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TECHNICAL MEMORANDUM

To:	Mr. Eric Neil, PE Bowen Collins & Associates, Inc. (BCA) 154 East 14000 South Draper, Utah 84020 (801) 495-2224 eneil@bowencollins.com
From:	Taylor Hall, PE
Reviewed by:	Travis Gerber, PhD, PE
Date:	February 7, 2019
Job Number:	18-1138
Subject:	Ogden – Hinckley Airport Well House

INTRODUCTION AND BACKGROUND

This technical memo (TM) summarizes findings from our field study and provides geotechnical design recommendations and construction considerations for the Ogden – Hinckley Airport Well House project in Ogden, Utah. We understand that Bowen Collins & Associates (BCA) has been retained by Ogden City (Owner) to design a new well house building located near 3800 S Airport Road. The well house will replace an existing well house, and the new well house will overlie portions of the old well house as well as a portion of an abandoned subsurface water reservoir which has recently been demolished. BCA has retained Gerhart Cole (GC) to assist it with its design efforts. We understand that the proposed structure will be on the order of 30 by 55 feet in plan, and consist of a tall one-story CMU (block) building with a chlorine room, pump room and maintenance room. With a vertical turbine pump, we understand the lowest portion of the well house building will only be several feet below existing (and final) grade.

SCOPE OF WORK

The studies were performed under GC's professional services agreement with BCA dated October 15, 2010, and more specifically Task Order No 18-07 which incorporates our proposal for services dated October 31, 2018. The scope of services completed includes:

- A field study consisting of one test hole with sampling of subsurface soil, together with in-hole infiltration testing,
- Laboratory testing,
- Geotechnical engineering analyses and development of recommendations, and
- Preparation of this technical memorandum.

FIELD STUDY

Test Hole

One test hole was drilled for this study on December 12, 2018. The test hole was drilled by A Cache Corp. under subcontract to GC, utilizing a Simco 2800 truck mounted drill rig, to a depth of about 52 feet below existing site grade. Soil samples were taken at near continuous intervals (every 2.5 feet) to a depth of 15 feet and every 5 feet thereafter. The test hole was backfilled with soil cuttings and bentonite chips. Drilling was performed using rotary wash (without mud) to a depth of 10 feet to accommodate infiltration testing. Following the sample recovered at 10 feet, drilling mud was added to the water to assist in removing soil cuttings from the test hole and maintained to the terminal depth of the test hole. Table 1 summarizes details regarding the test hole, and Figure 2 illustrates the test hole location relative to the proposed site improvements.

Subsurface conditions were logged by a GC field engineer at the time of drilling. Standard Penetration Tests (SPT) were performed using an automatic hammer. The energy efficiency of the auto hammer was measured to be approximately 83 percent by GC in October 2018. The number of hammer blows required to advance the sampler in 6-inch increments was recorded in the field, with the sum of the second and third 6-inch intervals constituting the SPT blowcount or "N-value." Logs of the test hole are presented in Appendix A. Lines designating boundaries between different materials shown on the logs should be considered approximate; transitions between subsurface materials may be gradual or occur between sampling depths.

Infiltration Testing

Infiltration testing was performed at two intervals within the upper 10 feet of the test hole. The infiltration testing was performed as the pump house may lose excess water within the near surface soils during its lifetime. Constant head infiltration tests were performed at two separate intervals, between 3 to 5 feet and 8 to 10 feet below existing site grade. Testing was performed by drilling a two foot interval of interest, backing out the tooling and driving a solid steel casing to the top of the interval of interest. Water is then used to fill the annulus and approximately 10 minutes of saturation time was allowed prior to measurements being taken. Results of the test are presented and discussed later in this document.

LAB TESTING

Laboratory testing was performed on select soil specimens obtained during the field study in order to further classify them and evaluate their engineering properties. Laboratory testing included index testing (particle-size distributions and natural moisture contents) on various samples, and one moisture-density relationship (i.e., "proctor compaction") test and one corresponding, one-point California bearing ratio (CBR) test on a near-surface bulk sample. Laboratory test results are tabulated in Table 3.

GEOLOGIC SETTING

The project site is located within the Basin and Range Province, near the western slope of the Wasatch Mountain Range on a historic delta along the Weber River. During pre-historic times, the Ogden area was largely filled by the ancestral Lake Bonneville which stabilized at several 'stands' or 'benches' between the Great Lake Lake's current elevation of approximately 4,200 feet above sea level and Lake Bonneville's peak elevation of approximately 5,100 feet above sea level (reached about 14,500 years ago during the Pleistocene Epoch). During Lake Bonneville's existence, finer grained lacustrine materials were deposited within the lake with typically coarser alluvial and fluvial soils intruding from the margins. Coarser deltaic deposits also formed at the mouths of the canyons and into the valleys where the Weber and Ogden rivers flowed into Lake Bonneville. These processes were a continuation of similar ones occurring during even older lake cycles within the basin.

The surficial geology of the site has been mapped by Sack (2005) as part of the Roy 7.5 minute quadrangle (see Figure 3). Surficial soils are mapped as Holocene “sand-dominated deltaic deposits from the early and middle post-Provo regressive phase of Lake Bonneville”. This material is further described as primarily fine and medium sand, with occasional deposits of gravel from the channel of the Weber River, with maximum thicknesses ranging from 50 to 125 feet.

The Ogden City area itself is located within the Intermountain Seismic Belt (ISB), one of the most seismically active areas in the interior western U.S. Earthquakes of a moment magnitude 7 and greater have occurred repeatedly along the nearby Wasatch Fault. The nearest mapped fault is the Weber Section of the Wasatch Fault Zone which is approximately 3.3 miles east of the site (USGS, 2018a). This fault is considered to rupture with a characteristic moment magnitude of 7.16 and have a long-term return interval on the order of 1,300 years. Current evidence indicates that the last rupture along this fault segment occurred approximately 600 years ago.

SURFACE CONDITIONS

The site is at approximately the same elevation as the rest of the adjacent airport, and elevated from the adjacent I-15 freeway to the southeast by more than 70 feet. The primary existing structures onsite (well house and water reservoir) have been partially to fully demolished shortly before prior the time that our field studies were performed. This excludes a small generator building located to the southwest of the existing well house. Some debris was still on site. The remaining surface area at the site is generally undeveloped and has either sparse vegetation or has been obscured by demolition-related debris and/or mud trafficked by construction equipment.

