GEODESIGN INC

April 22, 2015

Lewis & Clark College
Facilities, MSC 76
0615 SW Palatine Hill Road
Portland, OR 97219

Attention: Ms. Denise King

Report of Geotechnical Engineering Services
Outfall 10/Sewer Pipe Crossing
Lewis & Clark College
Portland, Oregon
GeoDesign Project: LCCollege-2-07

15-186839-SD

INTRODUCTION

GeoDesign, Inc. is pleased to present this letter report summarizing our geotechnical engineering services for a proposed improvement. A sewer pipe crosses over an eroded gully downstream of Outfall 10 on the Lewis & Clark College campus, north of the pool house and tennis courts. Figure 1 shows the site relative to existing physical features. A support structure is planned to support the pipe where it comes to eroded gully. Foundations for the pipe crossing structure will consist of shallow foundations, drilled piers, or micropile foundations bearing on native soil on either side of the gully.

SCOPE

The purpose of our services was to evaluate subsurface conditions and provide geotechnical engineering recommendations for use in design of the proposed pipe crossing improvements, including the following scope of services:

- Drilled up to three hand auger borings at the pipe crossing site to a depth of 10.0 feet below ground surface (BGS). The borings were located as follows: one in the channel bottom and one on each side of the gully near the existing pipe.
- Observed and logged soil conditions encountered and collected representative soil samples.
- Completed the following laboratory tests on selected soil samples:
  - Four moisture content determinations
  - Two Atterberg limits determinations

- Provided geotechnical engineering recommendations for use in design and construction of foundation support for the pipe crossing, including allowable design bearing pressure, minimum footing depth and width, and resistance to lateral loads in the form of passive resistance and base friction.
- Provided seismic design parameters in accordance with the procedures outlined in the 2014 State of Oregon Structural Specialty Code.
- Provided this report summarizing our findings and recommendations for design and construction of the proposed improvements.

SITE CONDITIONS

SITE GEOLOGY

Bedrock mapped in the vicinity of the site consists of the Eocene (54 million to 33 million years old) Basalt of Waverly Heights, a sequence of subaerial basaltic lava flows and associated undifferentiated sedimentary rocks[1,2]. Miocene (16 million to 10 million years old) Columbia River Basalts are also mapped in the site vicinity that overlie Basalt of the Waverly Heights. We have observed outcrops of basalt exposed in creeks at the base of the ravines at the Lewis & Clark College campus, but not at the Outfall 10/Sewer Pipe site. At the base and occasionally along the sidewalls of the eroded gully at the site, we observed soil consisting of silt and clay that we interpret to be wind-blown Pleistocene loess (Portland Hills Silt) and colluvium derived from the loess deposits.

SURFACE CONDITIONS

We have conducted surface reconnaissance for several years while observing the general condition of the outfalls on the Lewis & Clark College campus. The Outfall 10/Sewer Pipe site is located north of the tennis courts and pool area of the campus toward the top of a northeast-facing slope at elevations between 360 and 380 feet above mean sea level. The site slopes are forested with mature, second-growth Douglas fir and deciduous trees and a dense understory comprised of ivy, blackberry, and native shrubs.

Stormwater Outfall 10 is located at the head of a north-trending ditch that gradually becomes deeper and more incised until it turns northeastward and joins a larger drainage ravine. The outfall consists of a clay pipe that daylights mid-slope at the head of the ditch. Approximately 100 feet north of the outfall and where the ditch becomes more of an eroded gully, there is a black, steel sewer pipe approximately 8 inches in diameter that crosses over the gully in an east-west direction. The pipe is suspended in air over the gully at a height of approximately 5 feet. An old, decrepit brick foundation is located in the bottom of the gully below and slightly upstream of the sewer pipe. The brick foundation is not in contact with the pipe, and it is not clear what purpose the foundation served, as any remaining structure was apparently removed long ago.

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[1] Beeson, M. H.; Tolan, T. L.; Madin, I. P., 1989, *Geological Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon*: Oregon Department of Geology and Mineral Industries, GMS-59, scale 1:24,000.
[2] Madin, I. P., 1990, *Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon*: Oregon Department of Geology and Mineral Industries, Open-File Report O-90-2, 21p., 8 plates.

