GeoStrata

MEMORANDUM

To: **Gerber Construction**
Attn: Kyle Leishman

From: **Mark I. Christensen, PE**

Date: **May 12, 2011**

Subject: **Bearing Capacity Recommendations**
Murdock Trail
800 East Box Culvert
Orem, Utah

LICENSED PROFESSIONAL ENGINEER
MARK I. CHRISTENSEN
No. 188039
LICENSED
STATE OF UTAH

At your request GeoStrata has prepared this Memorandum to present bearing capacity recommendations for a box culvert planned for the 800 East crossing of the Murdock Canal Trail in Orem, Utah. The location of the proposed box culvert is shown on Plate 1. It is our understanding that the box culvert is to be about 19 feet wide, to extend about 18 feet below the 800 East surface and be founded on spread footings. We further understand that unfactored structural loads of the box culvert walls will be on the order of 18 kips per lineal foot.

Review of Data Report and Subsurface Investigation

A geotechnical data report prepared by RB & G Engineering, Inc. dated October 2010 was provided to us and we relied on this data report together with additional geotechnical information obtained from the investigation for this memorandum, to assist us in our analysis. The RB & G data report provided a boring log from a boring drilled on the west side of 800 East and south side of the canal with laboratory testing. A copy of the RB & G boring log is attached in the appendix to this memo and is identified as DH 10-4. In addition to the boring drilled for the data report GeoStrata drilled a boring on the east side of 800 East and on the south side of the canal to a depth of 25 ½ feet below existing site grade. The boring was completed with a CME 55 truck mounted rig using hollow-stem augers. The approximate location of the RB & G and GeoStrata explorations are shown on the exploration location map, Plate 2. A log of the subsurface conditions, as encountered in the GeoStrata exploration, was recorded at the time of excavation by a qualified engineer and is presented on Plate 3. A Key to USCS Soil Symbols and Terminology used on the boring log is found on Plate 4.

Soil sampling occurred at varying depths throughout the boring. The soils observed in the exploration were classified according to the Unified Soil Classification System (USCS) by the engineer.

Laboratory Testing

In addition to the testing performed by RB & G, representative soil samples collected during drilling of the GeoStrata boring were tested in the laboratory to assess pertinent engineering properties. Moisture content and density determinations were performed to estimate the in-place moisture and

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density conditions of the on-site soils. Grain size distributions and Atterberg limits were also performed to aid in developing engineering recommendations for the site. Direct shear tests were performed to estimate the in-situ soil strength. Results of the laboratory tests are included on the boring logs and on Plates 5 through 7.

Subsurface Conditions
Soils exposed in the borings generally consist of interbedded zones of soft to stiff Silty Clay with gravel (CL-ML), Sandy Silty Clay with gravel (CL-ML), and Sandy Silt (ML) to a depth of 16 to 20 feet which is underlain by zones of loose to very dense Silty Sand (SM), Silty Gravel with sand (GM), and Poorly Graded Gravel with silt and sand (GP-GM). Groundwater was encountered at a depth of 45 feet below existing site grade in the RB & G boring and was not encountered in the GeoStrata boring to a depth of 25 ½ feet.

Site Preparation
In areas beneath or adjacent to footings or fill sections, topsoil should be stripped and stockpiled for use in landscape areas or disposal. Any undocumented fill, debris, vegetation, roots, loose, soft or other deleterious materials should also be removed and replaced with structural fill. The exposed subgrade should be proof-rolled with heavy equipment to identify loose, soft or otherwise disturbed soils. The geotechnical engineer should be present during the testing of the subgrade to assess the deflections noted from the heavy equipment and the need to require soil stabilization. If soft soils are observed, they should be stabilized in accordance with our recommendations in the Soft Soil Stabilization Section below; if loose soils are observed, they should be compacted as recommended in the Structural Fill Section below.

Excavations
Based on Occupational Safety and Health Administration (OSHA) guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied, however, the presence of fill soils, loose soils, or wet soils may require that the walls be flattened to maintain safe work conditions. When the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Based on our soil observations, laboratory testing, and OSHA guidelines, native soils at the site classify as Type C soils. Deeper excavations, if required, should be constructed with side slopes no steeper than one and one-half horizontal to one vertical (1.5H:1V). If wet conditions are encountered, side slopes should be further flattened to maintain slope stability. Alternatively shoring or trench boxes may be used to improve safe work conditions in trenches. The contractor is ultimately responsible for trench and site safety. Pertinent OSHA requirements should be met to provide a safe work environment. If site specific conditions arise that require engineering analysis in accordance with OSHA regulations, GeoStrata can respond and provide recommendations as needed.

We recommend that a GeoStrata representative be on-site during all excavations to assess the exposed foundation soils. We also recommend that the Geotechnical Engineer be allowed to review the grading plans when they are prepared in order to evaluate their compatibility with these recommendations.

Soft Soil Stabilization

Although unlikely, excavations at the site may extend into high moisture content soft pumping soils. Once exposed, all subgrade surfaces beneath footings, structures, areas of concrete flatwork, and pavement should be proof rolled with heavy construction equipment. If soft or pumping soils are encountered, these soils should be stabilized prior to construction of footings. Stabilization of the subgrade soils can be accomplished using a clean, coarse angular material worked into the soft subgrade. We recommend the material be greater than 2 inches diameter, but less than 6 inches. A locally available pit-run gravel may be suitable but should contain a high percentage of particles larger than 2 inches and have less than 7 percent fines (material passing the No. 200 sieve). A pit-run gravel may not be as effective as a coarse, angular material in stabilizing the soft soils and may require more material and greater effort. The stabilization material should be worked (pushed) into the soft subgrade soils until a firm relatively unyielding surface is established. Once a firm, relatively unyielding surface is achieved, the area may be brought to final design grade using granular borrow or granular backfill borrow.

In large areas of soft subgrade soils, stabilization of the subgrade may not be practical using the method outlined above. In these areas it may be more economical to place a woven geotextile stabilization fabric against the soft soils covered by 18 inches of coarse gravel material over the woven geotextile. An inexpensive non-woven separation geotextile “filter” fabric should also be placed over the top of the coarse fill prior to placing structural fill or pavement section soils to reduce infiltration of fines from above. The stabilization and separation geotextiles should meet the requirements of section 02075 of the UDOT Standard Specifications for Road and Bridge Construction. The type of gravel fill material selected, the geosynthetics used, and field conditions of the near-surface subgrade soils will determine in large part how much over-excavation of the subgrade soils will be required to achieve an adequate level of stabilization before placing the minimal-thickness of granular borrow or granular backfill borrow below the foundation elements.

Borrow, Granular Borrow, Granular Backfill Borrow and Compaction

All fill placed for the support of box culvert, flatwork or pavements, should consist of either Borrow, Granular Borrow or Granular Backfill Borrow in accordance with UDOT standards. Based on the soils encountered in the borings for this site, we anticipate that the majority of the native subgrade soils may be used for Borrow; however, the native silty clays and sandy silts encountered along the alignment can be difficult to moisture condition and compact. The contractor should make provisions for this possibility. All borrow material should meet the requirements and be placed in accordance with UDOT Standard Specifications Section 02056. Any imported fill materials should be approved prior to importing. Also, prior to placing any fill, the excavations should be observed by the Geotechnical Engineer to confirm that unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

Seismicity

To assist development of the probabilistic spectral accelerations, the peak ground acceleration and site coefficients (Fa and Fv) have been developed using the criteria outlined in the 2007 AASHTO LRFD Bridge Design Specifications with a 7 percent chance of exceedance in 75 years (1,000 year return interval). In addition, the peak ground acceleration with a 2 percent chance of exceedance in 50 years was determined using the NEHRP-based software program published by the USGS. The field explorations completed at the site by R B & G Engineering and GeoStrata encountered interbedded zones of soft to stiff Silty Clay with gravel (CL-ML), Sandy Silty Clay with gravel (CL-ML), and Sandy Silt (ML) to a depth of 16 to 20 feet which is underlain by zones of loose to very dense Silty Sand (SM), Silty Gravel with sand (GM), and Poorly Graded Gravel with silt and sand (GP-GM) consistent with a Site Class D (stiff soil profile). Based on the geologic setting of the subject site, subsurface conditions below the borings likely consist of similar soils. Therefore it is our opinion that soils in the upper 100 feet are best described by Site Class D having Site Coefficients of Fa= 1.25 and Fv=1.90. From the AASHTO procedure the Peak Ground Acceleration (PGA) was estimated to be 0.293 g. The following table presents response accelerations for 0.2 and 1.0 second periods.

Table 1 - Seismic Response Values

Seismic Response Spectrum Spectral Acceleration Values for Site Class D	
Site Location: Latitude = 40.322942 N Longitude = -111.676820 W	**Site Class D Site Coefficients: Fa = 1.25 Fv = 1.90**
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.2 (SDs)	0.858
1.0 (SD1)	0.475
PGA (7% in 75 yrs) = 0.293g	
PGA (2% in 50 yrs) = 0.548g	

Foundations

The foundation for the box culvert may consist of conventional strip, spread footings, or a mat foundation. We recommend that strip and spread footings for the proposed structure be supported entirely on at least 12 inches of granular borrow or granular backfill borrow. Strip footings should be a minimum of 24-inches wide and exterior shallow footings should be embedded at least 30-inches below final grade for frost protection and confinement. Spread footings should be at least 40-inches in the smallest dimension.

Conventional strip footings founded on at least 12 inches of granular borrow or granular backfill borrow may be proportioned for a factored bearing resistance of 1,900 psf. This bearing resistance applies only to the use of conventional strip and spread footings. All footing excavations should be observed by the Geotechnical Engineer prior to footing placement.

