Location Restriction Demonstrations

LOCATION RESTRICTION DEMONSTRATIONS

40 CFR §257.60 – §257.64

Chesterfield FFCP Management Facility - SWP#609

Submitted To: Dominion Energy – Chesterfield Power Station
500 Coxendale Road
Chester, VA 23836

Submitted By: Golder Associates Inc.
2108 W. Laburnum Avenue, Suite 200
Richmond, VA 23227

August 2017 **Project No. 152-0610**

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation

Table of Contents

1.0 CERTIFICATION 1
2.0 INTRODUCTION 2
2.1 Landfill Site Background 2
2.2 Disposal Facility Permitting 2
2.3 Location Restrictions 2
3.0 PLACEMENT ABOVE THE UPPERMOST AQUIFER 3
3.1 Requirement 3
3.2 Demonstration 3
4.0 WETLANDS 4
4.1 Requirement 4
4.2 Demonstration 4
5.0 FAULT AREAS 5
5.1 Requirement 5
5.2 Demonstration 5
6.0 SEISMIC IMPACT ZONES 6
6.1 Requirement 6
6.2 Demonstration 6
6.2.1 Liquefaction Potential 6
6.2.2 Global Slope Stability 7
6.2.3 Veneer Stability 7
7.0 UNSTABLE AREAS 8
7.1 Requirement 8
7.2 Demonstration 8
7.2.1 Soil Conditions 8
7.2.2 Site Geology 8
7.2.3 Man-Made Features 9
8.0 CONCLUSION 10

List of Attachments

Attachment A Wetland Approvals and Site Plan Map
Attachment B Supplementary Seismic Evaluation

1.0 CERTIFICATION

I certify that the information contained within this Location Restriction Demonstration Report was prepared by me or under my direct supervision, and meets the requirements of Sections §257.60 through §257.64 of the Federal Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals (CCR) from Electric Utilities; Final Rule (40 CFR 257; the ***CCR rule*).**

As used herein, the word "certification" shall mean an expression of the Engineer's professional opinion to the best of his or her information, knowledge, and belief, and does not constitute a warranty or guarantee by the Engineer.

Daniel McGrath	Associate and Senior Consultant
Print Name	Title
Daniel McGrath	8/7/17
Signature	Date

2.0 INTRODUCTION

This Location Restriction Demonstration was prepared for the Chesterfield Power Station Fossil Fuel Combustion Products (FFCP) Management Facility CCR Landfill (***Landfill***) located in Chesterfield County, Virginia, in accordance with 40 CFR §257.60 through §257.64 (collectively – the ***Location Restrictions***). This demonstration documents how the Landfill meets the requirements of each condition in the CCR Rule.

2.1 Landfill Site Background

The Landfill is permitted as an approximately 67-acre lined facility for the disposal of CCR from the Chesterfield Power Station. A Site Location Map is included as Figure 1. The property is located near the James River and is near the western boundary of the Coastal Plain physiographic province of Virginia. The Landfill will occupy the high ground to the west of the James River and north of Proctors Creek.

2.2 Disposal Facility Permitting

The Landfill was permitted as a Solid Waste Disposal Facility under the Virginia Solid Waste Management Regulations (VSWMR - 9VAC20-81) with the following approval dates:

- Part A Application (Siting Criteria): January 6, 2012
- Part B Application (Technical Criteria): June 29, 2016

As part of the requirements for the Part A application, the site was vetted using the VSWMR criteria with regards to geology, groundwater, seismic hazards, wetlands, flood plains, etc. The Part B application considered the technical design of the Landfill for global stability, liner design, leachate containment, and stormwater control. Construction of Phase 1 of the landfill began in June 2016.

2.3 Location Restrictions

The location restrictions in the CCR Rule, Sections §257.60 through §257.64, require a demonstration to show compliance with each restriction. The following sections in this report address each restriction individually, and supporting documentation is included as attachments as required.

- §257.60 – Placement above the uppermost aquifer
- §257.61 – Wetlands
- §257.62 – Fault Areas
- §257.63 – Seismic Impact Zones
- §257.64 – Unstable Areas

3.0 PLACEMENT ABOVE THE UPPERMOST AQUIFER

3.1 Requirement

§257.60 (a): ***New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must be constructed with a base that is located no less than 1.52 meters (five feet) above the upper limit of the uppermost aquifer, or must demonstrate that there will not be an intermittent, recurring, or sustained hydraulic connection between any portion of the base of the CCR unit and the uppermost aquifer due to normal fluctuations in groundwater elevations (including the seasonal high water table).***

3.2 Demonstration

In the summer of 2007, as part of the hydrogeological investigation of the site for permitting, observation wells and groundwater monitoring wells were installed throughout the property. In 2015, four wells were added to the existing network, and one well was decommissioned and relocated. Three existing on-site wells were also used for water level measurements. A total of 30 wells were included in the water level measurements of the site. Water levels were taken approximately monthly from July 2007 through mid-2010 until suspended; measurement resumed on a semi-annual basis in 2015. From July 2007 through the first quarter of 2017, approximately 850 individual water level readings have been taken at the site.

Groundwater contour maps based on the projected seasonal high groundwater elevation were first developed with the groundwater elevation data compiled from 2007 through 2010. Base grade contours for the landfill, representing the lowest base elevation of the landfill liner system, were initially established based on this water level data. Within each of the four landfill phases, a low area on the north and south (N and S) of each phase serves as a leachate collection sump. The base elevation of these sumps were compared to the seasonal high water level. With the adoption of the Federal CCR Rule in 2015, the sump grades were again evaluated to verify no less than five feet of separation existed between the seasonal high groundwater level and the landfill base grades. Small elevation adjustments were made to sump 1N and 1S to raise them above the computed seasonal high groundwater as computed in July, 2016.

Table 1 shows the base elevations of each sump and the corresponding seasonal high groundwater elevation at that same point to demonstrate a separation of no less than five feet.

Table 1 – Groundwater Separation Distance

Sump	GW El	Base El	Difference, ft
1N	20	26	6
1S	13	22	9
2N	31	40	9
2S	26	34	8
3N	39	48	9
3S	32	44	12
4N	53	58	5
4S	40	48	8

4.0 WETLANDS

4.1 Requirement

§257.61 (a): ***New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in wetlands, as defined in § 232.2 of this chapter, unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that the CCR unit meets the requirements of paragraphs (a)(1) through (5) of this section.***

4.2 Demonstration

The entire Landfill property and proposed access road areas were evaluated for the presence of wetland areas. Certification of the wetland areas on the property was provided by the U.S. Army Corps of Engineers on October 9, 2007.

The Landfill is not located in a wetland area. Access to the site from the Chesterfield Power Station did impact a small amount of wetland areas; however, the road and bridge impacts were mitigated and permitted through both the U.S. Army Corps of Engineers and the Virginia Department of Environmental Quality. **Attachment A** includes the wetland approvals and a site plan showing the landfill boundary with respect to the permitted wetland areas. Figure 1 shows the landfill footprint is not located on mapped wetlands on site.

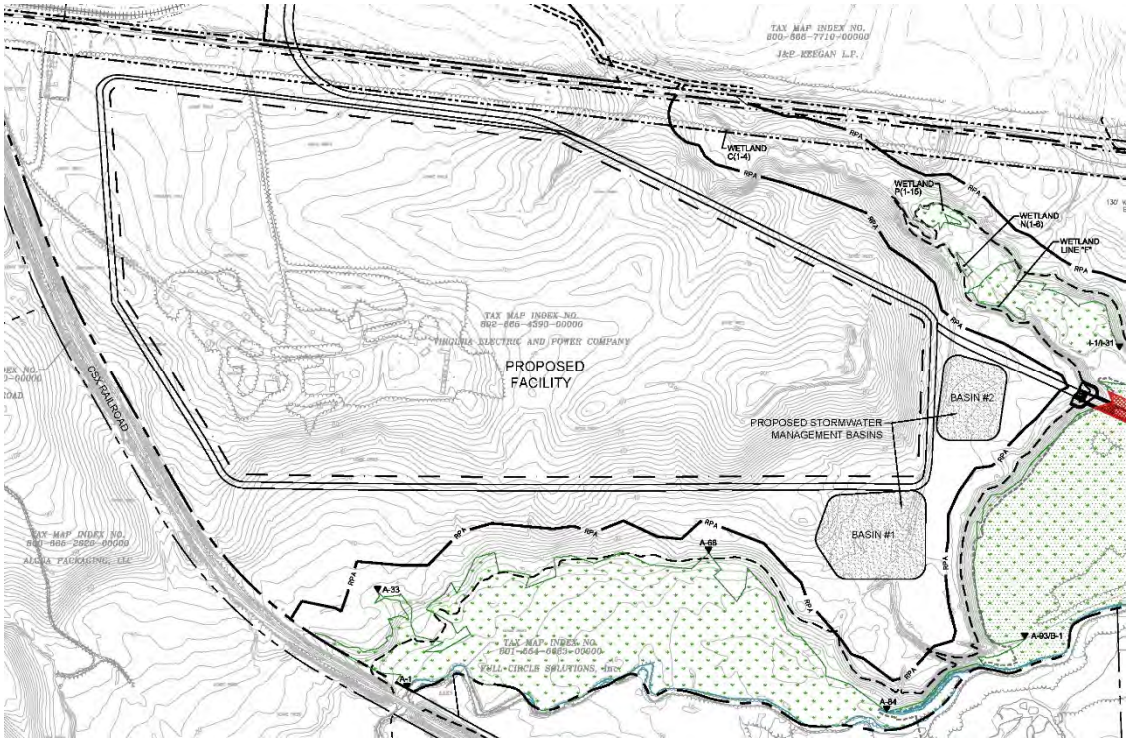


Figure 1 – Mapped Wetland Areas in Landfill Vicinity

5.0 FAULT AREAS

5.1 Requirement

§257.62 (a): ***New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located within 60 meters (200 feet) of the outermost damage zone of a fault that has had displacement in Holocene time unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that an alternative setback distance of less than 60 meters (200 feet) will prevent damage to the structural integrity of the CCR unit is defined as stormwater that may flow towards the active portion of the landfill from non-disposal areas.***

5.2 Demonstration

The Landfill is not located in an area of recent or active faulting. The closest fault system is the Paleocene age Dutch Gap Fault System south of the Landfill; this system does not indicate any recent (Holocene) movement that would result in failure of containment structures at the Landfill (Dischinger, 1987). The closest area known to have evidence of recent displacement is in central Virginia, which experienced displacement in Quaternary time (i.e., up to 1.8 million years ago) and is at least 20 miles from the site (see yellow hatched area in figure below).

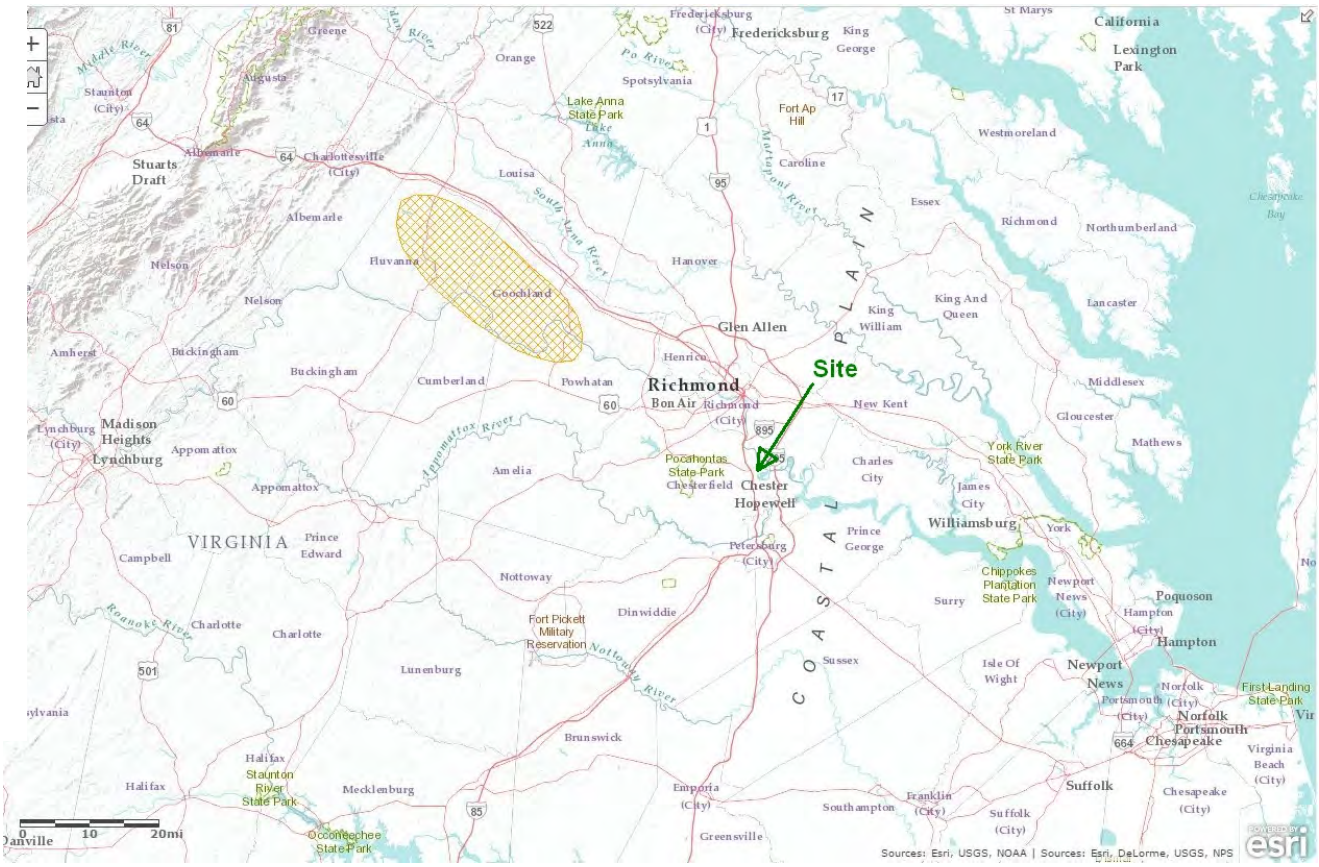


Figure 2 - Areas of Quaternary Deformation and Liquefaction, Virginia

Source:
http://www.arcgis.com/home/webmap/viewer.html?url=http%3A%2F%2Fearthquake.usgs.gov%2Farcgis%2Frest%2Fservices%2Fhaz%2Fqfaults%2FMapServer&source=sd

6.0 SEISMIC IMPACT ZONES

6.1 Requirement

§257.63 (a): ***New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site.***

6.2 Demonstration

A seismic impact zone, as defined in the CCR Rule, means ***an area having a 2% or greater probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth's gravitational pull (g) will exceed 0.10 g in 50 years.*** The site was determined to have a Site Class D adjusted Peak Ground Acceleration (PGA) of 0.183 g, thereby making it a seismic impact zone.

Golder evaluated the site and the design of the Landfill under seismic conditions for liquefaction potential, global slope stability, and veneer stability. In all evaluated cases, the site and design of the Landfill demonstrated adequate factors of safety with respect to seismic activity. **Attachment B** includes the complete supplementary seismic evaluation report with supporting calculations for each condition.

6.2.1 Liquefaction Potential

The liquefaction assessment screens foundation soils for susceptibility to sudden strength loss due to seismic loading. Foundation soils in three locations within the footprint of the Landfill were evaluated to a depth of 50 feet below ground surface. In all cases, the computed FS was greater than 1.2, indicating the foundation soils are not susceptible to liquefaction.

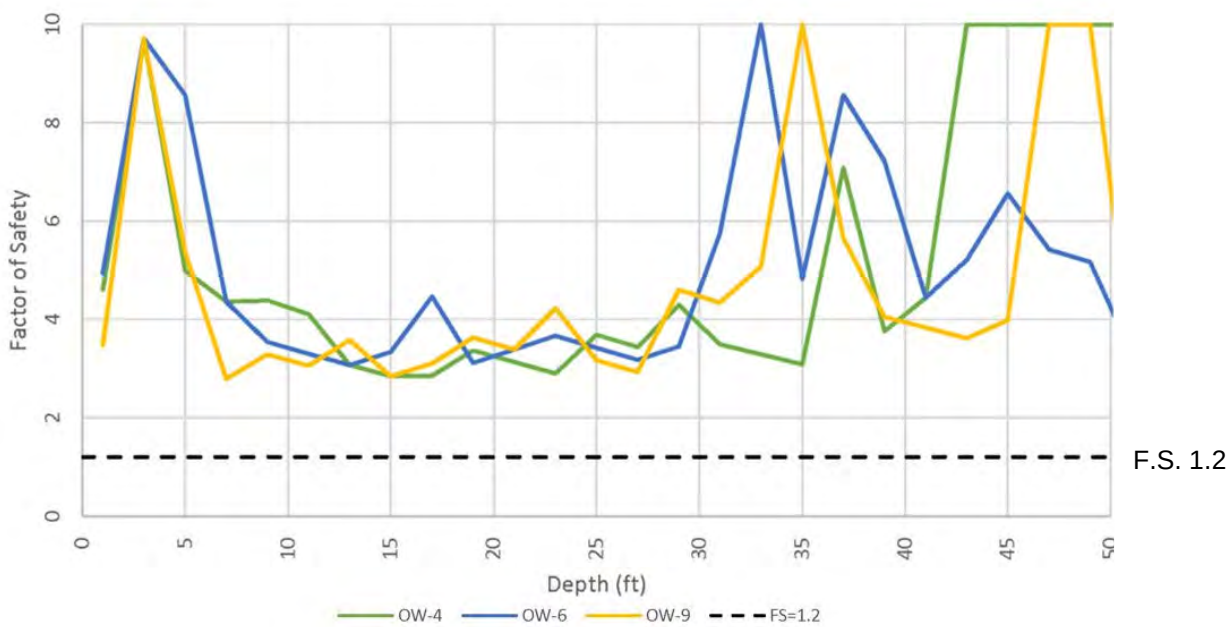


Figure 3 – Foundation Liquefaction Potential

6.2.2 Global Slope Stability

The global stability calculations evaluate the static and seismic stability of the Landfill and its foundation relating to deep seated failures (> 5 ft deep). The global stability of slopes were assessed along the two critical cross sections for the Landfill geometry. Slopes were evaluated under both long-term static and seismic conditions. As summarized in the table below, all slopes meet the target factors of safety for slope stability. Note that the two FS values listed for the steady state circular analysis case of Section A-A West are related to a large global and small local surface, respectively.

	Analysis Type	Steady-State	Steady-State	Seismic	Seismic
	Slip Surface Type	Circular	Block	Circular	Block
	Target FS	1.5	1.5	1.0	1.0
Figure	**Section**	**Factors of Safety**			
2	A-A, West	2.2 / 1.9	2.1	2.0	2.0
3	A-A, East	2.6	2.6	1.4	2.6
4	B-B, South	1.9	1.8	1.8	1.7
5	B-B, North	1.8	1.8	1.7	1.7

6.2.3 Veneer Stability

Veneer stability calculations were completed to evaluate shallow slope stability of both the base liner and cap liner for long-term static conditions and short-term construction conditions. In addition, veneer stability of the cap liner system was evaluated under seismic loading conditions and seepage conditions. For all the considered scenarios, calculated factors of safety meet or exceed target factors of safety as shown in the table below.

Case Analyzed	Target FS	Base Liner FS	Cap Liner FS
Long-Term Static	1.5	2.7	2.7
Short-Term with Dozer	1.1	2.3	2.5
Seepage Analysis	1.1	N/A	1.1
Seismic Analysis	1.0	N/A	1.8

7.0 UNSTABLE AREAS

7.1 Requirement

§257.64 (a): ***An existing or new CCR landfill, existing or new CCR surface impoundment, or any lateral expansion of a CCR unit must not be located in an unstable area unless the owner or operator demonstrates by the dates specified in paragraph (d) of this section that recognized and generally accepted good engineering practices have been incorporated into the design of the CCR unit to ensure that the integrity of the structural components of the CCR unit will not be disrupted***.

7.2 Demonstration

Assessment of unstable areas includes an evaluation of the soil conditions at the site, which may result in significant differential settling, a review of site geologic or geomorphologic features, and consideration of man-made features on site that may cause unstable conditions. Full documentation of the site history, field investigations, and site geology can be found in the Landfill's Part A application. A summary of the unstable area evaluation is presented in this document.

7.2.1 Soil Conditions

Site investigation activities conducted by Golder Associates, Inc. (2007 through 2010) did not identify any features that would suggest recent landslide activities. Based on the soil boring records and geotechnical testing of soils encountered, the subsurface conditions at the Landfill are expected to adequately support construction without significant differential settlement. A total of 45 investigative test borings were used to characterize the hydrogeologic and geotechnical properties of the subsurface soils. Geotechnical test borings were advanced to various depths ranging between 8 and 122 feet below grade. In general, the test borings drilled during this investigation were advanced to a depth required to characterize the thickness of the unconsolidated deposits for a minimum 20 feet below the lowest elevation of the bottom liner, and to measure the depth to groundwater at each location. Based on the information obtained during this investigation, the site soils are classified primarily as silty fine sand, clayey silt to sandy clay, silty clay, and sand and gravel. The results of the soil testing indicate that the Landfill is not sited in a geologically unstable area.

Evaluation of liquefaction potential is addressed in Section 6.2.1.

7.2.2 Site Geology

The Landfill is located on layers of competent soils from various episodes of sedimentary deposition due to the proximity of the James River and Proctors Creek. The subsurface soil layers were determined to be of adequate strength to support the Landfill. The Landfill is not located in an area of karst topography.

The Landfill is immediately underlain by unconsolidated sediments of the Pleistocene Charles City Formation and Pleistocene to late Pliocene Windsor Formation. The entire limits of the waste management unit are underlain by the Windsor Formation, which is a gray to yellowish to reddish-brown sand, gravel, silt, and clay (Bondurant ***et al***., 2007). The thickness of the Windsor Formation within the footprint of the Landfill ranges up to 52 feet beneath the upland area as observed in test boring OW-6.

Below the Windsor Formation as interpreted from boring logs and available published and unpublished literature is the Miocene Epoch lower Chesapeake Group. Site observations and available literature indicate that the lower Chesapeake Group sediments generally consist of light gray to dark bluish gray sand and silty to clayey fine-grained sand with subangular to subrounded quartz granules, and commonly, abundant mica flakes (Bondurant ***et al***., 2007).

Below the Windsor Formation as interpreted from boring logs and available published and unpublished literature is the Miocene Epoch lower Chesapeake Group. Site observations and available literature indicate that the lower Chesapeake Group sediments generally consist of light gray to dark bluish gray

sand and silty to clayey fine-grained sand with subangular to subrounded quartz granules, and commonly, abundant mica flakes (Bondurant ***et al***., 2007). As observed in test boring OW-8D, the thickness of the interpreted Chesapeake Group sediments within the footprint of the Landfill ranges up to 48 feet beneath the upland areas.

Bedrock was not encountered during the site investigation. Based on available information (published geologic maps), the uppermost competent bedrock beneath the proposed Facility is likely to be the Paleozoic Petersburg Granite. Based on site observations and boring records contained in the U.S. Geological Survey Professional Paper 1731 titled ***The Virginia Coastal Plain Hydrogeologic Framework*** (McFarland and Bruce, 2006), the depth to competent bedrock beneath the Landfill is expected to be greater than 145 feet below ground surface (elevation of less than -100 ft MSL).

7.2.3 Man-Made Features

An evaluation of the site's history does not reveal, nor has evidence been found of, man-made conditions on site that could cause unstable conditions. Historical research as part of the cultural resources evaluation indicates the site was used exclusively for farming from the mid-1700's through the mid-20th century. No evidence of surficial or shaft mining on the site has been encountered in either the literature or during on-site evaluations.

During the American Civil War, the site did see combat activity as part of the Confederate defensive line for Richmond. Anecdotal evidence of mortars and cannon fire impacting the site was encountered during the site research. An extensive metal detection survey, targeted excavation, and ground penetrating radar survey was undertaken in 2008 in an attempt to locate any large metal or possibly unexploded Civil War-era ordinance. Initial site excavations for construction were carried out under the direct observation of a trained archeologist. To date, no unexploded ordinance has been discovered on site.

The site was owned by Reynolds Metals Company from 1956 to 2001, where it was used as a materials testing facility for Reynolds. Evaluation of the site after sale to Dominion revealed several concrete foundations for buildings and towers, but no underground structures were suspected nor found.

8.0 CONCLUSION

Based upon the demonstrations made in this report, the Chesterfield Power Station FFCP Management Facility CCR Landfill is in compliance with the location restrictions outlined in Sections §257.60 through §257.64 of the Federal Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals (CCR) from Electric Utilities; Final Rule (40 CFR 257; the ***CCR rule***).

DEPARTMENT OF THE ARMY
NORFOLK DISTRICT, CORPS OF ENGINEERS
FORT NORFOLK, 803 FRONT STREET
NORFOLK, VIRGINIA 23510-1096

October 9, 2007

GOLDER ASSOCIATES
REC'D | PN
FN
CC | ROUTE
OCT 12 2007
RICHMOND, VA

CENAO-REG
Southern Virginia Regulatory Section
2007-02441 (Proctors Creek)

Golder Associates
Attn: Mr. Michael Williams
3719 Saunders Avenue
Richmond, Virginia 23227

Dear Mr. Williams:

This letter is in reference to the delineation completed by Golder and Associates on an approximately 228-acre site located on the north line of Proctors Creek, west of the James River generally east of I-95, and in Chesterfield County, Virginia.

The revised wetland and stream delineation map prepared by Golder Associates entitled "Reymet Road, Chesterfield County, Virginia" dated September 24, 2007, and received by the Corps on October 9, 2007, is hereby verified as accurate. Our basis for this includes application of the Corps' 1987 Wetland Delineation Manual and the positive indicators of wetland hydrology, hydric soils, and hydrophytic vegetation and the presence of an ordinary high water mark.

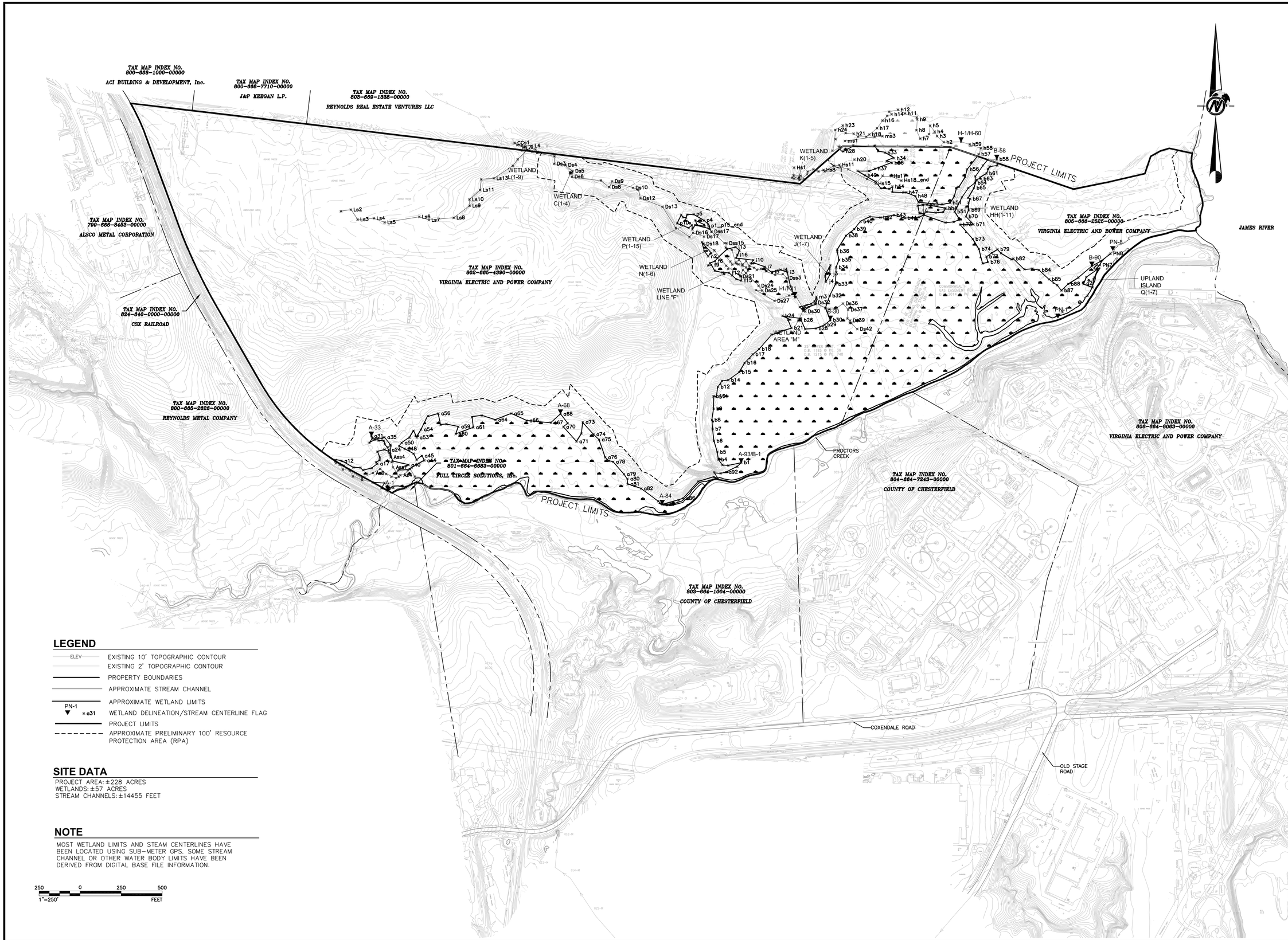
Any mechanized landclearing that disturbs the soil surface, such as with a bulldozer and/or root rake, and/or any filling or excavation in the wetlands and streams on this site may require a permit from the Department of the Army and/or the Virginia Department of Environmental Quality prior to such activities occurring.

Please contact Dr. Silvia Gazzera in the Richmond Field Office at 9100 Arboretum Parkway, Suite 235, Richmond, Virginia 23236, (804) 323-3780 with any questions.

Sincerely,

Lynette R. Rhodes
Chief, Southern Virginia
Regulatory Section

CF:
Chesterfield County, Department of Environmental Engineering, Chesterfield, Virginia
Chesterfield County, Department of Planning, Chesterfield, Virginia



LEGEND

- EXISTING 10' TOPOGRAPHIC CONTOUR
- EXISTING 2' TOPOGRAPHIC CONTOUR
- PROPERTY BOUNDARIES
- APPROXIMATE STREAM CHANNEL
- APPROXIMATE WETLAND LIMITS
- PN-1 / a31 — WETLAND DELINEATION/STREAM CENTERLINE FLAG
- PROJECT LIMITS
- APPROXIMATE PRELIMINARY 100' RESOURCE PROTECTION AREA (RPA)

SITE DATA

PROJECT AREA: ±228 ACRES
WETLANDS: ±57 ACRES
STREAM CHANNELS: ±14455 FEET

NOTE

MOST WETLAND LIMITS AND STEAM CENTERLINES HAVE BEEN LOCATED USING SUB-METER GPS. SOME STREAM CHANNEL OR OTHER WATER BODY LIMITS HAVE BEEN DERIVED FROM DIGITAL BASE FILE INFORMATION.

250 0 250 500
1"=250' FEET

Golder Associates
Richmond, Virginia

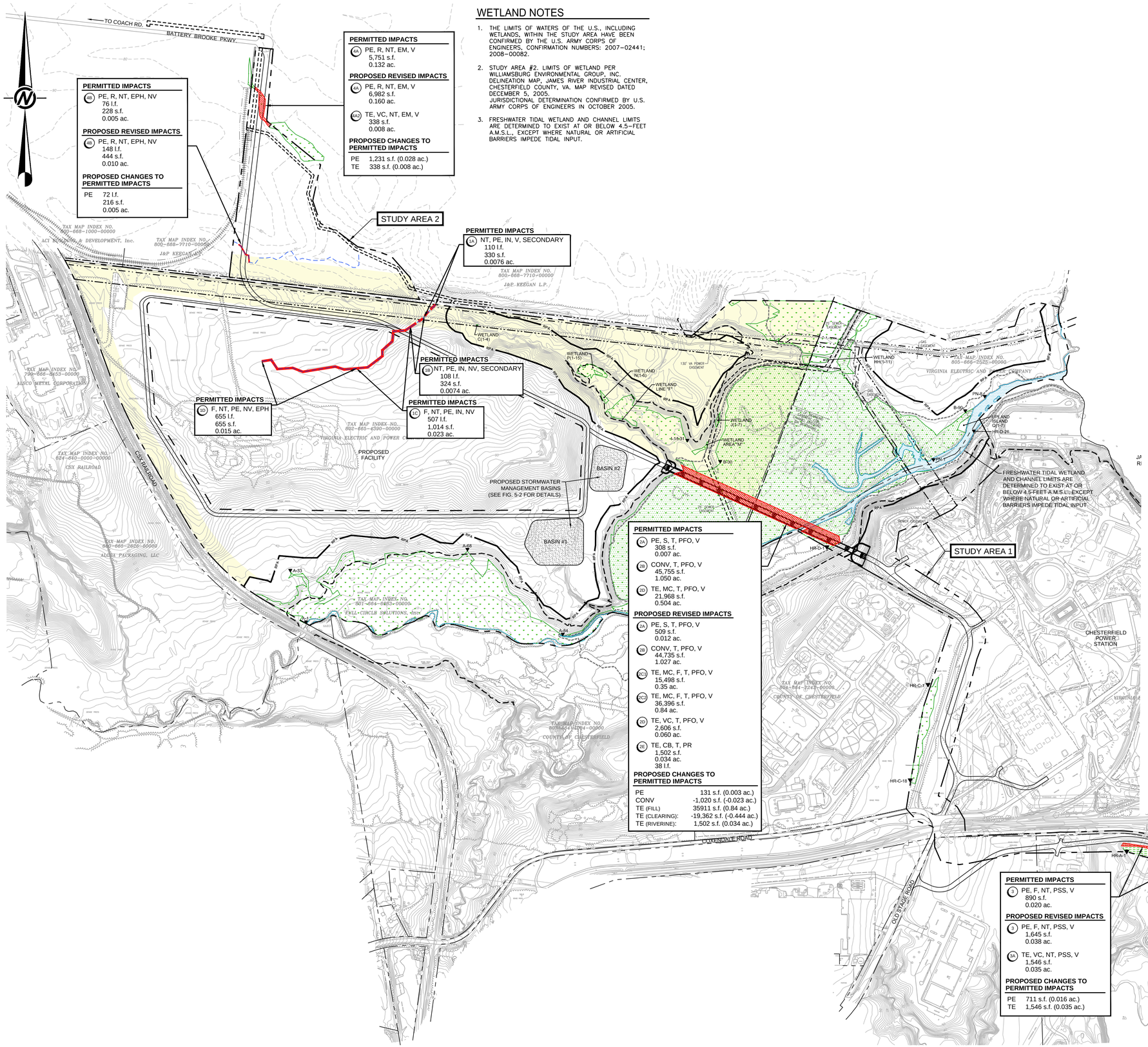
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
REYMET ROAD
CHESTERFIELD COUNTY, VIRGINIA

TITLE
REVISED WETLAND DELINEATION PLAN

PROJECT No.		073-6607
FILE No.		0736607A08
REV. 0	SCALE	AS SHOWN
DESIGN	ARC	09/24/07
CADD	OSY	09/24/07
CHECK		
REVIEW		

DRAWING 8



WETLAND NOTES

1. THE LIMITS OF WATERS OF THE U.S., INCLUDING WETLANDS, WITHIN THE STUDY AREA HAVE BEEN CONFIRMED BY THE U.S. ARMY CORPS OF ENGINEERS, CONFIRMATION NUMBERS: 2007-02441; 2008-00082.
2. STUDY AREA #2. LIMITS OF WETLAND PER WILLIAMSBURG ENVIRONMENTAL GROUP, INC. DELINEATION MAP, JAMES RIVER INDUSTRIAL CENTER, CHESTERFIELD COUNTY, VA. MAP REVISED DATED DECEMBER 5, 2005. JURISDICTIONAL DETERMINATION CONFIRMED BY U.S. ARMY CORPS OF ENGINEERS IN OCTOBER 2005.
3. FRESHWATER TIDAL WETLAND AND CHANNEL LIMITS ARE DETERMINED TO EXIST AT OR BELOW 4.5-FEET A.M.S.L., EXCEPT WHERE NATURAL OR ARTIFICIAL BARRIERS IMPEDE TIDAL INPUT.

