GEOTECHNICAL INVESTIGATION

UVU WEST CAMPUS

SMALL BUSINESS DEVELOPMENT BUILDING

Orem, Utah

Prepared for: DFCM

May 2009

RB&G ENGINEERING, INC.

200901-023

RB&G
ENGINEERING, INC.

May 27, 2009

Mike Ambre
DFCM
4110 State Office Building
Salt Lake City, UT 84114

Subject: UVU West Campus Small Business Development Building
Geotechnical Investigation

Gentlemen:

A Geotechnical Investigation has been completed for the proposed Small Business Development Building to be located on the UVU West Campus in Orem, Utah. The results of the study are summarized in the report transmitted herewith.

We appreciate the opportunity of providing this service for you. If there are any questions relating to the information contained herein, please call.

Sincerely,

RB&G ENGINEERING, INC.

Bradford E. Price

REGISTERED PROFESSIONAL ENGINEER
NO.162291
BRADFORD E.
PRICE
5/27/09
STATE OF UTAH

Bradford E. Price, P.E.

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Geotechnical Investigation

UVU West Campus

Small Business Development Building

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UVU WEST CAMPUS
SMALL BUSINESS
DEVELOPMENT BUILDING
Orem, Utah

Geotechnical Investigation

INTRODUCTION

This report outlines the results of a geotechnical investigation performed for the Small Business Development Building to be located on the Utah Valley University (UVU) West Campus at about 987 South Geneva Road in Orem, Utah, at the location shown on the vicinity map in Figure 1. The purpose of this investigation was to determine the characteristics of the subsurface material throughout the site so that satisfactory substructures can be designed to support the proposed facilities.

The information contained in the report is discussed under the following headings: (1) Geological and Existing Site Conditions, (2) Field and Laboratory Testing Procedures, (3) Subsurface Soil and Water Conditions, (4) Site Preparation and Compacted Fill Requirements, and (5) Foundation Considerations and Recommendations.

I. GEOLOGICAL AND EXISTING SITE CONDITIONS

The natural surface materials in this general area have been mapped as Lacustrine silt and clay deposits of the Provo (regressive) phase of the Bonneville lake cycle (upper Pleistocene).

The Wasatch Fault Zone is located approximately 5 miles east of the site. Faults have been identified in Utah Lake, approximately 3 miles west of the site. Utah County Natural hazards maps identify this area as having moderate liquefaction potential.

The topography throughout the site is relatively flat with a slight slope down to the west.
The vegetative cover consists of small evergreen trees and lawn grass, as shown on the Site Plan in Figure 2.

The one/two-story brick structure immediately east of the site appears to be supported on spread footings. Foundations for structure appear to be performing in a satisfactory manner, in that no significant cracking was observed in foundation walls.

No major water conveyance facilities or other water bodies exist in the immediate vicinity, which would influence the groundwater level at this site. Other than the information provided above, no conditions appear to exist at this site which would adversely affect foundation performance.

II. FIELD AND LABORATORY TESTING PROCEDURES

The subsurface investigation was performed using a CME 55 rotary drill rig with a tri-cone rock bit and NW casing to advance the boring and water as the drilling fluid. During the subsurface investigation, sampling was performed at one- to three-foot intervals in the upper 15 feet of the soil profile and at five-foot intervals thereafter. Both disturbed and undisturbed samples were obtained during the field investigations. Disturbed samples were obtained by driving a 2-inch split spoon sampling tube through a distance of 18 inches using a 140-pound weight dropped from a distance of 30 inches. The number of blows to drive the sampling spoon through each 6 inches of penetration is shown on the boring logs. The sum of the last two blow counts, which represents the number of blows to drive the sampling spoon through 12 inches, is defined as the standard penetration value. The standard penetration value, corrected for overburden and hammer energy, provides a good indication of the in-place density of sandy material; however, it only provides an indication of the relative stiffness of the cohesive material, since the penetration resistance of materials of this type is a function of the moisture content.

Undisturbed samples were obtained at select locations by pushing a thin-walled sampling tube into the subsurface material using the hydraulic pressure on the drill rig. The location at which the undisturbed samples were obtained is shown on the boring logs.

Miniature vane shear tests, which provide an indication of the undrained shearing strength of cohesive materials, were performed on samples of the clay soil during the field investigations. The results of these tests are shown on the boring logs as the torvane value in tsf.

Each sample obtained in the field was classified in the laboratory according to the Modified Unified Soil Classification System. The symbol designating the soil type according to this system, is presented on the boring logs. A description of the Modified Unified Soil Classification

System is presented in the appendix, and the meaning of the various symbols, shown on the logs, can be obtained from this figure.

Laboratory tests performed during this investigation to define the characteristics of the subsurface material throughout the proposed site included in-place dry unit weight, natural moisture content, Atterberg Limits, mechanical analyses, unconfined compressive strength, consolidation, pH, resistivity, sulfate and chloride tests. Testing was performed following procedures outlined in the American Society for Testing and Materials (ASTM) standards.

III. SUBSURFACE SOIL AND WATER CONDITIONS

The characteristics of the subsurface material were evaluated by drilling one boring to a depth of 61.5 feet and one boring to 31.5 feet at the approximate locations shown in Figure 2. The logs for the borings are presented in the appendix, and it will be observed that the subsurface profile consists predominantly of interbedded medium dense silty sand and stiff lean clay layers.

In Boring 09-1, medium dense silty sand extended to a depth of 2 feet and was underlain by stiff lean clay from 2 to 11 feet. The lean clay was underlain predominantly by medium dense to dense sand and silty sand, with occasional stiff lean clay layers extending to a depth of 61.5 feet.

Boring 09-2 encountered medium dense silty sand in the upper 4 feet, followed by very stiff lean clay from 4 to 13 feet, loose to medium dense sand from 13 to 20 feet, stiff clay from 20 to 23 feet, and medium dense sandy silty from 23 to 31.5 feet.

Groundwater was measured at a depth of between 8.5 and 10 feet below the existing ground surface at the time the field investigation was performed (May 2009).

The results of classification, density and moisture tests are presented on the boring logs, and the results of all laboratory tests, with exception of the consolidation tests, are summarized in Table 1, Summary of Test Data in the appendix. It will be noted from Table 1 that the in-place dry unit weight of the cohesive material ranges from 88.2 to 99.6 pcf and the natural moisture content varies from 24.0 to 31.0%. The unconfined compressive strength of the lean clay varies from 1660 to 2920 psf. The lean clay has a liquid limit of 35 to 42 and a plasticity index of 15 to 18. The silty clay obtained at a depth of 20 feet in Test Hole 09-B2 has a liquid limit of 27 and a plasticity index of 6.