Settlements of native soils beneath footings constructed as recommended in this memorandum are anticipated to be less than 1.0 inch. Differential settlements should be on the order of ½ the total settlement over 30 feet.

Mat foundations for the proposed structure should be supported entirely on at least 24 inches of granular borrow or granular backfill borrow. Mat foundations supported on 24 inches of granular borrow or granular backfill borrow should be designed using a modulus of subgrade reaction of 150 pci.

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. As recommended above, footings for the box culvert should be founded on at least 12 inches of granular borrow or granular backfill borrow. Given this the majority of footings will be founded on granular borrow or granular backfill borrow. In determining the frictional resistance, a coefficient of friction of 0.60 for granular borrow or granular backfill borrow with an internal angle of friction of 33° should be used. A coefficient of friction of 0.50 may be used for native sandy silt soils with an internal angle of friction of 28° and 0.45 native silty clay soils with an internal angle of friction of 26° where encountered.

Lateral Earth Pressures

Ultimate lateral earth pressures for granular backfill borrow used as wall backfill with an internal angle of friction of 28° acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table based on Rankine's equation:

Table 2 - Lateral Earth Pressure Values for Level Backfill

Condition	Lateral Pressure Coefficient	Equivalent Fluid Density (pounds per cubic foot)
Active	0.36	41
At-rest	0.50	58
Passive	2.77	320
Seismic Active	0.60	69

These coefficients and densities assume no buildup of hydrostatic pressures and that buried structures will be backfilled with sand or gravel soils. The force of the water should be added to the presented values if hydrostatic pressures are anticipated.

Walls and structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation, the at-rest condition should be used. These values are unfactored and should be used with an appropriate reduction factor for overturning and sliding.

For seismic analyses, the active earth pressure coefficient provided in the table is based on the Mononobe-Okabe pseudo-static approach and only accounts for the dynamic horizontal thrust produced by ground motion. Hence, the resulting dynamic thrust pressure should be added to the static pressure to determine the total pressure on the wall. The pressure distribution of the dynamic horizontal thrust may be closely approximated as an inverted triangle with stress decreasing with depth and the resultant acting at a distance approximately 0.6 times the loaded height of the structure, measured upward from the bottom of the structure.
The coefficients shown assume a vertical wall face. Hydrostatic and surcharge loadings, if any, should be added. Overcompaction behind walls should be avoided. Resisting passive earth pressure from soils subject to frost or heave, or otherwise above prescribed minimum depths of embedment, should usually be neglected in design.

Stability Analysis

As part of our analysis for retaining walls, the global and pseudo static stability of the retaining walls for the box culvert were evaluated. For the analysis the retaining walls were assumed to have a maximum height of 18 feet. The analysis only included the concrete wall with no footings. The wall was assumed to extend 2 feet below final grade; we consider this model to be conservative. Soil strengths used in our analysis were based on laboratory testing performed by RB & G and GeoStrata. The analysis assumed that the upper 20 feet consist of silt/clay and below 20 feet consist of sand. The analysis was performed once using a silt/clay soil strength based on the lowest Torvane test result of 1250 psf (c of 625 psf) and once based on a direct shear test result of an internal friction angle of 28 degrees and a cohesion of 155 psf. For both analyses the sand below 20 feet was based on a direct shear test performed on the sand (an internal friction angle of 36 degrees and 55 psf cohesion). The analysis was performed with the XSTABL computer program and the bishop's simplified method of slices. For the pseudo static analysis half of the peak ground acceleration with a 2 percent exceedance in 50 years was used in our analysis (0.28g). The results of our analysis indicate static factors of safety of 1.5 and 1.6 and pseudo static factors of safety of 1.1 and 1.0. The results of our analyses are attached in the appendix.

Liquefaction

The liquefaction analysis for the borings was based on the method outlined by Youd and Idriss in the Technical Report NCEER-97-0022. This method uses in-situ soil properties to calculate a CSR and blow count information from the borings to calculate a CRR for a 7.5 magnitude earthquake. Our analysis assumed that the largest earthquake at this site to have a magnitude 7.5 and therefore no scaling factor for the CRR was required. Our analysis assumed groundwater 45 feet below grades at the site and used the peak ground acceleration of 0.55 (2 percent in 50 yrs exceedance). A factor of safety (FS) was then calculated (CRR/CSR). Our analysis indicates that the saturated sand soils are not liquefiable.

Limitations

The recommendations contained in this Memorandum are based on limited field exploration, laboratory testing, and our understanding of the proposed construction. The subsurface data used in the preparation of this report was obtained from the explorations made for this investigation. It is possible that variations in subsurface conditions could exist outside the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to the recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, we should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety.

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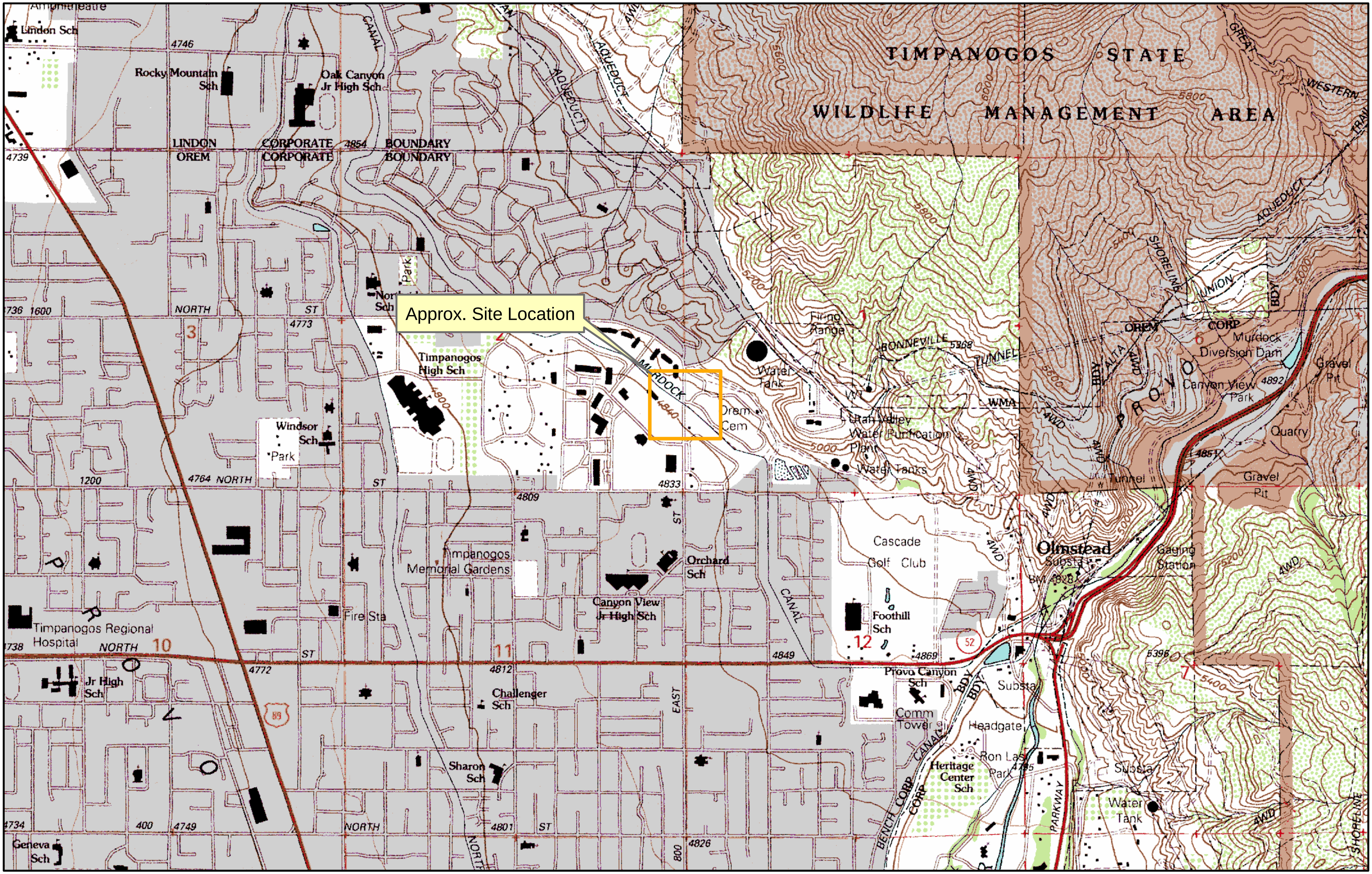

BASE MAP:
USGS Topographic Map obtained from the AGRC.