LEGEND

- EX. GROUND CONTOUR (2' INTERVAL)
- EX. GROUND CONTOUR (5' INTERVAL) *FROM CHESTERFIELD CO. G.I.S.
- STUDY AREA
- PROPERTY BOUNDARY
- PROPOSED LANDFILL FACILITY LIMITS OF DISTURBANCE
- HAUL ROAD
- PROPOSED LIMITS OF FFCP DISPOSAL
- PROPOSED SEDIMENT BASIN
- EXISTING TREELINE
- GAS EASEMENT
- POWER EASEMENT
- SEWER EASEMENT
- DREDGING EASEMENT
- APPROXIMATE RPA BOUNDARY (100-FOOT WETLAND OFFSET)
- APPROXIMATE 100-YEAR FLOODPLAIN
- EX. 4.5' GROUND CONTOUR (SEE WETLAND NOTE 3)
- ▼ B-90 DELINEATION FLAG
- FRESHWATER TIDAL WETLAND (SEE WETLAND NOTE 3)
- NON-TIDAL WETLAND
- PSS WETLAND
- PROPOSED WETLAND IMPACT
- NON-VEGETATED TIDAL CHANNEL (SEE WETLAND NOTE 3)
- STREAM CHANNEL
- PROPOSED STREAM CHANNEL IMPACT
- PROPOSED STREAM CHANNEL SECONDARY IMPACT
- (1) IMPACT SITE IDENTIFICATION (SEE FIGURES 5-1-A, 5-1-B, AND 5-1-C FOR 200 SCALE ENLARGEMENTS)
- PROPOSED BUFFER

SITE DATA

STUDY AREA 1:	256.84 AC.±
ACCESS ROAD STUDY AREA 2:	16.958 AC.±

JURISDICTIONAL AREAS
(STUDY AREA 1 - FACILITY AND HAUL ROAD)

FRESHWATER TIDAL WETLAND:	37.53 AC.
NON-TIDAL WETLAND:	20.90 AC.
PSS WETLAND:	0.48 AC.
NON-VEGETATED, TIDAL CHANNEL:	5,250 L.F.
STREAM CHANNEL:	8,765 L.F.

JURISDICTIONAL AREAS
(STUDY AREA 2 - ACCESS ROAD)

NON-TIDAL WETLAND:	0.51 AC.
STREAM CHANNEL:	1,511 L.F.

LEVEL OF IMPACTS

PE = PERMANENT IMPACT
TE = TEMPORARY IMPACT
CONV = CONVERSION IMPACT

TYPES OF IMPACTS

R = ROAD IMPACT
F = FILL IMPACT
VC = VEGETATION CUT
S = STRUCTURE (BRIDGE PILING)
MC = MECHANIZED CLEARING
CB = CONSTRUCTION BRIDGE (SPAN)

RESOURCE DESIGNATION

T = TIDAL
NT = NON-TIDAL
IN = INTERMITTENT STREAM CHANNEL
PR = PERENNIAL STREAM CHANNEL
EPH = EPHEMERAL STREAM CHANNEL
PFO = PALUSTRINE FORESTED WETLAND
PSS = PALUSTRINE SCRUB-SHRUB WETLAND
EM = EMERGENT WETLAND
V = VEGETATED
NV = NON-VEGETATED

Golder Associates
Richmond, Virginia

REV	DATE	DES	MGW	EXTENTS AND QUANTITIES OF IMPACTS	BPG	CADD	CHK	RVW
1	02/18/15			REVISION DESCRIPTION				

PROJECT
DOMINION
CHESTERFIELD POWER STATION
FFCP MANAGEMENT FACILITY
CHESTERFIELD COUNTY, VA

TITLE
JURISDICTIONAL AREA IMPACTS MAP

PROJECT No.	073-660711	
FILE No.073660711-I01 NO TABLES		
REV. 0	SCALE	AS SHOWN
DESIGN	KVS	12/01/10
CADD	OSY	12/01/10
CHECK		
REVIEW		

DRAWING 3

G:\Plan Production Data Files\Drawing Data Files\073-660711\I - Revised Wetland Impacts\Active Drawings\073660711-I01 NO TABLES.dwg

July 15, 2016 Project No. 1520610

Dominion
5000 Dominion Boulevard
Glen Allen, VA 23060

Attn: Mr. Chris Gee
Project Manager

RE: SUPPLEMENTAL STABILITY AND SEISMIC EVALUATION
DOMINION – CHESTERFIELD POWER STATION
CHESTERFIELD FFCP MANAGEMENT FACILITY – SWP #609

Dear Chris:

Dominion contracted Golder Associates Inc. (Golder) to design a landfill to receive ash from the Chesterfield Power Station (CPS) in Chesterfield County, Virginia. The proposed location of the landfill is near the CPS between Reymet Road and the CPS. In response to recent changes in coal ash storage regulations from the United States Environmental Protection Agency (USEPA) and the Virginia Department of Environmental Quality (VADEQ), design revisions and additional supporting calculations were completed to comply with these regulations. Golder issued drawings detailing the design revisions in November 2015. These revisions included the addition of a composite geosynthetic base liner system and updates to the final closure cap.

Geotechnical calculations further supporting these revisions are discussed and attached to this letter. Specifically, updated material properties and a seismic hazard assessment were developed for the proposed site, and calculations evaluating liquefaction potential, global slope stability, and veneer stability were completed and are attached.

1.0 REVIEW AND SUPPLEMENT OF MATERIAL PROPERTIES

As part of the additional calculations and design revisions, the geotechnical material properties were reviewed and supplemented from previous work for use in the current analyses. Properties of soils at the site were primarily developed from laboratory and in-situ test data obtained from Golder's 2007 exploration program. Properties of the ash fill were developed from recent laboratory testing on ash samples collected from CPS. Base and cap liner system properties were assigned based on typical control values from Golder's extensive internal geosynthetics database. Details of the material properties and laboratory test results on the ash can be found in the attached Material Properties Package (Attachment A).

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation

2.0 SITE SEISMIC HAZARD

The seismic hazard at the landfill site was assessed from data developed by and publically available through the National Earthquake Hazards Reduction Program (NEHRP) and the United States Geological Survey (USGS). The seismic hazard associated with a 2% probability of exceedance in 50 years (2475 year return period) was used for design as required in the applicable CCR and VADEQ regulations, and the following seismic hazard characteristics associated with the proposed landfill site were used in design:

- Site Class D
- B/C PGA = 0.117g
- Site Class D Adjusted PGA = 0.183g
- Mean Magnitude = 5.52

For details about the seismic hazard at the proposed landfill site, refer to the attached Seismic Hazard Evaluation (Attachment B).

3.0 GEOTECHNICAL CALCULATION PACKAGES

Geotechnical calculations supporting the design revisions are categorized into three types: Liquefaction Assessment, Global Stability, and Veneer Stability. The liquefaction assessment screens foundation soils for susceptibility to sudden strength loss due to seismic loading. The global stability calculations evaluate the static and seismic stability relating to deep seated failures (> 5 ft deep), while the veneer stability calculations evaluate the static and seismic stability of shallow slip surfaces (< 5 ft deep).

3.1 Liquefaction Assessment

The proposed landfill site was assessed for liquefaction susceptibility using the NCEER method. Profiles from the three most critical boreholes (OW-4, OW-6, and OW-9) drilled as part of Golder's 2007 geotechnical exploration were evaluated. All soils screened were calculated to have a factor of safety (FS) against liquefaction greater than 1.2 (FS > 1.2) and thus were not found to be susceptible to liquefaction. See the Liquefaction Assessment Package provided as Attachment C for the full analysis.

Boreholes	Calculated Liquefaction FS < 1.2?
OW-4, OW-6, OW-9	NO

3.2 Global Stability

Golder previously completed slope stability analyses for the landfill that were re-evaluated and updated to account for the recent landfill design changes in the current calculations. The previous analyses are supplemented with the attached global stability package which contains additional slope stability calculations to account for the liner design modifications and to check performance under seismic loading.

In the supplemental calculations, the global stability of slopes were assessed along the two critical cross sections for the proposed landfill geometry. Slopes were evaluated under both long-term static and seismic conditions. As summarized in the table below, all slopes meet the target factors of safety for slope stability. Note that the two FS values listed for the steady state circular analysis case of Section A-A West are related to a large global and small local surface, respectively. Further details are available in the Supplemental Slope Stability Analysis Package provided as Attachment D.

	Analysis Type	Steady-State	Steady-State	Seismic	Seismic
	Slip Surface Type	Circular	Block	Circular	Block
	Target FS	1.5	1.5	1.0	1.0
Figure	Section	Factors of Safety			
2	A-A, West	2.2 / 1.9	2.1	2.0	2.0
3	A-A, East	2.6	2.6	1.4	2.6
4	B-B, South	1.9	1.8	1.8	1.7
5	B-B, North	1.8	1.8	1.7	1.7

3.3 Veneer Stability

Veneer stability calculations were completed to evaluate shallow slope stability of both the base liner and cap liner for long-term static conditions and short-term construction conditions. In addition, veneer stability of the cap liner system was evaluated under seismic loading conditions and seepage conditions. For all the considered scenarios, calculated factors of safety exceed target factors of safety as shown in the table below.

Case Analyzed	Target FS	Base Liner FS	Cap Liner FS
Long-Term Static	1.5	2.7	2.7
Short-Term with Dozer	1.1	2.3	2.5
Seepage Analysis	1.1	N/A	1.1
Seismic Analysis	1.0	N/A	1.8

The stability of the liner systems meet the recommended factors of safety for the range of analyzed cases. For more details, see the attached Veneer Stability Calculation Package provided as Attachment E.

4.0 SUMMARY

To comply with regulations put forth by the USEPA and VADEQ, after the initial design of the Landfill, supplemental geotechnical calculations were completed in support of associated design modifications. Golder reviewed and extended the previous calculations where necessary, including updating the material properties summary and evaluating the current seismic hazard to assist in the new seismic stability and liquefaction evaluations. Geotechnical calculations consisted of a liquefaction assessment, global slope stability analyses (both static and seismic), and veneer stability analyses (both static and seismic). All calculated results were found to be acceptable for the updated landfill design.

5.0 LIMITATIONS

Golder's evaluation of subsurface conditions is based on our understanding of the site and project information. Findings presented in this report are based on data and samples from discrete locations from the site. It is not uncommon for subsurface conditions to vary significantly over short horizontal and vertical distances. Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions between borings will differ from those at the boring locations. If conditions are observed that differ materially from those observed herein, we should be notified to review our findings and see if revised recommendations or calculations are required.

Our professional services have been performed and our findings derived in accordance with generally accepted geotechnical engineering principles and standards. Golder is not responsible for the conclusions, opinions, or recommendations of others based on these data.

6.0 CLOSING

Golder appreciates the opportunity to provide continued services to Dominion on this project. Should you have any questions concerning this report, please contact one of the undersigned.

Sincerely,

GOLDER ASSOCIATES INC.

Gregory L. Hebeler, PE, PhD
Associate and Senior Consultant

Daniel McGrath, PE
Associate and Senior Consultant

Attachments or Enclosures:

Attachment A – Material Properties Package
Attachment B – Seismic Hazard Evaluation
Attachment C – Liquefaction Assessment Package
Attachment D – Supplemental Slope Stability Analysis Package
Attachment E – Veneer Stability Calculation Package

ATTACHMENT A

Materials Properties Package

Golder Associates	**SUBJECT:** Material Properties Selected for Stability Analyses **Project Number:** 1520610 **Project Name:** Chesterfield Reymet Road Landfill **Prepared by:** JGM **Checked by:** GLH **Date:** June 2016 **Reviewed by:** GLH

Objective

Update previous material package to include landfill liner components and for use in both seismic and static slope stability analyses.

Method

For material properties not previously selected, properties are evaluted using laboratory data and in-situ testing from previous geotechincal exploratory programs (Golder 2012). Where lab data and in-situ testing are lacking, additonal laboratory testing (triaxial testing) and Golder's experience were used to select material properties.

SUMMARY OF SELECTED PROPERTIES

The material properties, as summarized in the table below, are selected as representative values for the materials for use in slope stability analyses.

Selection of properties are discussed in detail for each material in the following pages.

Summary of Geotechnical Strength Properties

Material	*Total Unit Weight (pcf)*	*Drained (Effective)*		*Undrained (Total)*	
		ϕ' (°)	c' (psf)	ϕ (°)	c or $[S_u/\sigma'_v]$ (psf)
Stratum #1 (Silty Clayey Sand)	115*	31*	130*	23	170
Stratum #2 (Clayey Silt to Sandy Clay)	112*	28*	200*	N/A	[0.51]
Stratum #3 (Silty Clay)	120*	29*	200*	N/A	[0.42]
Stratum #4 (Clayey Sand & Gravel)	125*	32*	100*	N/A	
*Ash Fill***	100	31	40	N/A	
*Base Liner System****	120	24	90	N/A	
*Cap Liner System****	120	25	90	N/A	

**Properties previously selected in original material properties package (See package attached to end of this document.*

***Updated based on recent laboratory test data on site specific ash samples*

****Updated Base and Cap sections to comply with recently updated CCR regulations.*

References

Golder Associates (2012). Design Report - Chesterfield FFCP Facility - Permit #609

Golder Associates	Stratum #1 (Silty Clayey Sand) Material Properties for Seismic Conditions			
	Project Number:	**1520610**		
	Project Name:	**Chesterfield Reymet Road Landfill**		
	Prepared by:	JGM	*Checked by:*	GLH
	Date:	June 2016	*Reviewed by:*	GLH

Objective

Develop total (undrained) strength properties for use in seismic slope stability analyses. Effective (drained) strength properties were developed in the previous material property package (Golder 2012)

Method

Consolidated undrained (CU) triaxial testing completed as part of the 2012 material property packaged was used to interpret total stress strengths.

Triaxial Test Data

The following information was given in the triaxial test results.

Sample Number	A	B	C
Cell Pressure (psi)	93	53	63
Back Pressure (psi)	73	23	23
Consolidation Stress (psi)	20	30	40
Max Deviator Stress (psi)	29.6	42.6	35.6
σ'_1 (psi)	40.2	58.2	49.6
σ'_3 (psi)	10.6	15.6	14

From this information, the following total stress values were calculated.

Sample Number	A	B	C
Pore Pressure, u (psi)	9.4	14.4	26
σ_1 (psi)	49.6	72.6	75.6
σ_3 (psi)	20	30	40

Mohr's circles representing the stress data were plotted for total stresses, and potential Mohr-Coulomb strength envelopes were plotted with the circles. By inspection of test data, results from Sample C do not follow the trend set in the results of Samples A and B and were not considered respresentatvie of Stratum #1.

Strength Parameter Selection

Given irregularities in Sample C, only Samples A and B were used to develop a total stress envelope, and the following total stress strength parameters were selected as representative.

Total Stress Parameters	
ϕ (deg)	23
c (psf)	170

Golder Associates	**Stratum #2 (Clayey Silt to Sandy Clay) Material Properties for Seismic Conditions**		
	Project Number:	**1520610**	
	Project Name:	**Chesterfield Reymet Road Landfill**	
	Prepared by:	JGM	*Checked by:* GLH
	Date:	June 2016	*Reviewed by:* GLH

Objective

Develop total (undrained) strength properties for use in seismic slope stability analyses. Effective (drained) strength properties were developed in the previous material property package (Golder 2012)

Method

An undrained strength (Su) was correlated from blow counts as part of the original material property package. This undrained strength is converted to a SANSEP (Su/σ'_v) stress dependent type strength to account for the strength increases from consolidation effects below the landfill during filling.

Equivalent SANSEP Strength

The SANSEP type strength was calculated from the in-situ vertical effective stress (σ'_v) and Su in the center of the stratum. The table below shows the calculation and resulting SANSEP type strength.

Su in center of stratum (psf)	3,000
Depth to Stratum #2 Center Point (ft)	53
Average Unit Weight (pcf)	115
Average Depth of Water (ft)	50
Calculated σ'_v (psf)	5,908
S_u/σ'_v	**0.51**

Golder Associates	**Stratum #3 (Silty Clay) Material Properties for Seismic Conditions**
	Project Number: **1520610**
	Project Name: **Chesterfield Reymet Road Landfill**
	Prepared by: JGM *Checked by:* GLH
	Date: June 2016 *Reviewed by:* GLH

Objective

Develop total (undrained) strength properties for use in seismic slope stability analyses. Effective (drained) strength properties were developed in the previous material property package (Golder 2012)

Method

An undrained strength (Su) was correlated from blow counts as part of the original material property package. This undrained strength is converted to a SANSEP (Su/σ'_v) stress dependent type strength to account for the strength increases from consolidation effects below the landfill during filling.

Equivalent SANSEP Strength

The SANSEP type strength was calculated from the in-situ vertical effective stress (σ'_v) and Su in the center of the stratum. The table below shows the calculation and resulting SANSEP type strength.

Su in center of stratum (psf)	3,000
Depth to Stratum #2 Center Point (ft)	77.5
Average Unit Weight (pcf)	115
Average Depth of Water (ft)	50
Calculated σ'_v (psf)	7,197
S_u/σ'_v	**0.42**

Golder Associates	**Ash Material Properties**		
	Project Number:	**1520610**	
	Project Name:	**Chesterfield Reymet Road Landfill**	
	Prepared by:	JGM	*Checked by:* GLH
	Date:	June 2016	*Reviewed by:* GLH

Objective

Develop effective strength properties for use in slope stability analyses. Total (undrained) strength properties were not developed since all ash material will be placed in the landfill above the water level where the material will only be in a drained condition.

Method

Recently produced ash samples collected from the Chesterfield Power Station were shipped to Golder's geotechnical laboratory for testing. A Standard Proctor test was completed for compaction, and two consolidated undrained (CU) triaxial tests were conducted on laboratory-compacted samples. Two CU tests were completed at different levels of compaction. The first triaxial test was completed on ash compacted to 95% of the maximum dry density calculated from the Standard Proctor test, and the second test was completed on ash compacted to 90% of the calculated maximum dry density. Additional details of lab testing can be found in the summary of laboratory testing appended to the discussion of material properties.

Standard Proctor Test Data

Results from the standard proctor test completed on the ash sample presented a maximum dry density of 75 pcf with an optimum moisture content of 28%.

Triaxial Test Data

The following results were calculated from the triaxial lab testing.

Sample	1	2
Moisture Content (%)	30	30.5
Compaction Level	95%	90%
Dry Unit Weight (pcf)	71	68
Wet Unit Weight (pcf)	92	88
ϕ' (deg)	33.0	31.4
c' (psf)	72	43

Strength Parameter Selection

From the results of the Standard Proctor and triaxial tests, a conservative (for stability analyses) unit weight of 100 pcf was selected, and strengths from the 90% compacted triaxial test were rounded to obtain the design strength parameters.

Unit Weight	
γ (psf)	100
Effective Strength Parameters	
ϕ' (deg)	31
c' (psf)	40

	Base Liner System Strength Properties		
	Project Number:	**1520610**	
	Project Name:	**Chesterfield Reymet Road Landfill**	
	Prepared by:	JGM	*Checked by:* GLH
	Date:	June 2016	*Reviewed by:* GLH

Objective:

Develop strength parameters of the base liner system for use in slope stability analyses.

Liner System Shear Normal Function

The strength of the composite bottom liner and overlying protective cover is a composite of the lower bound strengths of the internal and interface shear strength of the various components. Therefore, the strength of a liner system is defined in terms of a shear normal function which discretely defines a shear strength as a function of a given stress.

Golder's internal database of geosynthetic and soil interface strengths were used to provide strength estimates for components where applicable. The geomembrane interface is estimated to engage the full compacted clay liner strength below the geomembrane since a texture geomembrane is being used. For the clay, an average of the peak and residual strength based on the anticipated plasticity of the soil was used for the interfaces and the fully softened strength according to the definition by Mesri and Shahien (2003). The internal and interface shear strength of each component is summarized in the table below:

Liner Component Strength - New Cells

Material	Peak Strength		Residual Strength		Design Strength		Basis
	ϕ' (Deg)	c' / a' (psf)	ϕ' (Deg)	c' / a' (psf)	ϕ' (Deg)	c' / a' (psf)	
Ash / CCR	-	-	-	-	31	40	Lab Testing
PC - GDM	31	40	-	-	31	40	Golder's Internal Direct Shear Database (1996 - 2016)
GDM - TGM	25	90	15	90	25	90	
TGM - GCL	24	90	12	90	24	90	
Internal GCL	27	400	6	400	27	400	
GCL-Clay	25	110	18	340	25	110	Mesri and Shahien (2003) based on Plasticity Index = 30
Clay	25	110	-	-	25	110	

Abbreviations: PC = Protective Cover; TGM = Textured Geomembrane; GDM = Geocomposite Drainage Material; GCL = Geosynthetic Clay Liner; Clay = Compacted Clay Liner

Shear Normal Function for Bottom Liner

Vertical Overburden Stress, σ'_v (psf)	PC	PC-GDM	GDM-TGM	TGM - GCL	Internal GCL	GCL-Clay	Clay	Shear Strength, τ_f' (psf)
0	40	40	90	90	400	110	110	40
300	220	220	230	224	553	250	250	220
1000	641	641	556	535	910	576	576	535
2000	1242	1242	1023	980	1419	1043	1043	980
4000	2443	2443	1955	1871	2438	1975	1975	1871
5000	3044	3044	2422	2316	2940	2442	2442	2316
6000	3645	3645	2888	2761	3457	2908	2908	2761
10000	6049	6049	4753	4542	5495	4773	4773	4542
15000	9053	9053	7085	6768	8043	7105	7105	6768
20000	12057	12057	9416	8995	10591	9436	9436	8995
25000	15062	15062	11748	11221	13138	11768	11768	11221
30000	18066	18066	14079	13447	15686	14099	14099	13447

For this liner, the strength can be defined with a single friction angle (24 degrees) and cohesion (90 psf) since the critical strength comes from the single interface between the textured geomembrane and the geosynthetic clay liner for all vertical overburden stresses except those at low stresses (σ'_v < 300 psf).

Golder Associates	**Cap Liner System**		
	Project Number:	**1520610**	
	Project Name:	**Chesterfield Reymet Road Landfill**	
	Prepared by:	JGM	*Checked by:* GLH
	Date:	June 2016	*Reviewed by:* GLH

Objective:

Develop strength parameters of the cap liner system for use in slope stability analyses.

Liner System Shear Normal Function

The strength of the composite cap liner and overlying protective cover is a composite of the lower bound strengths of the internal and interface shear strength of the various components. Therefore, the strength of a liner system is defined in terms of a shear normal function which discretely defines a shear strength as a function of a given stress.

Golder's internal database of geosynthetic and soil interface strengths were used to provide strength estimates for components where applicable. The geomembrane interface is estimated to engage the full strength of the intermediate cover below the geomembrane since a texture geomembrane is being used. For the intermediate cover, the strength values were assumed for the native soil (protective cover). The internal and interface shear strength of each component is summarized in the table below:

Cap Liner Component Strength

Material	Peak Strength		Residual Strength		Design Strength		Basis
	ϕ' (Deg)	c' / a' (psf)	ϕ' (Deg)	c' / a' (psf)	ϕ' (Deg)	c' / a' (psf)	
PC (Site Soil)	-	-	-	-	28	100	Conservative Value
PC - GDM	28	100	-	-	28	100	Golder's Internal Direct Shear Database (1996 - 2016)
GDM - TGM	25	90	15	90	25	90	
TGM - IC	28	100	-	-	28	100	
IC	-	-	-	-	28	100	Conservative Value
Ash	-	-	-	-	31	40	See Page on Ash

Abbreviations: PC = Protective Cover; GDM = Geocomposite Drainage Material; TGM = Textured Geomembrane; IC = Intermediate Cover

Shear Normal Function for Cap Liner

Vertical Overburden Stress, σ'_v (psf)	PC	PC - GDM	GDM - TGM	TGM - IC	IC	Ash	Shear Strength, τ_f' (psf)
0	100	100	90	100	100	40	40
300	260	260	230	260	260	220	220
1000	632	632	556	632	632	641	556
2000	1163	1163	1023	1163	1163	1242	1023
4000	2227	2227	1955	2227	2227	2443	1955
5000	2759	2759	2422	2759	2759	3044	2422
6000	3290	3290	2888	3290	3290	3645	2888
10000	5417	5417	4753	5417	5417	6049	4753
15000	8076	8076	7085	8076	8076	9053	7085
20000	10734	10734	9416	10734	10734	12057	9416
25000	13393	13393	11748	13393	13393	15062	11748
30000	16051	16051	14079	16051	16051	18066	14079

For this liner, the strength can be defined with a single friction angle (25 degrees) and cohesion (90 psf) since the critical strength comes from the single interface between the textured geomembrane and the geocomposite drainage material for all vertical overburden stresses except those at low stresses (s'v < 300 psf).

DOMINION/REYMET LF CONSTRUCTION/VA
SUMMARY OF SOIL DATA

Sample Identification	Sample Type	Sample Depth	Soil Classi-fication	Natural Moisture %	Atterberg Limits				Grain Size Distribution			Compaction		Gs	Unit Weight		Permeability (cm/sec)	Additional Tests Conducted (See Notes)
					L.L.	P.L.	P.I.	L.I.	% Finer No. 4 Sieve	% Finer No. 200 Sieve	% Finer .005 mm	Maximum Dry Density (pcf)	Optimum Moisture %		Moisture %	Dry (pcf)		
CPS-FA#1	Bulk	-	(ML)	40.0	-	-	-	-	100.0	83.3	-	75.0	28.0	2.24	30.3	71.0	-	T-CU w/pp
CPS-FA#1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	30.5	67.4	-	T-CU w/pp

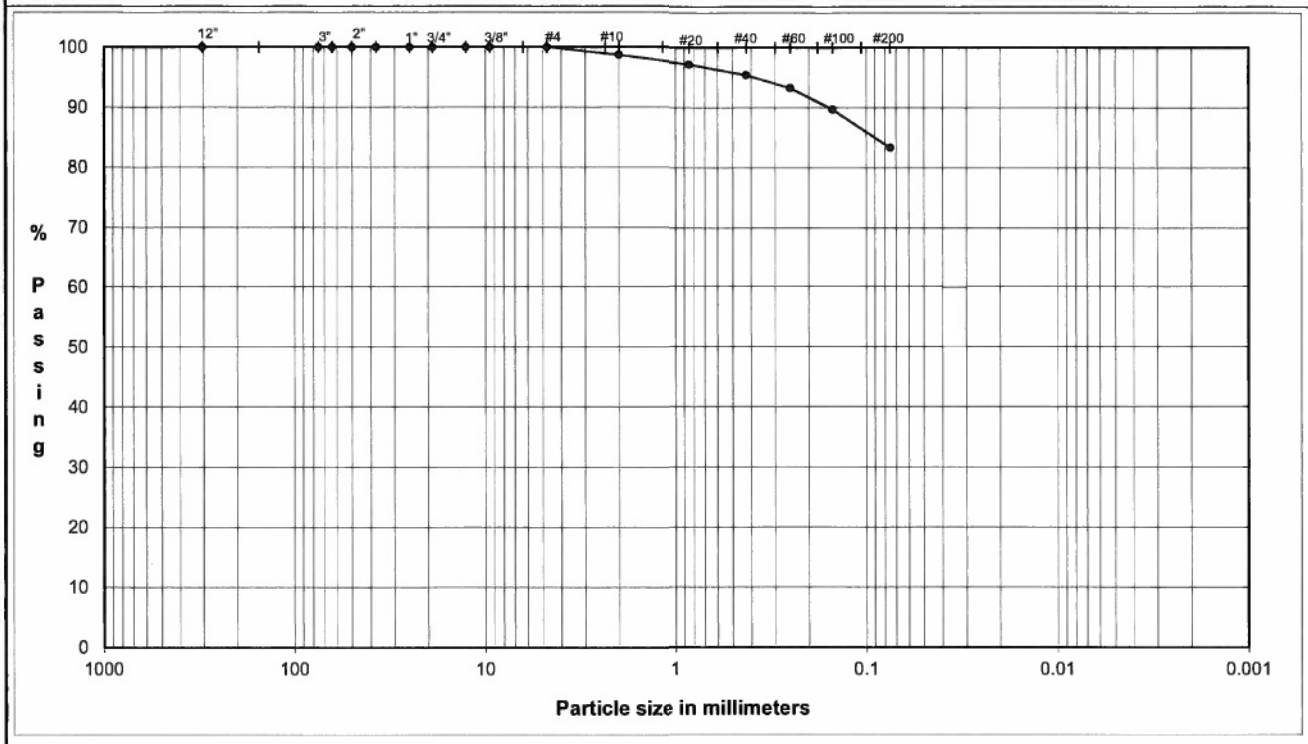
ABBREVIATIONS: LIQUID LIMIT (LL)
PLASTIC LIMIT (PL)
PLASTICITY INDEX (PI)
LIQUIDITY INDEX (LI)
SPECIFIC GRAVITY (Gs)
MOISTURE (Mc)

NOTES: T = TRIAXIAL TEST
U = UNCONFINED COMPRESSION TEST
C = CONSOLIDATION TEST
DS = DIRECT SHEAR TEST
O = ORGANIC CONTENT
P = pH

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D6913, D4318

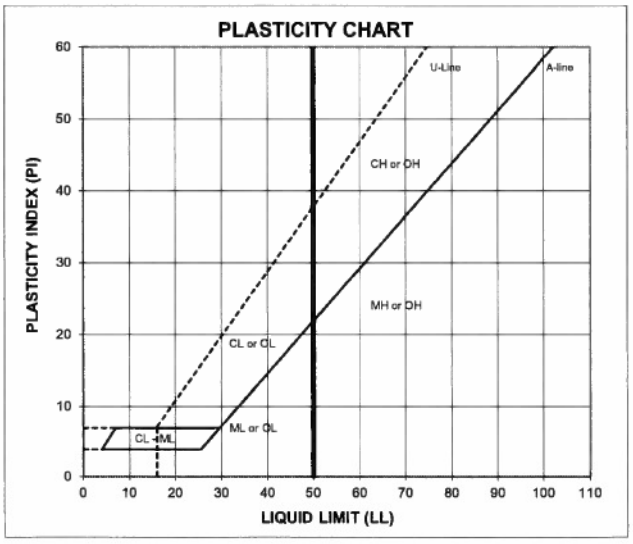
PROJECT NAME: **DOMINION/REYMET LF CONSTRUCTION/VA**
SAMPLE ID: **CPS-FA#1** Depth: -
TYPE: **Bulk**



	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay
COBBLES	GRAVEL		SAND			FINES

U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	**100.0**		
3.0"	75.0	**100.0**	Cobbles	**0.0**
2.5"	63.5	**100.0**		
2.0"	50.0	**100.0**		
1.5"	37.5	**100.0**		
1.0"	25.0	**100.0**		
0.75"	19.0	**100.0**	Coarse Gravel	**0.0**
0.50"	12.7	**100.0**		
0.375"	9.5	**100.0**		
#4	4.8	**100.0**	Fine Gravel	**0.0**
#10	2.00	**98.7**	Coarse Sand	**1.3**
#20	0.85	**97.2**		
#40	0.43	**95.5**	Medium Sand	**3.3**
#60	0.25	**93.3**		
#100	0.15	**89.7**		
#200	0.075	**83.3**	Fine Sand	**12.1**
			Fines	**83.3**



ATTERBERG LIMITS
Method -B (Dry preparation)

M_c	LL	PL	PI	LI
40.0				

LL (oven-dried) / < 0.75 = ORGANIC (OL/OH)

DESCRIPTION: ASH - sandy SILT, fine to coarse; dark gray.

