State of Utah

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Department of Administrative Services
KIMBERLY K. HOOD
Executive Director

Division of Facilities Construction and Management
DAVID G. BUXTON
Director

ADDENDUM NO. 1

Date:	April 12, 2010
To:	Contractors
From:	Dave McKay - Project Manager
Reference:	Pope Science Building Addition Utah Valley University – Orem, Utah DFCM Project No. 09020790
Subject:	**Addendum No. 1**

Pages			
	Addendum Cover Sheet	1	page
	Revised Project Schedule	1	page
	Geotechnical Investigation dated May 29, 2009	31	pages
	Geotechnical Investigation dated November 30, 2009	41	pages
	Total	74	pages

Note: This Addendum shall be included as part of the Contract Documents. Items in this Addendum apply to all drawings and specification sections whether referenced or not involving the portion of the work added, deleted, modified, or otherwise addressed in the Addendum. Acknowledge receipt of this Addendum in the space provided on the Bid Form. Failure to do so may subject the Bidder to Disqualification.

While we contend that SB220 should only be potentially applicable to a contract issued after the effective date of said bill, this is to clarify that for purposes of this contract, regardless of the execution or effective dates of this contract, the status of Utah Law and remedies available to the State of Utah and DFCM, as it relates to any matter referred to or affected by said SB220, shall be the Utah law in effect at the time of the issuance of this Addendum.

1.1 SCHEDULE CHANGES: See attached Revised Project Schedule - changes highlighted in yellow.

1.2 GENERAL ITEMS:

1.2.1 Please note that there will be no requirement to address construction schedule in the initial submission with Statements of Qualification, Management Plan, and References. Scoring in the shortlisting selection will be only on Past Performance, Strength of Team, and Project Management Approach.

1.2.2 The geotechnical work by RB&G, Brad Price, is included as attachments to this addendum

PROJECT SCHEDULE – REVISED
PER ADDENDUM NO. 1 DATED APRIL 13, 2010

PROJECT NAME: **POPE SCIENCE BUILDING ADDITION**
UTAH VALLEY UNIVERSITY – OREM, UTAH

DFCM PROJECT NO. **09020790**

Event	Day	Date	Time	Place
Request for Proposals and Construction Documents Available	Tuesday	March 30, 2010	12:00 NOON	DFCM 4110 State Office Bldg SLC, UT and the DFCM web site *
Mandatory Pre-Proposal Site Meeting	Thursday	April 8, 2010	3:00 PM	Student Center, SC213A Utah Valley University Orem, UT
Last Day to Submit Questions prior to submittal of Statements of Qualifications	Monday	April 19, 2010	4:00 PM	E-mail dmckay@utah.gov or call Dave McKay 801-541-9019
Addendum Deadline	Tuesday	April 20, 2010	4:00 PM	DFCM web site *
Prime Contractors turn in References, Statements of Qualifications, Management Plans (including Schedule), and Termination/Debarment Certifications	Thursday	April 22, 2010	12:00 NOON	DFCM 4110 State Office Bldg SLC, UT
Short Listing by Selection Committee (if applicable)	Wednesday	April, 28, 2010	TBD	DFCM web site *
Mandatory Finalists Meeting (Contract Documents to be distributed on CD)	Wednesday	May 5, 2010	3:00 PM	Student Center SC213A Utah Valley University Orem, UT Use Student Center Parking
Last Date to Submit Questions for Final Addendum	Thursday	May 13, 2010	12:00 NOON	E-mail dmckay@utah.gov or call Dave McKay 801-541-9019
Final Addendum Deadline (exception for bid delays)	Tuesday	May 18, 2010	4:00 PM	DFCM web site *
Prime Contractors Turn In Cost Proposals and Cost Reduction Proposals	Tuesday	May 25, 2010	12:00 NOON	DFCM 4110 State Office Bldg SLC, UT
Subcontractor List Due	Wednesday	May 26, 2010	12:00 NOON	DFCM 4110 State Office Bldg SLC, UT Fax 801-538-3677
Interviews	Wednesday	June 2, 2010	TBD	To be determined
Announcement	Thursday	June 3, 2010	4:00 PM	DFCM web site *
Substantial Completion Date	Saturday	December 31, 2011		

* DFCM's web site address is **http://dfcm.utah.gov**.

GEOTECHNICAL INVESTIGATION

UVU SCIENCE BUILDING ADDITION

Orem, Utah

Prepared for: DFCM

May 2009

RB&G ENGINEERING, INC.

200901-022

RB&G
ENGINEERING, INC.

May 29, 2009

Dave McKay
DFCM
4110 State Office Building
Salt Lake City, UT 84114

Subject: Utah Valley University
Science Building Addition
DFCM Project No. 0920790/Contract No. 097429
Geotechnical Investigation

Gentlemen:

A Geotechnical Investigation has been completed for the proposed addition to the Science Building located on the UVU 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.

REGISTERED PROFESSIONAL ENGINEER
NO.162291
BRADFORD E.
PRICE
6/1/09
STATE OF UTAH

Bradford E. Price, P.E.

bep/jal

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Geotechnical Investigation

UVU Science Building Addition

Orem, Utah

Prepared for: DFCM

May 2009

RB&G ENGINEERING, INC.

RB&G
ENGINEERING, INC.

UVU
Science Building Addition
Orem, Utah

Geotechnical Investigation

INTRODUCTION

This report outlines the results of a geotechnical investigation performed for the proposed addition to the Science Building located on the Utah Valley University (UVU) Campus in Orem, Utah, at the location shown on the Vicinity and Site Plan Maps in Figures 1 and 2. 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.

RB&G Engineering performed geotechnical investigations for the Science Building in 1986 and 1987, and applicable information from these investigations has been used during this study.

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 sand deposits laid down during the Provo regressive phase of the Bonneville lake cycle (upper Pleistocene). The Wasatch Fault Zone is located approximately 3 miles east of the site. Utah County Natural hazards maps identify this area as having moderate liquefaction potential.

The area where the proposed addition will be located is presently landscaped in lawn grass with some trees and shrubs. In general, the topography slopes relatively steeply downward from the existing building on the north to a relatively flat area between the building and ponds shown in Figure 2. Much of the area south of the PS and EN buildings is landscaped with terraces planted in shrubs.

Structures in the immediate vicinity are supported using spread foundations on compacted fill. Foundations appear to be performing in a satisfactory manner, in that no significant cracking was observed in foundation walls.

As shown in Figure 2, lined detention ponds are located immediately south and west of the proposed addition. 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 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, and sulfate 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 two borings to a depth of 81.5 feet and one boring to 33 feet at the approximate locations shown in Figure 2. The logs for the borings are presented in the appendix. It will be noted from the boring logs that the ground elevation at each boring location was within 0.5 feet of the floor elevation at the west entrance of the restricted lab area.

It will be observed that the subsurface profile consists of a surface layer of medium dense silty sand and firm silt extending 5 to 7 feet below the existing ground surface, underlain generally by firm to stiff lean clay with occasional silty sand layers extending to a depth of about 43 feet. From a depth of 43 feet to the bottom of the borings at 81.5 feet, the soil profile consists predominantly of dense to very dense sand with silt.

Groundwater was encountered at a depth of between 3.3 and 6 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 soils varies from 86.5 to 101.6 pcf, and that the natural moisture content of the cohesive material ranges from 22.9 to 36.1%. The unconfined compressive strength of the cohesive soil varies from 526 to 2840 psf.

The compressibility characteristics of the subsurface material were evaluated by performing five consolidation tests on samples obtained from Boring B1 at 9 to 10.5 feet, Boring B2 at 6 to 7.5

feet, and Boring B3 at 9 to 10.5 feet, 30 to 31.5 feet, and 40 to 41.5 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. It will be noted that the clay is over-consolidated with relatively low compressibility characteristics for load intensities less than 3 tsf.

In order to obtain an indication of the corrosive nature of the subsurface material at this site, resistivity, pH, sulfate and chloride tests were performed on samples obtained in Borings 1 and 3 at a depth of to 4 feet below the ground surface. It will be noted that the sandy material has have a pH of 7.6, resistivity of 2600 ohm cm, sulfate less than .0005%, and chloride of 41 mg/Kg. The plastic silt has a pH of 8.1, resistivity of 1500 ohm cm, sulfate of .011%, and chloride of 740 mg/Kg. This material has relatively poor corrosion resistance. It is recommended that Type II cement be used for concrete in contact with the native soils due to its increased resistance to sulfate attack.

IV. FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

A. FOUNDATION TYPES AND BEARING CAPACITIES

We understand that the proposed addition will be 4 to 5 stories with a footprint of between 30,000 to 35,000 sq ft and no basement; however, excavation into slope adjacent to existing buildings may be necessary. We also understand that one or more buildings may be demolished to make room for the new addition. 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 600 kips and that wall loads will not likely exceed 8 klf.

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 within the medium dense silty sand and soft to firm silt and clay.

The allowable bearing capacity of the cohesive material in the upper 15 feet of the soil profile ranges from about 800 to 2100 psf, with an average of about 1200 psf. It is readily apparent that supporting the structure using spread footings on the native soil would result in excessively large foundations.

Foundation support options considered during preparation of this report include (1) spread footings on compacted granular fill, (2) deep foundations, and (3) mat foundations. Each of these foundation types are discussed separately below:

1. Spread Footings on Compacted Granular Fill

A significant increase in bearing capacity can be achieved by placing the footings on compacted fill. The following table shows the required fill thickness to obtain a given allowable bearing capacity for continuous and spot footings:

ALLOWABLE BEARING CAPACITY	CONTINUOUS FOOTINGS Depth of Fill (ft)	SQUARE FOOTINGS Depth of Fill (ft)
2000	1.0 x B, (B≥ 2 ft)	0.41 x B, (B≥ 3 ft)
2500	1.5 x B, (B≥ 2 ft)	0.58 x B, (B≥ 4 ft)
3000	2.0 x B, (B≥ 2 ft)	0.73 x B, (B≥ 4 ft)
4000	N/A	1.00 x B, (B≥ 4 ft)

B = Width of Footing

If the spot footings are designed using an allowable bearing capacity of 3000 psf, column loads of 600,000 pounds would require a 14 foot square footing and about 10.5 feet of granular fill over the lean clay. Using an allowable bearing capacity of 4000 psf requires at 12.3 foot square footing and about 12.3 feet of fill. Placement of this magnitude of compacted fill results in excavation below the existing groundwater level, requiring dewatering and stabilization of the subgrade. Dewatering of the lean clay can best be accomplished using closely spaced well points. The groundwater level should be lowered to at least 2 feet below the base of the excavation. The width of footing excavations should be equal to the width of the footing plus the depth of fill, plus 2 feet.

Due to the anticipated footing width and required depth of fill, it may be more efficient to excavate the entire footprint and replace with compacted fill, if the spread footing option is used. The on-site silty sand can be used as structural fill below a depth of 5 feet from footing subgrade. Structural fill within 5 feet of footing subgrade should consist of relatively well graded sand gravel having a maximum size of 4 inches with less than 20%

passing a No. 200 sieve. The fines should have a plasticity index less than 5. The fill should be placed in lifts not exceeding 1 foot in thickness and compacted to an in-place dry unit weight of 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 criteria is tantamount to a differential settlement of about 0.5 inch for column spacing's of 20 feet and 0.7 inch for column spacing's 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.

2. Mat Foundation

The relatively low bearing capacity of the natural subgrade soils and the anticipated large structural loads make a mat foundation an efficient option to provide uniform distribution of soil pressure under the foundation.

Settlement analyses have been performed assuming net allowable bearing capacities of 800 psf and 1200 psf for the mat foundation. A total estimated consolidation settlement at the center of the mat of 0.91 inches for the 800 psf loading and 1.26 inches for the 1200 psf loading has been computed. Differential settlement will be less than 0.6 inches, which should be entirely satisfactory for a mat type foundation.

It is recommended that a coefficient of subgrade reaction (k_1) of 50 lb/in^3 be used for the lean clay and plastic silt subgrade. It is also recommended that consideration be given to over excavating the subgrade and placing 1 foot of compacted sandy gravel beneath the mat foundation. The fill should be compacted to an in-place unit weight equal to at least 92% of the maximum density as determined by ASTM D 1557. If this action is taken, the coefficient of subgrade reaction can be increased to 100 lb/in^3. The fill can include the 6" layer of free draining gravel beneath the mat discussed in Section VI.A below.

3. Deep Foundations

Driven Piles

Consideration has been given to supporting the structure on driven piles extending at least 5 feet into the dense silty sand which was encountered at a depth of about 43 feet throughout the site. Axial compressive capacities for 12.75-inch, 14-inch, and 16-inch (outside diameter) closed-end concrete-filled steel pipe piles are summarized on the following table.

Pipe Pile Outside Diameter (inches)	Ultimate Skin Friction (kips)	Ultimate End Bearing (kips)	Allowable Capacity Assuming Factor of Safety = 2.25 (kips)
12.75	116	133	110
14.0	133	162	131
16.0	162	212	166

Pile layouts should be designed with a minimum center-to-center spacing of 3 pile diameters between piles. It will be noted that a factor of safety of 2.25 has been used to calculate the allowable capacities. This factor of safety assumes that PDA testing will be performed during driving of one pile for columns located near the four corners and center of the structure. Pile uplift capacity can be estimated by using a factor of safety of 3 with the ultimate skin friction values. If this option is selected, pile lateral capacities, along with estimated pile group settlement, can be evaluated. It is anticipated that group settlement will be tolerable for column loads less than 600 kips. We recommend that the geotechnical engineer's representative be present during pile installation.

Drilled Shafts

Drilled shafts have also been considered as a foundation option for supporting the structure. It has been assumed that the shafts will be drilled at least 5 feet into the dense silty sand referenced in the Driven Pile section above. Procedures outlined in FHWA-H1-88-042, Drilled Shafts: Construction Procedures and Design Methods, have been used to determine the ultimate axial compressive capacity (nominal resistance) of drilled shafts. Capacity analyses have been performed for straight-sided drilled shafts using soil parameters obtained from the borings. If allowable stress design methods are used, we recommend that a factor of safety of 2.5 be applied to the ultimate capacity to determine the allowable capacity. It has been assumed that high quality construction, good specifications and excellent inspection will exist for each foundation. The estimated capacities of the drilled shafts can be taken from the table below.

Shaft Diameter (ft)	Ultimate Side Resistance (kips)	Ultimate End Resistance (kips)	Total Ultimate Capacity (kips)	Allowable Capacity (kips)
3	258	322	580	232
3.5	301	439	740	296
4	344	573	917	367
4.5	387	725	1112	445
5	403	895	1298	519

The allowable uplift resistance of a single drilled shaft may be taken as the ultimate side resistance value shown on the table above divided by a factor of safety of 3.0. A center-to-center spacing of at least three shaft diameters should exist to minimize interaction and overlapping stresses between shafts, which would result in reduced capacity.

The design of rebar and concrete should follow established guidelines. If the foundation recommendations presented above are followed, the maximum settlement of any drilled shaft should not be greater than about 1 inch. Due to the high water table and presence sand layers, drilling slurry or casing will likely be required for shaft excavation. Concrete should be placed by tremie methods to ensure that no voids exist within the shafts. Concrete used for shafts should have a relatively high slump (6 inches or greater) to allow workability and proper placement between reinforcement and the sides of the shafts. Within each shaft, concrete should be placed in a generally continuous manner to prevent cold joints and other problems associated with excessive waits between concrete trucks. It is essential that drilled shaft construction be carefully inspected to ensure that loose material is removed from the base and that the concrete is placed using proper procedures.

While all options discussed above will provide satisfactory support, it is our opinion that supporting the structure on a mat type foundation may be the most efficient design.

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 40.2778° North and longitude 111.7153° 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	17.70	50.57
0.2 sec SA	42.19	114.89
1.0 sec SA	14.24	48.47

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 0.34g, which is 2/3's of the event having a probability of exceedence of 2% in 50 years. The results of the analysis indicate that the sand in the upper 80 feet will have a factor of safety greater than 1.5 against liquefaction. Based upon the results of the analysis, it is concluded that problems associated with liquefaction during a seismic event are unlikely at this site, and no special mitigation of the foundation soils is required.

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 0.34. 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.

V. SITE PREPARATION AND COMPACTED FILL REQUIREMENTS

As indicated above, the vegetative cover throughout the building site consists of lawn grass, shrubs and trees. 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.

Temporary excavations extending to a depth of less than 20 feet can be sloped at 1 horizontal to 1 vertical.

It is anticipated that stabilization of the foundation excavations may be required prior to placement of structural fill. Care should be taken to prevent heavy construction equipment from traversing directly on the subgrade soils. Stabilization techniques are dependant upon conditions encountered and construction methods. Where very soft clay exists, it is anticipated that cobble rock will provide the most effective means of stabilization. Where cobble rock is required, it should consist of 3 to 8 inch rock placed in single lifts, tamped into the clay such that the voids are filled. Excess cobbles which cannot be tamped into the clay should be removed to prevent migration of fines into the voids, which would result in settlement. Placement of a geotextile fabric, such as Mirafi 600X or equivalent will be effective in stabilizing moderately soft areas.

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 20% passing a No. 200 sieve. We recommend that the material passing a No. 200 sieve have a plasticity index less than 5. 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 IV.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 100 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.

VI. 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. Since the bearing capacity for foundation design is dependent upon adequate compaction of imported fill, it is requested that testing of the fill be performed under the direct supervision of the soils engineer.

It is recommended that a soils engineer observe the foundation excavations prior to placement of footings. If driven pile or drilled shaft foundations are used, it is requested that we be present during the initial driving/drilling, and that we have the opportunity to observe all test results.