The compressibility characteristics of the subsurface material were evaluated by performing four consolidation tests on samples obtained from Test Hole 09-B1 at 3 feet, and Test Hole 09-B2 at depths of 4.5, 7, and 10 feet. The results of these tests are also presented in the appendix.

During performance of the consolidation tests, each sample was permitted to absorb water at the beginning of the test to determine the effect of moisture on the compressibility characteristics of these materials. Expansive soils always experience an increase in void ratio on absorbing water. It will be observed from these tests that no increase in the void ratio occurred as the sample absorbed moisture. It is concluded from the consolidation and classification tests that the subsurface clays at this site do not have expansive characteristics. Furthermore, there is no indication that any of the samples tested have collapsible characteristics. The clay has relatively low consolidation characteristics for load intensities less than 1 tsf.

IV. SITE PREPARATION AND COMPACTED FILL REQUIREMENTS

As indicated above, the vegetative cover throughout the building site consists of small evergreen trees and lawn grass. We recommend that the upper 6 inches be stripped from the area and that tree roots be grubbed to remove the excess organic matter in the upper portion of the soil profile.

We recommend that imported fill used to establish final grade throughout the site consist of granular soil having a maximum size of 4 inches with less than 30% passing a No. 200 sieve. We recommend that the material passing a No. 200 sieve have a plasticity index less than 6. The fill should be compacted to an in-place density equal to at least 92% of the maximum density as determined by ASTM D 1557. Structural fill beneath foundations, if needed, should meet requirements outlined in Section V.A.

We recommend that a free-draining granular layer be placed beneath ground level floor slabs. The free-draining granular layer should be at least 6 inches thick and should have a maximum size less than 1 inch and not more than 5% passing a 200 sieve. The free-draining material should be densified using at least 4 passes of a smooth drum 5-ton vibratory roller or equivalent. If the above specifications are followed, the granular layer will prevent the accumulation of moisture beneath the floor slab and will also serve adequately as a base beneath the floor slabs. A subgrade modulus of 150 pci can be used for design.

Grading around the structure should be performed in such a manner that all surface water will flow freely from the area and that no ponding will occur adjacent to the structure which will permit deep percolation into the foundation area. Roof drains should extend well beyond the building lines to prevent seepage into the foundation soils. Sprinkler heads located adjacent to the building should be directed away from the structure to prevent the percolation of water into the foundation zone.

Backfilling around foundation walls should be performed using granular material densified to an in-place unit weight equal to at least 90% of the maximum laboratory density indicated above.

V. FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

A. FOUNDATION TYPES AND BEARING CAPACITIES

We understand that the proposed facility will be a two-story building having a footprint of about 6,000 sq ft. It is anticipated that the facility will be supported using continuous and spot footings. The magnitude of the structural loads are not known as of the preparation of this report; however, it has been assumed that the column loads will not likely exceed 200 kips and that wall loads will not likely exceed 6 klf.

The structure can be supported using spread footings and no other foundation type has been considered during preparation of this report. We recommend that all exterior foundations be located at a depth below finished grade sufficient to provide frost protection, which is about 2.5 feet in this area, and that interior footings be located at least 1 foot below floor level. If this action is taken, it is apparent from the boring logs that the zone of significant stress for foundations will be located primarily within the stiff to very stiff lean clay. Footing excavation should be performed using a smooth-edged bucket to prevent disturbance of the subgrade soil. If the subgrade is disturbed, the disturbed area should be over-excavated, with footings extending to undisturbed soil, or the area should be backfilled with granular fill meeting gradation and compaction requirements outlined in Section IV.

Based upon the results of field and laboratory testing, it is recommended that an allowable soil bearing capacity of 1800 psf be used for the lean clay. The allowable bearing capacity can be increased, if desired, by over excavating the foundation areas and placing the footings on compacted sandy gravel.

ALLOWABLE BEARING CAPACITY (psf)	DEPTH OF FILL (ft)	
	Square Footings	Continuous Footings
2500	0.18 x B, (B≥ 3 ft)	0.39 x B, (B ≥ 2 ft)
3000	0.29 x B, (B ≥ 4 ft)	0.67 x B, (B ≥ 2 ft)
4000	0.49 x B, (B ≥ 4 ft)	N/A

B = Width of Footing

The width of the excavation should extend 2 feet beyond the footing perimeter. Granular fill used to support structural foundations should be compacted to at least 95% of the maximum laboratory density as determined by ASTM D 1557. To ensure that compaction requirements are met, each lift should be tested, with testing performed at 50 foot intervals along continuous footing lines and at each spot footing. Testing should be performed in accordance with ASTM D 6938 (nuclear method), or ASTM D 1556 (sand cone method).

If the foundations for the proposed facility are designed in accordance with the recommendations outlined above, the maximum settlement of any footing should not exceed one inch and differential settlement throughout the structure should not exceed 0.5 inch. It is generally recognized that the tolerable differential settlement for steel and concrete structures is about 0.002 times the column spacing. This criterion is tantamount to a differential settlement of about 0.5 inch for column spacings of 20 feet and 0.7 inch for column spacings of 30 feet. Since it is not anticipated that the column spacing for this structure will be less than 20 feet, a differential settlement of 0.5 inch should be satisfactory for the proposed facility.

B. SEISMIC CONSIDERATIONS

The site is classified as Site Class D, as per Section 1613 of the 2006 International Building Code. The site is located at latitude 40.2802° North and longitude 111.7303° West. Probabilistic peak ground acceleration (PGA) values are tabulated below:

Probabilistic ground motion values in %g.

	10%PE in 50 yr	2%PE in 50 yr
PGA	17.79	49.13
0.2 sec SA	42.42	112.60
1.0 sec SA	14.31	47.28

The allowable soil bearing pressure indicated above may be increased by one-third where seismic forces are involved in the structural loads. If the frictional resistance of the footings and floor slabs are used to resist seismic forces, we recommend a coefficient of friction of 0.40 be used to calculate these forces. See Section C below for recommendations related to resistance provided by passive earth pressures.