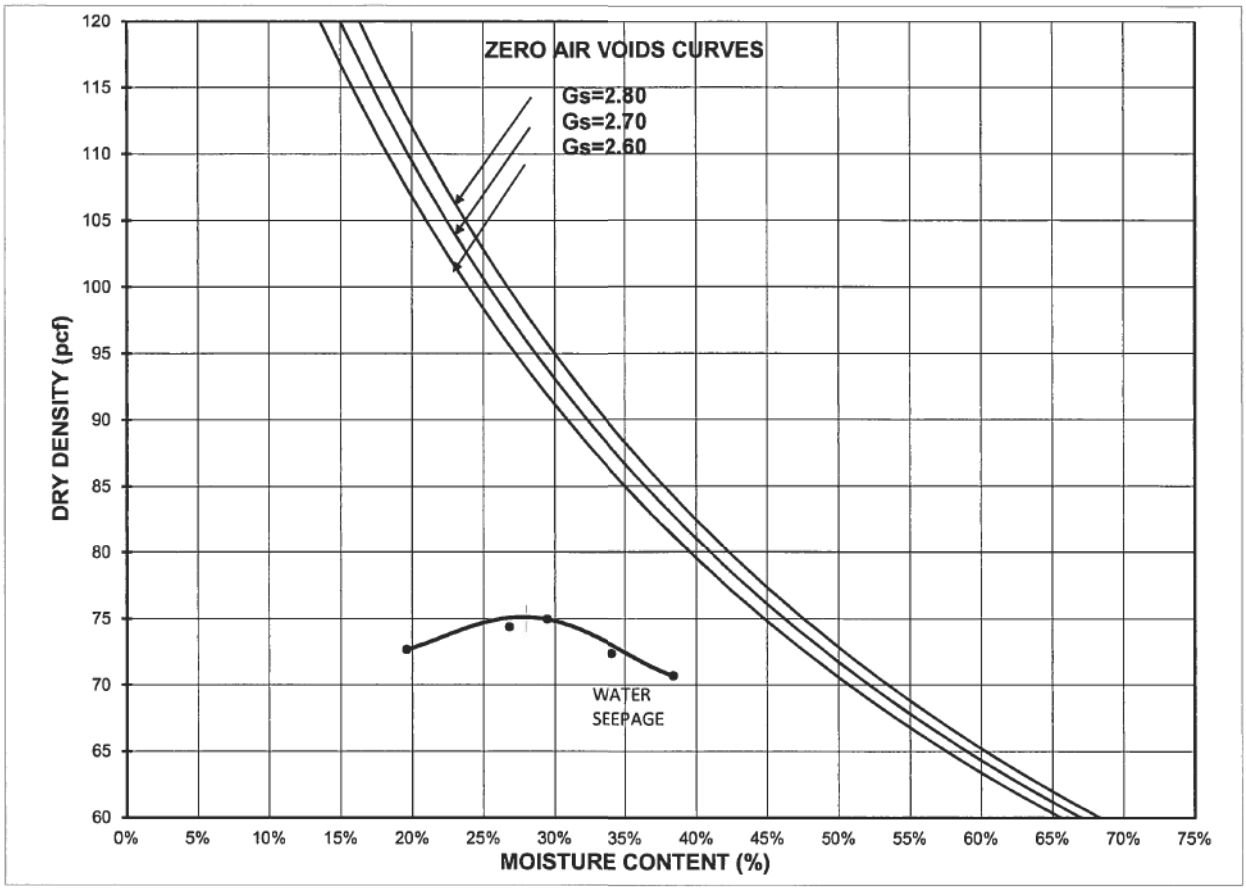
USCS: (ML)

TECH	PWM/SM
DATE	5/26/16
CHECK	
REVIEW	
APPROVE	

MOISTURE / DRY DENSITY CURVE
ASTM D 698 Method A

Mechanical	Standard	Dry Method

PROJECT NAME: DOMINION/REYMET LF CONSTRUCTION/VA
PROJECT NUMBER: 1520610
SAMPLE ID: CPS-FA#1 DEPTH: - SAMPLE TYPE: Bulk



COMPACTION POINTS		
Specimen Number	Dry Density (pcf)	Moisture Content (%)
1	72.7	19.6%
2	74.4	26.8%
3	74.9	29.5%
4	72.4	34.1%
5	70.7	38.4%

Maximum Dry Density (pcf)	75.0
Optimum Moisture (%)	28.0
Corrected Maximum Dry Density (pcf)	
Corrected Optimum Moisture (%)	
As-Received Moisture Content	40.0%
% Retained on # 4 sieve	
% Retained on 3/8" sieve	
% Retained on 3/4" sieve	

DESCRIPTION: ASH - sandy SILT, fine to coarse; dark gray.

USCS: (ML)

CHECK
REVIEW
APPROVE

Boring or Test Pit:	CPS-FA#1		Boring or Test Pit:	CPS-FA#1		Boring or Test Pit:	CPS-FA#1	
Sample:	-		Sample:	-		Sample:	-	
Depth:	-	ft	Depth:	-	ft	Depth:	-	ft
Point No.:	1		Point No.:	2		Point No.:	3	

	Initial			Initial			Initial	
Length =	5.997	in	Length =	6.008	in	Length =	6.001	in
Diameter =	2.790	in	Diameter =	2.790	in	Diameter =	2.790	in
Wet Mass =	1.966	lb	Wet Mass =	1.966	lb	Wet Mass =	1.964	lb
Area =	6.114	in^2	Area =	6.114	in^2	Area =	6.114	in^2
Volume =	36.663	in^3	Volume =	36.731	in^3	Volume =	36.688	in^3
Specific Gravity =	2.42	(ASTM D854)	Specific Gravity =	2.42	(ASTM D854)	Specific Gravity =	2.42	(ASTM D854)
Dry Mass of Solids =	1.507	lb	Dry Mass of Solids =	1.507	lb	Dry Mass of Solids =	1.510	lb
Moisture Content =	30.4%		Moisture Content =	30.4%		Moisture Content =	30.1%	
Wet Unit Weight =	92.7	pcf	Wet Unit Weight =	92.5	pcf	Wet Unit Weight =	92.5	pcf
Dry Unit Weight =	71.1	pcf	Dry Unit Weight =	70.9	pcf	Dry Unit Weight =	71.1	pcf
Void Ratio =	1.13		Void Ratio =	1.13		Void Ratio =	1.12	
Percent Saturation =	66%		Percent Saturation =	65%		Percent Saturation =	65%	

	After Consolidation			After Consolidation			After Consolidation	
Length =	5.970	in	Length =	5.994	in	Length =	5.948	in
Diameter =	2.720	in	Diameter =	2.712	in	Diameter =	2.682	in
Area =	5.812	in^2 (Method B)	Area =	5.776	in^2 (Method B)	Area =	5.649	in^2 (Method B)
Volume =	34.700	in^3	Volume =	34.621	in^3	Volume =	33.603	in^3
Moisture Content =	41.8%		Moisture Content =	41.6%		Moisture Content =	39.0%	
Wet Unit Weight =	106.4	pcf	Wet Unit Weight =	106.5	pcf	Wet Unit Weight =	107.9	pcf
Dry Unit Weight =	75.1	pcf	Dry Unit Weight =	75.2	pcf	Dry Unit Weight =	77.6	pcf
Void Ratio =	1.01		Void Ratio =	1.01		Void Ratio =	0.95	
Percent Saturation =	100%		Percent Saturation =	100%		Percent Saturation =	100%	

B Parameter =	0.95		B Parameter =	0.96		B Parameter =	0.96	
Shear Rate =	0.045%	/min.	Shear Rate =	0.045%	/min.	Shear Rate =	0.045%	/min.
t_{50} =	0.15	min.	t_{50} =	0.15	min.	t_{50} =	0.15	min.
Strain at Failure =	8.0%		Strain at Failure =	9.0%		Strain at Failure =	7.2%	

Cell Pressure =	80.0	psi	Cell Pressure =	100.0	psi	Cell Pressure =	120.0	psi
Back Pressure =	60.0	psi	Back Pressure =	60.0	psi	Back Pressure =	60.0	psi
Confining Pressure =	20.0	psi	Confining Pressure =	40.0	psi	Confining Pressure =	60.0	psi

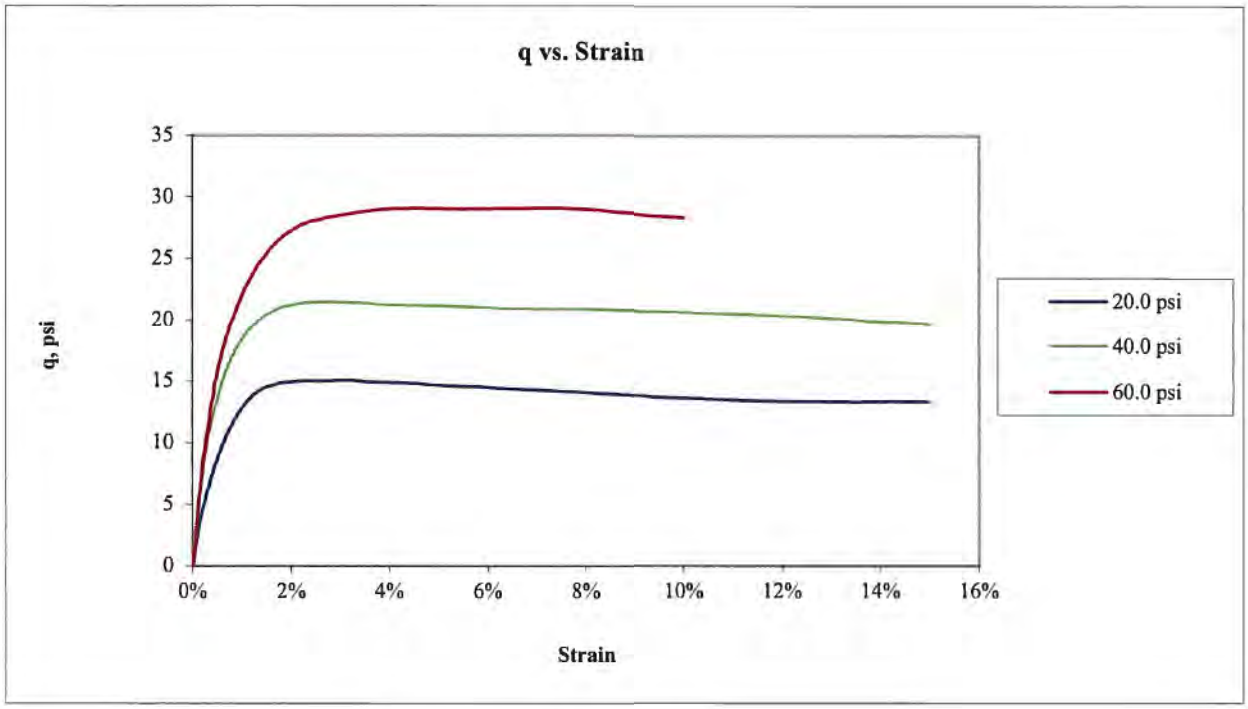
Notes: Sample description: ASH - (ML) sandy SILT, fine to coarse; dark gray.

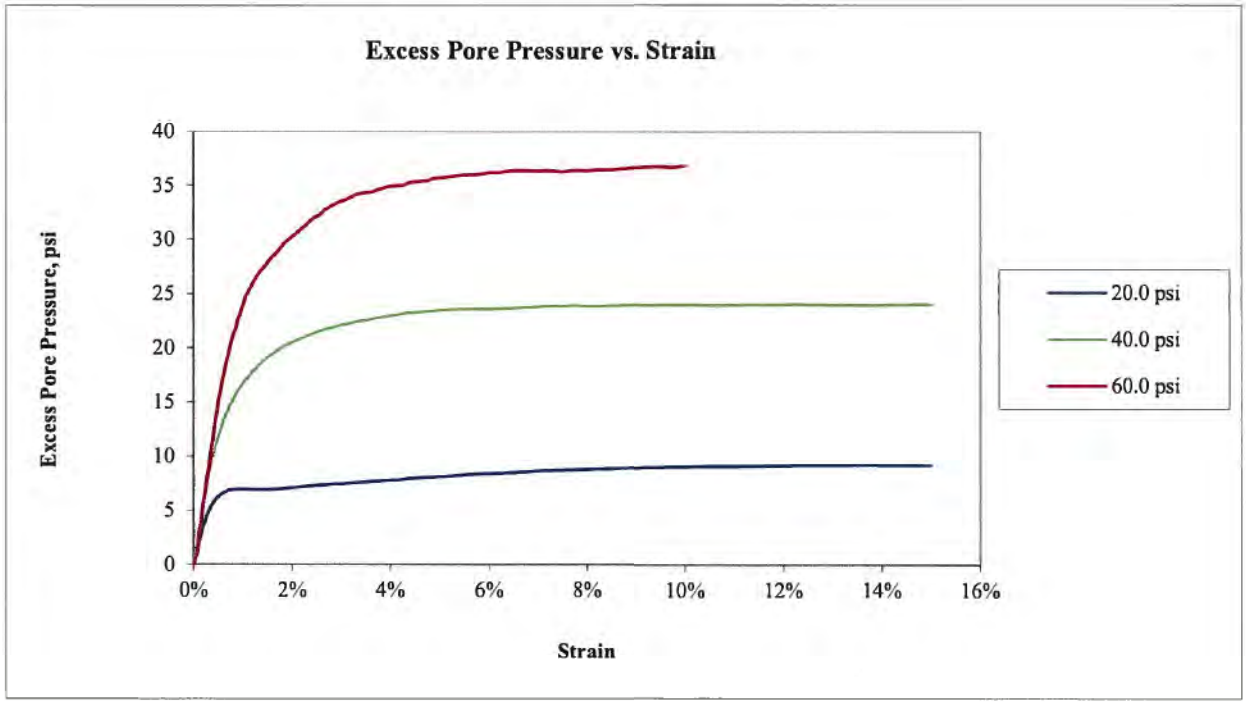
Atterberg limits: LL = - PL = - PI = - (ASTM D4318)

Percent finer: 3/4 in. = 100% No. 4 = 100% No. 200 = 83% (ASTM D422, refer to separate report for gradation curve)

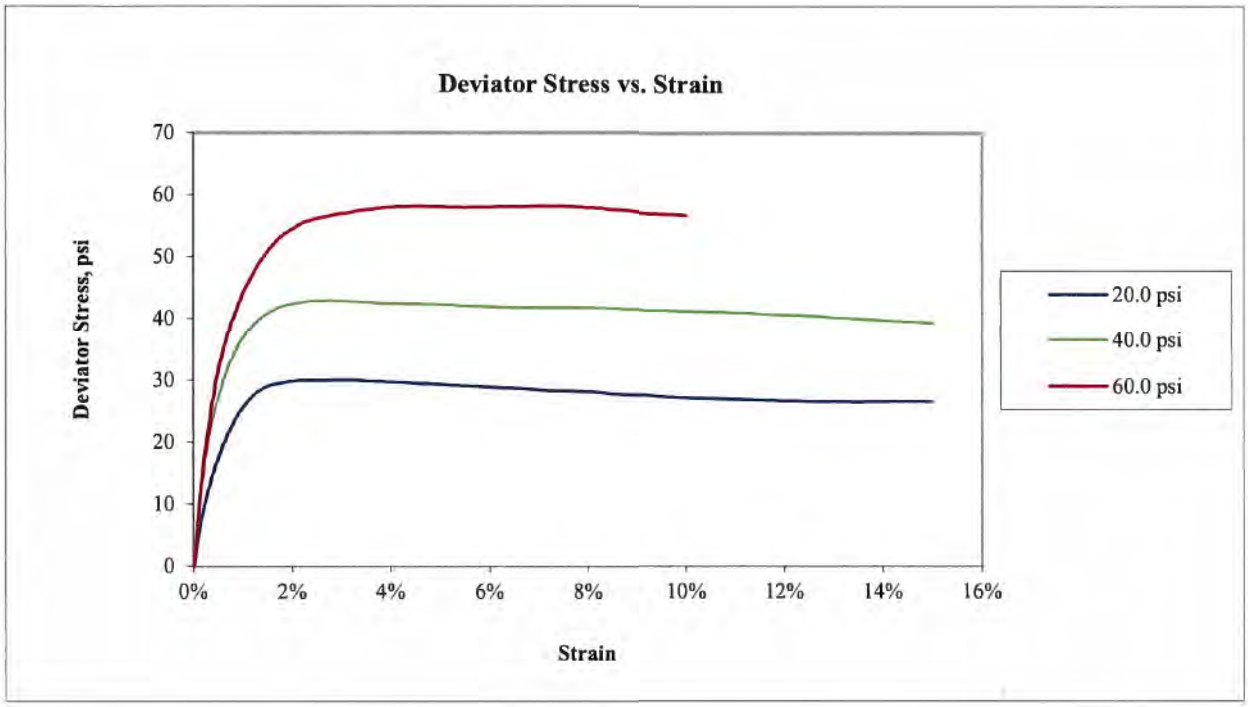
Specimen type:		Intact	X	Reconstituted	Remold targets:	71.3	pcf	30.0% moisture
Moisture from:		Cuttings	X	Entire specimen				
Saturation method:	X	Wet		Dry				
Failure criterion:	X	$(\sigma'_1/\sigma'_3)_{max}$		$(\sigma'_1-\sigma'_3)_{max}$	% strain			
Membrane effect:	X	Corrected		Not Corrected				

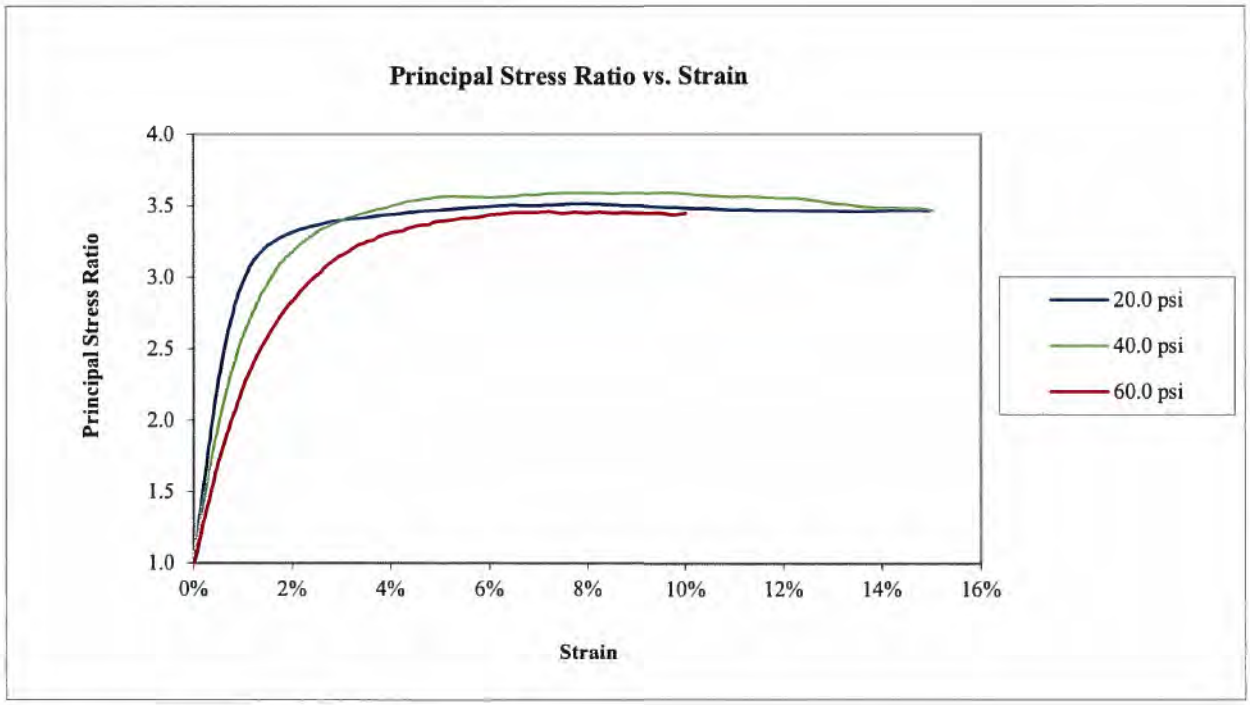
Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **SAMPLE AND TEST DATA**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:** MW	**Reviewed:** **Approved:**	**Start Date:** **5/30/2016**	**Job Number:** **1520610**	**Figure:** **1**



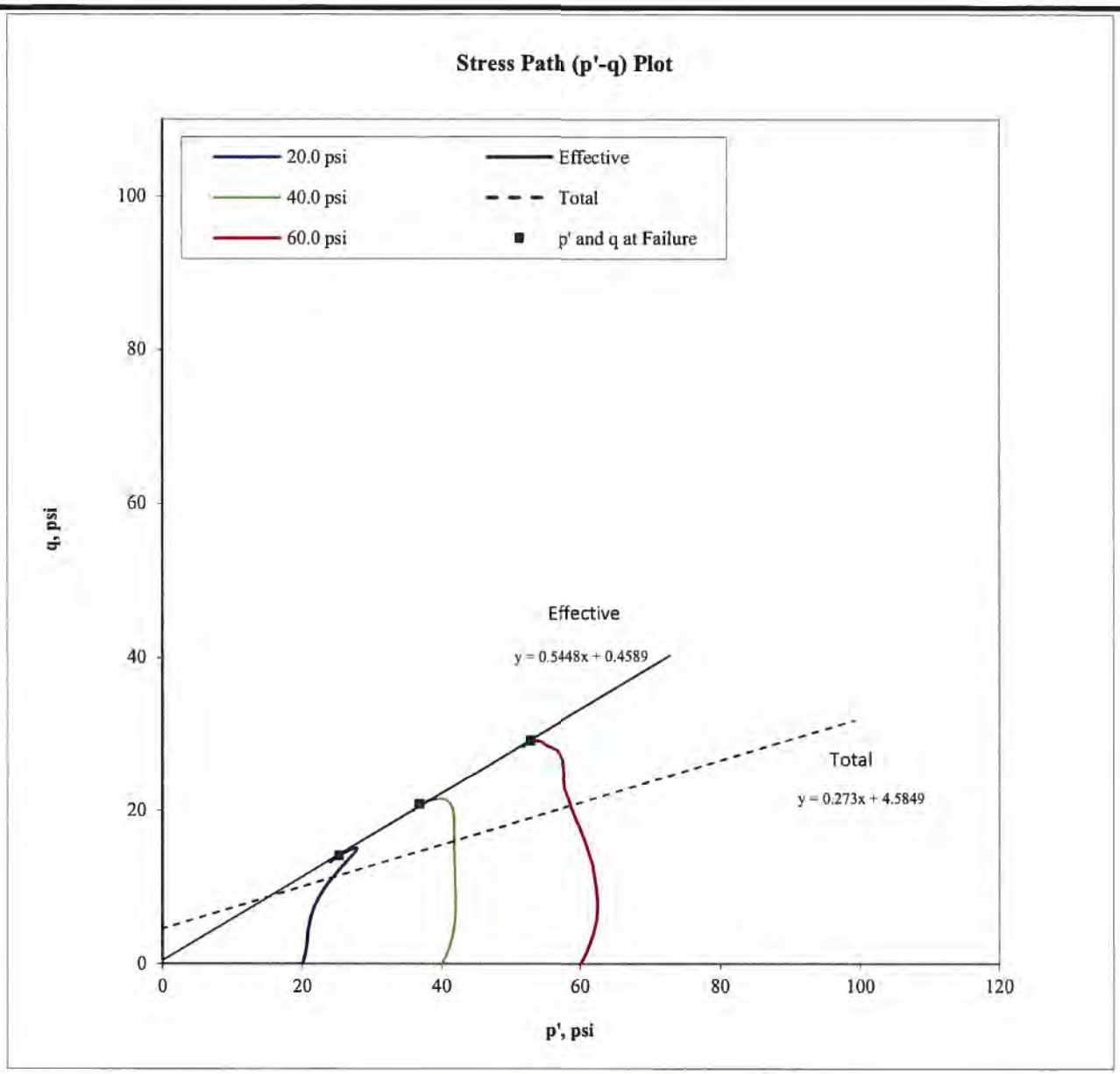


Golder Associates Inc. Atlanta, Georgia	Title: ASTM D4767 CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT q AND EXCESS PORE PRESSURE PLOTS				
Job Short Title: DOMINION/REYMET LF CONSTRUCTION/VA					
Sample:	Technician: SM/SDM	Reviewed:	Start Date:	Job Number:	Figure:
	Check:	Approved:			
CPS - FA#1			5/30/2016	1520610	2





Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **DEVIATOR STRESS AND PRINCIPAL STRESS RATIO PLOT**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:**	**Reviewed:** **Approved:**	**Start Date:** **5/30/2016**	**Job Number:** **1520610**	**Figure:** **3**



Confining Pressure (psi)	p at failure (psi)	p' at failure (psi)	q at failure (psi)
20.0	34.1	25.3	14.1
40.0	60.8	36.8	20.8
60.0	89.1	52.8	29.1

Effective

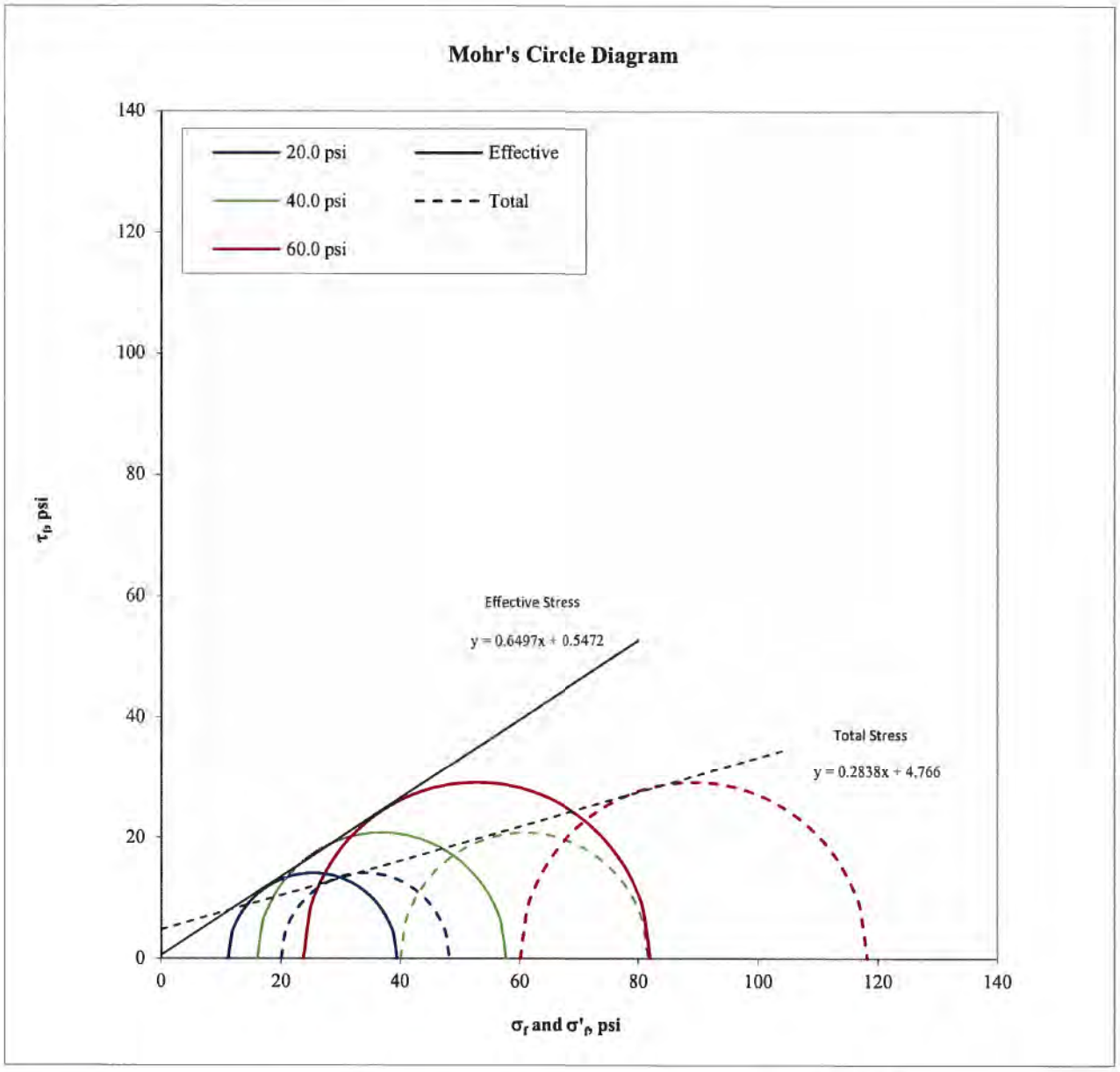
α ' =	28.6	degree
a' =	0.5	psi

Total

α =	15.3	degree
a =	4.6	psi

Note: The laboratory testing relates only to the sample tested. GAI neither accepts responsibility for nor makes claims to the final use and purpose of the material.

Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **STRESS PATH PLOT**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:** MWM	**Reviewed:** **Approved:**	**Start Date:** **5/30/2016**	**Job Number:** **1520610**	**Figure:** **4**



Confining Pressure (psi)	σ'_1 at failure (psi)	σ'_3 at failure (psi)	σ_1 at failure (psi)	σ_3 at failure (psi)
20.0	39.4	11.2	48.2	20.0
40.0	57.6	16.0	81.6	40.0
60.0	81.9	23.7	118.2	60.0

Effective

ϕ' =	33.0	degree
c' =	0.5	psi

Total

ϕ =	15.8	degree
c =	4.8	psi

Note: The laboratory testing relates only to the sample tested. GAI neither accepts responsibility for nor makes claims to the final use and purpose of the material.

Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **MOHR'S CIRCLE DIAGRAM**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:**	**Reviewed:** **Approved:**	**Start Date:** **5/30/2016**	**Job Number:** **1520610**	**Figure:** **5**

20.0 psi

40.0 psi

60.0 psi

NOTE: Pore pressure dropped and stress load increased beyond 10% strain. Eliminated readings after 10% strain.

Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **SPECIMENS PHOTOGRAPH - 20.0 40.0 60.0 psi**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:**	**Reviewed:** **Approved:**	**Start Date:** **5/30/2016**	**Job Number:** **1520610**	**Figure:** **6**

Boring or Test Pit:	CPS-FA#1		Boring or Test Pit:	CPS-FA#1		Boring or Test Pit:	CPS-FA#1	
Sample:	-		Sample:	-		Sample:	-	
Depth:	-	ft	Depth:	-	ft	Depth:	-	ft
Point No.:	1		Point No.:	2		Point No.:	3	

	Initial		Initial		Initial	
Length =	5.997	in	5.993	in	5.982	in
Diameter =	2.790	in	2.790	in	2.790	in
Wet Mass =	1.865	lb	1.865	lb	1.865	lb
Area =	6.114	in^2	6.114	in^2	6.114	in^2
Volume =	36.663	in^3	36.639	in^3	36.572	in^3
Specific Gravity =	2.24	(ASTM D854)	2.24	(ASTM D854)	2.24	(ASTM D854)
Dry Mass of Solids =	1.427	lb	1.429	lb	1.430	lb
Moisture Content =	30.6%		30.4%		30.4%	
Wet Unit Weight =	87.9	pcf	87.9	pcf	88.1	pcf
Dry Unit Weight =	67.3	pcf	67.4	pcf	67.6	pcf
Void Ratio =	1.08		1.07		1.07	
Percent Saturation =	64%		64%		64%	

	After Consolidation		After Consolidation		After Consolidation	
Length =	5.950	in	5.919	in	5.924	in
Diameter =	2.746	in	2.739	in	2.724	in
Area =	5.922	in^2 (Method B)	5.894	in^2 (Method B)	5.826	in^2 (Method B)
Volume =	35.235	in^3	34.887	in^3	34.511	in^3
Moisture Content =	44.4%		43.4%		42.4%	
Wet Unit Weight =	101.1	pcf	101.5	pcf	102.0	pcf
Dry Unit Weight =	70.0	pcf	70.8	pcf	71.6	pcf
Void Ratio =	1.00		0.97		0.95	
Percent Saturation =	100%		100%		100%	

B Parameter =	0.97		B Parameter =	0.96		B Parameter =	0.96	
Shear Rate =	0.045%	/min.	Shear Rate =	0.045%	/min.	Shear Rate =	0.045%	/min.
t_{50} =	0.57	min.	t_{50} =	0.15	min.	t_{50} =	0.15	min.
Strain at Failure =	9.5%		Strain at Failure =	10.4%		Strain at Failure =	10.8%	

Cell Pressure =	80.0	psi	Cell Pressure =	100.0	psi	Cell Pressure =	120.0	psi
Back Pressure =	60.0	psi	Back Pressure =	60.0	psi	Back Pressure =	60.0	psi
Confining Pressure =	20.0	psi	Confining Pressure =	40.0	psi	Confining Pressure =	60.0	psi

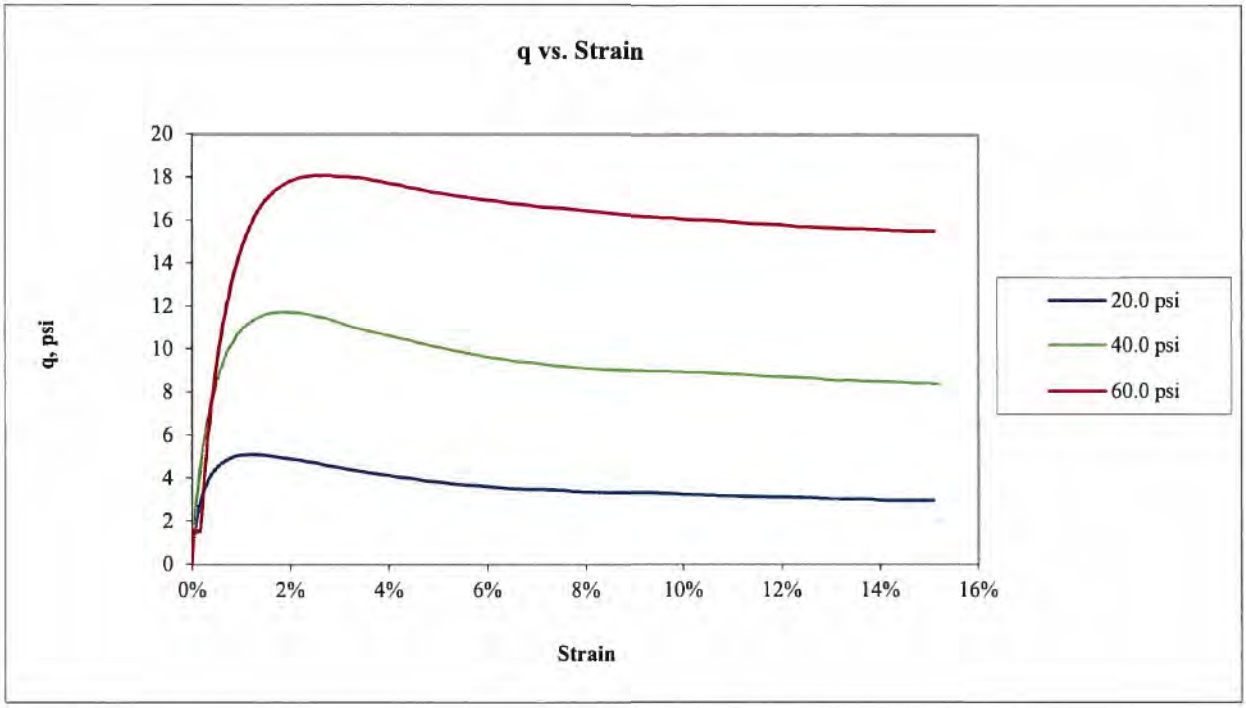
Notes: Sample description: ASH - (ML) sandy SILT, fine to coarse; dark gray