SUBSURFACE CONDITIONS

We completed three hand auger explorations (HA-1 through HA-3) on April 9, 2015 to a depth of 10.0 feet BGS. The approximate exploration locations are shown on Figure 2. The details of our field exploration program, exploration logs, and a summary of laboratory testing are provided in the Attachment of this report.

In general, the subsurface conditions encountered by our explorations consist of native clay representing the Portland Hills Silt overlain by less than 2 feet of fill and colluvium at the top of the gully side slopes. The bottom of the gully exposes stiff clay interpreted to represent the Portland Hills Silt that is underlain at depth by residual soil derived from decomposition of basalt bedrock. The following sections present a description of the soil units encountered in our explorations.

Fill

Undocumented fill was encountered at the ground surface in exploration HA-1 to a depth of approximately 1 foot BGS. The fill consists of soft, dark brown silt with some clay and trace brick fragments, possibly derived from construction or demolition of the nearby brick foundation in the gully.

Colluvium

Colluvium was encountered below fill in boring HA-1 and at the ground surface at boring HA-3 for a thickness of 1 foot. The colluvium consists of soft to medium stiff, brown silt.

Native Clay

Native clay was encountered below the colluvium in borings HA-1 and HA-3 to the maximum depth explored (10 feet BGS) and at the bottom of the gully in boring HA-2 to an approximate depth of 7 feet BGS. The clay contains some silt and minor amounts of sand. Field observations indicate the native clay is medium stiff to stiff. Laboratory testing on selected samples of the clay resulted in moisture contents ranging from 32 to 36 percent. Atterberg limits testing of a sample of the native clay indicated it is of low plasticity. We interpret the native clay to represent the Portland Hills Silt.

Residual Soil and Basalt Bedrock

Boring HA-2 encountered stiff, gray clay below the native clay to the maximum depth explored, 10 feet BGS. The lower clay unit likely represents residual soil derived from decomposition of basalt bedrock. We did not observe relict rock texture but did observe that the soil had a less homogeneous structure than the overlying native clay. Laboratory testing on selected samples of the clay resulted in moisture contents ranging from 32 to 43 percent. Atterberg limits testing of a sample of the residual clay indicated it is of high plasticity.

Intact basalt bedrock was not encountered in our hand auger explorations at the site. Based on geologic mapping and our experience in the area, intact basalt bedrock can be encountered at shallow depths below the native clay.

Groundwater
Groundwater was observed in boring HA-1 at 2 feet BGS as an isolated seep. Zones of perched water may be encountered within a few feet of the ground surface during the wet season or extended periods of precipitation. The depth to groundwater is expected to fluctuate in response to seasonal changes, changes in surface topography, and other factors not observed in the site vicinity.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our geotechnical engineering analyses, it is our opinion that the project can be constructed as planned, provided the recommendations in this report are incorporated into design and construction. The following factors will have an impact on design and construction of the proposed improvements:

- The proposed structure can be established on shallow, isolated foundations bearing on the native clay. The footings should be established at a depth so there is at least 10 horizontal feet between the bottom of the footing and the face of adjacent slopes.
- We recommend that trees and other vegetation remain on the existing slopes where they would not otherwise interfere with the proposed structure. If vegetation has to be removed, bare soil should be protected against erosion until permanent vegetation can be re-established.
- We did not encounter basalt bedrock during our explorations to depths of up to 10 feet BGS; therefore, bedrock excavation is not expected to be necessary for cuts up to these depths.
- The on-site soil will provide inadequate support for construction equipment during periods of persistent rainfall.

Our specific recommendations for site development are presented in the following sections of this report. These recommendations should be incorporated into the design and implemented during construction.