A liquefaction analysis has been performed for the site assuming a seismic event having an acceleration of 0.33g, which is two-thirds of the PGA for an event having a probability of exceedence of 2% in 50 years. The results of the analysis indicate that the sand layer between 13.5 and 20 feet in Boring 09-2 has the potential to liquefy during the design event. The consequence of liquefaction of this layer would be strain related settlement estimated to be in the order of 1.5 inches. This settlement will be partially mitigated by the 13 feet of non-liquefiable soil overlying the layer; however, it is anticipated that up to 0.5 inch of differential settlement may occur during the design seismic event.

C. LATERAL EARTH PRESSURES

It is not anticipated that earth-retaining structures will be required for the proposed facility. If earth-retaining structures are required, however, and if backfilling is performed using granular material, and if the backfill behind the wall is horizontal, we recommend that the earth pressures be calculated using the following equation, along with the earth pressure coefficient outlined below:

$$P = \tfrac{1}{2}\,\gamma\, K\, H^2$$

Where P = total lateral force on wall, plf
K = earth pressure coefficient
γ = unit weight of soil (125 pcf)
H = height of retained soil against wall

The earth pressure coefficient used in designing the walls will depend upon whether the wall is free to move during backfilling operations, or whether the wall is restrained during backfilling. If the wall is free to move during backfilling operations and the backfill material is granular soil, we recommend an active earth pressure coefficient of 0.30 be used in the above equation to calculate the lateral earth pressures. If the walls are restrained from any movement during backfilling and the backfill material is granular soil, we recommend an at-rest earth pressure coefficient of 0.45 be used to calculate the lateral earth pressure. A passive earth pressure coefficient of 3.0 may be used to estimate the lateral resistance of the soil in cases where the wall tends to move toward the backfill. In each of these cases, the earth pressure diagram may be approximated as a triangle, such that the resultant earth pressure force P acts at a height of approximately H/3 above the base of the wall.

For the seismic event having a 2-percent probability of exceedance in 50 years, the additional active earth pressure due to ground acceleration may be estimated using a coefficient of 0.2. The seismic ground motion will reduce the available passive resistance. This reduction may be accounted for as an earth pressure acting in the direction opposite the passive resistance, and computed using a coefficient of 0.5. The pressure diagrams for these forces may be roughly approximated as inverted triangles, such that the resultant forces of the seismic components act at heights of approximately 2H/3 above the base of the wall.

For non-yielding walls, the increase in earth pressure corresponding to the seismic event may be estimated using the equation $P_{EQ} = a_h \gamma H^2$, where a_h is a seismic coefficient of 0.33. This force is in addition to the at-rest pressure, and acts at a height of about 0.53H above the base of the wall.

It should be recognized that the pressures calculated by the above equation are earth pressures only and do not include hydrostatic pressures. Where hydrostatic pressures may exist behind a retaining structure, we recommend either the wall be designed to resist hydrostatic pressure, or that a drainage system be placed behind the wall to prevent the development of hydrostatic pressures.

VII. LIMITATIONS

The conclusions and recommendations presented in this report are based upon the results of the field and laboratory tests which, in our opinion, define the characteristics of the subsurface material throughout the site in a satisfactory manner. It should be recognized that soil materials are inherently heterogeneous and that conditions may exist throughout this site which could not be defined during this investigation.

It is recommended that a soils engineer observe the foundation excavations prior to placement of footings. If during construction, conditions are encountered which appear to be different than those presented in this report, it is requested that we be advised in order that appropriate action may be taken.

The information contained in this report is provided for the specific location and purpose of the client named herein and is not intended or suitable for reuse by any other person or entity whether for the specified use, or for any other use. Any such unauthorized reuse, by any other party is at that party's sole risk and RB&G Engineering, Inc. does not accept any liability or responsibility for its use.