0 500 1,000 2,000 3,000 4,000 Feet

1:24,000

N

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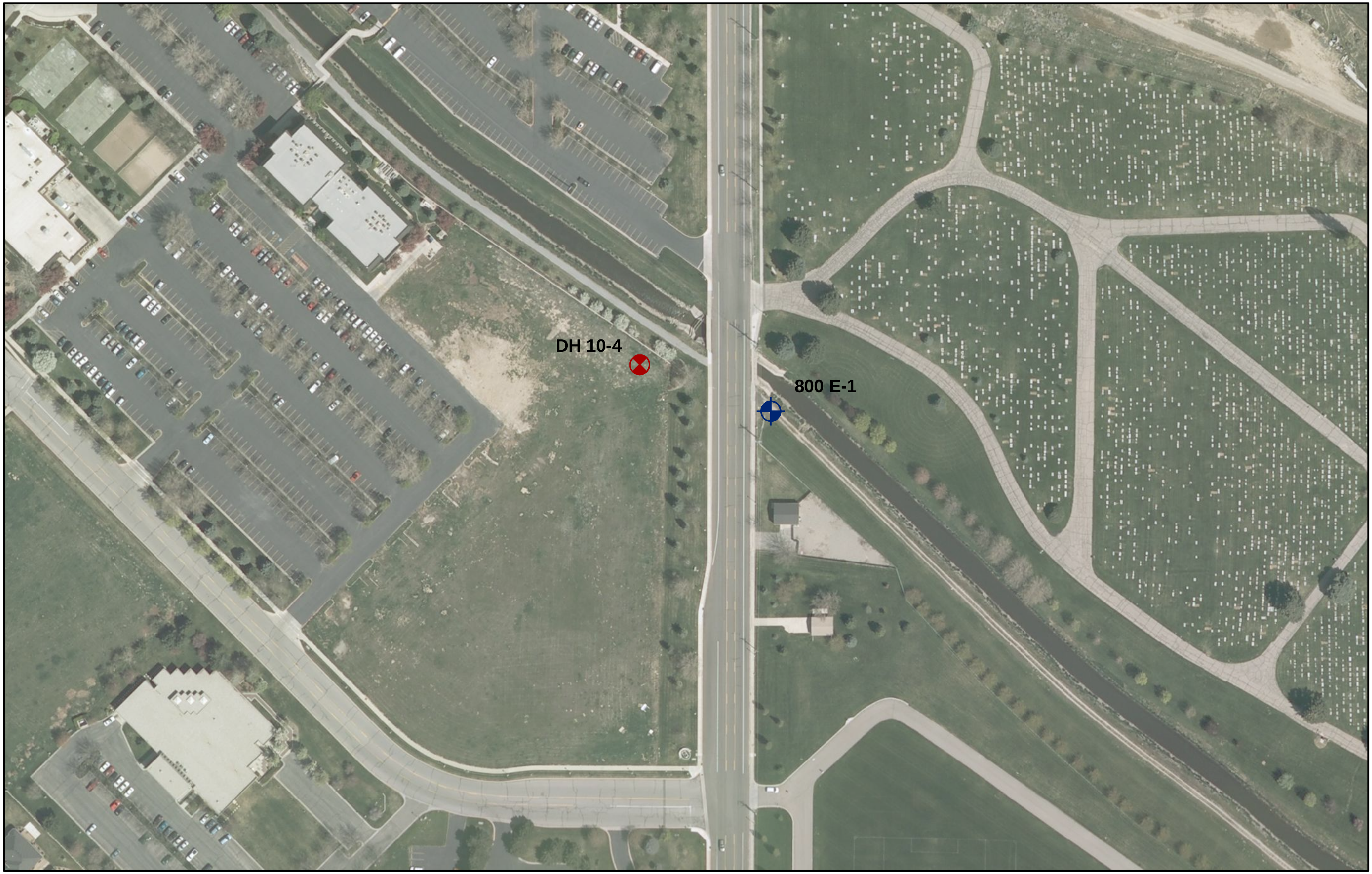
Murdock Canal Trail
Gerber Construction
Lehi, Utah
Project Number: 426-004

Site Vicinity Map

Plate 1



0 35 70 140 210 280
Feet

1:2,000

N

Legend

Boring

GeoStrata Boring Location

RB&G Boring Location

BASE MAP:
2009 1 Foor Orthophotography obtained from the AGRC.
All locations are Approximate.

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MurdockCanal Trail
Gerber Construction
Lehi, Utah
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Site Exploration Map

Plate 2

DATE	
STARTED:	3/11/11
COMPLETED:	3/11/11
BACKFILLED:	3/11/11

Murdock Canal Trail
Gerber Construction
Lehi, Utah
Project Number 426-004

GeoStrata Rep: J. Mattson
Rig Type: CME-55
Boring Type:

BORING NO:
800 E-1

LOCATION
NORTHING EASTING ELEVATION

Depth (feet)	Samples	USCS	MATERIAL DESCRIPTION	N	N*	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0		GM	Fill: Silty GRAVEL - dark brown, medium stiff, moist.							
2.5	2" split spoon	CL-ML	Silty CLAY with gravel - dark brown, medium stiff to stiff, moist.	6	12					
5	3" I.D. sampler			9	10					
7.5	3" I.D. sampler	CL-ML	Sandy Silty CLAY with gravel - medium brown, stiff, moist.	17	18		11.1	52.0	22	5
10	2" split spoon	ML	Sandy SILT - medium brown, medium stiff, moist.	6	9					
12.5	3" I.D. sampler			6	5					
15	Shelby					109.4	16.5	59.7	21	3
20	2" split spoon	SM	Silty SAND - medium brown, loose, moist.	7	9		12.5	39.5	NP	NP
25	3" I.D. sampler	GM	Silty GRAVEL with sand - medium brown, very dense, moist.	50-5	100					

Auger refusal at 25.5 feet on cobbles.

Bottom of boring @ 25.5 Feet

N - OBSERVED UNCORRECTED BLOW COUNT
* N - CORRECTED, N_1 (60) BLOW COUNT

SAMPLE TYPE
- 2" O.D./1.38" I.D. SPLIT SPOON SAMPLER
- 3" O.D./2.42" I.D. SAMPLER
- 3" O.D. THIN-WALLED SHELBY SAMPLER
- GRAB SAMPLE
- Modified California Sampler

NOTES:

WATER LEVEL
- MEASURED - ESTIMATED

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Plate
3

NEW_LOG OF BORING (A) - INPROGRESS PROVO CANAL BORING LOGS.GPJ GEOSTRATA.GDT 5/12/11

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2 - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

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Soil Symbols Description Key

Murdock Canal Trail
Gerber Construction
Provo, UT
Project Number 426-004

Plate 4

Boring No.	Sample Depth (feet)	USCS Soil Classification	Natural Dry Density	Natural Moisture Content (%)	Gradation			Atterberg Limits		Direct Shear	
					Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plasticity Index	Cohesion (psf)	Friction Angle
800 E-1	7.5	CL-ML		11.1%	19.6	28.4	52.0	22	5		
800 E-1	15.0	ML	109.4	16.5%	0.2	40.1	59.7	21	3	155	28
800 E-1	20.0	SM		12.5%	0.9	59.6	39.5	NP	NP	55	36

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Summary Table of Laboratory Testing

Murdock Canal Trail
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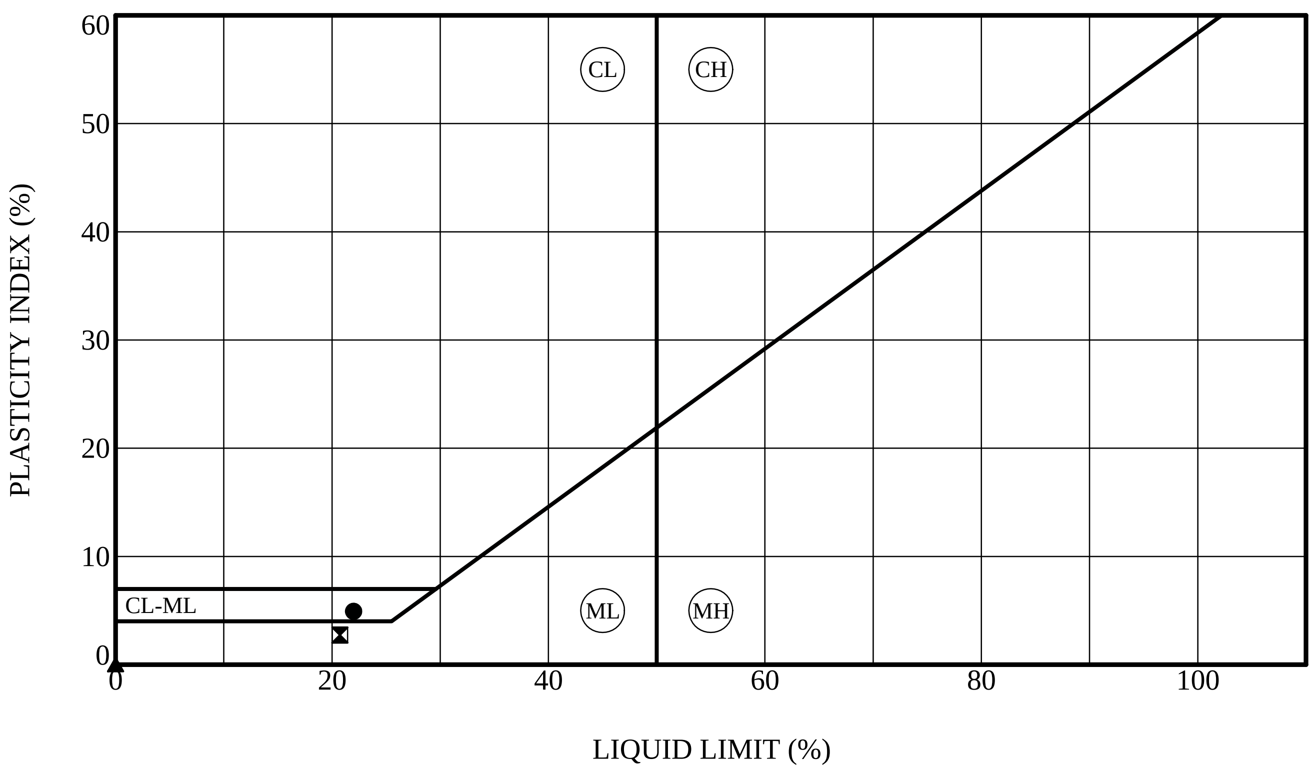
Project Number 426-004

Plate 5



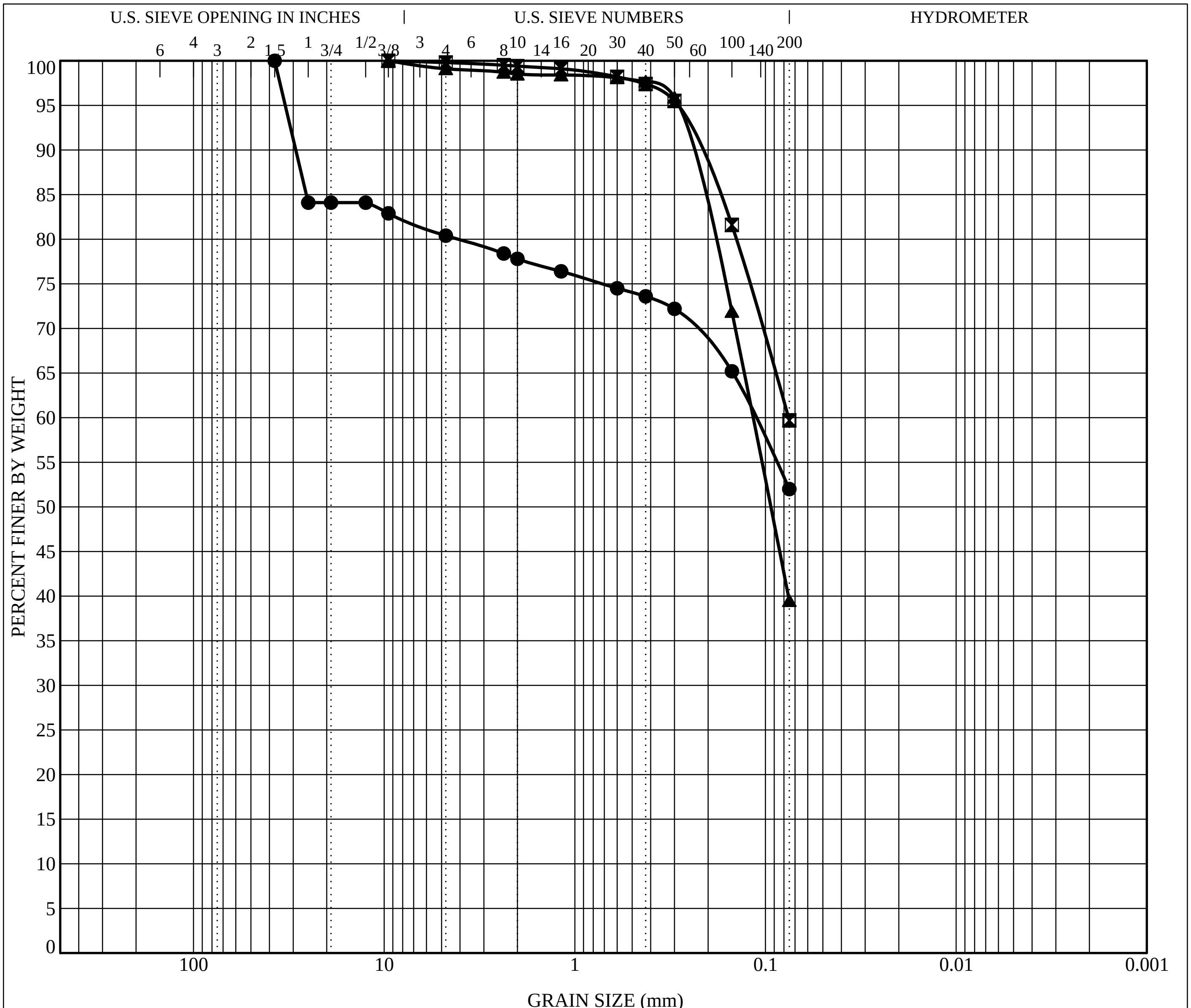
	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Fines (%)	Classification
●	**800 E-1**	**7.5**	**22**	**17**	**5**		**Sandy Silty CLAY w/gravel**
⧗	**800 E-1**	**15.0**	**21**	**18**	**3**		**Sandy SILT**
▲	**800 E-1**	**20.0**	**NP**	**NP**	**NP**		**Silty SAND**

ATTERBERG LIMITS' RESULTS

Murdock Canal Trail
Gerber Construction
Lehi, Utah

Project Number: 426-004

Plate 6



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**800 E-1**	**7.5**	**Sandy Silty CLAY w/gravel**	**22**	**17**	**5**		
⊠	**800 E-1**	**15.0**	**Sandy SILT**	**21**	**18**	**3**		
▲	**800 E-1**	**20.0**	**Silty SAND**	**NP**	**NP**	**NP**		

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**800 E-1**	**7.5**	**37.5**	**0.114**			**19.6**	**28.4**	**52.0**	
⊠	**800 E-1**	**15.0**	**9.5**	**0.076**			**0.2**	**40.1**	**59.7**	
▲	**800 E-1**	**20.0**	**9.5**	**0.116**			**0.9**	**59.6**	**39.5**	

GeoStrata

GRAIN SIZE DISTRIBUTION

Murdock Canal Trail
Gerber Construction
Lehi, Utah

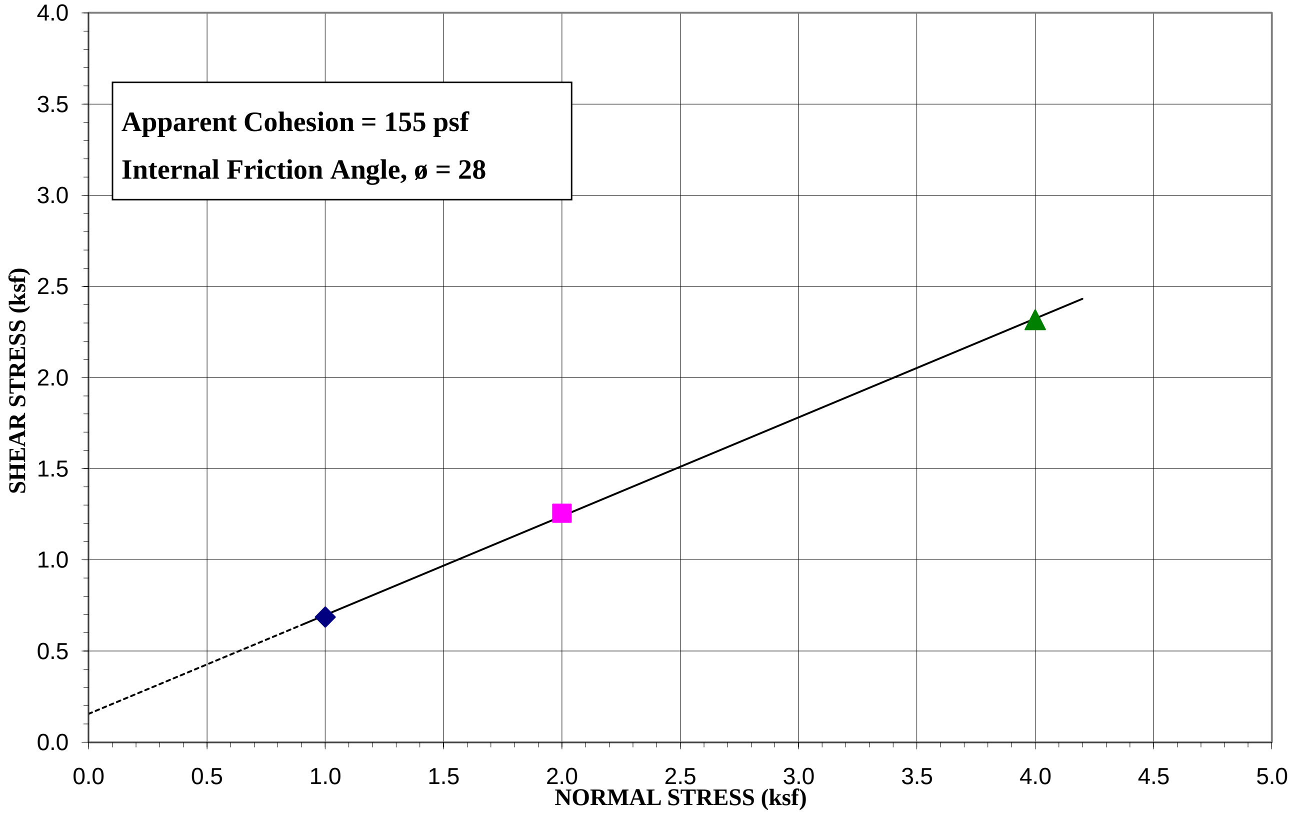
Project Number: 426-004

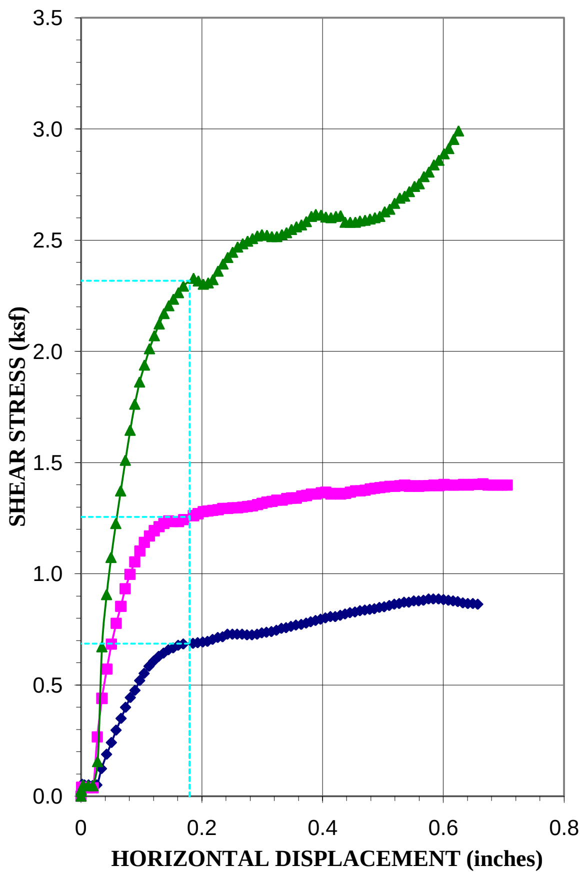
Plate

7

C_GSD PROVO CANAL BORING LOGS.GPJ GEOSTRATA.GDT 5/12/11

DIRECT SHEAR TEST





Source:	**800 E-1**	**Depth:**	**15 ft.**
Type of Test:	Consolidated Drained/Saturated		

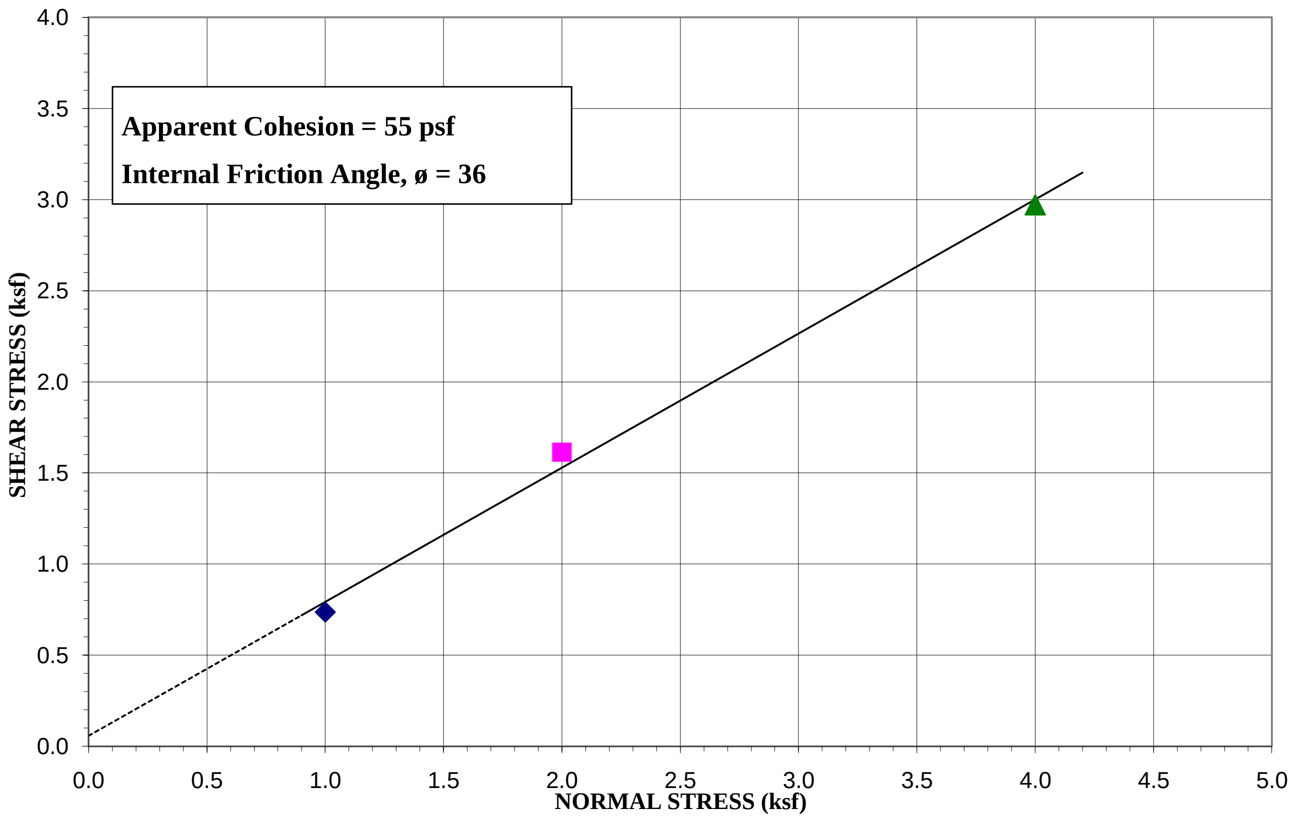
Test No. (Symbol)	**1 (◆)**	**2 (■)**	**3 (▲)**
Sample Type	Undisturbed		
Initial Height, in.	1	1	1
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	110.6	106.9	109.4
Dry Density After, pcf	110.6	106.9	109.4
Moisture % Before	16.7	20.8	13.1
Moisture % After	19.5	17.9	19.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.69	1.26	2.32
Strain Rate	.0046 INCHES/MIN		

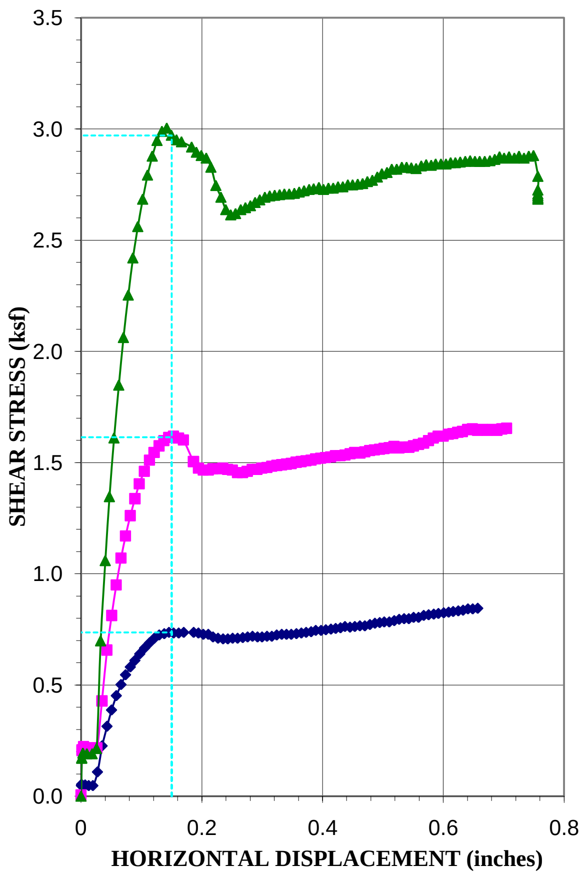
Sample Properties	
Cohesion, psf	155
Friction Angle, ϕ	28
Liquid Limit, %	21
Plasticity Index, %	3
Percent Gravel	0.2
Percent Sand	40.1
Percent Passing No. 200 sieve	59.7
Classification	ML

PROJECT:

PROJECT NO.: 426-004 | GeoStrata | **FIGURE NO.: 8**

DIRECT SHEAR TEST





Source:	**800 E-1**	**Depth:**	**20 ft.**
Type of Test:	Consolidated Drained/Saturated		

Test No. (Symbol)	1 (◆)	2 (■)	3 (▲)
Sample Type	Remolded		
Initial Height, in.	1	1	1
Diameter, in.	2.5	2.5	2.5
Dry Density Before, pcf	108.0	104.9	107.7
Dry Density After, pcf	108.0	104.9	107.7
Moisture % Before	15.3	16.7	16.8
Moisture % After	17.3	19.3	17.8
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.74	1.61	2.97
Strain Rate	.0128 INCHES/MIN		

Sample Properties	
Cohesion, psf	55
Friction Angle, ϕ	36
Liquid Limit, %	NP
Plasticity Index, %	NP
Percent Gravel	0.9
Percent Sand	59.6
Percent Passing No. 200 sieve	39.5
Classification	SM

PROJECT:

PROJECT NO.: 426-004 | GeoStrata | **FIGURE NO.: 9**

DRILL HOLE LOG

BORING NO. 10-4

SHEET 1 OF 2

PROJECT: UDOT - MURDOCK CANAL TRAIL PEDESTRIAN STRUCTURES

CLIENT: J-U-B ENGINEERS

LOCATION: 800 EAST, OREM, UTAH (SEE SITE PLAN)

DRILLING METHOD: 08-CME-55 / N.W. CASING

DRILLER: T. KERN

DEPTH TO WATER - INITIAL: ▽ 45.0' **AFTER 24 HOURS:** ▼ 45.4'

PROJECT NUMBER: 201001.031

DATE STARTED: 7/28/10

DATE COMPLETED: 7/29/10

GROUND ELEVATION: NOT MEASURED

LOGGED BY: J. OLSEN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample: Type	Sample: Rec. (in)	Sample: See Legend	Sample: USCS (AASHTO)	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter.: Liquid Limit	Atter.: Plast. Index	Gradation: Gravel (%)	Gradation: Sand (%)	Gradation: Silt/Clay (%)	Other Tests
				18	3,7,7,(30) *1.30*	SM CL-ML	brown, moist brown, moist, very stiff	SILTY SAND W/GRAVEL (fill) SANDY SILTY CLAY W/GRAVEL								
				11	2,5,4,(19)	CL-ML	dk. brown, moist, stiff	GRAVELLY SILTY CLAY W/SAND								
	5			14	Pushed	CL-ML (*A-4(1)*)	brown, moist	SANDY SILTY CLAY		17.6	21	6	5	40	55	
						GC-GM	brown, very moist	SILTY CLAYEY GRAVEL W/SAND								
				12	7,4,3,(13)	ML	brown, very moist, med. dense	SANDY SILT								
	10			7	Pushed *0.35*	CL-ML (*A-4(1)*)	brown, moist, firm		105.6	13.3	22	6	1	37	62	UC 1751 psf Chem.
				15	0/12",1,(2) *0.15*	CL-ML	brown, moist, soft	SANDY SILTY CLAY occasional gravels								
				17	0/18",(0) *0.10*	CL-ML (*A-4(2)*)	brown, very moist, soft			20.6	23	6	0	28	72	
	15			12	Pushed *0.29*	ML (*A-4(0)*)	brown, very moist, firm	SANDY SILT slightly plastic	98.5	16.6	21	3	1	33	66	UC 1250 psf
						SM	brown, wet									
				4	7,2,3,(6)	SM	brown, wet, loose									
	20			8	0/18",(0)	SM	brown, wet, very loose	SILTY SAND								Chem.
	25			15	1,2,10,(13)	SM (*A-4(0)*)	brown, wet, very loose			19.9		NP	0	57	43	
						GM	brown, wet, very loose	SILTY GRAVEL W/SAND possible cobbles								
	30			5	27,50/3"	GM	brown, wet, very dense									
	35			8	29,50,50/3"	GP-GM (*A-1-a(0)*)	brown, wet, very dense	GRAVEL W/SILT & SAND possible cobbles		3.3		NP	81	14	5	
	40					GP-GM	brown, wet									
				6	12,15,17,(27)	SM	brown, wet, med. dense	SILTY SAND W/GRAVEL								
								SILTY SAND possible clay layers (driller's								

DH_LOGV1 MURDOCKCANALTRAIL.10.11.10.GPJ US EVAL.GDT 10/11/10

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2,(6) ← Blow Count per 6" ← $(N_1)_{60}$ Value; *0.45* ← Torvane (tsf)

UNDISTURBED SAMPLE — PUSHED *0.45* ← Torvane (tsf)

OTHER TESTS

- UC = Unconfined Compression
- CT = Consolidation
- DS = Direct Shear
- UU = Unconsolidated, Undrained Triaxial
- CU = Consolidated, Undrained Triaxial
- HYD = Hydrometer
- SS = Soluble Salt
- DC = Dispersive Clay
- Chem. = pH, Resistivity, Sulfate, Chloride

DRILL HOLE LOG

BORING NO. 10-4

SHEET 2 OF 2

PROJECT: UDOT - MURDOCK CANAL TRAIL PEDESTRIAN STRUCTURES
CLIENT: J-U-B ENGINEERS
LOCATION: 800 EAST, OREM, UTAH (SEE SITE PLAN)
DRILLING METHOD: 08-CME-55 / N.W. CASING
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ∇ 45.0' **AFTER 24 HOURS:** ▼ 45.4'

PROJECT NUMBER: 201001.031
DATE STARTED: 7/28/10
DATE COMPLETED: 7/29/10
GROUND ELEVATION: NOT MEASURED
LOGGED BY: J. OLSEN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample Type	Rec. (in)	See Legend	USCS (AASHTO)	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
								observation)								
				13	17,20,24,(37)	SM (A-2-4(0))	brown, wet, dense	SILTY SAND possible clay layers (driller's observation)		20.2		NP	1	83	16	DS
	50			16	11,12,10,(18)	ML	brown, wet, med. dense	SANDY SILT clay lenses								
	55			16	18,21,12,(26)	SM (A-2-4(0))	brown, wet, med. dense	SILTY SAND clay seams and/or layers to 3" thick		24.1		NP	0	79	21	
	60			15	7,10,12,(17)	SM	brown, wet, med. dense									
	65			18	0.85	CL (A-4(10))	brown w/rust, very moist, stiff	LEAN CLAY		28.2	32	10	0	3	97	
					8,18,28,(35)	SM	brown w/rust, wet, dense	SILTY SAND								
	70			17	21,36,41,(58)	SM	brown, wet, very dense									
	75			16	37,34,27,(45)	SM (A-2-4(0))	brown, wet, dense			19.6		NP	3	77	20	
	80			17	30,28,29,(41)	SM	brown, wet, dense									
								BOH								
	85															

DH_LOGV1 MURDOCKCANALTRAIL.10.11.10.GPJ US EVAL.GDT 10/11/10

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2,(6) Blow Count per 6", $(N_1)_{60}$ Value; 0.45 Torvane (tsf)

UNDISTURBED SAMPLE — PUSHED; 0.45 Torvane (tsf)

OTHER TESTS
UC = Unconfined Compression
CT = Consolidation
DS = Direct Shear
UU = Unconsolidated, Undrained Triaxial
CU = Consolidated, Undrained Triaxial
HYD = Hydrometer
SS = Soluble Salt
DC = Dispersive Clay
Chem. = pH, Resistivity, Sulfate, Chloride

800 E

$$q_u = 1.3 c N_c + q N_q + 0.4 \gamma B N_\gamma$$

$D = 1 ft \quad \gamma = 115 pcf \quad B = 2 ft$

$q = \gamma D = 115 pcf (1 ft) = 115 psf$

$\phi = 28^\circ \quad c = 155 psf$

$N_c \cong 37 \quad N_q \cong 20 \quad N_\gamma \cong 18$

$$q_u = 1.3(155 psf)37 + 115 psf(20) + 0.4(115 pcf) 2 ft (18)$$

$= 7460 psf + 2300 psf + 1660 psf$

$= 11400 psf$

$$q_f = 11400 psf (0.45)$$

$= 5140 psf$

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

800 E

Settlement on Sand (granular Soil)

By Meyerhof Modified by Bowles

$$q_{net(all)} = \frac{N_{1(60)}}{4}\left(\frac{B+1}{B}\right)^2 F_d\, S$$

$N_{1(60)} = 5 \quad B = 2\,ft \quad S = 1 \quad D_f = 1\,ft$

$$F_d = 1 + 0.33\left(\frac{D_f}{B}\right) = 1 + 0.33\left(\frac{1\,ft}{2\,ft}\right) = 1.165$$

$$q_{net(all)} = \frac{5}{4}\left(\frac{2+1}{2}\right)^2 1.165(1)$$

$$= 3.3\ ksf$$

$N_{1(60)} = 5 \quad B = 5\,ft \quad S = 1 \quad D_f = 1\,ft$

$$F_d = 1 + 0.33\left(\frac{1}{5}\right) = 1.066$$

$$q_{net(all)} = \frac{5}{4}\left(\frac{5+1}{5}\right)^2 1.066\,(1)$$

$$= 1.9\ psf$$

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

Active and Passive Earth Pressure Coefficients

Number: 426-004
Project: 800 E

Friction angle	ϕ =	28	(deg)
Wall friction angle	δ =	18.7	(deg)
Wall angle (from vert.)	θ =	0.0	(deg)
Backfill Slope	β =	0.0	(deg)
Seismic Parameters:			
Horiz.Yield Accleration	K_H =	0.293	
Vert. Yield Accleration	K_V =	0	
	ψ =	16.3	

	Active Case		Passive Case	
Method	Ka	Kah	Kp	Kph
Rankine	0.3610	0.3610	2.7698	2.7698
Coulomb	0.3213	0.3044	5.1525	4.8815
Average:	0.3412	0.3327	3.9612	3.8256

Seismic Coefficients:	Active Case		Passive Case	
Method	Ka	Kah	Kp	Kph
Mononobe-Okabe Method	0.602	0.570	3.675	3.481

Rankine Ka = cos b [cos b - (cos2 b -cos2 f)1/2] / [cos b + (cos2 b -cos2 f)1/2]

Kp = [cos b + (cos2 b -cos2 f)1/2] / [cos b - (cos2 b -cos2 f)1/2]

Navfac Ka = {cos f / 1 + [sin f (sin f - cos f tan b)]1/2 }2

Kp = {cos f / 1 - [sin f (sin f + cos f tan b)]1/2 }2

Coulomb:

$$Ka = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\delta + \varphi)\left[1 + \sqrt{\frac{\sin(\delta + \varphi)\sin(\phi - \beta)}{\cos(\delta + \varphi)\cos(\beta - \theta)}}\right]^2}$$

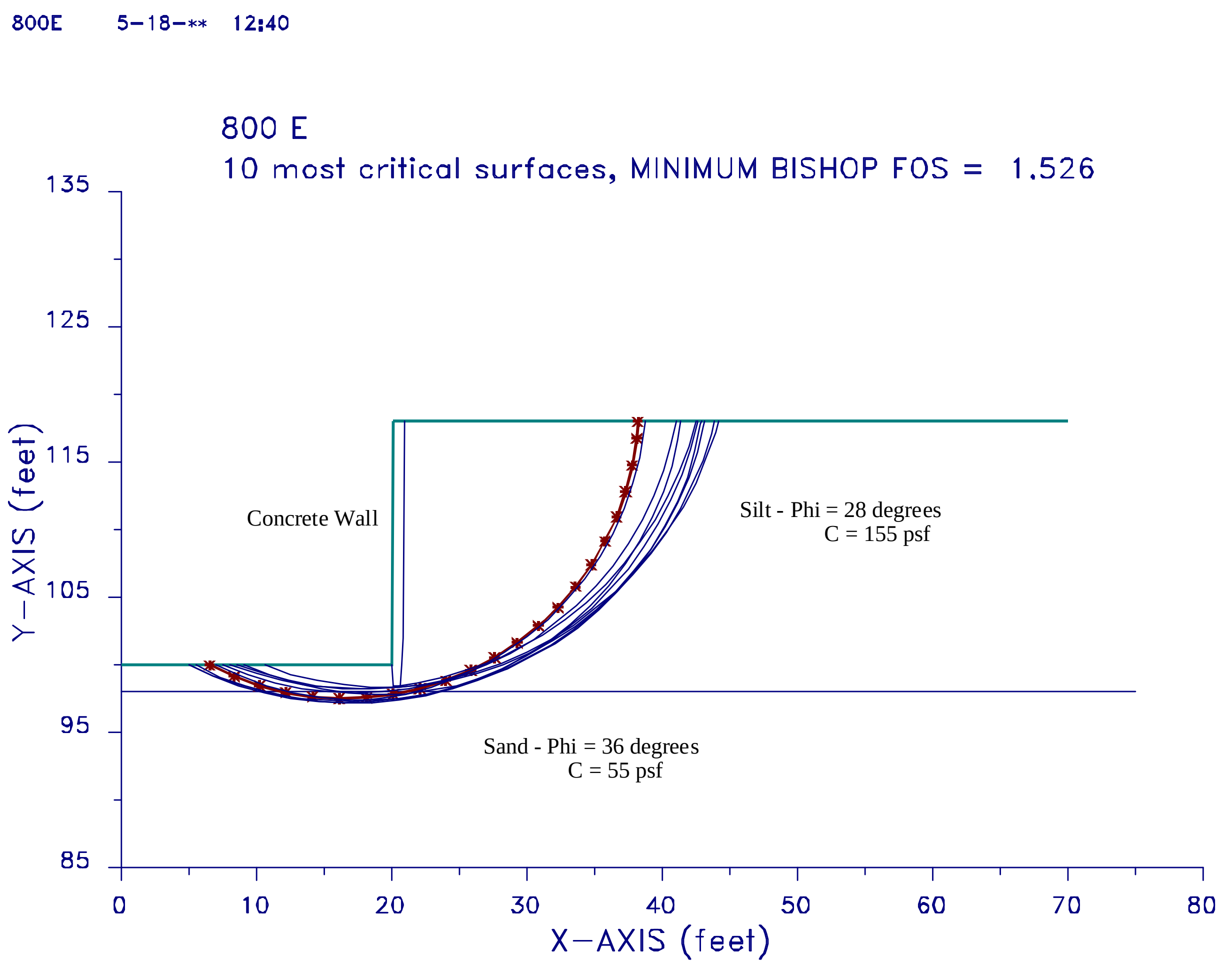
Kp: Apply appropraite sign convention changes

Log Spiral: Interpolation from Caquot, A. and Kerisel J. (1948) Acitve and Passive Earth Pressure Tables

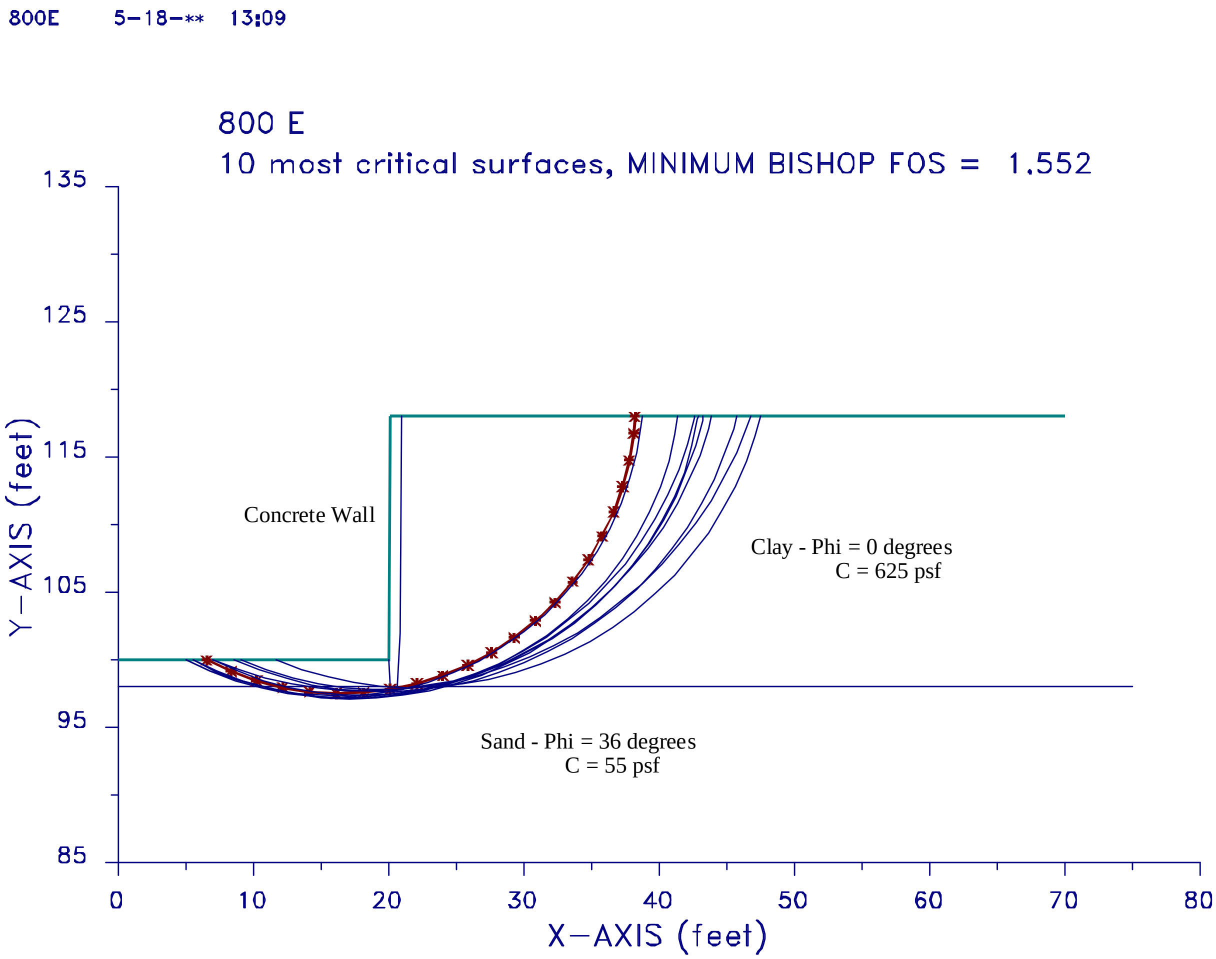
Mononobe-Okabe

$$Kae = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \varphi + \psi)\left[1 + \sqrt{\frac{\sin(\delta + \varphi)\sin(\phi - \beta - \psi)}{\cos(\delta + \varphi + \psi)\cos(\beta - \theta)}}\right]^2}$$

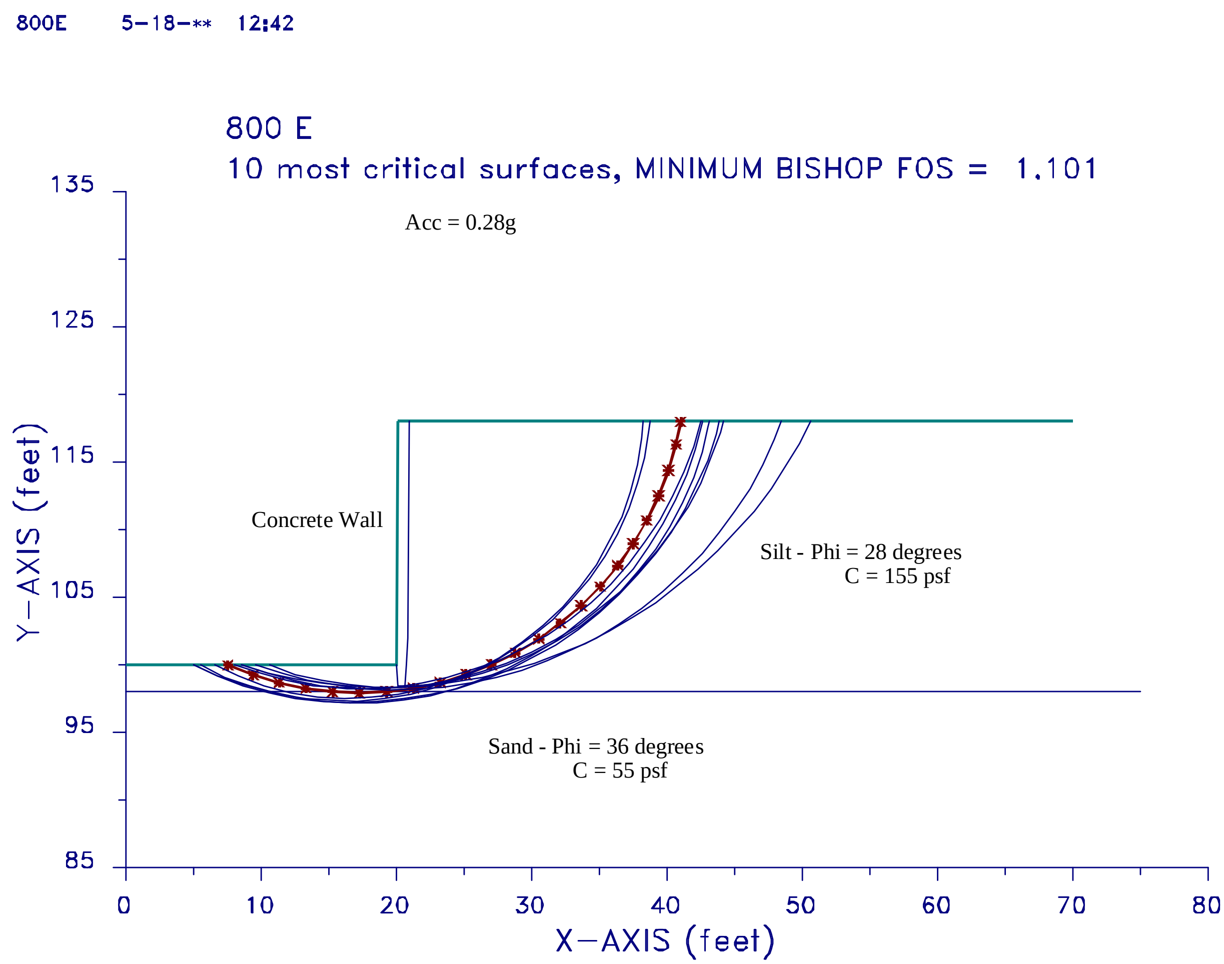
Kpe: Apply appropraite sign convention changes



Retaining Wall Global Stability

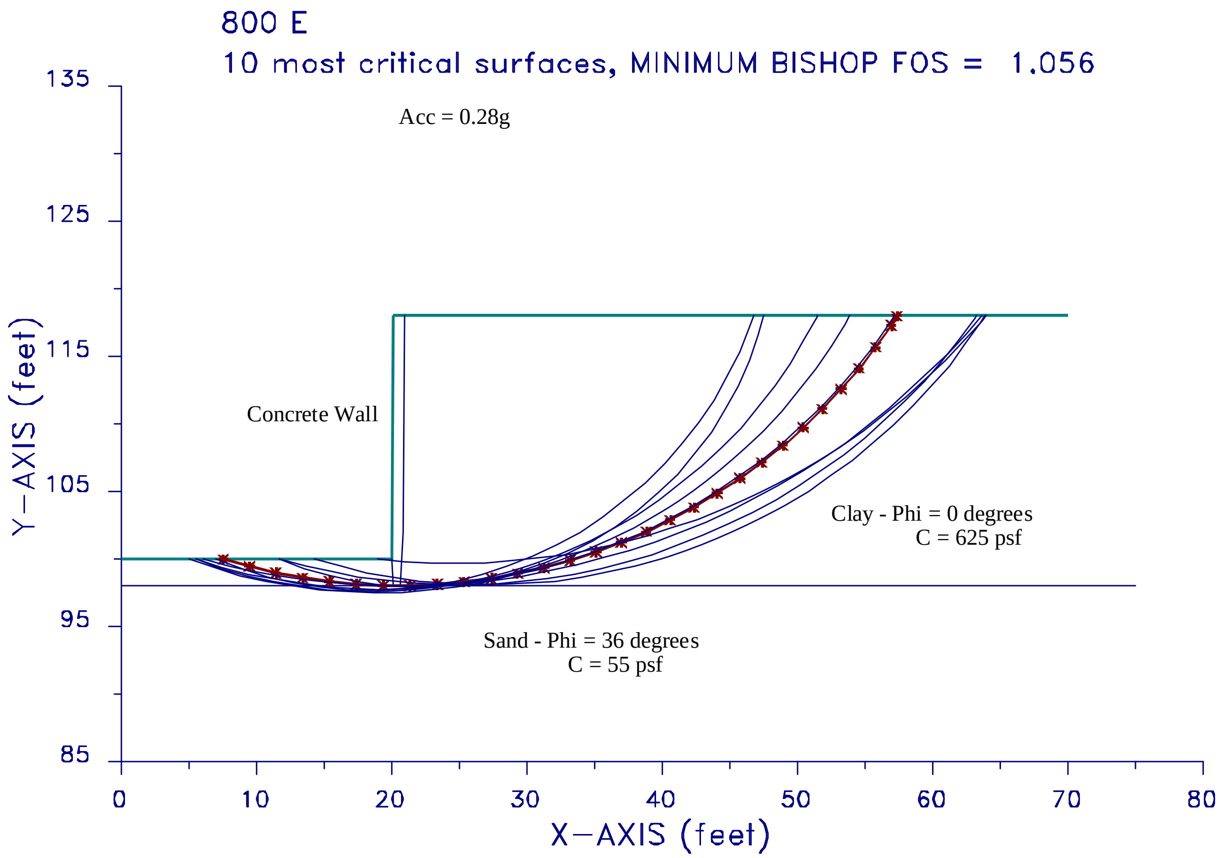


Retaining Wall Global Stability



Retaining Wall Pseudo Static Stability

800E 5-18-** 13:08



Retaining Wall Pseudo Static Stability

LIQUEFACTION POTENTIAL[1]

Job: 800 E
Job No: 426-004
May. 17, 2011

soil layer	layer bottom depth, ft	total unit wt., pcf
1	13.0	115
2	14.0	115
3	40.0	115
4	60.0	115

hammer energy E	Boring Diameter in.	earthquake magnitude M_R	magnitude scaling fact MSF[8]
90	8	7.5	1.00

(E: for safety hammer use 75 w/cathead, 90 w/auto; use 50 for donut w/cathead)

USGS Probabilisitic Horizontal Ground Acceleration, a_{max} (PE probability of exceedence)		Probability of Liquefaction	Liquefaction Hazard Rating
10% PE in 50 yrs (g)		10% or greater in 50 yrs	high
5% PE in 50 yrs (g)		5% to less than 10% in 50 yrs	moderate
2% PE in 50 yrs (g)	0.55	2% to less than 5% in 50 yrs	low
		Less tha 2% in 50 yrs	very low

= input values

test hole no.	USCS class	water depth z_w, ft	sample depth z, ft	blow count n	sampler 3=mcal 1=cal, 2=spt	liners 1=yes, 2=no	percent fines	total stress σ_{vo}, tsf	effective stress σ_{vo}', tsf	reduction coefficient r_d[2]	CSR[3] a = 10% PE in 50	CSR[3] a = 5% PE in 50	CSR[3] a = 0.55 2% PE in 50	spt correction factors[4] stress C_N	energy C_E	boring dia C_B	rod C_R	sampler C_S	sample dia C_D	corrected blow count $(n_1)_{60}$[5]	adjusted for fines $(n1)_{60cs}$[6]	cyclic stress ratio $CRR_{7.5}$[7]	cyclic stress ratio CRR[9]	10% PE 50	Factor of Safety (FS)[10] 5% PE 50	2% PE 50	FS = 1.0 acceleration a_L[11], g	liquefaction hazard rating
1		45.0	45	44	2	2	16	2.588	2.433	0.44	0.00	0.00	0.17	0.63	1.50	1.15	1.00	1.10	1.00	52.5	58.1	-	-	-	-	-	-	**Won't Liquefy**
1		45.0	50	22	2	2	50	2.875	2.565	0.42	0.00	0.00	0.17	0.61	1.50	1.15	1.00	1.10	1.00	25.3	35.4	-	-	-	-	-	-	**Won't Liquefy**
1		45.0	55	33	2	2	21	3.163	2.803	0.41	0.00	0.00	0.17	0.57	1.50	1.15	1.00	1.10	1.00	35.8	42.7	-	-	-	-	-	-	**Won't Liquefy**
1		45.0	60	22	2	2	21	3.450	2.829	0.40	0.00	0.00	0.17	0.57	1.50	1.15	1.00	1.10	1.00	23.7	29.5	0.44	0.44	#DIV/0!	#DIV/0!	2.54	**1.39**	**very low**
1		45.0	65	46	2	2	20	3.450	2.985	0.38	0.00	0.00	0.16	0.55	1.50	1.15	1.00	1.10	1.00	47.8	55.2	-	-	-	-	-	-	**Won't Liquefy**
1		45.0	70	77	2	2	20	3.450	2.985	0.37	0.00	0.00	0.15	0.55	1.50	1.15	1.00	1.10	1.00	79.9	89.9	-	-	-	-	-	-	**Won't Liquefy**
1		45.0	75	61	2	2	20	3.450	2.985	0.36	0.00	0.00	0.15	0.55	1.50	1.15	1.00	1.10	1.00	63.3	72.0	-	-	-	-	-	-	**Won't Liquefy**
1		45.0	80	57	2	2	20	3.450	2.986	0.35	0.00	0.00	0.15	0.55	1.50	1.15	1.00	1.10	1.00	59.2	67.5	-	-	-	-	-	-	**Won't Liquefy**

1 Ref: Youd, T.L., & Idriss, I.M., etc, (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEEF/NSF Workshops on Evaluation of Liquefaction Resistance in Soils", *J. Geotech. and Geoenvir. Engrg*., 127(10), 817-833.
2 Stress reduction coefficient, r_d, *ibid*, equation (3), page 819
3 Cyclic Stress Ratio, CSR, *Ibid*, equation (1), page 818
4 Blow count corrections, C_n = 2.2/(1.2+se/1), *ibid*, equation 10, & Table 2, pg 820 &821
5 Corrected Blow Count, *ibid*, Equation (8), pg 820
6 Correct for fines content, *ibid*, equations 5-7, pg 820
7 $CRR_{7.5}$, *ibid.*, equation (4), page 820
8 f_m = -.037778 M_{R3} + 9.6762 MR2 -8.5015 M_R + 26.271, magnitude correction factor. Curve fit: average of Idriss and Andrus & Stoke for M<7.5, Idriss for values M>7, *Ibid*, pg 827, Table 3 columns 1, 3 and 7
9 CRR = $CRR_{7.5}$ MSF, cyclic stress ratio for liquefaction corrected for magnitude, *ibid*, pg
10 FS = ($CRR_{7.5}$/CSR)
11 a_L = CRR/ [0.65 *(s_t/s_e) * r_d], estimated acceration necessary to induce liquefaction

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 40.322942
Longitude = -111.676820
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.293	PGA - Site Class B
0.2	0.686	Ss - Site Class B
1.0	0.250	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 40.322942
Longitude = -111.676820
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.21, Fa = 1.25, Fv = 1.90
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.366	As - Site Class D
0.2	0.858	SDs - Site Class D
1.0	0.475	SD1 - Site Class D