SUBSURFACE CONDITIONS

The soil profile logged at the time of drilling consisted of predominantly granular soils with some interbedded fine-grained seams to the terminal depth explored, 52 feet. Near surface soils to a depth of approximately 5 feet consisted of a loose to medium dense silty sand. This material was observed to be somewhat less pervious than the gravel to sandy gravel found beneath the sand. Based on the test hole, this coarser layer extends to an approximate depth of 12 feet below existing site grade. The gravel and sandy gravel was found to be medium dense, with a decreasing silt content with depth. From a depth of 12 to approximately 45 feet, a sand with varying silt and gravel content was found in a medium dense state. The sand contained iron oxide staining in some areas. At a depth of 45 feet, a clay interbed was found to be approximately 1/2 –foot thick. The clay is medium stiff to stiff with 1/4 –inch sand seams. From an approximate depth of 45.5 to 50.5 feet, a medium dense sand with frequent clay seams up to 1/4-inch thick was present. A 4-inch clay seam was found between 50 and 50.5 feet where the material transitioned back to the medium dense sand with 1/4-inch clay seams to the terminal depth explored of 52 feet. Additional details regarding the soil profile at the test location are shown on the test hole log. It should be noted that some spatial variations in subsurface conditions should be expected across the site.

Groundwater

Groundwater was not measured during our field studies due to the mud-rotary drilling method used. Also, absent a temporary monitoring well which was not included in the scope due to budgetary constraints, groundwater levels may have otherwise been difficult to measure due to the infiltration testing performed. We understand from BCA that the City has reported that the in-ground reservoir (the bottom elevation of which at a sump is shown on drawings provided by the City to be approximately 4430 feet, which is about 13 feet below surrounding grade) leaked so profusely that the entire reservoir drained from full to empty over the course of three days (hence the reason for its abandonment). This suggests that site soils present a relatively high permeability, and that groundwater levels are at least lower than 13 feet below existing grade. Examination of laboratory test results indicates that there is a marked increase in water (moisture) content, exceeding 20%, beginning at a similar depth. However, moisture content at depth may have been influenced by the infiltration testing. For design purposes, we have conservatively assumed groundwater to be no higher than 4430 feet. Groundwater levels could be deeper. Actual groundwater levels vary at least seasonally.

SEISMICITY

Ground Shaking

The level of ground shaking expected at the site has been expressed in probabilistic terms by the US Geological Survey as part of the National Seismic Hazard Mapping Project. Based on site data and geologic similarity with other sites in the area, we have assigned an IBC-based seismic site classification of ‘D’ to the site. Table 4 identifies seismic design parameters consistent with the generalized horizontal acceleration response spectrum procedure (with 5% damping) of the 2015 International Building Code (IBC). Specifically, these values were obtained from the USGS’ website for seismic design parameters (USGS, 2018b). Acceleration parameters presented in the table represent 5% damping and have not been adjusted to account for any particular occupancy category or seismic importance factor.

The MCE geometric mean peak ground acceleration (PGAm) provided in Table 4 is used for geotechnical engineering assessments such as liquefaction. This value generally represents ground motions having a 2% chance of exceedance in 50 years (i.e., 2PE50), and is different than the PGA value shown in the previous row of the table (which itself corresponds to an estimated 1% probability of structural collapse with ground motions oriented in the maximum direction).

Liquefaction

As discussed previously, the subsurface soil profile consists of predominately granular soils, which below a depth of about 5 feet are in a medium dense state. Based on the seismic design demand (i.e., design level of ground shaking) and the assumed ground water level, liquefaction is expected to be triggered, resulting in several inches of calculated settlement. However, we note that liquefaction triggering analyses need to be evaluated in the context of geologic setting (see Youd and Perkins, 1978). Surficial soils at the site are mapped as unit Qd6, the youngest of a sequence of deltaic deposits made by the Weber River that are between 14 and 12.2 thousand years old (Stack, 2005). Deeper soils are older. As such, despite analytical methods indicating liquefaction triggering and subsequent settlement, we are of the opinion that the probability of liquefaction and sufficiently large settlement to necessitate mitigation of typical constructed works is low for this site.

EARTHWORK

General

Site grading should be performed to provide adequate support for foundations, building floor slabs, asphalt concrete pavement. Of particular concern at this site is the abandonment of the previous water reservoir, the footprint into which the new well house is expected to extend. We understand that any changes in site grade will be limited to 2 feet or less. Supplementary earthwork recommendations are presented together with our foundation recommendations.

Subgrade Preparation

Prior to site grading and fill placement, the existing well house and reservoir should be removed in its entirety. Undocumented fill and deleterious material (e.g., concrete, timber, plastic, etc. associated with the existing well house and reservoir, including all underlying old piping and abandoned foundations) should be removed prior to backfilling the areas with structural fill. Adjacent piping which is left in place should be capped and sealed.

Prior to backfilling any excavation and placement of structural fill to raise site grades, the onsite soils should be scarified to a depth of 8 inches, moisture conditioned to within 2 percent of optimum moisture content, and compacted to a minimum of 95 percent of the maximum dry density (MDD) as determined by ASTM D 1557 (Modified Proctor).

Fill material used to backfill excavations should meet the recommendations discussed in later sections. Site grading activities and compaction of subgrade materials should be observed by the Geotechnical Engineer or qualified persons to note compliance with these recommendations.

Excavation

The Contractor should rely upon his own methods to determine and maintain safe and stable excavations during construction subject to his particular construction procedures and to those subsurface conditions more fully exposed during construction. All excavations should comply at a minimum with the Occupational Safety and Health Administration's (OSHA) construction standards. All excavations should be observed by qualified personnel. The Contractor is ultimately responsible for excavation, trench and site safety.

Fill and Compaction

All structural fill placed for the support of the foundations and building slab should consist of structural fill. This would include the area within the historic reservoir from its prepared subgrade to the elevation of the proposed footing and concrete slab. Structural fill should be limited to approved onsite granular fill soils, or approved imported granular structural fill. All granular structural fill should have a maximum particle size of 3-inches, a fines content (material passing the #200 mesh sieve) between 5 and 25 percent, and a plasticity index of 10 or less. Onsite soils can likely meet this requirement so long as the over-sized particles are screened prior to placement and are free from deleterious materials (including snow, ice or frozen materials). Materials used as structural fill should not be chemically aggressive toward concrete or ferrous materials. Imported fill materials should be approved by the Geotechnical Engineer prior to importing.

General fill, associated with backfilling the historic reservoir and well house, can utilize onsite soils removed as part of site preparation or an import material approved by the Geotechnical Engineer. Onsite soils are suitable for reuse so long over-sized particles (greater than 3 inches in nominal diameter) are removed in addition to all deleterious materials. Deleterious materials consist of historic construction debris and foreign objects that are not soil should not be used. Imported soil associated with the general fill and raising site grades should consist of material with a maximum particle size of 3 inches, a fines content less than 35 percent and an plasticity index of 10 or less. Materials used as structural fill should not be chemically aggressive toward concrete or ferrous materials.