Figures

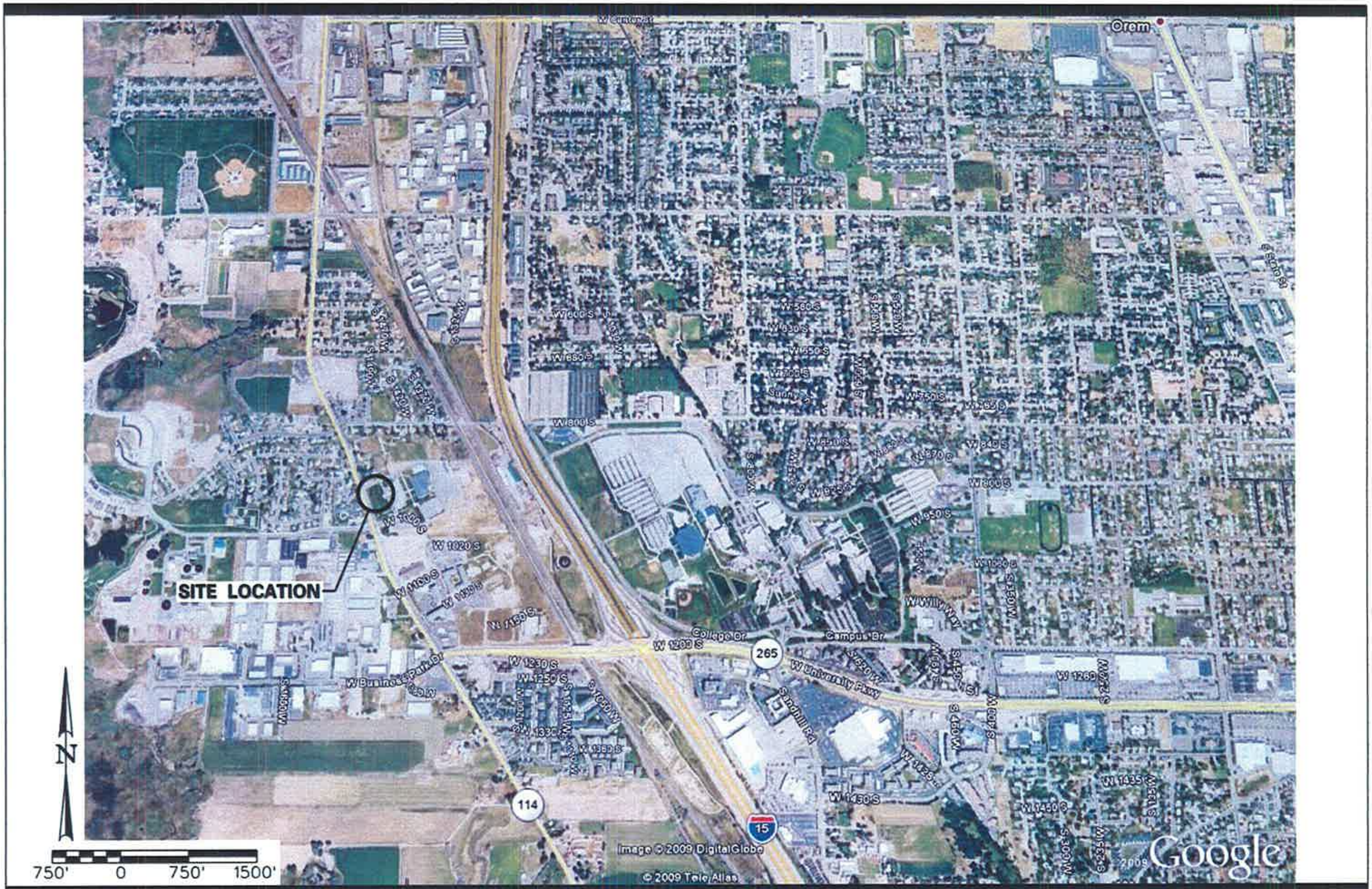

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Figure 1 **VICINITY MAP**

UVU West Campus - Small Business Development Building
Orem, Utah

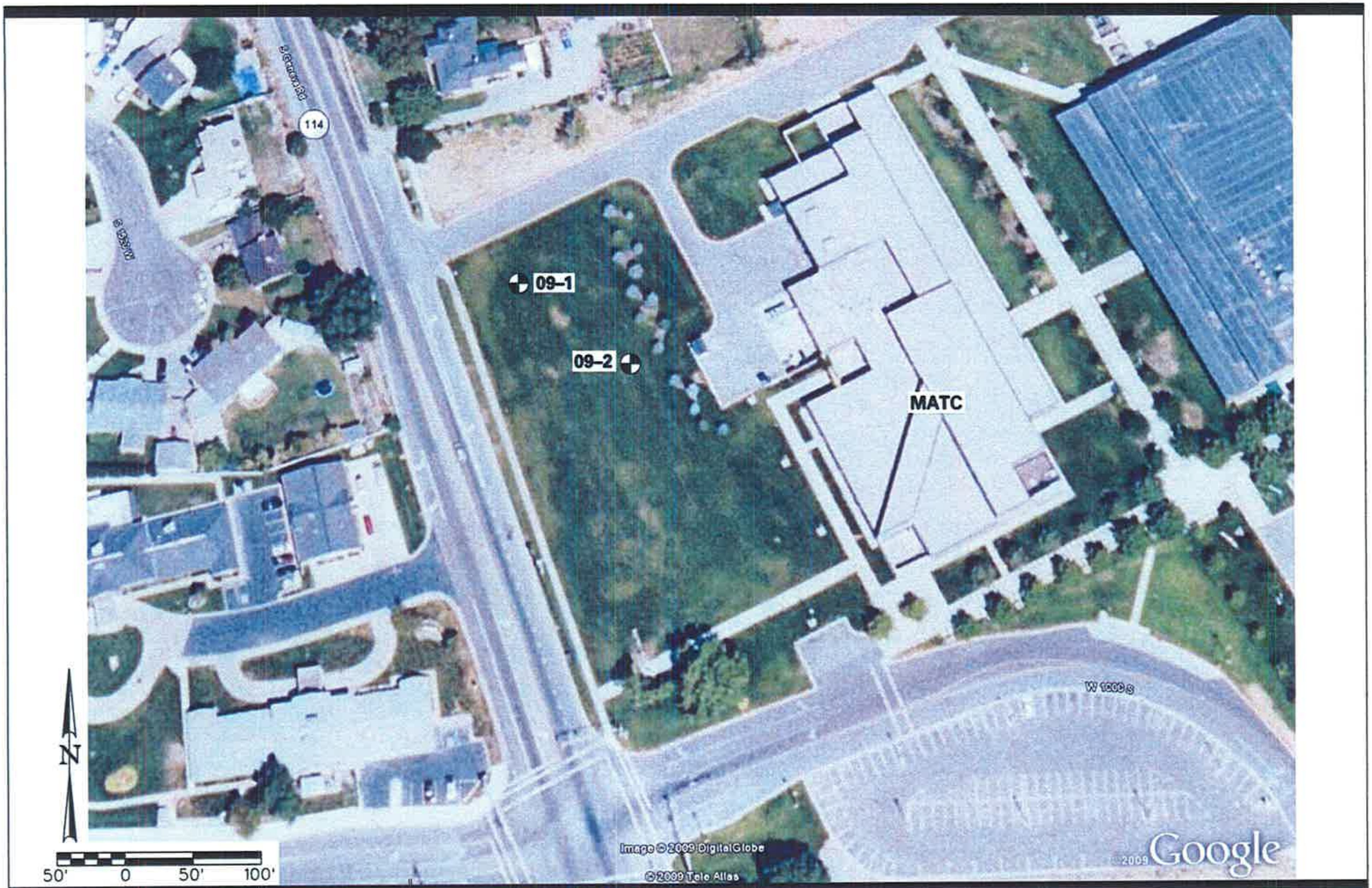

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Figure 2 **SITE PLAN & TEST HOLE LOCATIONS**

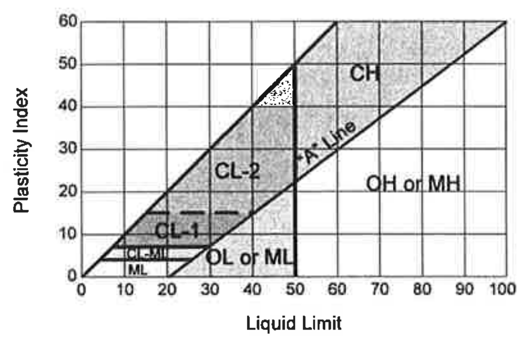
UVU West Campus - Small Business Development Building