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

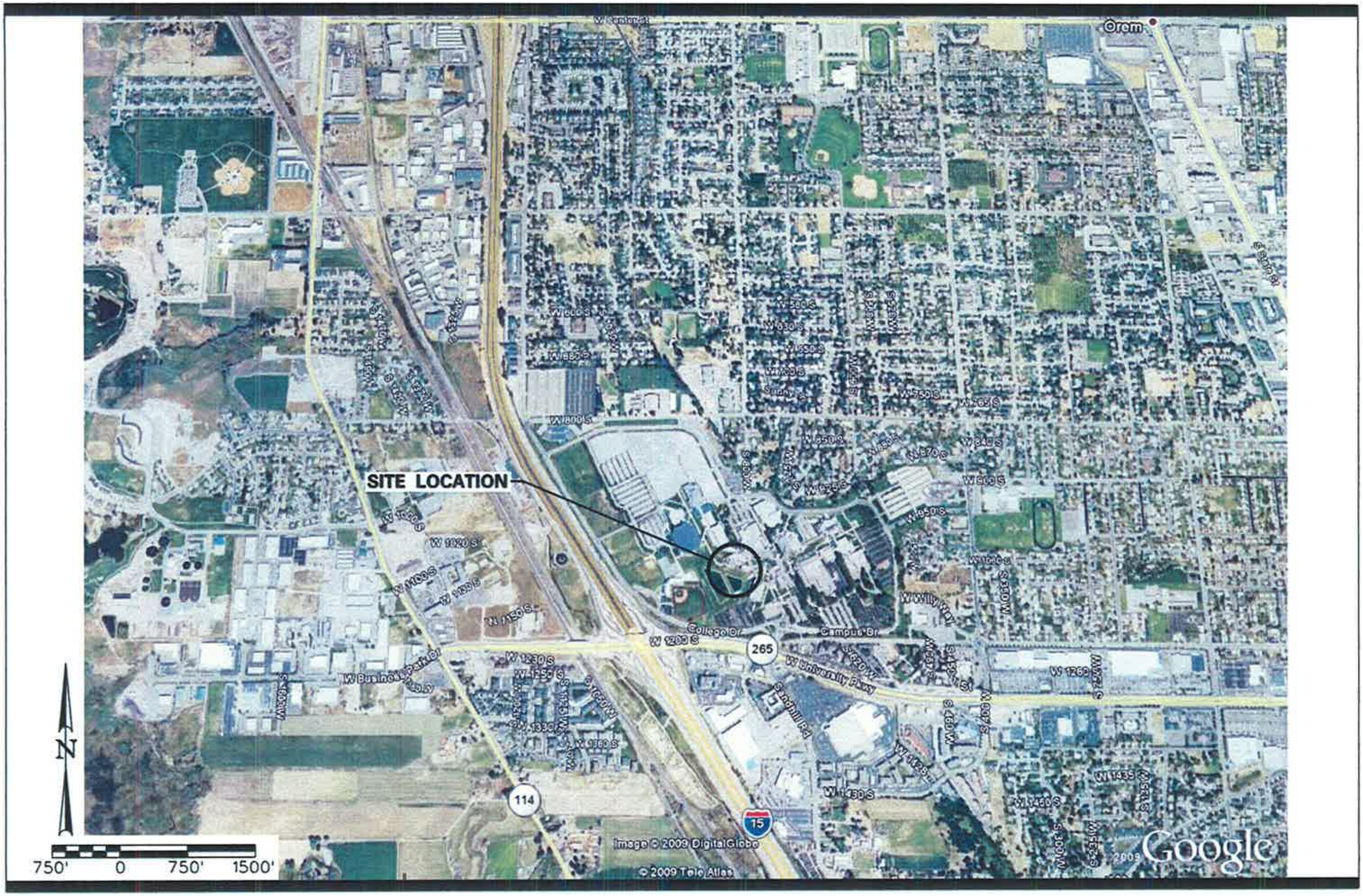

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Figure 1 **VICINITY MAP**

UVU - Science Building Addition

Orem, Utah

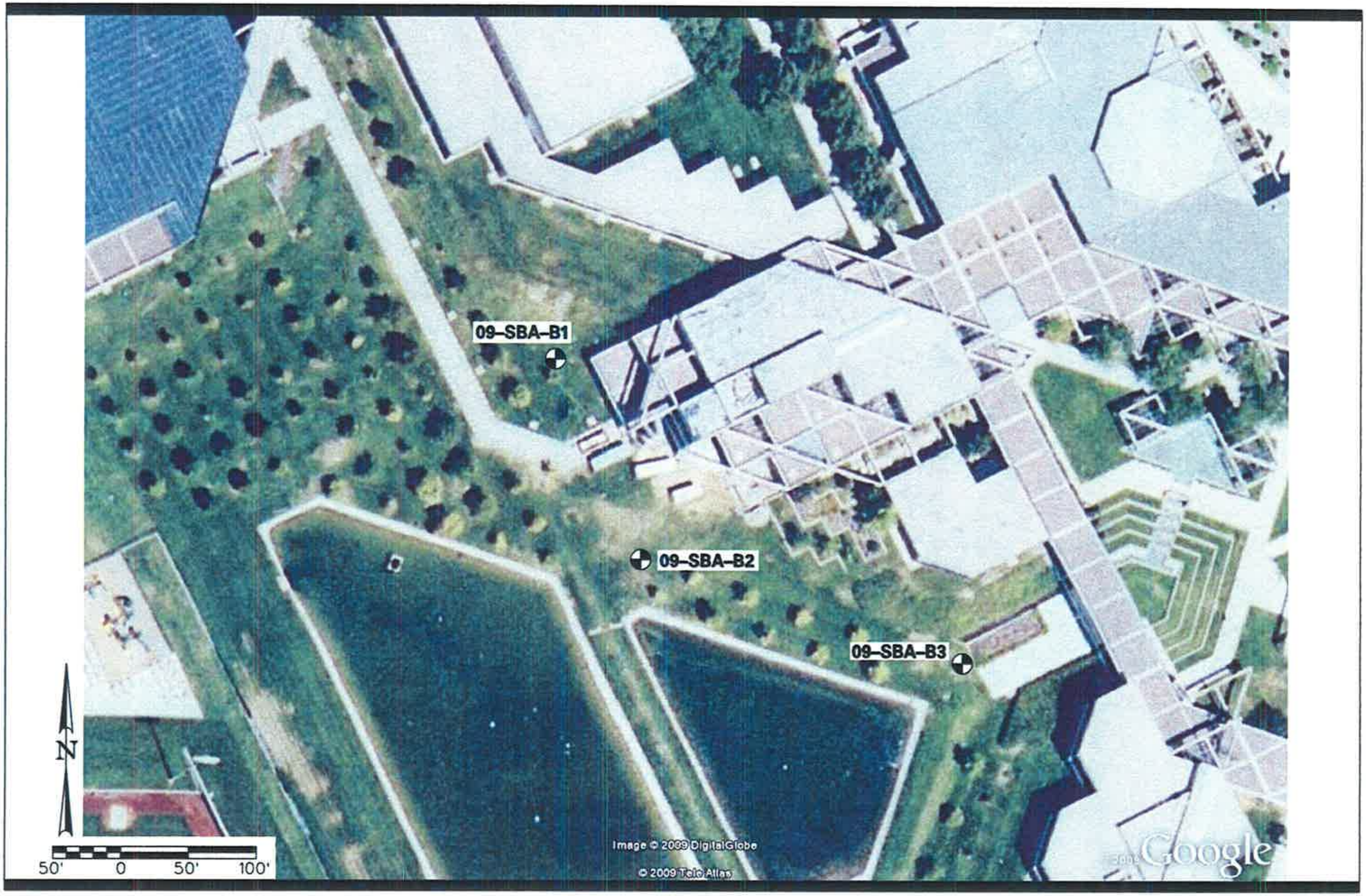

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Figure 2 **SITE PLAN & TEST HOLE LOCATIONS**

UVU - Science Building Addition

Orem, Utah

Appendix

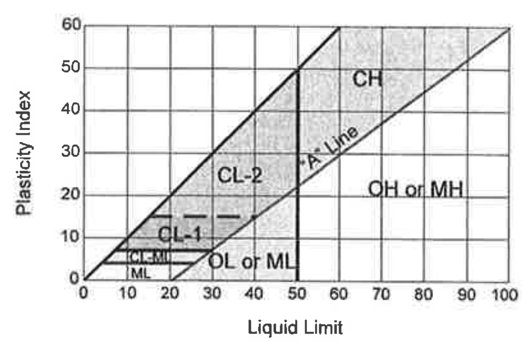
Unified Soil Classification System

Major Divisions			Group Symbols		Typical Names
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
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels With Fines *appreciable amount of fines*	**GM***	**d**	Silty gravels, poorly graded gravel-sand-silt mixtures
				u	
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures
	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
			SP		Poorly graded sands, gravelly sands, little or no fines
		Sands with Fines *appreciable amount of fines*	**SM***	**d**	Silty sands, poorly graded sand-silt mixtures
				u	
			SC		Clayey sands, poorly graded sand-clay mixtures
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
			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

Laboratory Classification Criteria

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	
Determine percentage of gravel and sand from grain-size curve.	Not meeting all gradation requirements for GW	
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
	Atterberg limits above "A" line, or PI greater	
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	
More than 12% GM, GC, SM, SC	Not meeting all gradation requirements for SW	
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
	Atterberg limits above "A" line, or PI greater	

For laboratory classification of fine-grained 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-SBA-B1

SHEET 1 OF 2

PROJECT: UVU - SCIENCE BUILDING ADDITION
CLIENT: DFCM
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING TO 18.5'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ▽ 6.0'   AFTER 24 HOURS: ▼ N.M.

PROJECT NUMBER: 200901.022
DATE STARTED: 5/7/09
DATE COMPLETED: 5/7/09
GROUND ELEVATION: 100.4'*
LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Sample Type	Rec. (in)	See Legend	USCS		Material Description	Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
100		Disturbed	16	5,7,8,(31)	SM	brown, moist, dense	Organics in top 6"		16.0		NP	8	46	46	
		Disturbed	16	4,2,5,(16)	SM	brown, very moist, med. dense	SILTY SAND		20.7		NP	0	61	39	
95 ▽	5														
		Pushed	15	Pushed	SM	brown, wet		93.5	28.3		NP	0	55	45	UC
		Disturbed	18	1,1,3,(8) 0.14	CL	brown, very moist, soft									
90	10	Pushed	14	Pushed 0.28	CL-2	brown, moist, firm		86.5	34.2	39	19				CT UC
		Pushed	16	Pushed 0.38	CL	brown, moist, firm									
85	15	Disturbed	18	1,2,2,(7) 0.30	CL-1	brown, moist, firm	LEAN CLAY W/SAND silt lenses		34.3	35	12				
80	20	Pushed	15	Pushed 0.57	CL-1	brown, moist, stiff		101.2	25.4	26	8				
75	25	Disturbed	15	3,3,7,(14) 0.40	CL	brown, moist, firm	LEAN CLAY W/SAND sand layers to 1" thick								
70	30	Pushed	15	0.36 Pushed	CL SM	brown, moist, firm / brown, wet	SILTY SAND								
		Disturbed	18	3,7,8,(19) 0.61	CL	brown, moist, stiff									
65	35	Disturbed	18	3,6,7,(16) 0.86	CL	brown, moist, stiff	LEAN CLAY W/SAND numerous silt and/or sand lenses & layers to 1/2" thick								
60	40	Pushed	16	Pushed 0.52	CL,SM	brown, moist/wet, stiff/med. dense	INTERBEDDED LEAN CLAY & SILTY SAND LAYERS 4"-8" THICK								
		Disturbed	18	8,13,11,(28) 0.54	CL,SM	brown, moist/wet, stiff/med. dense									
							SAND W/SILT occasional clay layers to 1 1/2" thick								

DH_LOGV6 UVUSCIENCEBLDGADD.GPJ US EVAL.GDT 5/29/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-SBA-B1

PROJECT: UVU - SCIENCE BUILDING ADDITION
CLIENT: DFCM
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING TO 18.5'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ∇ 6.0' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.022
DATE STARTED: 5/7/09
DATE COMPLETED: 5/7/09
GROUND ELEVATION: 100.4'*
LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample: Type	Sample: Rec. (in)	Sample: See Legend	Sample: USCS	Material Description	Dry Density (pcf)	Moisture Content (%)	Atter.: Liquid Limit	Atter.: Plast. Index	Gradation: Gravel (%)	Gradation: Sand (%)	Gradation: Silt/Clay (%)	Other Tests
55			■	6	15,20,20,(45)	SP-SM	gray, wet, dense								
50	50		■	15	12,21,21,(45)	SP-SM	brown, wet, dense								
45	55		■	6	22,27,30,(59)	SP-SM	gray, wet, very dense								
							SAND W/SILT occasional clay layers to 1 1/2" thick								
40	60		■	18	15,17,14,(31)	SP-SM	gray, wet, dense								
35	65														
30	70		■	16	21,7,11,(17) 0.37	SP-SM CL	gray, wet, med. dense gray, moist, firm — LEAN CLAY sand lenses								
25	75		■	14	21,37,39,(68)	SP-SM	gray, wet, very dense SAND W/SILT clay lenses								
20	80		■	17	30,31,18,(43)	SP-SM	gray, wet, dense								
							*Assumed finish floor elevation of 100.0' at west entance of restricted lab area, grass level								
15	85														

DH_LOGV6 UVUSCIENCEBLDGADD.GPJ US EVAL.GDT 5/29/09

RB&G
ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE ■ 2,3,2 ← Blow Count per 6"
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

DRILL HOLE LOG

BORING NO. 09-SBA-B2

SHEET 1 OF 1

PROJECT: UVU - SCIENCE BUILDING ADDITION
CLIENT: DFCM
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING TO 15'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ∇ N.M. **AFTER 24 HOURS:** ▼ 3.3'

PROJECT NUMBER: 200901.022
DATE STARTED: 5/7/09
DATE COMPLETED: 5/7/09
GROUND ELEVATION: 99.6' *
LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Type	Rec. (in)	See Legend	USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plast. Index	Gravel (%)	Sand (%)	Silt/Clay (%)	Other Tests
		Disturbed	17	9,12,9,(44) 0.28	SM	brown, moist, dense	SILTY SAND								
					ML	brown, moist, firm	SILT W/SAND plastic								
		Pushed	9	Pushed 0.66	ML	brown, moist, stiff		93.0	30.1	32	8				UC
95	5														
		Pushed	16	Pushed 0.53	CL-2	brown, moist, stiff	LEAN CLAY W/SAND occasional silt lenses	93.6	29.1	37	17				CT
90	10	Pushed	18	Pushed 0.35	CL-1	brown, moist, firm		92.9	32.3	35	14				UC
		Disturbed	17	1,2,2,(8) 0.64	CL	brown, moist, stiff									
85	15	Disturbed	18	0,3,3,(11) 0.32	CL-2	brown, moist, firm			27.6	37	18				
80	20	Pushed	18	Pushed 0.24	SM	brown, wet	SILTY SAND								
					CL	brown, moist, soft	LEAN CLAY frequent silt and/or sand lenses								
75	25	Disturbed	18	4,5,6,(17) 0.61	CL	brown, moist, stiff									
70	30	Pushed	15	Pushed 0.50	CL	brown, moist, stiff	LEAN CLAY sand layers to 1 1/2" thick								
		Disturbed	18	4,8,8,(22) 0.69	CL	brown, moist, stiff									
65	35						*Assumed finish floor elevation of 100.0' at west entance of restricted lab area, grass level								

DH_LOGV6 UVUSCIENCEBLDGADD.GPJ US EVAL.GDT 5/29/09

LEGEND:

DISTURBED SAMPLE 2,3,2 ← Blow Count per 6" ; 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

RB&G ENGINEERING, INC.

DRILL HOLE LOG

BORING NO. 09-SBA-B3

SHEET 1 OF 2

PROJECT: UVU - SCIENCE BUILDING ADDITION
CLIENT: DFCM
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING TO 15'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ∇ 3.9' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.022
DATE STARTED: 5/8/09
DATE COMPLETED: 5/8/09
GROUND ELEVATION: 100.5'*
LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample Type	Rec. (in)	See Legend	USCS	Field Description	Material Description	Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
100						SM		SILTY SAND organics (grass)								
						-	brown, moist	4" CONCRETE								
			Disturbed	15	10,8,5,(27)	GM	brown, moist, med. dense	SILTY GRAVEL W/SAND								
						SM										
						SM	brown, very moist	SILTY SAND								
∇			Disturbed	9	3,1,3,(9) 0.30	ML	brown, very moist, firm	SILT W/SAND plastic		30.7	30	7				
95	5															
			Pushed	17	Pushed 0.18	CL-2	brown, moist, soft		87.1	36.1	42	20				UC
90	10		Pushed	17	Pushed 0.37	CL-1	brown, very moist, firm		87.2	32.7	32	11				CT UC
			Disturbed	17	1,2,3,(10) 0.58	CL	brown, moist, stiff									
85	15		Disturbed	18	0,4,4,(15) 0.33	CL-1	brown, moist, firm	LEAN CLAY W/SAND silt lenses, occasional sand layers to 1/2" thick		26.8	32	13				
80	20		Pushed	18	Pushed 0.58	CL-1	brown, moist, stiff			28.7	31	11				
75	25		Disturbed	18	0,3,2,(8) 0.40	CL	brown, moist, firm									
70	30		Pushed	17	Pushed 0.71	CL-1	brown, moist, stiff		95.1	23.1	31	12				CT
65	35		Disturbed	16	4,7,5,(16) 0.44	CL	brown, moist, firm	LEAN CLAY frequent silt lenses, occasional sand layers to 1" thick - becoming more frequent w/depth								
60	40		Pushed	15	Pushed 0.60	CL-1	brown, moist, stiff		101.6	22.9	29	8				CT
								SILTY SAND few clay layers to 1" thick								

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE ■ 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-SBA-B3

SHEET 2 OF 2

PROJECT: UVU - SCIENCE BUILDING ADDITION
CLIENT: DFCM
LOCATION: SEE SITE PLAN
DRILLING METHOD: 96-CME-55 / N.W. CASING TO 15'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ▽ 3.9' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.022
DATE STARTED: 5/8/09
DATE COMPLETED: 5/8/09
GROUND ELEVATION: 100.5'*
LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample: Type	Sample: Rec. (in)	Sample: See Legend	Sample: USCS	Material Description	Dry Density (pcf)	Moisture Content (%)	Atter.: Liquid Limit	Atter.: Plast. Index	Gradation: Gravel (%)	Gradation: Sand (%)	Gradation: Silt/Clay (%)	Other Tests
55			■	18	16,10,9,(22)	SM	brown, wet, med. dense — SILTY SAND few clay layers to 1" thick		25.9		NP	0	62	38	
50	50		■	0	21,23,24,(52)	-	no recovery								
45	55		■	15	9,11,25,(38)	SP-SM	gray, wet, dense — SAND W/SILT occasional clay lenses & layers to 1" thick								
40	60		■	14	18,18,20,(38)	SP-SM	gray, wet, dense								
35	65														
30	70		■	12	17,24,30,(51)	SP-SM	gray, wet, very dense — SAND W/SILT clay lenses								
25	75														
20	80		■	12	29,33,36,(61)	SP-SM	gray, wet, very dense								
15	85														

*Assumed finish floor elevation of 100.0' at west entance of restricted lab area, grass level

DH_LOGV6 UVUSCIENCEBLDGADD.GPJ US EVAL.GDT 5/29/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

Table 1

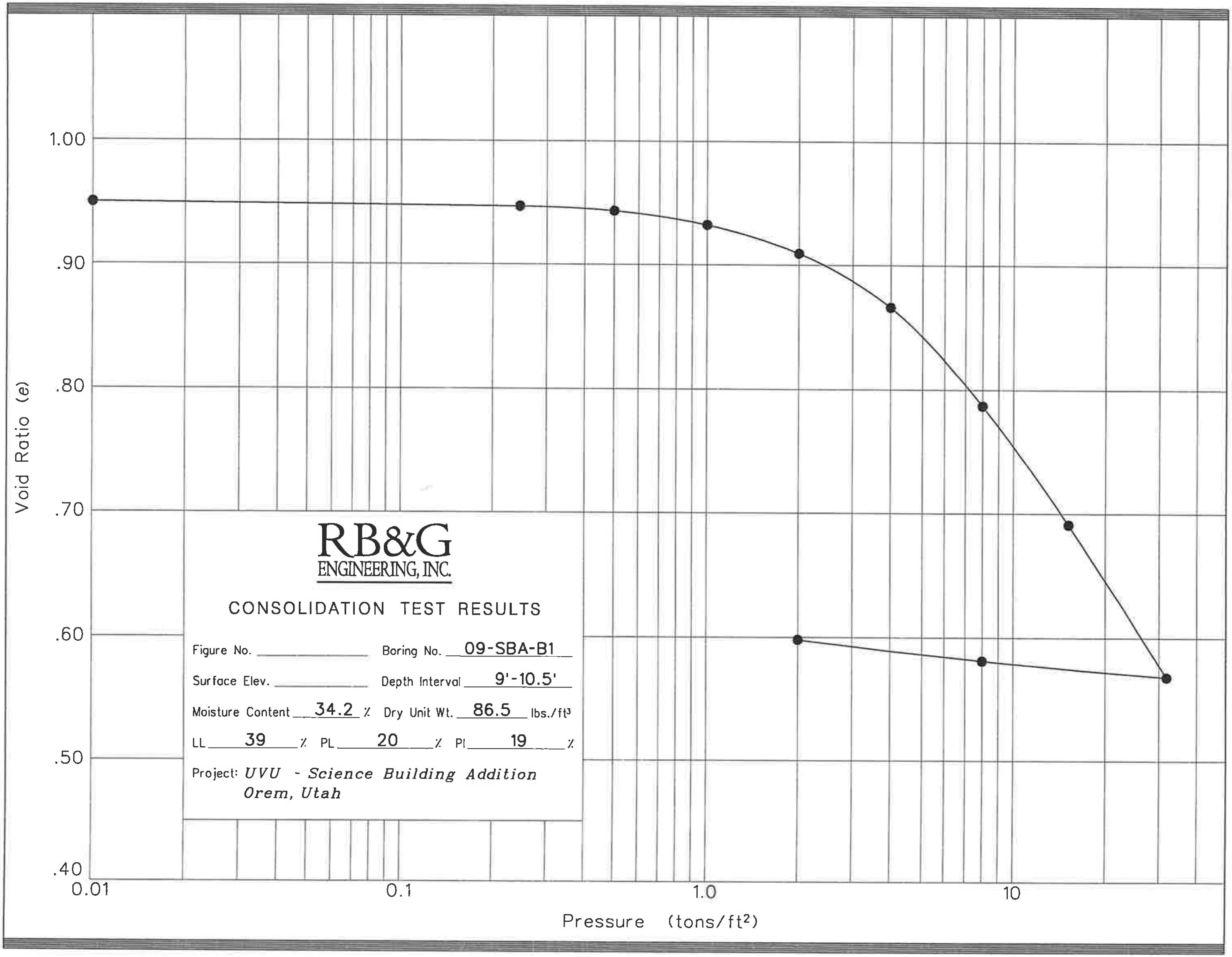
SUMMARY OF TEST DATA

PROJECT	UVU Science Building Addition	PROJECT NO.	200901-022
LOCATION	Orem, Utah	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-SBA-B1	0-1.5	31		16.0				NP	8	46	46	SM
	3-4.5	16		20.7				NP	0	61	39	SM
	6-7.5	Shelby	93.5	28.3	763			NP	0	55	45	SM
	9-10.5	Shelby	86.5	34.2	1076	39	20	19				CL-2
	15-16.5	7		34.3	1200*	35	23	12				CL-1
	20-21.5	Shelby	101.2	25.4	2280*	26	18	8				CL-1
09-SBA-B2	3-4.5	Shelby	93.0	30.1	1976	32	24	8				ML
	6-7.5	Shelby	93.6	29.1	2120*	37	20	17				CL-2
	9-10.5	Shelby	92.9	32.3	1124	35	21	14				CL-1
	15-16.5	11		27.6	1280*	37	19	18				CL-2
09-SBA-B3	3-4.5	9		30.7	1200*	30	23	7				ML
	6-7.5	Shelby	87.1	36.1	526	42	22	20				CL-2
	9-10.5	Shelby	87.2	32.7	1448	32	21	11				CL-1
	15-16.5	15		26.8	1320*	32	19	13				CL-1
	20-21.5	Shelby		28.7	2320*	31	20	11				CL-1
	30-31.5	Shelby	95.1	23.1	2840*	31	19	12				CL-1
	40-41.5	Shelby	101.6	22.9	2400*	29	21	8				CL-1
	45-46.5	22		25.9				NP	0	62	38	SM

		Resistivity (ohm cm)	pH	Sulfate (%)	Chloride Mg/Kg-dry
09-SBA-B1	3-4.5	2600	7.6	<0.0005	41
09-SBA-B3	3-4.5	1500	8.1	0.011	740

NP=Nonplastic
Torvane value used to estimated unconfined compressive strength.



RB&G ENGINEERING, INC.

CONSOLIDATION TEST RESULTS

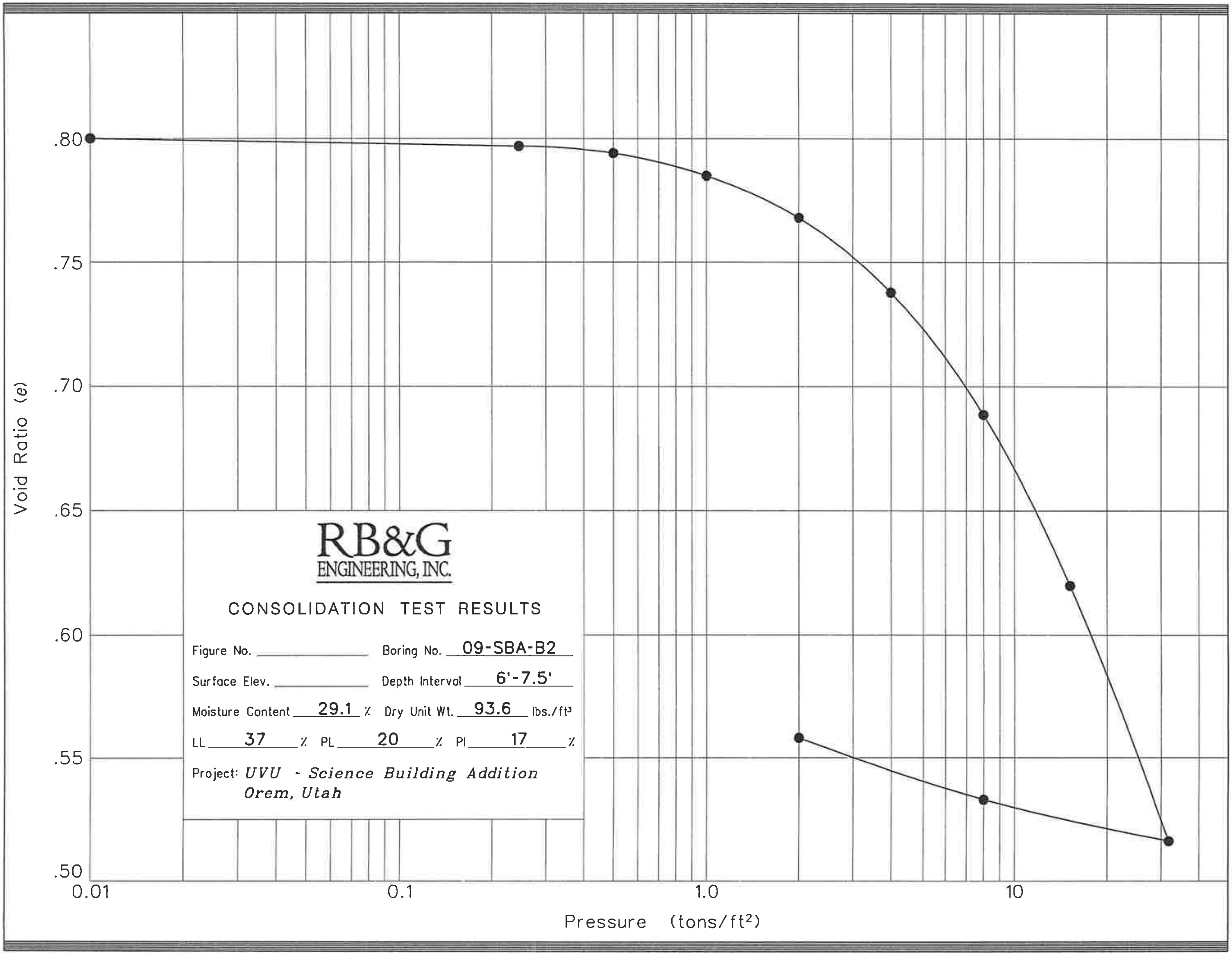
Figure No. ______ Boring No. 09-SBA-B1

Surface Elev. ______ Depth Interval 9'-10.5'

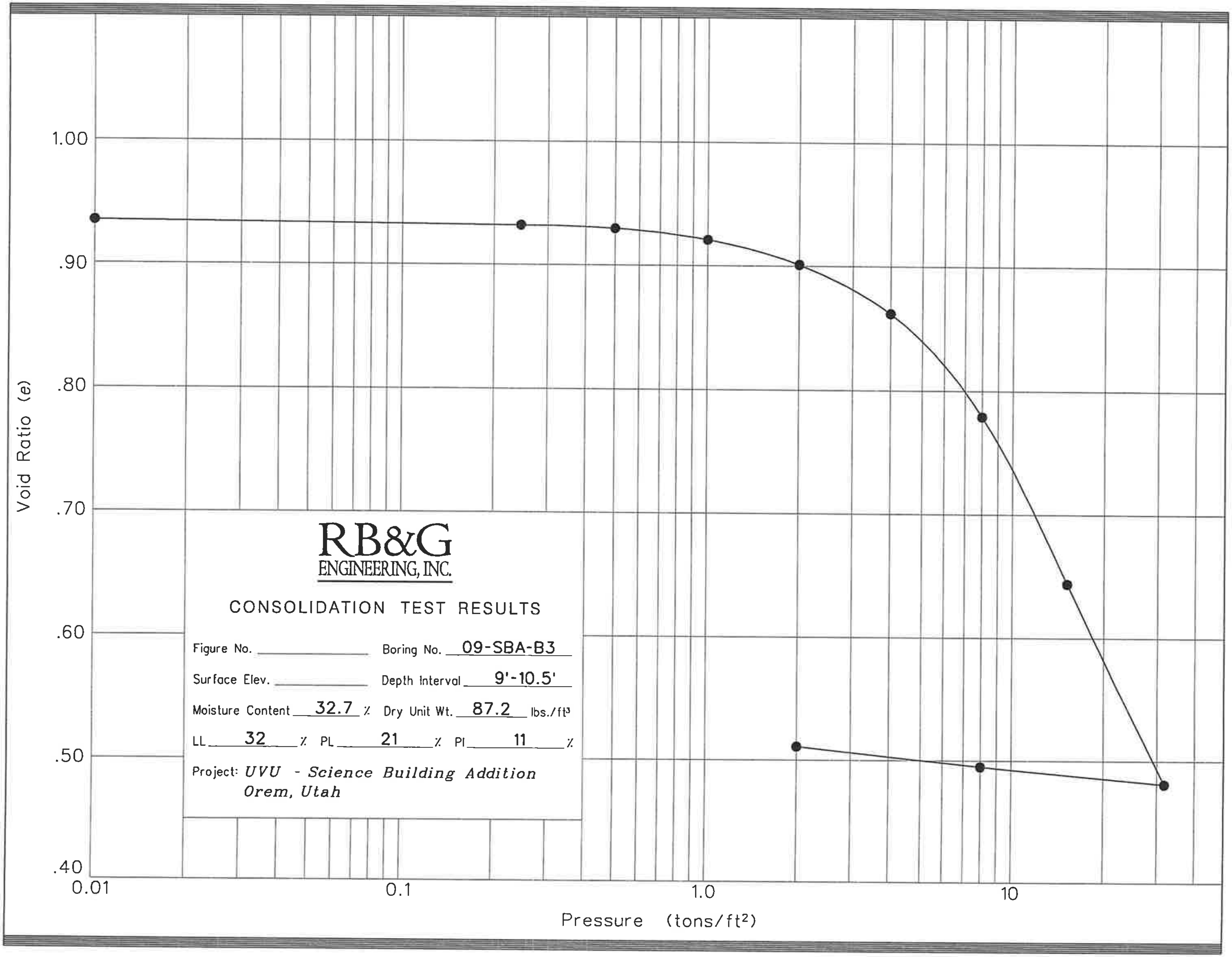
Moisture Content 34.2 % Dry Unit Wt. 86.5 lbs./ft³

LL 39 % PL 20 % PI 19 %

Project: *UVU - Science Building Addition*
Orem, Utah



RB&G
ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-SBA-B2
Surface Elev.
Depth Interval 6'-7.5'
Moisture Content 29.1 %
Dry Unit Wt. 93.6 lbs./ft³
LL 37 % PL 20 % PI 17 %
Project: UVU - Science Building Addition
Orem, Utah
Void Ratio (e)
.80
.75
.70
.65
.60
.55
.50
0.01
0.1
1.0
10
Pressure (tons/ft²)



RB&G ENGINEERING, INC.

CONSOLIDATION TEST RESULTS

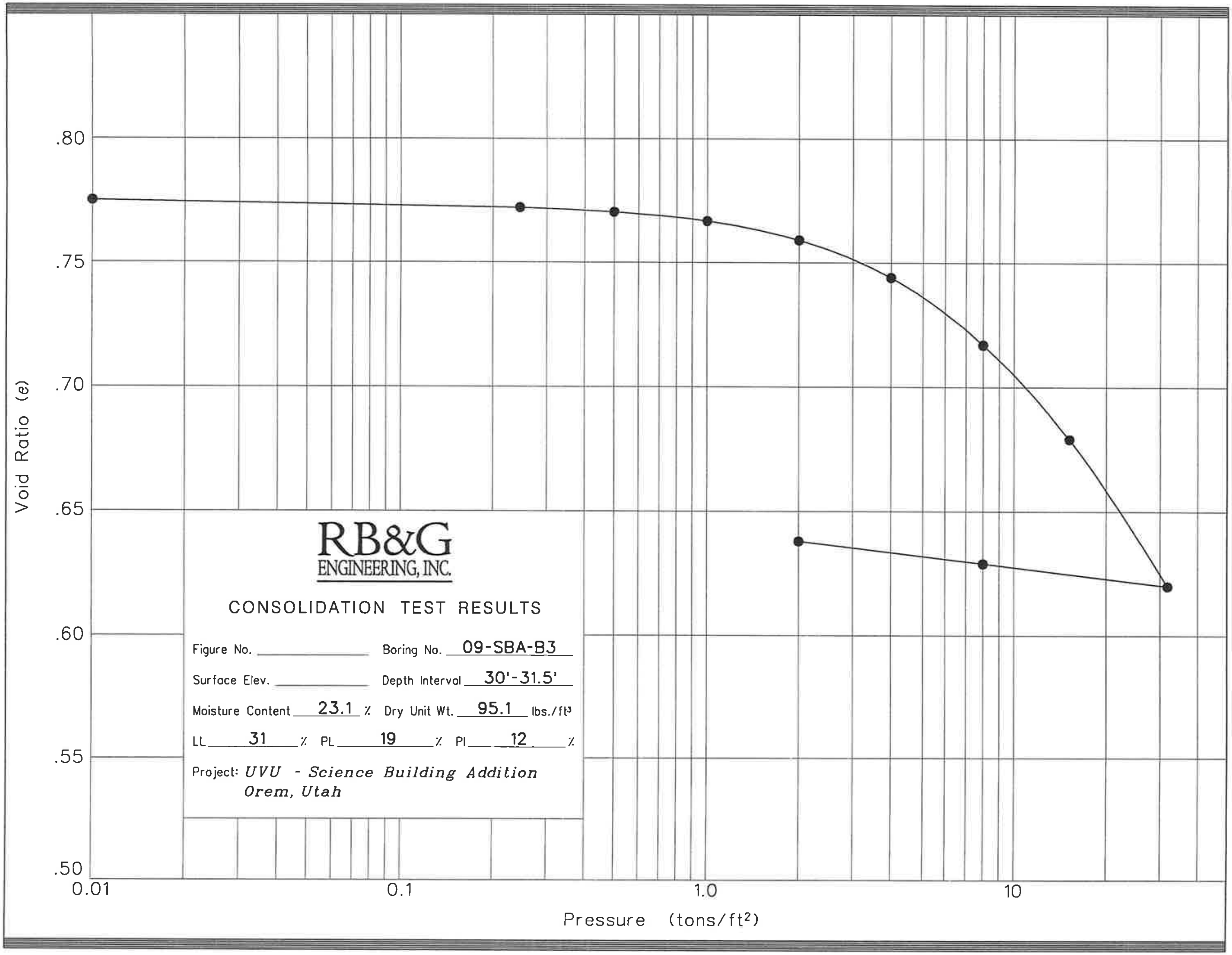
Figure No. ______ Boring No. 09-SBA-B3

Surface Elev. ______ Depth Interval 9'-10.5'

Moisture Content 32.7 % Dry Unit Wt. 87.2 lbs./ft³

LL 32 % PL 21 % PI 11 %

Project: *UVU - Science Building Addition*
Orem, Utah



RB&G ENGINEERING, INC.

CONSOLIDATION TEST RESULTS

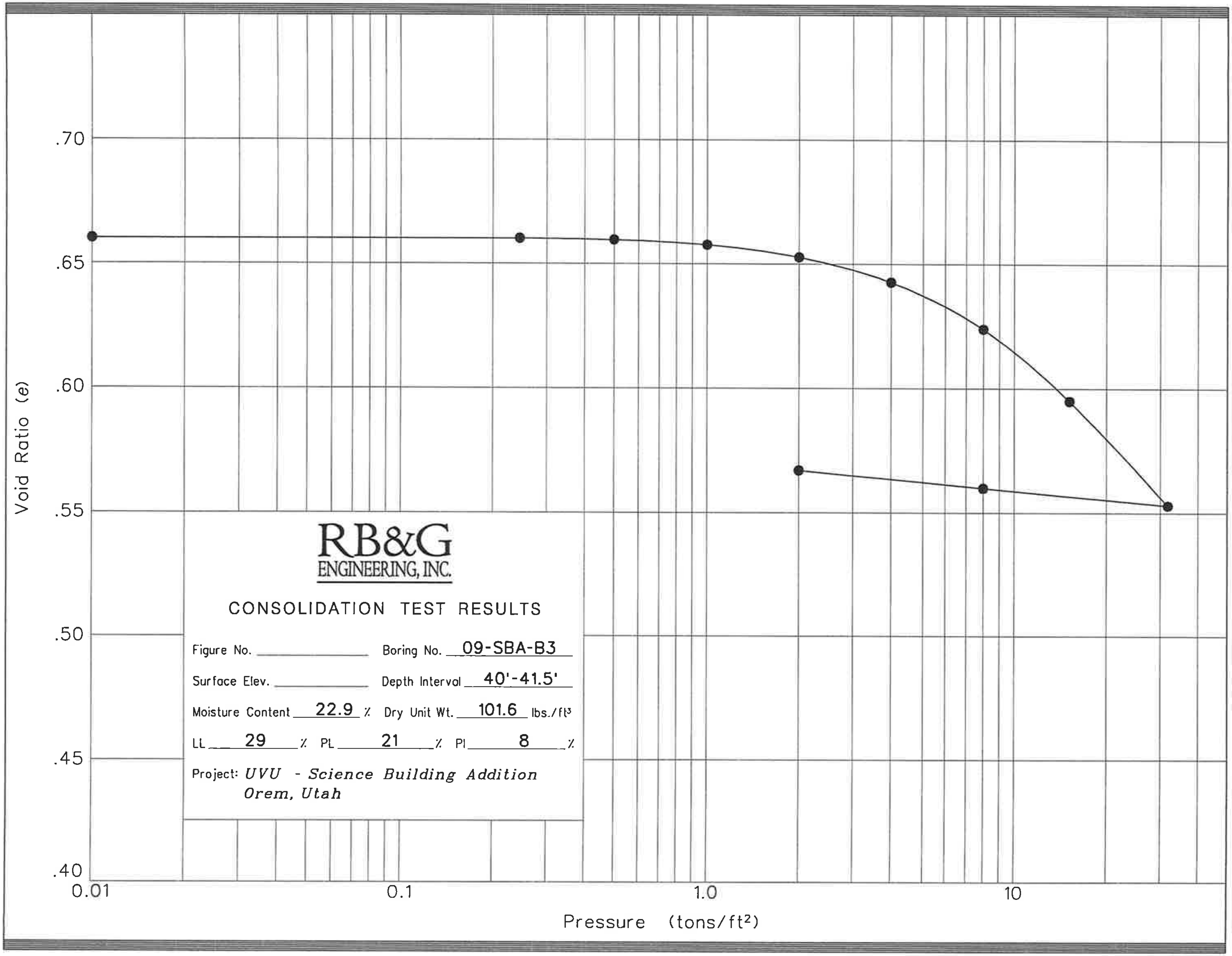
Figure No. ______ Boring No. 09-SBA-B3

Surface Elev. ______ Depth Interval 30'-31.5'

Moisture Content 23.1 % Dry Unit Wt. 95.1 lbs./ft³

LL 31 % PL 19 % PI 12 %

Project: *UVU - Science Building Addition*
Orem, Utah


Void Ratio (e)
.70
.65
.60
.55
.50
.45
.40
0.01
0.1
1.0
10
Pressure (tons/ft²)
RB&G ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No. ______ Boring No. 09-SBA-B3
Surface Elev. ______ Depth Interval 40'-41.5'
Moisture Content 22.9 % Dry Unit Wt. 101.6 lbs./ft³
LL 29 % PL 21 % PI 8 %
Project: UVU - Science Building Addition
Orem, Utah

GEOTECHNICAL INVESTIGATION

UVU POPE SCIENCE BUILDING ADDITION

Orem, Utah

Prepared for: DFCM

November 2009

RB&G ENGINEERING, INC.

200901-051

RB&G
ENGINEERING, INC.

November 30, 2009

Dave McKay
DFCM
4110 State Office Building
Salt Lake City, UT 84114

Re: Utah Valley University
Pope Science Building Addition
DFCM Project No. 0920790/Contract No. 097429, Mod. 1
Geotechnical Investigation

Dear Mr. Bankhead:

A Geotechnical Investigation has been completed for the proposed addition to the Pope Science Building located on the UVU 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

Bradford E. Price, P.E.

REGISTERED PROFESSIONAL ENGINEER
NO.162291
BRADFORD E.
PRICE
12/2/09
STATE OF UTAH

bep/jal

Geotechnical Investigation

UVU
Pope Science
Building Addition

Orem, Utah

Prepared for: DFCM

November 2009

RB&G ENGINEERING, INC.

RB&G
ENGINEERING, INC.

UVU
POPE SCIENCE
BUILDING NEW ADDITION
Orem, Utah

Geotechnical Investigation

INTRODUCTION

This report outlines the results of a geotechnical investigation performed for the proposed new addition to the Pope Science Building located on the Utah Valley University (UVU) Campus in Orem, Utah. The site location in relation to the surrounding area is shown on the Vicinity Map in Figure 1, and the Site Plan is presented in Figure 2. 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.

RB&G Engineering performed geotechnical investigations for the Science Building in 1986 and 1987. RB&G also performed a geotechnical investigation for a proposed addition to the Science Building in May 2009. Applicable information from these investigations has been used during this study.

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 sand deposits laid down during the Provo regressive phase of the Bonneville lake cycle (upper Pleistocene). The Wasatch Fault Zone is located approximately 3 miles east of the site. Utah County Natural hazards maps identify this area as having moderate liquefaction potential.

The area where the proposed addition will be located is presently landscaped in lawn grass with some trees and shrubs. The topography west of the walkway shown in Figure 2 and in the vicinity of Borings 09-SBA-B1, 09-2, 3, 4, 5, and 6 is relatively flat at approximately elevation 88 feet. The topography slopes upward to the east at about 4 horizontal to 1 vertical to elevation ~103 feet in the vicinity of Borings 09-3 and 09-1, as shown in Figure 2.

Structures in the immediate vicinity are supported using spread foundations on compacted fill. Foundations appear to be performing in a satisfactory manner, in that no significant cracking was observed in foundation walls.

As shown in Figure 2, lined detention ponds are located immediately south of the proposed addition. 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 two- to three-foot intervals in the upper 15 feet and at about 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 five borings to depths of between 30 and 45 feet and one boring to 100 feet at the approximate locations shown in Figure 2. In addition, one boring from the May 2009 study was located at the southerly end of the site at the location shown in Figure 2, extending to a depth of 81 feet. Borings 09-1 and 09-3 were located at about elevation 103 feet, with the remainder of the borings at about elevation 88 feet. The logs for the borings are presented in the appendix.

It will be noted from the boring logs that the subsurface profile consists of silty sand and sandy silt layers to depths varying from 7.5 to 25 feet below the surface, followed predominately by lean clay extending to between 45 and 50 feet, followed by silty sand and sand layers to 100 feet. Several of the silt and sand deposits in the upper 25 feet are in a loose to medium dense state and are susceptible to liquefaction during a seismic event. The silt and sand layers deeper in the soil profile range from dense to very dense and will not liquefy during a seismic event.

Groundwater was encountered at a depth of about 8 feet below the ground surface in the holes drilled at the 88 foot level (approx. elevation 80 feet), and 15 to 16 feet below the ground surface in the borings drilled at the 103 foot level on the east side (approx. elevation 87 to 88 feet). It will be noted that the borings were drilled in November 2009, when groundwater is nearing its seasonal low. Up to 2 foot rise in the groundwater level should be expected due to seasonal and precipitation fluctuations.

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 silty sand (SM) material has 19 to 49% non-plastic fines and that the silt and sandy silt layers (ML) tested have 7 to 39% sand size particles. The in-place dry unit weight of the cohesive soils varies from 87.3 to 98.9 pcf, and that the natural moisture content of the cohesive material ranges from 26.0 to 34.9%. The unconfined compressive strength of the cohesive soil varies from 612 to 1735 psf, with an average of 1256 psf.

The compressibility characteristics of the clay material were evaluated by performing seven consolidation tests on samples obtained from Boring 1 at a depth of 25 feet, Boring 2 at 20 feet, Boring 3 at 30 and 40 feet, Boring 4 at 9 feet, and Boring 6 at depths of 12 and 25 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. It will be noted that the clay is over-consolidated with relatively low compressibility characteristics for load intensities less than 3 tsf.

In order to obtain an indication of the corrosive nature of the subsurface material at this site, resistivity, pH, sulfate and chloride tests were performed on samples obtained in Borings 2 and 6 at a depth of 3 to 4.5 feet below the ground surface. It will be noted that the silty sand and sandy silt material has a pH of 7.9, resistivity of 6100 and 3450 ohm cm, chlorides of 5.7 and 14 mg/kg, and sulfates of 14 and 35 mg/kg, respectively.

These soils have low percentages of water soluble sulfate. While Type I or Type II cement is acceptable, it is recommended that Type II cement be used for concrete in contact with the native soils due to its increased resistance to sulfate attack.

IV. FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

A. FOUNDATION TYPES AND BEARING CAPACITIES

We understand that the proposed addition will be 3 stories with the lower floor level at elevation 88 ft. The magnitude of structural loads is not known as of the preparation of this report; however, it is our understanding that the column loads will be in the order of 350 to 400 kips and that wall loads will not likely exceed 8 klf.

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.

At stated previously, groundwater was encountered at about elevation 80 feet in the westerly borings drilled at the 88 foot level and at 87 to 88 feet in the easterly borings located at the 103 foot level. The silty sand and sandy silt layers below the groundwater level and above the clay layers are loose to medium dense and will liquefy during the design seismic event, as discussed in Section 4B below. The thickness of liquefiable layers varies from 2 to 10 feet. Liquefaction will result in a loss of shear strength, settlement and potential lateral spread displacement. It is recommended that no footings be supported directly on material susceptible to liquefaction. We recommend that either ground modification techniques be used to mitigate liquefaction potential, or that the foundation support extend below the liquefiable layers.

Ground modification techniques include (1) removal and replacement, (2) densification, (3) reinforcement and (4) hardening and mixing.

The removal and replacement option can be performed by conventional means. This will require dewatering, which can best be performed using well points. Material from the required excavation can be used as backfill, provided that it is moisture conditioned, placed in lifts not exceeding 8 inches in thickness, and compacted to at least 95% of the maximum laboratory density as determined by ASTM D 1557. The 2 to 8 feet of silty sand and sandy silt below the groundwater level is wet of optimum and will require spreading and drying to achieve optimum.

Due to the high amount of fines, vibro-replacement bottom feed stone columns/aggregate piers provide a good combined densification/reinforcement and replacement option.

Extending foundation support below the liquefiable layers mitigates loss of shear strength and settlement concerns; however, the potential for lateral spread displacement remains. As discussed in Section 4B, lateral displacement in the order of 2 to 3 feet has been estimated. It is recommended that, if final design results in extending the foundation support below the liquefiable layers, sufficient ground modification be performed to mitigate the lateral spread concern.

Foundation support options considered during preparation of this report include (1) spread footings on compacted fill, (2) spread footings on stone columns/aggregate piers, (3) mat foundation, and (4) deep foundations. Each of these foundation types are discussed separately below:

1. Spread Footings on Compacted Fill

This option requires removing the liquefiable soils below the water level to elevation 72 ft. or until lean clay is encountered, whichever comes first. The width of the excavations should be equal to at least the footing width plus the depth of fill placed below the footing. For example, a 10 ft. square footing placed on 10 feet of compacted fill requires the footing excavation to be 20 feet wide. With column spacing in the order of 20 by 30 ft., it will likely be more efficient to excavate the entire footprint. Material from the required excavation can be used as backfill, provided that it is moisture conditioned and compacted to the requirements stated above. It is expected that the saturated sandy silt layers will be difficult to condition.

Assuming the footing elevations to be at about elevation 85 feet results in excavation of at least 10 feet of silty sand and sandy silt throughout most of the site. Two borings (09-1 and SBA-1 encountered clay at elevation 80 feet. It is recommended that all footing areas be excavated to at least elevation 75 feet, with the excavation width extending at least 5 feet beyond the footing perimeter.

It is also recommended that 2 feet of structural fill be placed directly beneath all footings. The structural fill should extend 1 foot beyond the footing perimeter and should consist of relatively well graded sandy gravel having a maximum size of 3 inches with less than 15% passing the No. 200 sieve. The granular fill should be compacted to at least 95% of the maximum density as determined by ASTM D 1557.

If the above recommendations are complied with, footings can be designed using the bearing capacity charts shown in Figures 3 and 4. Figure 3 is applicable for continuous footings, while Figure 4 is valid for spot footings. It will be noted from these figures that the allowable soil bearing pressure is a function of the width of the footing, and that the bearing pressure decreases as the footing width increases. This condition occurs since the portion of the zone of significant stress within the cohesive material increases with increased footing width.

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 criteria is tantamount to a differential settlement of about 0.5 inch for column spacing's of 20 feet and 0.7 inch for column spacing's 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.

2. Spread Footings on Stone Columns/Aggregate Piers

It is anticipated that an allowable bearing capacity in the order of 5000 psf can be achieved by extending vibro-replacement stone columns or rammed aggregate piers to elevation 70 feet beneath all footings covering the footing area plus a lateral distance of 8 feet on each side of the footing. We recommend that at least 2 feet of structural fill be placed beneath the footings to provide uniform support across the pier system. This option mitigates both liquefaction and lateral concerns with respect to structural support. The lower floor slab, however, will remain susceptible to liquefaction induced settlement in the order of 1.5 to 2.5 inches during the design seismic event.

Design and installation of the pier system should be completed by a specialty contractor with a minimum of 5 years experience. The system plans should be designed and stamped by a registered professional engineer. We recommend that at least 2 load tests be performed during construction following procedures outlined in ASTM D-1143. It is recommended that the geotechnical engineer review the specialty contractor's design and provide Quality Assurance during installation and modulus testing.

3. Mat Foundation

If the excavation and replacement method is used to mitigate liquefaction and lateral spread, a mat foundation may be an efficient option to provide uniform distribution of soil pressure under the foundation.

Settlement analyses have been performed assuming net allowable bearing capacities of 800 psf and 1200 psf for the mat foundation. A total estimated consolidation settlement at the center of the mat of 0.91 inches for the 800 psf loading and 1.26 inches for the 1200 psf loading has been computed. Differential settlement will be less than 0.6 inches, which should be entirely satisfactory for a mat type foundation.

It is recommended that a coefficient of subgrade reaction (k_1) of 100 lb/in^3 be used for the silty sand subgrade.

3. Deep Foundations

If the deep foundation option is used, sufficient ground modification should be performed to mitigate the lateral spread concern. Lateral spread mitigation will require installing 20 foot widths of stone column/aggregate piers at the east, center, and west end of the footprint.

Driven Piles

Consideration has been given to supporting the structure on driven piles extending to into the dense silty sand which was encountered between elevation 35 and 45 feet throughout the site. Axial compressive capacities for 16-inch (outside diameter) closed-end concrete-filled steel pipe piles are summarized on the following table.

Pile Toe Elevation (feet)	Ultimate Skin Friction (kips)	Ultimate End Bearing (kips)	Allowable Capacity Assuming Factor of Safety = 2.25 (kips)
30	196	150	152
25	252	181	190
20	317	212	232

Pile layouts should be designed with a minimum center-to-center spacing of 3 pile diameters between piles. It will be noted that a factor of safety of 2.25 has been used to calculate the allowable capacities. This factor of safety assumes that PDA testing will be performed during driving of one pile for columns located near the four corners and center of the structure. Pile uplift capacity can be estimated by using a factor of safety of 3 with the ultimate skin friction values. If this option is selected, pile lateral capacities, along with estimated pile group settlement, can be evaluated. It is anticipated that group settlement will be tolerable for column loads less than 600 kips. We recommend that the geotechnical engineer's representative be present during pile installation.

Drilled Shafts

Drilled shafts have also been considered as a foundation option for supporting the structure. It has been assumed that the shafts will be drilled at least 5 feet into the dense silty sand referenced in the Driven Pile section above. Procedures outlined in FHWA-H1-88-042, Drilled Shafts: Construction Procedures and Design Methods, have been used to determine the ultimate axial compressive capacity (nominal resistance) of drilled shafts. Capacity analyses have been performed for straight-sided drilled shafts using soil parameters obtained from the borings. If allowable stress design methods are used, we recommend that a factor of safety of 2.5 be applied to the ultimate capacity to determine the allowable capacity. It has been assumed that high quality construction, good specifications and excellent inspection will exist for each foundation. The estimated capacities of the drilled shafts can be taken from the table below.

Shaft Diameter (ft)	Ultimate Side Resistance (kips)	Ultimate End Resistance (kips)	Total Ultimate Capacity (kips)	Allowable Capacity (kips)
3	258	322	580	232
3.5	301	439	740	296
4	344	573	917	367
4.5	387	725	1112	445
5	403	895	1298	519

The allowable uplift resistance of a single drilled shaft may be taken as the ultimate side resistance value shown on the table above divided by a factor of safety of 3.0. A center-to-center spacing of at least three shaft diameters should exist to minimize interaction and overlapping stresses between shafts, which would result in reduced capacity.

The design of rebar and concrete should follow established guidelines. If the foundation recommendations presented above are followed, the maximum settlement of any drilled shaft should not be greater than about 1 inch. Due to the high water table and presence sand layers, drilling slurry or casing will likely be required for shaft excavation. Concrete should be placed by tremie methods to ensure that no voids exist within the shafts. Concrete used for shafts should have a relatively high slump (6 inches or greater) to allow workability and proper placement between reinforcement and the sides of the shafts. Within each shaft, concrete should be placed in a generally continuous manner to prevent cold joints and other problems associated with excessive waits between concrete trucks. It is essential that drilled shaft construction be carefully inspected to ensure that loose material is removed from the base and that the concrete is placed using proper procedures.

While all options discussed above will provide satisfactory support, it is our opinion that supporting the structure on stone columns or aggregate piers may be the most efficient design.

B. SEISMIC CONSIDERATIONS

The site is located at latitude 40.2781° North and longitude 111.7160° 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	17.71	50.52
0.2 sec SA	42.21	114.80
1.0 sec SA	14.25	48.43

Liquefaction/lateral spread analyses have been performed for the site assuming a seismic event having an acceleration of 0.34g, which is 2/3's of the event having a probability of exceedance of 2% in 50 years. The results of the analyses indicate that the following zones will liquefy and or spread during the design event:

Boring No.	Approximate Ground Elevation (feet)	Elevation of Liquefiable Zones (feet)	Elevation of Zones Susceptible to Lateral Spread (feet)
09-1	103	80 to 88	80 to 88
09-2	88	70 to 79 65 to 67 55 to 57	71 to 79 65 to 67
09-3	103	78 to 86	78 to 86
09-4	88	68 to 76 60 to 63	68 to 76
09-5	88	70 to 80	70 to 80
09-6	88	74 to 80	74 to 80
SBA-B1	88	80 to 82	n/a

The site is classified as Site Class F, as per Section 1613 of the 2006 International Building Code. Mitigating the liquefaction concern as recommended herein will allow the use of Site Class D for design.

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.

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.

Assuming a seismic event having an acceleration of 0.34g, which is two-thirds of the PGA for an event having a probability of exceedence of 2% in 50 years, the additional active earth pressure due to ground acceleration may be estimated using a coefficient of 0.19. 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 0.53. 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 0.34. 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.

V. SITE PREPARATION AND COMPACTED FILL REQUIREMENTS

As indicated above, the vegetative cover throughout the building site consists of lawn grass, shrubs and trees. 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.

Temporary excavations extending to a depth of less than 20 feet can be sloped at 1 horizontal to 1 vertical.

Dewatering requirements will depend upon the option used to mitigate liquefaction. If stone columns/aggregate piers or deep foundations are used, dewatering will be minor; limited to the easterly side where the existing ground is at elevation ~103 feet. Groundwater in this area was encountered between elevation 87 and 88 feet. It is recommended that a perimeter drain and cross drains be installed to maintain the water level below elevation 86 feet. If the excavation and replacement option is used to mitigate liquefaction, construction dewatering should be designed to maintain the groundwater level at least 2 feet below the base of the excavation. In our opinion, well points will be the most efficient method of lowering the water level during construction.

If the excavation and replacement method is used, it is anticipated that stabilization of the foundation excavations may be required prior to placement of fill. Care should be taken to prevent heavy construction equipment from traversing directly on the subgrade soils.

Stabilization techniques are dependant upon conditions encountered and construction methods. Where very soft clay exists, it is anticipated that cobble rock will provide the most effective means of stabilization. Where cobble rock is required, it should consist of 3 to 8 inch rock placed in single lifts, tamped into the clay such that the voids are filled. Excess cobbles which cannot be tamped into the clay should be removed to prevent migration of fines into the voids, which would result in settlement. Placement of a geotextile fabric, such as Mirafi 500X or equivalent will be effective in stabilizing moderately soft areas.

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 20% passing a No. 200 sieve. We recommend that the material passing a No. 200 sieve have a plasticity index less than 5. The fill should be compacted to an in-place density equal to at least 95% of the maximum density as determined by ASTM D 1557. If the over excavation and replacement option is used during winter and spring months, it may be more efficient to import fill and waste the sandy silt below the water level rather than dry the soil to optimum moisture content.

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 100 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.

VI. 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.

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.

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Figures

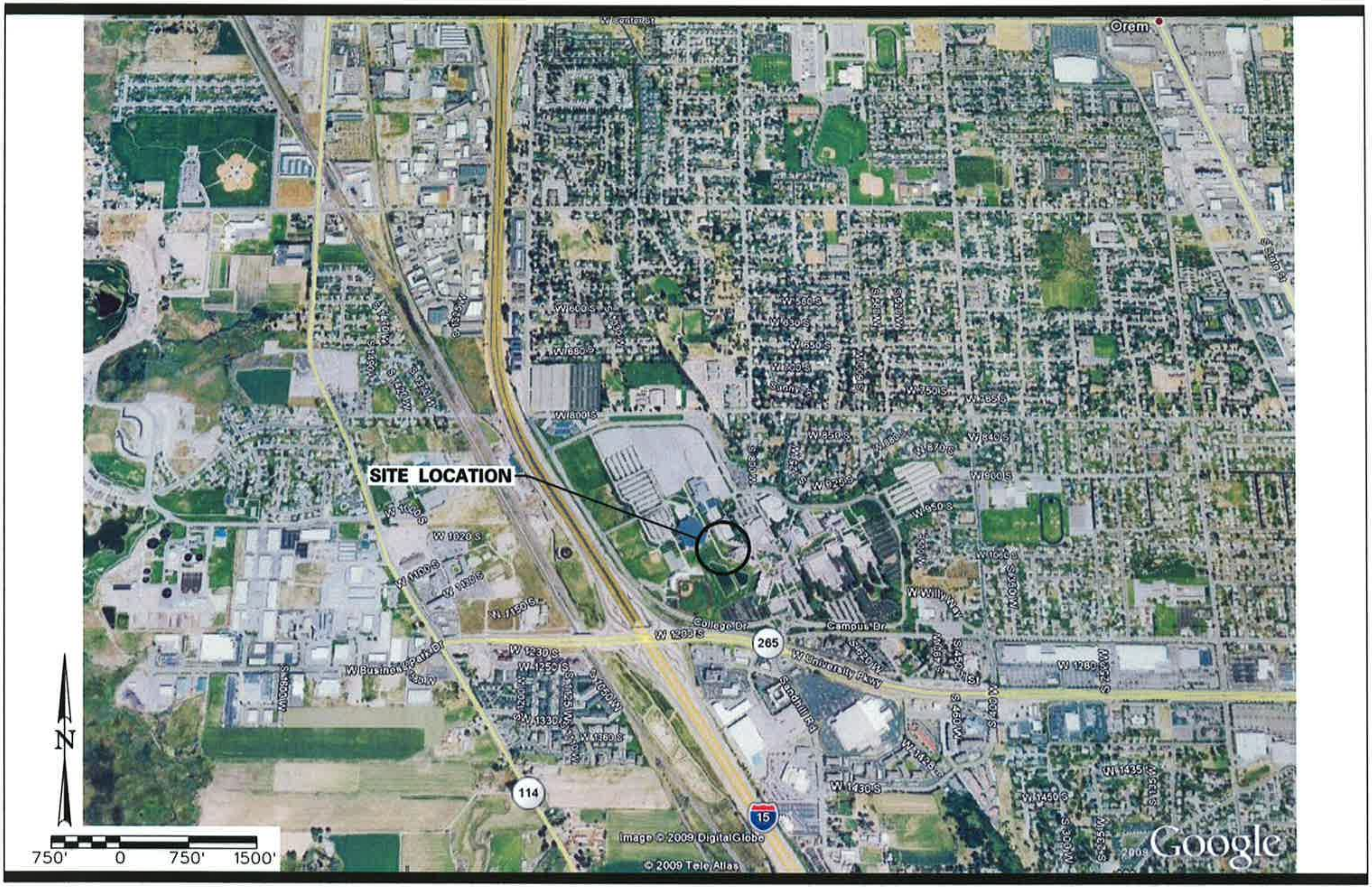

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Figure 1 **VICINITY MAP**

UVU - Pope Science Building New Addition

Orem, Utah

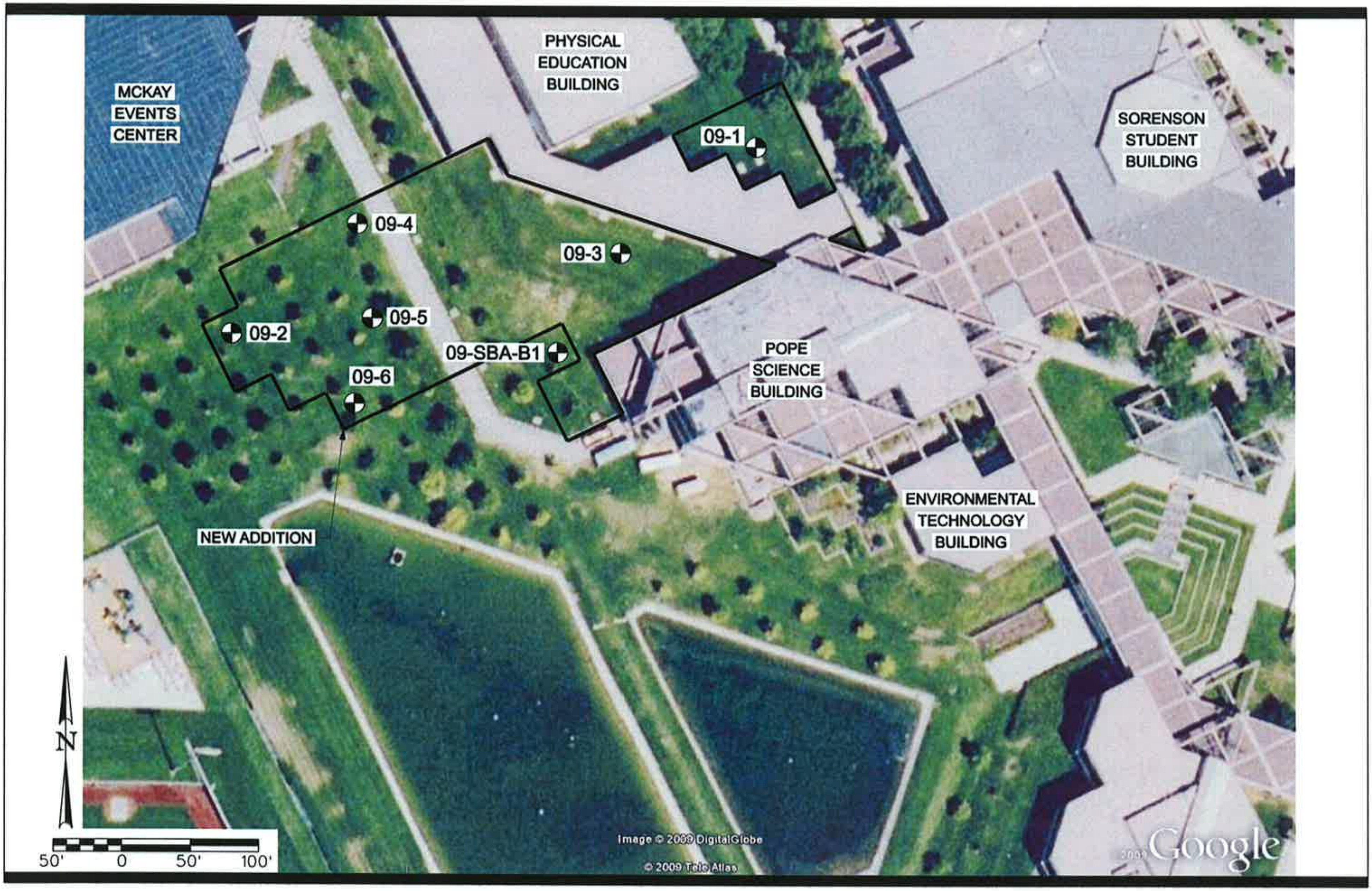

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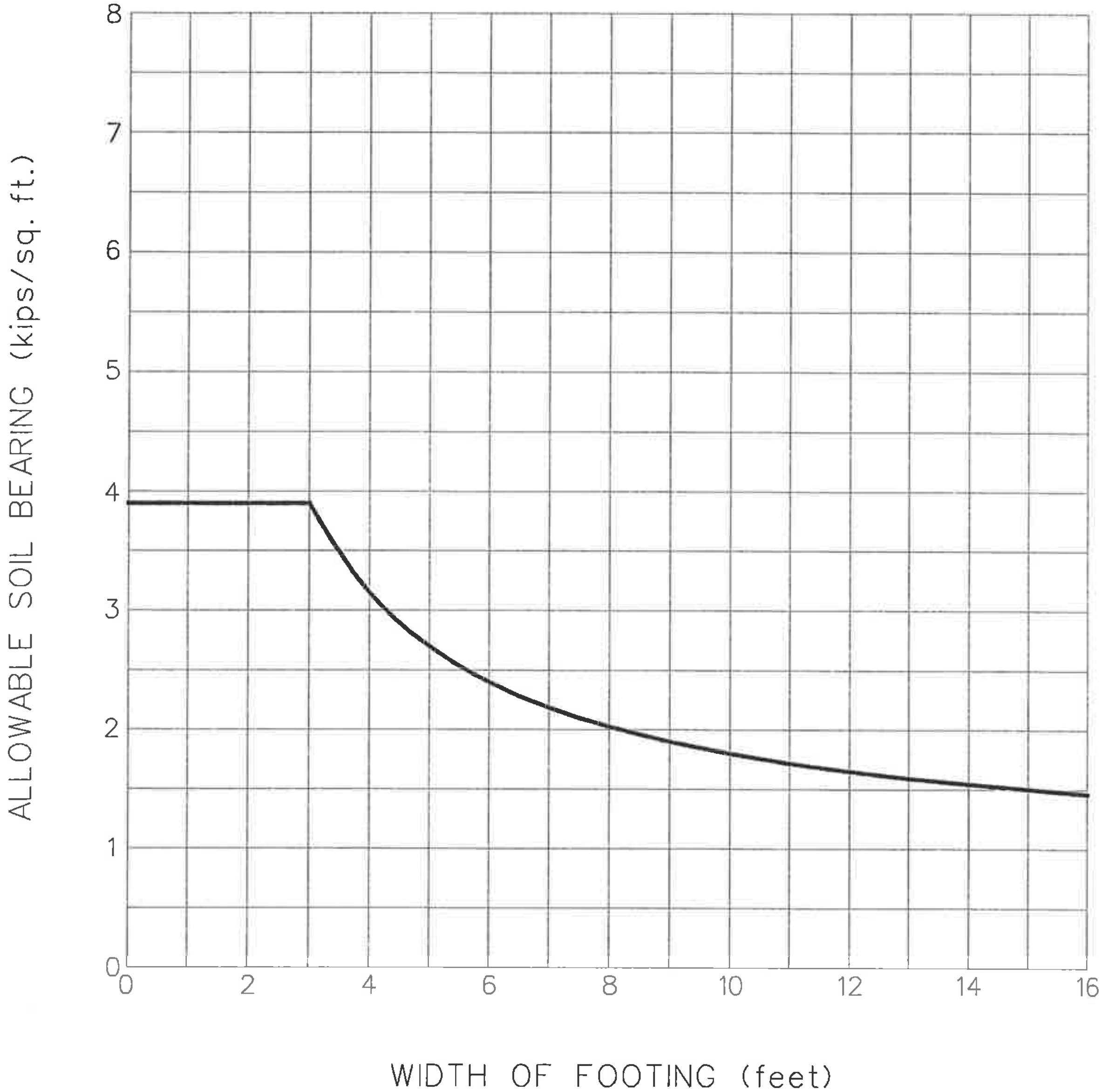
Figure 2 SITE PLAN & TEST HOLE LOCATIONS

UVU - Pope Science Building New Addition

Orem, Utah



Recommended allowable soil bearing pressures on:

- ☐ natural material
- ☒ compacted fill

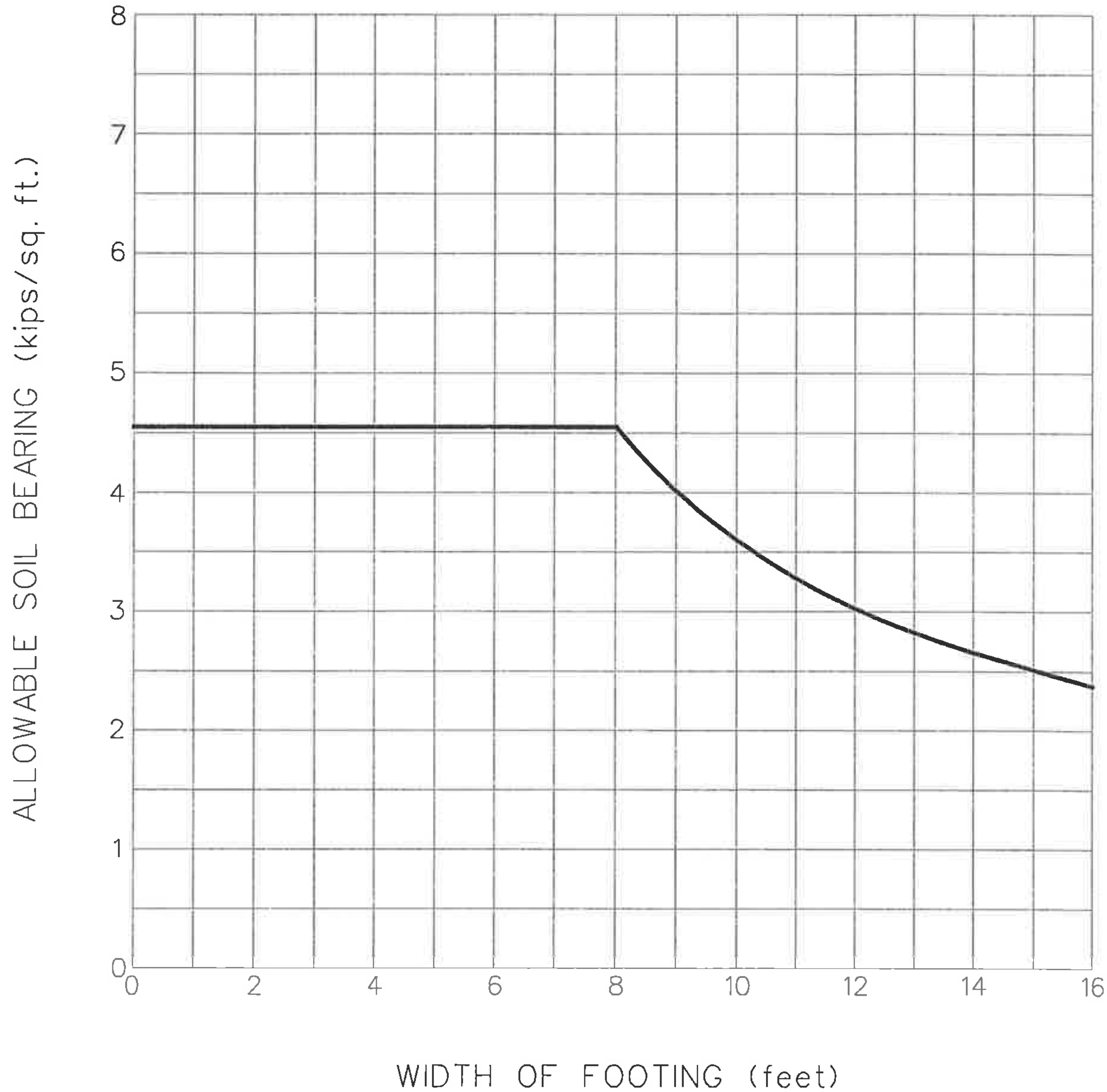
Type of footings:

- ☒ spread
- ☐ spot
- ☒ continuous

RB&G ENGINEERING, INC.

Figure 3 BEARING CAPACITY CHART

UVU Pope Science Building New Addition
Orem, Utah



Recommended allowable soil bearing pressures on:

- ☐ natural material
- ☒ compacted fill

Type of footings:

- ☒ spread
- ☒ spot
- ☐ continuous

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Figure 4 BEARING CAPACITY CHART

UVU Pope Science Building New Addition
Orem, Utah

Appendix

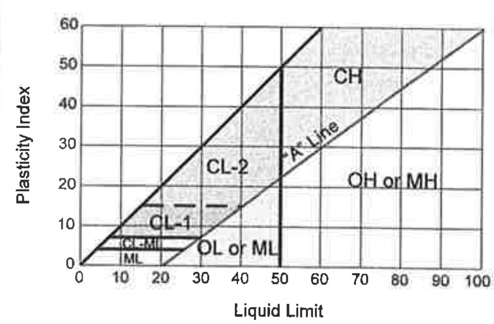
Unified Soil Classification System

Major Divisions			Group Symbols		Typical Names
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
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels With Fines *appreciable amount of fines*	**GM***	**d**	Silty gravels, poorly graded gravel-sand-silt mixtures
				u	
			GC		Clayey gravels, poorly graded gravel-sand-clay mixtures
	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
			SP		Poorly graded sands, gravelly sands, little or no fines
		Sands with Fines *appreciable amount of fines*	**SM***	**d**	Silty sands, poorly graded sand-silt mixtures
				u	
			SC		Clayey sands, poorly graded sand-clay mixtures
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
			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

Laboratory Classification Criteria

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	
Determine percentage of gravel and sand from grain-size curve.	Not meeting all gradation requirements for GW	
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
Less than 5% GW, GP, SW, SP	Atterberg limits above "A" line, or PI greater	
More than 12% GM, GC, SM, SC	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
5% to 12% Borderline cases requiring use of dual symbols**	Not meeting all gradation requirements for SW	
	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
	Atterberg limits above "A" line, or PI greater	

For laboratory classification of fine-grained 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 POPE SCIENCE BUILDING NEW ADDITION

CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT

LOCATION: SEE SITE PLAN

DRILLING METHOD: 08-CME-55 / N.W. CASING TO 9'

DRILLER: T. KERN

DEPTH TO WATER - INITIAL: ▽ 16.0' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.051

DATE STARTED: 10/29/09

DATE COMPLETED: 10/29/09

GROUND ELEVATION: ~103.0' *

LOGGED BY: M. HANSEN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample: Type	Sample: Rec. (in)	Sample: See Legend	Sample: USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter.: Liquid Limit	Atter.: Plast. Index	Gradation: Gravel (%)	Gradation: Sand (%)	Gradation: Silt/Clay (%)	Other Tests
			Disturbed	18	4,8,8,(34)	SM	brown, moist, dense									
100																
			Disturbed	9	2,2,1,(7)	SM	brown, moist, loose	SILTY SAND occasional gravelly layer								
	5															
			Disturbed	11	8,6,8,(30)	SM	brown, moist, med. dense			10.8		NP	11	70	19	
95																
	10		Disturbed	11	7,5,6,(30)	SM	brown, moist, med. dense									
								SILTY SAND few clay lenses								
90			Disturbed	15	5,5,7,(19)	SM	brown, very moist, med. dense									
	15															
▽			Disturbed	18	3,2,3,(7)	ML	brown, wet, loose	SANDY SILT interbedded w/silty sand layers to 2" thick		31.7		NP	0	45	55	
85																
	20															
			Disturbed	18	3,2,3,(7)	ML	brown, wet, loose	SANDY SILT few clay lenses		33.5		NP	0	47	53	
80																
	25															
			Undisturbed	18	Pushed 0.20	CL-2	lt. brown, very moist, soft	LEAN CLAY W/SAND vertical sand lenses & layers	87.3	34.9	40	20				CT UC
75																
	30							LEAN CLAY W/SAND sand lenses								
			Disturbed	18	2,2,3,(6) 0.24	CL	lt. brown, very moist, soft									
								BOH								
70								*Note: Elevation is assumed.								

200901.051 UVUPOPESCIENCEADD.GPJ US EVAL.GDT 12/2/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE: 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-2

SHEET 1 OF 2

PROJECT: UVU POPE SCIENCE BUILDING NEW ADDITION

CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT

PROJECT NUMBER: 200901.051

LOCATION: SEE SITE PLAN

DATE STARTED: 11/2/09

DRILLING METHOD: 08-CME-55 / N.W. CASING TO 48.5'

DATE COMPLETED: 11/2/09

DRILLER: T. KERN

GROUND ELEVATION: ~88.0' *

DEPTH TO WATER - INITIAL: ▽ N.M. **AFTER 24 HOURS:** ▼ 8.6' / 8.0' 11/4

LOGGED BY: K. MARTINEZ, J. BOONE

Elev. (ft): 85, 80, 75, 70, 65, 60, 55, 50, 45, 40, 35

Depth (ft): 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

Rec. (in)	See Legend	USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plast. Index	Gravel (%)	Sand (%)	Silt/Clay (%)	Other Tests
18	5,6,7,(29)	SM	brown, moist, med. dense	Organics in top 4" SILTY SAND								
9	5,4,4,(18)	SM	brown, moist, med. dense									Chem.
12	3,3,4,(15)	ML	brown, wet, med. dense	SANDY SILT		31.9		NP	0	33	67	
4	Pushed	SM	brown, wet	SILTY SAND clay lenses								
10	2,3,3,(11)	SM	brown, wet, med. dense									
13	2,2,2,(7)	SM	brown, wet, loose			33.6		NP	0	58	42	
12	0,1,2,(5)	ML	brown, wet, soft	SANDY SILT slightly plastic								
18	0.23 Pushed	CL-1	brown, wet, soft	LEAN CLAY W/SAND sand lenses	89.5	32.0	30	8				CT UC
		SM	brown, wet	SILTY SAND		26.6		NP	0	58	42	
13	2,2,4,(8) 0.33	CL	brown, wet, firm	LEAN CLAY W/SAND sand lenses & layers								
18	0.40 Pushed	CL-2	brown, very moist, firm		94.1	29.4	35	16				UC
		SM	brown, wet	SILTY SAND		22.2		NP	0	51	49	
18	1,3,4,(9) 0.52	CL	brown, wet, stiff	LEAN CLAY W/SAND								
17	Pushed	ML	brown, wet	SANDY SILT		24.2		NP	0	42	58	
		SM	brown, wet	SILTY SAND clay lenses & layers		19.3		NP	0	76	24	
11	3,7,10,(19)	SM	brown, wet, med. dense									
14	4,14,12,(29)	SM	brown, wet, med. dense									
18	21,7,15,(23) 0.99+	CL-ML	brown, wet, very stiff	SANDY SILTY CLAY		23.1	20	4				
				SILTY SAND clay lenses & layers								

200901.051 UVUPOPESCIENCEADD.GPJ US EVAL.GDT 11/24/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-2

PROJECT: UVU POPE SCIENCE BUILDING NEW ADDITION

CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT

PROJECT NUMBER: 200901.051

LOCATION: SEE SITE PLAN

DATE STARTED: 11/2/09

DRILLING METHOD: 08-CME-55 / N.W. CASING TO 48.5'

DATE COMPLETED: 11/2/09

DRILLER: T. KERN

GROUND ELEVATION: ~88.0' *

DEPTH TO WATER - INITIAL: ▽ N.M. **AFTER 24 HOURS:** ▼ 8.6' / 8.0' 11/4

LOGGED BY: K. MARTINEZ, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample Type	Rec. (in)	See Legend	USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
				15	13,22,16,(39)	SM	gray, wet, dense	SILTY SAND clay lenses & layers								
30																
	60			15	15,20,15,(34)	SM	gray, wet, dense	SILTY SAND occasional clay lenses		24.1		NP	0	64	36	
25																
	65			18	25,31,30,(58)	SP-SM	gray, wet, very dense									
20																
	70			12	14,30,30,(55)	SP-SM	gray, wet, very dense	SAND W/SILT occasional clay lenses								
15																
	75			14	11,15,26,(37)	SP-SM	gray, wet, dense									
10																
	80			14	30,42,47,(77)	SM	gray, wet, very dense			18.2		NP	0	83	17	
5								SILTY SAND occasional clay lenses								
	85			16	30,31,26,(48)	SM	gray, wet, dense									
0																
	90			18	7,13,14,(22) *0.46*	CL-1	gray, wet, firm	LEAN CLAY W/SAND sand lenses & layers		35.0	31	14				
-5																
	95			15	30,37,35,(58)	SP-SM	gray, wet, very dense	SAND W/SILT clay lenses & layers to 0.5" thick								
-10																
	100			16	19,31,40,(56)	SM	gray, wet, very dense	SILTY SAND clay lenses & layers to 0.5" thick								
								BOH *Note: Elevation is assumed.								
-15																
	105															
-20																

200901.051 UVUPOPESCIENCEADD.GPJ US EVAL.GDT 11/24/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; *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

DRILL HOLE LOG

BORING NO. 09-3

SHEET 1 OF 1

PROJECT: UVU POPE SCIENCE BUILDING NEW ADDITION

CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT

LOCATION: SEE SITE PLAN

DRILLING METHOD: 08-CME-55 / N.W. CASING TO 15'

DRILLER: T. KERN

DEPTH TO WATER - INITIAL: ∇ 15.0' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.051

DATE STARTED: 11/3/09

DATE COMPLETED: 11/3/09

GROUND ELEVATION: ~103.0' *

LOGGED BY: K. MARTINEZ, J. BOONE

Elev. (ft)	Depth (ft)	Sample Type	Rec. (in)	See Legend	USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
		Disturbed	16	5,13,13,(55)	SM	brown, moist, med. dense	Organics in top 4"								
100							SILTY SAND occasional gravelly layer								
	5	Disturbed	16	15,21,28,(99+)	SM	brown, very moist, very dense									
95															
	10	Undisturbed	15	Pushed	ML	brown, moist			27.4		NP	0	28	72	
90							SILT W/SAND few clay lenses								
	15 ∇	Disturbed	12	2,3,5,(12)	ML	brown, wet, med. dense									
85															
		Undisturbed	13	Pushed	SM	brown, wet	SILTY SAND		30.1		NP	0	58	42	
	20	Disturbed	17	1,1,3,(5)	SM	brown, wet, loose									
		Disturbed	16	1,2,3,(7)	ML	brown, wet, loose									
80							SANDY SILT								
	25	Undisturbed	12	Pushed	ML	brown, wet									
		Disturbed	18	2,2,2,(5) 0.18	CL	brown, very moist, soft to firm	LEAN CLAY sand lenses								
75		Disturbed	18	1,2,3,(6) 0.26	CL	brown, very moist, firm									
	30	Undisturbed	16	Pushed 0.40	CL-1	brown, very moist, firm		89.5	30.9	34	14				CT UC
70															
	35	Disturbed	18	2,3,3,(7) 0.42	CL	brown, very moist, firm									
65							LEAN CLAY W/SAND sand lenses & layers								
	40	Undisturbed	16	Pushed 0.35	CL-1	brown, very moist, firm		99.7	27.2	31	11				CT UC
60															
	45	Disturbed	18	2,4,5,(9) 0.46	CL	brown, very moist, firm									
							BOH								
55							*Note: Elevation is assumed.								

200901.051 UVUPOPESCIENCEADD.GPJ US EVAL.GDT 11/24/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-4

SHEET 1 OF 1

PROJECT: UVU POPE SCIENCE BUILDING NEW ADDITION

CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT

LOCATION: SEE SITE PLAN

DRILLING METHOD: 08-CME-55 TO 18' THEN 96-CME-55 / N.W. CASING TO 5'

DRILLER: T. KERN, K. CONLIN

DEPTH TO WATER - INITIAL: ∇ 8.4' **AFTER 24 HOURS:** ▼ 8.1'

PROJECT NUMBER: 200901.051

DATE STARTED: 11/3/09

DATE COMPLETED: 11/24/09

GROUND ELEVATION: ~88.0' *

LOGGED BY: K.M., J.O., J.B.

Elev. (ft)	Depth (ft)	Type	Rec. (in)	See Legend	USCS	Material Description	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plast. Index	Gravel (%)	Sand (%)	Silt/Clay (%)	Other Tests
	0	Disturbed	16	4,11,16,(45)	SM	brown, moist, dense — Organics in top 4" SILTY SAND W/GRAVEL								
85		Disturbed	17	12,12,10,(47)	SM	brown, moist, dense — SILTY SAND								
	5	Disturbed	18	2,4,4,(17)	ML	brown, moist, med. dense/stiff — SANDY SILT slightly plastic								
80														
	10	Undisturbed	13	Pushed 0.23	ML	brown, wet, soft	86.3	30.6	26	2				CT UC
		Disturbed	15	2,5,3,(15)	SM	brown, wet, med. dense — SILTY SAND								
75		Disturbed	15	1,2,2,(7)	SM	brown, wet, loose		31.4		NP	0	52	48	
	15	Disturbed	18	1,2,2,(7)	ML	brown, wet, loose — SANDY SILT		34.3		NP	0	39	61	
70		Disturbed	18	1,3,2,(8)		brown, wet, loose								
	20	Disturbed	18	1,2,2,(6) 0.15		brown, wet, soft — SANDY SILTY CLAY TO SANDY SILT slightly plastic, clay layers to 2" thick								
65						brown, very moist, firm — LEAN CLAY silt lenses								
	25	Disturbed	18	0.43 2,2,3,(7)		brown, wet, loose — SILT occasional clay lenses		28.3		NP	0	7	93	
		Disturbed	13	5,4,3,(9) 0.05 0.33		brown, wet								
60						brown, very moist, firm — LEAN CLAY silt lenses								
	30	Disturbed	18	2,2,4,(8) 0.33		brown, very moist, firm — LEAN CLAY many silt lenses & layers to 2" thick								
55		Disturbed	18	2,3,4,(9) 0.55		brown, very moist, stiff — LEAN CLAY W/SAND occasional silt lenses								
	35	Disturbed	18	3,6,6,(14) 0.60		brown, very moist, stiff — LEAN CLAY W/SAND many silt lenses & layers to 1" thick								
50														
	40	Disturbed	18	4,6,7,(15) 0.80		brown, moist, stiff								
						BOH								
45														

*Note: Elevation is assumed.

200901.051 UVUPOPESCIENCEADD4.GPJ US EVAL.GDT 12/1/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-5

SHEET 1 OF 1

PROJECT: UVU POPE SCIENCE BUILDING NEW ADDITION
CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT
LOCATION: SEE SITE PLAN
DRILLING METHOD: 08-CME-55 / N.W. CASING TO 5'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ▽ 8.5' **AFTER 24 HOURS:** ▼ 8.2'

PROJECT NUMBER: 200901.051
DATE STARTED: 11/3/09
DATE COMPLETED: 11/3/09
GROUND ELEVATION: ~88.0' *
LOGGED BY: K. MARTINEZ, J. BOONE

Depth (ft)	Lithology	Sample Type	Rec. (in)	See Legend	USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Atter. Liquid Limit	Atter. Plast. Index	Gradation Gravel (%)	Gradation Sand (%)	Gradation Silt/Clay (%)	Other Tests
0–3	SM	Disturbed	18	6,15,14,(62)	SM	brown, moist, very dense	Organics in top 4" SILTY SAND								
3–8	ML	Disturbed	17	5,6,6,(27)	ML	brown, very moist, med. dense	SANDY SILT		25.8		NP	0	36	64	
		Disturbed	17	2,3,5,(17) 0.03	ML	brown, very moist, stiff	SANDY SILT slightly plastic								
8–15	SM	Undisturbed	11	Pushed	SM	brown, wet	SILTY SAND clay lenses		29.3		NP	0	57	43	
		Disturbed	15	1,2,3,(9)	SM	brown, wet, loose			33.8		NP	0	61	39	
15–18	ML	Disturbed	1	1,2,2,(7)	ML	brown, very moist, soft to firm	SANDY SILT slightly plastic								
18–28	CL	Disturbed	18	0,2,1,(5) 0.20	CL	brown, very moist, soft	LEAN CLAY W/SAND sand lenses & layers								
		Undisturbed	15	Pushed 0.42	CL-1	brown, moist, firm		98.9	26.0	32	12				UC
28–31.5	CL	Disturbed	16	7,4,4,(10) 0.45	CL	brown, moist, firm	SANDY LEAN CLAY sand lenses & layers								
							BOH								

Elev. (ft) scale: 85, 80, 75, 70, 65, 60, 55. Depth (ft) scale: 5, 10, 15, 20, 25, 30.

*Note: Elevation is assumed.

200901.051 UVUPOPESCIENCEADD.GPJ US EVAL.GDT 11/24/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-6

SHEET 1 OF 1

PROJECT: UVU POPE SCIENCE BUILDING NEW ADDITION
CLIENT: STATE OF UTAH DEPT. OF FACILITES CONSTRUCTION & MANAGEMENT
LOCATION: SEE SITE PLAN
DRILLING METHOD: 08-CME-55 / N.W. CASING TO 5'
DRILLER: T. KERN
DEPTH TO WATER - INITIAL: ▽ 7.5' **AFTER 24 HOURS:** ▼ 7.5'

PROJECT NUMBER: 200901.051
DATE STARTED: 11/3/09
DATE COMPLETED: 11/3/09
GROUND ELEVATION: ~87.0' *
LOGGED BY: K. MARTINEZ, J. BOONE

Elev. (ft)	Depth (ft)	Sample Type	Rec. (in)	See Legend	USCS	Material Description		Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plast. Index	Gravel (%)	Sand (%)	Silt/Clay (%)	Other Tests
85	0	Disturbed	18	6,11,10,(45)	SM	brown, moist, dense	Organics in top 4" SILTY SAND								
		Disturbed	16	4,4,6,(23)	ML	brown, very moist, med. dense	SANDY SILT clay lenses		29.7		NP	0	30	70	Chem.
80	5	Disturbed	14	2,3,5,(17)	ML	brown, wet, med. dense									
	10	Disturbed	11	1,1,2,(6)	SM	brown, wet, loose	SILTY SAND		30.5		NP	1	66	33	
75		Undisturbed	14	Pushed 0.13	ML	brown, wet, soft	SANDY SILT slightly plastic	83.6	34.7	28	3				CT UC
70	15	Disturbed	18	0,1,1,(3) 0.20	CL	brown, wet, soft									
65	20	Disturbed	18	2,2,2,(6) 0.32	CL	brown, very moist, firm	LEAN CLAY W/SAND sand lenses & layers								
60	25	Undisturbed	16	Pushed 0.40	CL-1	brown, very moist, firm		95.0	26.7	29	11				CT UC
55	30	Disturbed	18	3,3,7,(13) 0.05	CL	brown, very moist, soft	SANDY LEAN CLAY sand lenses & layers								
							BOH								

*Note: Elevation is assumed.

200901.051 UVUPOPESCIENCEADD.GPJ US EVAL.GDT 11/24/09

RB&G ENGINEERING, INC.

LEGEND:
DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-SBA-B1

SHEET 1 OF 2

PROJECT: UVU - SCIENCE BUILDING ADDITION

CLIENT: DFCM

LOCATION: SEE SITE PLAN

DRILLING METHOD: 96-CME-55 / N.W. CASING TO 18.5'

DRILLER: T. KERN

DEPTH TO WATER - INITIAL: ▽ 6.0' **AFTER 24 HOURS:** ▼ N.M.

PROJECT NUMBER: 200901.022

DATE STARTED: 5/7/09

DATE COMPLETED: 5/7/09

GROUND ELEVATION: ~88.0'*

LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Lithology	Sample: Type	Sample: Rec. (in)	Sample: See Legend	Sample: USCS		Material Description	Dry Density (pcf)	Moisture Content (%)	Atter.: Liquid Limit	Atter.: Plast. Index	Gradation: Gravel (%)	Gradation: Sand (%)	Gradation: Silt/Clay (%)	Other Tests
				16	5,7,8,(31)	SM	brown, moist, dense	Organics in top 6"		16.0		NP	8	46	46	
85																
				16	4,2,5,(16)	SM	brown, very moist, med. dense	SILTY SAND		20.7		NP	0	61	39	
	5															
	▽															
				15	Pushed	SM	brown, wet		93.5	28.3		NP	0	55	45	UC
80																
				18	1,1,3,(8) 0.14	CL	brown, very moist, soft									
	10			14	Pushed 0.28	CL-2	brown, moist, firm		86.5	34.2	39	19				CT UC
75				16	Pushed 0.38	CL	brown, moist, firm									
	15							LEAN CLAY W/SAND silt lenses								
				18	1,2,2,(7) 0.30	CL-1	brown, moist, firm			34.3	35	12				
70																
	20															
				15	Pushed 0.57	CL-1	brown, moist, stiff		101.2	25.4	26	8				
65																
	25															
				15	3,3,7,(14) 0.40	CL	brown, moist, firm	LEAN CLAY W/SAND sand layers to 1" thick								
60																
	30			15	0.36	CL	brown, moist, firm									
					Pushed	SM	brown, wet	SILTY SAND								
				18	3,7,8,(19) 0.61	CL	brown, moist, stiff									
55								LEAN CLAY W/SAND numerous silt and/or sand lenses & layers to 1/2" thick								
	35			18	3,6,7,(16) 0.86	CL	brown, moist, stiff									
50																
	40			16	Pushed 0.52	CL,SM	brown, moist/wet, stiff/med. dense	INTERBEDDED LEAN CLAY & SILTY SAND LAYERS 4"-8" THICK								
				18	8,13,11,(28) 0.54	CL,SM	brown, moist/wet, stiff/med. dense									
45																
								SAND W/SILT occasional clay layers to 1 1/2" thick								

DH_LOGV6 UVUPOPESCIENCEADD2.GPJ US EVAL.GDT 11/24/09

RB&G ENGINEERING, INC.

LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

DRILL HOLE LOG

BORING NO. 09-SBA-B1

SHEET 2 OF 2

PROJECT: UVU - SCIENCE BUILDING ADDITION

CLIENT: DFCM

LOCATION: SEE SITE PLAN

DRILLING METHOD: 96-CME-55 / N.W. CASING TO 18.5'

DRILLER: T. KERN

DEPTH TO WATER - INITIAL: ∇ 6.0' AFTER 24 HOURS: ▼ N.M.

PROJECT NUMBER: 200901.022

DATE STARTED: 5/7/09

DATE COMPLETED: 5/7/09

GROUND ELEVATION: ~88.0'*

LOGGED BY: C. SANBORN, J. BOONE

Elev. (ft)	Depth (ft)	Rec. (in)	See Legend	USCS	Sample description	Material Description
		6	15,20,20,(45)	SP-SM	gray, wet, dense	
40	50	15	12,21,21,(45)	SP-SM	brown, wet, dense	
35	55	6	22,27,30,(59)	SP-SM	gray, wet, very dense	SAND W/SILT occasional clay layers to 1 1/2" thick
30	60	18	15,17,14,(31)	SP-SM	gray, wet, dense	
25	65					
20	70	16	21,7,11,(17) 0.37	SP-SM CL	gray, wet, med. dense gray, moist, firm	LEAN CLAY sand lenses
15	75	14	21,37,39,(68)	SP-SM	gray, wet, very dense	SAND W/SILT clay lenses
10	80	17	30,31,18,(43)	SP-SM	gray, wet, dense	
5	85					BOH *Note: Elevation is assumed.
0						

Columns also present (empty): Dry Density (pcf), Moisture Content (%), Atter. (Liquid Limit, Plast. Index), Gradation (Gravel (%), Sand (%), Silt/Clay (%)), Other Tests.

DH_LOGV6 UVUPOPESCIENCEADD2.GPJ US EVAL.GDT 11/24/09

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LEGEND:

DISTURBED SAMPLE — 2,3,2 ← Blow Count per 6"; 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

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Table 1

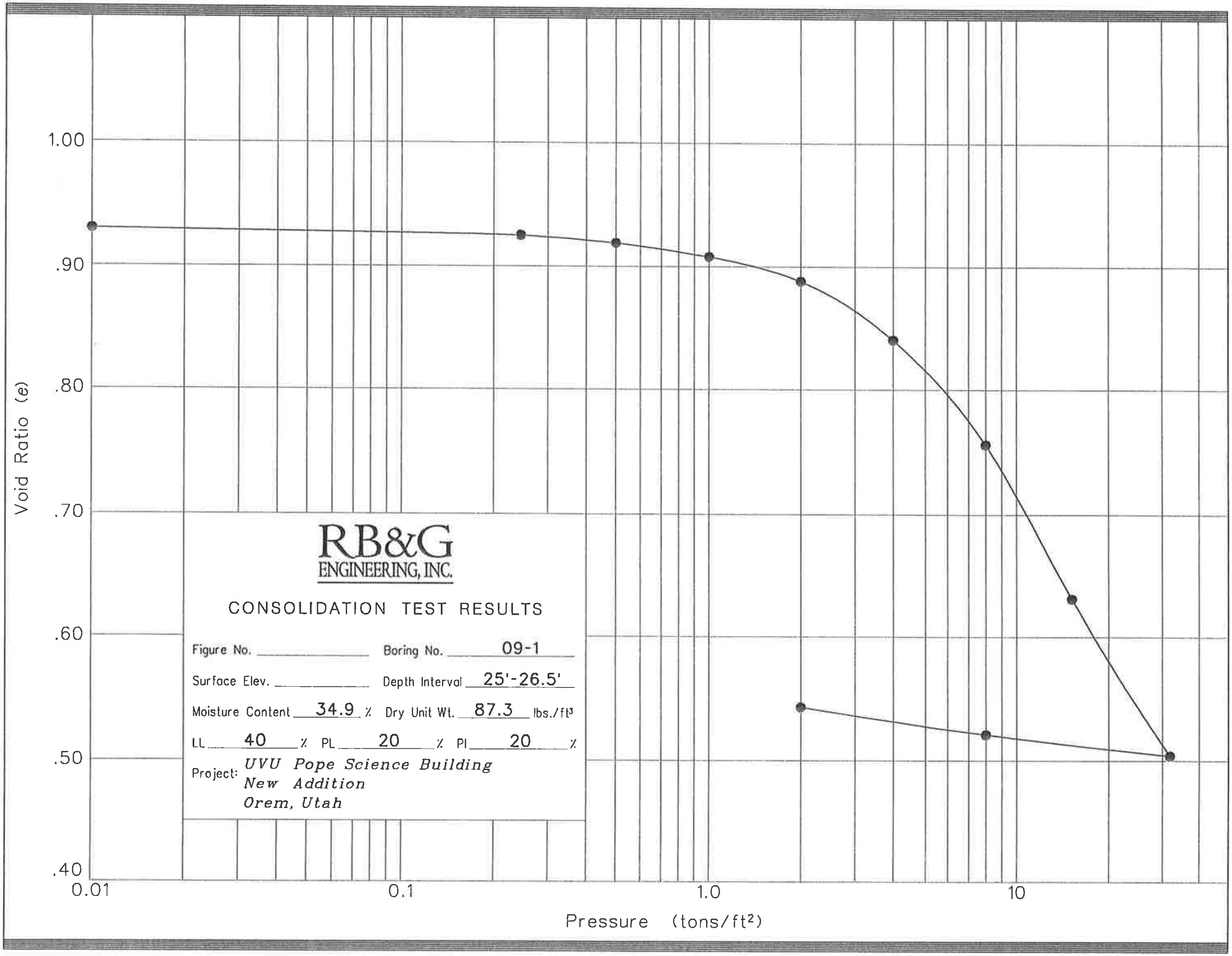
SUMMARY OF TEST DATA

PROJECT	UVU Pope Science Bldg. Addition	PROJECT	200901-051
LOCATION	Orem, Utah	FEATURE	Foundations

HOLE NO.	DEPTH BELOW GROUND SURFACE (ft)	IN-PLACE		UNCONFINED OR UU TRIAXIAL COMPRESSIVE STRENGTH (psf)	ATTERBERG LIMITS			MECHANICAL ANALYSIS			PERCENT FINER THAN 0.005 mm	UNIFIED SOIL CLASSIFICATION SYSTEM / (AASHTO CLASSIFICATION)
		DRY UNIT WEIGHT (pcf)	MOISTURE (%)		LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	PERCENT GRAVEL	PERCENT SAND	PERCENT SILT & CLAY		
09-1	6-7.5		10.8				NP	11	70	19		SM
	15-16.5		31.7				NP	0	45	55		ML
	20-21.5		33.5				NP	0	47	53		ML
	25-26.5	87.3	34.9	uc 897	40	20	20					CL-2
09-2	6-7.5		31.9				NP	0	33	67		ML
	12.5-14		33.6				NP	0	58	42		SM
	20-20.7	89.5	32.0	uc 958	30	22	8					CL-1
	20.7-21.5		26.6				NP	0	58	42		SM
	30-30.7	94.1	29.4	uc 1649	35	19	16					CL-2
	30.7-31.5		22.2				NP	0	51	49		SM
	40-40.8		24.2				NP	0	42	58		ML
	40.8-41.5		19.3				NP	0	76	24		SM
	50-51.5		23.1		20	16	4					CL-ML
	60-61.5		24.1				NP	0	64	36		SM
	80-81.5		18.2				NP	0	83	17		SM
	90-91.5		35.0		31	17	14					CL-1
09-3	10-11.5		27.4				NP	0	28	72		ML
	18-19.5		30.1				NP	0	58	42		SM
	30-31.5	89.5	30.9	uc 1634	34	20	14					CL-1
	40-41.5	99.7	27.2	uc 1488	31	20	11					CL-1
09-4	9-10.5	86.3	30.6	uc 672	26	24	2					ML
	12-13.5		31.4				NP	0	52	48		SM
	15-16.5		34.3				NP	0	39	61		ML
	25-26.5		28.3				NP	0	7	93		ML
09-5	3-4.5		25.8				NP	0	36	64		ML
	9-10.5		29.3				NP	0	57	43		SM
	12-13.5		33.8				NP	0	61	39		SM
	25-26.5	98.9	26.0	uc 1735	32	20	12					CL-1
09-6	3-4.5		29.7				NP	0	30	70		ML
	9-10.5		30.5				NP	1	66	33		SM
	12-13.5	83.6	34.7	uc 612	28	25	3					ML
	25-26.5	95.0	26.7	uc 1663	29	18	11					CL-1

		pH	Resistivity	Chloride (mg/kg-dry)	Sulfate (mg/kg-dry)
09-2	3-4.5	7.9	6100	5.7	14
09-6	3-4.5	7.9	3450	14	35

NP=Non-Plastic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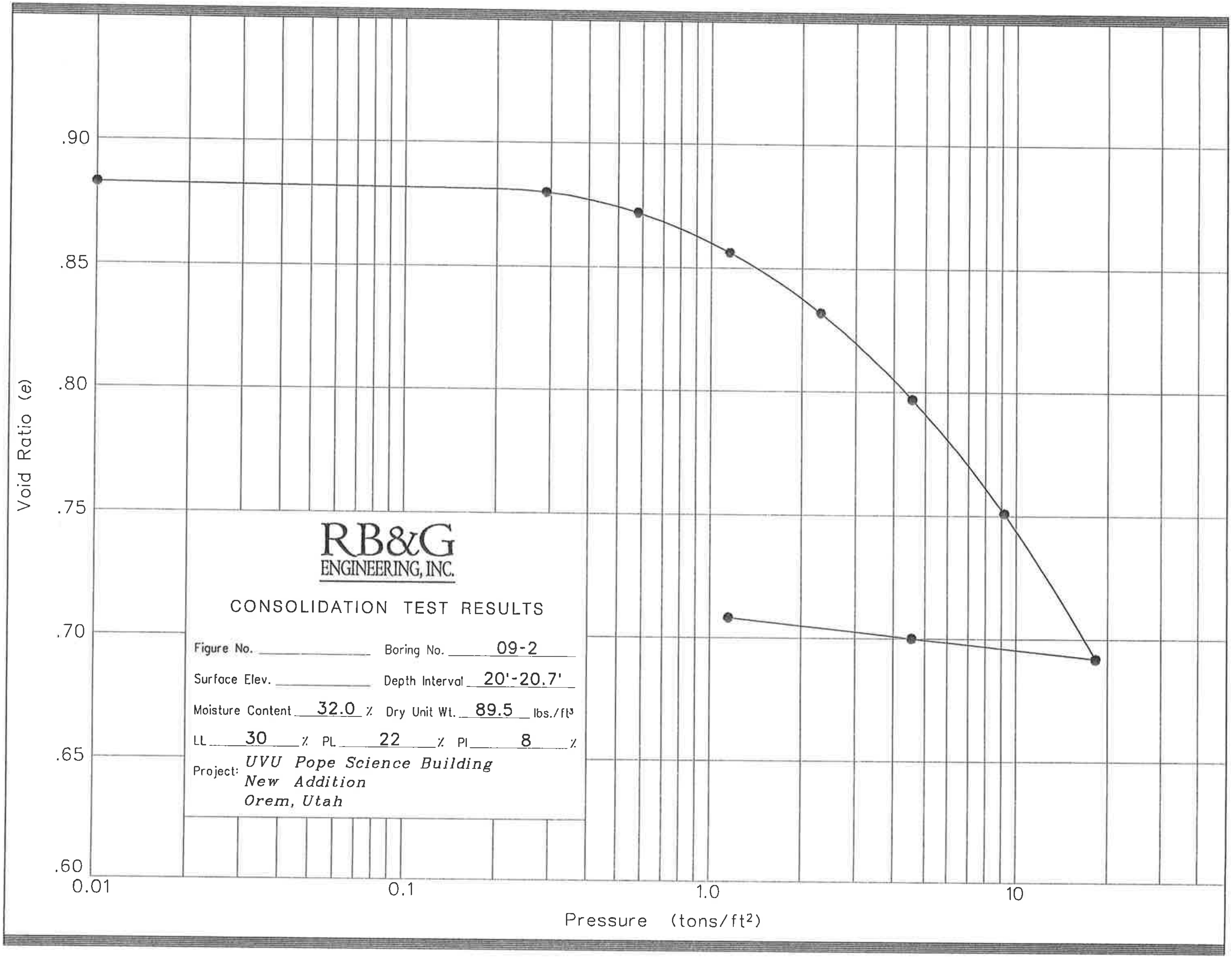
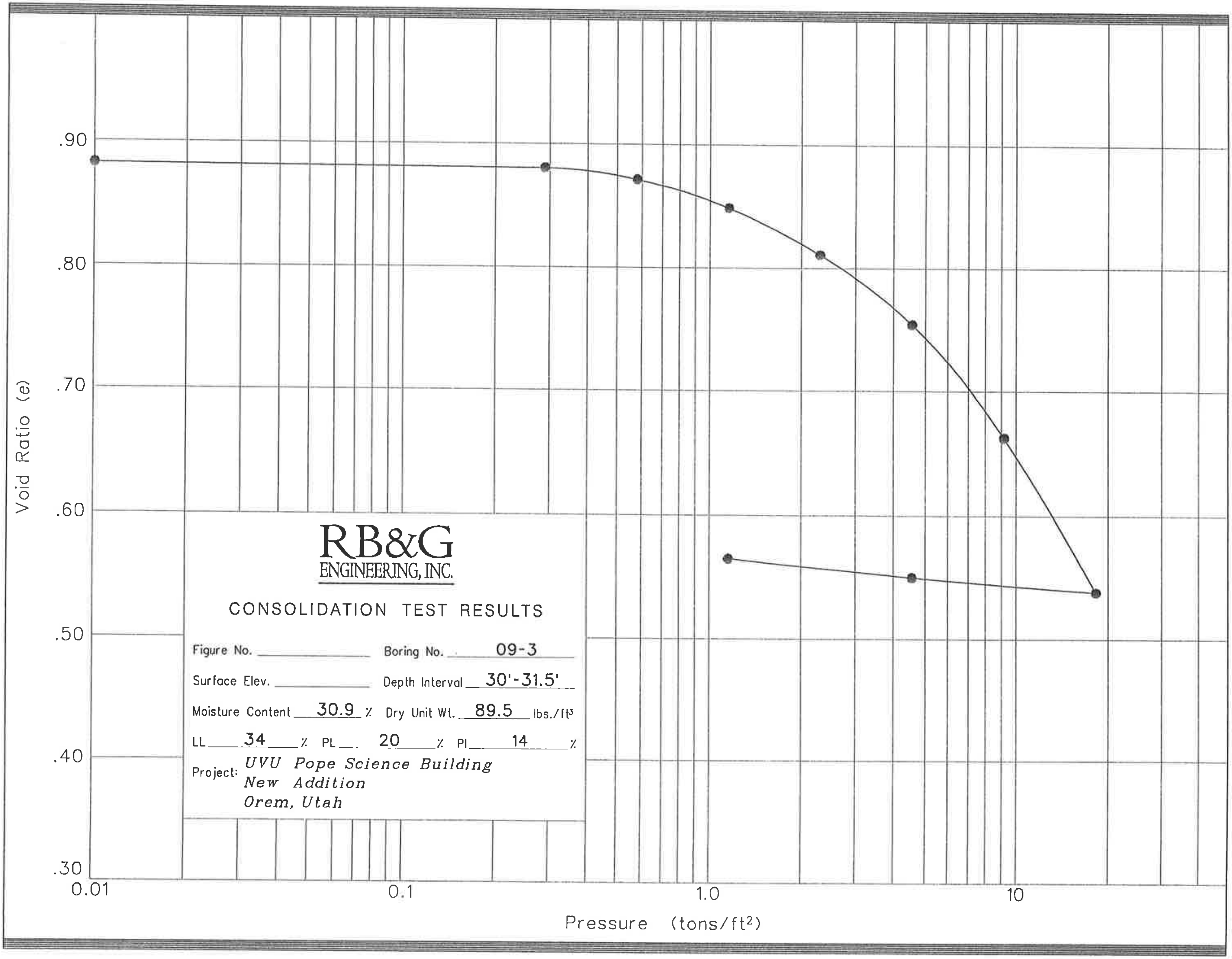
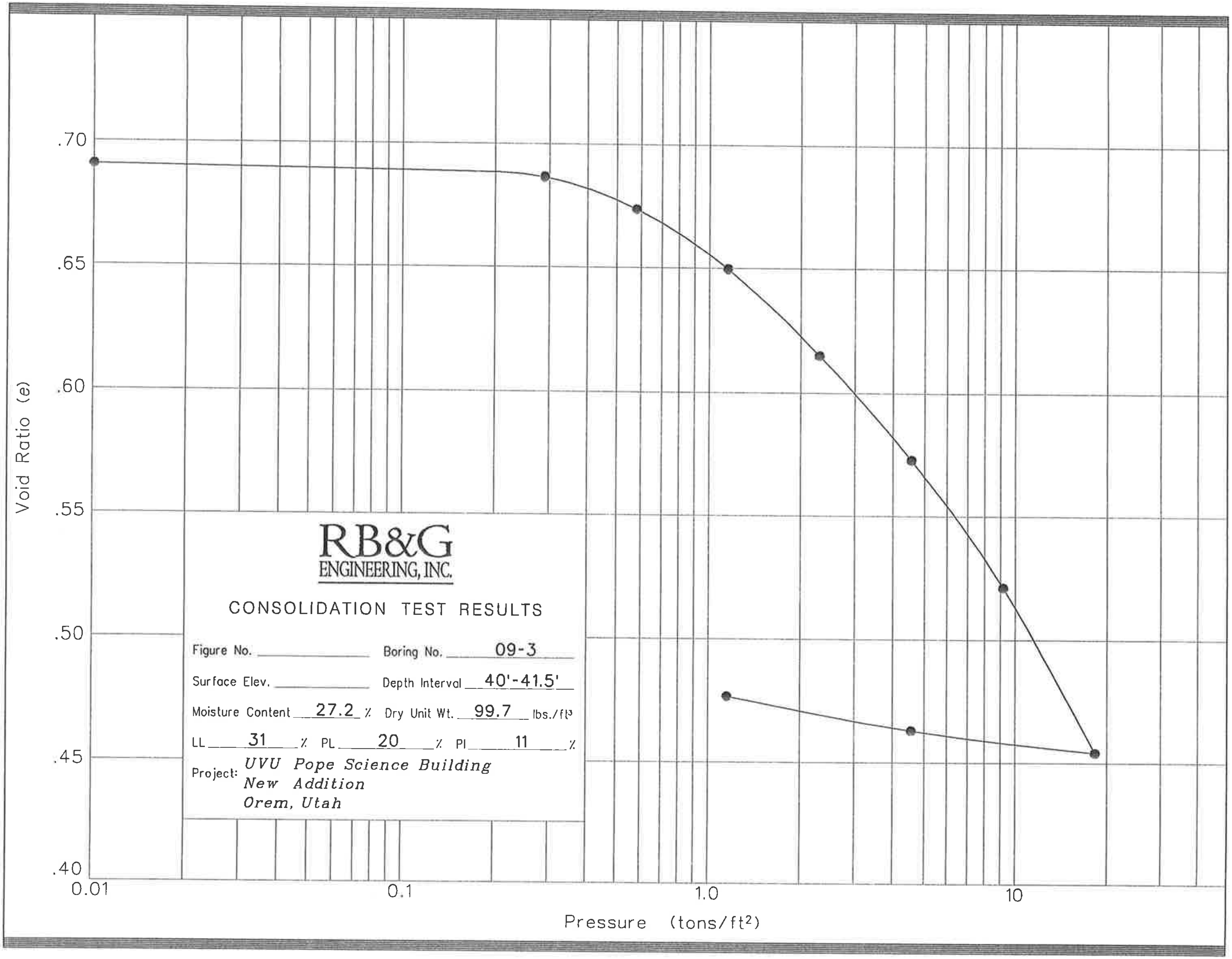
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-1
Surface Elev.
Depth Interval 25'-26.5'
Moisture Content 34.9 %
Dry Unit Wt. 87.3 lbs./ft³
LL 40 %
PL 20 %
PI 20 %
Project: UVU Pope Science Building
New Addition
Orem, Utah
Void Ratio (e)
1.00
.90
.80
.70
.60
.50
.40
0.01
0.1
1.0
10
Pressure (tons/ft²)

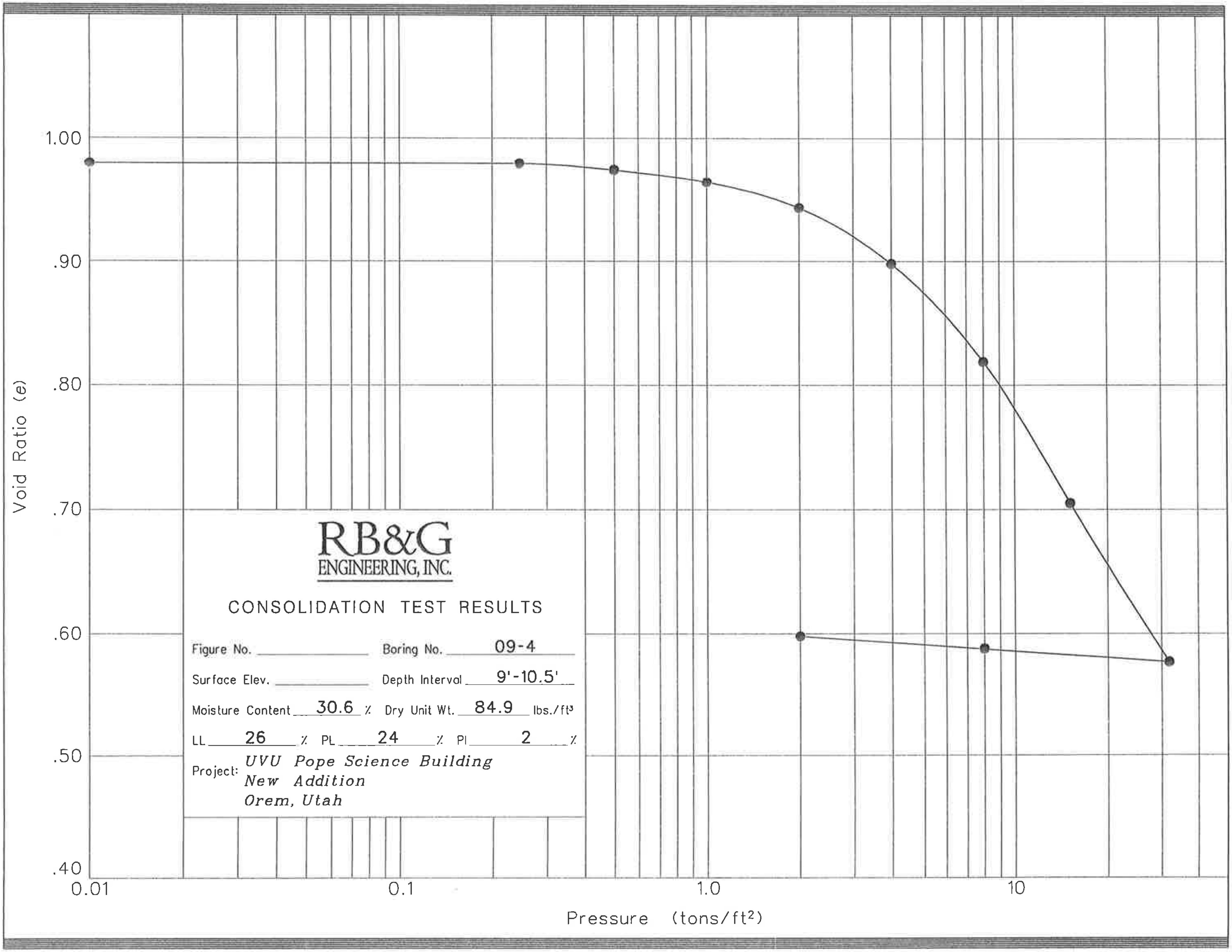

.90
.85
.80
.75
.70
.65
.60
Void Ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
RB&G ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-2
Surface Elev.
Depth Interval 20'-20.7'
Moisture Content 32.0 %
Dry Unit Wt. 89.5 lbs./ft³
LL 30 %
PL 22 %
PI 8 %
Project: UVU Pope Science Building
New Addition
Orem, Utah

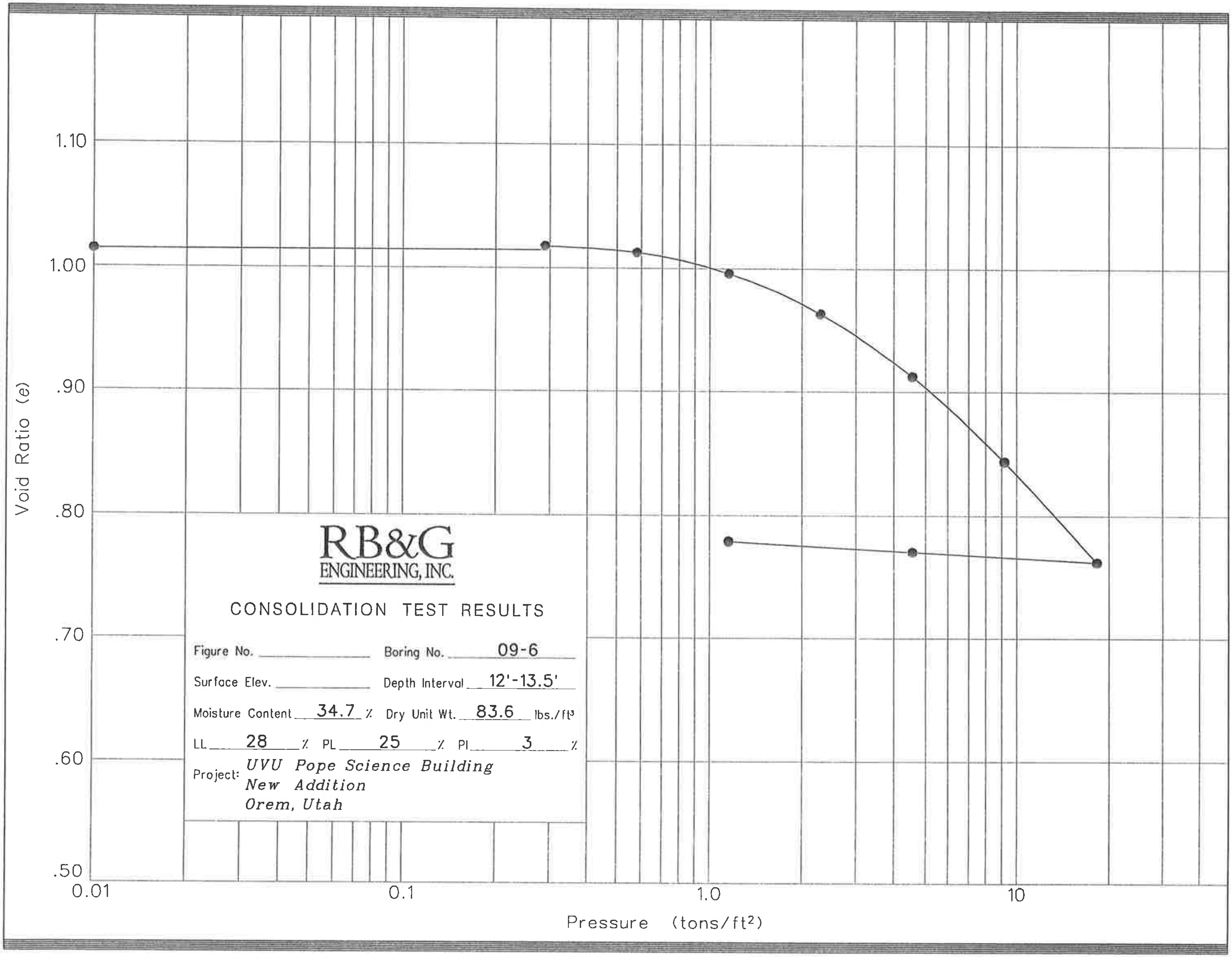


RB&G ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-3
Surface Elev.
Depth Interval 30'-31.5'
Moisture Content 30.9 %
Dry Unit Wt. 89.5 lbs./ft³
LL 34 %
PL 20 %
PI 14 %
Project: UVU Pope Science Building
New Addition
Orem, Utah
Void Ratio (e)
.90
.80
.70
.60
.50
.40
.30
0.01
0.1
1.0
10
Pressure (tons/ft²)

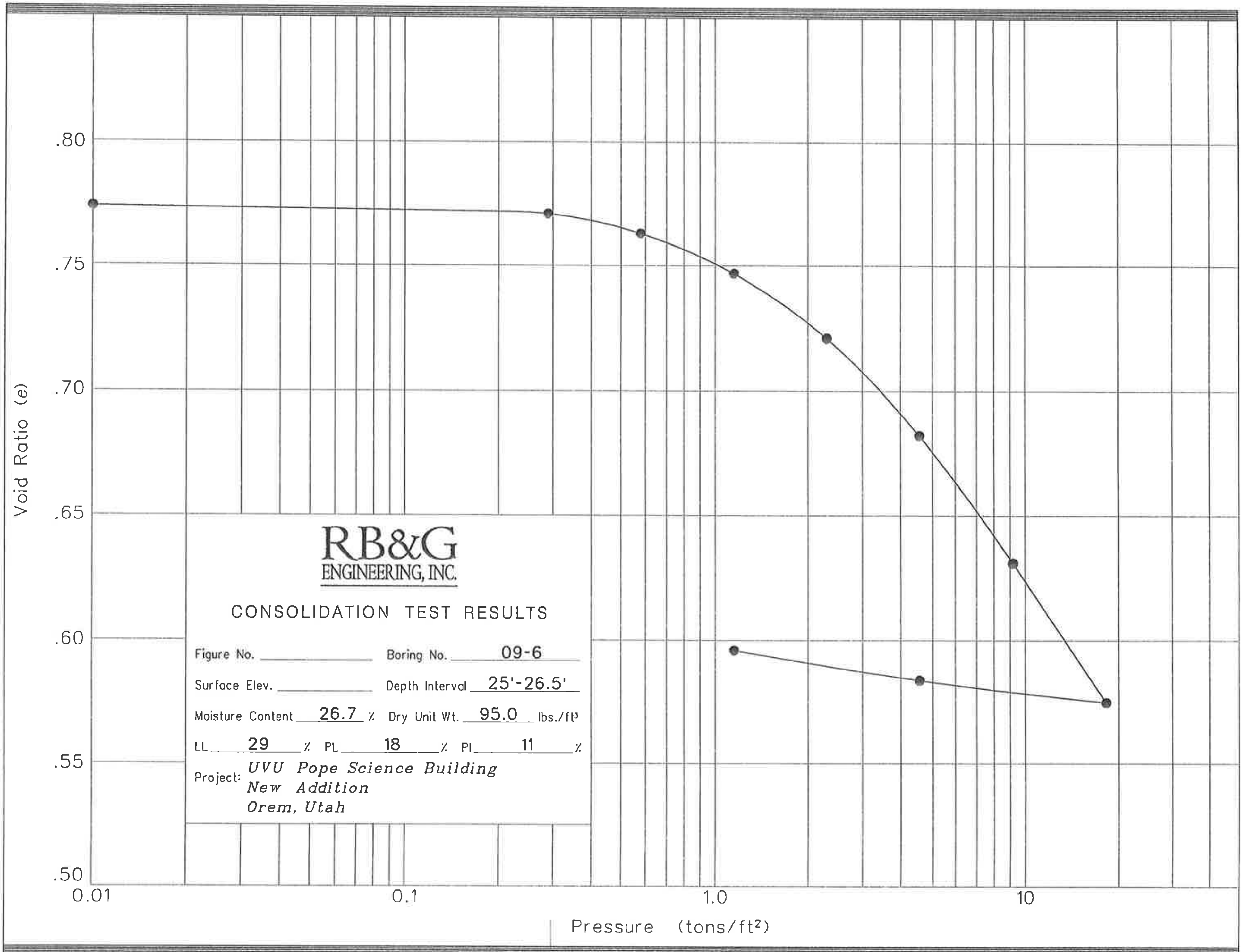


.70
.65
.60
.55
.50
.45
.40
Void Ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
RB&G ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-3
Surface Elev.
Depth Interval 40'-41.5'
Moisture Content 27.2 %
Dry Unit Wt. 99.7 lbs./ft³
LL 31 %
PL 20 %
PI 11 %
Project: UVU Pope Science Building
New Addition
Orem, Utah


1.00
.90
.80
.70
.60
.50
.40
0.01
0.1
1.0
10
Void Ratio (e)
Pressure (tons/ft²)
RB&G ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-4
Surface Elev.
Depth Interval 9'-10.5'
Moisture Content 30.6 %
Dry Unit Wt. 84.9 lbs./ft³
LL 26 %
PL 24 %
PI 2 %
Project: UVU Pope Science Building
New Addition
Orem, Utah



1.10
1.00
.90
.80
.70
.60
.50
Void Ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
RB&G
ENGINEERING, INC.
CONSOLIDATION TEST RESULTS
Figure No.
Boring No. 09-6
Surface Elev.
Depth Interval 12'-13.5'
Moisture Content 34.7 %
Dry Unit Wt. 83.6 lbs./ft³
LL 28 %
PL 25 %
PI 3 %
Project: UVU Pope Science Building
New Addition
Orem, Utah



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CONSOLIDATION TEST RESULTS

Figure No. ______ Boring No. 09-6

Surface Elev. ______ Depth Interval 25'-26.5'

Moisture Content 26.7 % Dry Unit Wt. 95.0 lbs./ft³

LL 29 % PL 18 % PI 11 %

Project: *UVU Pope Science Building*
New Addition
Orem, Utah