All fill material (structural and general) should be moisture conditioned to within 2 percent of optimum moisture content and compacted on a horizontal plane in maximum 8-inch loose lifts to a minimum of 96 percent (MDD) in accordance with ASTM 1557 (modified proctor compaction effort).

When installing fill against an existing slope, such as in the case of the sloped walls/floors of the former water reservoir, fill should be keyed into the existing slope. In this case, steps of the key should be about 2-foot high and result in a minimum cut width of 4 feet. Fill material should be worked into the key during compaction using horizontal lifts.

FOUNDATIONS

General

We understand that BCA proposes to design the well house using shallow foundations. We understand that net service loads will be on the order of 2,100 lbs/lineal foot and 8,000 lbs for wall and column loads respectively.

Bearing Capacity and Settlement

We understand that one of the design concerns is the potential for differential settlement where the footprints of the new well house and former water reservoir overlap. Previously presented earthwork recommendations address this issue in part. Additionally, we recommend that all of the foundations and floor for the well house bear on at least two feet of compacted structural fill. Implementation of this recommendation will require overexcavation of the entire footprint of the well house.

We recommend that footings be founded at a depth of at least 30 inches below the finished floor elevation to reduce potential frost effects.

For such footings, an allowable bearing capacity of 3,000 psf may be used for design. This value is based on an approximate factor of safety of 3 with respect to shear failure and assumes a minimum footing width of 24 inches. The associated settlement is expected to be 1 inch or less differential settlements less than ½-inch over a distance of 25 feet.

Allowable bearing pressures provided in this document are net allowable bearing pressures, meaning that the weight of all components above the foundation bearing level up to the lowest adjacent grade need not be included in the calculation of the bearing load. The allowable bearing pressure may be increased by one-third for temporary loading conditions such as transient wind and seismic loadings.

Lateral Sliding Resistance

Foundations may be designed with a coefficient of friction of 0.45 when bearing on structural fill. Being an ultimate value, this factor should be considered as representing the maximum resistance to sliding before displacement occurs (i.e., it contains no inherent factor of safety against sliding.

Lateral Earth Pressures

Lateral earth loads acting on short foundation stem walls under static and seismic conditions may be computed using the earth pressure coefficients listed in Table 5. ”At-rest” lateral earth pressures are generally assumed for buried structural elements that are designed for little or no movement/rotation. Elements that can move or deflect sufficiently to develop the strength of the soils and backfill behind a wall can be designed assuming “active” lateral earth pressures for structures. A movement or rotation equal to about 0.1 percent of the buried depth of the element is usually considered to be required to develop lateral earth pressures adjacent to granular soils. Passive lateral earth pressures are generally assumed to resist structure movement. Structure movement of at least 2 percent of the buried depth of the structure element is generally associated with full passive lateral earth pressures. Lateral earth pressures have been provided for sloping and flat ground conditions.

For seismic analyses, the active earth pressure coefficient provided in the table is based on the Mononobe-Okabe pseudo-static approach and only accounts for the dynamic horizontal thrust produced by ground motion. The resulting dynamic thrust pressure *should be added* to the static pressure to determine total pressures on the wall. The pressure distribution of the dynamic horizontal thrust may be treated as a triangle with the point of application at 1/3 the wall height from the base. Unless indicated otherwise, the lateral earth pressure coefficients shown in the table assume horizontal backfill and vertical wall face conditions. Hydrostatic pressures and surcharge loadings should be added to lateral earth pressures as applicable. Over-compaction behind walls should be avoided. Resistive passive earth pressures developed from soils subject to frost or heave, or otherwise above prescribed minimum depths of foundation embedment, should usually be neglected in design.

SITE DRAINAGE AND INFILTRATION

Grading should be planned and executed to provide positive surface drainage away from the foundation of the structure during construction and afterward. Ponding of water around the structures should be avoided. We recommend that all runoff from the roof of the structures and foundations be conveyed directly into an appropriate storm water collection system to avoid depositing water adjacent to the foundation. We also recommend that landscape watering adjacent to structures be avoided to reduce the risk of moisture infiltration to the foundation soils.

We understand that the City is considering the possibility of discharging some unused well water into the natural subgrade as part of well operations, rather than discharging all unused water to a storm drain. Results of the infiltration tests performed are summarized in Table 2. A range of 0.7 inch/hr to greater than 10 inches/hr was calculated for the strata between 3 to 5 feet and 8 to 10 feet below existing grade, respectively. It should be recognized that as shallower soils at the site saturate, the apparent infiltration rate will decrease. We have not evaluated the implications of such a discharge into the natural subgrade. The lateral extent of the strata at the site is unknown, therefore the total water storage capacity (storativity) of the soil is unknown. Where such water would go and what its environmental and engineering impacts might be (including the stability of the slope east of the site above I-15) are also unknown, being beyond our requested scope of work. In any event, any discharge plan should consider the quantity and rate of discharge as well as the potential need for filtering to help provide resistance to piping and internal soil erosion.

PAVEMENT

We understand that a driveway and parking area is planned along the perimeter of the pump house. Limited site-specific traffic loading information has been provided. However, initial considerations by BCA have identified a “smaller tanker type truck hauling about 6,000 to 8,000 gallons” which would access the site once a week, together with a “city dump truck accessing the site two to three times a day and the same for [a rubber tired] backhoe.” Based on this information, and assuming other minimal traffic loadings such as personal cars and pickup trucks for facility personnel, our recommended pavement section (representing an asphalt pavement with a nominal 20-year design life and regularly performed pavement maintenance) consists of 4.5 inches of hot-mix asphalt over 6 inches of untreated base course. Because pavements are susceptible to the effects of frost, a frost-resistance subbase (or additional base) is often used to increase reliability and long-term performance. If such is desired by the City, we recommend a total pavement section thickness of at least 24 inches be used; this means at least an additional 14 inches of frost resistant material (either base or subbase) should be provided below the basic pavement section described above.

All subgrade preparation and pavement section materials (plant mix asphalt, untreated base course and subbase) should conform to the recommendations presented in this document and American Public Works Association (APWA) specifications. Additionally, untreated base course should possess a minimum CBR value of 70, and the granular subbase should possess a minimum CBR value of 25. The asphalt should be compacted to a minimum of 96% of the Marshall (50 blow) maximum density.

LIMITATIONS

Subsurface conditions are inherently variable. It is important that subsurface materials and conditions exposed at the subject area(s) during construction be observed, thereby taking advantage of opportunities to recognize potentially differing site conditions and reduce the risk of unanticipated and/or adverse outcomes. We also recommend that we review project plans and specifications for compatibility with our assessments and recommendations. Additional information regarding such services can be obtained from our office.