Orem, Utah

Appendix

Unified Soil Classification System

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria		
COARSE-GRAINED SOILS *more than half of material is larger than No. 200 sieve*	**Gravels** *more than half of coarse fraction is larger than No. 4 sieve size*	**Clean Gravels** *little or no fines*	GW		Well graded gravels, gravel-sand mixtures, little or no fines	*For laboratory classification of coarse-grained soils*	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines	Determine percentage of gravel and sand from grain-size curve.	Not meeting all gradation requirements for GW	
		Gravels With Fines *appreciable amount of fines*	GM*	d	Silty gravels, poorly graded gravel-sand-silt mixtures	Depending on percentage of fines (*fraction smaller than No. 200 sieve size*), coarse-grained soils are classified as follows:	Atterberg limits below "A" line, or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring uses of dual symbols
				u				
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, or PI greater	
	Sands *more than half of coarse fraction is smaller than No. 4 sieve size*	**Clean Sands** *little or no fines*	SW		Well graded sands, gravelly sands, little or no fines	**Less than 5%** GW, GP, SW, SP	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines	**More than 12%** GM, GC, SM, SC	Not meeting all gradation requirements for SW	
		Sands with Fines *appreciable amount of fines*	SM*	d	Silty sands, poorly graded sand-silt mixtures	**5% to 12%** Borderline cases requiring use of dual symbols**	Atterberg limits below "A" line, or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring uses of dual symbols
				u				
			SC		Clayey sands, poorly graded sand-clay mixtures		Atterberg limits above "A" line, or PI greater	
FINE-GRAINED SOILS *more than half of material is smaller than No. 200 sieve*	**Silts and Clays** *liquid limit is less than 50*		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	*For laboratory classification of fine-grained soils* (see Plasticity Chart)		
			CL	1	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
				2				
			OL		Organic silts and organic silt-clays of low plasticity			
	Silts and Clays *liquid limit is greater than 50*		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils			



Plasticity Chart

NOTE: USCS Modified to include CL-type subcategories

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when liquid limit is 28 or less and the PI is 6 or less, the suffix u used when liquid limit is greater than 28.

***Borderline classification*: Soils possessing characteristics of two groups are designated by combinations of group symbols. (For example GW-GC, well graded gravel-sand mixture with clay biner.)

DRILL HOLE LOG

BORING NO. 09-1

SHEET 1 OF 1

PROJECT: UVU WEST CAMPUS - SMALL BUSINESS DEVELOPMENT BUILDING
CLIENT: DFCM
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING TO 25'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ▽ N.M. **AFTER 24 HOURS:** ▼ 8.5'

PROJECT NUMBER: 200901.023
DATE STARTED: 5/1/09
DATE COMPLETED: 5/4/09
GROUND ELEVATION: NOT MEASURED
LOGGED BY: C. SANBORN, 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
			■	10	4,4,4,(17)	SM	dk. brown, moist, med. dense	SILTY SAND organics, clay fragments								
	5		⊠	8	Pushed 0.73	CL-2	brown, moist, stiff	LEAN CLAY W/SAND many silt and/or sand lenses	93.7	25.3	40	15				CT
	▼		⊠	13	Pushed 0.77	CL-2	brown, moist, stiff		88.2	31.0	42	18				UC
	10		■	18	7,9,9,(33) 1.80	CL	brown, moist, stiff									
			■	14	5,6,10,(28)	SP-SM	brown, wet, med. dense	SAND W/SILT occasional gravels								
	15		■	15	3,7,10,(27)	SP-SM	brown, wet, med. dense			23.7		NP	1	93	6	
	20		■	15	3,10,11,(31)	SP-SM	brown & gray, wet, dense									
	25		■	18	6,11,13,(33)	SP-SM	brown, wet, dense			23.0		NP	1	94	5	
	30		■	18	5,10,14,(31)	SP-SM ML SP-SM	brown, wet ~~gray, moist~~ gray-brown, wet, dense	SILT plastic SAND W/SILT								
	35		■	18	7,11,10,(25)	ML	gray, wet, med. dense	SANDY SILT clay layers less than 1" thick								
	40		■	18	11,11,13,(27)	SM	gray, wet, med. dense	SILTY SAND clay lenses								
	45		■	16	0.63 6,18,21,(43)	CL ML	gray, moist, stiff gray, wet, dense	LEAN CLAY silt & sand lenses SANDY SILT								
	50		■	18	8,10,12,(23) 0.56	SM,CL	gray-brown, wet/moist, med. dense/stiff	INTERBEDDED SILTY SAND & LEAN CLAY LAYERS LESS THAN 2" THICK								
	55		■	16	24,32,37,(69)	SP-SM	gray-brown, wet, very dense	SAND W/SILT occasional clay lenses								
	60		■	17	12,24,26,(48)	SP-SM	gray, wet, dense									

DH_LOGV1 UVUWSBDEVBLDG.GPJ US EVAL.GDT 5/27/09

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LEGEND:

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

UNDISTURBED SAMPLE ⊠ PUSHED 0.45 — Torvane (tsf)

OTHER TESTS
UC = Unconfined Compression
CT = Consolidation
DS = Direct Shear
UU = Unconsolidated, Undrained
CU = Consolidated, Undrained
HYD = Hydrometer
SS = Soluble Salt
DC = Dispersive Clay

DRILL HOLE LOG

BORING NO. 09-2

SHEET 1 OF 1

PROJECT: UVU WEST CAMPUS - SMALL BUSINESS DEVELOPMENT BUILDING

CLIENT: DFCM

LOCATION: SEE SITE PLAN

DRILLING METHOD: 96-CME-55 / N.W. CASING TO 25'

DRILLER: T. KERN

DEPTH TO WATER - INITIAL: ∇ 10.0' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.023

DATE STARTED: 5/5/09

DATE COMPLETED: 5/5/09

GROUND ELEVATION: NOT MEASURED

LOGGED BY: C. SANBORN, 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
			Disturbed	16	6,6,8,(29)	SM	dk. brown, moist, med. dense	SILTY SAND								
			Disturbed	18	3,2,3,(11)	SM	brown, moist, med. dense									
						SC	brown, moist	CLAYEY SAND								
	5		Undisturbed	16	Pushed 1.23	CL-2	brown, moist, very stiff		88.7	28.0	42	17				CT UC
			Undisturbed	9	Pushed 2.23	CL-2	brown, moist, very stiff	LEAN CLAY W/SAND interbedded w/silt and/or sand lenses	96.3	24.9	41	16				CT
∇	10		Undisturbed	12	Pushed 1.43	CL-2	brown, moist, very stiff		98.8	24.0	35	15				CT UC
			Undisturbed	15	Pushed 1.50	CL-2	brown, moist, very stiff		99.6	24.0	36	18				UC
			Disturbed	10	1,2,4,(10)	SP-SM	brown, wet, loose			25.4		NP	0	93	7	
	15		Disturbed	14	4,4,4,(13)	SP-SM	brown & gray, wet, med. dense	SAND W/SILT few silt layers less than 1/2" thick								
	20					SP-SM	gray, wet									
			Disturbed	14	4,3,3,(9) 0.56	CL-ML	gray-brown, moist, stiff	SILTY CLAY sand lenses		27.0	27	6				
	25		Disturbed	13	3,3,8,(15) 0.71	CL	gray-brown, moist, stiff	LEAN CLAY many silt and/or sand lenses								
	30							SANDY SILT few clay lenses								
			Disturbed	12	7,10,14,(30)	ML	gray-brown, wet, med. dense									