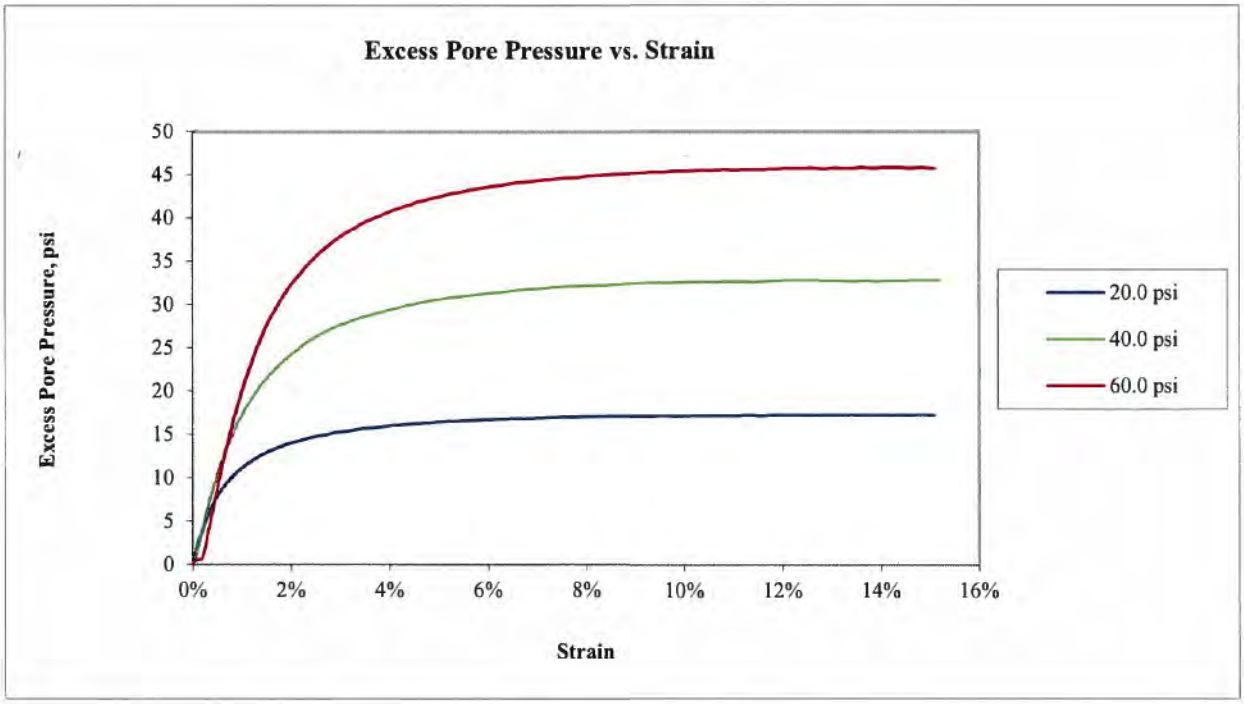
Atterberg limits: LL = - PL = - PI = - (ASTM D4318)

Percent finer: 3/4 in. = 100% No. 4 = 100% No. 200 = 83% (ASTM D422, refer to separate report for gradation curve)

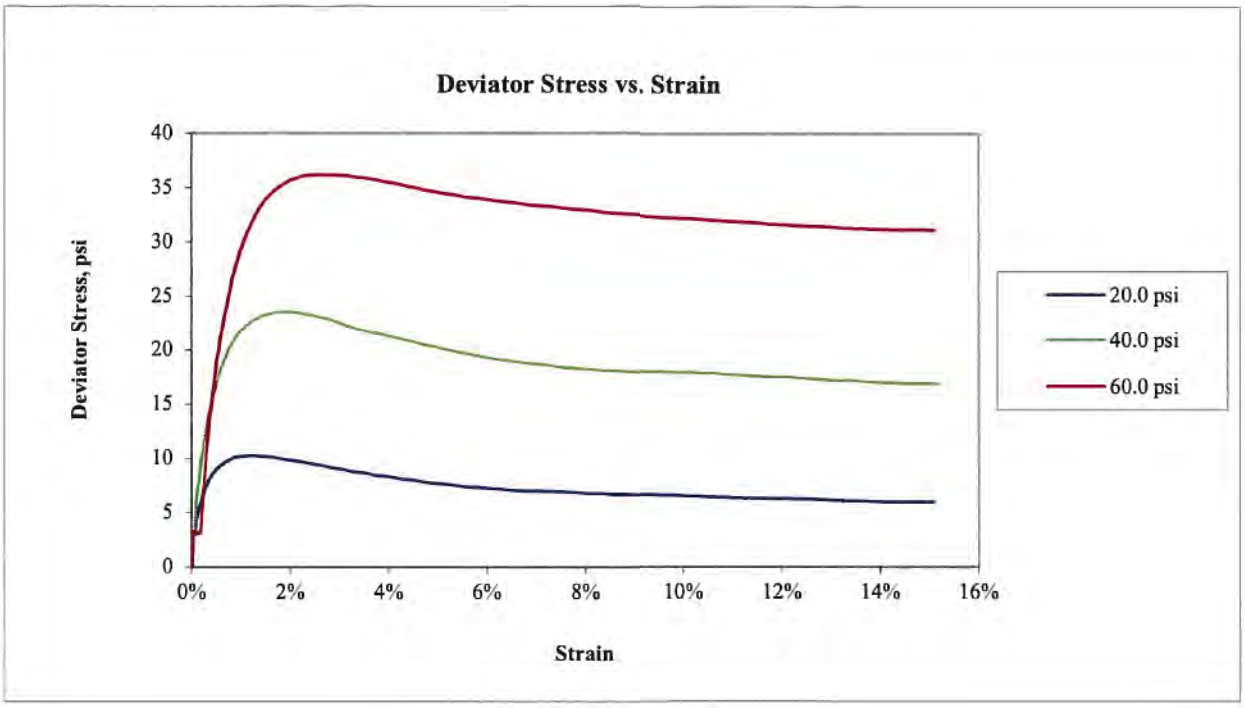
Specimen type:		Intact	X	Reconstituted	Remold targets:	67.5	pcf	30.0% moisture
Moisture from:		Cuttings	X	Entire specimen				
Saturation method:	X	Wet		Dry				
Failure criterion:	X	$(\sigma'_1/\sigma'_3)_{max}$		$(\sigma'_1-\sigma'_3)_{max}$	% strain			
Membrane effect:	X	Corrected		Not Corrected				

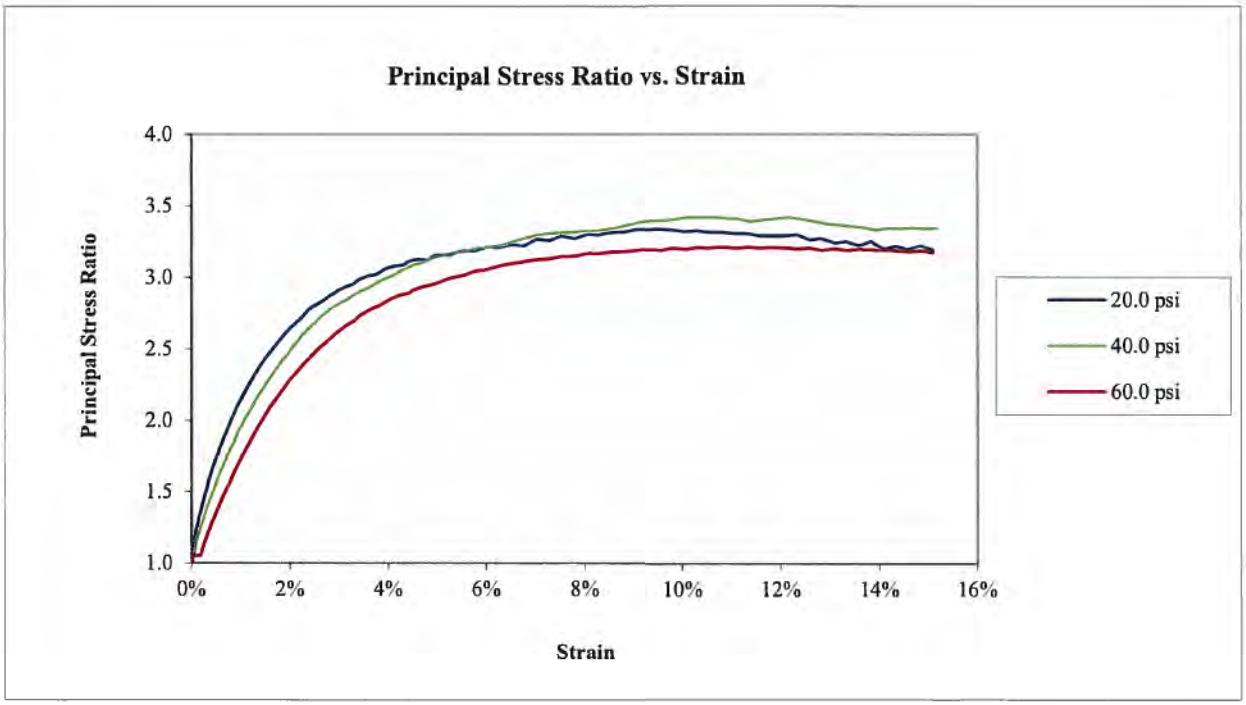
Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **SAMPLE AND TEST DATA**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:** PWN	**Reviewed:** **Approved:**	**Start Date:** **6/1/2016**	**Job Number:** **1520610**	**Figure:** **1**



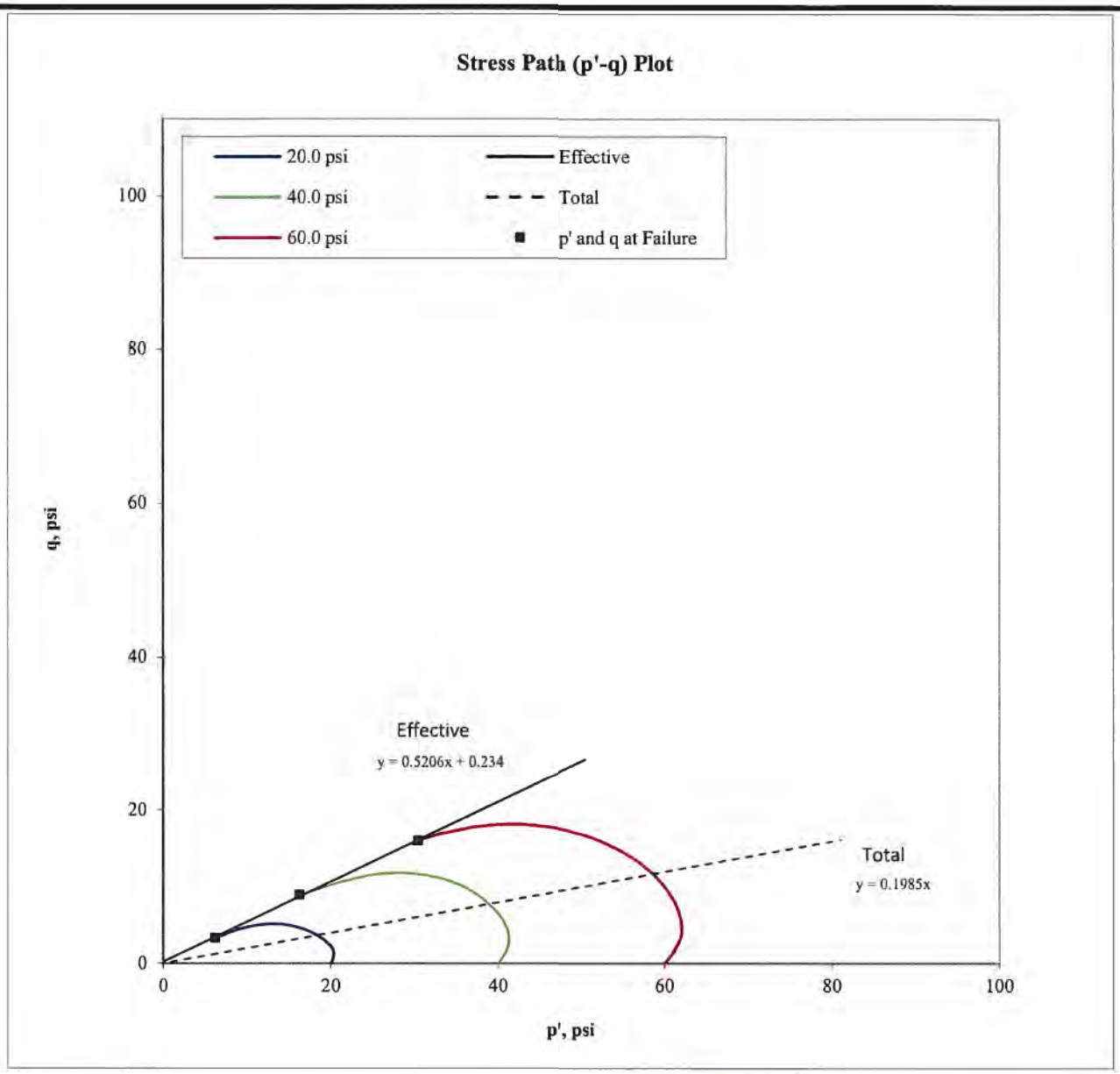


Golder Associates Inc. Atlanta, Georgia	Title: ASTM D4767 CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT q AND EXCESS PORE PRESSURE PLOTS				
Job Short Title: DOMINION/REYMET LF CONSTRUCTION/VA					
Sample: CPS - FA#1	Technician: SM/SDM Check:	Reviewed: Approved:	Start Date: 6/1/2016	Job Number: 1520610	Figure: 2





Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **DEVIATOR STRESS AND PRINCIPAL STRESS RATIO PLOT**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:**	**Reviewed:** **Approved:**	**Start Date:** **6/1/2016**	**Job Number:** **1520610**	**Figure:** **3**



Confining Pressure (psi)	p at failure (psi)	p' at failure (psi)	q at failure (psi)
20.0	23.3	6.2	3.3
40.0	48.9	16.3	8.9
60.0	76.0	30.4	16.0

Effective

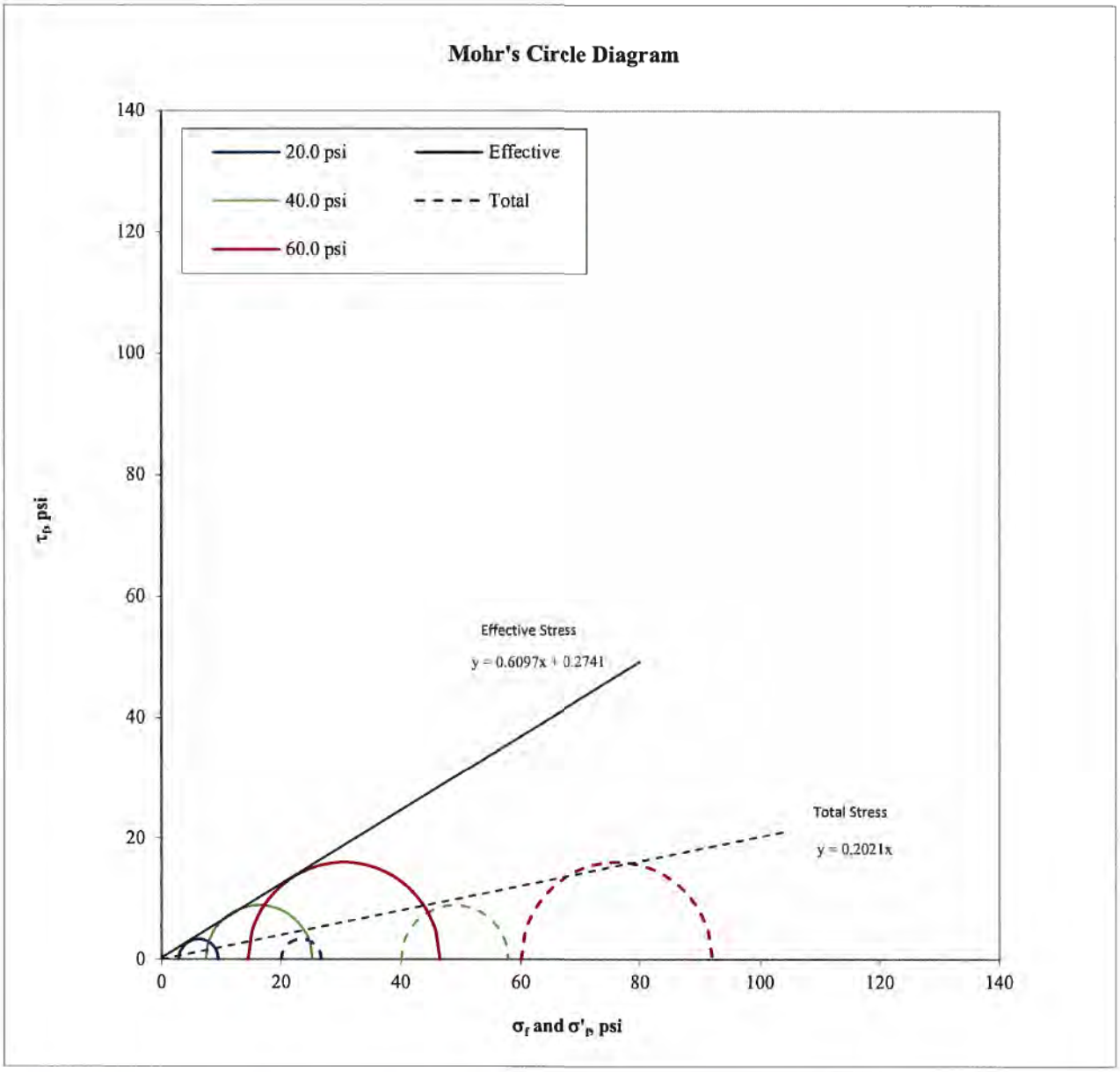
α ' =	27.5	degree
a' =	0.2	psi

Total

α =	11.2	degree
a =	0.0	psi

Note: The laboratory testing relates only to the sample tested. GAI neither accepts responsibility for nor makes claims to the final use and purpose of the material.

Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **STRESS PATH PLOT**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:**	**Reviewed:** **Approved:**	**Start Date:** **6/1/2016**	**Job Number:** **1520610**	**Figure:** **4**



Confining Pressure (psi)	σ'_1 at failure (psi)	σ'_3 at failure (psi)	σ_1 at failure (psi)	σ_3 at failure (psi)
20.0	9.5	2.8	26.6	20.0
40.0	25.2	7.4	57.8	40.0
60.0	46.4	14.4	92.0	60.0

Effective

ϕ' =	31.4	degree
c' =	0.3	psi

Total

ϕ =	11.4	degree
c =	0.0	psi

Note: The laboratory testing relates only to the sample tested. GAI neither accepts responsibility for nor makes claims to the final use and purpose of the material.

Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **MOHR'S CIRCLE DIAGRAM**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:** PWK	**Reviewed:** **Approved:**	**Start Date:** **6/1/2016**	**Job Number:** **1520610**	**Figure:** **5**

20.0 psi

40.0 psi

60.0 psi

Golder Associates Inc. **Atlanta, Georgia**	**Title:** **ASTM D4767** **CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST REPORT** **SPECIMENS PHOTOGRAPH - 20.0 40.0 60.0 psi**				
Job Short Title: **DOMINION/REYMET LF CONSTRUCTION/VA**					
Sample: **CPS - FA#1**	**Technician:** **SM/SDM** **Check:**	**Reviewed:** **Approved:**	**Start Date:** **6/1/2016**	**Job Number:** **1520610**	**Figure:** **6**

ATTACHMENT B

Seismic Hazard Evaluation

Date:	June 10, 2016	**Made by:**	Grant Martin
Project No.:	1520610	**Checked by:**	Gregory Hebeler
Site Name:	Chesterfield Reymet Road Landfill	**Reviewed by:**	Gregory Hebeler
Subject	**SEISMIC HAZARD EVALUATION**		

1.0 INTRODUTION

In 2010, Golder submitted plans for an ash landfill to be constructed near Reymet Rd in Richmond VA to receive ash from operations at Dominion's Chesterfield Power Station. As part of CCR rule requirements issued by the federal EPA under subtitle D, calculations relating to the seismic hazard were required to be updated. Specifically, calculations for slope stability and liquefaction potential are required. This package discusses the seismic hazard used as input for the required calculations.

2.0 SEISMIC HAZARD EVALUATION

The seismic hazard at the proposed landfill location was evaluating using two main methods: (1) spectral response curves provided by the National Earthquake Hazards Reduction Program (NEHRP) and (2) seismic hazard maps and tools provided by the United States Geological Survey (USGS).

2.1 2015 NEHRP Provisions

A spectral response curve is often useful in designing structures of known period including buildings and geotechnical structures. A spectral response curve gives accelerations as a function of period such that an appropriate acceleration can be applied in the design of structures. The 2015 NEHRP Provisions provide a method of calculating a spectral response curve and peak ground acceleration (PGA) for a given location in the United States. This method requires site location, seismic site class, and risk category to calculate the response curve.

The seismic site class can be defined using shear wave velocities, blow counts, or undrained shear strengths as shown in the table below.

Golder Associates Inc.
3730 Chamblee Tucker Road
Atlanta, GA 30341 USA
Tel: (770) 496-1893 Fax: (770) 934-9476 www.golder.com

Golder Associates: Operations in Africa, Asia, Australasia, Europe, North America and South America

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation

Project No.:	1520610	Made by:	G. Martin
Site Name:	Chesterfield / Reymet Rd Landfill	Checked by:	G. Hebeler
Date:	June 10, 2016	Reviewed by:	G. Hebeler

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20 • Moisture content w ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		
For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²			

The site class was assessed using blow counts averaged in the upper 100 ft using the following equation:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

The average blow count for this site is 27, and the site exploration did not yield any zone of soft clay which had moisture contents greater than 40%; therefore, site class D was used in calculation of the spectral response curve. This average blow count of 27 can be roughly equated to an average shear wave velocity (Vs30) of 800 ft/s (~250 m/s) by interpolating from the table above.

Using the 2015 NEHRP Provisions tools found on USGS's website, the response curve was calculated for the proposed landfill site location. The resulting response curve is shown in the figure below, and the PGA for the site class B/C boundary was calculated to be 0.117g. Adjusted for the site class (D), the PGA was calculated to be 0.183g. For more details on the calculation, see the attached output document provided by the online hazard website.

Project No.:	1520610	Made by:	G. Martin
Site Name:	Chesterfield / Reymet Rd Landfill	Checked by:	G. Hebeler
Date:	June 10, 2016	Reviewed by:	G. Hebeler

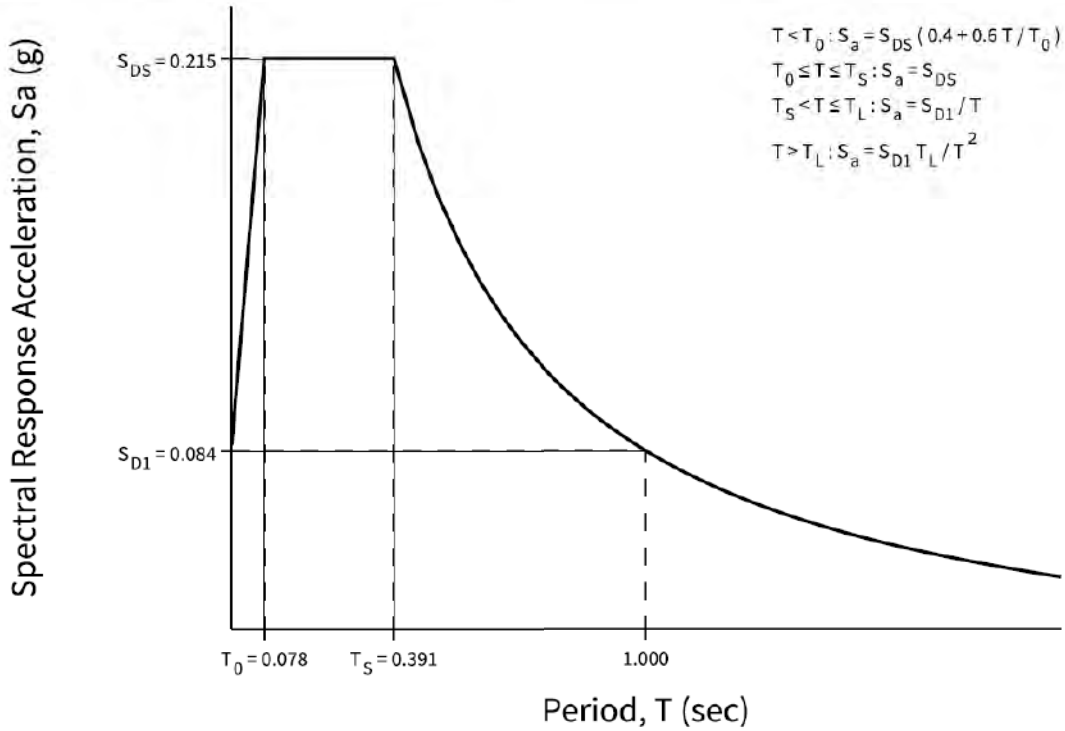


Figure 1: Spectral Response Curve calculated using the 2015 NEHRP Provisions

2.2 USGS Seismic Hazard Maps and Tools

The USGS provides seismic hazard maps and tools for aid in designing structures against seismic hazards. The most recent documents provided by the USGS were published in 2014 and include only maps quantifying the hazard in terms of PGA and accelerations at periods of 1.0 s and 0.2 s. The USGS has not released additional tools to quantify other important hazard considerations such as earthquake magnitude and deaggregation of probabilistic motions. These additional hazard considerations are not included in the USGS's 2014 published documents. However, the USGS's previous publication in 2008 includes these additional hazard considerations.

2.2.1 2014 vs 2008 Seismic Hazard Publications

In the area near the Chesterfield Power Station and proposed landfill location, seismic hazard maps saw significant changes in earthquake hazard magnitudes between 2008 and 2014. Specifically, the PGA associated with an earthquake event having a 2% probably of exceedance in 50 years (2% / 50yr event, associated with a return period of 2475 years) increased at the proposed landfill location from 0.095g to 0.117g. The 2014 PGA represents about a 23% increase from the 2008 PGA.

Project No.:	1520610	Made by:	G. Martin
Site Name:	Chesterfield / Reymet Rd Landfill	Checked by:	G. Hebeler
Date:	June 10, 2016	Reviewed by:	G. Hebeler

Petersen *et al.* (2014) documented additional detail on the differences and noted that changes were primarily due to updates in the historical earthquake catalog and adaptive smoothing techniques used to calculate the earthquake hazard. For the seismic hazard at the proposed landfill location, the most influential addition to the historical earthquake catalog was the 2011 **M**[1] 5.8 Mineral, Virginia earthquake. The Mineral, Virginia earthquake epicenter was located about 48 miles (77 km) northwest of the proposed landfill site. Inclusion of the Mineral earthquake in the earthquake catalog for the 2014 National Seismic Hazard Model (NSHM) increases the earthquake activity rate estimate near the project site as compared to the 2008 NSHM, which was developed before the Mineral earthquake. Additionally, the 2014 NSHM model adopted new adaptive smoothing techniques that were not included in the 2008 NSHM. These techniques increase the activity rate surrounding larger, historical earthquake epicenters such as the 2011 Mineral earthquake. Thus, the inclusion of the Mineral Earthquake likely had a greater increase in the activity rate of the 2014 NSHM than would have occurred in the 2008 NSHM. Together, the updated seismic hazard catalog and the newly adopted smoothing techniques result in an increased seismic hazard at the project site.

2.2.2 2014 Seismic Hazard Maps

From the 2014 seismic hazard maps, the PGA for the 2%/50yr event was determined to be 0.117g (see figure below). This PGA is consistent with the PGA derived from the NEHRP method and is appropriate to use in seismic analyses requiring PGA.

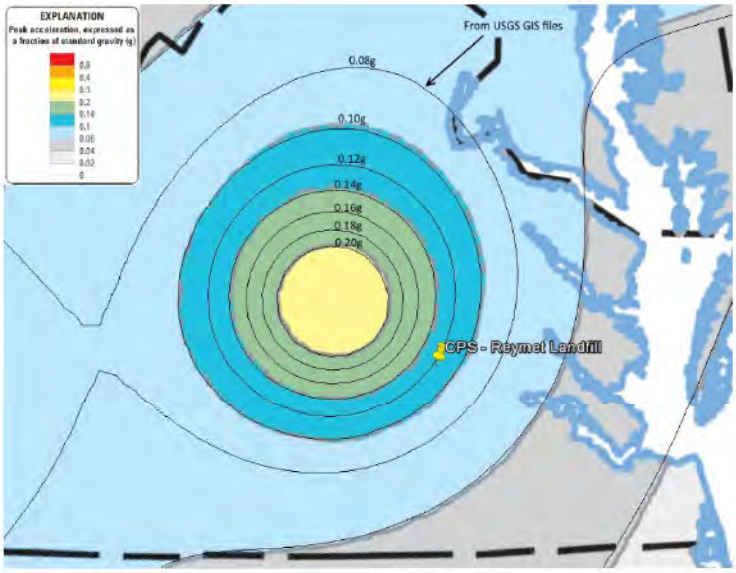


Figure 2: USGS 2014 Seismic Hazard Map - PGA, 2%/50yr (USGS 2014)

[1] Moment magnitude.

Project No.:	1520610	Made by:	G. Martin
Site Name:	Chesterfield / Reymet Rd Landfill	Checked by:	G. Hebeler
Date:	June 10, 2016	Reviewed by:	G. Hebeler

2.2.3 Input to Liquefaction Analysis

Golder used the NCEER method (Youd et al. 2001) to calculate a screening level factor of safety against liquefaction. This liquefaction analysis requires an estimate of the maximum acceleration at the ground surface (calculated above) and an estimate of the design earthquake magnitude.

The selection of either the mean or modal magnitude produces inconsistent risks of liquefaction because the relationship between duration (represented by magnitude) and liquefaction potential is non-linear. Kramer (2008) suggests that the best way to handle this issue is to perform liquefaction calculations for all magnitudes and to weight the results according to the relative contribution of each magnitude.

Golder implemented this approach by recognizing that the MSF is the only term in the NCEER approach that is affected by the magnitude selection. Golder calculated a weighted-average MSF (weighted by the relative contribution of each magnitude) and then calculated the magnitude corresponding to that MSF. The latest deaggregation hazard data (2008 deaggregated NSHM model, available on the USGS website) was used to provide the listing of the contributing earthquake magnitudes to feed the weighted MSF calculation. Golder calculated the weighted equivalent earthquake magnitude to be 5.52 for a 2% in 50 year seismic hazard. This value (5.52) is less than the mean magnitude (5.71) and is greater than the modal magnitude (4.80).

3.0 SUMMARY

The seismic hazard corresponding to a 2% probability of exceedance in 50 years was taken from the following online USGS Tools: 2015 NEHRP Provisions, 2014 USGS Seismic Hazard Maps, and 2008 USGS Deaggregation Seismic Hazard Tools. The following seismic hazard characteristics were used in design:

- Site Class D
- B/C PGA = 0.117g
- Site Class D Adjusted Ground Surface PGA = 0.183g
- Design Earthquake Magnitude = 5.52

4.0 REFERENCES

Jibson, R.W. and Harp, E.L. 2012. Extraordinary Distance Limits of Landslides Triggered by the 2011 Mineral, Virginia, earthquake. Bulletin of the Seismological Society of America, Vol. 102 No. 6, pp. 2368-2377.

Kramer, S.L. (2008). "Evaluation of Liquefaction Hazards in Washington State" Final Research report WA-RD 668.1, December 2008.

Project No.:	1520610	Made by:	G. Martin
Site Name:	Chesterfield / Reymet Rd Landfill	Checked by:	G. Hebeler
Date:	June 10, 2016	Reviewed by:	G. Hebeler

"NEHRP Recommended Seismic Provisions for New Buildings and Other Structures. 2015 Edition." FEMA.gov. N.p., n.d. Web. 01 June 2016.

Petersen, M.D., Frankel, A. D., Harmsen, S.C., Mueller, C.S., Haller, K. M., Wheeler, R.L., Wesson, R.L., Zeng, Y., Boyd, O.S., Perkins, D.M., Luco, N., Field, E.H., Wills, C.J., and Rukstales K.S. 2008. Documentation for the 2008 Update of the United States National Seismic Hazard Maps. USGS Open-File Report 2008–1128.

USGS 2008 Seismic Hazard Deaggregation Tool, http://geohazards.usgs.gov/deaggint/2008/

USGS. 2014 "Lower 48 Maps and Data." Lower 48 Maps and Data. N.p., n.d. Web. 01 June 2016.

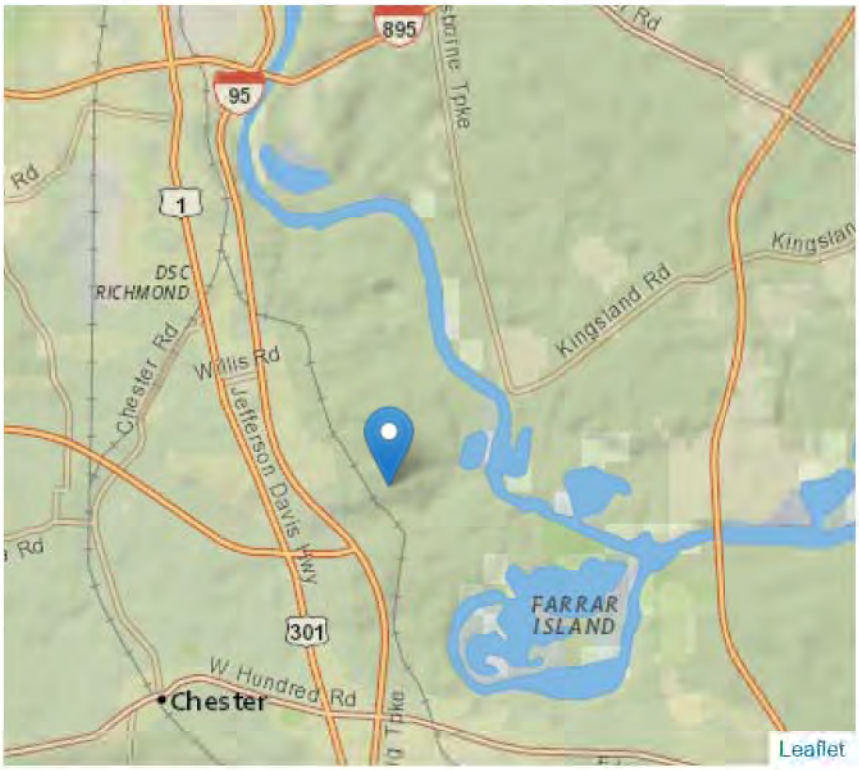
Youd, T.L. et al. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, vol. 127, No. 4, April 2001.