The assessments and recommendations presented in this document are based on limited field studies and laboratory testing, as well as our understanding of the project's design and manner of construction. If the project's design or manner of construction changes, or if conditions are found that are different from those described, we should be notified immediately so that we can make revisions as necessary.

This document was prepared solely for the use of the addressee and may not contain sufficient information for other parties or uses.

We represent that our services are performed within the limitations prescribed by our Client, in a manner consistent with the level of care and skill ordinarily exercised by other professional consultants under similar circumstances. No other representation, expressed or implied, and no warranty or guarantee is included or intended. We do not assume responsibility for the accuracy of information provided by others.

TABLES

FIGURES

APPENDICES

REFERENCES

Sack, D. (2005). Geologic Map of the Roy 7.5' Quadrangle, Weber and Davis Counties, Utah. Utah Geological Survey Miscellaneous Publication MP-05-03.

United States Geological Survey (2018a). “Quaternary fault and fold database for the United States”, accessed December 17, 2018 from USGS web site: https://earthquake.usgs.gov/hazards/qfaults/

United States Geological Survey (2018b). “ASCE 7-10 Web Service Documentation”, accessed December 18, 2018 from USGS web site: https://earthquake.usgs.gov/ws/designmaps/asce7-16.html

Table 1 Test Hole Location

Ogden-Hinckley Airport Well House (18-1138)

Test Hole	Test Hole Date	Test Hole Latitude[a]	Test Hole Longitude[a]	Test Hole Elevations (ft)[b]	Test Hole Total Depth (ft)	Drilling Method	Comments
TH-01	12/12/18	41.176220	-111.94539	4444	52	Water- and Mud-Rotary	Backfilled with cuttings

Notes: a) Latitude and longitude estimated by recreational grade hand-held GPS device with reported accuracy of 20 feet.

b) Elevations estimated from survey data provided by Bowen Collins and Assocaites

Table 2 : Summary of Infiltration Tests and Results
Ogden-Hinckley Airport Well House (18-1138)

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Test Hole Designation	Test Hole Elevation[a] (ft)	Depth to Groundwater[b] (ft)	Depth Below Ground Surface (ft)	Field Test Type[c]	Field Measured Infiltration Rate (in/hr)
TH-01	4444	N.M.	3-5	Interval - C	0.67
			8-10	Interval - C	>10

a) Test hole elevations estimated from site survey provided by Bowen Collins. NAVD88 datum
b) N.M. - Not measured at the time of drilling
c) C = Constant head test

Table 3 Laboratory Test Results Summary

Ogden-Hinckley Airport Well House (18-1138)

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Test Hole	Depth (ft)	Moisture content (%)	Grain-Size			Grain-Size Analysis (Percent Finer)											Hydrometer Analysis							Maximum dry unit weight, g_{d-max} (pcf)	Optimum moisture content, w_{opt} (%)	California bearing ratio, CBR (%)
			GRAVEL (No.4 - 3")	SAND (No.200-No.4)	FINES (<No.200)	3-in (75 mm)	1.5-in (37.5 mm)	3/4-in (19 mm)	3/8-in (9.5 mm)	No.4 (4.75 mm)	No.10 (2 mm)	No.20 (0.85 mm)	No.40 (0.425 mm)	No.60 (0.25 mm)	No.100 (0.15 mm)	No.200 (0.075 mm)	0.06 mm	0.05 mm	0.04 mm	0.02 mm	0.01 mm	0.005 mm	0.002 mm			
TH-01	0-5	7.0	33.8	44.1	22.1	100	97	85	71	66	63	61	56	45	33	22	22	21	18	13	10	9	7	127.7	9	9.2
TH-01	3-5	14.6			33.9											34										
TH-01	5-7	4.3	61.5	32.4	6.1	100	100	78	49	38	32	27	22	16	11	6	5	5	5	4	3	2	1			
TH-01	7.5-9.5	11.3	52.2	45.1	2.7	100	100	90	64	48	36	24	11	7	5	3	3	3	3	2	2	1	1			
TH-01	13-15	24.8	0.0	92.1	7.9	100	100	100	100	100	100	99	98	84	30	8	7	7	6	6	5	4	3			
TH-01	20-22	26.7	0.2	85.1	14.7	100	100	100	100	100	100	100	99	86	43	15	13	12	11	8	7	4	3			
TH-01	35-37	23.1	0.0	71.3	28.7	100	100	100	100	100	100	100	100	89	60	29	25	21	18	12	9	8	5			

Table 4 Seismic Design Parmeters

Ogden-Hinckley Airport Well House (18-1138)

Site Class	Type of MCE Acceleration	Mapped Site Class B Acceleration (g)			Site Coefficient			Design Acceleration (g)			
D	Risk-targeted (Structural)	- - -	S_S	S_1	- - -	F_a	F_v	Multiplier	PGA	S_{DS}	S_{D1}
		- - -	1.36	0.47	- - -	1.00	1.53	2/3	0.36	0.91	0.48
	Geo-mean (Geotechnical)	PGA	- - -	- - -	F_{pga}	- - -	- - -	Multiplier	PGA_m	- - -	- - -
		0.59	- - -	- - -	1.00	- - -	- - -	1.0	0.59	- - -	- - -

Table 5 Lateral Earth Pressures

Ogden-Hinckley Airport Well House (18-1138)

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Material	Moist Unit Weight (pcf)	Earth Pressure Coefficients				
		Active Static	Active Seismic Component	At-Rest	At-Rest Seismic Component	Passive Static
Compacted Structural Fill	125	0.28	0.09	0.44	0.36	3.54

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Ogden - Hinckley Airport Well House (18-1138)

Vicinity Map

N

0 1 2 4 Miles

Figure 1

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GERHART COLE	Ogden – Hinckley Airport Well House (18-1138)	N	0 25 50 100 Feet
	Field Study Locations		Figure 2

LEGEND

Ogden - Hinckley Airport Well House Field Study Locations (Gerhart Cole 2018)