DH_LOGV1 UVUWSBDEVBLDG.GPJ US EVAL.GDT 5/19/09

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
CU = Consolidated, Undrained
HYD = Hydrometer
SS = Soluble Salt
DC = Dispersive Clay

Table 1

SUMMARY OF TEST DATA

PROJECT UVU West Campus
Small Business Development Building

LOCATION Orem, Utah

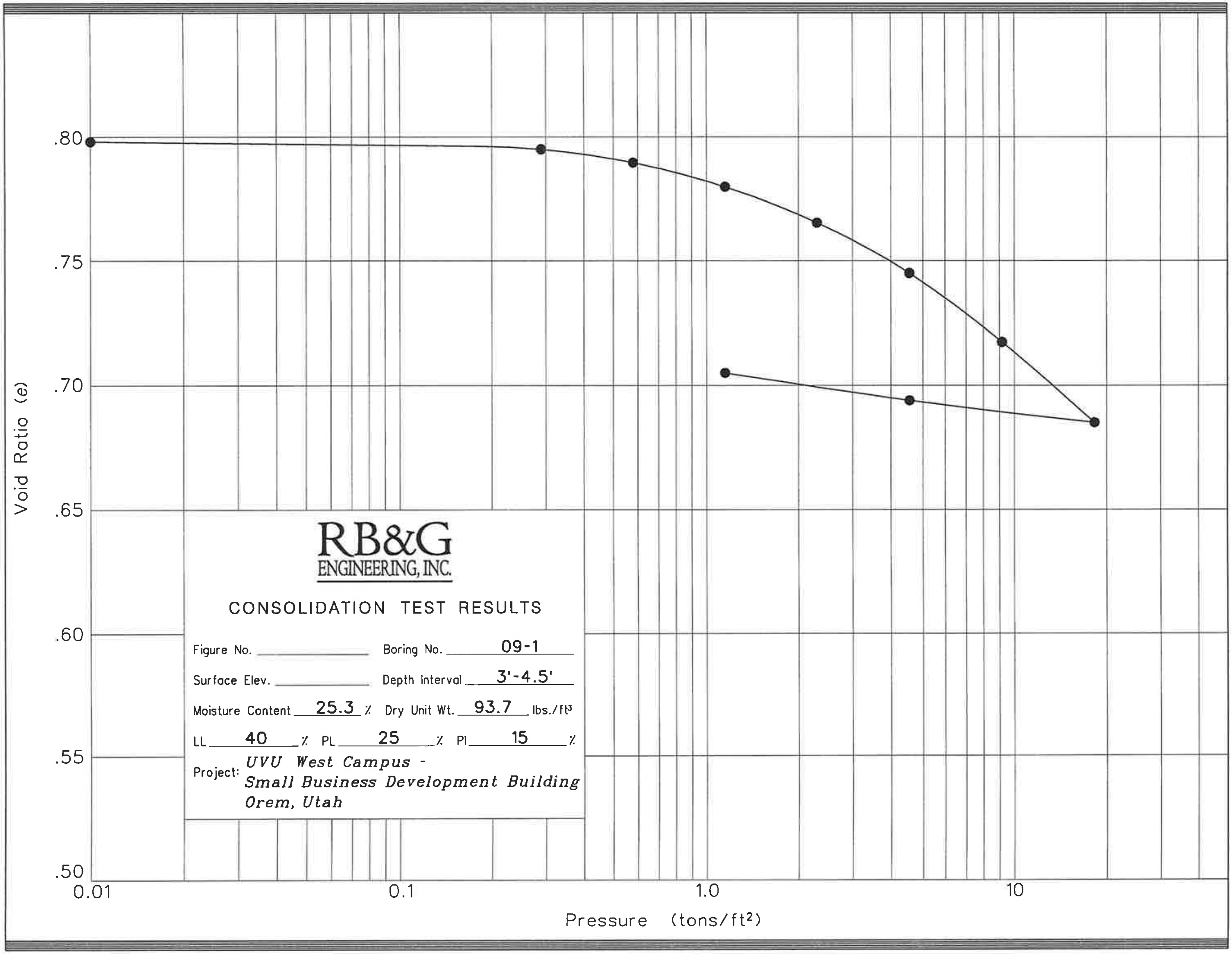
PROJECT NO. 200901-023

FEATURE Foundations

HOLE NO.	DEPTH BELOW GROUND SURFACE (ft)	STANDARD PENETRATION BLOWS PER FOOT Corrected $(N_1)60$	IN-PLACE		UNCONFINED COMPRESSIVE STRENGTH (psf)	ATTERBERG LIMITS			MECHANICAL ANALYSIS			UNIFIED SOIL CLASSIFICATION SYSTEM (modified)
			DRY UNIT WEIGHT (pcf)	MOISTURE (%)		LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	PERCENT GRAVEL	PERCENT SAND	PERCENT SILT& CLAY	
09-B1	3-4.5	Shelby	93.7	25.3	2920*	40	25	15				CL-2
	6-7.5	Shelby	88.2	31.0	1660	42	24	18				CL-2
	15-16.5	27		23.7				NP	1	93	6	SP-SM
	25-26.5	33		23.0				NP	1	94	5	SP-SM
09-B2	4.5-6	Shelby	88.7	28.0	2150	42	25	17				CL-2
	7-8.5	Shelby	96.3	24.9		41	25	16				CL-2
	10-11.5	Shelby	98.8	24.0	2308	35	20	15				CL-2
	12-13.5	Shelby	99.6	24.0	2858	36	18	18				CL-2
	13.5-15	10		25.4				NP	0	93	7	SP-SM
	20-21.5	13		27.0		27	21	6				CL-ML

		Resistivity (ohm cm)	pH	Sulfate (%)
09-B2	4.5-6	1300	8.4	

NP=Nonplastic
*Torvane value used to estimate unconfined compressive strength.



RB&G ENGINEERING, INC.

CONSOLIDATION TEST RESULTS

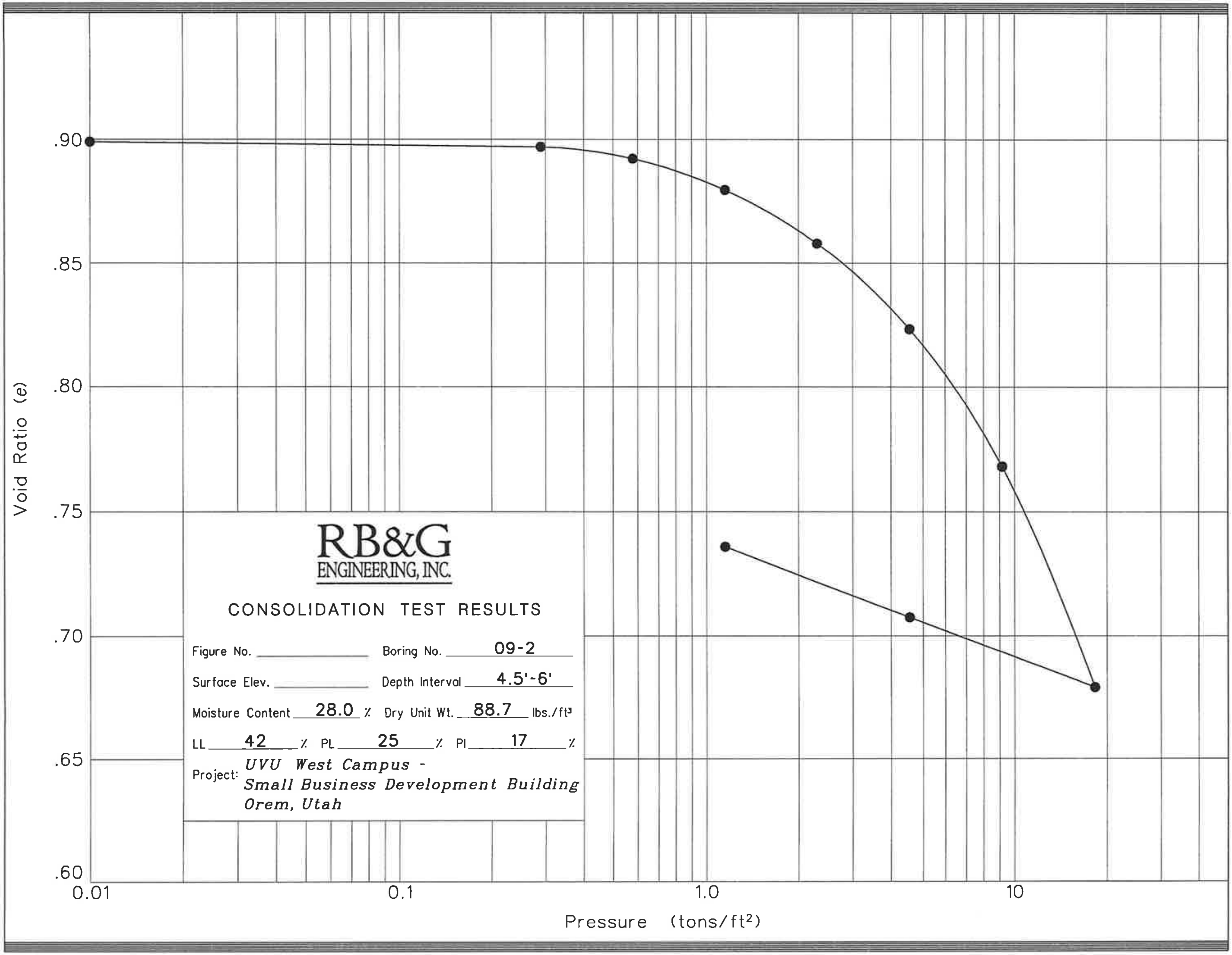
Figure No. ______ Boring No. 09-1

Surface Elev. ______ Depth Interval 3'-4.5'

Moisture Content 25.3 % Dry Unit Wt. 93.7 lbs./ft³

LL 40 % PL 25 % PI 15 %

Project: *UVU West Campus -*
Small Business Development Building
Orem, Utah



RB&G ENGINEERING, INC.

CONSOLIDATION TEST RESULTS

Figure No. ______ Boring No. 09-2

Surface Elev. ______ Depth Interval 4.5'-6'

Moisture Content 28.0 % Dry Unit Wt. 88.7 lbs./ft³

LL 42 % PL 25 % PI 17 %

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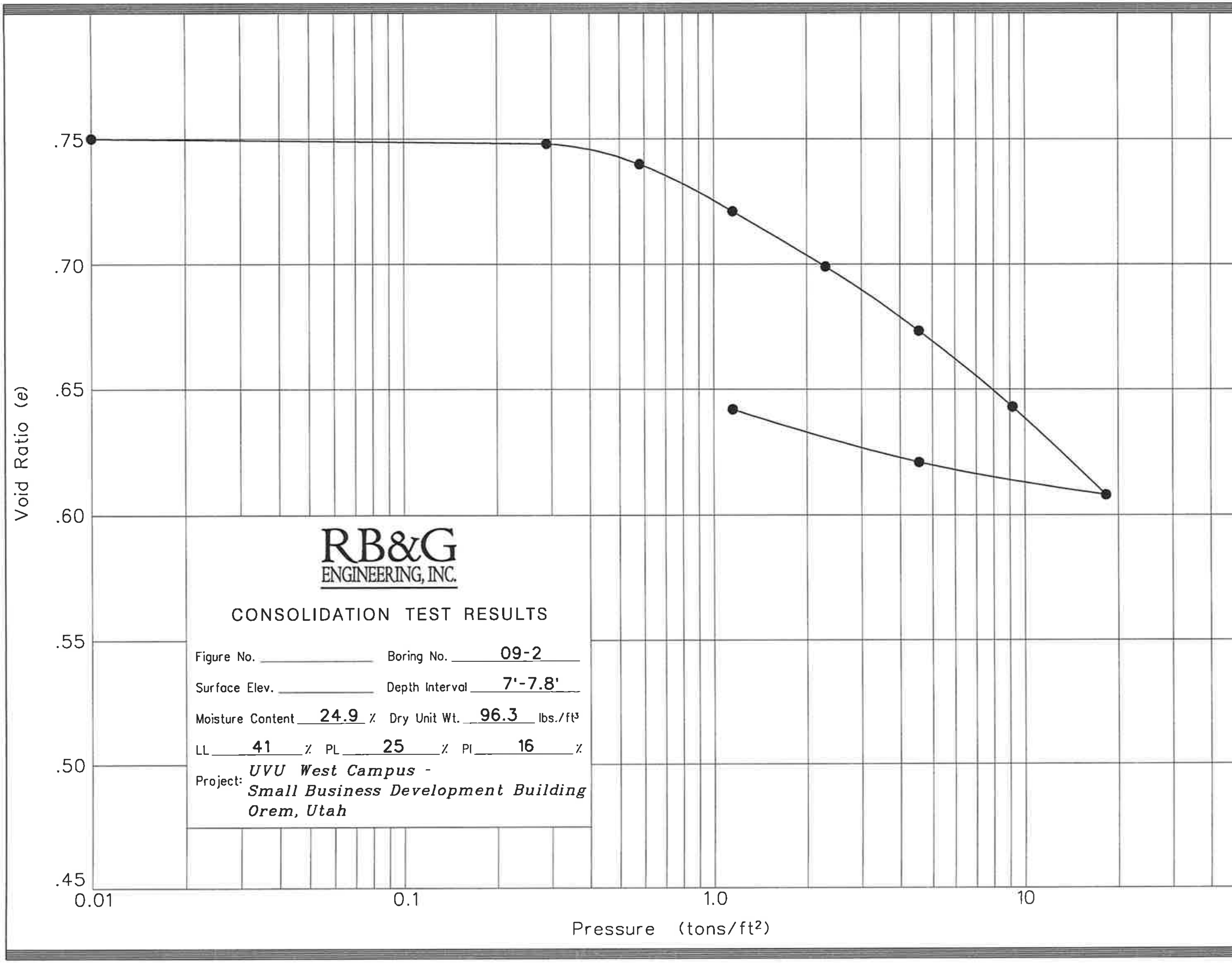
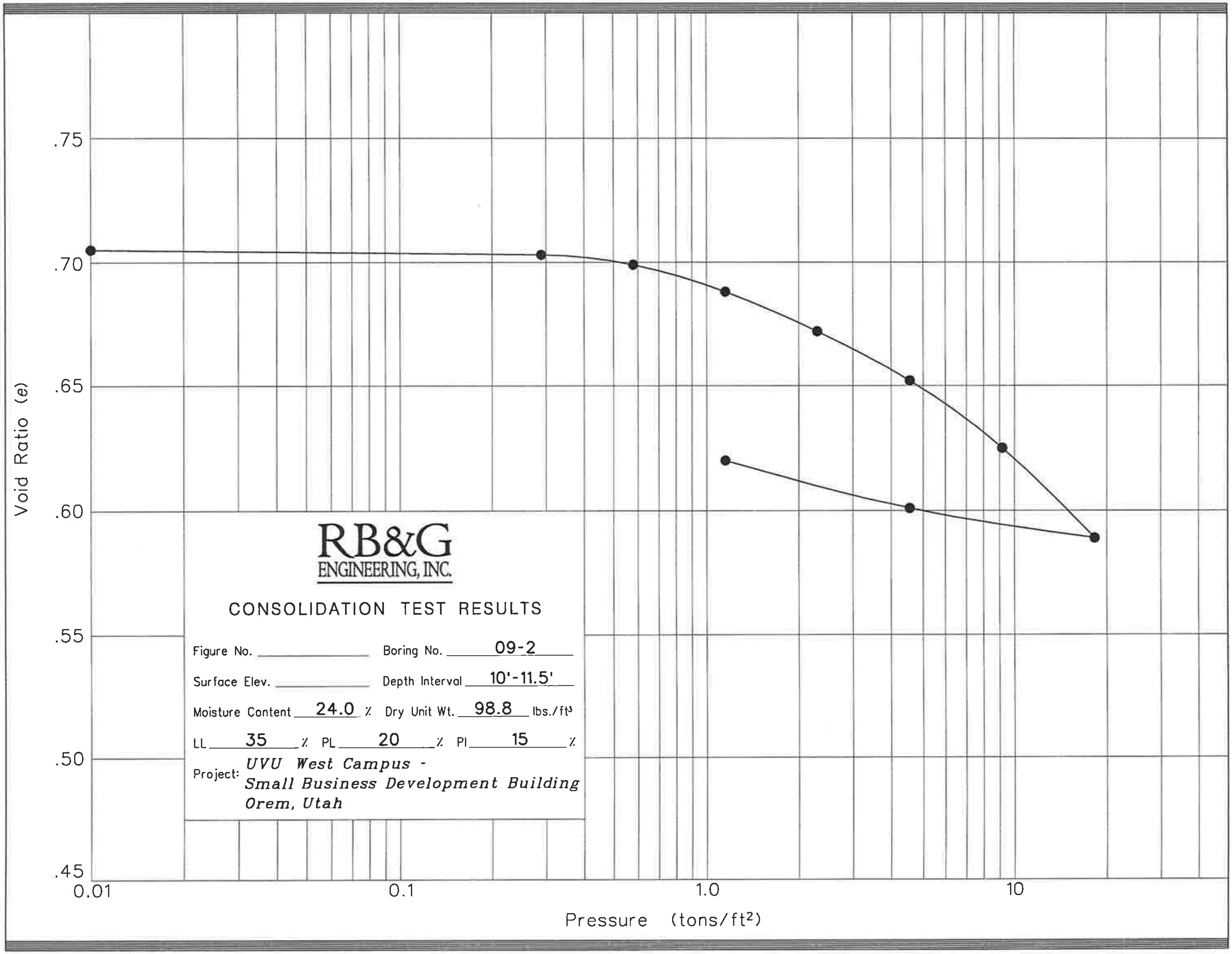
Small Business Development Building
Orem, Utah



.75
.70
.65
.60
.55
.50
.45
Void Ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
RB&G ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-2
Surface Elev.
Depth Interval 7'-7.8'
Moisture Content 24.9 % Dry Unit Wt. 96.3 lbs./ft³
LL 41 % PL 25 % PI 16 %
Project: UVU West Campus - Small Business Development Building Orem, Utah



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CONSOLIDATION TEST RESULTS

Figure No. ______ Boring No. 09-2

Surface Elev. ______ Depth Interval 10'-11.5'

Moisture Content 24.0 % Dry Unit Wt. 98.8 lbs./ft³

LL 35 % PL 20 % PI 15 %

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