U.S. Geological Survey - Earthquake Hazards Program

Chesterfield

Latitude = 37.386°N, Longitude = 77.403°W

Location



Reference Document

2015 NEHRP Provisions

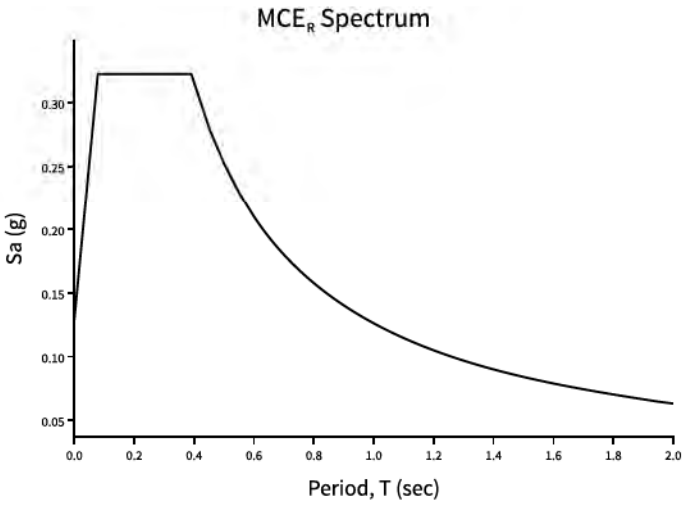
Site Class

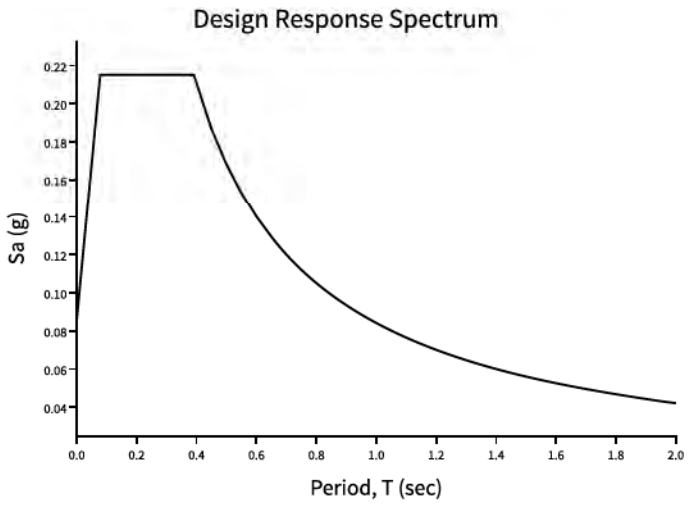
D (determined): Stiff Soil

Risk Category

I or II or III

S_S =	0.202 g	S_{MS} =	0.323 g	S_{DS} =	0.215 g
S_1 =	0.053 g	S_{M1} =	0.126 g	S_{D1} =	0.084 g


MCE$_R$ Spectrum
Sa (g)
0.30
0.25
0.20
0.15
0.10
0.05
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Period, T (sec)


Design Response Spectrum
Sa (g)
0.22
0.20
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Period, T (sec)

Mapped Acceleration Parameters, Long-Period Transition Periods, and Risk Coefficients

Note: The S_S and S_1 ground motion maps provided below are for the direction of maximmum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) 1.3 (to obtain S_1).

- FIGURE 22-1 S_S Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B
- FIGURE 22-2 S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B
- FIGURE 22-9 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %g, Site Class B for the Conterminous United States
- FIGURE 22-14 Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States
- FIGURE 22-18 Mapped Risk Coefficient at 0.2 s Spectral Response Period, C_{RS}
- FIGURE 22-19 Mapped Risk Coefficient at 1.0 s Spectral Response Period, C_{R1}

Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site class as Site Class , based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20 • Moisture content w ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		
For SI: 1ft/s = 0.3048 m/s 1lb/ft^2 = 0.0479 kN/m^2			

Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Risk-targeted Ground Motion (0.2 s)

$C_{RS}S_{SUH} = 0.942 \times 0.214 = 0.202$ g

Deterministic Ground Motion (0.2 s)

$S_{SD} = 1.500$ g

$S_S \equiv$ "Lesser of $C_{RS}S_{SUH}$ and S_{SD}" = 0.202 g

Risk-targeted Ground Motion (1.0 s)

$C_{R1}S_{1UH} = 0.924 \times 0.057 = 0.053$ g

Deterministic Ground Motion (1.0 s)

$S_{1D} = 0.600$ g

$S_1 \equiv$ "Lesser of $C_{R1}S_{1UH}$ and S_{1D}" = 0.053 g

Table 11.4-1: Site Coefficient F_a

	Spectral Reponse Acceleration Parameter at Short Period					
Site Class	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 1.50$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.3	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.2	1.1	1.0	1.0
D (default)	1.6	1.4	1.2	1.2	1.2	1.2
E	2.4	1.7	1.3	1.2 *	1.2 *	1.2 *
F	See Section 11.4.7					

* For Site Class E and $S_S \geq 1.0$ g, see the requirements for site-specific ground motions in Section 11.4.7 of the 2015 NEHRP Provisions. Here the exception to those requirements allowing F_a to be taken as equal to that of Site Class C has been invoked.

Note: Use straight-line interpolation for intermediate values of S_S.

Note: Where Site Class B is selected, but site-specific velocity measurements are not made, the value of F_a shall be taken as 1.0 per Section 11.4.2.

Note: Where Site Class D is selected as the default site class per Section 11.4.2, the value of F_a shall not be less than 1.2 per Section 11.4.3.

For Site Class = D (determined) and S_S = 0.202 g, F_a = 1.600

Table 11.4-2: Site Coefficient F_v

Site Class	Spectral Response Acceleration Parameter at 1-Second Period					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.8	0.8	0.8	0.8	0.8	0.8
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.5	1.5	1.5	1.5	1.5	1.4
D (determined)	2.4	2.2 [1]	2.0 [1]	1.9 [1]	1.8 [1]	1.7 [1]
D (default)	2.4	2.2 [1]	2.0 [1]	1.9 [1]	1.8 [1]	1.7 [1]
E	4.2	3.3 [1]	2.8 [1]	2.4 [1]	2.2 [1]	2.0 [1]
F	See Section 11.4.7					

[1] For Site Class D or E and $S_1 \geq 0.2$ g, site-specific ground motions might be required. See Section 11.4.7 of the 2015 NEHRP Provisions.

Note: Use straight-line interpolation for intermediate values of S_1.

Note: Where Site Class B is selected, but site-specific velocity measurements are not made, the value of F_v shall be taken as 1.0 per Section 11.4.2.

For Site Class = D (determined) and S_1 = 0.053 g, F_v = 2.400

Site-adjusted MCE_R (0.2 s)

$$S_{MS} = F_a S_S = 1.600 \times 0.202 = 0.323 \text{ g}$$

Site-adjusted MCE_R (1.0 s)

$$S_{M1} = F_v S_1 = 2.400 \times 0.053 = 0.126 \text{ g}$$

Design Spectral Acceleration Parameters

Design Ground Motion (0.2 s)

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.323 = 0.215 \text{ g}$$

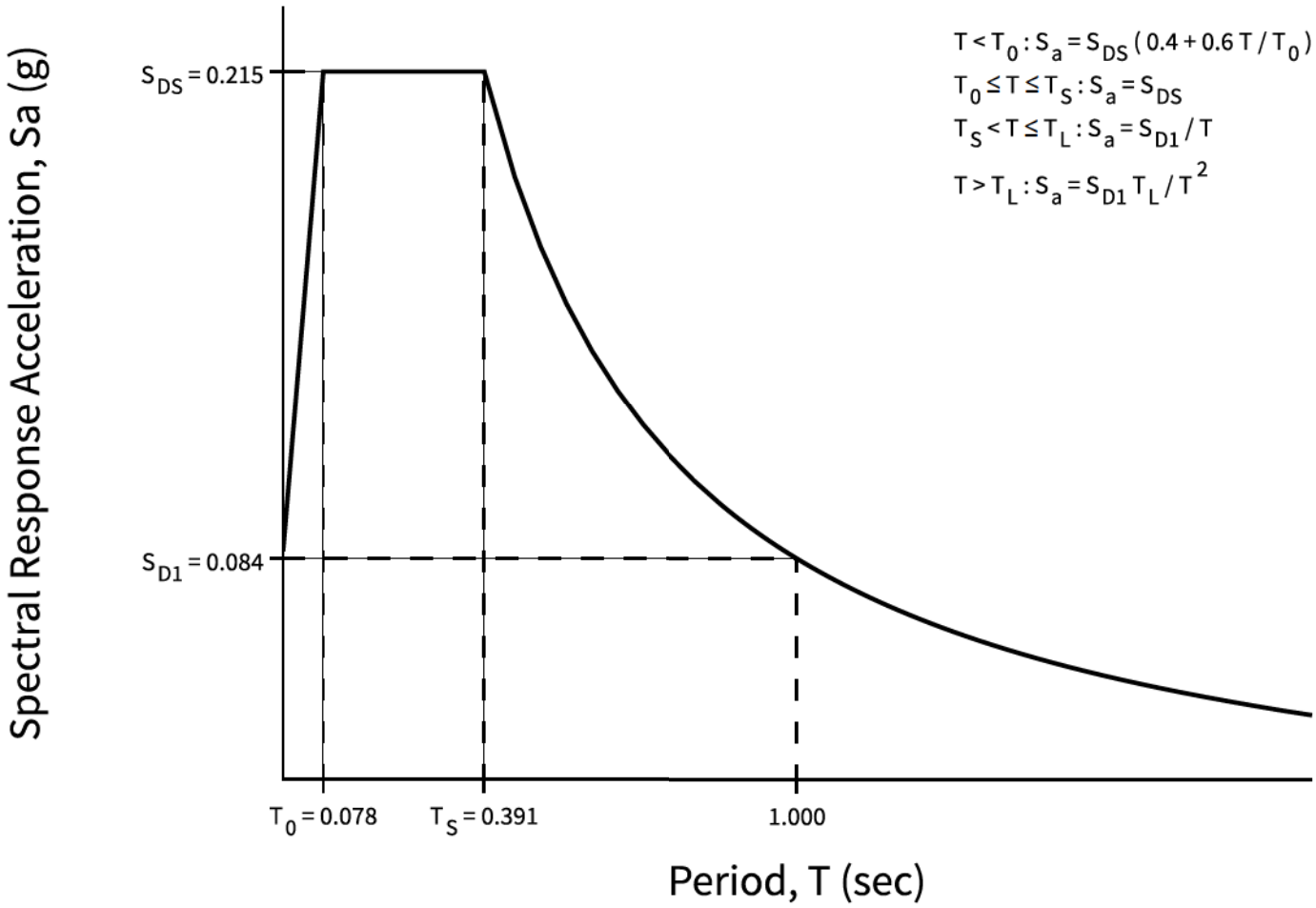
Design Ground Motion (1.0 s)

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.126 = 0.084 \text{ g}$$

Design Response Spectrum

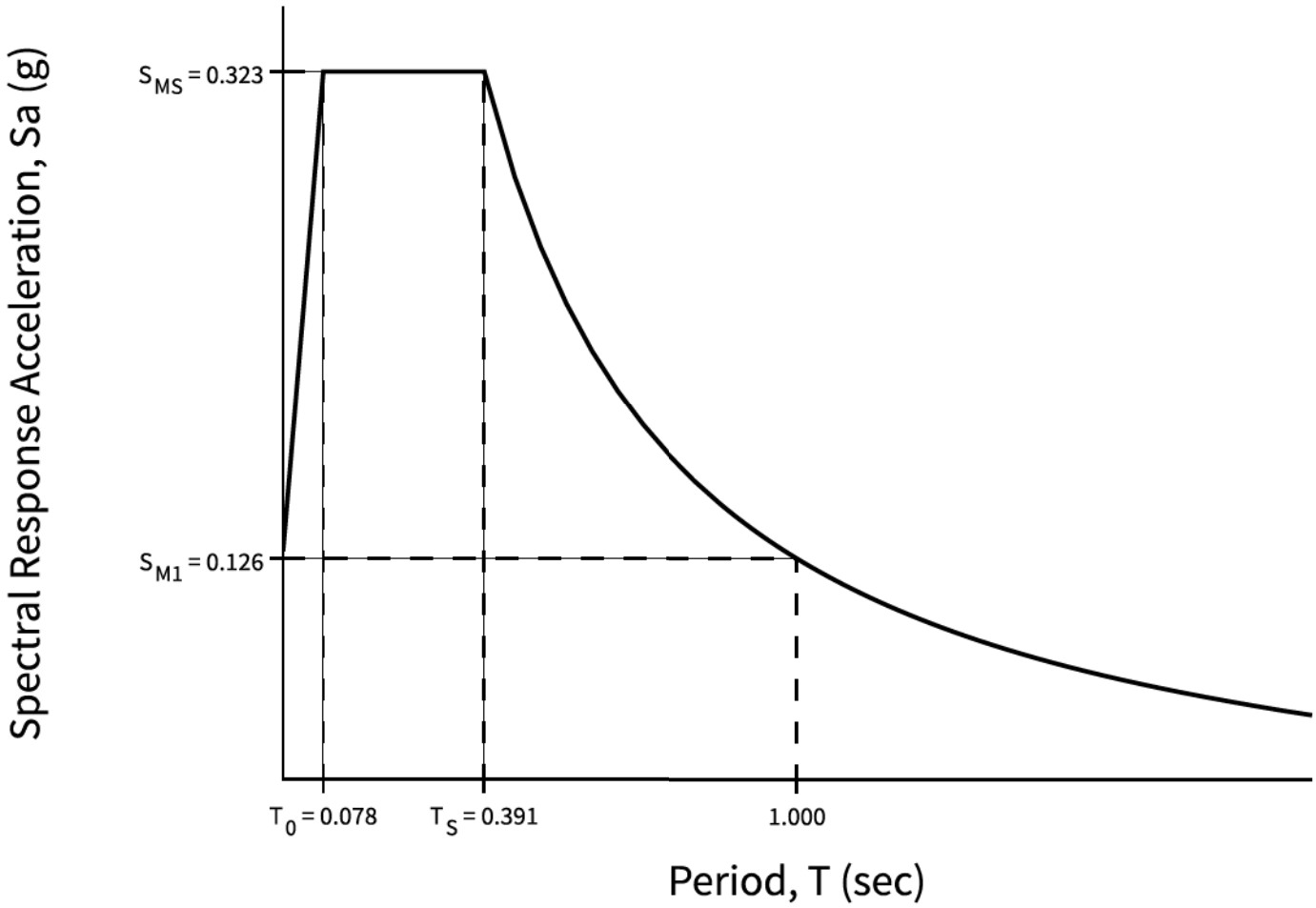
Long-Period Transition Period = T_L = 8 s

Figure 11.4-1: Design Response Spectrum


$T < T_0 : S_a = S_{DS}(0.4 + 0.6\,T/T_0)$
$T_0 \le T \le T_S : S_a = S_{DS}$
$T_S < T \le T_L : S_a = S_{D1}/T$
$T > T_L : S_a = S_{D1}T_L/T^2$
$S_{DS} = 0.215$
$S_{D1} = 0.084$
$T_0 = 0.078$
$T_S = 0.391$
1.000
Spectral Response Acceleration, Sa (g)
Period, T (sec)

MCE_R Response Spectrum

The MCE_R response spectrum is determined by multiplying the design response spectrum above by 1.5.


Spectral Response Acceleration, Sa (g)
S_{MS} = 0.323
S_{M1} = 0.126
T_0 = 0.078
T_S = 0.391
1.000
Period, T (sec)

Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

Table 11.8-1: Site Coefficient for F_{PGA}

	Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration					
Site Class	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA = 0.50	PGA ≥ 0.60
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.2	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.3	1.2	1.1	1.1
D (default)	1.6	1.4	1.3	1.2	1.2	1.2
E	2.4	1.9	1.6	1.4	1.2	1.1
F	See Section 11.4.7					

Note: Use straight-line interpolation for intermediate values of PGA

Note: Where Site Class D is selected as the default site class per Section 11.4.2, the value of F_{pga} shall not be less than 1.2.

For Site Class = D (determined) and PGA = 0.117 g, F_{PGA} = 1.567

Mapped MCE_G

PGA = 0.117 g

Site-adjusted MCE_G

$PGA_M = F_{PGA}PGA = 1.567 \times 0.117 = 0.183$ g

ATTACHMENT C

Liquefaction Assessment Package

Date:	6/10/16	**Made by:**	Grant Martin
Project No.:	1520610	**Checked by:**	Gregory Hebeler
Subject:	Liquefaction Assessment for Proposed Ash Landfill Site	**Reviewed by:**	Gregory Hebeler
Project Short Title:	**CHESTERFIELD / REYMET RD ASH LANDFILL**		

1.0 OBJECTIVE

A landfill is proposed for construction near Reymet Rd in Richmond, VA to accept ash from Dominion's Chesterfield Power Station. In this calculation package, the final closure design is evaluated for liquefaction potential.

2.0 METHODOLOGY

For liquefaction to occur during a seismic event, all three of the below conditions must exist:

- The particulate material (soil, ash, etc.) must be saturated or very near to saturated
- The material must generate positive pore pressures when stressed during rapid / undrained loading
- The seismic event must produce a strong enough motion to overcome the material's liquefaction resistance.

If any of these three factors does not exist during the seismic event, the soil will not liquefy. The liquefaction potential of a soil can be screened with respect to each factor. For example, soils or materials that are above the water table and that are not expected to become saturated in the long term (such as materials in a landfill) are not susceptible to liquefaction in the long term. The soil's response to rapid loading is primarily dependent on the soil type and soil structure. Typically, loose sands are most likely to behave contractively and thus are usually more susceptible to liquefaction. Dense sands and plastic soils such as clays are usually not susceptible to liquefaction. Specifically, Bray et al. (2004) noted that soils with a plasticity index greater than 20 are too clayey to liquefy. If a soil's stress-induced behavior is uncertain, in-situ or lab tests may be used to assess the soil's behavior and to quantify the soil's liquefaction resistance. Finally, the seismic hazard is used to define the strength of the ground motion, which can then be compared to the liquefaction resistance to determine a factor of safety against liquefaction.

In typical conditions, landfilled ash will not be susceptible to liquefaction in the long term since it will not be saturated for the long term. Foundation soils at the Chesterfield landfill site are likely to plastic to liquefy since the majority of lab tests quantified the plasticity index of these soils to be greater than 20. However, for foundation materials near or below the water table, detailed screening methods developed from the 1996 and 1998 NCEER/NSF Workshops on Liquefaction Evaluation (Youd et al 2001) are used in this

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation

calculation package to evaluate liquefaction susceptibility. These methods calculate a soil's liquefaction resistance from in-situ test data and calculate the stresses induced by a deterministic earthquake scenario. A factor of safety against liquefaction is then calculated from the ratio of the soil's liquefaction resistance to the earthquake stresses.

2.1 NCEER Liquefaction Susceptibility Screening Method

As part of Golder's 2007 geotechnical investigation (Golder 2012), boreholes were drilled in the foundation soils of the proposed landfill site. Standard Penetration Tests (SPTs) were completed at regular intervals in boreholes. The factor of safety against liquefaction was determined for materials in the upper 50 ft using the procedure discussed during the 1996 and 1998 NCEER/NSF Workshop on liquefaction evaluation (Youd et al 2001). The ratio of the cyclic stress ratio (CSR) to the cyclic resistance ratio (CRR) gives the factor of safety against liquefaction. Factors of safety (FS) greater than 1.2 are considered to not be susceptible to liquefaction during a given seismic event per the EPA CCR Rule (EPA 2015). The factor of safety against liquefaction susceptibility is calculated as

$$FS = \frac{CRR_{7.5}}{CSR} \times MSF$$

where $CRR_{7.5}$ is the cyclic resistance ratio for a Mw=7.5 earthquake, CSR is the cyclic stress ratio induced by the earthquake event, and MSF is the magnitude scaling factor to adjust CRR to magnitudes other than Mw=7.5 earthquake. The calculation of these factors is discussed in the following sections. Factors of safety for liquefaction susceptibility were completed for three most critical borehole locations (OW-4, OW-6, and OW-9).

2.2 CSR Determination

The CSR is defined as

$$CSR = \frac{\tau_{ave}}{\sigma'_v} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right)r_d$$

where a_{max} is the peak horizontal ground acceleration (PGA), g is the acceleration due to gravity, σ_v is the total vertical overburden stress, σ'_v is the effective vertical overburden stress, and r_d is a depth-dependent stress reduction factor defined as

$$r_d = 1.0 - 0.00765z \quad for\ z \leq 9.15\ m$$

$$r_d = 1.174 - 0.0267z \quad for\ 9.15\ m < z \leq 23\ m$$

$$r_d = 0.744 - 0.008z \quad for\ 23\ m < z \leq 30\ m$$

$$r_d = 0.50 \quad for\ z > 30\ m$$

where z is the depth in meters. The reference PGA for the site at the Site Class B/C boundary provided by the 2014 USGS Seismic Hazard maps is 0.117g. The USGS NEHRP seismic hazard tool was used to provide the PGA at the ground surface amplified from the reference B/C boundary (a_{max} = 0.183g) as discussed in the Seismic Hazard Evaluation document.

2.3 Cyclic Resistance Ratio (CRR) Calculation

For the current evaluation, the CRR was calculated based on SPT blow count data. The CRR for an earthquake magnitude (M) of 7.5 is given as the following:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{10 * (N_1)_{60cs} + 45^2} - \frac{1}{200}$$

where $(N_1)_{60}$ is the blow count for clean sands. This equation applies only to materials where $(N_1)_{60} < 30$. Where $(N_1)_{60} \geq 30$, soils are considered too dense to liquefy and are classified as non-liquefiable. Sands at the proposed site contain large percentages of clay, so the following formula was used to calculate equivalent clean sand blow counts for the computation of $CRR_{7.5}$:

$$(N_1)_{60cs} = \alpha + \beta (N_1)_{60}$$

$$\alpha = 0 \; for \; FC \leq 5\%$$

$$\alpha = \exp\left[1.76 - \left(\frac{190}{FC^2}\right)\right] for \; 5\% < FC < 35\%$$

$$\alpha = 5.0 \; for \; FC \geq 35\%$$

$$\beta = 1.0 \; for \; FC \leq 5\%$$

$$\beta = \left[0.99 + \left(\frac{FC^{1.5}}{1000}\right)\right] for \; 5 < FC < 35\%$$

$$\beta = 1.2 \; for \; FC \geq 35\%$$

According to lab tests (Golder 2012), the fines content in the foundation soils at the Chesterfield landfill site average 43%. Therefore, the blow counts were corrected according to the factors for soils with fines content greater than 35%.

2.4 Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) adjusts the CRR for magnitudes other than 7.5. Based on Idriss as presented in Youd et al. (2001), the MSF is calculated with the following equation:

$$MSF = \frac{10^{2.24}}{M^{2.56}}$$

where M is the moment magnitude of the earthquake being assessed.

3.0 CONCLUSIONS

As discussed in the seismic hazard evaluation document, a design earthquake scenario having a magnitude of 5.52 and peak ground acceleration of 0.183g was considered in the liquefaction analysis. For the design earthquake scenario, the foundation soils at the Chesterfield landfill site were calculated to have a factor of safety against liquefaction greater than 1.2, so these soils are calculated to not be susceptible to liquefaction. The figure below shows the factors of safety against liquefaction calculated with depth in the three boreholes for the earthquake scenario, and Table 3 summarizes the liquefaction analysis results. Detailed results of the critical earthquake scenario are presented in the attached plots.

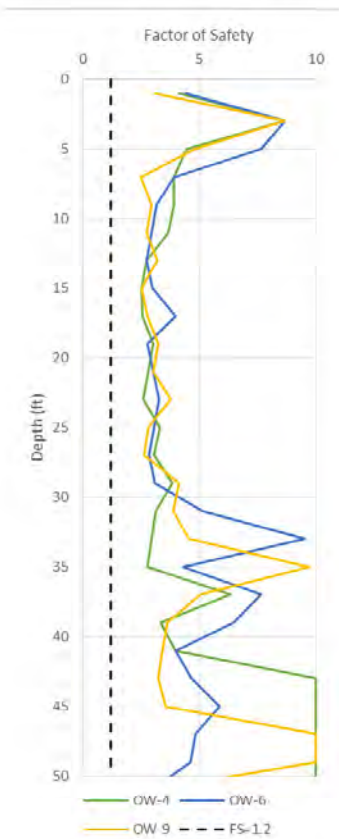


Figure 2: Factor of Safety against Liquefaction for the Critical Earthquake Scenario

Table 3 – Summary of Liquefaction Analysis Results

Borehole	Calculated Liquefaction (Factor of Safety <1.2?)
OW-4	NO LIQUEFACTION
OW-6	NO LIQUEFACTION
OW-9	NO LIQUEFACTION

4.0 ATTACHMENTS

Borehole Liquefaction Analysis Plots for OW-4, OW-6, and OW-9

5.0 REFERENCES

Atkinson, G.M. and D.M. Boore (2006) "Earthquake Ground-Motion Prediction Equations for Eastern North America," *Bulletin of the Seismological Society of America,* Vol. 96, No. 6, pp. 2181-2205.

Bray, Jonathan D., et al. "Liquefaction susceptibility of fine-grained soils." Proc., *11th Int. Conf. on Soil Dynamics and Earthquake Engineering and 3rd Int. Conf. on Earthquake Geotechnical Engineering.* Vol. 1. Stallion Press, Singapore, 2004.

Environmental Protection Agency (2015) Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities, 40 CFR Parts 257 and 261

Golder Associates Inc. (2012). Design Report - Chesterfield FFCP Facility - Permit #609

Golder Associates Inc. (2016). Seismic Hazard Evaluation Package

Olson, Scott M. and Timothy D. Stark (2003) "Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 129, No. 8, pp. 727-737.

Robertson, P.K. and C.E. (Fear) Wride (1998) "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test," *Canadian Geotechnical Journal*, Vol. 35, pp. 442-459.

Robertson, P.K. (2010) "Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 136, No. 6, pp. 842-853.

US Environmental Protection Agency (2015), "Final Rule: Disposal of Coal Combustion Residuals from Electric Utilities," 40 CFR Parts 257 and 261. Federal Register Vol. 80, No. 74

Youd, T.L., I.M. Idriss, Ronald D. Andrus, Ignacio Arango, Gonzalo Castro, John T. Christian, Ricardo Dobry, W.D. Liam Finn, Leslie F. Harder Jr., Mary Ellen Hynes, Kenji Ishihara, Joseph P. Koester, Sam S.C. Liao, William F. Marcuson III, Geoffrey R. Martin, James K. Mitchell, Yoshiharu Moriwaki, Maurice S. Power, Peter K. Robertson, Raymond B. Seed, Kenneth H. Stokoe II (2001) "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, 127(10), pp. 817-833.

2008 USGS National Seismic Hazard Map;
http://earthquake.usgs.gov/hazards/products/conterminous/2008/maps/

ATTACHMENT D

Supplemental Slope Stability Package

Date:	June 2016	**Made by:**	G. Martin
Project No.:	1520610	**Checked by:**	G. Hebeler
Subject:	Supplemental Slope Stability Analysis	**Reviewed by:**	G. Hebeler
Short Title:	Chesterfield Reymet Road Landfill, Stability Analyses		

1.0 OBJECTIVE

Slope stability analyses were initially conducted for critical slopes of the proposed Chesterfield Reymet Road Ash Landfill in Golder's 2010 report (Golder 2010). Slope stability analyses are to be update to included newly added liner systems and seismic stability analyses.

2.0 METHODOLOGY

Dike geometry for the model was developed based on the proposed design grades, topography, and geotechnical site exploration results presented in Golder's 2012 design report.

Stability analyses were completed using the computer program SLIDE 7.0 Version 7.016 (2016). SLIDE computes potential failure surfaces using a general limit equilibrium (GLE) method developed by Morgenstern and Price (Abramson et al., 2002). The method is based on the principle of limit equilibrium (i.e., the method calculates the shear strengths that would be required to maintain equilibrium and then calculates a factor of safety by dividing the available shear strength by the shear strength required to maintain stability). An auto-refine, circular failure surface search-method was primarily used in this study and was supplemented with block failure search methods. For these iterations, safety factors in excess of 1.0 indicate stability, and those less than 1.0 indicate a potential for instability. Shallow surfaces (< 5 ft) were not considered in the global stability analysis (see Veneer Stability Calculation Package (Golder 2016)).

2.1 Material Properties

Golder completed a material properties package with their 2012 report. Material properties selected in the report were supplemented to include total (undrained) strength properties for all materials and liner system properties in general. The table below summarizes the material properties selected for use in stability analyses. Details of material property selection are included in the Material Property Calculation Package.

Table 1: Summary of Geotechnical Strength Properties

Summary of Geotechnical Strength Properties					
Material	***Total Unit Weight (pcf)***	***Drained (Effective)***		***Undrained (Total)***	
		ϕ' (°)	**c' (psf)**	**ϕ (°)**	**c or [S_u/σ'_v] (psf)**
Stratum #1 (Silty Clayey Sand)	115	31	130	23	170
Stratum #2 (Clayey Silt to Sandy Clay)	112	28	200	N/A	[0.51]
Stratum #3 (Silty Clay)	120	29	200	N/A	[0.42]
Stratum #4 (Clayey Sand & Gravel)	125	32	100	N/A	
Ash Fill	100	31	40	N/A	
Base Liner System	120	24	90	N/A	
Cap Liner System	120	25	90	N/A	

2.2 Stability Cases

Critical slopes along two (2) sections of the proposed landfill were evaluated for two (2) scenarios:

(A) Long Term (Drained) Steady State Conditions

(B) Seismic Loading Conditions

For long-term steady state conditions, effective strength material properties were used. For seismic loading conditions, total strength material properties were used. Section locations are shown on Figure 1.

2.3 Seismic Analysis

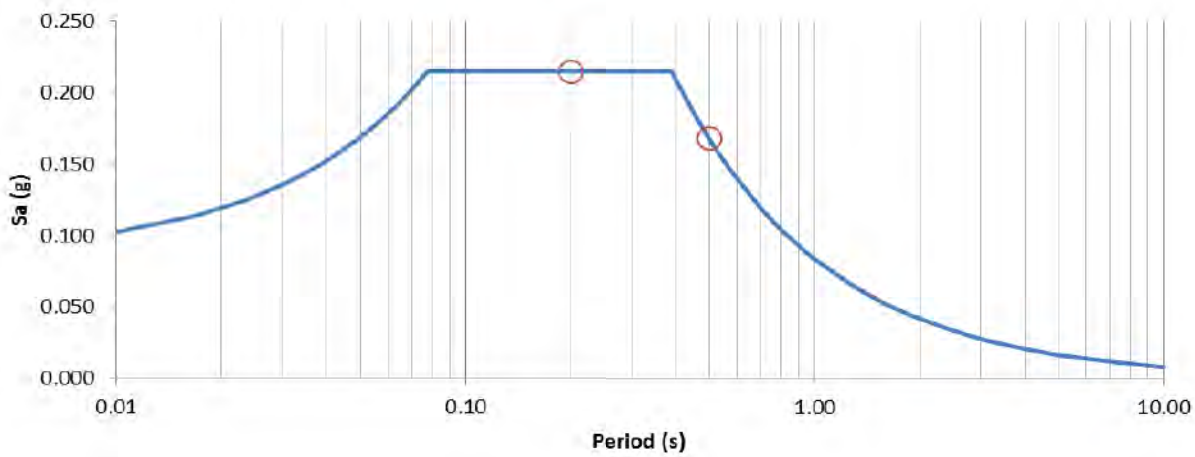
Stability under seismic conditions is evaluated using the pseudo-static screening method presented by Bray and Travasarou (2009). This method uses a seismic coefficient to model horizontal seismic forces as the product of the seismic coefficient (k) and the weight of the sliding mass (vertical seismic forces are typically neglected). A pseudostatic slope stability analysis is then completed with the seismic coefficient. If the resultant factor of safety is greater than 1, the slope is not predicted to displace greater than the specified allowable magnitude.

Bray and Travasarou (2009) present equations to calculate the seismic coefficient for an allowable displacement of six inches (15 cm) at two periods (0.2 s and 0.5 s). The equation representing a period close to that of the slope being evaluated is used. Since the period of the slope is not know for this project, the seismic coefficient was calculated using both equations, and the more critical of the two was used in analyses. The equations for both periods are shown below:

$$k_{15\ cm} = (0.036M - 0.004)Sa\ - 0.030 > 0\ for\ Sa = Sa(T = 0.2\ s) < 2.0g$$

$$k_{15\ cm} = (0.038M - 0.006)Sa\ - 0.026 > 0\ for\ Sa = Sa(T = 0.5\ s) < 1.5g$$

The spectral acceleration (Sa) for both periods was taken from the 2015 NEHRP provisional method as provided in a beta tool format on the USGS website. The corresponding spectral accelerations are 0.22g and 0.17g for periods of 0.2s and 0.5s, respectively. The figure below shows the selection of these values on the NEHRP spectral acceleration curve. A mean magnitude (M) of 5.71 was selected as provided in the USGS 2008 deaggregation tool.



Thus, the seismic coefficients corresponding to periods of 0.2s and 0.5s for the Bray and Travasarou dispalcement based evaluation method were calculated as 0.013 and 0.012, respectively. Since the coefficient corresponding to a period of 0.2s is more critical, a seismic coefficient of 0.013 was selected for use in seismic stability analyses.

Date:	June 2016	**Made by:**	G. Martin
Project No.:	1520610	**Checked by:**	G. Hebeler
Subject:	Supplemental Slope Stability Analysis	**Reviewed by:**	G. Hebeler
Short Title:	Chesterfield Reymet Road Landfill, Stability Analyses		

3.0 RESULTS

Table 2 shows the results of each analysis completed. All sections meet target factors of safety for both long-term steady state conditions and conditions for the design seismic hazard.

Table 2: Summary of Geotechnical Stability Analyses

		A	B	C	D
	Analysis Type	Steady-State	Steady-State	Seismic	Seismic
	Slip Surface Type	Circular	Block	Circular	Block
	Target FS	1.5	1.5	1.0	1.0
Figure	**Section**	**Factors of Safety**			
2	A-A, West	2.2 / 1.9	2.1	2.0	2.0
3	A-A, East	2.6	2.6	1.4	2.6
4	B-B, South	1.9	1.8	1.8	1.7
5	B-B, North	1.8	1.8	1.7	1.7

Note - the localized surface at the toe in Section A-A West observed in the static search (resulting in a FS of 1.9) was not observed in the seismic surface search.

4.0 Conclusion

All analyzed sections show calculated factors of safety well above the target factors of safety for both long-term steady state conditions and the design seismic hazard conditions.

5.0 References

Abramson, L.W., T.S. Lee, S. Sharma, and G.M. Boyce (2002), Slope Stability and Stabilization Methods, 2nd edition, John Wiley & Sons, New York.

Bray, J.D., and Travasarou, T. (2009), Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 9: pp. 1336-1340.