FOUNDATION DESIGN RECOMMENDATIONS

Based on results of our subsurface exploration and experience in the project area, the proposed structure can be established on shallow, isolated foundations bearing on the native clay. We recommend that isolated spread footings have minimum widths of 18 inches. The footings should be established at a depth so there is at least 10 horizontal feet between the bottom of the footing perimeter and the faces of the gully side slopes and the north-facing hillside. Based on the site slope gradients observed, the resulting footing depths will be greater than the anticipated frost depth. The following sections present specific information for design and construction of shallow foundations.

BEARING CAPACITY
We recommend that conventional spread footings founded on the native soil be proportioned using a maximum allowable bearing pressure of 2,500 pounds per square foot. If soft material is encountered at the footing subgrade elevations, we recommend the material be removed and the footings extended to firm soil. This bearing pressure applies to the total of dead and long-

term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes.

TOTAL FOUNDATION SETTLEMENT

Foundations designed and constructed as recommended in this report are expected to experience less than 1 inch of settlement. Differential settlement between adjacent footings should be less than ½ inch. The majority of settlement should occur during construction as loads are applied.

LATERAL RESISTANCE

Lateral loads can be resisted by passive earth pressure on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an allowable equivalent unit weight of 250 pounds per cubic foot if footings are confined by undisturbed native soil or structural fill. We recommend a friction coefficient of 0.35 for foundations placed on the native soil and 0.40 if footings are placed on at least 4 inches of compacted crushed rock. The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total. The lateral resistance values include a safety factor of 1.5, which is appropriate when designing for dead loads plus frequently applied live loads.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and that static groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the foundation area is covered with pavement or is inside the building.

SUBGRADE PREPARATION

We recommend that any loose or disturbed soil be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, it should be removed before placing reinforcing steel or concrete.

We recommend that an experienced geotechnical engineer observe all foundation excavations before placing reinforcing steel in order to determine if bearing surfaces have been adequately prepared and that the soil conditions are consistent with those revealed by our explorations.

SEISMIC DESIGN CRITERIA

We understand that the development will be designed and constructed in accordance with the 2014 State of Oregon Structural Specialty Code. Base shear forces can be computed using the parameters provided in Table 1.

Table 1. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
Maximum Considered Earthquake Spectral Acceleration, S	S_s = 0.989 g	S_1 = 0.424 g
Site Class	D	
Site Coefficient, F	F_a = 1.105	F_v = 1.576
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.092 g	S_{M1} = 0.668 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.728 g	S_{D1} = 0.445 g
Design Peak Ground Acceleration, S_{aPGA}	0.291 g	

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking. The site soil we encountered in our borings is not susceptible to liquefaction under design levels of ground shaking.

OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractors' activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that a geotechnical engineer be retained to observe excavation, fill placement, and subgrade preparation.

Subsurface soil and groundwater conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

LIMITATIONS

We have prepared this report for use by Lewis & Clark College, Harper Houf Peterson Righellis Inc., and other members of the design and construction team. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Exploration observations indicate soil conditions at specific locations and to the depths explored. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the buildings or retaining walls, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty, express or implied, should be understood.

◆ ◆ ◆

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Erick J. Staley, C.E.G.
Associate Engineering Geologist

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
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EXPIRES: 6/30/16

CERTIFIED
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ERICK J. STALEY
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ENGINEERING GEOLOGIST
Expires 05/31/2015

cc: Mr. Scott Banker, Harper Houf Peterson Righellis Inc. (via email only)

EJS:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: LCCollege-2-07-042215-geolr.docx

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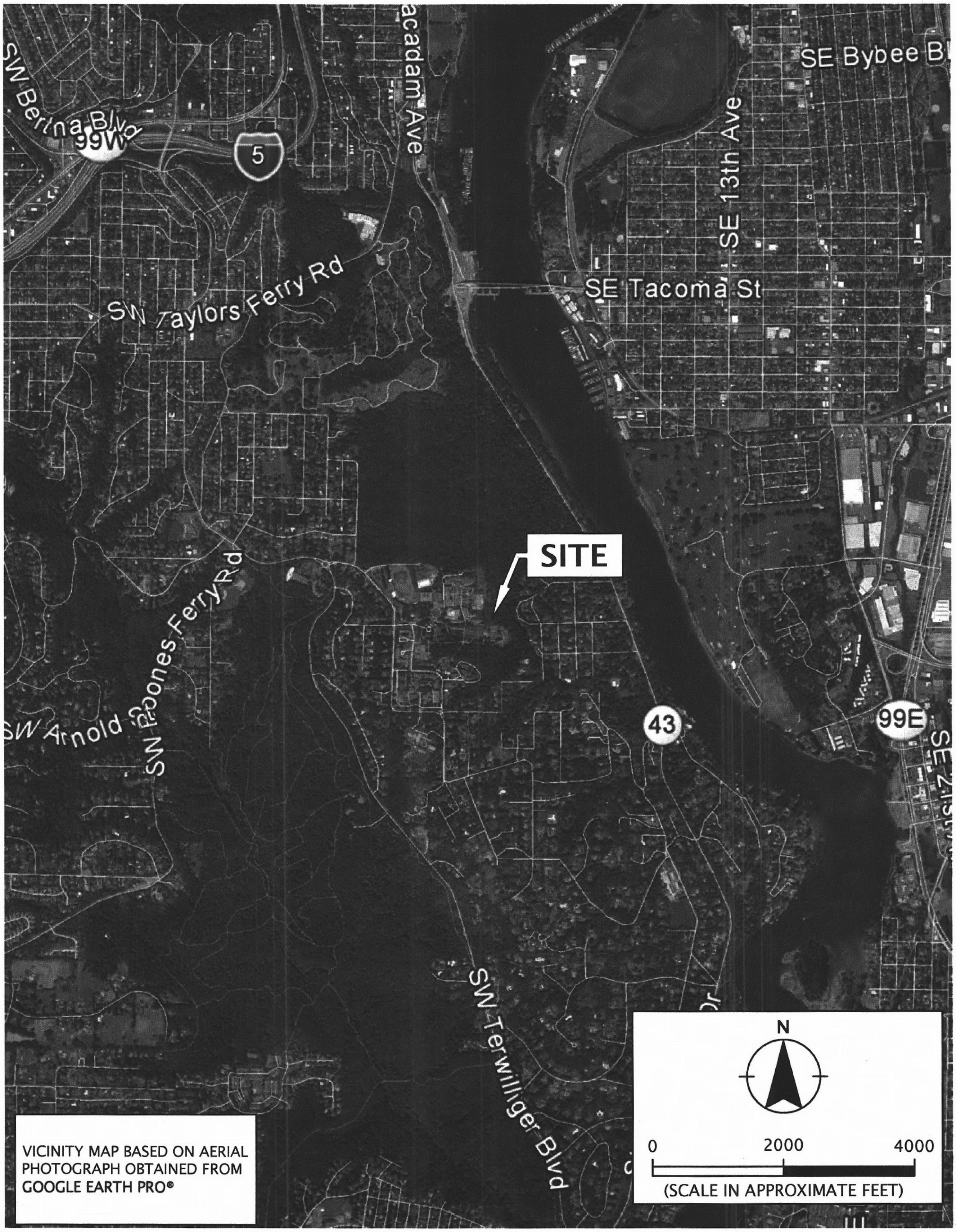
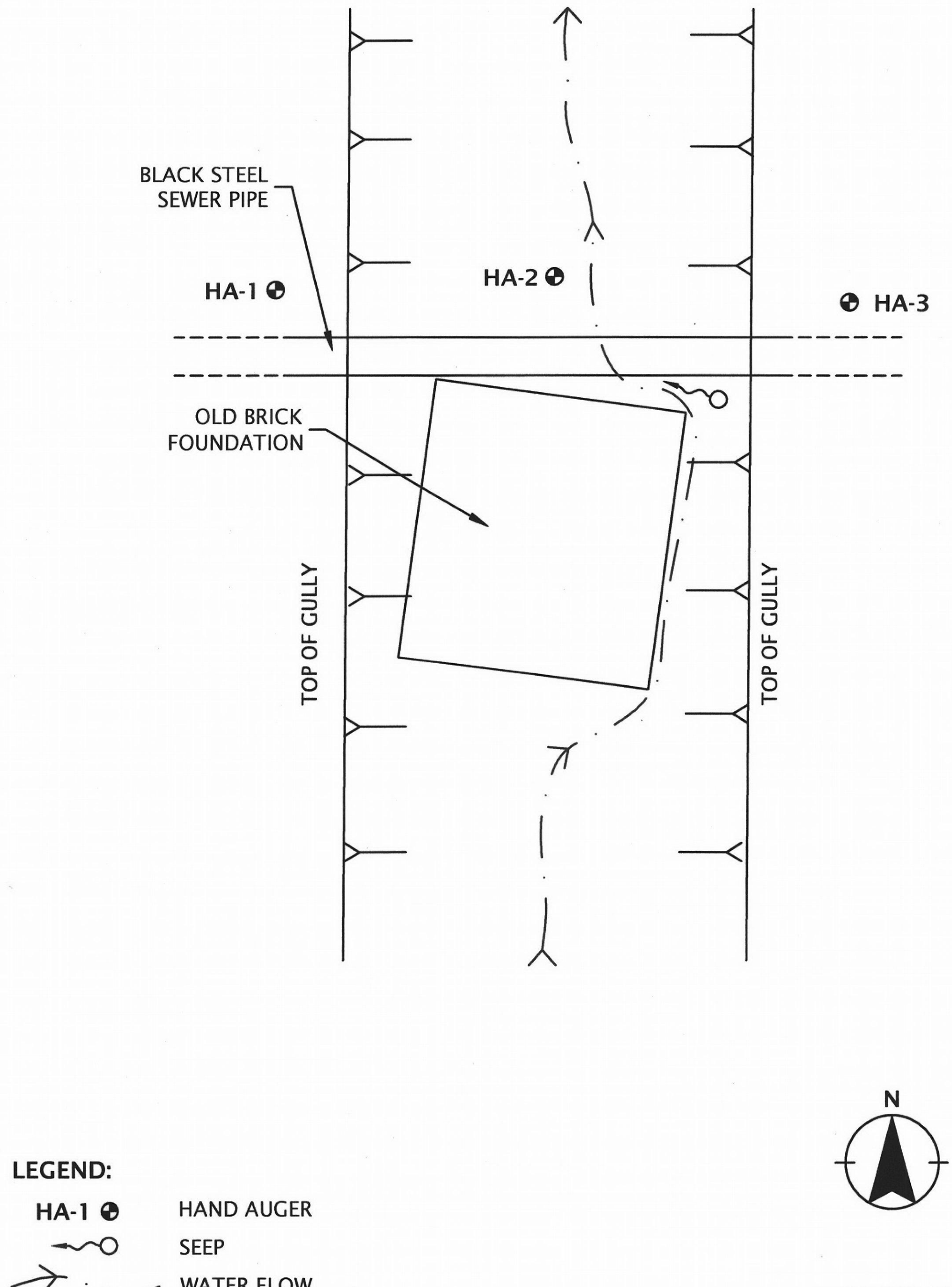
File Name: J:\E-L\LCCollege\LCCollege-2\LCCollege-2-07\Figures\CAD\LCCollege-2-07-VM01.dwg | Layout: FIGURE 1



GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	LCCOLLEGE-2-07	**VICINITY MAP**	
	APRIL 2015	OUTFALL 10 PIPE CROSSING AND GULLY REPAIR PORTLAND, OR	**FIGURE 1**



(NOT TO SCALE)

SITE PLAN BASED ON OBSERVATIONS AND FIELD MEASUREMENTS BY GEODESIGN STAFF

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LCCOLLEGE-2-07	**SITE PLAN**	
APRIL 2015	OUTFALL 10 PIPE CROSSING AND GULLY REPAIR PORTLAND, OR	**FIGURE 2**

ATTACHMENT

FIELD EXPLORATIONS

The proposed gully crossing at the Outfall 10/Sewer Pipe site was explored by drilling three hand auger borings (HA-1 through HA-3) to a maximum depth of 10.0 feet BGS on April 9, 2015. Figure 2 shows the approximate exploration locations. The exploration locations were located by measuring from the existing sewer pipe and were staked in the field using wooden lath for later survey by others. Latitudinal and longitudinal coordinates were collected using a smartphone-grade GPS in dense tree cover and should be considered approximate.

The explorations were conducted by our field geologist. We obtained select samples of the soil encountered at representative intervals. The soil samples were classified in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are included in this attachment. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Detailed exploration logs are presented in this attachment.

LABORATORY TESTING

CLASSIFICATION AND MOISTURE CONTENT

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are presented on the exploration logs if those classifications differed from the field classifications.

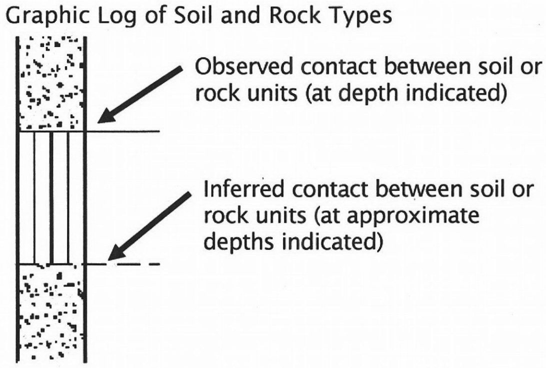
We tested the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The moisture contents are presented in this attachment.

ATTERBERG LIMITS

The plastic limit and liquid limit (Atterberg limits) of selected soil samples were determined in accordance with ASTM D 4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil. The test results are presented in this attachment.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOILS

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVELS WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOILS			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soils	Silt and Clay In: Coarse-Grained Soils	Percent	Sand and Gravel In: Fine-Grained Soils	Sand and Gravel In: Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % (0 – 50 – 100)	COMMENTS

HA-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Soft, dark brown SILT (ML), some clay, trace brick fragments; moist - **FILL.**				
1.0	Medium stiff, brown with orange mottled SILT with sand (ML), minor gravel; moist (colluvium).	1.0		☒	
2.0–2.5	Medium stiff, brown with orange mottled CLAY (CL), some silt, trace to minor sand; moist, sand is fine (Portland Hills Silt).	2.0		☒ ●	Groundwater seep at 2.0 feet.
5.0	stiff at 5.0 feet				
7.5	gray at 7.0 feet			☒	
10.0	Exploration completed at a depth of 10.0 feet. Latitude: 45.45077 Longitude: -122.66724 (determined from iPhone 6 GPS)	10.0		☒	Surface elevation was not measured at the time of exploration.

HA-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, gray CLAY (CL), some silt, trace sand; moist, sand is fine (Portland Hills Silt).			☒ ●	
5.0	stiff at 6.0 feet			☒	
7.5	Stiff, gray with brown mottled CLAY (CH), some silt; moist (suspect residual soil from Boring Lavas).	7.0		☒ ●	Change in soil structure - crumbly and less homogeneous/massive at 7.0 feet.
10.0	Exploration completed at a depth of 10.0 feet. Latitude: 45.45082 Longitude: -122.66731 (determined from iPhone 6 GPS)	10.0	ATT	☒ ●	LL = 60% PL = 25% Surface elevation was not measured at the time of exploration.

DRILLED BY: GeoDesign, Inc. staff LOGGED BY: CR COMPLETED: 04/09/15

BORING METHOD: hand-auger (see document text) BORING BIT DIAMETER: 4-inch

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HAND AUGER

OUTFALL 10 PIPE CROSSING AND GULLY REPAIR
PORTLAND, OR

FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % (0 50 100)	COMMENTS
HA-3							
0.0		Soft, dark brown SILT (ML); moist (colluvium).					
		Medium stiff, brown with gray mottled CLAY (CL), some silt, trace sand; moist, sand is fine (Portland Hills Silt).	1.0		☒	●	
2.5							
5.0		minor sand at 4.5 feet			☒		
		stiff, light blue-gray at 6.5 feet		ATT	☒	●	LL = 38% PL = 23%
7.5					☒		
10.0		Exploration completed at a depth of 10.0 feet. Latitude: 45.45078 Longitude: -122.66720 (determined from iPhone 6 GPS)	10.0				Surface elevation was not measured at the time of exploration.
12.5							

DRILLED BY: GeoDesign, Inc. staff LOGGED BY: CR COMPLETED: 04/09/15

BORING METHOD: hand-auger (see document text) BORING BIT DIAMETER: 4-inch

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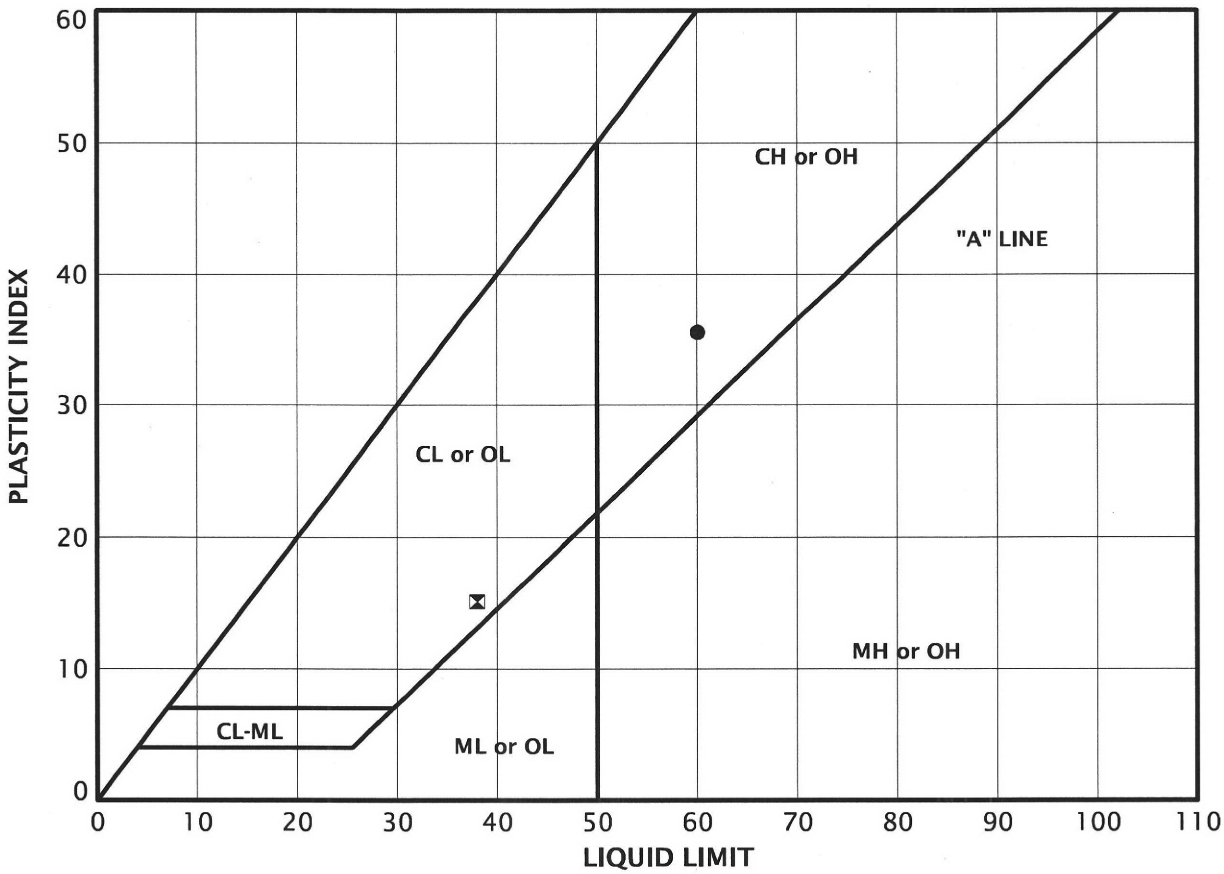
	APRIL 2015	OUTFALL 10 PIPE CROSSING AND GULLY REPAIR PORTLAND, OR	**FIGURE A-2**



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	HA-2	9.5	43	60	25	35
⧗	HA-3	7.0	32	38	23	15

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ATTERBERG LIMITS TEST RESULTS

OUTFALL 10 PIPE CROSSING AND GULLY REPAIR
PORTLAND, OR

FIGURE A-3

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
HA-1	3.5		36							
HA-2	1.0		36							
HA-2	7.5		32							
HA-2	9.5		43					60	25	35
HA-3	1.5		34							
HA-3	7.0		32					38	23	15

LAB SUMMARY LCCOLLEGE-2-07-HA1_3.GPJ GEODESIGN.GDT PRINT DATE: 4/17/15:KT

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APRIL 2015

SUMMARY OF LABORATORY DATA

OUTFALL 10 PIPE CROSSING AND GULLY REPAIR
PORTLAND, OR

FIGURE A-4

GEODESIGN INC

Addendum Transmittal

To:	Denise King	From:	Brett Shipton
Company:	Lewis & Clark College	Date:	July 8, 2015
Address:	Facilities, MSC 76 0615 SW Palatine Hill Road Portland, OR 97219		

cc:	Scott Banker, Harper Houf Peterson Righellis, Inc. (via email only)

GDI Project:	LCCollege-2-07
RE:	Lewis & Clark College

Original File Name	Date	Document Title
LCCollege-2-07-042215-geolr	4/22/15	Report of Geotechnical Engineering Services; Outfall 10/Sewer Pipe Crossing; Lewis & Clark College; Portland, Oregon

Addendum Number	Date	Description
1	7/8/15	Helical Pier Recommendations (attached)

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Attachment
One copy submitted (via email only)
Document ID: LCCollege-2-07-070815-geoat-1.docx

15-188395-SI

GEODESIGN INC

July 8, 2015

Lewis & Clark College
Facilities, MSC 76
0615 SW Palatine Hill Road
Portland, OR 97219

Attention: Ms. Denise King

Addendum 1
Helical Pier Recommendations
Outfall 10/Sewer Pipe Crossing
Lewis & Clark College
Portland, Oregon
GeoDesign Project: LCCollege-2-07

INTRODUCTION

This addendum supplements our April 22, 2015 geotechnical engineering report for the sewer pipe crossing over an eroded gully downstream of Outfall 10 on the Lewis & Clark College campus. A structure is planned to support the pipe where it crosses the eroded gully. We understand that helical piers will support the structure. Both vertical loads and lateral loads will be supported by the anchors as follows:

- Maximum allowable vertical load per pier 5 kips
- Maximum allowable lateral load per pier 4 kips

HELICAL PIER RECOMMENDATIONS

Helical pier lead section consists of one of more circular steel helices attached to a square bar. This type of system is proprietary and designed by a specialty contractor, who will select the pier size and determine the capacity. The helices generally range between 6 and 14 inches in diameter. If the lead section is provided with more than one helix, they are generally spaced along the square rod at a minimum of 3 helix diameters to avoid influencing the bearing capacities of adjacent helices. The lead section is generally attached to an extension bar with a thread bar adaptor, which in turn is attached to a thread bar.

The piers should be installed to a length of at least 10 feet. At this length we anticipate that an allowable capacity of 5 kips is achievable, assuming a safety factor of 2.0. The anchors should be installed continuously from the ground surface using equipment capable of measuring installation torque, which can be correlated to ultimate capacity. At least one anchor that supports vertical loads and one anchor that supports lateral loads should be proof-tested to 200 percent of the design load. The test pier that supports lateral loads should be tested in tension in accordance with ASTM D 1143. The test pier that supports vertical loads can, at the contractor's option, be tested in compression or tension accordance with ASTM D 1143 or ASTM D 3689, respectively.

Helical piers installed in accordance with the recommendations in the addendum and the April 22, 2015 geotechnical engineering report should not have a significant impact on slope stability at the site.

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We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

cc: Mr. Scott Banker, Harper Houf Peterson Righellis Inc. (via email only)

BAS:kt
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Document ID: LCCollege-2-07-070815-geoa-1.docx