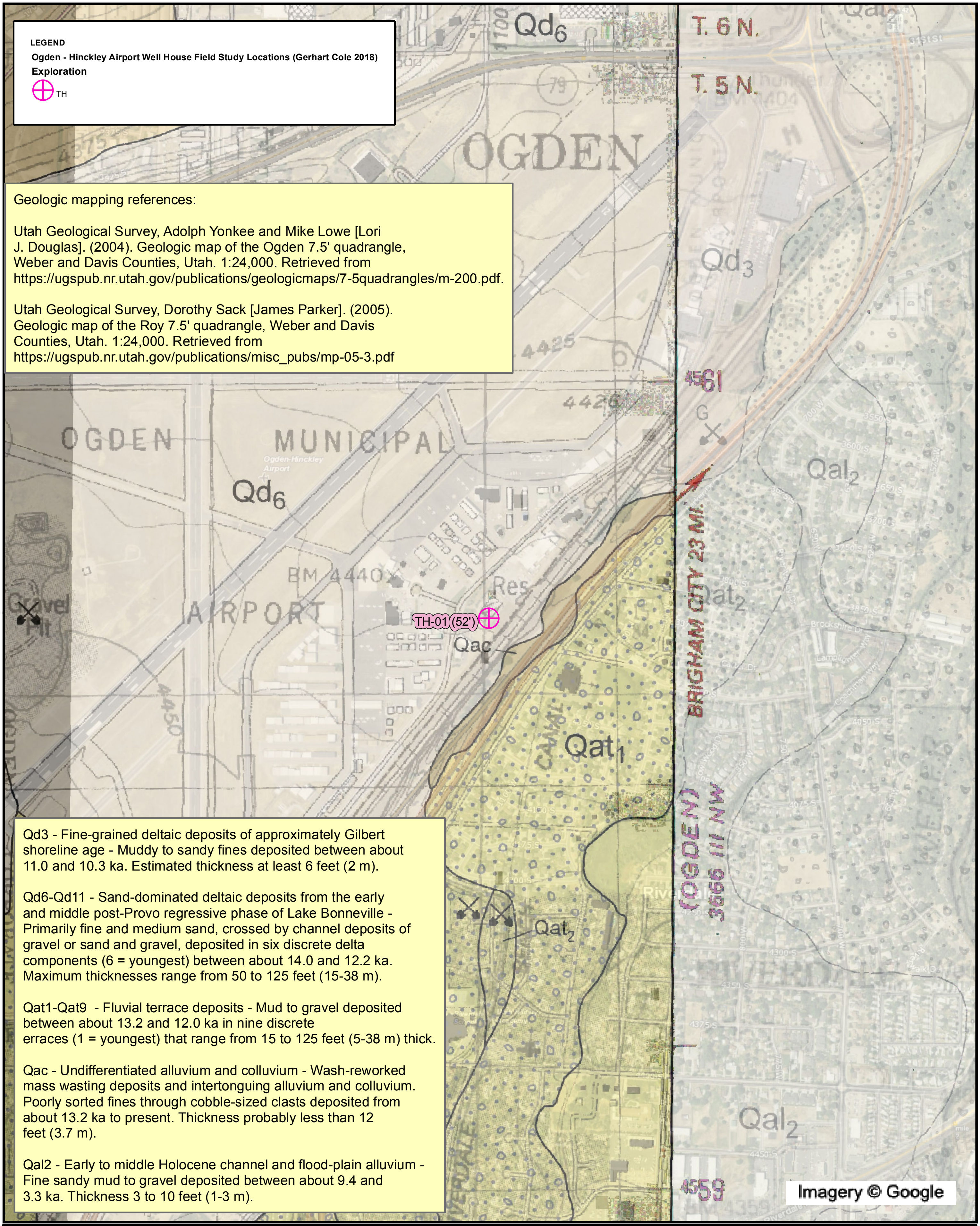
Exploration

⊕ TH

Geologic mapping references:

Utah Geological Survey, Adolph Yonkee and Mike Lowe [Lori J. Douglas]. (2004). Geologic map of the Ogden 7.5' quadrangle, Weber and Davis Counties, Utah. 1:24,000. Retrieved from https://ugspub.nr.utah.gov/publications/geologicmaps/7-5quadrangles/m-200.pdf.

Utah Geological Survey, Dorothy Sack [James Parker]. (2005). Geologic map of the Roy 7.5' quadrangle, Weber and Davis Counties, Utah. 1:24,000. Retrieved from https://ugspub.nr.utah.gov/publications/misc_pubs/mp-05-3.pdf



Qd3 - Fine-grained deltaic deposits of approximately Gilbert shoreline age - Muddy to sandy fines deposited between about 11.0 and 10.3 ka. Estimated thickness at least 6 feet (2 m).

Qd6-Qd11 - Sand-dominated deltaic deposits from the early and middle post-Provo regressive phase of Lake Bonneville - Primarily fine and medium sand, crossed by channel deposits of gravel or sand and gravel, deposited in six discrete delta components (6 = youngest) between about 14.0 and 12.2 ka. Maximum thicknesses range from 50 to 125 feet (15-38 m).

Qat1-Qat9 - Fluvial terrace deposits - Mud to gravel deposited between about 13.2 and 12.0 ka in nine discrete erraces (1 = youngest) that range from 15 to 125 feet (5-38 m) thick.

Qac - Undifferentiated alluvium and colluvium - Wash-reworked mass wasting deposits and intertonguing alluvium and colluvium. Poorly sorted fines through cobble-sized clasts deposited from about 13.2 ka to present. Thickness probably less than 12 feet (3.7 m).

Qal2 - Early to middle Holocene channel and flood-plain alluvium - Fine sandy mud to gravel deposited between about 9.4 and 3.3 ka. Thickness 3 to 10 feet (1-3 m).

GERHART COLE	Ogden – Hinckley Airport Well House (18-1138)	N	0 250 500 1,000 Feet
	Surficial Geologic Mapping		Figure 3

Appendix A

FIELD STUDIES: TEST HOLE DATA

Project: Ogden-Hinckley Airport Well House	**LOG OF TEST HOLE TH-01**
Project Location: Ogden Airport	
Project Number: 18-1138	**Sheet 1 of 2**

Date(s) Drilled	12/12/2018 to 12/12/2018	Logged By	M. Starkie	Checked By	T. Reed
Drilling Method	Rotary Wash/Mud Rotary	Drill Bit Size/Type	3-7/8" Bullet Bit	Total Depth Drilled (feet)	52.0
Drill Rig Type	Simco 2800	Drilling Contractor	A Cache Corp. (Trevor)	Hammer Weight/ Drop (lbs/in.)	Automatic
Apparent Groundwater Depth (feet)	Not Measured	Latitude / Longitude	41.17622 , -111.94539	Ground Surface Elevation (feet)	4444.0 (Approx.)
Comments		Test Hole Backfill	Cuttings	Elevation Datum	NAVD88