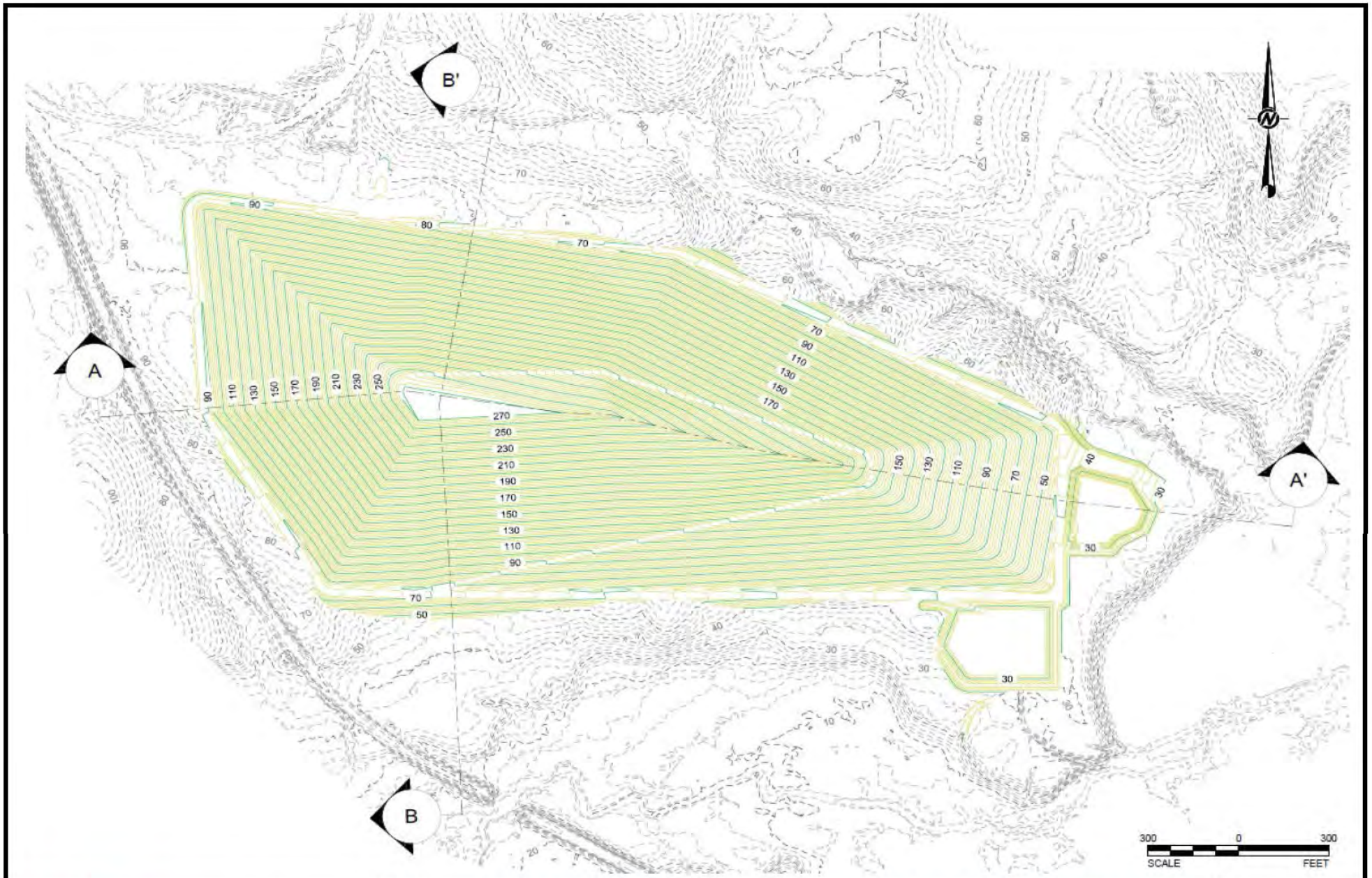
Golder Associates (2012). Design Report - Chesterfield FFCP Facility - Permit #609

Golder Associates (2016). Material Properties Package

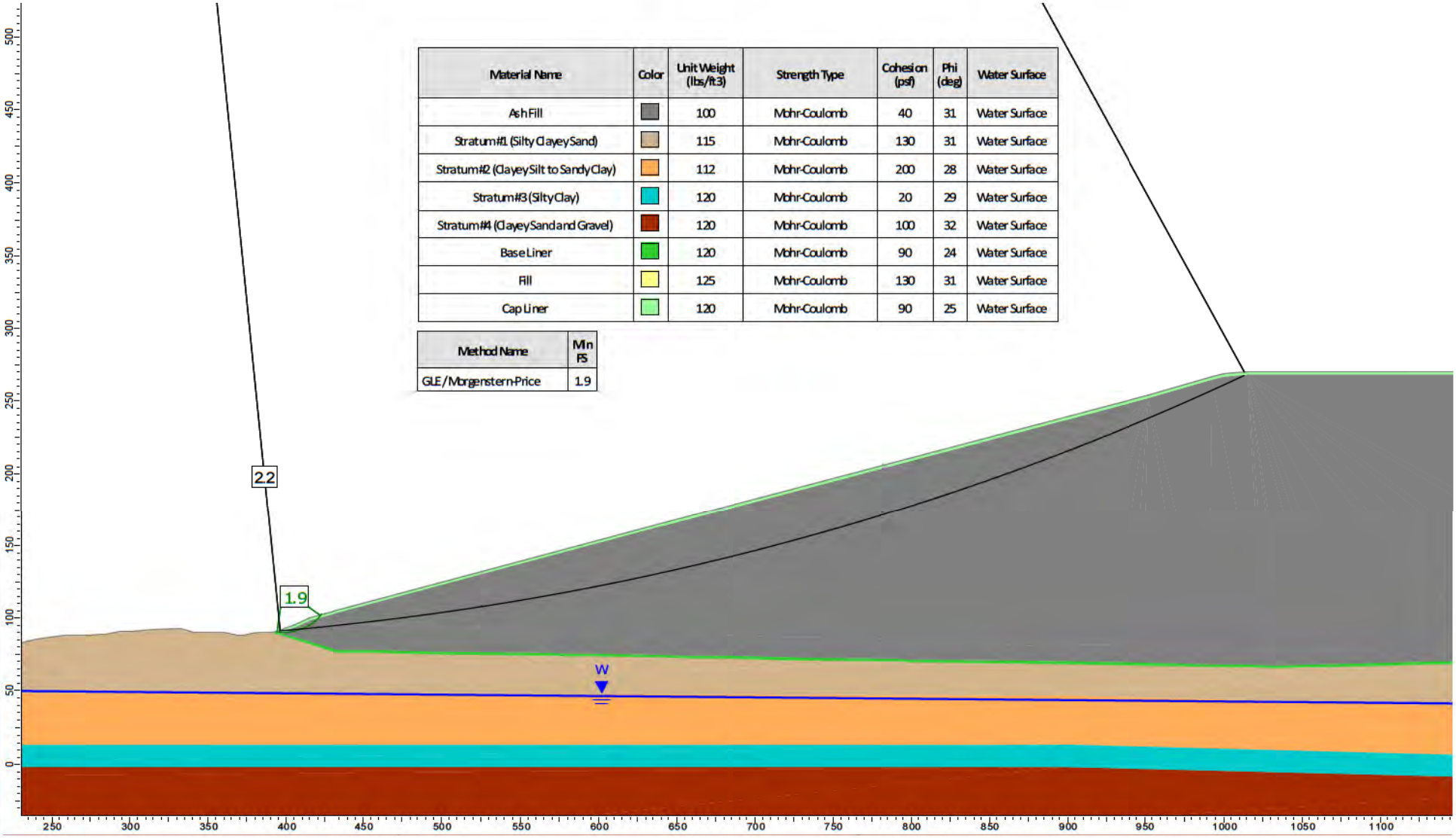
Golder Associates (2016). Seismic Hazard Evaluation Package

Golder Associates (2016). Veneer Stability Calculation Package

Rocscience (2016), SLIDE Version 7.017.



Golder Associates	SCALE: AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill	
	DATE: June 2016	TITLE: Section Location Plan	
	MADE BY: JGM		
	CAD: SEP		
FILE: STABILITY	CHECK: GLH	CLIENT: Dominion	FIGURE: 1
PROJECT No. 1520610 REV. 0	REVIEW: GLH		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31	Water Surface
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31	Water Surface
Stratum #2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28	Water Surface
Stratum #3 (Silty Clay)		120	Mohr-Coulomb	20	29	Water Surface
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32	Water Surface
Base Liner		120	Mohr-Coulomb	90	24	Water Surface
Fill		125	Mohr-Coulomb	130	31	Water Surface
Cap Liner		120	Mohr-Coulomb	90	25	Water Surface

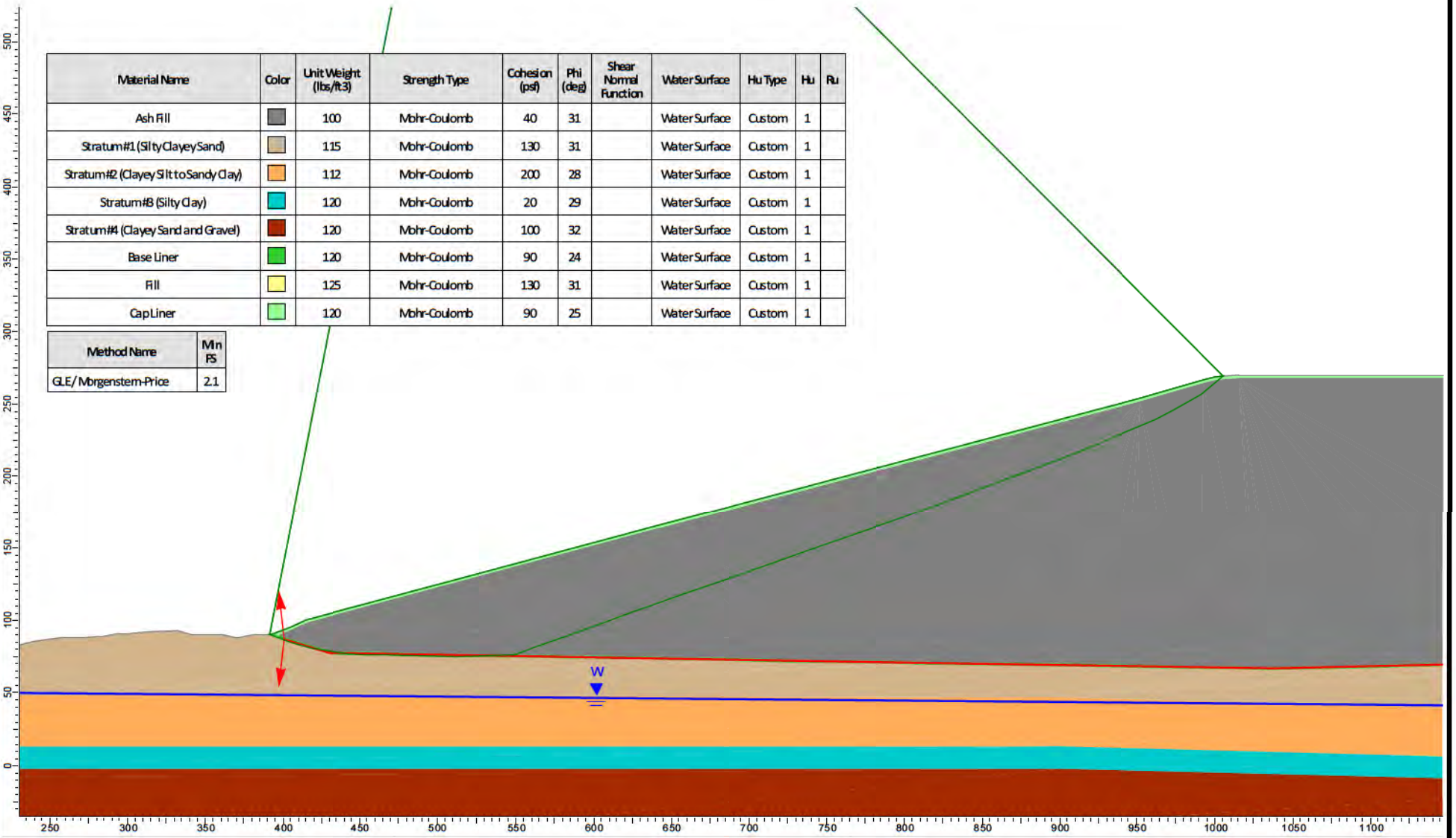
Method Name	Min FS
GLE/Morgenstern-Price	1.9

Golder Associates

SCALE	AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
DATE	June 2016	TITLE	Section A-A, West Long-Term (Effective) Stress, Steady State Analysis Circular Slip Surface Search
MADE BY	JGM		
CAD	-		
FILE	STABILITY	CHECK	GLH
PROJECT No. 1520610	REV. 0	REVIEW	GLH
CLIENT	Dominion	FIGURE	2A

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface	Hu Type	Hu	Ru
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface	Custom	1	
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface	Custom	1	
Stratum #2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface	Custom	1	
Stratum #3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface	Custom	1	
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface	Custom	1	
Base Liner		120	Mohr-Coulomb	90	24		Water Surface	Custom	1	
Fill		125	Mohr-Coulomb	130	31		Water Surface	Custom	1	
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface	Custom	1	

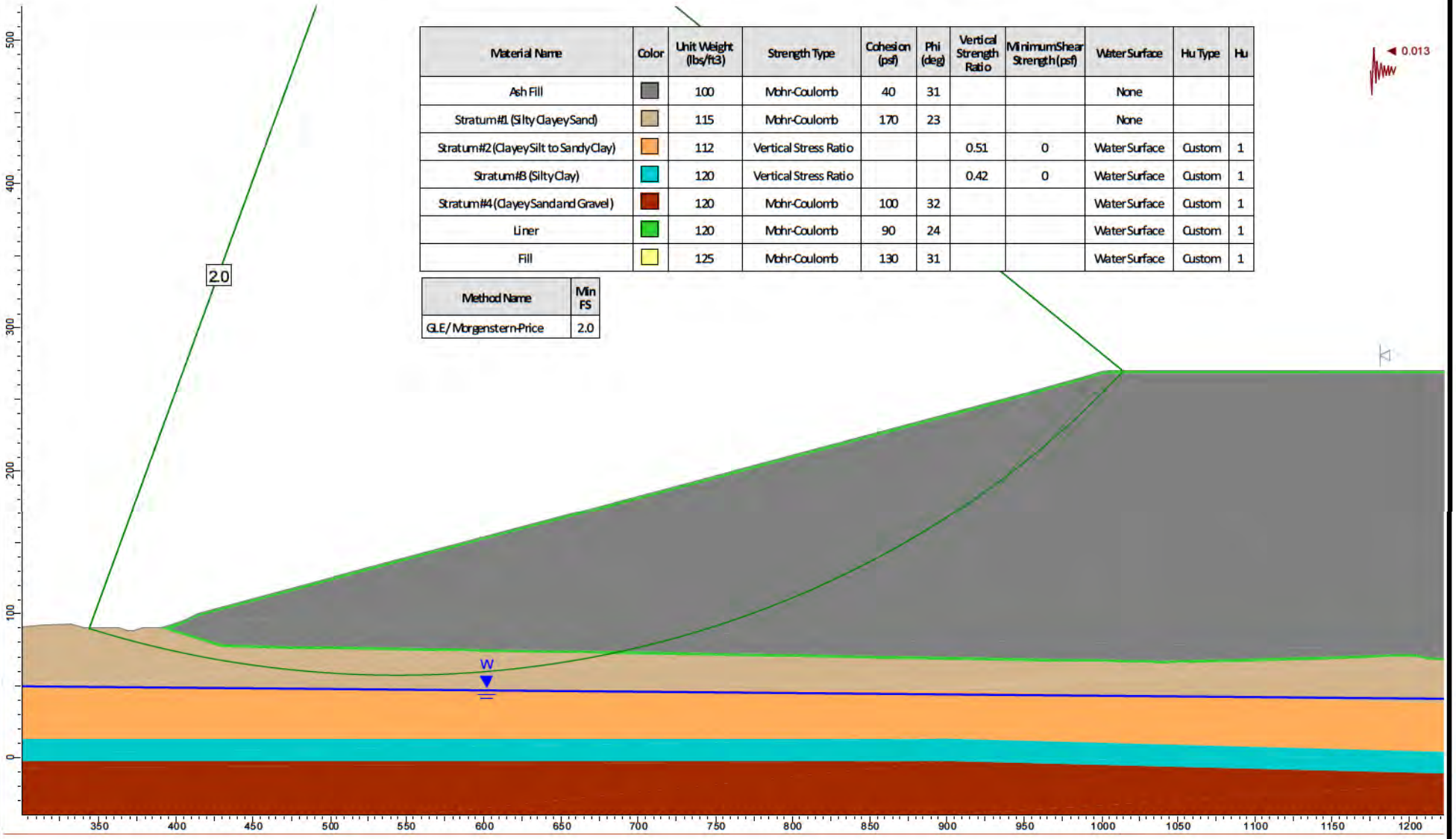
Method Name	Min FS
GLE/Morgenstern-Price	2.1



Golder Associates	SCALE	AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill
	DATE	June 2016	TITLE: Section A-A, West Long-Term (Effective) Stress, Steady State Analysis Non-Circular Slip Surface Search
	MADE BY	JGM	
	CAD	-	
FILE: STABILITY	CHECK	GLH	CLIENT: Dominion
PROJECT No. 1520610 REV. 0	REVIEW	GLH	FIGURE 2B

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Ash Fill		100	Mohr-Coulomb	40	31			None		
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23			None		
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio			0.51	0	Water Surface	Custom	1
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio			0.42	0	Water Surface	Custom	1
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32			Water Surface	Custom	1
Liner		120	Mohr-Coulomb	90	24			Water Surface	Custom	1
Fill		125	Mohr-Coulomb	130	31			Water Surface	Custom	1

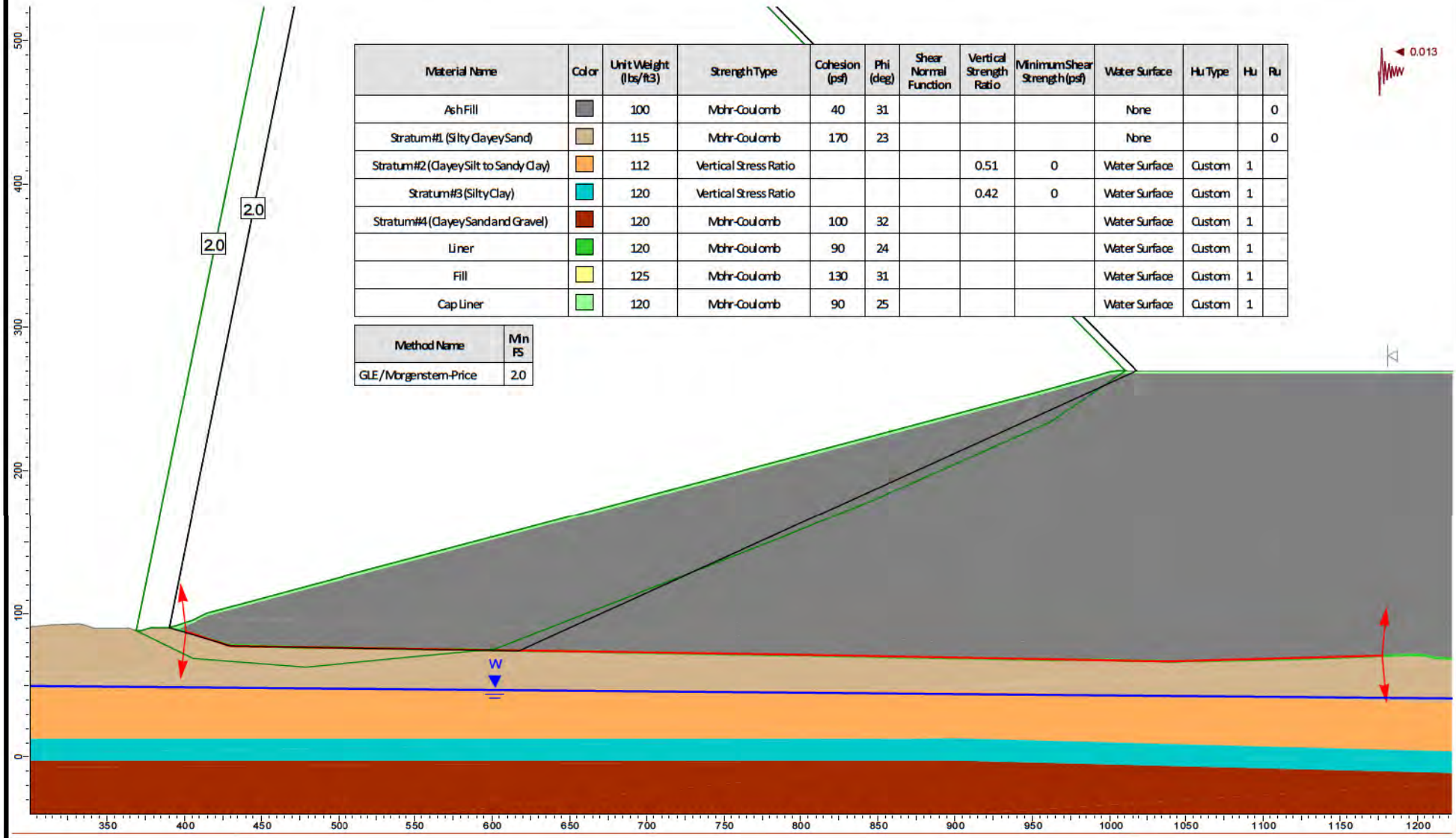
Method Name	Min FS
GLE/Morgenstern-Price	2.0



Golder Associates	SCALE AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
	DATE June 2016	TITLE	Section A-A, West Seismic Loading Conditions Circular Slip Surface Search
	MADE BY JGM		
	CAD -		
FILE STABILITY	CHECK GLH	CLIENT	Dominion
PROJECT No. 1520610 REV. 0	REVIEW GLH		FIGURE 2C

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu	Ru
Ash Fill		100	Mohr-Coulomb	40	31				None			0
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23				None			0
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio				0.51	0	Water Surface	Custom	1	
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio				0.42	0	Water Surface	Custom	1	
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32				Water Surface	Custom	1	
Liner		120	Mohr-Coulomb	90	24				Water Surface	Custom	1	
Fill		125	Mohr-Coulomb	130	31				Water Surface	Custom	1	
Cap Liner		120	Mohr-Coulomb	90	25				Water Surface	Custom	1	

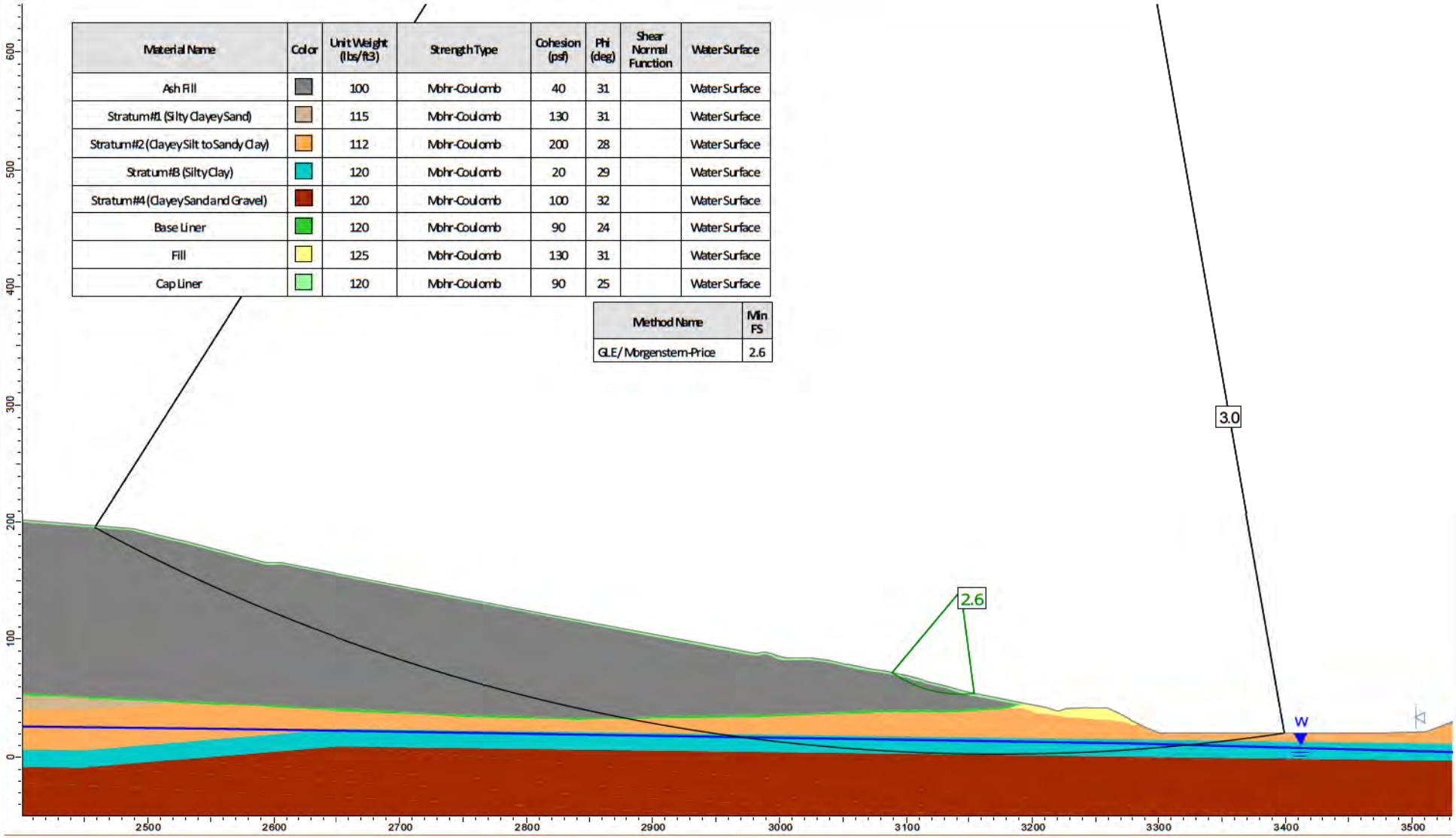
Method Name	Min FS
GLE/Morgenstern-Price	2.0



Golder Associates	SCALE AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
	DATE June 2016	TITLE	Section A-A, West Seismic Loading Conditions Non-Circular Slip Surface Search
	MADE BY JGM		
	CAD -		
FILE STABILITY	CHECK GLH	CLIENT	Dominion
PROJECT No. 1520610 REV. 0	REVIEW GLH		FIGURE 2D

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface
Stratum #2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface
Stratum #3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface
Base Liner		120	Mohr-Coulomb	90	24		Water Surface
Fill		125	Mohr-Coulomb	130	31		Water Surface
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface

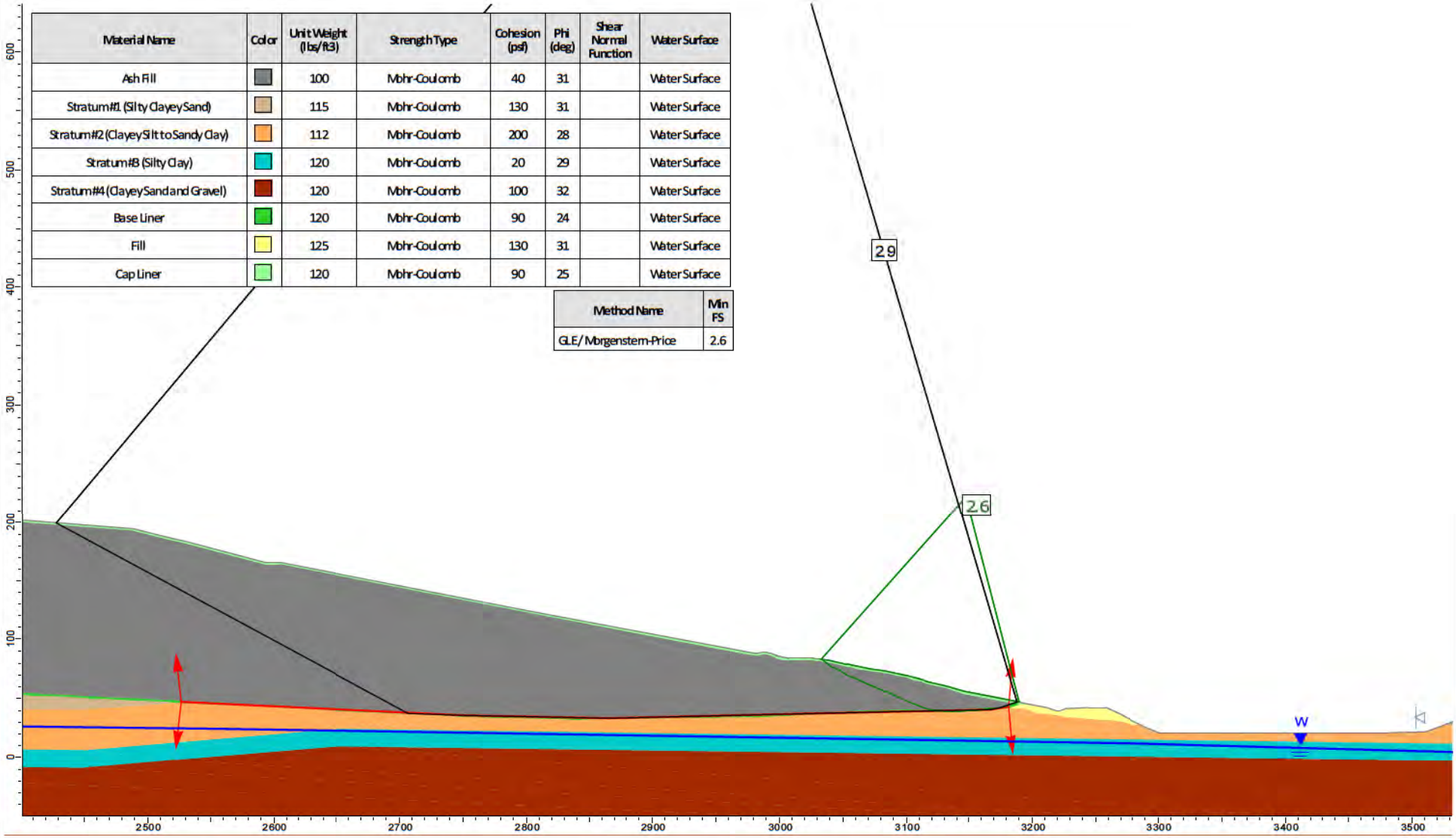
Method Name	Min FS
GLE/ Morgenstern-Price	2.6



Golder Associates	SCALE: AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill	
	DATE: June 2016	TITLE: Section A-A, East Long-Term (Effective) Stress, Steady State Analysis Circular Slip Surface Search	
	MADE BY: JGM		
	CAD: -		
FILE: STABILITY	CHECK: GLH	CLIENT: Dominion	FIGURE: 3A
PROJECT No. 1520610 REV. 0	REVIEW: GLH		

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface
Stratum #2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface
Stratum #3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface
Base Liner		120	Mohr-Coulomb	90	24		Water Surface
Fill		125	Mohr-Coulomb	130	31		Water Surface
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface

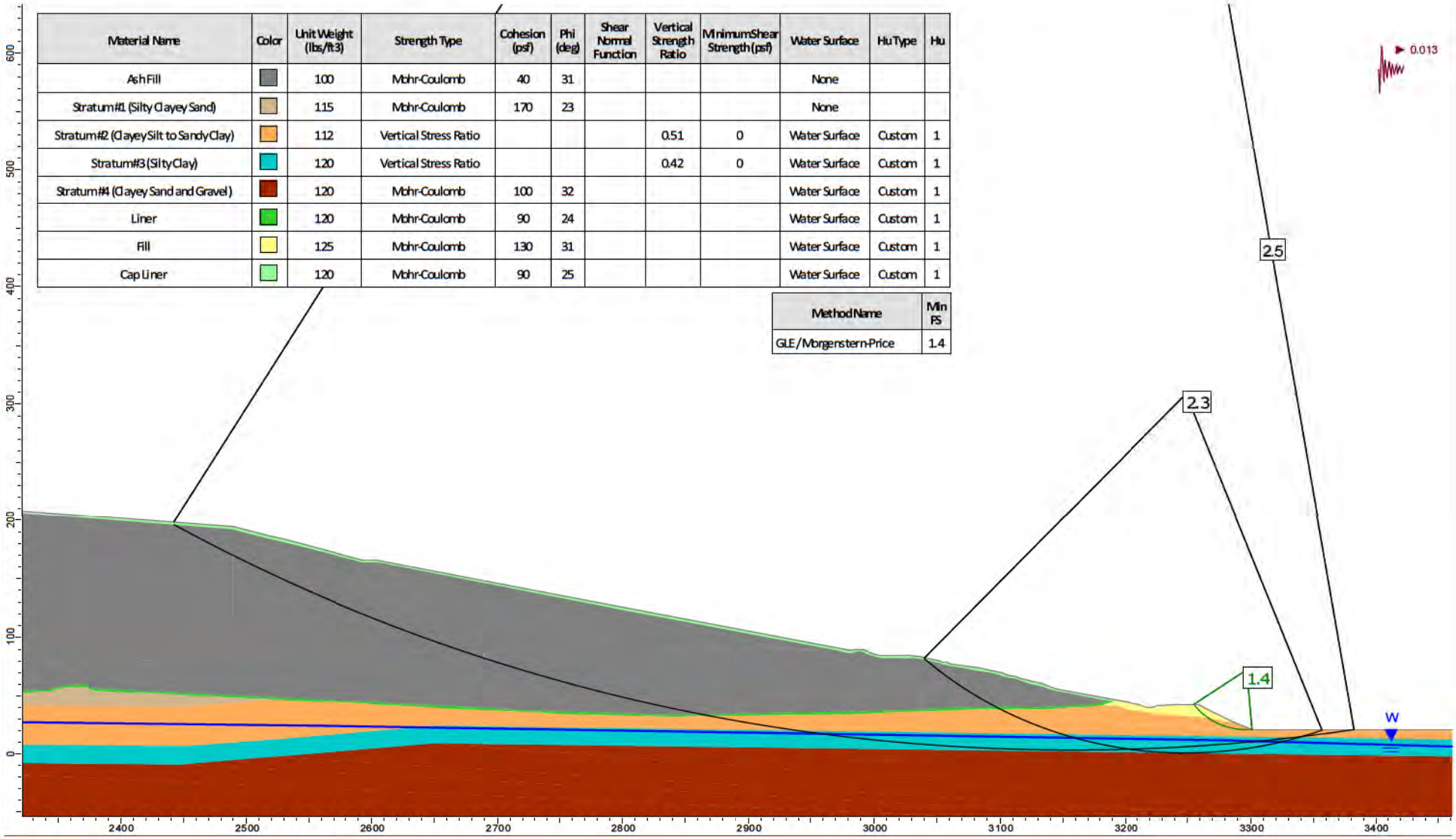
Method Name	Min FS
GLE/ Morgenstern-Price	2.6



Golder Associates	SCALE	AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill
	DATE	June 2016	TITLE: Section A-A, East Long-Term (Effective) Stress, Steady State Analysis Non-Circular Slip Surface Search
	MADE BY	JGM	
	CAD	-	
FILE: STABILITY	CHECK	GLH	CLIENT: Dominion
PROJECT No. 1520610 REV. 0	REVIEW	GLH	FIGURE 3B

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Ash Fill		100	Mohr-Coulomb	40	31				None		
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23				None		
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio				0.51	0	Water Surface	Custom	1
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio				0.42	0	Water Surface	Custom	1
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32				Water Surface	Custom	1
Liner		120	Mohr-Coulomb	90	24				Water Surface	Custom	1
Fill		125	Mohr-Coulomb	130	31				Water Surface	Custom	1
Cap Liner		120	Mohr-Coulomb	90	25				Water Surface	Custom	1

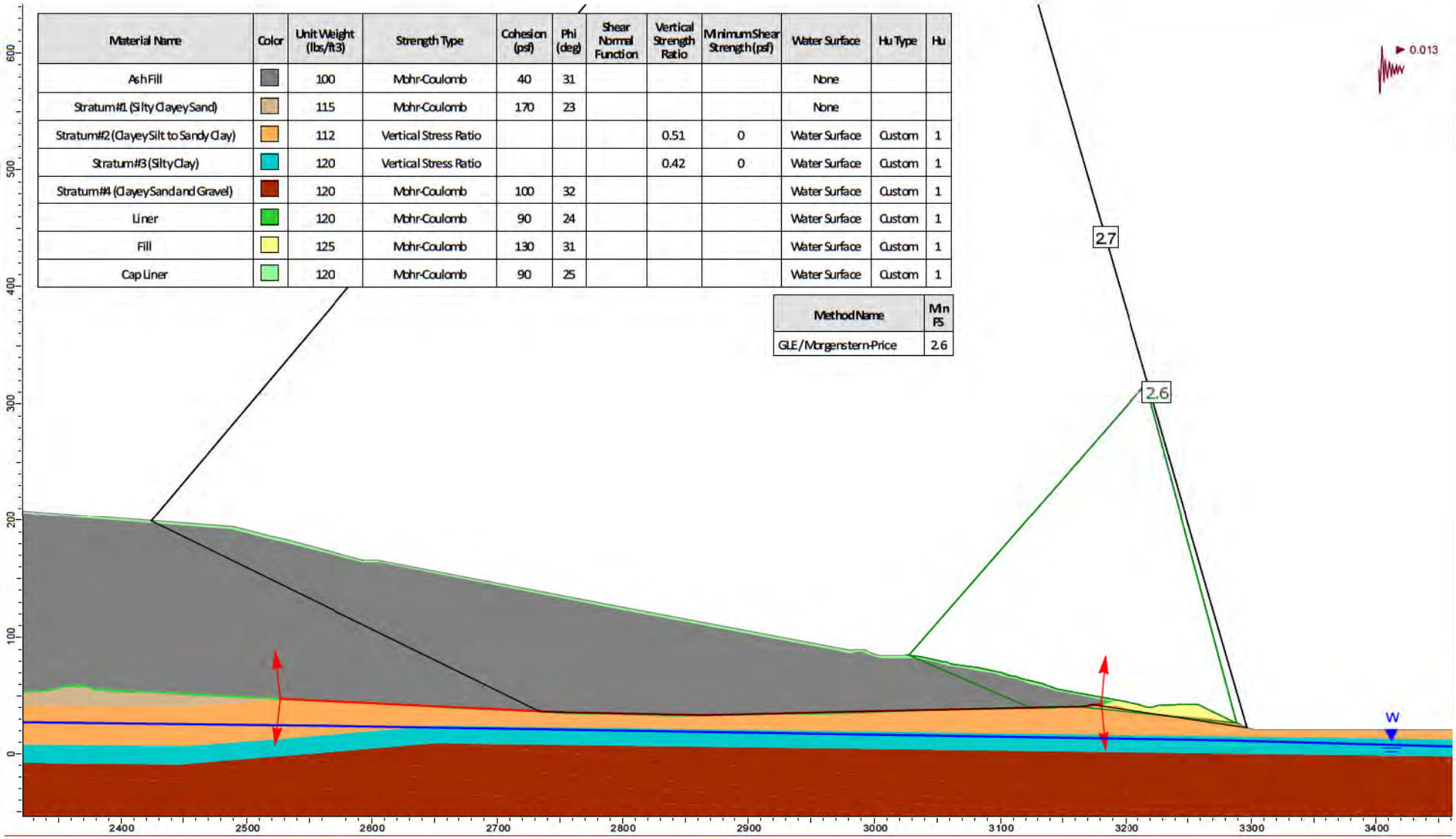
Method Name	Min FS
GLE/Morgenstern-Price	1.4



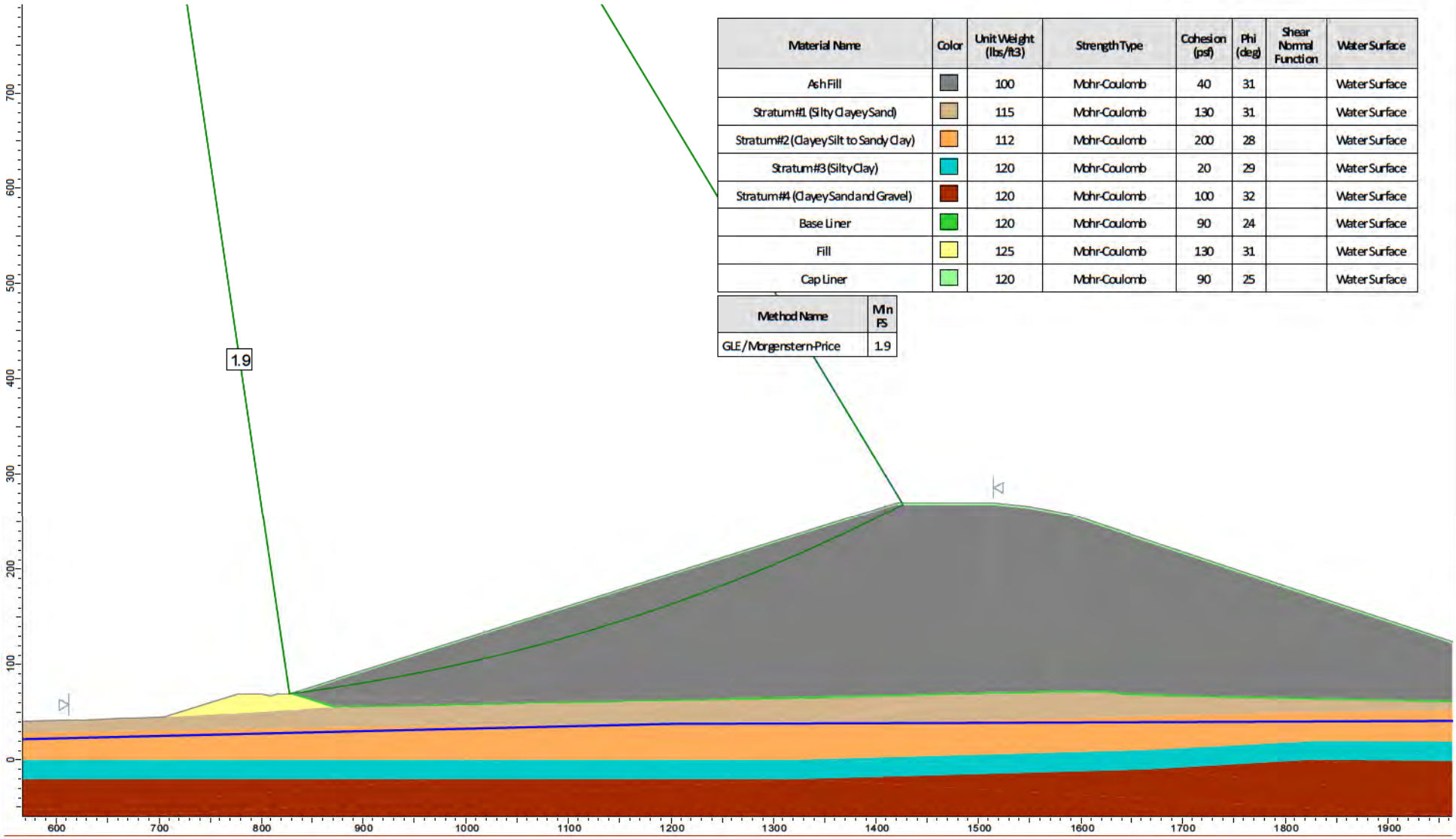
Golder Associates	SCALE	AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill
	DATE	June 2016	TITLE: Section A-A, East Seismic Loading Conditions Circular Slip Surface Search
	MADE BY	JGM	
	CAD	-	
FILE: STABILITY	CHECK	GLH	CLIENT: Dominion
PROJECT No. 1520610 REV. 0	REVIEW	GLH	FIGURE 3C

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Ash Fill		100	Mohr-Coulomb	40	31				None		
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23				None		
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio				0.51	0	Water Surface	Custom	1
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio				0.42	0	Water Surface	Custom	1
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32				Water Surface	Custom	1
Liner		120	Mohr-Coulomb	90	24				Water Surface	Custom	1
Fill		125	Mohr-Coulomb	130	31				Water Surface	Custom	1
Cap Liner		120	Mohr-Coulomb	90	25				Water Surface	Custom	1

Method Name	Min FS
GLE/Morgenstern-Price	2.6



Golder Associates	SCALE AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
	DATE June 2016	TITLE	Section A-A, East Seismic Loading Conditions Non-Circular Slip Surface Search
	MADE BY JGM		
	CAD -		
FILE STABILITY	CHECK GLH	CLIENT	Dominion
PROJECT No. 1520610 REV. 0	REVIEW GLH		FIGURE 3D

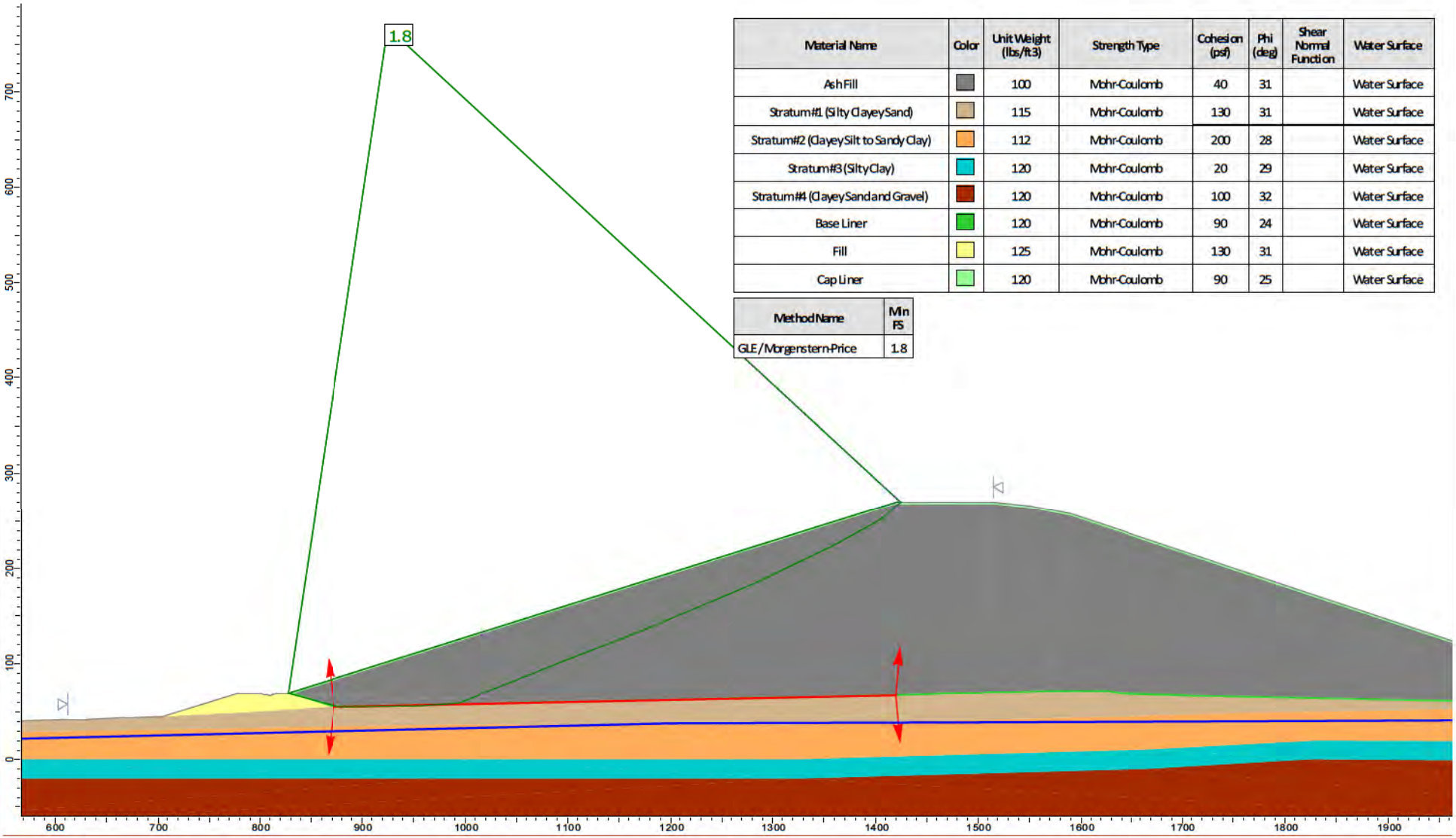


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface
Stratum #2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface
Stratum #3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface
Base Liner		120	Mohr-Coulomb	90	24		Water Surface
Fill		125	Mohr-Coulomb	130	31		Water Surface
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface

Method Name	Min FS
GLE/Morgenstern-Price	1.9

Golder Associates

SCALE	AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
DATE	June 2016	TITLE	Section B-B, South Long-Term (Effective) Stress, Steady State Analysis Circular Slip Surface Search
MADE BY	JGM		
CAD	-		
FILE	STABILITY	CHECK	GLH
PROJECT No. 1520610	REV. 0	REVIEW	GLH
CLIENT	Dominion	FIGURE	4A



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface
Stratum #2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface
Stratum #3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface
Base Liner		120	Mohr-Coulomb	90	24		Water Surface
Fill		125	Mohr-Coulomb	130	31		Water Surface
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface

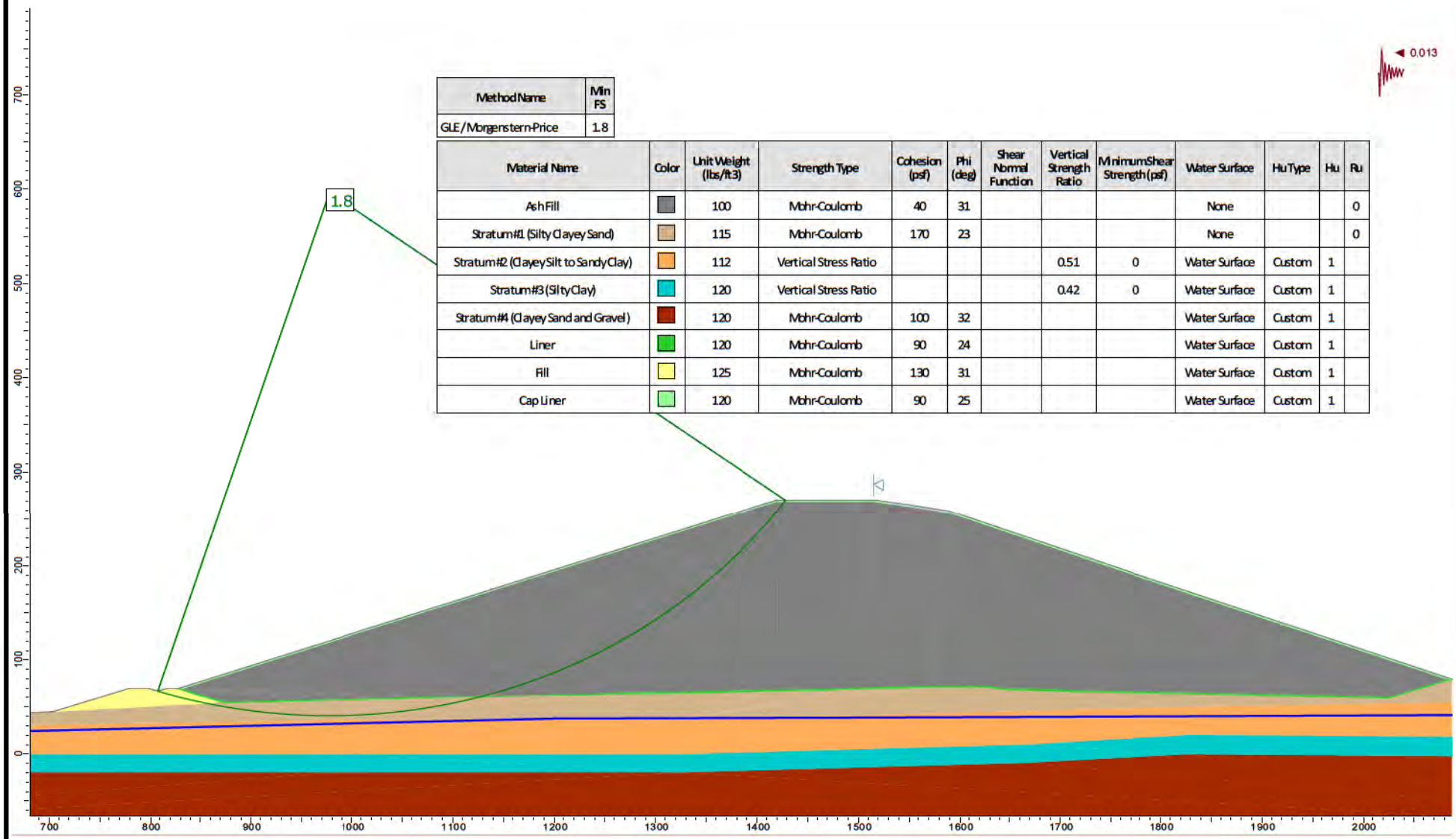
Method Name	Min FS
GLE/Morgenstern-Price	1.8

Golder Associates

SCALE	AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
DATE	June 2016	TITLE	Section B-B, South Long-Term (Effective) Stress, Steady State Analysis Non-Circular Slip Surface Search
MADE BY	JGM		
CAD	-		
FILE	STABILITY	CLIENT	Dominion
CHECK	GLH		
PROJECT No. 1520610	REV. 0	FIGURE	4B
REVIEW	GLH		

Method Name	Min FS
GLE/Morgenstern-Price	1.8

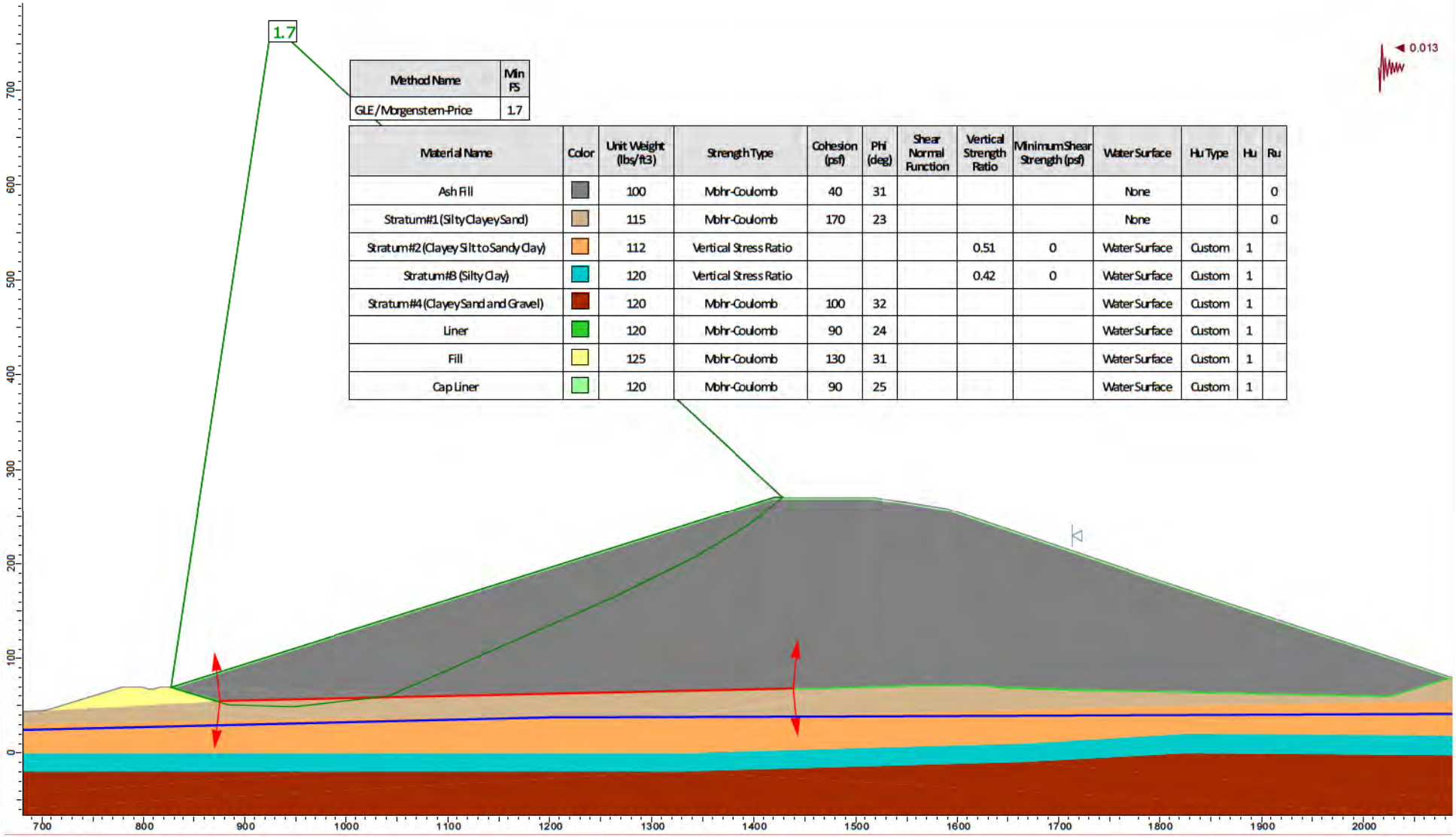
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu	Ru
Ash Fill		100	Mohr-Coulomb	40	31				None			0
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23				None			0
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio				0.51	0	Water Surface	Custom	1	
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio				0.42	0	Water Surface	Custom	1	
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32				Water Surface	Custom	1	
Liner		120	Mohr-Coulomb	90	24				Water Surface	Custom	1	
Fill		125	Mohr-Coulomb	130	31				Water Surface	Custom	1	
Cap Liner		120	Mohr-Coulomb	90	25				Water Surface	Custom	1	



Golder Associates	SCALE AS SHOWN	PROJECT	Chesterfield / Reymet Road Landfill
	DATE June 2016	TITLE	Section B-B, South Seismic Loading Conditions Circular Slip Surface Search
	MADE BY JGM		
	CAD -		
FILE STABILITY	CHECK GLH	CLIENT	Dominion
PROJECT No. 1520610 REV. 0	REVIEW GLH		FIGURE 4C

Method Name	Min FS
GLE/Morgenstern-Price	1.7

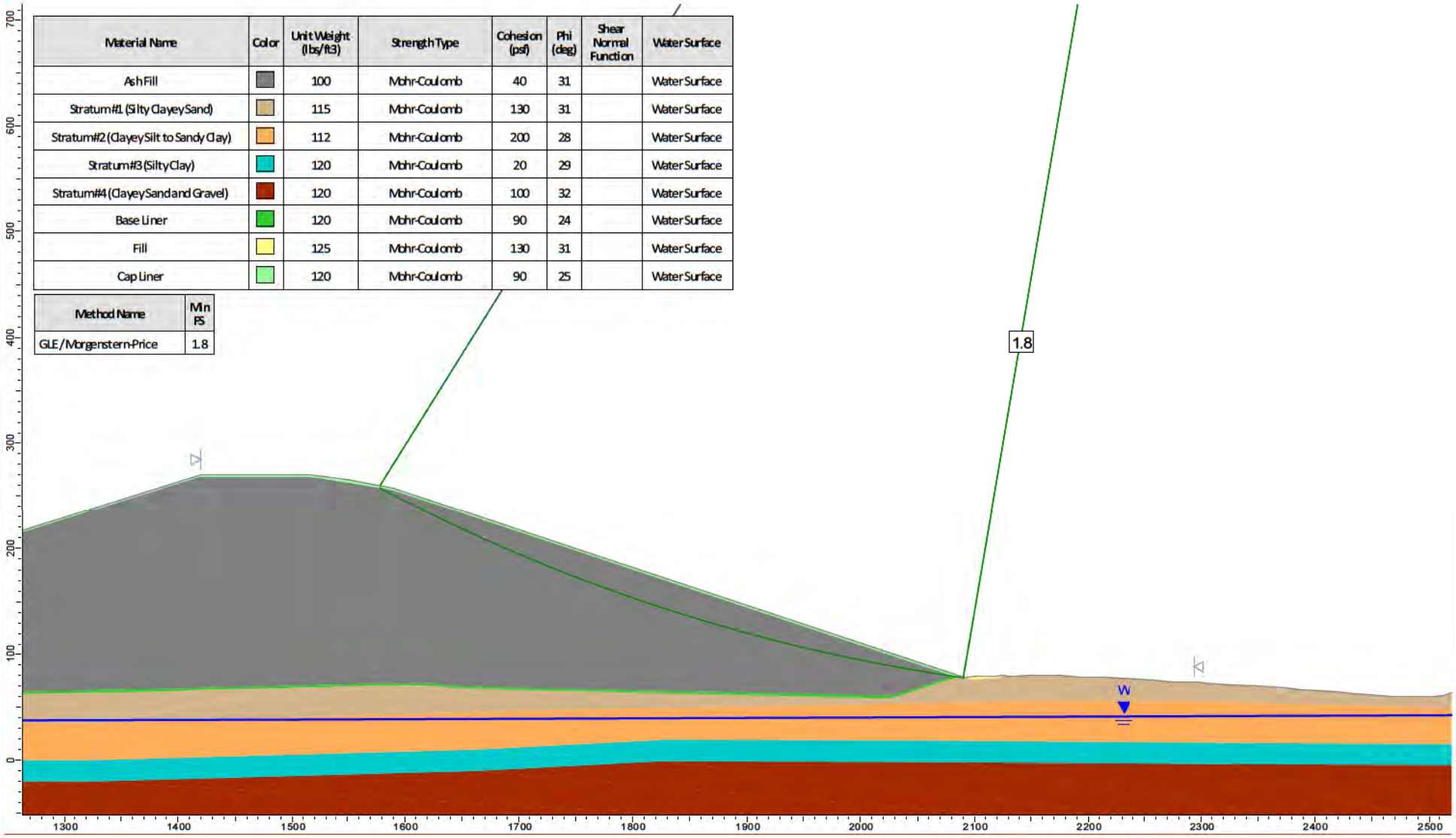
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu	Ru
Ash Fill		100	Mohr-Coulomb	40	31				None			0
Stratum#1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23				None			0
Stratum#2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio				0.51	0	Water Surface	Custom	1	
Stratum#3 (Silty Clay)		120	Vertical Stress Ratio				0.42	0	Water Surface	Custom	1	
Stratum#4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32				Water Surface	Custom	1	
Liner		120	Mohr-Coulomb	90	24				Water Surface	Custom	1	
Fill		125	Mohr-Coulomb	130	31				Water Surface	Custom	1	
Cap Liner		120	Mohr-Coulomb	90	25				Water Surface	Custom	1	



Golder Associates	SCALE: AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill	
	DATE: June 2016	TITLE: Section B-B, South Seismic Loading Conditions Non-Circular Slip Surface Search	
	MADE BY: JGM		
	CAD: -		
FILE: STABILITY	CHECK: GLH	CLIENT: Dominion	FIGURE: 4D
PROJECT No. 1520610 REV. 0	REVIEW: GLH		

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface
Stratum#1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface
Stratum#2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface
Stratum#3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface
Stratum#4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface
Base Liner		120	Mohr-Coulomb	90	24		Water Surface
Fill		125	Mohr-Coulomb	130	31		Water Surface
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface

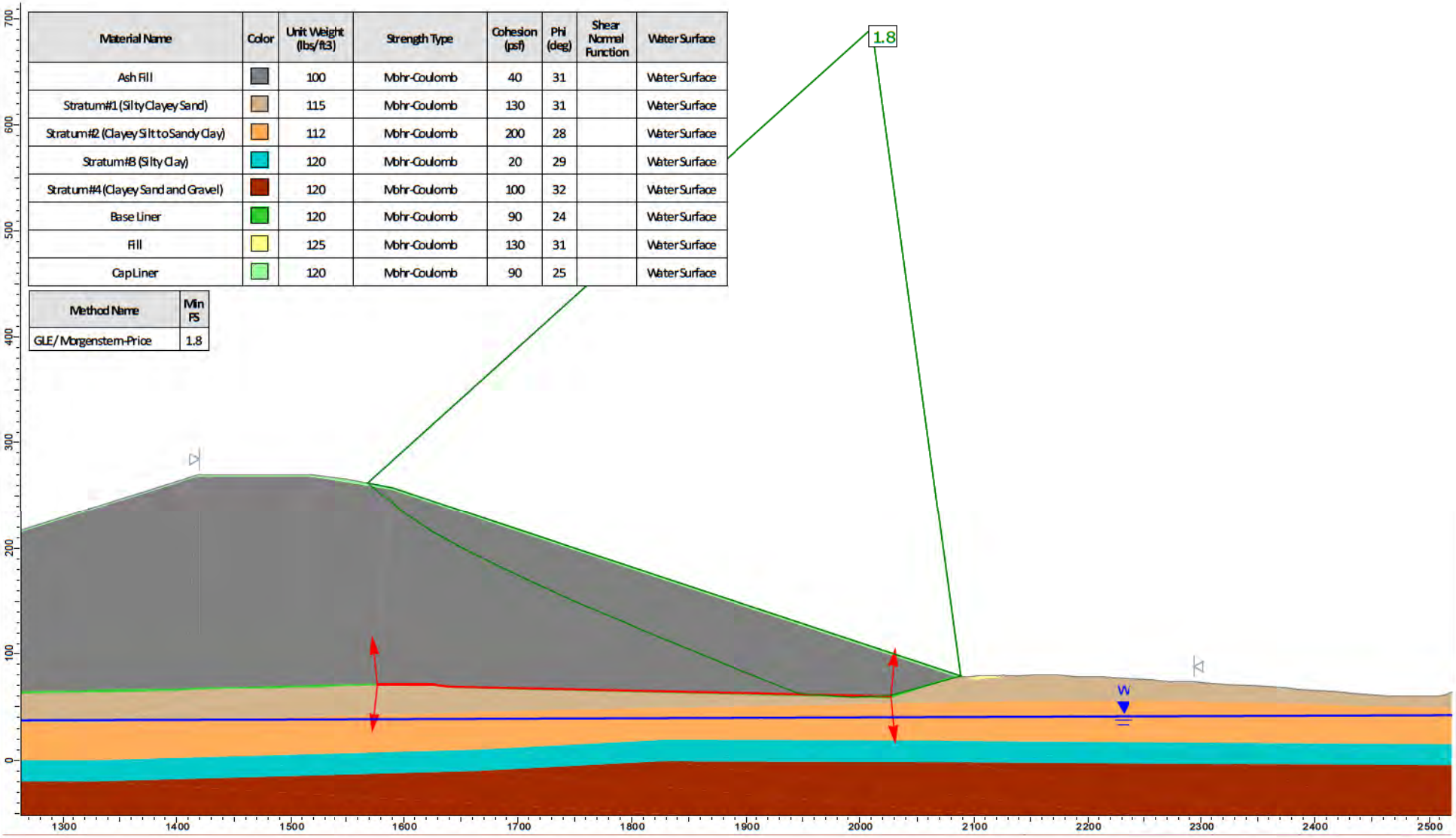
Method Name	Min FS
GLE/Morgenstern-Price	1.8



Golder Associates	SCALE	AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill
	DATE	June 2016	TITLE: Section B-B, North Long-Term (Effective) Stress, Steady State Analysis Circular Slip Surface Search
	MADE BY	JGM	
	CAD	-	
FILE: STABILITY	CHECK	GLH	CLIENT: Dominion
PROJECT No. 1520610 REV. 0	REVIEW	GLH	FIGURE 5A

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Water Surface
Ash Fill		100	Mohr-Coulomb	40	31		Water Surface
Stratum#1 (Silty Clayey Sand)		115	Mohr-Coulomb	130	31		Water Surface
Stratum#2 (Clayey Silt to Sandy Clay)		112	Mohr-Coulomb	200	28		Water Surface
Stratum#3 (Silty Clay)		120	Mohr-Coulomb	20	29		Water Surface
Stratum#4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32		Water Surface
Base Liner		120	Mohr-Coulomb	90	24		Water Surface
Fill		125	Mohr-Coulomb	130	31		Water Surface
Cap Liner		120	Mohr-Coulomb	90	25		Water Surface

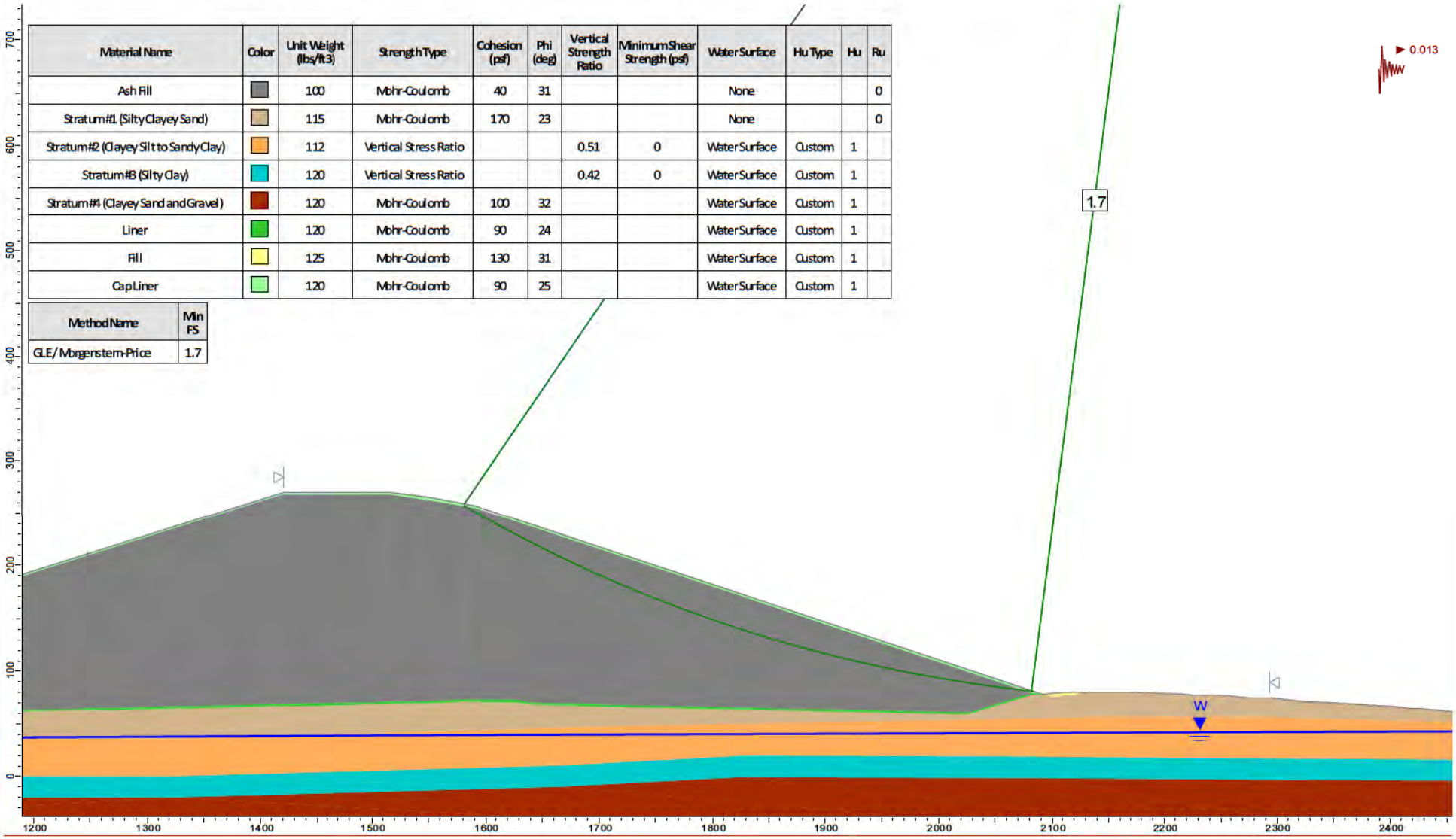
Method Name	Min FS
GLE/ Morgenstern-Price	1.8