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Samples: Sampling Resistance	Samples: Recovery, inches	Graphic Log	Material Description	Field Notes
	0						SAND, gravelly, with silt - loose, moist, brown to dark brown, fine to medium sand, (SM)	- Rotary wash method from 0-10 ft. - Driller advanced casing to 3.0 ft.
		X	SPT-01	4-5-5-6 10	16			
4439	5	X	SPT-02	9-13-13-18 26	10		GRAVEL, sandy, some silt - med. dense, moist, brown to dark brown, fine to coarse sand, fine to coarse gravel, (GP-GM)	- Driller noted rough drilling at 5.0 ft. gravel with possible cobbles.
		X	SPT-03	4-10-11-10 21	12		GRAVEL, sandy, trace silt - medium dense, moist to wet, light brown to brown, fine to coarse gravel, fine to coarse sand (GP)	- SPT- 03 @ 7.5 ft., driller noted the SPT sampler was sitting 2-3 in higher than it should. With rotary wash, water alone could not wash out some of the more coarse cuttings out of the hole.
4434	10	X	SPT-04	6-8-7-9 15	2			- SPT - 04 @ 10 ft., sample possibly disturbed by infiltration test and washing hole out. SPT was sitting high again. - Mud rotary method from 10-52 ft
		X	SPT-05	6-7-8-9 15	12		SAND, some silt - medium dense, moist, light brown to brown, fine to coarse sand, (SP-SM)	
4429	15	X	SPT-06	6-8-9-9 17	16			
							SAND, with silt, trace gravel - medium dense, moist to wet, brown to dark brown, fine to coarse sand. (SM)	
4424	20	X	SPT-07	5-5-6-7 11	18			
4419	25	X	SPT-08	5-9-10-12 19	18			
4414	30							

GERHART COLE

Project: Ogden-Hinckley Airport Well House	LOG OF TEST HOLE TH-01
Project Location: Ogden Airport	
Project Number: 18-1138	Sheet 2 of 2

Date(s) Drilled	12/12/2018 to 12/12/2018	Logged By	M. Starkie	Checked By	T. Reed
Drilling Method	Rotary Wash/Mud Rotary	Drill Bit Size/Type	3-7/8" Bullet Bit	Total Depth Drilled (feet)	52.0
Drill Rig Type	Simco 2800	Drilling Contractor	A Cache Corp. (Trevor)	Hammer Weight/ Drop (lbs/in.)	Automatic
Apparent Groundwater Depth (feet)	Not Measured	Latitude / Longitude	41.17622 , -111.94539	Ground Surface Elevation (feet)	4444.0 (Approx.)
Comments		Test Hole Backfill	Cuttings	Elevation Datum	NAVD88

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Samples: Sampling Resistance	Samples: Recovery, inches	Graphic Log	Material Description	Field Notes
		X	SPT-09	6-6-6-7 12	18		SAND, with silt, trace gravel - medium dense, moist to wet, brown to dark brown, fine to coarse sand. (SM) - iron oxide staining	
4409	35	X	SPT-10	5-6-7-9 13	17			
4404	40	X	SPT-11	5-7-7-9 14	19			
4399	45	X	SPT-12	2-6-9-10 15	19		CLAY, with sand - medium stiff to stiff, wet, light brown to brown, occasional sand seams (up to 1/4-in. thick). (CL) SAND, some clay - medium dense, moist to wet, light brown to brown, frequent clay seams (up to 1/4-in. thick). (SP-SC)	
4394	50	X	SPT-13	4-6-9-11 15	16		CLAY - medium stiff to stiff, wet, light brown to brown. (CL) SAND, some clay - medium dense, moist to wet, light brown to brown, frequent clay seams (up to 1/4-in. thick). (SP-SC) Bottom of Hole at 52 feet	
4389	55							
4384	60							

GERHART COLE

Unified Soil Classification System (USCS)

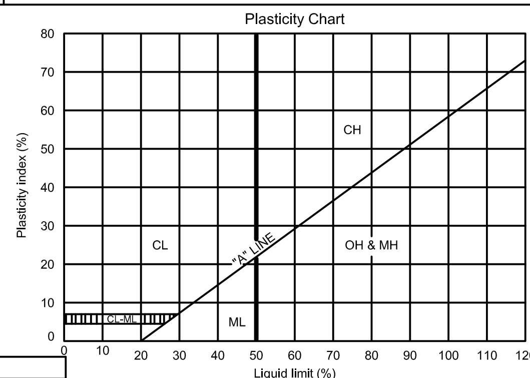
Material Types	Major Soil Divisions		Group Symbol and Legend	Typical Names
COARSE-GRAINED SOILS >50% retained on No. 200 sieve	GRAVELS >50% of coarse fraction retained on No. 4 Sieve	Clean GRAVELS (little or no fines)	GW	Well-Graded GRAVEL, GRAVEL-sand mixtures, few fines
			GP	Poorly-Graded GRAVEL, GRAVEL-sand mixtures, few fines
		GRAVELS with fines (appreciable amount of fines)	GM	Silty GRAVEL, GRAVEL-sand silt mixtures
			GC	Clayey GRAVEL, GRAVEL-sand clay mixtures
	SANDS >50% of coarse fraction passing the No. 4 sieve	Clean SANDS (little or no fines)	SW	Well-Graded SAND, SAND-gravel mixtures, few fines
			SP	Poorly-Graded SAND, SAND-gravel mixtures, few fines
		SANDS with fines (appreciable amount of fines)	SM	Silty SAND, SAND-silt mixtures
			SC	Clayey SAND, SAND-clay mixtures
FINE-GRAINED SOILS >50% Passing No. 200 Sieve	SILTS and CLAYS liquid limit < 50	Inorganic 1) CF > 30%: + Sandy/Gravelly 2) CF = 15-30% + with sand/gravel	CL	Lean CLAY, Gravelly/Sandy CLAY, low to med. plasticity
			ML	SILT, Gravelly/Sandy SILT, no to slight plasticity
		Organic	OL	Organic CLAY or SILT
	SILTS and CLAYS liquid limit > 50	Inorganic 1) CF > 30%: + Sandy/Gravelly 2) CF = 15-30% + with sand/gravel	CH	Fat CLAY, Gravelly/Sandy Fat CLAY, high plasticity
			MH	Elastic SILT, Gravelly/Sandy Elastic SILT, low to high plasticity
		Organic	OH	Organic CLAY or SILT
Highly organic soils		Primarily Organic Matter; Organic Odor	PT	PEAT
Boulders / Cobbles		> 50% (by volume) particles > 3"	COBBLES BOULDERS	Boulders (>12"); Cobbles (>3" and <12")

Other Material Symbols

- Bedrock
- Asphalt
- Concrete
- Topsoil
- Fill

Sample Types

- Auger Cuttings
- Continuous sampler
- Grab Sample
- No Recovery
- Shelby Tube
- Standard Penetration Test (SPT)
- California Sampler
- Rock Core
- Modified California Sampler
- Other (see remarks)
- Piston Sampler (Shelby Tube)
- Vane Shear



Apparent water level   Measured water level

Descriptors for Coarse Grained Soils

Apparent Density	Dr (%)	SPT	MC	CAL
Very Loose	0-15	<4	<6	<8
Loose	15-35	4-10	6-15	8-20
Med. Dense	35-65	10-30	15-42	20-56
Dense	65-85	30-50	42-72	56-96
Very Dense	85-100	>50	>72	>96

Descriptors for Fine Grained Soils

Consistency	Su (psf)	SPT	MC	CAL
Very Soft	< 250	<2	<2	<2
Soft	250-500	2-4	2-4	2-5
Med. Stiff	500-1000	4-8	4-10	5-11
Stiff	1000-2000	8-15	10-19	11-22
Very Stiff	2000-4000	15-30	19-37	22-45
Hard	>4000	>30	>37	>45

SPT - Standard split spoon (SPT): 2" OD, 1.375" ID
MC - Modified California: 2.5" OD, 1.875" ID
CAL - California: 3" OD, 2.375" ID
R - Practical Refusal For Sampling Method

Stratification

Description	Criteria
Seam	1/16" to 1/2"
Layer	1/2" to 12"
Occasional	<= 1 per ft. thickness
Frequent	> 1 per ft. thickness

Modifiers

Description	Est. (%)
Trace	<5
Some	5-12
With	12-30
-ly	>30

Descriptors for Moisture

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

Descriptors for Particle Size

Description	Criteria
Boulder	>12" : larger than a basketball
Cobble	3-12" : larger than a grapefruit
Coarse Gravel	3/4-3" : larger than a grape
Fine Gravel	No.4-3/4" : larger than a pea
Coarse Sand	No.10-4 : larger than a rock salt grain
Medium Sand	No.40-10 : larger than window screen opening
Fine Sand	No.200-40 : larger than a sugar grain

Descriptors for Particle Angularity

Description	Criteria
Angular	Sharp edges, rel. plane sides, unpolished surface
Subangular	Similar to angular, but with rounded edges
Subrounded	Nearly plane sides, well-rounded corners & edges
Rounded	Smoothly curved sides and no edges

General Notes:
1) Stratigraphic lines on the logs represent approximate boundaries.
2) No warranty is provided as to the continuity of soil conditions beyond or between points explored and sample locations.
3) Logs represent soil conditions observed at the point of exploration on the date indicated.
4) Visual methods were used to classify the materials in general accordance with Unified Soil Classification System; actual designations based on laboratory methods may vary.

Abbreviated Soil Classification Symbols (after ASTM D2488 X.5)

Prefix
s = sandy
g = gravelly

Suffix
s = with sand
g = with gravel
c = with cobbles
b = with boulders

Abbreviated system for supplementary presentations when complete description is referenced. Examples:

Group Symbol and Full Name	Abbreviated
Sandy Lean CLAY	s(CL)
Poorly Graded SAND with silt and gravel	(SP-SM)g
Poorly Graded Gravel with sand, cobbles, and boulders	(GP)scb
Gravelly SILT with sand and cobbles	g(ML)sc

Abbreviations for Laboratory Testing

Description	Criteria
w	Water/moisture content
UU	Unconsolidated undrained Triaxial
CON	One-dimensional consolidation
AL	Atterberg Limits (LL=Liquid Limit & PI=Plasticity Index)
SV	Sieve / Grain-Size Distribution Testing
PR	Lab Compaction Characteristics of Soil (Proctor)
CBR	California Bearing Ratio
FC	Percent Passing #200 Sieve

GERHART COLE

Legend to Soil Descriptions

Figure A-1

Appendix B

LAB TEST DATA

Laboratory Compaction Characteristics of Soil

after ASTM D698 / D1557

GERHART COLE

Project: Ogden-Hinckley Airport Wellhouse
No: 18-1138
Date: 11-Dec-18
Tested by: MGS
Reduced by: MGS
Reviewed by: zmg

TH/TP/Sample: TH-01
Depth: 0-5 Bulk
Location: Ogden, UT

Comments:

Test Summary

Method: ASTM D698 C
Mold volume (ft^3): 0.0750

Laboratory sample description: BWN-dk. BWN
Engineering Classification: Not requested
As-received moisture content (%): Not requested
Preparation method: Moist
Rammer: Manual
Rock Correction: Yes

Optimum moisture content (%): 10.5
Maximum dry unit weight (pcf): 123.6

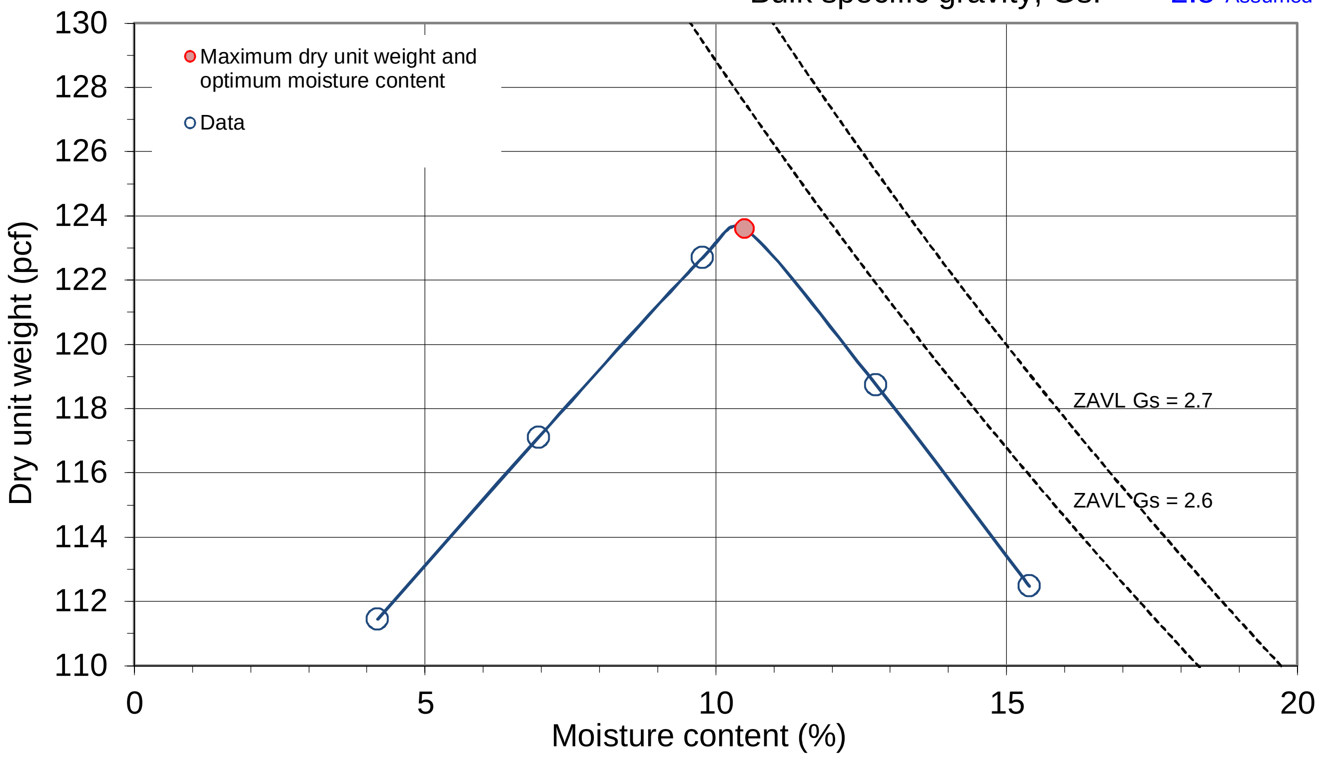
Point Number	as-is	-3	+6	+3	+9
Wt. mold + wet soil (g)	10749.4	10438.7	11043.4	11071.0	10904.0
Wt. mold (g)	6487.56	6487.56	6487.56	6487.56	6487.56
Moist unit wt., gd (pcf)	125.2	116.1	133.9	134.7	129.8
Wet soil + tare (g)	969.88	1335.08	1137.21	1275.93	1071.6
Dry soil + tare (g)	935.49	1306.7	1078.56	1218.39	1012.49
Tare (g)	440.73	628.45	618.73	629.71	628.45
Moisture content, w (%)	7.0	4.2	12.8	9.8	15.4
Dry unit wt., gd (pcf)	117.1	111.4	118.7	122.7	112.5

*Correction of Unit Weight and Water Content for Soils Containing Oversize Particles

(ASTM D4718)

Corrected moisture content (%): 9.0
Corrected dry unit weight (pcf): 127.7

Oversized fraction, +3/4-in. (%): 15.5
Moisture content, +3/4-in. (%): 0.8
Sieve for oversized fraction: 3/4-in.
Bulk specific gravity, Gs: 2.5 Assumed



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California Bearing Ratio

(After ASTM D 1883 and AASHTO T193)

GERHART COLE

Project: **Ogden-Hinckley Airport Wellhouse**
No: **18-1138**
Date: 13-Dec-18
Tested by: MGS
Reduced by: MGS
Reviewed by: zmg
Comments:

TH/TP/Sample: **TH-01**
Depth: **0-5 ft**
Location: **Ogden, UT**

Test Summary

Maximum dry unit weight (pcf):	123.6	Reference method:	ASTM D698 C
Optimum moisture content (%):	10.5	Eng. classification:	Not requested
Relative compaction (%):	100.4	Condition of sample:	Soaked
Corrected CBR at 0.1-in. (%)	**7.8**	Scalp and replace:	No
Corrected CBR at 0.2-in. (%)	**9.2**		

Compaction Data

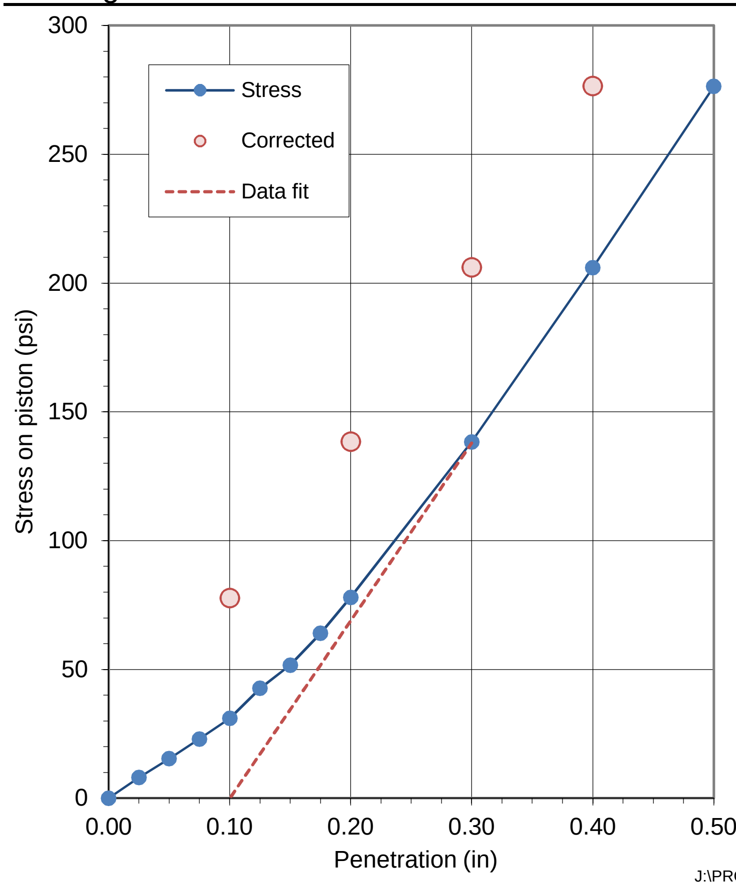
	As-Comp.	After Soak	Top 1-in.
Wt. mold + moist soil (g)	8986.76	8987.95	
Wt. mold (g)	4318.00	4318.00	
Mold volume (ft^3)	0.0750	0.07489	
Moist unit wt., gm (pcf)	137.236	137.481	
Moist soil + tare (g)	743.26	1570.74	443.99
Dry soil + tare (g)	688.89	1466.8	415.65
Tare (g)	172.98	440.45	145.89
Moisture content, w (%)	10.5	10.1	10.5
Dry unit wt., gd (pcf)	124.2	124.8	

Swell Data

Date	Time	Dial (in)
12/14	8:30	0.344
12/18	8:25	0.337

Soaking Period (hr)	96
Ho (in)	4.584
Hf (in)	4.577
Swell (%)	-0.15
Surcharge (psf)	50

Bearing Test Results



Penetration (in)	Meas. Stress (psi)	Corrected Str. (psi)	Standard Stress (psi)	Bearing Ratios
0	0	32		
0.025	8	37		
0.05	15	46		
0.075	23	55		
0.1	31	78	1000	7.8
0.125	43	79	1125	7.0
0.15	52	101	1250	8.1
0.175	64	120	1375	8.7
0.2	78	138	1500	9.2
0.3	138	206	1900	10.8
0.4	206	276	2300	12.0
0.5	276		2600	