Golder Associates	SCALE	AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill
	DATE	June 2016	TITLE: Section B-B, North Long-Term (Effective) Stress, Steady State Analysis Non-Circular Slip Surface Search
	MADE BY	JGM	
	CAD	-	
FILE: STABILITY	CHECK	GLH	CLIENT: Dominion
PROJECT No. 1520610 REV. 0	REVIEW	GLH	FIGURE: 5B

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu	Ru
Ash Fill		100	Mohr-Coulomb	40	31			None			0
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23			None			0
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio			0.51	0	Water Surface	Custom	1	
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio			0.42	0	Water Surface	Custom	1	
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32			Water Surface	Custom	1	
Liner		120	Mohr-Coulomb	90	24			Water Surface	Custom	1	
Fill		125	Mohr-Coulomb	130	31			Water Surface	Custom	1	
Cap Liner		120	Mohr-Coulomb	90	25			Water Surface	Custom	1	

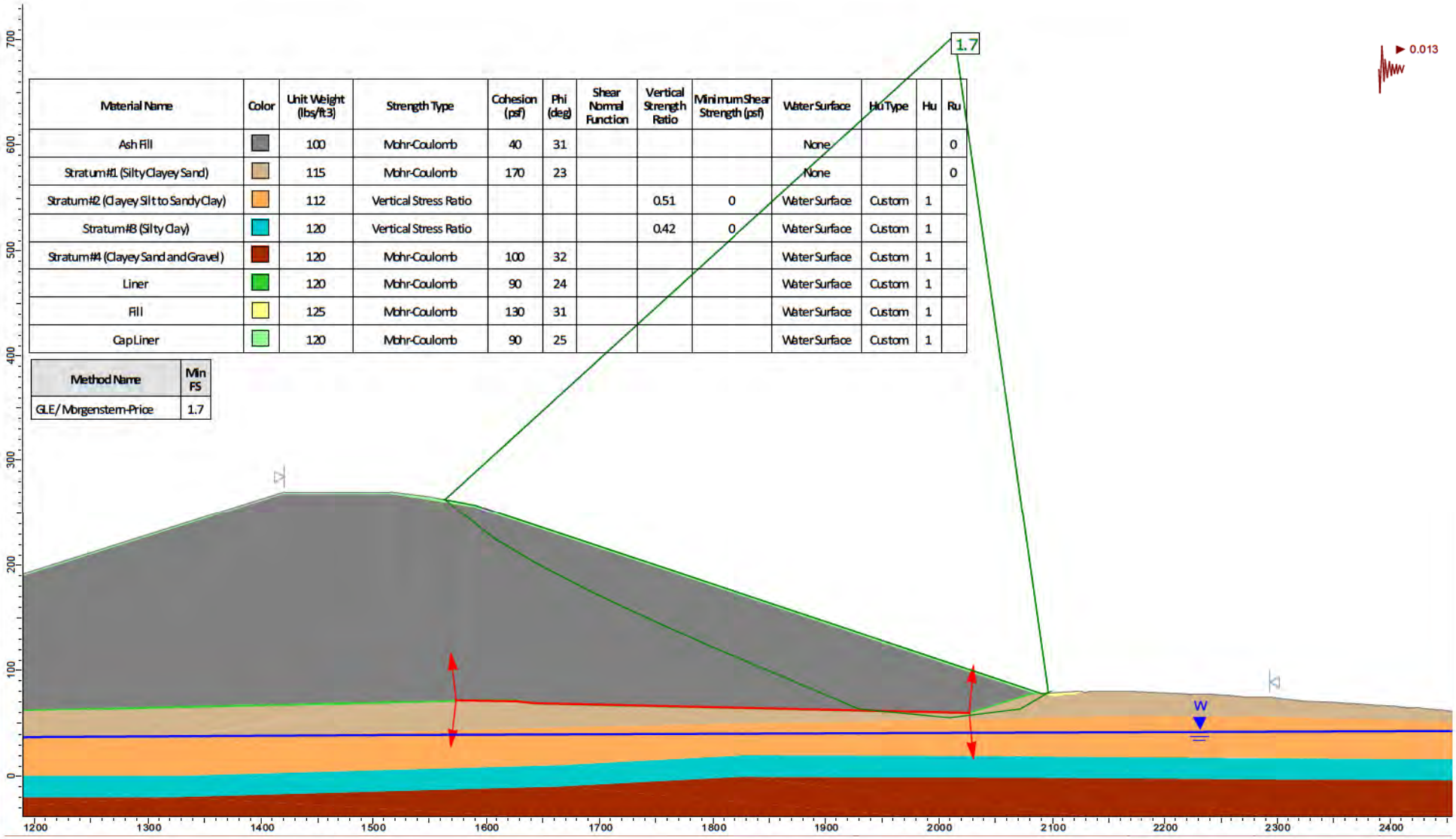
Method Name	Min FS
GLE/Morgenstern-Price	1.7



Golder Associates	SCALE: AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill	
	DATE: June 2016	TITLE: Section B-B, North Seismic Loading Conditions Circular Slip Surface Search	
	MADE BY: JGM		
	CAD: -		
FILE: STABILITY	CHECK: GLH	CLIENT: Dominion	FIGURE: 5C
PROJECT No. 1520610 REV. 0	REVIEW: GLH		

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shear Normal Function	Vertical Strength Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu	Ru
Ash Fill		100	Mohr-Coulomb	40	31				None			0
Stratum #1 (Silty Clayey Sand)		115	Mohr-Coulomb	170	23				None			0
Stratum #2 (Clayey Silt to Sandy Clay)		112	Vertical Stress Ratio				0.51	0	Water Surface	Custom	1	
Stratum #3 (Silty Clay)		120	Vertical Stress Ratio				0.42	0	Water Surface	Custom	1	
Stratum #4 (Clayey Sand and Gravel)		120	Mohr-Coulomb	100	32				Water Surface	Custom	1	
Liner		120	Mohr-Coulomb	90	24				Water Surface	Custom	1	
Fill		125	Mohr-Coulomb	130	31				Water Surface	Custom	1	
Cap Liner		120	Mohr-Coulomb	90	25				Water Surface	Custom	1	

Method Name	Min FS
GLE/Morgenstern-Price	1.7



Golder Associates	SCALE: AS SHOWN	PROJECT: Chesterfield / Reymet Road Landfill	
	DATE: June 2016	TITLE: Section B-B, North Seismic Loading Conditions Non-Circular Slip Surface Search	
	MADE BY: JGM		
	CAD: -		
FILE: STABILITY	CHECK: GLH	CLIENT: Dominion	FIGURE: 5D
PROJECT No. 1520610 REV. 0	REVIEW: GLH		

ATTACHMENT E

Veneer Stability Calculations

Golder Associates	SUBJECT:	Stability of Base Liner System - Veneer Stability			
	Job No.	1520610	Made by	JGM	Date 6/10/2016
	Ref. :	Chesterfield / Reymet Rd Landfill	Checked	GLH	1 of 5
			Reviewed	GLH	

OBJECTIVE:

Analyze the stability of the bottom liner system under the critical case of loading during construction.

Use design strength parameters as defined in the material properties package and the bottom liner detail shown in the November 2015 Golder Drawings.

GOLDER RECOMMENDED FACTORS OF SAFETY FOR LANDFILL BASE LINER

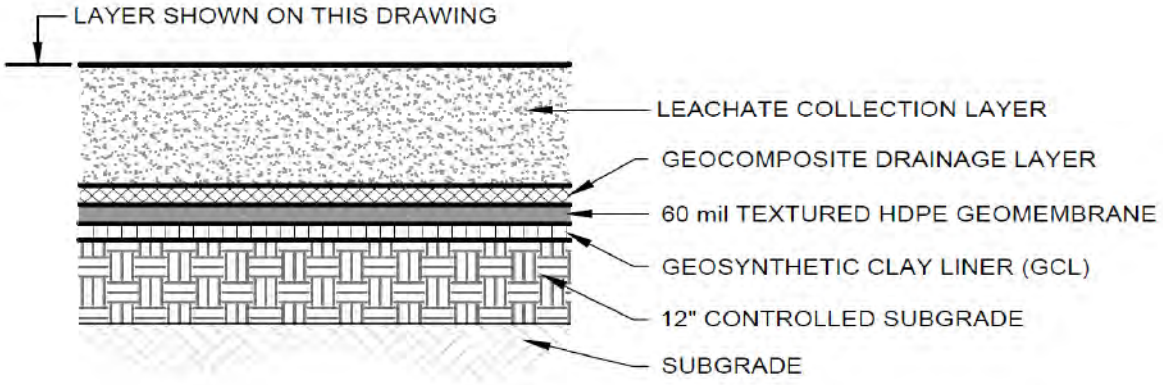
Shear Strength	During Construction (Short term)	Long Term
Design	1.1	1.5

If the calculated factors of safety based on the bottom liner system are higher than the recommended factors of safety, the stability requirements are met.

Top Elevation of Critical Slope	52 ft
Toe Elevation of Critical Slope:	26 ft
Critical Slope is:	3 H:1V

The critical slope used for analysis is a 3:1 slope in Phase 1, as seen on the FFCP Management Facility Site Plan dated November 2015.

GEOMETRY:



LINER SECTION

NOT TO SCALE

Bottom Liner Detail from Golder Drawing Sheet 17.

Material Properties (refer to Material Properties Package)

Material	c (psf)	c_a (psf)	ϕ (°)	δ (°)	γ (pcf)	Thickness (ft)
Protective Cover (Ash)	40	-	31	-	120	2.00
Liner System [1]	-	90	-	24	120	0.03
Controlled Subgrade [2]	130		31		115	1.00

[1] Liner System includes Geocomposite Drainage Layer, Textetured goemembrane, GCL, and Clay. Strength is minium of interface and internal strengths. See Material Properties Package for more details.

[2] Stratum #1 effective stress properties from the material properties package used for subgrade properties

Where:

c = Cohesion of the cover soil

c_a = Adhesion between cover soil of the active wedge and the geomembrane

δ = Interface friction angle between cover soil and geomembrane

ϕ = Friction Angle of cover soil

γ = Unit weight of the cover soil

Slope Angle =	β (°) =	18.4
Slope Height =	H (ft) =	26.0

CALCULATIONS:

LONG TERM VENEER STABILITY based on Koerner/Soong Method (page 487 to 490, ref. 2)

Using the Koerner/Soong Method, the factor of safety is calculated using the following equation (Eq. 13.9, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_a - N_a \times \cos\beta) \cos\beta$

$b = -[(W_a - N_a \times \cos\beta) \times \sin\beta \tan f + (N_a \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan f) \times \sin\beta]$

$c = (N_a \times \tan\delta + C_a) \times \sin^2\beta \times \tan f$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

$N_a = W_a \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

Where:

W_a = Total weight of the active wedge

N_a = Effective force normal to the failure plane of the active wedge

C_a = Adhesive force between cover soil of the active wedge and the geomembrane

W_p = Total weight of the passive wedge

C = Cohesive force along the failure plane of the passive wedge

γ = Unit Weight of protective cover soil

h = Thickness of cover soil

β = Slope Angle

L = Length of slope measured along the geosynthetic interface

c = Cohesion of the cover soil

c_a = Adhesion between cover soil of the active wedge and the geomembrane

δ = Interface friction angle between cover soil and geomembrane

ϕ = Friction Angle of cover soil

Where:

h = Thickness of Cover (ft) = 2.00

β = Cover Slope Angle (°) = 18.4

H_{max} = Maximum height = 26.0 feet

L= 82.2 feet

Since h and L are known for **LONG-TERM Conditions**, solve for the FS:

W_a (lbs/ft) =	18,135		
N_a (lbs/ft) =	17,204	W_p (lbs/ft) =	800
C_a (lbs/ft) =	76 x c_a	C (lbs/ft) =	253

$(W_a - N_a \times \cos\beta)$ =	1,813
$(C + W_p \times \tan\phi)$ =	734
$\cos\beta$ =	0.95
$\sin\beta$ =	0.32
$\sin\beta \times \tan\phi$ =	0.19
$\sin^2\beta \times \tan\phi$ =	0.06
$\sin\beta \times \cos\beta$ =	0.30
$\tan\phi$ =	0.60

a= 1720.4

-b= 576.6 + 0.30 x $(N_a \times \tan\delta + C_a)$

c = $(N_a \times \tan\delta + C_a)$ x 0.06

Solve for FS with different combinations of δ an c_a:

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	$(N_a \times \tan\delta + C_a)$	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
24.00	*90*	*0.4*	*6,831*	*14,490*	*-4,924*	*871*	*4272.1*	*2.7*

SHORT TERM CONDITIONS (*Dozer on the slope without acceleration)*

Veneer Stability based on Koerner/Soong Method (page 490-497, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_{a+e} - N_{a+e} \times \cos\beta) \cos\beta$

$b = -[(W_{a+e} - N_{a+e} \times \cos\beta) \times \sin\beta \times \tan\phi + (N_{a+e} \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan\phi) \times \sin\beta]$

$c = (N_{a+e} \times \tan\delta + C_a) \times \sin^2\beta \times \tan\phi)$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

W_e = Equipment Weight, see below

$W_{a+e} = W_a + W_e$

$N_{a+e} = W_{a+e} \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

The definitions of all the parameters are as same as those in long term FS calculation except W_e, W_{a+e}, and N_{a+e}

$L_{short\ term}$=	82.2	ft
$h_{short\ term}$=	2.00	ft
ϕ =	31.0	degrees
c =	40	psf
$\gamma_{soil\ cover}$ =	120	pcf

Determination of W_e (See dozer specifications from manufacturer, ref. 3):

Width of Dozer Track =	3.00	ft
Contact Area =	64.26	sq.ft.
Ground Pressure =	4.8	psi
Influence factor (I) =	0.95	(obtained from Figure 13.7, page 493, ref. 2)
Ground Pressure at Geosynthetics =	652.4	psf
Length of Dozer Track =	10.7	ft

W_e= 6987 lbs/ft

W_a+W_e (lbs/ft) =	25,124		
N_{a+e} (lbs/ft) =	23,835	W_p (lbs/ft) =	800
C_a (lbs/ft) =	76 x c_a	C (lbs/ft) =	253

$(W_{a+e} - N_{a+e} \times \cos\beta)$ =	2,512
$(C + W_p \times \tan\phi)$ =	734
$\cos\beta$ =	0.95
$\sin\beta$ =	0.32
$\sin\beta \times \tan\phi$ =	0.19
$\sin^2\beta \times \tan\phi$ =	0.06
$\sin\beta \times \cos\beta$ =	0.30
$\tan\phi$ =	0.60

a=	2383.5			
- b=	709.4	+	0.30	x $(N_a \times \tan\delta + C_a)$
c =	$(N_a \times \tan\delta + C_a)$	x	0.06	

Solve for FS :

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	$(N_{a+e} \times \tan\delta + C_a)$	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
24.00	*90.00*	*0.4*	6830.52	*17,443*	*-5,942*	*1,048*	*5,032*	*2.3*

SUMMARY OF RESULTS

CASE ANALYZED	TARGET FACTOR OF SAFETY	ACTUAL FACTOR OF SAFETY	EVALUATION
Long Term using Design Shear Strength	1.5	2.7	Satisfactory
Short Term using Design Shear Strength - Dozer on Slope	1.1	2.3	Satisfactory

Therefore, the stability of the base liner meets the recommended factors of safety for both short and long term conditions.

References:

1. Material Properties Package, Golder Associates, June 2016

2. Qian, X., Koerner, R. M., Gray, D. H., Geotechnical Aspects of Landfill Design and Construction, Prentice Hall, New Jersey, US, 2002.

3. Dozer Specifications from Manufacturer

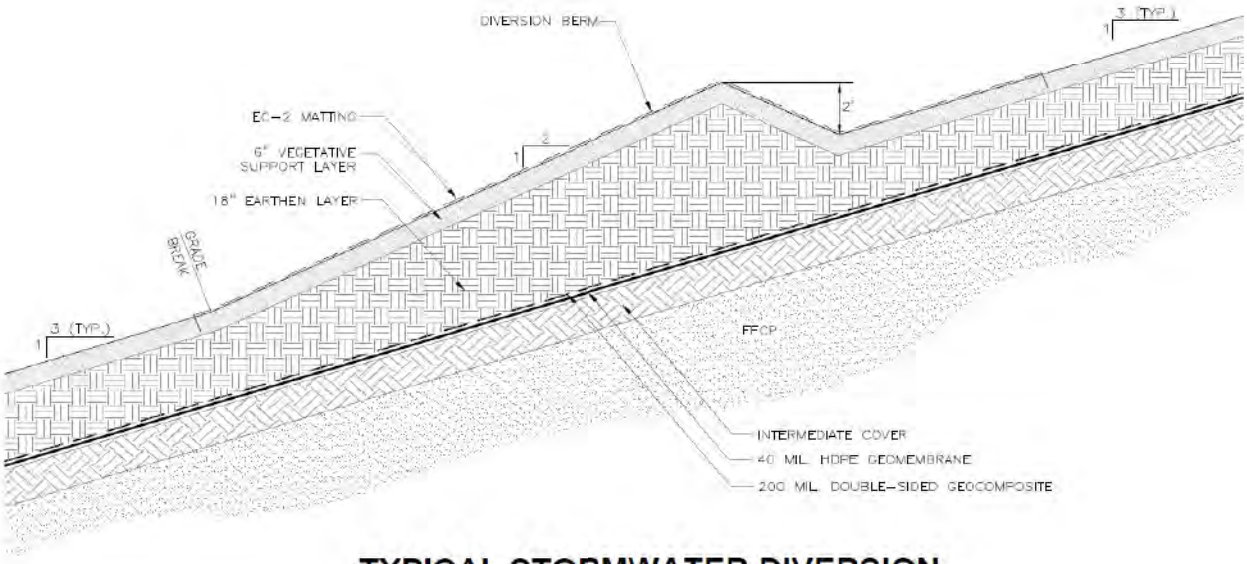
4. Golder Associates Inc. Drawing Sheets, FFCP Management Facility Site Plan, November 2015

	SUBJECT:	Stability of Cover System - Veneer Stability				
Golder Associates	Job No.	1520610	Made by	JGM	Date	6/10/2016
	Ref. :	Chesterfield / Reymet Rd Landfill	Checked	GLH		
			Reviewed	GLH		

OBJECTIVE:

Analyze the stability of the final cover system for the proposed Chesterfield landfill.
Use design strength parameters and analyze for conditions with and without seepage forces.

GEOMETRY:



TYPICAL STORMWATER DIVERSION BERM AND FINAL COVER DETAIL

4 / 27

NOT TO SCALE

GOLDER RECOMMENDED FACTORS OF SAFETY FOR LANDFILL FINAL COVER

Shear Strength	During Construction (Short term)	Long Term
Design	1.1	1.5, 1.1 [a]

[a] Recommended factor of safety with seepage forces included
If the calculated factors of safety based on the final cover conditions are higher than the recommended factors of safety for landfill final cover, the stability of the final cover meets the requirement.

Based on Proposed Cover Grades:	
Top Elevation of Cover	270 ft
Approximate Toe Elevation :	70 ft
Slope:	3H:1V

Material Properties (Golder Material Properties Package 2016)

Material	c (psf)	c_a (psf)	ϕ (°)	δ (°)	γ (pcf)	Thickness (ft)
Cover soil (CS)	100	-	28	-	120	2.00
Cap Liner System [1]	-	90	-	25	-	0.03

[1] Veneer stability was calculated for ash strengths which control for overburden pressures less than 300 psf. The displayed strength parameters were found to be more critical for this calculation due to factors of safety calculated in the seepage analysis. See Material Properties Package for more details on selection of strength properties.

Where:
- c = Cohesion of the cover soil
- c_a = Adhesion between cover soil of the active wedge and the geomembrane
- δ = Interface friction angle between cover soil and geomembrane
- f = Friction Angle of cover soil
- γ = Unit weight of the cover soil

Slope Angle =	β (°) =	18.4	
Slope Height =	200.0	ft	(H)

CALCULATIONS:

LONG TERM VENEER STABILITY based on Koerner/Soong Method (page 487 to 490, ref. 2)

Using the Koerner/Soong Method, the factor of safety is calculated using the following equation (Eq. 13.9, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_a - N_a \times \cos\beta) \cos\beta$

$b = -[(W_a - N_a \times \cos\beta) \times \sin\beta \tan f + (N_a \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan f) \times \sin\beta]$

$c = (N_a \times \tan\delta + C_a) \times \sin^2\beta \times \tan f$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

$N_a = W_a \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

Where:
- W_a = Total weight of the active wedge
- N_a = Effective force normal to the failure plane of the active wedge
- C_a = Adhesive force between cover soil of the active wedge and the geomembrane
- W_p = Total weight of the passive wedge
- C = Cohesive force along the failure plane of the passive wedge
- γ = Unit Weight of protective cover soil
- h = Thickness of cover soil
- β = Slope Angle
- L = Length of slope measured along the geosynthetic interface
- c = Cohesion of the cover soil
- c_a = Adhesion between cover soil of the active wedge and the geomembrane
- δ = Interface friction angle between cover soil and geomembrane
- f = Friction Angle of cover soil

Where:

h = Thickness of Cover (ft) =	2.00	
β = Cover Slope Angle (°) =	18.4	
H_{max} = Maximum height =	200.0	feet
L=	200.0	feet

Golder Associates	SUBJECT:	Stability of Cover System - Veneer Stability				
	Job No.	1520610	Made by	JGM	Date	6/10/2016
	Ref. :	Chesterfield / Reymet Rd Landfill	Checked	GLH		
			Reviewed	GLH		

Since h and L are known for **LONG-TERM Conditions**, solve for the FS:

W_a (lbs/ft) = 46,402

N_a (lbs/ft) = 44,021

C_a (lbs/ft) = 194 x c_a

W_p (lbs/ft) = 800

C (lbs/ft) = 632

(W_a - N_a x cos β) = 4,640

(C + W_p x tan f) = 1058

cos β = 0.95

sin β = 0.32

sin β x tan f = 0.17

$\sin^2 \beta$ x tan f = 0.05

sin β x cos β = 0.30

tan f = 0.53

a= 4402.1

- b= 1114.7 0.30

c = × 0.05

Solve for FS with different combinations of δ an c_a:

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	(N_a x tan δ + C_a)	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
25.00	*90*	*0.5*	*17,431*	*37,958*	*-12,502*	*2,018*	*10989.3*	*2.7*

Golder Associates	SUBJECT:	Stability of Cover System - Veneer Stability				
	Job No.	1520610	Made by	JGM	Date	6/10/2016
	Ref. :	Chesterfield / Reymet Rd Landfill	Checked	GLH		
			Reviewed	GLH		

SHORT TERM CONDITIONS (*Dozer on the slope without acceleration)*

Veneer Stability based on Koerner/Soong Method (page 490-497, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_{a+e} - N_{a+e} \times \cos\beta) \cos\beta$

$b = -[(W_{a+e} - N_{a+e} \times \cos\beta) \times \sin\beta \times \tan f + (N_{a+e} \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan f) \times \sin\beta]$

$c = (N_{a+e} \times \tan\delta + C_a) \times \sin^2\beta \times \tan f)$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

W_e = Equipment Weight, see below

$W_{a+e} = W_a + W_e$

$N_{a+e} = W_{a+e} \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

The definitions of all the parameters are as same as those in long term FS calculation except W_e, W_{a+e}, and N_{a+e}

For SHORT-Term Conditions, look at soil being placed up slope with a Low Ground Pressure Dozer

$L_{short\ term}$=	200.0	ft
$h_{short\ term}$=	2.00	ft
f =	28.00	degrees
c =	100.00	psf
$\gamma_{soil\ cover}$ =	120.00	pcf

Determination of W_e (See dozer specifications from manufacturer, ref. 3):

Width of Dozer Track =	3.00	ft
Contact Area =	64.26	sq.ft.
Ground Pressure =	4.8	psi
Influence factor (I) =	0.95	(obtained from Figure 13.7, page 493, ref. 2)
Ground Pressure at Geosynthetics =	652.4	psf
Length of Dozer Track =	10.7	ft
W_e=	**6987**	**lbs/ft**

W_a+W_e (lbs/ft) =	53,392		
N_{a+e} (lbs/ft) =	50,652	W_p (lbs/ft) =	800
C_a (lbs/ft) =	194 x c_a	C (lbs/ft) =	632

$(W_{a+e} - N_{a+e} \times \cos\beta)$ =	5,339
$(C + W_p \times \tan f)$ =	1058
$\cos\beta$ =	0.95
$\sin\beta$ =	0.32
$\sin\beta \times \tan f$ =	0.17
$\sin^2\beta \times \tan f$ =	0.05
$\sin\beta \times \cos\beta$ =	0.30
$\tan f$ =	0.53

a=	5065.2	
- b=	1232.2	0.30
c =		0.05

Solve for FS :

δ (°)	c_a (psf)	$\tan\delta$	C_a (lbs/ft)	$(N_{a+e} \times \tan\delta + C_a)$	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
25.00	*90.00*	*0.5*	17430.79	*41,050*	*-13,547*	*2,183*	*11,803*	*2.5*

Golder Associates	SUBJECT:	Stability of Cover System - Veneer Stability				
	Job No.	1520610	Made by	JGM	Date	6/10/2016
	Ref. :	Chesterfield / Reymet Rd Landfill	Checked	GLH		
			Reviewed	GLH		

SEEPAGE BUILD-UP CONDITION

Veneer Stability based on Koerner/Soong Method (page 501-508, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = W_a \sin\beta \cos\beta + U_H \times (1 - \cos^2\beta)$

$b = -[Wp \times \tan f + W_a \times (\sin^2\beta \times \tan f + \cos^2\beta \times \tan\delta) - U_{AN} \times \cos\beta \times \tan\delta - U_{PN} \times \tan f + U_H \times \sin\beta \times \cos\beta \times (\tan f - \tan\delta)]$

$c = (W_a \times \cos\beta - U_{AN} + U_H \times \sin\beta) \times \sin\beta \times \tan\delta \times \tan f)$

$U_{AN} = \gamma_w \times h_w \times (H - 0.5 \times h_w \times \cos\beta) / \tan\beta$

$U_H = 0.5 \times \gamma_w \times h_w^2$

$U_{PN} = 0.5 \times \gamma_w \times h_w^2 / \tan\beta$

$W_a = 0.5 \times [\gamma \times (h - h_w) \times (2 \times H \times \cos\beta - h - h_w) + \gamma_{sat} \times h_w \times (2 \times H \times \cos\beta - h_w)] / (\sin\beta \times \cos\beta)$

$W_p = 0.5 \times [\gamma \times (h^2 - h_w^2) + \gamma_{sat} \times h_w^2] / (\sin\beta \times \cos\beta)$

U_H = Resultant of the pore water pressures acting on lateral side of the active wedge or passive wedge
U_{PN} = Resultant of the pore water pressures acting on bottom of the passive wedge
U_{AN} = Resultant of the pore water pressures acting on bottom of the active wedge
h = Thickness of the soil layer
h_w = Depth of seepage water in the soil layer (perpendicular to the slope)
γ_w = Unit weight of water
γ = Moisture unit weight of the soil layer
γ_{sat} = Saturated unit weight of the soil layer

Other parameters are same as in the above calculations

γ_w =	62.4	lb/ft^3	
h_w =	10.0	inches	
H =	200.0	ft	
γ_{sat} =	130	lb/ft^3	(Assumed)
U_{AN} =	31138	lbs/ft	
U_H =	21.67	lbs/ft	
U_{PN} =	65.00	lbs/ft	
W_a =	156248	lbs/ft	
W_p =	812	lbs/ft	

Wp x tan f =	432	$\sin^2\beta \times \tan f$ =	0.05
$U_{PN} \times \tan f$ =	34.56	$\sin\beta$ =	0.32
$U_H \times \sin\beta \times \cos\beta$ =	6.5000	$\tan f$ =	0.53
		$\cos^2\beta$ =	0.90
		$\cos\beta$ =	0.95

a =	46877		
- b=	8708	+	111076 *tan δ
c =	19689.1		

δ (°)	c_a (psf)	tan δ	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
25.0	*90.0*	*0.466*	*-60,504*	*9,181*	*44037*	*1.1*

SUMMARY OF RESULTS

CASE ANALYZED	REQUIRED FACTOR OF SAFETY	ACTUAL FACTOR OF SAFETY	MEET REQUIREMENT
Long Term using Design Shear Strength	1.5	2.7	Yes
Short Term using Design Shear Strength - Dozer on Slope	1.1	2.5	Yes
Seepage Analysis	1.1	1.1	Yes

Therefore, the stability of the final cover meets the recommended factors of safety provided the cover drainage layer maintains a maximum fluid head condition of no greater than 10 inches above the liner. As such, the cover drainage layer should be designed to maintain this condition.

References:

1. Material Properties Package, Golder Associates, June 2016

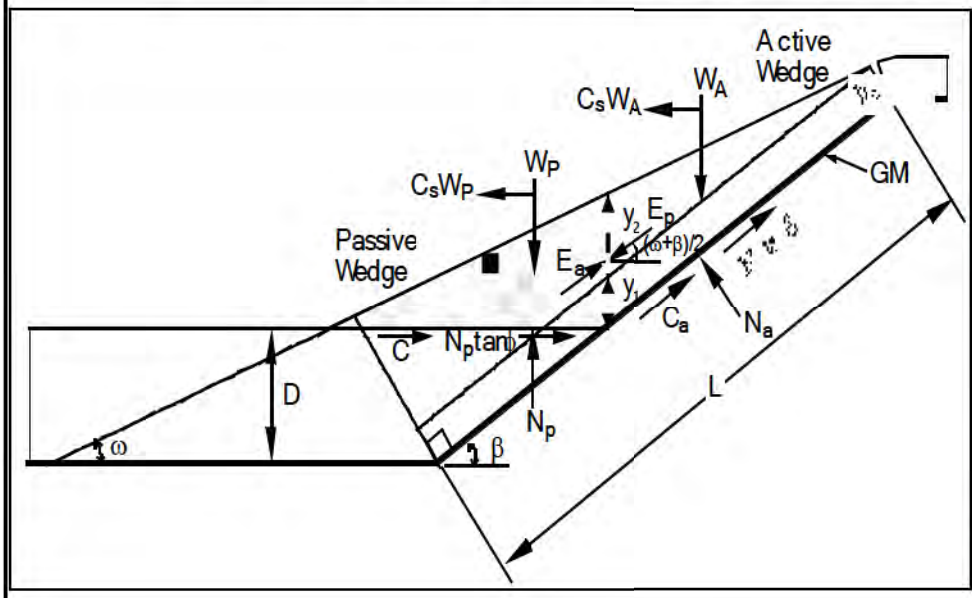
2. Qian, X., Koerner, R. M., Gray, D. H., Geotechnical Aspects of Landfill Design and Construction, Prentice Hall, New Jersey, US, 2002.

3. Dozer Specifications from Manufacturer

4. Golder Associates Inc. Drawing Sheets, FFCP Management Facility Site Plan, November 2015

Cap Liner Stability Analysis Worksheet

Uniform and/or Tapered Cover Soil with Consideration of Seismic Forces



Calculation of FS

Active Wedge:

Wa= 467 kips
Na= 443 kips
Ca= 77.9 kips

Passive Wedge:

Wp= 2.6 kips
C= 2.1 kips

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

a= 174
b= -344
c= 54.8

FS= 1.81

(Note: for uniform cover soil thickness the input value of $\omega = \beta$)

thickness of cover soil at top (crest) of the slope = hc =	2.0	ft	
thickness of cover soil along the bottom of the site = D =	2.0	ft	
soil slope angle beneath the geomembrane = β =	18.4	°	*= 0.32 (rad.)*
finished cover soil slope angle = ω =	18.4	°	*= 0.32 (rad.)*
length of slope measured along the geomembrane = L =	600.0	ft	

y2= 0.00 (ft)
y1= 2.11 (ft)
$(\omega+\beta)/2$= 0.32 (rad.)
(= 18.4 °)

unit weight of the cover soil = γ =	120	pcf	
friction angle of the cover soil = ϕ =	28.0	°	*= 0.49 (rad.)*
cohesion of the cover soil = c =	100.0	psf	
interface friction angle between cover soil and geomembrane = δ =	31.0	° *	*= 0.54 (rad.)*
adhesion between cover soil and geomembrane = ca =	40.0	psf*	
seismic coefficient = Cs =	0.09	g**	

*Strength parameters for stresses greater than 300 psf (δ=25, c=90 psf) were found to be less critical for this scenario. See Material Properties Package for more details (Golder 2016).
** Cs = 0.5 * Site specific PGA (0.183); Based on Kavazanjian et al. 2007.

Note: numbers in boxes are input values
numbers in Italics are calculated values

References:

Kavazanjian, E., JR., N. Matasovic, 1997. T. Hadj-Hamou, and P. J. Sabatini. Geotechnical Engineering Circular No. 3: Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I - Design Principles. US Department of Transportation.

R. M. Koerner, and T-Y. Soong, 1998. "Analysis and Design of Veneer Cover Soils". Proceedings of 6th International Conference on Geosynthetics, Vol. 1, pp. 1-23, Atlanta, Georgia, USA.

Golder Associates Inc., (2016) Material Properties Package

Golder Associates Inc. (2015) Drawing Sheets, FFCP Management Facility Site Plan, November

Established in 1960, Golder Associates is a global, employee-owned organization that helps clients find sustainable solutions to the challenges of finite resources, energy and water supply and management, waste management, urbanization, and climate change. We provide a wide range of independent consulting, design, and construction services in our specialist areas of earth, environment, and energy. By building strong relationships and meeting the needs of clients, our people have created one of the most trusted professional services organizations in the world.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Inc.
2108 W. Laburnum Ave, Suite 200
Richmond, VA 23227
Tel: (804) 358-7900

Golder Associates

Engineering Earth's Development, Preserving Earth's Integrity

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation