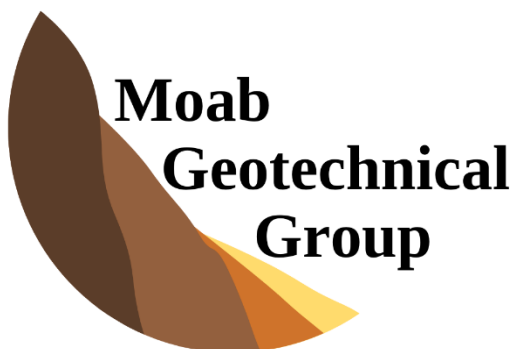


**GEOTECHNICAL DESIGN  
REPORT**

**WELLINGTON CITY BUILDING  
150 W MAIN STREET  
WELLINGTON, UTAH**

**May 5, 2026**





May 5, 2026

Jones & DeMille Engineering  
c/o Wyatt Hansen, PE  
1675 South Highway 10  
Price, UT 84501

Mr. Hansen,

We prepared this design-level geotechnical report for the Wellington City Building renovation and addition in general conformance with the 2021 International Building Code (IBC). The following design recommendations and construction considerations were prepared based on findings from the field studies and subsequent laboratory testing. Based on our findings, the proposed development is feasible provided the following recommendations and considerations are followed.

We appreciate the opportunity to provide this service for you. If there are any questions or comments related to the information contained within, please call and we will be glad to discuss them with you.

Sincerely,

A blue ink handwritten signature is written over a circular professional seal. The seal is for Taylor Quinn Hall, a Registered Professional Engineer in the State of Utah, with license number 10113401-2202. The seal also includes the text "ELECTRONIC SEAL".

Taylor Hall, PE  
Principal

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# 1 INTRODUCTION

This report provides design recommendations and construction considerations for the proposed addition and renovation to the Wellington City Building in Wellington, Utah. The recommendations are based on the findings from the field investigation and subsequent laboratory testing. Should the structures and proposed site development change from what has been detailed in this report, Moab Geotechnical Group (MGG) should be contacted immediately to determine the relevance of the following recommendations and considerations.

## 1.1 Scope of Services

The following scope of services were performed in accordance with the Professional Services Agreement, executed on October 20, 2025, between MGG and Jones & DeMille Engineering, in support of the design-level geotechnical investigation:

- Three (3) Dynamic Cone Penetrometer (DCP) tests were performed using a 17.6-pound hammer to estimate in situ strength and bearing resistance.
- Three (3) borings drilled using a CME-75 drill rig to characterize subsurface conditions.
- A licensed Professional Engineer logged the borings, classified soil samples in the field in general accordance with the Unified Soil Classification System (USCS), and collected representative soil samples for laboratory testing,
- Collected soil samples were transported to geotechnical laboratories for further testing and classification,
- Results of the field and laboratory investigations were interpreted and incorporated into this report, including figures and appendices.

Additional services to support the construction of the project are discussed later in this report for your consideration. Experience has shown these additional services help ensure the following recommendations are consistent with findings during construction. Subsurface conditions are inherently heterogenous and often change over short distances.

## 1.2 Project Description

The proposed project involves improvements to the Wellington City Building in Wellington, Utah, including construction of a new 7,800-square-foot municipal building for City Administration, Council Chambers, and the Police Department, as well as a 3,372-square-foot Fire Apparatus Bay. The project also includes renovation of approximately 7,340 square feet of existing fire facilities and associated site improvements such as grading, paving, accessibility upgrades, landscaping, and demolition of roughly 25,050 square feet of existing structures. It is understood that the existing northeast wing is on a crawl space with a concrete floor within the crawl space and main floor. The southeast portion has a basement mechanical room where the boiler is and has crawl space for the main wood floor. The old gym on the northeast contains a wood floor and crawl space and a basement for the mechanical room in the far north. The existing city administration building on the southwest corner is mostly slab on grade but does have a concrete basement room for mechanical equipment. This geotechnical investigation was performed to support the design and construction of these planned improvements.

It is understood that the proposed structures are planned as one to two-story, at-grade buildings with no basement.

### 1.3 Existing and Historic Information

Publicly available historical aerial imagery (NETR, 2025) was reviewed to assess past development and land use at the project site. The earliest available imagery from 1952 shows portions of the existing structure, with building additions first appearing in the 1962 and 1997 photographs. A small structure in the southwest corner of the property is visible in 1952 but absent from 1962 onward. Throughout the available record, the site has remained predominantly developed, consisting of buildings, pavement, landscape islands, several scattered trees, and a grass area along the east side of the property. No significant changes in site development are apparent between the 1997 imagery and present. A current aerial image is provided in Figure 2.

A review of the Utah Geological Survey (UGS) GeoData Archive did not identify any pertinent subsurface data or geotechnical reports for the project site.

## 2 SITE CHARACTERIZATION

Moab Geotechnical Group (MGG) performed a subsurface investigation to evaluate soil conditions at the project site. The scope included exploratory borings, in situ penetration testing, and laboratory testing of recovered samples to characterize engineering properties. Field explorations were completed within and adjacent to proposed building footprints and planned parking and drive-lane areas to assess soil strength, composition, and variability with depth. Exploration locations were selected based on the project plans available at the time of drilling. Some locations were offset from proposed improvement due to the presence of existing structures. Approximate boring and test locations are shown on Figures 2a and 2b.

### 2.1 Exploratory Borings

MGG retained South Slope Drilling of Altonah, Utah to perform the exploratory borings. Three borings (B-01 through B-03) were advanced throughout the project area using a truck-mounted CME-75 drill rig equipped with 6.5-inch outer diameter hollow-stem augers. The borings were advanced to a depths ranging approximately 21.5 to 26.5 feet below the ground surface.

Sampling was conducted to classify the subsurface materials and evaluate their strength and consistency. Disturbed samples were collected into sealed plastic bags or 5-gallon buckets at the time of excavation. Undisturbed samples were obtained using thin-walled Shelby tubes, that were sealed with caps and tape immediately upon collection. Each boring was backfilled with drill cuttings and an asphalt cold patch upon completion. A description of the subsurface materials encountered is provided in Section 2.6 and a graphical summary of the boring logs is provided in Appendix A.

The latitude and longitude coordinates of the borings were estimated using handheld GPS devices with a reported accuracy of approximately 10 feet. Surface elevations were estimated based on the USGS 1/3 arc-second digital elevation model (USGS, 2024).

### 2.2 Dynamic Cone Penetrometer

Six Dynamic Cone Penetrometer (DCP) tests (DCP-01 through DCP-03) were conducted in accordance with ASTM D6951 using a hand-held 17.6-pound drop hammer and sacrificial 60° cone tips with a diameter of 0.79 inches. The purpose of the testing was to characterize the strength and stiffness of near-surface soils across the site. Test locations were selected within and nearby the drive lanes, flatwork, and one within the building footprint. Exploration locations are illustrated in Figures 2a and 2b.

DCP testing provides empirical correlations to California Bearing Ratio (CBR), elastic modulus, and allowable bearing capacity, as summarized in Exhibit 2-1. Correlation charts and interpreted plots are presented in Appendix C.

#### Exhibit 2-1 DCP Correlations

Parameter	Correlation	Reference
Depth per Impact (DPI)	Calculated in inches/blow	N/A
California Bearing Ratio (CBR)	$CBR (\%) = 292 / (DPI * 25.4)^{1.12}$ [DPI in in/blow]	ASTM, 2023
Elastic Modulus	$E_{DPI} (MPa) = 10^{3.04758 - [1.06166 \log(DPI)]}$ [DPI in mm/blow]	Lockwood et al., 1992
Bearing Capacity	$q (psi) = 3.794 * CBR^{0.664}$	Packard, R.G., 1973



### 2.3 Laboratory Testing

Laboratory testing was performed on select soil samples by MGG and by Intermountain GeoEnvironmental Services, Inc. (IGES), of South Salt Lake, Utah. The testing program included the following:

- Index Testing (Moisture Content, ASTM D2216; Unit Weight, ASTM D7263; Particle Size Distribution – Sieve Analysis, ASTM D6913; Hydrometer Analysis, ASTM D7928; Fines Content, ASTM D1140; Atterberg Limits, ASTM D4318);
- Swell or Collapse Potential (ASTM D4546)
- Compaction Characteristics (Modified Proctor, ASTM D1557);
- Corrosion and Resistivity (Minimum Resistivity, AASHTO T 288; Soil pH, AASHTO T 289; Soluble Chloride, HACH 8113; Soluble Sulfate, HACH 8051).

Results of the laboratory tests are provided in Appendix B. Select properties are also summarized on the boring logs provided in Appendix A.

### 2.4 Surface Conditions

The proposed building site is located on a relatively flat parcel approximately 1,000 feet north-northwest of the Price River and about 25 feet above the riverbank elevation. The site is bordered to the south by US Highway 191, with a railroad corridor running roughly parallel to the highway. Ground surface elevations gently decline from northwest to southeast, ranging from approximately 5,419 to 5,413 feet (USGS 1/3-arc-second DEM, 2024). A shallow topographic depression is present along the north side of the existing building in the pavement. The site is primarily developed with existing structures and paved surfaces, with limited vegetation consisting of lawn areas, sparse shrubs, and several trees. Vegetation is mainly concentrated near the north wing of the existing structure and along the highway frontage within landscaped strips and parking islands.

### 2.5 Subsurface Conditions

Subsurface conditions generally consisted of 2½ to six (6) inches of asphalt, underlain in B-01 and B-02 by suspected fill, comprising lean clay and clayey gravel, respectively, extending to approximately 2 feet below ground surface (bgs). Beneath the fill, or beneath the asphalt in the case of B-03, medium stiff lean clay (USCS CL) was encountered to depths of approximately six (6) to 6½ feet bgs. This was underlain by dense to very dense poorly graded to clayey gravel (USCS GP and GC) to depths of approximately 14 to 16 feet bgs. Below this, highly to slightly weathered shale interpreted as Mancos Shale was encountered, extending to the total depths explored (approximately 21.5 to 26.5 feet). Based on field identification and Standard Penetration Tests (SPT), the shale was interpreted to range from hard clay to weak rock. A graphical summary of the boring logs is provided in Appendix A.

### 2.6 Groundwater

Groundwater was encountered in two borings at depths ranging from approximately 16 to 20.5 feet below ground surface. These levels were observed during and shortly after drilling. Based on the stratigraphy, the encountered water is interpreted as a perched condition within the gravel layer overlying the shale bedrock, rather than the regional groundwater table. Regional groundwater is anticipated to be significantly deeper at this location; however, no nearby well logs were available for confirmation. Groundwater levels can fluctuate seasonally and from year to year in response to irrigation practices, surface runoff, and precipitation.

## 3 GEOLOGICAL AND GEOTECHNICAL HAZARDS AND FINDINGS

Geological and geotechnical hazards exist at nearly every site in the world. Due to the chaotic environment in which the soil and rock are deposited, uplifted, faulted, and compressed, hazards are to be expected. The following hazards are prevalent throughout the Colorado Plateau and Southwest Regions. Results of the site characterization and experience were used to characterize the risk that each of the following hazards will impose on the proposed development.

### 3.1 Geologic Setting

Figure 3 illustrates the project site in relation to regional geologic mapping (Witkind, 1979). The site is mapped within the Quaternary Slope Wash unit (Qsw), described as clay- and silt-rich material derived from the disintegration of the Mancos Shale and reworked by sheetwash processes not confined to defined channels. Subsurface conditions encountered during the investigation are generally consistent with this mapping. However, the gravel layer underlying the upper clay appears more characteristic of nearby Quaternary Alluvium (Qal), which is mapped along the Price River floodplain and may represent a localized alluvial deposit not fully delineated at the regional mapping scale.

### 3.2 Landslide and Slope Stability

The project site lies within an area mapped by USGS (2024) as having no landslide susceptibility. The relatively flat topography and absence of steep slopes or unstable terrain support this designation. The risk of slope instability adversely impacting the site is considered negligible.

### 3.3 Radon

Radon is a common geologic hazard throughout the Colorado Plateau due to the natural decay of uranium-bearing minerals present in bedrock and alluvial soils. This colorless, odorless gas can accumulate in enclosed structures and poses health risks with prolonged exposure.

Both the new construction and renovation components of the project should consider the current ANSI/AARST Radon Standards (2023). Separate standards apply to new buildings and to existing structures undergoing renovation, each providing prescriptive requirements and mitigation practices for reducing indoor radon and other soil gases. Project-specific recommendations can be provided upon request.

### 3.4 Problematic Soils

Problematic soils generally consist of soluble, collapsible, and expansive soils in the Southwest and Colorado Plateau Regions. The following subsections detail each hazard as it relates to the Project.

#### 3.4.1 Collapsible Soils

Collapsible soils are soils that have the potential to undergo significant volumetric change once loaded or wetted. Generally, the soils are deposited during flood, debris flow, or landslides and can become cemented from naturally occurring processes. Once wetted, or loaded beyond the compressive strength of the soil, the soils can break down and collapse. This behavior is often non-uniform and may result in differential settlement.

The lean clay observed in the exploratory borings has the potential to be collapsible. Collapse potential was evaluated in accordance with ASTM D4546. Results are summarized in Exhibit 3-1, and complete data is presented in Appendix B. The recommendations presented in Sections 4.0 and 5.0 should be followed to mitigate the potential effects of collapsible soils on proposed improvements.

### Exhibit 3-1 Collapse Test Results

Exploration	Sample ID	Depth (ft)	Inundation Stress (psf)	Swell/Collapse (%)
B-01	ST-02	5 – 7	1000	0.6 Collapse
B-02	ST-02	4 – 6	1000	0.5 Collapse
B-03	ST-02	4 – 6	1000	0.6 Collapse

#### 3.4.2 Expansive Soils

Expansive soils generally occur when cohesive soils or weathered shales become wet. The plate-like nature of the material and its mineralogy determine how expansive these materials are. Wetting due to irrigation practices, site drainage, precipitation, and utilities often occurs irregularly around structures and can lead to differential movement of expansive soils. This differential movement is often presented by ‘heaving’ flatwork, cracking foundations, jamming windows and doors, and inoperable utilities.

Swell potential was evaluated in accordance with ASTM D4546. Results are summarized in Exhibit 3-1, with complete data provided in Appendix B. The tested upper lean clay layer exhibited collapse behavior under wetting rather than swell. The underlying weathered shale may contain clay minerals with some expansion potential; however, relatively undisturbed Shelby Tube samples could not be obtained due to the shale’s strength and fissility. Index testing of disturbed shale samples indicates an Activity (A) between 0.44 and 0.73, which is characterized by Skempton (1953) as having low swell potential. Based on available data, the potential for swelling soils to adversely impact the proposed improvements is considered low.

#### 3.4.3 Soluble Soils and Rock

Soluble soil and rock are a known geologic hazard within the Colorado Plateau, originating from the region’s marine depositional history. Prehistoric ocean environments produced widespread accumulations of soluble minerals, such as salts, gypsum, and carbonates, that have since been transported, concentrated, and redistributed by springs, flooding, and groundwater fluctuations. Because these minerals can occur in discontinuous lenses or thin horizons, their presence in the subsurface is difficult to predict.

During this investigation, white, chalky material, suspected to be calcium carbonate, was observed in boring B-02 at various depths, with an estimated volume of less than 5 percent. Calcium carbonate is considered moderately soluble, and its dissolution may reduce cementation and contribute to ground softening or volume loss, particularly where present in higher concentrations. However, based on the limited observed quantity of potentially soluble material, the potential for soluble soils or rock to adversely impact the proposed improvements is considered low. If distinct horizons or pockets of white, potentially soluble material (e.g., salts, gypsum, or more concentrated carbonate deposits) are encountered during construction, MGG should be contacted for further evaluation.

#### 3.5 Reactive Soils

Corrosive or chemically aggressive soil environments can pose hazards to unprotected steel, concrete, and buried utilities. Corrosion and resistivity testing was performed on select samples obtained during the field investigation. Results are summarized in Exhibit 3-2 and provided in full in Appendix B.

**Exhibit 3-2 Corrosion and Resistivity Test Results**

Exploration	Sample ID	Depth (ft)	pH	Soluble Sulfate (ppm)	Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-03	BULK-03	2 – 4	8.3	320	355	665

The minimum measured resistivity of 665 ohm-cm indicates an extremely corrosive environment for unprotected steel elements, based on industry criteria (Roberge, 2008). All metallic components in direct contact with native soils should be designed for extremely corrosive conditions. Where feasible, the use of non-metallic utility materials is recommended.

Soluble sulfate concentrations were measured at 320 ppm, corresponding to Exposure Class S1 per ACI 201.2R-16 (ACI, 2016). Concrete placed in contact with native soils should comply with ACI requirements for this exposure class to mitigate the potential for sulfate attack.

Prior to use, future imported fill materials should be tested for corrosion potential (pH, sulfates, chlorides, resistivity) to determine their potential for corrosion towards concrete and untreated steel.

**3.6 Piping and Erosion**

Erosional gullies can form where runoff becomes concentrated during heavy rain events, and piping (or tunnel erosion) can occur when subsurface flow moves through permeable soils under a sufficient hydraulic gradient, discharging at a free face. Fines-rich soils, including those with high silt or clay content or soluble minerals, are generally more susceptible to these processes.

At the project site, surficial soils contain high fines and could experience localized erosion if left unprotected during storm events. However, the site is largely paved and bounded by curb and gutter, and significant erosion is not expected with proper site grading. Piping is also unlikely due to the limited topographic relief and lack of conditions that would induce a strong hydraulic gradient.

Recommendations are provided in the *Drainage and Landscaping* section of this report. With appropriate grading around building foundations and implementation of a comprehensive site grading plan prepared by a licensed Civil Engineer, the risk of erosion or piping affecting planned structures is considered negligible.

**3.7 Shallow Bedrock**

Shallow bedrock can influence excavation conditions and contribute to localized perched groundwater. Weathered Mancos Shale was encountered at depths of approximately 14 to 16 feet and was classified as moderately to highly weathered and very weak. Because the planned structures are at-grade and no deep excavations are anticipated, shallow bedrock is not expected to pose excavation challenges.

### 3.8 Seismic

The site could be subject to strong to very strong ground shaking, according to the Utah Geologic Survey (UGS, 2025), from nearby or more distant, large magnitude earthquakes occurring during the expected life span of the project. This hazard is managed by structural design of the buildings per the applicable edition of the International Building Code (IBC). Seismic design parameters are provided in the *Recommendations* section of this report.

The potential for earthquake surface fault rupture at the site is considered low. Surface fault rupture is the result of movement on an active fault reaching the ground surface. Surface rupture occurs when displacement on an active fault propagates to the ground surface. No evidence of active faulting was observed during the desktop study or field exploration. In addition, the United States Geological Survey (USGS) Quaternary Faults interactive map does not show any mapped faults within the project boundaries.

The potential for liquefaction and associated secondary effects is also considered low. Liquefaction involves the rapid loss of soil shear strength within saturated, loose to medium-dense sands or non-plastic silts due to pore-pressure buildup during earthquake shaking. Secondary manifestations may include sand boils, settlement, and slope instabilities such as lateral spreading or flow deformation. Because the subsurface profile lacks loose, saturated fine grained granular soils, the risk of liquefaction is low.

## 4 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following geotechnical design recommendations have been developed based on the results of the field studies, laboratory testing, review of publicly available information, knowledge of the proposed structures, and experience. The performance of the planned structures and infrastructure is dependent on conformance with the following recommendations.

The following recommendations and design criteria are based on the Project description in Section 1.2 of this report. If changes are made to the Project, Moab Geotechnical Group should be contacted immediately to determine the validity of the following recommendations.

### 4.1 Seismic Design Parameters

The site is located within a seismically active region. Seismic design parameters were developed using the ASCE 7 Hazard Tool based on 2021 IBC and ASCE 7-16 requirements. Site Class D was selected as a conservative assumption given the available subsurface information; this is appropriate for the proposed low-rise structures, which are expected to have fundamental periods less than 0.5 seconds. It is understood that the risk category may be different for each building and therefore seismic design parameters have been prepared for Risk Category II, III, and IV. Based on a Site Class D designation of this site, the seismic design parameters do not fluctuate. Exhibits 4-1 summarize the resulting seismic design parameters for use in structural design.

#### Exhibit 4-1 Seismic Design Parameters for Risk Category 2 - 4

Period (seconds)	Spectral Acceleration (g)
Site Modified Peak Ground Acceleration, $PGA_M$	0.310
$MCE_R$ Spectral Response Acceleration at Short Periods, $S_{MS}$ (g)	0.608
$MCE_R$ Spectral Response Acceleration at 1-second Period, $S_{M1}$ (g)	0.283
Design Spectral Response Acceleration at Short Period, $S_{DS}$ (g)	0.406
Design Spectral Response Acceleration at 1-second Period, $S_{D1}$ (g)	0.188

Notes: 1) Based on assumed site location of Latitude: 39.542713° & Longitude: -110.737708°  
 2) Based on Risk Category II, III, IV

### 4.2 Lateral Earth Pressures

Lateral earth pressures will impart loads onto structures retaining earth loads. Exhibit 4-2 provides a summary of the lateral earth pressures for use in the design of walls, buried structures and other elements of the project. Active earth pressures are the earth pressures to be used for conditions where movements greater than 0.003H (H is the height of the wall in feet) are allowed for the structure. At rest pressures should otherwise be used as the earth pressures in the design of structures.

Passive earth pressures are to be used to resist the lateral earth pressures. Passive earth pressures should be ignored in the upper 24 inches of the ground surface due to the potential for frost to impact these soils. Passive earth pressures can be assumed to increase linearly in the frost susceptible zone. Exhibit 4-2 provides a summary for both native and imported structural fill soils.

**Exhibit 4-2 Summary of Lateral Earth Pressures**

Soil Type	Slope Behind Wall, $\beta$	Condition	Earth Pressure Coefficient			Equivalent Fluid Weight (psf/ft)		
			Active	At Rest	Passive	Active	At Rest	Passive
Native Soil	0°	Standard <sup>1</sup>	0.44	0.66	2.04	56	84	185
		Seismic <sup>2,3,4</sup>	0.15	0.27	1.80	19	36	165
Structural Fill <sup>5</sup>	0°	Standard <sup>1</sup>	0.25	0.44	3.54	35	60	420
		Seismic <sup>2,3,4</sup>	0.11	0.24	3.23	11	28	380

Notes: psf – pounds per square foot, ft – feet

1. The static passive resistance can be increased by 1/3 to address other extreme events due to wind or snow loading.
2. The seismic earth pressures were calculated using the Mononobe Okabe methodology (Mononobe, 1929) (Okabe, 1926).
3. The seismic active and at rest earth pressure coefficients should be added to their respective static term.
4. The seismic passive earth pressure coefficient is the discounted static term and does not require any further factoring.
5. Structural fill material and placement in accordance with the *Fill Placement* section of this report.

**4.3 Foundations**

The foundation subgrade should be prepared in accordance with the recommendations provided in Section 5.2 of this report. To provide uniform support and mitigate differential settlement between varying existing subsurface conditions (e.g., former basements vs. slab-on-grade areas), we recommend constructing the proposed structures on a uniform mat of engineered fill.

We understand finished grades across the site will be within about 1 foot (above or below) existing grades. The required frost embedment depth is 24 inches.

**4.3.1 Spread Footings on New Engineered Fill**

The foundation subgrade should be prepared in accordance with the recommendations provided in Section 5.2 of this report. The footings should be overexcavated to a depth of 24 inches below the foundation bearing elevation and replaced with engineered fill. Footings bearing entirely on at least 24 inches of compacted engineered fill may be designed for an allowable bearing resistance of 2,500 pounds per square foot (psf), provided the following conditions are met:

- Overexcavation and Replacement: A minimum of two foot of native materials below all future footing elements should be overexcavated and replaced with properly compacted Structural Fill.
  - a. Depth: Overexcavation must extend two feet below the bottom elevation of footing elements—not two feet below finished floor, existing grade, bottom of slab, or other reference points.
  - b. Lateral Extents:
    - i. For continuous spread footings, overexcavation should extend at least 24 inches beyond the footing edges on either side.
    - ii. For isolated pad footings, overexcavation should extend a minimum of 24 inches beyond the footing perimeter on all sides.
  - c. Subgrade Preparation: The subgrade (i.e., the bottom of the excavation prior to fill placement) should be prepared in accordance with Section 5.2 and observed to be in a dense and unyielding condition prior to placement of engineered fill.



- Continuous spread footings should be at least one (1) foot wide, and no more than four (4) feet wide. Isolated pad footings should be at least two (2) feet wide, and no more than eight (8) feet wide.
- The foundation system should be reinforced to span an unsupported length of 10 feet.
- The allowable bearing and lateral resistance may be increased by 1/3 for temporary (seismic and wind) loading conditions.
- Footing designed following these recommendations is expected to settle less than one inch. The structure should be designed to accommodate up to 1-inch differential settlement over 50 feet.

The footers' allowable bearing resistance is limited by one inch of settlement associated with the underlying lean clay. Therefore the allowable bearing resistance can be increased by 500 psf. for every foot of additional overexcavation made beyond the minimum recommended two feet. The extent of the overexcavation should reflect the total depth of over excavation made beneath the footers (i.e. four feet of overexcavation requires four feet of lateral overexcavation beyond the footers, etc.).

#### 4.3.2 Sliding Resistance

For footings and slabs on grade founded on properly prepared native soils, a coefficient of friction of 0.40 may be used for structural fill. These values can be increased by 1/3 for temporary loading conditions. Additional lateral support can be accommodated by utilizing the passive lateral earth pressures provided in Section 4.2. Note that the local frost depth should be considered when utilizing lateral earth pressures.

#### 4.4 Interior Slab on Grade

Slab-on-grade areas should be underlain by a minimum of 4 inches of compacted gravel as described in Section 4.4.1, underlain by 20 inches of engineered fill, as described in Section 5.3, placed over a subgrade prepared in accordance with Section 5.2 and observed to be firm and unyielding prior to placement of engineered fill. The intent with the fill beneath the slab is for the subgrade of the footers (24-inches below base of concrete) to be maintained level across the footprint of the building and to be built up uniformly with fill in accordance with Section 5. Therefore, the fill beneath the slab on grade may vary slightly and may fluctuate up to six (6) inches from the specified minimum.

Slab-on-grade constructed over subgrades prepared in accordance with the recommendations in this section may be designed using a modulus of subgrade reaction (k-value) of 50pounds per square inch per inch (psi/in). The provided k value is intended for use in slab-on-grade analyses and represents near-surface, small-strain support conditions. The recommended k value is applicable for slab-on-grade loading conditions with contact stresses generally on the order of 100 to 500 psf. For higher sustained loading conditions, the Geotechnical Engineer should be consulted to evaluate the applicability of the recommended k value. The k value is not intended for use in the evaluation of foundation bearing capacity or settlement, including mat or raft foundations.

Slabs should be designed and constructed in accordance with ACI 302.1R-15 Guide to Concrete Floor and Slab Construction (ACI, 2015). Independent slabs on grade should be isolated from all bearing walls and columns using isolation joints that allow for unrestrained vertical movement. Slab reinforcement should be designed by the project structural engineer and, at a minimum, should include welded wire fabric (WWF) or rebar to help limit cracking. Control joints should also be provided throughout the slab to manage cracking. Where plumbing lines enter through the slab, positive breaks should be installed, and connections to mechanical equipment should incorporate flexible couplings to accommodate movement.

#### 4.4.1 Vapor Reduction

Water from beneath the slab will migrate through the slab into the building over time. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. We recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

- Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders should conform to class A vapor retarder per ASTM E 1745-11 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”.
- Place 4-inches of clean, crushed rock below the vapor retarder membrane. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.
- Concrete should have a concrete water-cement ratio of no more than 0.50.
- Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.
- Moist cure slabs for a minimum of 3 days or use the equivalent curing specified by the Structural Engineer.

#### 4.5 Drainage and Landscaping

Proper site drainage is critical given the dry climate and the presence of collapsible and erodible soils, which are highly sensitive to moisture infiltration. Excess moisture in foundation or slab subgrade soils can lead to volume loss and differential settlement. Therefore, stormwater runoff and irrigation must be carefully managed to support the long-term performance of foundations and other site improvements.

The following is recommended to be incorporated into the project’s grading and drainage design:

- Water should not be allowed to pond near foundations, slabs-on-grade, pavements, or other improvements. Finished soil grades should slope away from structures at a minimum of five percent for the first 10 feet, consistent with 2021 IBC Section 1804.4. Flatwork can be designed to flow at two percent away from the structure.
- Roof downspouts and drainage from impervious surfaces should be routed away from foundation walls. Where possible, roof drains should be directly connected to the site’s stormwater management system. If direct connection is not feasible, discharge should occur a minimum of 10 feet from any foundation element and be directed into properly graded swales or collection areas.
- Landscaping immediately adjacent to buildings should be limited to drought-tolerant vegetation or dry-scape features that do not require frequent irrigation.
- Stormwater should be prevented from sheet flowing off roadways and eroding or saturating supporting subgrades. Instead, runoff should be captured and conveyed using a curb and gutter system integrated into the broader stormwater management plan.

Final grading and drainage design should be performed by a licensed Civil Engineer and should incorporate these geotechnical considerations to minimize the potential for moisture-related soil movement and associated structural distress.

## 4.6 Pavement

Pavement improvements on the project should be made in conformance to the following recommendations.

### Exhibit 4-3 Recommended Pavement Sections

Application	Surface Course	Base Course
Heavy-Duty Rigid Pavement (trash truck access and dumpster pads)	6-inch Portland Cement Concrete (Reinforced per ACI 330R)	8-inch Aggregate Base Course
Heavy-Duty Flexible Pavement	4-inch Hot Mix Asphalt	10-inch Aggregate Base Course
Light-Duty Flexible Pavement	3-inch Hot Mix Asphalt	6-inch Aggregate Base Course
Light-Duty Rigid Pavement	5-inch Portland Cement Concrete	4-inch Aggregate Base Course

Subgrade should be scarified to a depth of at least 12 inches, moisture conditioned, and compacted prior to base placement. Aggregate base should conform to UDOT Standard Specification Section 02721 (Untreated Base Course), or engineer-approved equivalent. All base and subgrade materials should be compacted to a minimum of 95% relative compaction per ASTM D1557, Modified. Concrete should achieve a minimum compressive strength of 4,000 psi at 28 days. Hot Mix Asphalt (HMA) should be designed and placed in accordance with FP-14 Section 401 or AASHTO M 320 (Superpave), and must meet applicable standards for asphalt content, gradation, and compaction.

Pavement sections designed for post-construction traffic may not withstand heavy construction vehicles. Consider temporary construction access routes or thicker interim sections if early use is expected. Regular maintenance such as crack sealing, seal coating (for asphalt), and joint resealing (for concrete) is recommended to extend pavement life.

## 4.7 Concrete Flatwork

Concrete flatwork, including exterior walkways, driveways, patios, and steps, should be designed for anticipated use and loading conditions, and constructed in accordance with recognized standards such as ACI 330R Guide for the Design and Construction of Concrete Parking Lots (ACI, 2008) and applicable municipal specifications.

The subgrade should be prepared as described in Section 5.2 and observed to be firm and unyielding prior to fill placement.

Slab reinforcement should be designed by the project structural engineer and should include, at a minimum, welded wire fabric (WWF) or rebar to help control cracking. Control and isolation joints should be incorporated in accordance with ACI 330R, with final layout based on slab geometry, thickness, and anticipated loading.

Positive drainage away from all flatwork should be maintained to prevent ponding and moisture-related subgrade degradation. Additional drainage guidance is provided in Section 4.5.

## 5 CONSTRUCTION CONSIDERATIONS

The success of the project and the considerations provided in this report have been prepared assuming the following construction considerations are incorporated into the development of the Project. Where the considerations cannot be followed, or alternative construction methods are to be employed to construct the Project, MGG should be notified.

### 5.1 Site Preparation

Prior to site development, undocumented fill, weeds and organic material should be cleaned to a firm, undisturbed soil surface determined by a Geotechnical Engineer. Excavations will be required to support removal of existing basement and associated foundations. All fill material associated with the demolition should be screened and removed from native soils.

### 5.2 Subgrade Preparation

Subgrade, defined as the lowest depth to which soil is excavated prior to fill or foundation placement, is expected to consist primarily of lean clay for the at grade structures and gravel for the basement portions of the existing buildings.

Following excavation, subgrade preparation should consist of scarifying the upper 12 inches, moisture conditioning, and compacting the material in place. Scarification can generally be performed using an excavator bucket with teeth. Subgrade soils should be compacted to at least 90 percent of the modified Proctor maximum dry density (ASTM D1557) and within one percentage point of the optimum moisture content.

A Geotechnical Engineer should observe and test all subgrade areas prior to fill placement to verify that the subgrade is firm, unyielding, and consistent with the conditions encountered during the subsurface investigation.

Where soft or loose zones are identified, additional excavation or installation of a geotextile reinforcing material may be required. Overexcavation should extend at least one foot laterally from any areas identified as being soft or loose. If the overexcavation required becomes too large, the geotextile reinforcing material can help limit the required overexcavation volume. A Mirafi HP270 or an MGG approved equivalent is recommended to reinforce soft or loose material.

### 5.3 Fill Placement

Fill placement is anticipated to be required to achieve the proposed site grades and to balance earthwork for the project. Onsite soils are considered suitable for reuse, provided they meet the criteria outlined in the following subsections. All fill materials intended to support structures, pavements, or flatwork should be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D1557) and within one percentage point of the optimum moisture content. This requirement may be reduced to 90 percent for fill placed outside of structural, pavement and flatwork support zones. Proctor test results for the native lean clay are provided in Appendix B.

The height of the loose lifts to be used as the fill material is placed is dependent on the machinery used to compact the soils. Where hand-held jumping jacks or walk-behind compactors are used, we recommend that loose lifts not exceed 8 inches. For excavators outfitted with a wheel compactor, 10-inch loose lifts should be used. For large sheep foot or drum rollers, 12-inch loose lifts can be used. Larger lift thicknesses can be used provided that the contractor can demonstrate adequate compaction through the compacted lift. Fill material should be free of frozen material or free water. Where free water is present, we recommend the material be blended with dry material to reach the required moisture content. Frozen soils should be set aside and not utilized until they are unfrozen.

### 5.3.1 Structural Fill

Structural Fill should consist of a 3-inch minus material with no more than 15 percent fines. The Atterberg Limit (ASTM 4318) of the material should result in a Plasticity Index less than 8. The material should not be open graded or well-rounded and free of organic and deleterious materials. It is recommended that any imported fill material be approved by a geotechnical engineer familiar with the project prior to use.

### 5.3.2 Recycled Onsite Materials

Onsite soils are suitable for reuse as engineered fill, provided they are screened to remove oversized and deleterious materials. Materials to be excluded include particles greater than 3 inches in nominal diameter (unless otherwise noted below), organics exceeding 1 inch in diameter or 5 percent by volume, concrete or asphaltic concrete debris, and other deleterious materials. Particles between 3 and 10 inches in nominal diameter may be reused, provided they are well distributed, not nested together, and compatible with compaction and lift thickness requirements. Where clay-dominated materials are sourced onsite for fill, the material should be blended with onsite sand, gravel, and non-plastic fine-grained material.

## 5.4 Utilities

Utilities should be installed in accordance with the requirements of the local authority having jurisdiction and applicable industry standards. In the absence of municipal guidance, trench excavation, bedding, and backfill should follow the Utah APWA Standards. Trench backfill should be placed in lifts and compacted in accordance with Section 5.3.

## 5.5 Excavations

Trench and excavation safety is the requirement of the contractor installing the utilities and performing excavations. We recommend the contractor be familiar with the necessary OSHA 1926 standards and have a person onsite who can perform the necessary duties of the Competent Person as defined by OSHA. Soil conditions can vary from those explored as part of this investigation and therefore soil types, in accordance with OSHA, should be assessed by the onsite Competent Person.

## 6 ADDITIONAL SERVICES

Moab Geotechnical Group (MGG) strongly encourages our continued involvement with the project leading up to and during construction. The additional services we can provide will help ensure the recommendations and construction considerations are followed and that adverse conditions encountered during construction are properly addressed.

### 6.1 Project Progression

As the design of the project advances, we recommend we be informed of changes that differ from those assumed or detailed in this report. Successful performance of the future structures and infrastructure are dependent on the findings used to prepare this report. This report can be revised from its current state with subsequent changes and evolution of the project.

### 6.2 Plan and Specification Review

It is recommended that MGG review the final plans and specifications for the project. The adherence to the recommendations and construction considerations will help the long-term performance of the proposed structures and associated improvements. The review can also check if any changes in the nature, design or location of the proposed improvements and provide opportunity to prepare a written response with updated recommendations, if needed.

### 6.3 Testing and Observation Services

Testing and observation services can be performed by moisture / density testing and by onsite observation and testing. These services should be considered in the future construction costs of the project. All earthwork operations should be performed under the fulltime to temporary observation of our representative to check that the site properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills have been performed in accordance with our recommendations and project specifications. Site observations also aid in addressing potentially problematic soils that were not found during the site characterization and field investigation phase.

## 7 LIMITATIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary states can be taken by the contractors and subcontractors to carry out such recommendations in the field.

Moab Geotechnical Group (MGG) strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in land development. MGG is unable to eliminate all risks or provide insurance; therefore, MGG is unable to guarantee or warrant the results of the work.

This report is based upon field and other conditions discovered at the time of preparation of our work. This document must not be subject to unauthorized reuse, that is, reuse without our written authorization. Such authorization is essential because it requires us to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications, to other changes to our work. Therefore, we must be engaged to prepare the necessary clarifications, adjustment modifications, or other changes before construction activities commence or further activity proceeds.

## REFERENCES

- American Concrete Institute [ACI], 2019, *Building Code Requirements for Structural Concrete*, ACI 318
- American Concrete Institute [ACI], 2016, *Guide to Durable Concrete*, ACI 201.2R-16
- American Concrete Institute [ACI], 2015, *Guide to Concrete Floor and Slab Construction*, ACI 302.1R-15
- American Concrete Institute [ACI], 2008, *Guide for the Design and Construction of Concrete Parking Lots*, ACI 330R-08.
- American National Standard [ANSI]/ AARST Consortium on National Radon Standards, 2023, *Soil Gas Control Systems in New Construction of Multifamily, School, Commercial and Mixed-Use Buildings*
- American National Standard [ANSI]/ AARST Consortium on National Radon Standards, 2023, *Soil Gas Mitigation Standards for Existing Multifamily, School, Commercial and Mixed-Use Buildings*
- American Society of Civil Engineers [ASCE], 2025, *ASCE 7 Hazard Tool*, [ASCE 7 Hazard Tool](#), accessed on November 20, 2025.
- American Society of Civil Engineers [ASCE], 2017, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16)*
- ASTM International, 2023, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications (ASTM D6951/D6951M-18, reapproved 2023)*: ASTM International, West Conshohocken, PA.
- International Code Council, 2021, *International Building Code*. Falls Church, Va. :International Code Council
- Lockwood, D., et al, 1992, *Analysis and Classification of DCP Survey Data, Technology & Information Management Programme*, CSIR Transporteck, Pretoria, South Africa
- Mononobe, N. and H. Matsuo, 1929, *On the Determination of earth Pressures During Earthquakes*, Proceedings of the World Engineering Congress
- Nationwide Environmental Title Research, LLC (NETR), 2025, *Historic aerial imagery viewer*: HistoricAerials.com, accessed November 18, 2025, at <https://www.historicaerials.com>.
- Okabe, S., 1926, *General Theory of Earth Pressure*, Journal of Japan Society of Civil Engineering Vol. 12, No. 1
- Packard, R.G., 1973, *Design of Concrete Airport Pavement*: Portland Cement Association Engineering Bulletin, Portland Cement Association, Skokie, IL, reprinted 1995.
- Roberge, Pierre R., 2000, *Handbook of Corrosion Engineering*, McGraw-Hill
- Skempton, A. W. (1953). "The Colloidal Activity of Clays." *Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 57–61.
- United States Geological Survey [USGS], 2024, *1/3 arc-second Digital Elevation Model (DEM), N40W111*: The National Map, U.S. Geological Survey, published October 31, 2024, accessed October 14, 2025, from <https://www.nationalmap.gov>.
- United States Geological Survey [USGS], 2024, *U.S. Landslide Inventory and Susceptibility Map*, [U.S. Landslide Inventory and Susceptibility Map | U.S. Geological Survey](#), accessed on November 19, 2025.


United States Geological Survey [USGS], 2024, *Quaternary Fault and Fold Database of the United States*, [Faults | U.S. Geological Survey](#), accessed on November 20, 2025.

Utah Geological Survey. (n.d.). *Hazard mapping interactive viewer: Earthquake ground shaking – statewide* [Web map]. <https://maps.geology.utah.gov/hazards>, accessed on November 20, 2025

Witkind, I.J., 1979, *Reconnaissance geologic map of the Wellington quadrangle, Carbon County, Utah*, U.S. Geological Survey, Miscellaneous Investigations Series Map I-1178, 1:24,000.



**LEGEND**


 Project Boundary



0 1,000 2,000 ft



Map Source: Google Maps

	<p>WELLINGTON CITY BUILDING 150 W MAIN STREET WELLINGTON, UTAH</p>	<p>VICINITY MAP</p>	<p>Project No: 2025-098</p>	
			<p>May 2026</p>	<p>FIG. 1</p>



**LEGEND**

Project Boundary
 
● Borings
 

 DCP

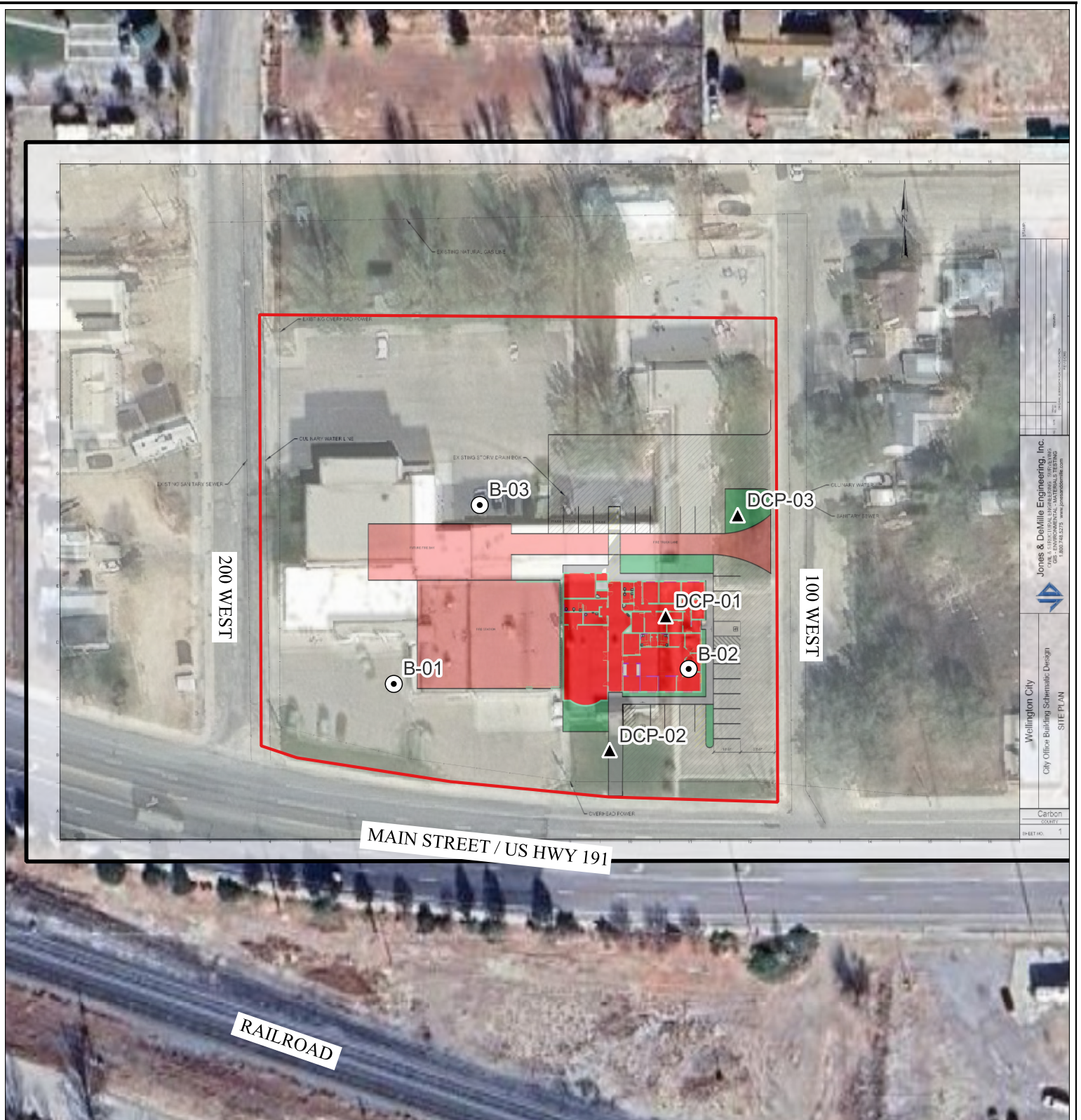


0 50 100 ft



Map Source: Google Satellite

 <p><b>Moab Geotechnical Group</b></p>	<p>WELLINGTON CITY BUILDING 150 W MAIN STREET WELLINGTON, UTAH</p>	<p><b>EXPLORATIONS MAP</b></p>	<p>Project No: 2025-098</p>	
			<p>May 2026</p>	<p><b>FIG. 2a</b></p>



Jones & DeMille Engineering, Inc.  
 CIVIL & STRUCTURAL ENGINEERING • WELLINGTON, UTAH  
 400 S. MAIN STREET, SUITE 200 • WELLINGTON, UTAH 84401  
 TEL: 435.734.2275 • WWW.JONESANDEMILLE.COM

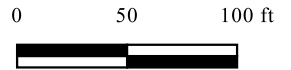
Wellington City  
 City Office Building Schematic Design  
 SITE PLAN

Carbon  
 100000000

SHEET NO. 1

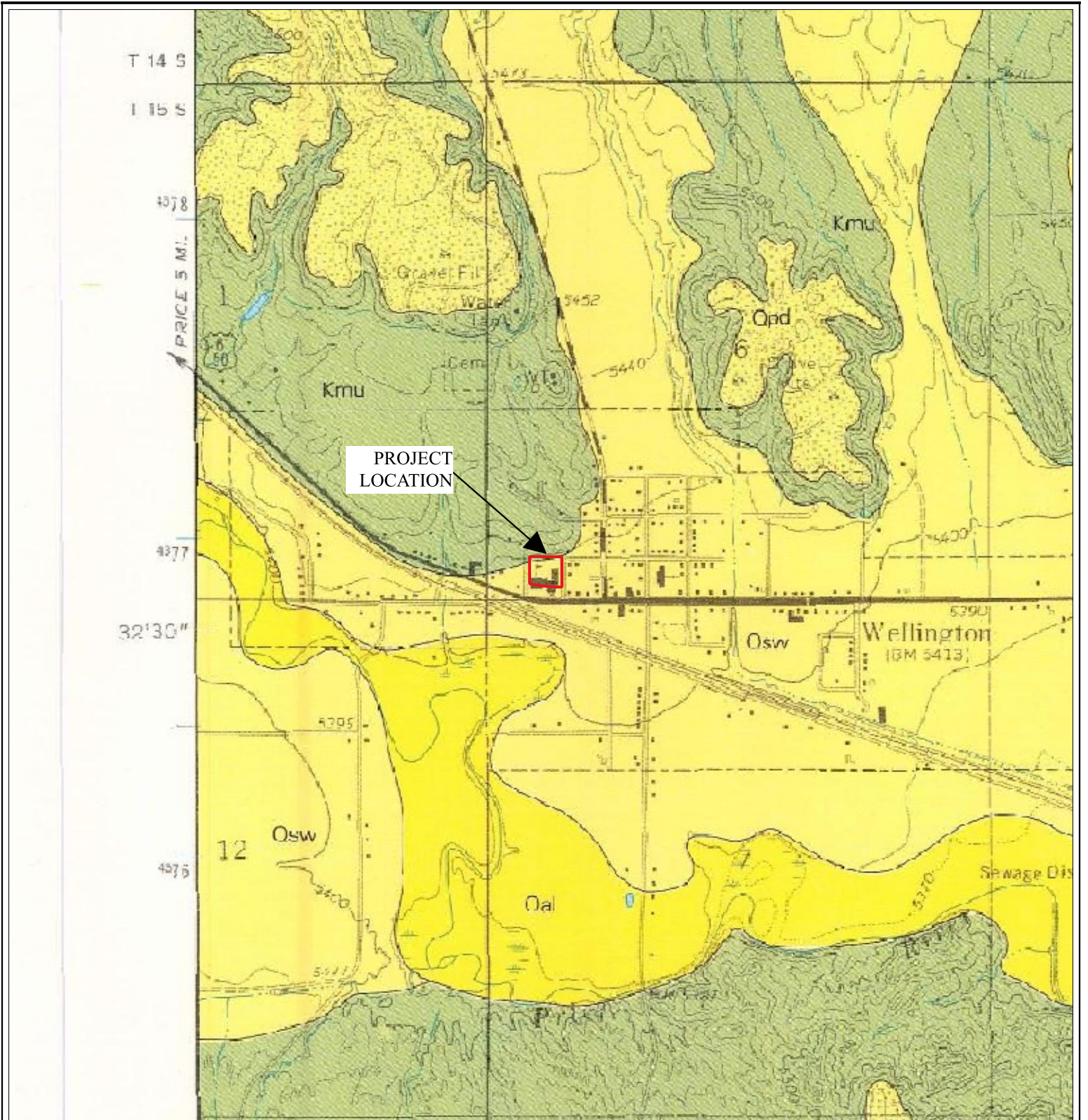
**LEGEND**

- Project Boundary
- Borings
- DCP




Map Source: (1) Jones & DeMille, 2025, (2) Google Satellite

	<b>WELLINGTON CITY BUILDING</b> 150 W MAIN STREET WELLINGTON, UTAH	<b>EXPLORATIONS MAP WITH          SCHEMATIC DESIGN          OVERLAY</b>	Project No: 2025-098	
			May 2026	<b>FIG. 2b</b>



**LEGEND**

 Project Boundary

**SELECT ABBREVIATED DESCRIPTIONS**

Qsw: Slope Wash (Holocene and Pleistocene)

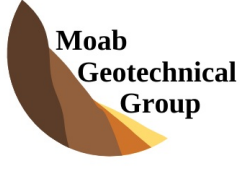
Kmu: Mancos Shale, unnamed upper unit (Upper Cretaceous)



0 1,000 2,000 ft



Map Source: Witkind, 1979

	WELLINGTON CITY BUILDING 150 W MAIN STREET WELLINGTON, UTAH	<b>GEOLOGIC MAP</b>		Project No: 2025-098	
				May 2026	<b>FIG. 3</b>

# APPENDIX A – BORING LOGS

RELATIVE CONSISTENCY - COHESIVE SOIL			
DESCRIPTION	SPT-N VALUE (# BLOWS/FT)	TORVANE, tsf SHEAR STRAIN	POCKET PENETROMETER, tsf UNCONFINED SHEAR STRAIN
Very Soft (VS)	< 2	< 0.13	<0.25
Soft (S)	2 to 4	0.13 to 0.25	0.25 to 0.5
Medium Stiff (MSt)	4 to 8	0.25 to 0.5	0.5 to 1
Stiff (St)	8 to 15	0.5 to 1	1 to 2
Very Stiff (VSt)	15 to 30	1 to 2	2 to 4
Hard (H)	Over 30	>2	>4

CEMENTATION	
DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

MOISTURE CONTENT	
CONSISTENCY	FIELD TEST
Dry (D)	Absence of moisture, dusty, dry to the touch
Moist (M)	Damp but no visible water
Wet (W)	Visible free water, usually soil is below water table

RELATIVE DENSITY - COHESIONLESS SOIL	
DESCRIPTION	SPT-N VALUE (# BLOWS/FT)
Very Loose (VL)	< 4
Loose (L)	4 to 10
Medium Dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

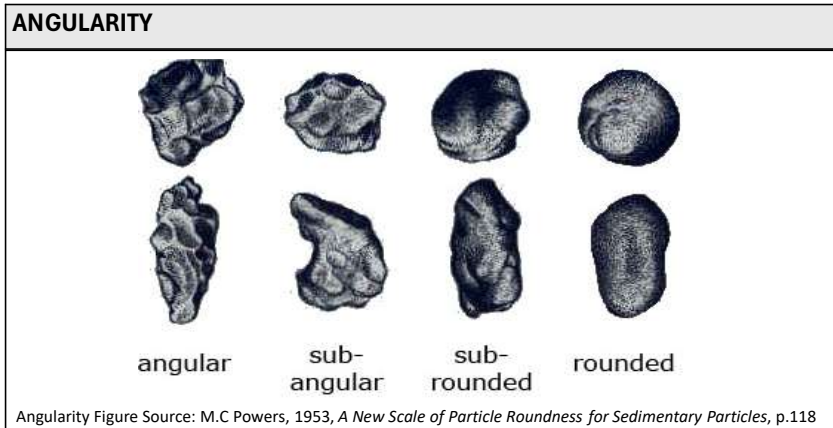
REACTION WITH HCL	
DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Strong reaction, with bubbles forming immediately

STRUCTURE	
DESCRIPTION	CRITERIA
Interbedded (int)	Alternating layers of varying material or color with layers at least 1/4-inch thick; note thickness; singular: Bed
Laminated (lam)	Alternating layers of varying material or color with layers less than 1/4-inch thick; note thickness; singular: Lamination
Fissured (fis)	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided (stk)	Fracture planes appear polished or glossy, sometimes striated
Blocky (blk)	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed (lnsd)	Inclusion of small packets of different soils such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous (hom)	Same color and appearance throughout

GRAIN SIZE		
DESCRIPTION	GRAIN SIZE / SIEVE NO.	
Boulders (Bo)	>12"	
Cobbles (Co)	3 to 12"	
Gravel (Gr)	coarse (c)	3/4 to 3"
	fine (f)	No. 4 to 3/4"
Sand (Sa)	coarse (c)	No. 10 to No. 4
	medium (m)	No. 40 to No.10
	fine (f)	No. 200 to No. 40
Fines (Fi)	< No. 200	

PERCENTAGE TERMS	
ADJECTIVE	PERCENT RANGE
Trace	<5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

Gravel, sand and silt estimated by mass.  
Organic cobbles and boulders estimated by volume.



WELLINGTON CITY BUILDING  
150 W MAIN STREET  
WELLINGTON, UTAH

## SOIL REFERENCE STANDARD

Project No.: 2025-098

May 2026

FIG. A-01

FABRIC TERMS
SEDIMENTARY ROCKS
<b>MASSIVE</b> - Rock without significant structure
<b>BEDDED</b> - Regular layering from sedimentation
<b>FISSILE</b> - Tendency to break along laminations
METAMORPHIC ROCKS
<b>FOLIATED</b> - Parallel arrangement or distribution of minerals
<b>SCHISTOSE</b> - Parallel arrangement of tabular minerals giving a planar fissility
<b>GNEISSOSE</b> - Segregation of minerals into bands
<b>CLEAVAGE</b> - Tendency to split along secondary, planar, textures or structures

DISCONTINUITY TERMS
<b>FRACTURE</b> - collective term for any natural break excluding shears, shear zones, and faults
<b>JOINT (JT)</b> - Planar break with little or no displacement
<b>FOLIATION JOINT (FJ) or BEDDING JOINT (BJ)</b> - Joint along foliation or bedding
<b>INCIPIENT JOINT (IJ) or INCIPIENT FRACTURE (IF)</b> - Joint or fracture not evident until wetted and dried; breaks along existing surface
<b>RANDOM FRACTURE (RF)</b> - Natural, very irregular fracture that does not belong to a set
<b>BEDDING PLANE SEPARATION or PARTING</b> - A separation along bedding after extraction from stress relief or slaking
<b>FRACTURE ZONE (FZ)</b> - Planar zone of broken rock without gouge
<b>MECHANICAL BREAK (MB)</b> - Breaks due to drilling or handling; drilling break (DB), hammer break (HB)
<b>SHEAR (SH)</b> - Surface of differential movement evident by presence of slickensides, striations, or polishing
<b>SHEAR ZONE (SZ)</b> - Zone of gouge and rock fragments bounded by planar shear surfaces
<b>FAULT (FT)</b> - Shear zone of significant extent; differentiation from shear zone may be site-specific

SPACING		
DESCRIPTION	SPACING	
Extremely close Spacing	<20 mm	<1 in
Very close Spacing	20 to 60 mm	1 to 1.25 in
Close Spacing	60 to 200 mm	2.5 to 8 in
Moderate Spacing	200 to 600 mm	8 to 24 in
Wide Spacing	600 to 2000 mm	24 in to 6 ft
Very wide Spacing	2000 to 6000 mm	6ft to 20 ft
Extremely wide Spacing	> 6000 mm	<20 ft

PERSISTENCE	
Very low persistence	<1 m
Low persistence	1-3 m
Medium persistence	3-10 m
High persistence	10-20 m
Very high persistence	>20 m

VESICULARITY	
Slightly Vesicular	1 to 10%
Moderately Vesicular	10 to 30%
Highly Vesicular	30 to 50%
Scoriaceous	>50%



WELLINGTON CITY BUILDING  
150 W MAIN STREET  
WELLINGTON, UTAH

**ROCK REFERENCE  
STANDARD**

Project No.: 2025-098

May 2026

**FIG. A-02**

WEATHERING		
GRADE	TERM	DESCRIPTION
I	Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
II	Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
III	Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
IV	Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
V	Completely Weathered	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.
VI	Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH				
GRADE	DESCRIPTION	FIELD IDENTIFICATION	APPROX. RANGE OF UNIAXIAL COMPRESSIVE STRENGTH	
			(MPa)	(psi)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025	<3.6
S2	Soft clay	Easily penetrated several inches by thumb	0.025 to 0.05	3.6 to 7.3
S3	Firm clay	Can be penetrated by thumb with moderate effort	0.05 to 0.10	7.3 to 14.5
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10 to 0.25	14.5 to 36
S5	Very stiff clay	Readily indented by thumbnail	0.25 to 0.5	36 to 73
S6	Hard clay	Indented with difficulty by thumbnail	>0.50	>73
R0	Extremely Weak rock	Indented by thumbnail	0.25 to 1.0	36 to 145
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0 to 5.0	145 to 700
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow of geological hammer	5.0 to 25	700 to 3,600
R3	Medium strong rock	Cannot be scraped or peeled or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25 to 50	3,600 to 7,200
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50 to 100	7,200 to 14,500
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100 to 250	14,500 to 36,250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	>250	>36250



WELLINGTON CITY BUILDING  
150 W MAIN STREET  
WELLINGTON, UTAH

**ROCK REFERENCE  
STANDARD**

Project No.: 2025-098

May 2026

**FIG. A-03**

Project: **Wellington City Building**  
 Project Location: **150 W Main Street, Wellington, Utah**  
 Project Number: **2025-098**

**Key to Log of Boring**  
**Sheet 1 of 1**

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/6-inches	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Gravel, %	Sand, %	Fines, %	LL, %	PI, %	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16






**COLUMN DESCRIPTIONS**

- 1** Elevation (feet): Elevation (MSL, feet)
- 2** Depth (feet): Depth in feet below the ground surface.
- 3** Sample Type: Type of soil sample collected at the depth interval shown.
- 4** Sample Number: Sample identification number.
- 5** Sampling Resistance, blows/6-inches: Number of blows to advance driven sampler 18 inches (in 6 inch intervals) using the hammer identified on the boring log.
- 6** Material Type: Type of material encountered.
- 7** Graphic Log: Graphic depiction of the subsurface material encountered.
- 8** MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
- 9** Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.
- 10** Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.
- 11** Gravel, %: The percent gravel (soil passing a 3-in. sieve and retained on a No. 4 sieve) in the sample.
- 12** Sand, %: The percent sand (soil passing a No. 4 sieve and retained on a No. 200 sieve) in the sample.
- 13** Fines, %: The percent fines (soil passing a No. 200 Sieve) in the sample.
- 14** LL, %: Liquid Limit, expressed as a water content.
- 15** PI, %: Plasticity Index, expressed as a water content.
- 16** REMARKS AND OTHER TESTS: Comments and observations regarding drilling or sampling made by driller or field personnel.




**FIELD AND LABORATORY TEST ABBREVIATIONS**

- CF: Clay Fraction, as determined by Hydrometer Analysis
- UU: Unconsolidated-Undrained Test, su: Undrained Shear Strength
- PP: Pocket Penetrometer reading
- LL: Liquid Limit, percent
- PI: Plasticity Index, percent






**MATERIAL GRAPHIC SYMBOLS**

-  Asphaltic Concrete (AC)
-  LEAN CLAY (CL), SANDY LEAN CLAY (CL), GRAVELLY LEAN CLAY (CL)
-  CLAYEY GRAVEL (GC), and CLAYEY GRAVEL (GC) with sand
-  POORLY-GRADED GRAVEL (GP)
-  Shale

**TYPICAL SAMPLER GRAPHIC SYMBOLS**

-  2.5-inch-OD Modified California (MC)
-  2-inch-OD unlined split spoon (SPT)
-  Shelby Tube (ST) thin-walled, fixed head

**OTHER GRAPHIC SYMBOLS**

-  Water level (at time of drilling, ATD)
-  Water level (after waiting, AW)
-  Minor change in material properties within a stratum
-  Inferred/gradational contact between strata
-  Queried contact between strata

**GENERAL NOTES**

- 1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
- 2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.
- 3: The latitude and longitude of the borings were estimated using handheld GPS equipment with a reported accuracy of approximately ±10 feet. Ground surface elevations were estimated from publicly available elevation data. These measurements are approximate and should not be considered survey-grade.

Project: **Wellington City Building**  
 Project Location: **150 W Main Street, Wellington, Utah**  
 Project Number: **2025-098**

**Log of Boring B-01**  
**Sheet 1 of 1**

Date(s) Drilled: <b>11/3/202</b>	Logged By: <b>CAV</b>	Checked By: <b>TQH</b>
Drilling Method: <b>HSA</b>	Drill Bit Size/Type: <b>6.5" O.D., 3.125" I.D.</b>	Total Depth of Borehole: <b>26.5 feet bgs</b>
Drill Rig Type: <b>Truck Mounted CME-75</b>	Drilling Contractor: <b>South Slope Drilling</b>	Approximate Surface Elevation: <b>5415 feet</b>
Groundwater Level and Date Measured: <b>Not Measured</b>	Sampling Method(s): <b>Modified California, SPT, Tube</b>	Hammer Data: <b>140lb Auto</b>
Borehole Backfill: <b>Cuttings and Asphalt Patch</b>	Location: <b>Lat: 39.5427397°, Long: -110.7381508°</b>	

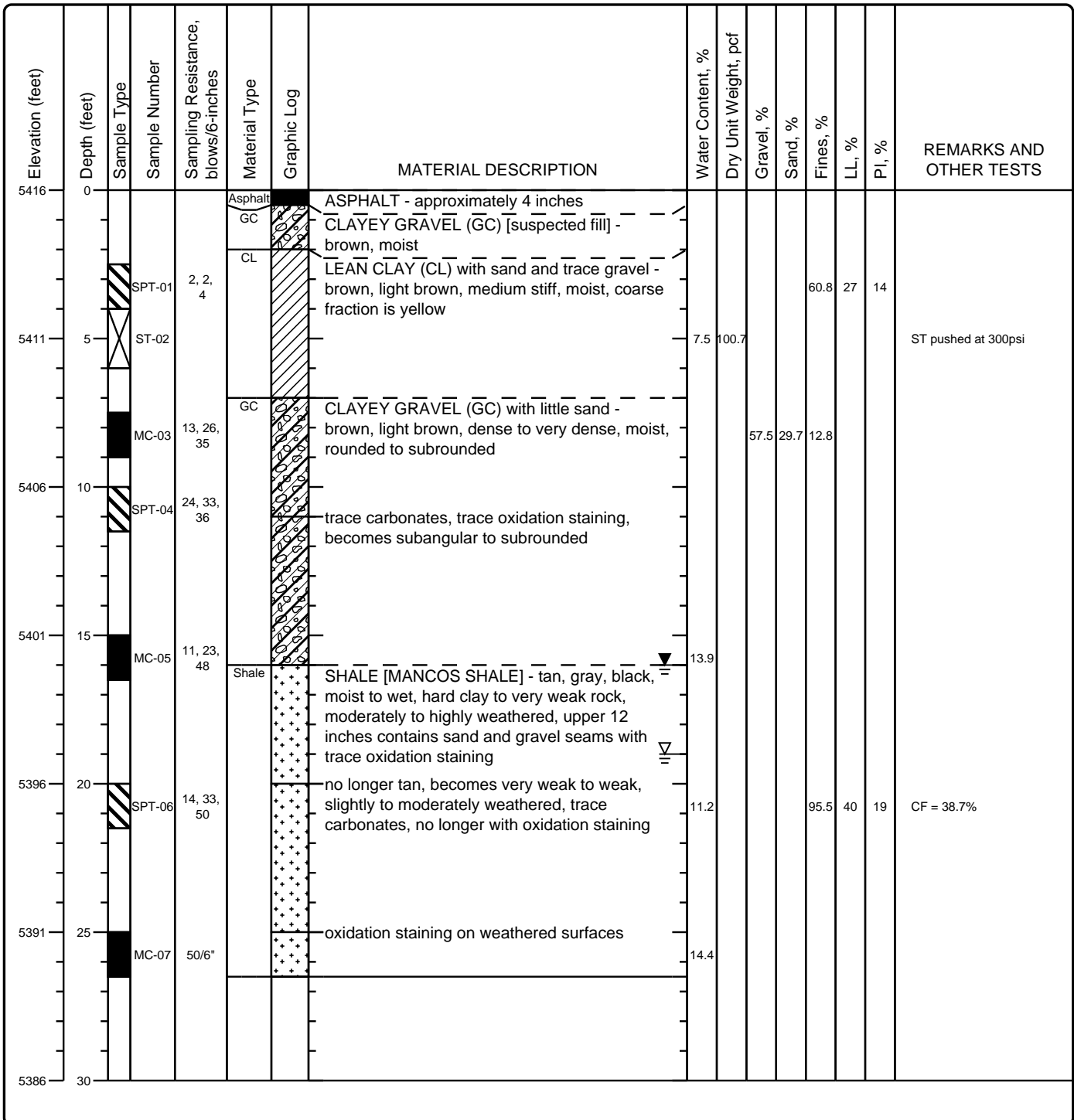
Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/6-inches	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Gravel, %	Sand, %	Fines, %	LL, %	PI, %	REMARKS AND OTHER TESTS
5415	0				Asphalt		ASPHALT - approximately 6 inches								
		CL			CL		LEAN CLAY (CL) [suspected fill] - dark brown, moist								
		CL			CL		LEAN CLAY (CL) - brown, light brown, medium stiff, moist					90.2	28	12	
5410	5	ST-02						14.9	96.9						ST pushed 20" at 300psi, 2" at 700psi
		GC			GC		CLAYEY GRAVEL (GC) with some sand and trace cobbles - tan, brown, very dense, moist			57.2	30.4	12.4			
5405	10	SPT-04					trace oxidation staining								
		Shale			Shale		SHALE [MANCOS SHALE] - tan, gray, black, hard clay to very weak rock, moderately to highly weathered, trace oxidation staining	11.4				93.1	33	13	CF = 29.7%
5395	20	SPT-06					no longer tan, becomes very weak to weak, slightly to moderately weathered, with thin carbonate veins, no longer with oxidation staining	10.1							
5390	25	MC-07					oxidation staining on weathered surfaces								
5385	30														

**Figure A-05**

Project: **Wellington City Building**  
 Project Location: **150 W Main Street, Wellington, Utah**  
 Project Number: **2025-098**

**Log of Boring B-02**  
**Sheet 1 of 1**

Date(s) Drilled: <b>11/3/202</b>	Logged By: <b>CAV</b>	Checked By: <b>TQH</b>
Drilling Method: <b>HSA</b>	Drill Bit Size/Type: <b>6.5" O.D., 3.125" I.D.</b>	Total Depth of Borehole: <b>26.5 feet bgs</b>
Drill Rig Type: <b>Truck Mounted CME-75</b>	Drilling Contractor: <b>South Slope Drilling</b>	Approximate Surface Elevation: <b>5416 feet</b>
Groundwater Level and Date Measured: <b>16 feet DD, 19 feet AD</b>	Sampling Method(s): <b>Modified California, SPT, Tube</b>	Hammer Data: <b>140lb Auto</b>
Borehole Backfill: <b>Cuttings and Asphalt Patch</b>	Location: <b>Lat: 39.54276661°, Long: -110.73747051°</b>	

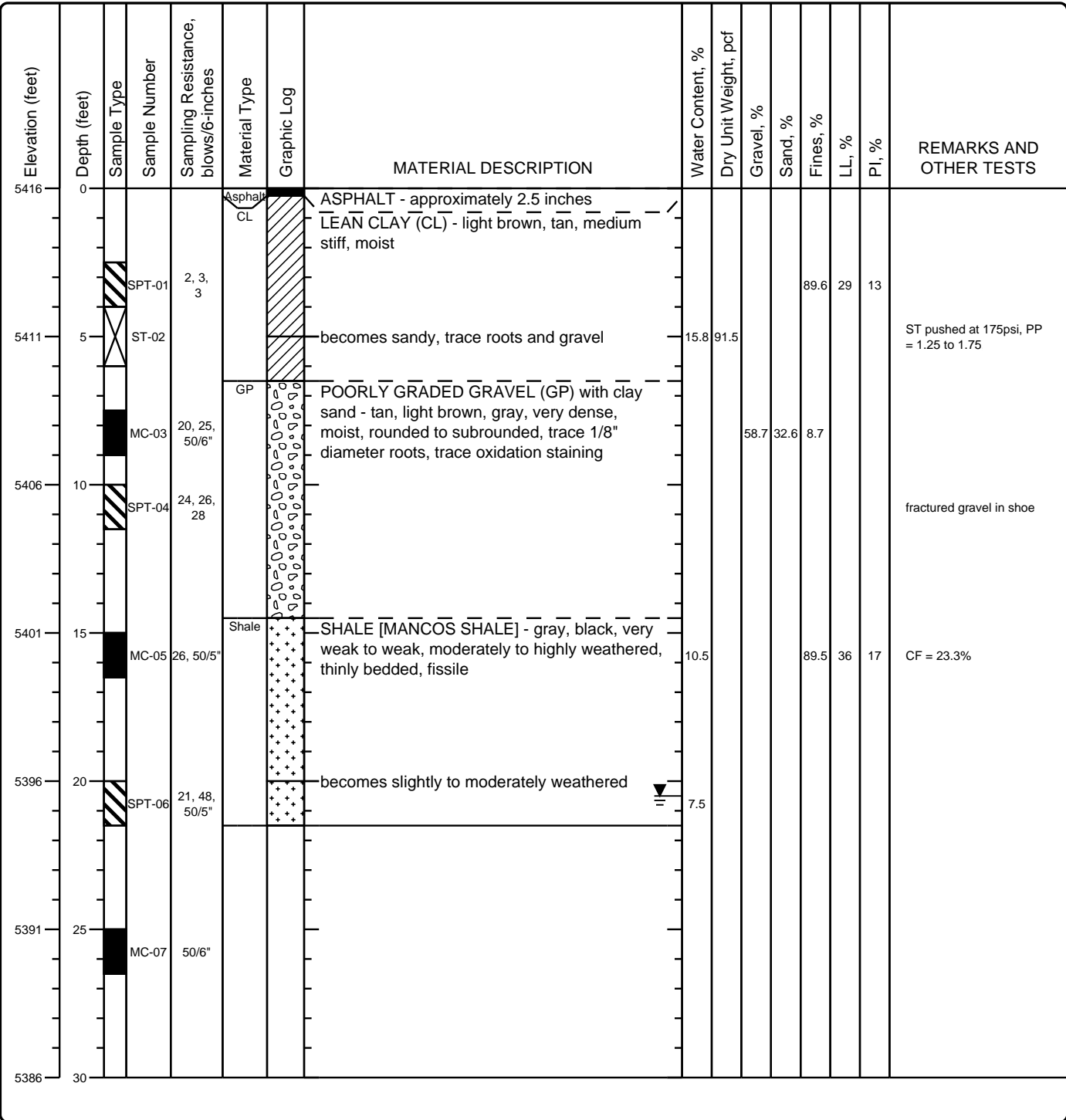


**Figure A-06**

Project: **Wellington City Building**  
 Project Location: **150 W Main Street, Wellington, Utah**  
 Project Number: **2025-098**

**Log of Boring B-03**  
**Sheet 1 of 1**

Date(s) Drilled: <b>11/3/202</b>	Logged By: <b>CAV</b>	Checked By: <b>TQH</b>
Drilling Method: <b>HSA</b>	Drill Bit Size/Type: <b>6.5" O.D., 3.125" I.D.</b>	Total Depth of Borehole: <b>21.5 feet bgs</b>
Drill Rig Type: <b>Truck Mounted CME-75</b>	Drilling Contractor: <b>South Slope Drilling</b>	Approximate Surface Elevation: <b>5416 feet</b>
Groundwater Level and Date Measured: <b>20.5 feet AD</b>	Sampling Method(s): <b>Modified California, SPT, Tube</b>	Hammer Data: <b>140lb Auto</b>
Borehole Backfill: <b>Cuttings and Asphalt Patch</b>	Location: <b>Lat: 39.54305893°, Long: -110.73795111°</b>	



**Figure A-07**

# APPENDIX B – LABORATORY TEST RESULTS

Job Name: Wellington City Building  
 Job Number: 2025-098  
 Location: Wellington, UT

Exploration:	Sample ID:	Depth (ft):	Description:	Moisture Content (%):	Dry Unit Weight (pcf):	Gravel (%):	Sand (%):	Fines (%):	Silt (%):	Clay (%):	LL	PI	USCS	Group Name
B-01	BULK-01	4-7	Brown silty clay w/ gravel			6	18	76					CL/ML	Silt or Clay w/ Sand
B-01	SPT-01	2.5-4	Brown silty clay					90.2			28	12	CL	Lean Clay
B-01	ST-02	5-7	Brown silty sand		96.9									
B-01	MC-03	7.5-9	Brown sandy gravel w/ clay			57	30	12					GC/GM	Clayey or Silty Gravel
B-01	MC-05	15-16.5	Grayish brown silty clay	11.4				93.12	60.7	32.4	33	13	CL	Lean Clay
B-01	SPT-06	20-21.5	Gray silty clay	10.1										
B-02	SPT-01	2.5-4	Brown sandy clay w/ gravel					60.8			27	14	CL	Lean Clay
B-02	ST-02	4-6	Brown clayey silt		100.7									
B-02	MC-03	7.5-9	Brown silt w/ gravel			58	30	13					GC/GM	Clayey or Silty Gravel
B-02	MC-05	15-16.5	Grayish brown silty sand with gravel	13.9										
B-02	SPT-06	20-21.5	Gray silty clay	11.2				95.49	55.4	40.1	40	19	CL	Lean Clay
B-02	MC-07	25-26.5	Gray silty clay	14.4										
B-03	SPT-01	2.5-4	Brown silty clay					89.6			29	13	CL	Lean Clay
B-03	ST-02	4-6	Brown clayey silt		91.5									
B-03	MC-03	7.5-9	Brown silty sand w/ gravel			59	33	9					GP-GC/GM	Poorly Graded Gravel with Silt or Clay w/ Sand
B-03	MC-05	15-16.5	Gray silty clay	10.5				89.48	59	30.5	36	17	CL	Lean Clay
B-03	SPT-06	20-21.5	Gray silty clay	7.5										

Project: Wellington City Building      **Water Content of Soil and Rock by Mass**  
 Project No: 2025-098      ASTM D2216  
 Location: Wellington, UT

Test Number	Exploration	Sample ID	Depth	Sample Date	Sample Description	Test Date	Tested By	Water Content
1	B-01	MC-05	15-16.5	11/3/2025	Grayish brown silty clay	11/12/2025	KC	11.4
2	B-01	SPT-06	20-21.5	11/3/2025	Gray silty clay	11/12/2025	DP/KC	10.1
3	B-02	MC-05	15-16.5	11/3/2025	Grayish brown silty sand with gravel	11/12/2025	KC	13.9
4	B-02	SPT-06	20-21.5	11/3/2025	Gray silty clay	11/12/2025	KC	11.2
5	B-02	MC-07	25-26.5	11/3/2025	Gray silty clay	11/12/2025	KC	14.4
6	B-03	MC-05	15-16.5	11/3/2025	Gray silty clay	11/12/2025	KC	10.5
7	B-03	SPT-06	20-21.5	11/3/2025	Gray silty clay	11/12/2025	KC	7.5

Wellington City Building      Density and Unit Weight of Soil  
 2025-098                              ASTM D7263

Wellington, UT

Test Number	Exploration	Sample ID	Depth	Description	Test Method	Specimen Type	Specimen Mass (g)	Specimen Volume (cm <sup>3</sup> )	Wet Density (g/cm <sup>3</sup> )	Wet Unit Weight (lb/ft <sup>3</sup> )	Dry Density (g/cm <sup>3</sup> )	Dry Unit Weight (lb/ft <sup>3</sup> )	Water Content
1	B-01	ST-02	5-7	Brown silty	B	Intact	4072.3	2283.4	1.8	111.3	1.6	96.9	14.9
2	B-02	ST-02	4-6	Brown clayey silt	B	Intact	4837.9	2788.1	1.7	108.3	1.6	100.7	7.5
3	B-03	ST-02	4-6	Brown clayey silt	B	Intact	4456.1	2627.2	1.7	105.9	1.5	91.5	15.8

Reviewed By:     TQH

Project: Wellington City Building **Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis**  
 Project No: 2025-098 ASTM C136/D6913  
 Location: Wellington, UT

Test Number:	Sample Source:	Sample ID:	Depth:	Ave Depth:	Descripton:	Gravel (%):	Sand (%):	Fines (%):	Cu	Cc	Poor/Well Graded	Test Notes:	Test Method:
1	B-01	BULK-01	4-7	5.5	Brown silty clay w/ gravel	6.0	17.6	76.4	6.0	1.5	Fines with Sand		A
2	B-01	MC-03	7.5-9	8.25	Brown sandy gravel w/ clay	57.2	30.4	12.4	224.0	2.9	Gravel with Fines		A
3	B-02	MC-03	7.5-9	8.25	Brown silt w/ gravel	57.5	29.7	12.8	209.8	2.7	Gravel with Fines		A
4	B-03	MC-03	7.5-9	8.25	Brown silty sand w/ gravel	58.7	32.6	8.7	194.1	0.8	Poorly Graded Gravel		A

**Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis**

ASTM D6913



Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-01
Sample ID:	BULK-01
Depth:	4-7
Sample Date:	11/3/2025
Sample Description:	Brown silty clay w/ gravel
Prep Method:	Moist
Test Method:	A
Number of Separations:	1

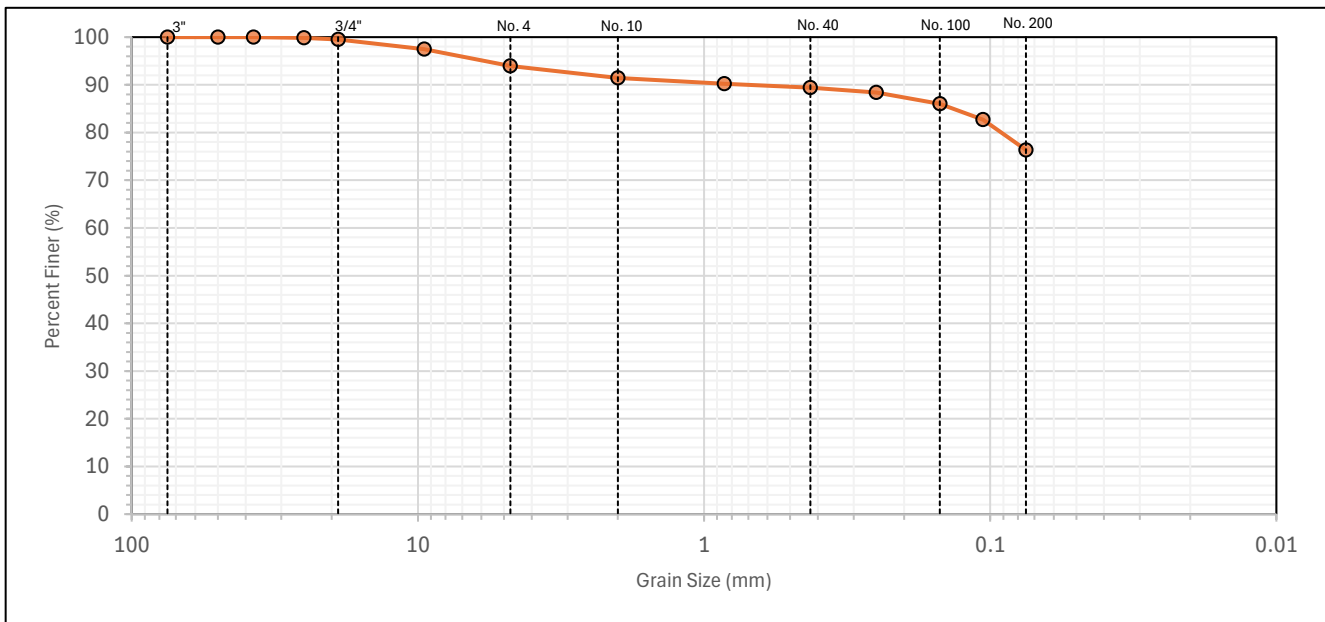
Test Date:	11/12/2025
Tested By:	RS/KC

Test Notes:

Gravel (%):	6.0
Sand (%):	17.6
Fines (%):	76.4

Coarse Fraction: >3/8"	
Dry Mass (g):	540.2
> 200 Mass (g):	499.3
Water Content (%):	1.6
Fine Fraction: <3/8"	
Fraction Dry Mass	17891.1
Tested Dry Mass (g):	248.2
Tested > No. 200 (g):	54.5
Water Content (%):	13.0

Sieve	Sieve Opening (mm)	Percent Finer (%)	Percent Retained (%)
5"	125	100	0
4"	100	100	0
3.5"	90	100	0
3"	75	100	0
2"	50	100	0
1.5"	37.5	100	0
1.0"	25	100	0
3/4"	19	99	1
3/8"	9.5	97	3
No. 4	4.75	94	6
No. 10	2	91	9
No. 20	0.85	90	10
No. 40	0.425	89	11
No. 60	0.25	88	12
No. 100	0.15	86	14
No. 140	0.106	83	17
No. 200	0.075	76	24
Pan	-	0.0	100.0



**Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis**

ASTM D6913



Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-01
Sample ID:	MC-03
Depth:	7.5-9
Sample Date:	11/3/2025
Sample Description:	Brown sandy gravel w/ clay
Prep Method:	Moist
Test Method:	A
Number of Separations:	1

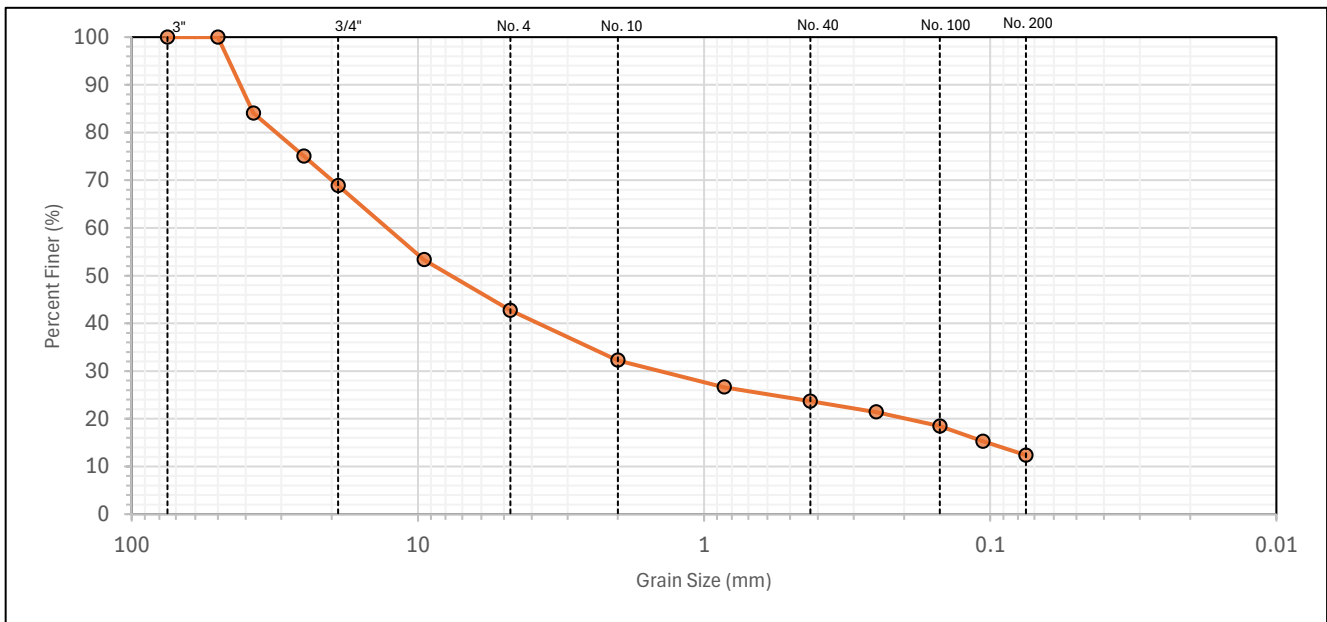
Test Date:	11/12/2025
Tested By:	RS/KC

Test Notes:

Gravel (%):	57.2
Sand (%):	30.4
Fines (%):	12.4

Coarse Fraction: >3/8"	
Dry Mass (g):	1402.5
> 200 Mass (g):	1382.9
Water Content (%):	3.7
Fine Fraction: <3/8"	
Fraction Dry Mass	1441.4
Tested Dry Mass (g):	214.4
Tested > No. 200 (g):	168.1
Water Content (%):	7.0

Sieve	Sieve Opening (mm)	Percent Finer (%)	Percent Retained (%)
5"	125	100	0
4"	100	100	0
3.5"	90	100	0
3"	75	100	0
2"	50	100	0
1.5"	37.5	84	16
1.0"	25	75	25
3/4"	19	69	31
3/8"	9.5	53	47
No. 4	4.75	43	57
No. 10	2	32	68
No. 20	0.85	27	73
No. 40	0.425	24	76
No. 60	0.25	21	79
No. 100	0.15	18	82
No. 140	0.106	15	85
No. 200	0.075	12	88
Pan	-	0.0	100.0



**Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis**

ASTM D6913



Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-02
Sample ID:	MC-03
Depth:	7.5-9
Sample Date:	11/3/2025
Sample Description:	Brown silt w/ gravel
Prep Method:	Moist
Test Method:	A
Number of Separations:	1

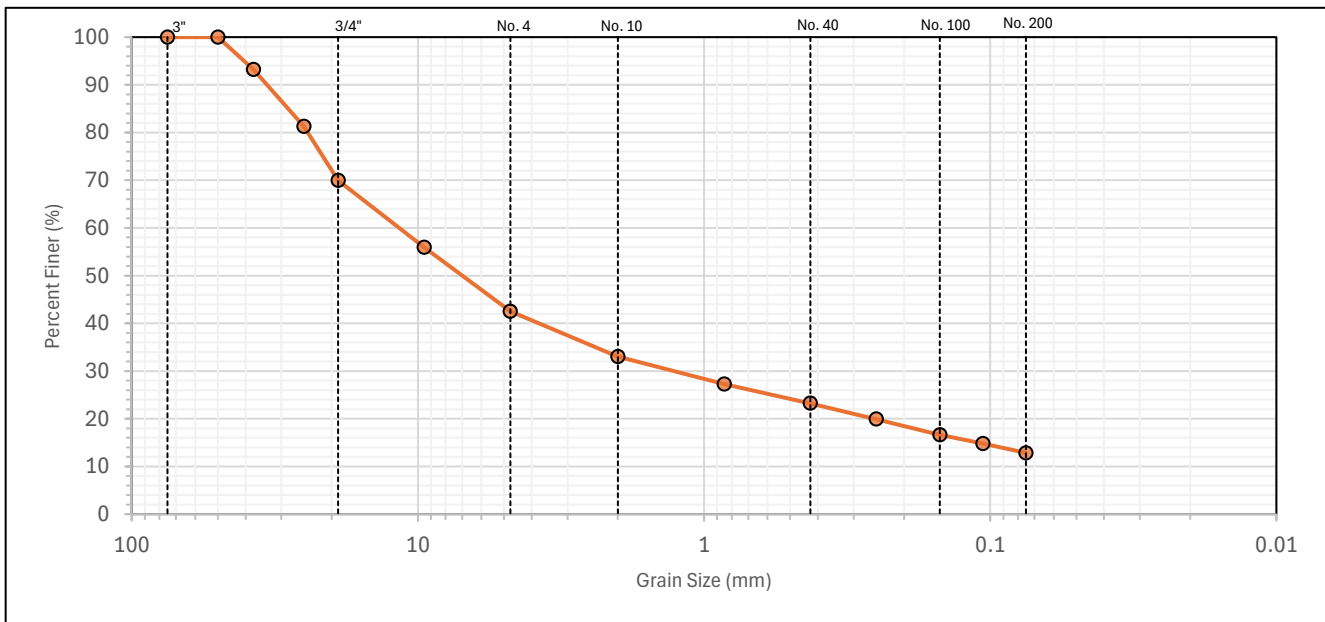
Test Date:	11/13/2025
Tested By:	KC

Test Notes:

Gravel (%):	57.5
Sand (%):	29.7
Fines (%):	12.8

Coarse Fraction: >3/8"	
Dry Mass (g):	1023.9
> 200 Mass (g):	1007.3
Water Content (%):	2.5
Fine Fraction: <3/8"	
Fraction Dry Mass	1175.1
Tested Dry Mass (g):	252.5
Tested > No. 200 (g):	197.6
Water Content (%):	5.7

Sieve	Sieve Opening (mm)	Percent Finer (%)	Percent Retained (%)
5"	125	100	0
4"	100	100	0
3.5"	90	100	0
3"	75	100	0
2"	50	100	0
1.5"	37.5	93	7
1.0"	25	81	19
3/4"	19	70	30
3/8"	9.5	56	44
No. 4	4.75	42	58
No. 10	2	33	67
No. 20	0.85	27	73
No. 40	0.425	23	77
No. 60	0.25	20	80
No. 100	0.15	17	83
No. 140	0.106	15	85
No. 200	0.075	13	87
Pan	-	0.0	100.0



**Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis**

ASTM D6913



Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-03
Sample ID:	MC-03
Depth:	7.5-9
Sample Date:	11/3/2025
Sample Description:	Brown silty sand w/ gravel
Prep Method:	Moist
Test Method:	A
Number of Separations:	1

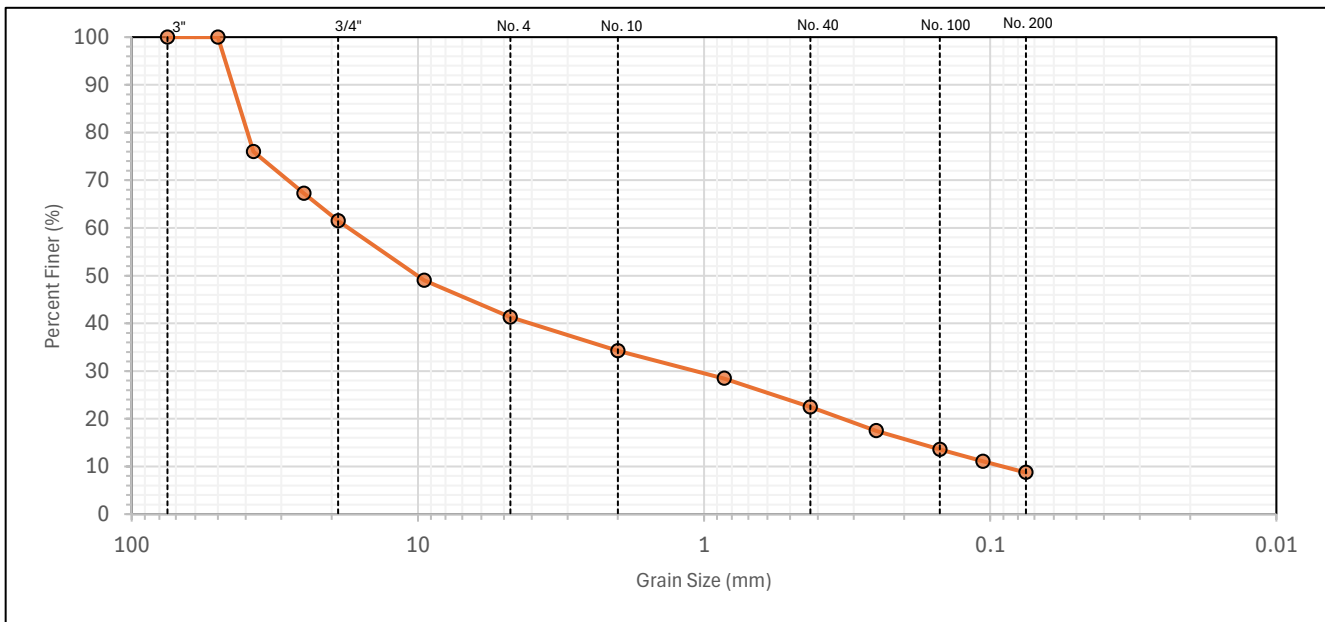
Test Date:	11/13/2025
Tested By:	KC

Test Notes:

Gravel (%):	58.7
Sand (%):	32.6
Fines (%):	8.7

Coarse Fraction: >3/8"	
Dry Mass (g):	1084.76
> 200 Mass (g):	1074.06
Water Content (%):	3.1
Fine Fraction: <3/8"	
Fraction Dry Mass	918.0
Tested Dry Mass (g):	263.41
Tested > No. 200 (g):	219.91
Water Content (%):	5.2

Sieve	Sieve Opening (mm)	Percent Finer (%)	Percent Retained (%)
5"	125	100	0
4"	100	100	0
3.5"	90	100	0
3"	75	100	0
2"	50	100	0
1.5"	37.5	76	24
1.0"	25	67	33
3/4"	19	61	39
3/8"	9.5	49	51
No. 4	4.75	41	59
No. 10	2	34	66
No. 20	0.85	28	72
No. 40	0.425	22	78
No. 60	0.25	17	83
No. 100	0.15	14	86
No. 140	0.106	11	89
No. 200	0.075	8.7	91.3
Pan	-	0.0	100.0



Project: Wellington City Building Material Finer than No. 200 Sieve  
 Project No: 2025-098 ASTM D1140  
 Location: Wellington, UT

\*Dry mass determined via companion

Test Number:	Exploration:	Sample ID:	Depth:	Sample Date:	Sample Description:	Test Date:	Tested By:	Test Method:	Soak Time (min):	Initial Dry Mass (g):	Water Content (%):	Fines (%):	Notes:
1	B-01	SPT-01	2.5-4	11/3/2025	Brown silty clay	11/12/2025	DP	B	390	122.54	17.6	90.2	Limited sample size. Fines tested after Atterberg.
2	B-01	MC-05	15-16.5	11/3/2025	Grayish brown silty clay	11/13/2025	KC	B	350	219.51	11.4	93.12	
3	B-02	SPT-01	2.5-4	11/3/2025	Brown sandy clay w/ gravel	11/12/2025	DP	B	380	105.63	12.1	60.8	Limited sample size. Fines tested after Atterberg.
4	B-02	SPT-06	20-21.5	11/3/2025	Gray silty clay	11/13/2025	KC	B	370	202.97	11.2	95.49	
5	B-03	SPT-01	2.5-4	11/3/2025	Brown silty clay	11/12/2025	DP	B	370	146.91	16.4	89.6	Limited sample size. Fines tested after Atterberg.
6	B-03	MC-05	15-16.5	11/3/2025	Gray silty clay	11/13/2025	KC	B	380	215.63	10.5	89.48	

Project: Wellington City Building  
 Project No: 2025-098  
 Location: Wellington, UT

**Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sedimentation (Hydrometer) Analysis**  
 ASTM D7928

Clay Fraction (%)

Test Number:	Sample Source:	Sample ID:	Depth:	Description:	Sand:	Fines:	Silt:	Clay:
1	B-01	MC-05	15-16.5	Grayish brown silty clay	14.7	85.3	55.6	29.7
2	B-02	SPT-06	20-21.5	Gray silty clay	7.8	92.2	53.5	38.7
3	B-03	MC-05	15-16.5	Gray silty clay	31.6	68.4	45.1	23.3

Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sedimentation (Hydrometer) Analysis

ASTM D7928

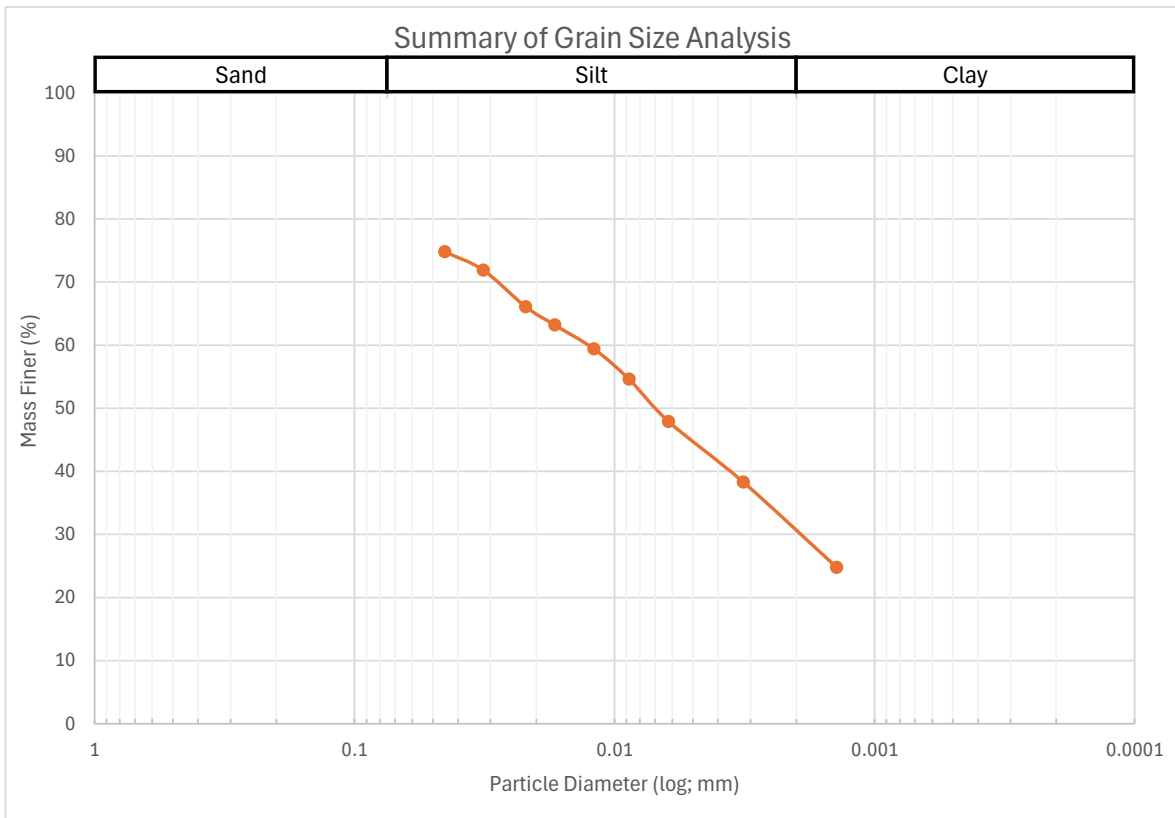


Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Sample Date:	11/3/2025
Exploration:	B-01
Sample ID:	MC-05
Depth (ft):	15-16.5
Description:	Grayish brown silty clay
Sample Moist Mass (0.01g):	46.25
Sample Dry Mass (0.01g):	41.51
Sample Water Content (%):	11.4
Companion Sample Used:	Yes

Tested By:	DP	
Test Sample Reduction:	Whole	
Prep Method:	Moist	
Separation Sieve:	No. 10	
Sample Specific Gravity:	2.65	Assumed

Summary of Results for Sedimentation Specimen		
Sand	Fines	
14.7	85.3	
	Silt	Clay
	55.6	29.7

Dispersant Method:	Mix in 100 mL	Dispersant (0.01g):	5.02	Dispersion Device:	Cup & Mixer	
Foam Inhibitor:	No	Test Cylinder:	5	Hydrometer:	151H	
Temp Density Correction:	A/B Constant	Test Start Date:	11/18/2025	Test Start Time:	8:45 AM	
Time elapsed (min)	Hydrometer Reading (r <sub>m</sub> )	Temp °C	Hydrometer Offset Read (r <sub>d,m</sub> )	Effective Depth (H <sub>m</sub> ; cm)	Particle Diameter (D; mm)	Mass Finer (N <sub>m</sub> ;%)
1	1.02225	22.0	1.0028	11	0.045	74.8
2	1.0215	22.0	1.0028	11	0.032	71.9
5	1.02	22.0	1.0028	12	0.022	66.1
8	1.01925	22.0	1.0028	12	0.017	63.2
15	1.01825	22.0	1.0028	12	0.012	59.4
30	1.017	22.0	1.0028	12	0.009	54.6
62	1.01525	22.1	1.0028	13	0.0062	47.9
241	1.01275	22.1	1.0028	14	0.0032	38.3
1439	1.00925	22.1	1.0028	14	0.0014	24.8



Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sedimentation (Hydrometer) Analysis

ASTM D7928

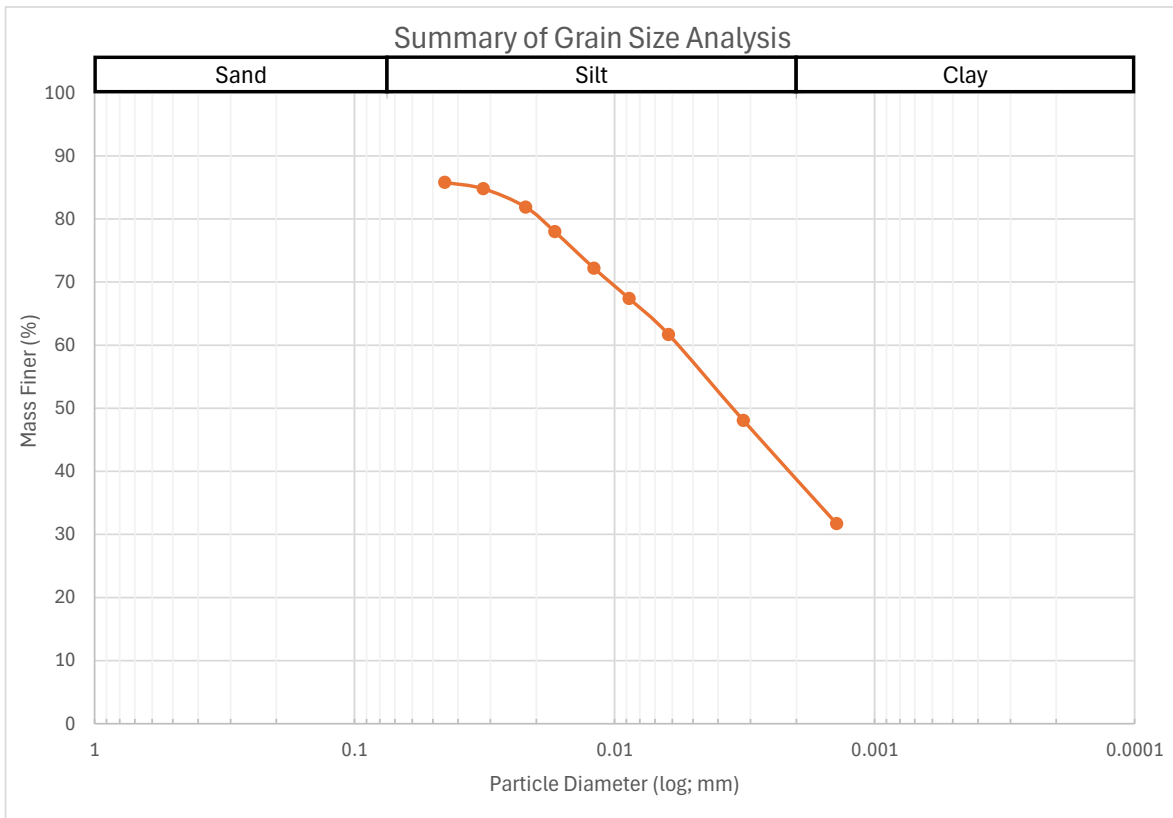


Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Sample Date:	11/3/2025
Exploration:	B-02
Sample ID:	SPT-06
Depth (ft):	20-21.5
Description:	Gray silty clay
Sample Moist Mass (0.01g):	45.92
Sample Dry Mass (0.01g):	41.28
Sample Water Content (%):	11.2
Companion Sample Used:	Yes

Tested By:	DP	
Test Sample Reduction:	Whole	
Prep Method:	Moist	
Separation Sieve:	No. 10	
Sample Specific Gravity:	2.65	Assumed

Summary of Results for Sedimentation Specimen		
Sand	Fines	
7.8	92.2	
	Silt	Clay
	53.5	38.7

Dispersant Method:	Mix in 100 mL	Dispersant (0.01g):	4.7	Dispersion Device:	Cup & Mixer	
Foam Inhibitor:	No	Test Cylinder:	6	Hydrometer:	151H	
Temp Density Correction:	A/B Constant	Test Start Date:	11/18/2025	Test Start Time:	8:55 AM	
Time elapsed (min)	Hydrometer Reading (r <sub>m</sub> )	Temp °C	Hydrometer Offset Read (r <sub>d,m</sub> )	Effective Depth (H <sub>m</sub> ; cm)	Particle Diameter (D; mm)	Mass Finer (N <sub>m</sub> ;%)
1	1.025	22.0	1.0028	10	0.044	85.8
2	1.02475	22.0	1.0028	10	0.031	84.8
4	1.024	22.0	1.0028	11	0.022	81.9
8	1.023	22.0	1.0028	11	0.016	78
15	1.0215	22.0	1.0028	11	0.012	72.2
30	1.02025	22.0	1.0028	12	0.008	67.4
60	1.01875	22.1	1.0028	12	0.0061	61.7
247	1.01525	22.1	1.0028	13	0.0031	48.1
1440	1.011	22.1	1.0028	14	0.0013	31.7



Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sedimentation (Hydrometer) Analysis

ASTM D7928

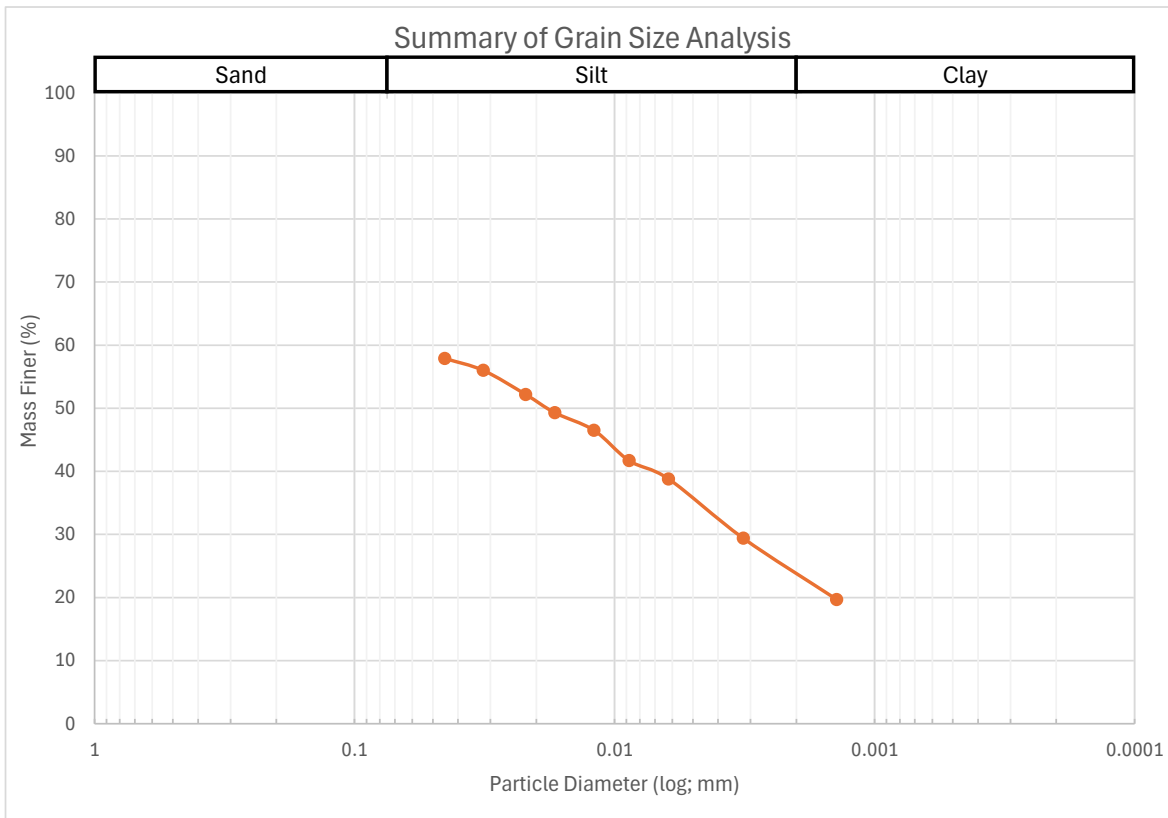


Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Sample Date:	11/3/2025
Exploration:	B-03
Sample ID:	MC-05
Depth (ft):	15-16.5
Description:	Gray silty clay
Sample Moist Mass (0.01g):	46.22
Sample Dry Mass (0.01g):	41.83
Sample Water Content (%):	10.5
Companion Sample Used:	Yes

Tested By:	DP	
Test Sample Reduction:	Whole	
Prep Method:	Moist	
Separation Sieve:	No. 10	
Sample Specific Gravity:	2.65	Assumed

Summary of Results for Sedimentation Specimen		
Sand	Fines	
31.6	68.4	
	Silt	Clay
	45.1	23.3

Dispersant Method:	Mix in 100 mL	Dispersant (0.01g):	4.95	Dispersion Device:	Cup & Mixer	
Foam Inhibitor:	No	Test Cylinder:	7	Hydrometer:	151H	
Temp Density Correction:	A/B Constant	Test Start Date:	11/18/2025	Test Start Time:	9:05 AM	
Time elapsed (min)	Hydrometer Reading (r <sub>m</sub> )	Temp °C	Hydrometer Offset Read (r <sub>d,m</sub> )	Effective Depth (H <sub>m</sub> ; cm)	Particle Diameter (D; mm)	Mass Finer (N <sub>m</sub> ;%)
1	1.018	22.0	1.0028	12	0.047	57.9
2	1.0175	22.0	1.0028	12	0.034	56
4	1.0165	22.0	1.0028	13	0.024	52.2
8	1.01575	22.0	1.0028	13	0.017	49.3
15	1.015	22.0	1.0028	13	0.013	46.5
30	1.01375	22.0	1.0028	13	0.009	41.7
60	1.013	22.0	1.0028	13	0.0065	38.8
238	1.0105	22.1	1.0028	14	0.0033	29.4
1439	1.008	22.0	1.0028	15	0.0014	19.7



Project: Wellington City Building    **Liquid Limit, Plastic Limit, and Plasticity Index of Soils (Atterberg)**  
 Project No: 2025-098    Standard: ASTM D4318  
 Location: Wellington, UT



Sheet Number:	Sample Source:	Sample ID:	Depth:	Description:	Liquid Limit (%):	Plastic Limit (%):	Plasticity Index:	USCS	Test Notes
1	B-01	SPT-01	2.5-4	Brown silty clay	28	16	12	CL	
2	B-01	MC-05	15-16.5	Grayish brown silty clay	33	20	13	CL	
3	B-02	SPT-01	2.5-4	Brown sandy clay w/ gravel	27	13	14	CL	
4	B-02	SPT-06	20-21.5	Gray silty clay	40	21	19	CL	Limited sample
5	B-03	SPT-01	2.5-4	Brown silty clay	29	16	13	CL	
6	B-03	MC-05	15-16.5	Gray silty clay	36	19	17	CL	

# Liquid Limit, Plastic Limit, and Plasticity Index of Soils

ASTM D4318



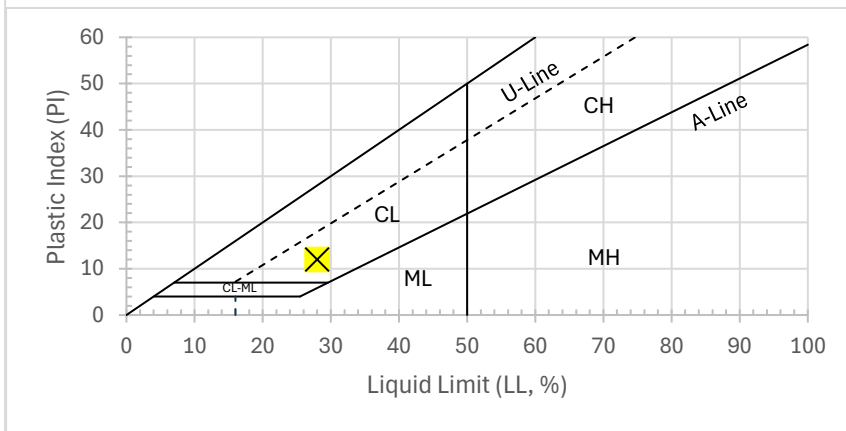
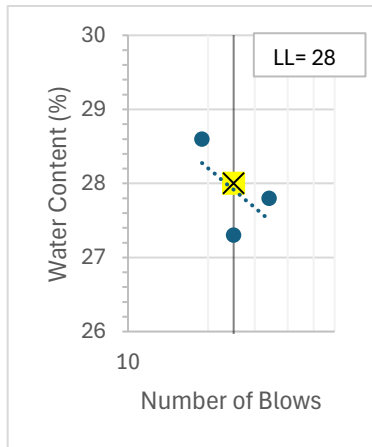
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration Name:	B-01
Sample ID:	SPT-01
Depth (ft):	2.5-4
Sample Date:	11/3/2025
Description:	Brown silty clay
Test Date:	11/25/2025
Tested By:	RS
Test Notes:	

USCS Soil Classification: **CL**

Liquid Limit (%):	28
Plastic Limit (%):	16
Plasticity Index:	12

Sample Preparation:	Wet
Liquid Limit Procedure:	Multi-Point (3)
Plastic Limit Procedure:	Rolling Device

	Liquid Limit			Plastic Limit	
	15-25	20-30	25-35		
Number of Blows:	19	25	34		
Tare (g):	21.02	20.96	21.04	21.05	21.05
Wet Sample + Tare (g):	31.17	31.7	31.71	31.5	31.16
Dry Sample + Tare (g):	28.91	29.4	29.39	30.08	29.82
Water Content (%):	<b>28.6</b>	<b>27.3</b>	<b>27.8</b>	<b>15.7</b>	<b>15.3</b>



# Liquid Limit, Plastic Limit, and Plasticity Index of Soils

ASTM D4318



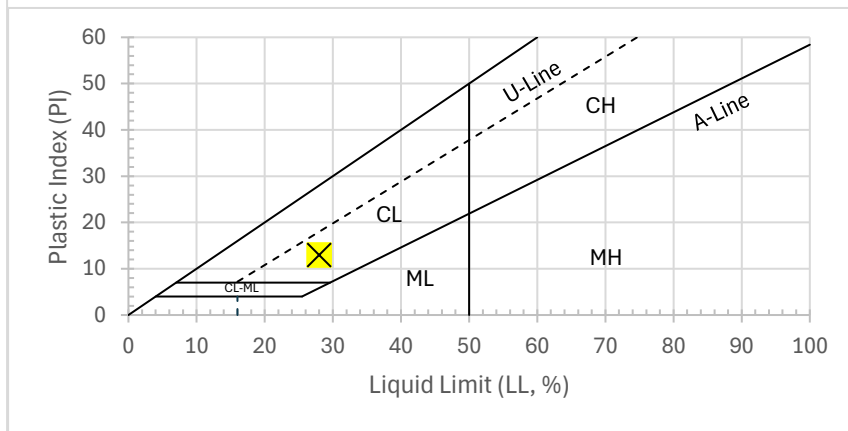
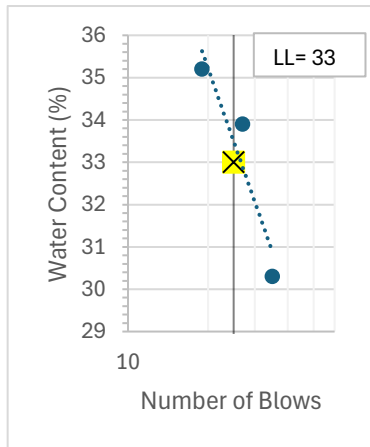
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration Name:	B-01
Sample ID:	MC-05
Depth (ft):	15-16.5
Sample Date:	11/3/2025
Description:	Grayish brown silty clay
Test Date:	11/25/2025
Tested By:	RS
Test Notes:	

USCS Soil Classification: **CL**

Liquid Limit (%): **33**  
 Plastic Limit (%): **20**  
 Plasticity Index: **13**

Sample Preparation: **Wet**  
 Liquid Limit Procedure: **Multi-Point (3)**  
 Plastic Limit Procedure: **Rolling Device**

	Liquid Limit			Plastic Limit	
	15-25	20-30	25-35		
Number of Blows:	19	27	35		
Tare (g):	21.01	21.18	21.14	21.11	21.04
Wet Sample + Tare (g):	31.5	32.12	32.1	31.68	31.87
Dry Sample + Tare (g):	28.77	29.35	29.55	29.9	30.1
Water Content (%):	<b>35.2</b>	<b>33.9</b>	<b>30.3</b>	<b>20.3</b>	<b>19.5</b>



# Liquid Limit, Plastic Limit, and Plasticity Index of Soils

ASTM D4318



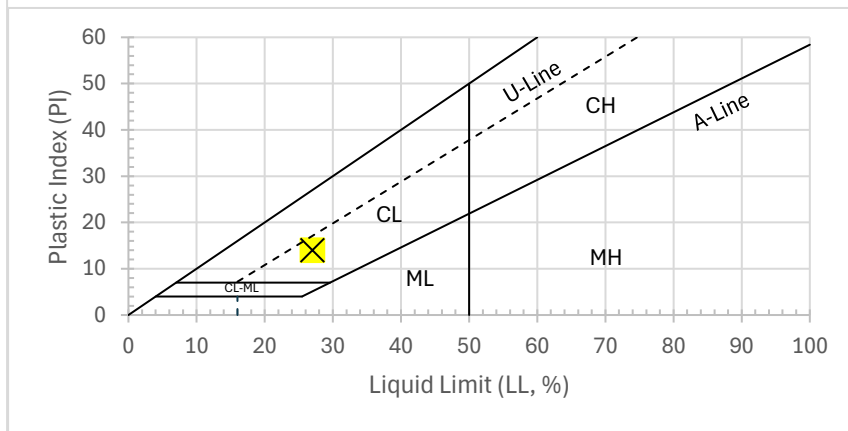
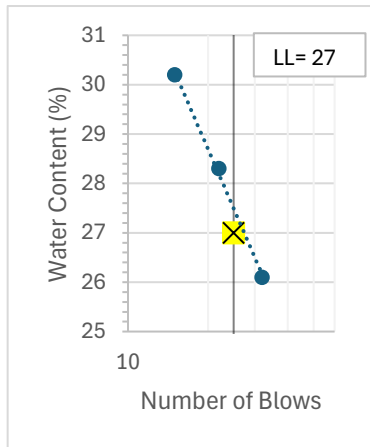
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration Name:	B-02
Sample ID:	SPT-01
Depth (ft):	2.5-4
Sample Date:	11/3/2025
Description:	Brown sandy clay w/ gravel
Test Date:	11/25/2025
Tested By:	RS
Test Notes:	

USCS Soil Classification: **CL**

Liquid Limit (%): **27**  
 Plastic Limit (%): **13**  
 Plasticity Index: **14**

Sample Preparation: **Wet**  
 Liquid Limit Procedure: **Multi-Point (3)**  
 Plastic Limit Procedure: **Rolling Device**

	Liquid Limit			Plastic Limit	
	15-25	20-30	25-35		
Number of Blows:	15	22	32		
Tare (g):	21.16	21.15	21.11	21.08	121.06
Wet Sample + Tare (g):	31.67	31.9	31.564	31.23	31.12
Dry Sample + Tare (g):	29.23	29.53	29.4	30.06	29.9
Water Content (%):	<b>30.2</b>	<b>28.3</b>	<b>26.1</b>	<b>13.0</b>	<b>#NUM!</b>



# Liquid Limit, Plastic Limit, and Plasticity Index of Soils

ASTM D4318



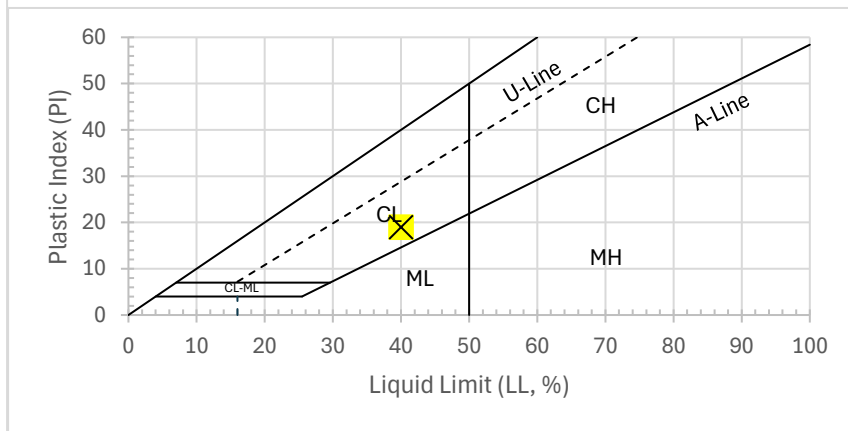
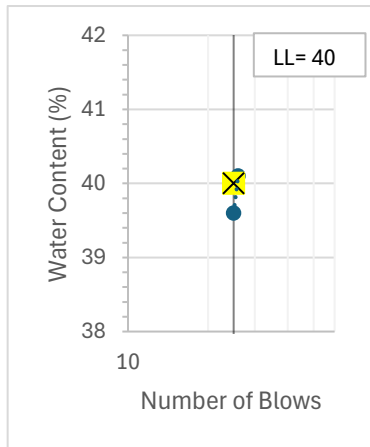
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration Name:	B-02
Sample ID:	SPT-06
Depth (ft):	20-21.5
Sample Date:	11/3/2025
Description:	Gray silty clay
Test Date:	11/26/2025
Tested By:	RS
Test Notes:	Limited sample

USCS Soil Classification: **CL**

Liquid Limit (%): **40**  
 Plastic Limit (%): **21**  
 Plasticity Index: **19**

Sample Preparation: **Wet**  
 Liquid Limit Procedure: **One-Point**  
 Plastic Limit Procedure: **Rolling Device**

	Liquid Limit		Plastic Limit	
	20-30	20-30		
Number of Blows:	26	25		
Tare (g):	21.08	21.13	21.07	21
Wet Sample + Tare (g):	31.6	31.63	31.17	31.42
Dry Sample + Tare (g):	28.59	28.65	29.43	29.64
Water Content (%):	<b>40.1</b>	<b>39.6</b>	<b>20.8</b>	<b>20.6</b>



**Liquid Limit, Plastic Limit, and Plasticity Index of Soils**

ASTM D4318



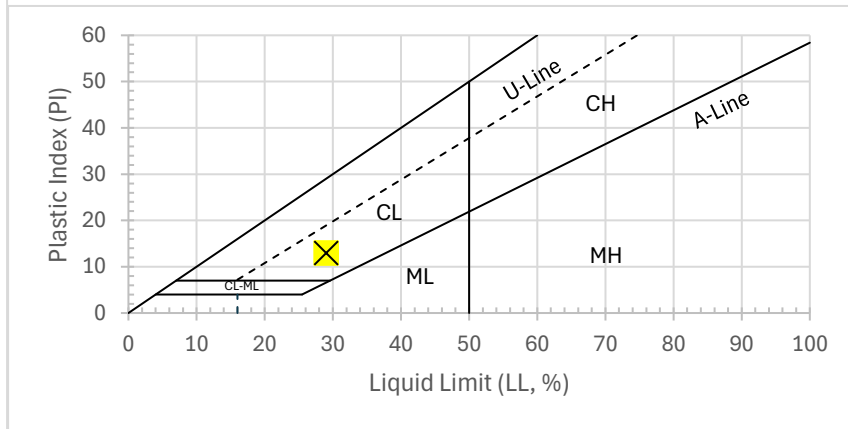
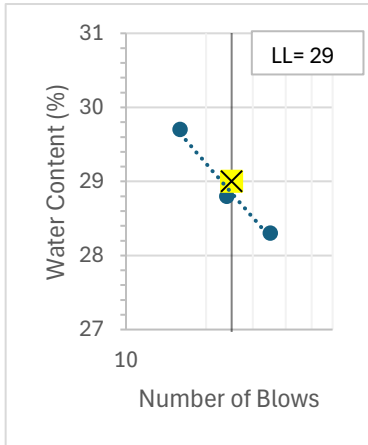
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration Name:	B-03
Sample ID:	SPT-01
Depth (ft):	2.5-4
Sample Date:	11/3/2025
Description:	Brown silty clay
Test Date:	11/25/2025
Tested By:	RS
Test Notes:	

USCS Soil Classification: **CL**

Liquid Limit (%):	29
Plastic Limit (%):	16
Plasticity Index:	13

Sample Preparation:	Wet
Liquid Limit Procedure:	Multi-Point (3)
Plastic Limit Procedure:	Rolling Device

	Liquid Limit			Plastic Limit	
	15-25	20-30	25-35		
Number of Blows:	16	24	35		
Tare (g):	21.21	21.01	20.97	21.09	21.11
Wet Sample + Tare (g):	31.77	32.09	32.13	31.26	31.25
Dry Sample + Tare (g):	29.35	29.61	29.67	29.84	29.87
Water Content (%):	29.7	28.8	28.3	16.2	15.8



**Liquid Limit, Plastic Limit, and Plasticity Index of Soils**

ASTM D4318



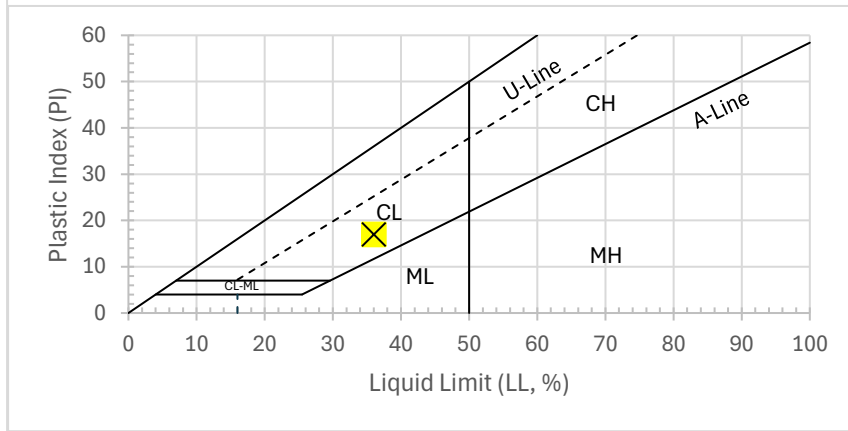
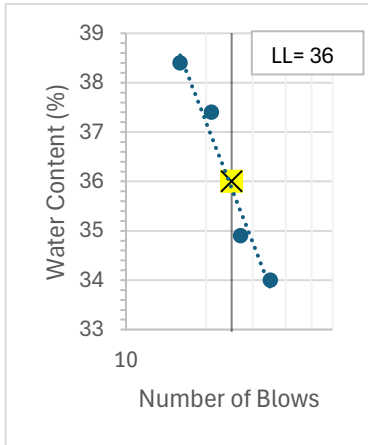
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration Name:	B-03
Sample ID:	MC-05
Depth (ft):	15-16.5
Sample Date:	11/3/2025
Description:	Gray silty clay
Test Date:	11/26/2025
Tested By:	RS
Test Notes:	

USCS Soil Classification: **CL**

Liquid Limit (%): **36**  
 Plastic Limit (%): **19**  
 Plasticity Index: **17**

Sample Preparation: **Wet**  
 Liquid Limit Procedure: **Multi-Point (4)**  
 Plastic Limit Procedure: **Rolling Device**

	Liquid Limit				Plastic Limit	
	15-20	20-25	25-30	30-35		
Number of Blows:	16	21	27	35		
Tare (g):	21.04	21.11	21.11	21.1	20.98	21.09
Wet Sample + Tare (g):	31.14	31.62	31.62	31.74	31.45	31.47
Dry Sample + Tare (g):	28.34	28.76	28.9	29.04	29.74	29.78
Water Content (%):	<b>38.4</b>	<b>37.4</b>	<b>34.9</b>	<b>34.0</b>	<b>19.5</b>	<b>19.4</b>

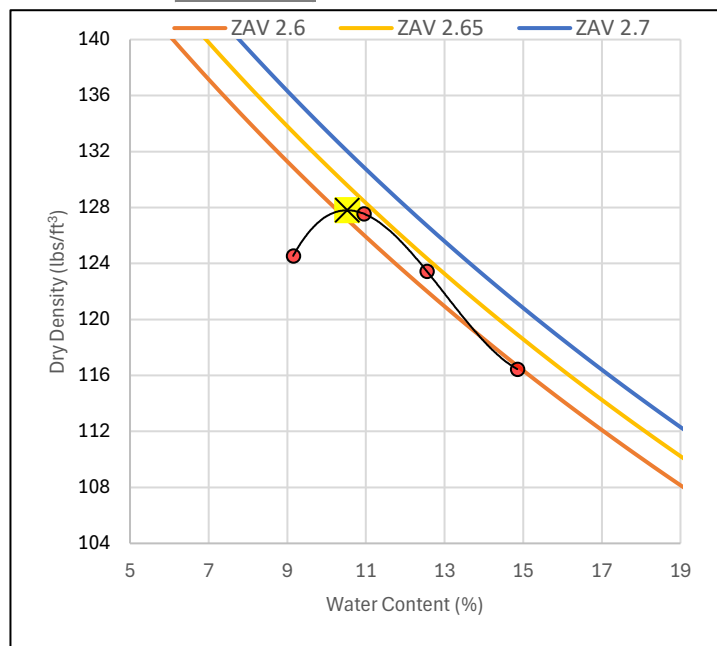


# Laboratory Compaction Characteristics of Soil Using Modified Effort

ASTM D1557



Project:	Wellington City Building	Exploration Name:	B-01
Project No:	2025-098	Sample ID:	BULK-01
Location:	Wellington, UT	Depth (ft):	4-7
Sample Date:	11/3/2025	Description:	Brown silty clay w/ gravel
Test Date:	11/13/2025	Prep Method:	Moist
Tested By:	RS	Test Method:	B
Reduced By:	RS	Rammer:	Manual
Test Notes		Proctor Effort:	Modified
		Percent Retained, P <sub>c</sub> (%):	2
		Percent Passing, P <sub>f</sub> (%):	98
Maximum Dry Unit Weight (lb/ft <sup>3</sup> ):	127.7	Corrected Dry Unit Weight (lb/ft <sup>3</sup> ):	N/A
Optimum Water Content (%):	10.5	Corrected Optimum Water Content (%):	N/A



Sieve for Oversized Correction:	3/8-in	Mold Size:	4
Oversized Water Content (%):	1.6	Mold Volume (ft <sup>3</sup> ):	0.0333
Passing Water Content (%):	13.0	Bulk Specific Gravity, G <sub>s</sub> :	2.6 Assumed

Weight of Mold (g):	4107.9	4107.8	4108	4107.9
Weight of Sample + Mold (g):	6156.9	6241.7	6203.2	6124.6
Moist Unit Weight (lb/ft <sup>3</sup> ):	135.7	141.3	138.7	133.5
Tare (g):	281.7	282.1	282.6	281.5
Wet Soil + Tare (g):	2318.6	2415.2	2376.1	2290.5
Dry Soil + Tare (g):	2148.2	2204.8	2142.8	2032.1
Water Content (%):	9.1	10.9	12.5	14.8
Dry Unit Weight (lb/ft <sup>3</sup> ):	124.4	127.4	123.3	116.3

Project: Wellington City Building      Standard Test Methods for 1-D Swell or Collapse of Soils  
 Project No: 2025-098                      ASTM D4546  
 Location: Wellington, UT

Test:	Sample Source:	Sample ID:	Depth:	Description:	Inundation Stress (psf):	Inundation Collapse (%):	Initial Void Ratio:	Final Void Ratio:	Test Notes:
1	B-01	ST-02	5-7	Brown silty sand	1000	0.6	0.732	0.700	
2	B-02	ST-02	4-6	Brown Clayey silt	1000	0.5	0.745	0.714	
3	B-03	ST-02	4-6	Brown clayey silt	1000	0.6	0.695	0.662	

Standard Test Methods for 1-D Swell or Collapse of Soils  
ASTM D4546



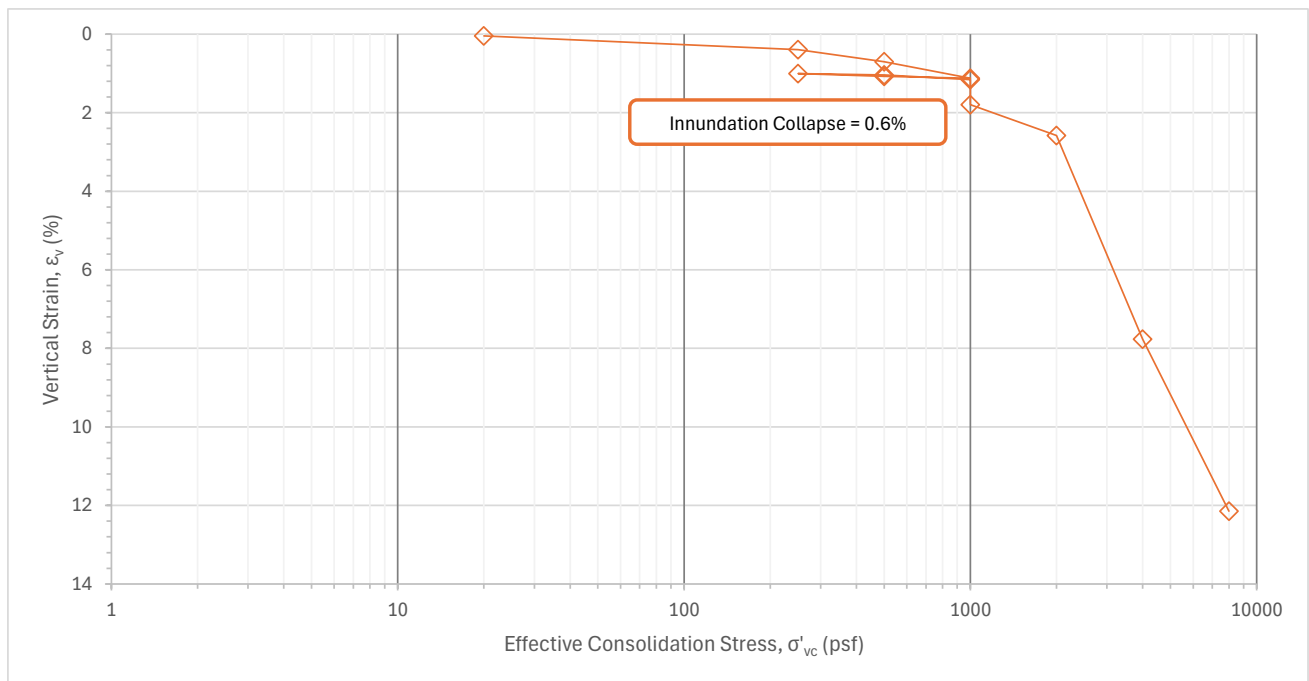
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-01
Sample ID:	ST-02
Depth (ft):	5-7
Sample Date:	11/3/2025
Description:	Brown silty sand
Tested By:	DP
Test Date:	11/17/2025
Test Method:	C
Sample Type:	Intact
Intact Sample Disturbed:	No
Sample Specific Gravity:	2.65 Assumed
Innundation Water Used:	Tap
Mass of Oversized (g):	0.00

	Stress (psf)	Dial Reading	$\epsilon_v$ (%)	Height (in)
Initial		0.0115	0.00	0.9881
Seating (20)		0.0120	0.05	0.9877
250		0.0155	0.40	0.9842
500		0.0185	0.70	0.9811
1000		0.0227	1.12	0.9769
500		0.0223	1.08	0.9774
250		0.0216	1.01	0.9781
500		0.0219	1.04	0.9778
1000		0.0231	1.16	0.9765
1000		0.0295	1.80	0.9702
2000		0.0373	2.58	0.9623
4000		0.0891	7.76	0.9105
8000		0.1329	12.14	0.8667

Innundation Stress (psf):	1000
Innundation Collapse (%):	0.6

	Initial	Final
Sample Mass (g):	141.24	145.25
Water Content (%):	14.76	19.9
Sample Height (h, in):	1.000	0.982
Sample Diameter (D, in):	2.502	2.502
Dry Unit Weight ( $\rho_d$ , pcf):	95.371	94.596
Saturation (%):	53.48	70.73
Void Ratio:	0.732	0.700

Test Notes:



Standard Test Methods for 1-D Swell or Collapse of Soils  
ASTM D4546



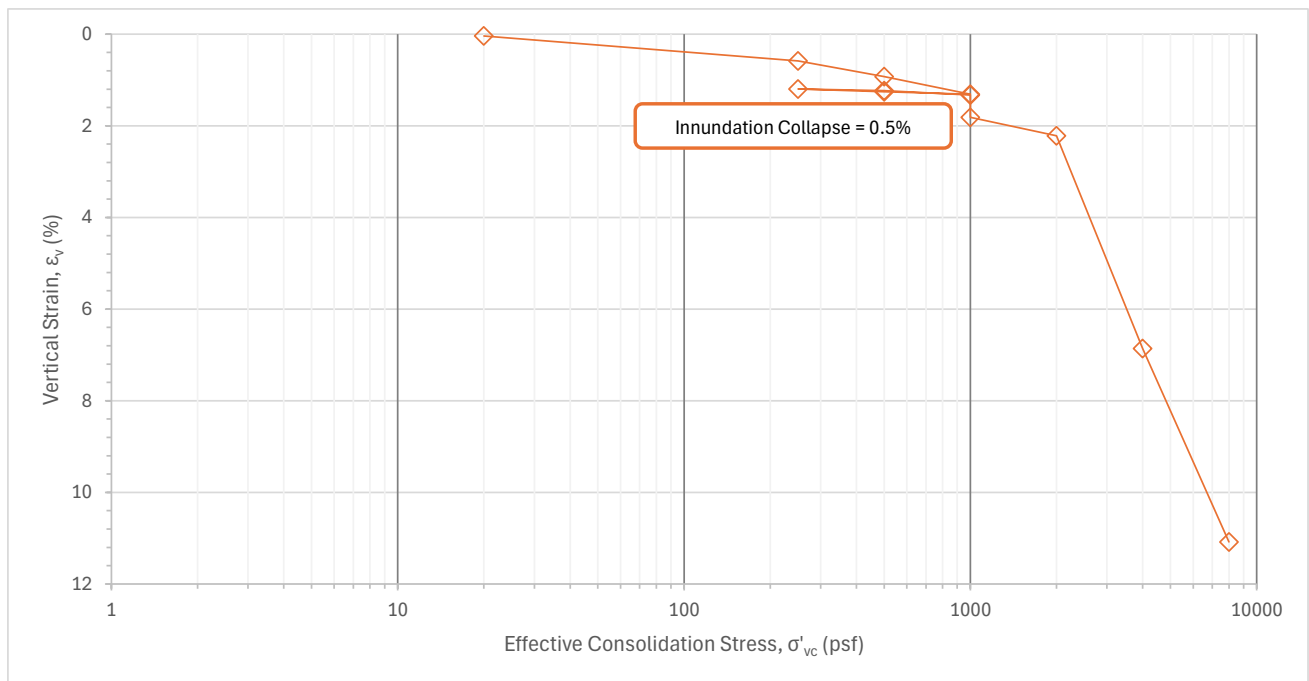
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-02
Sample ID:	ST-02
Depth (ft):	4-6
Sample Date:	11/3/2025
Description:	Brown Clayey silt
Tested By:	DP
Test Date:	11/24/2025
Test Method:	C
Sample Type:	Intact
Intact Sample Disturbed:	No
Sample Specific Gravity:	2.65 Assumed
Innundation Water Used:	Tap
Mass of Oversized (g):	0.00

	Stress (psf)	Dial Reading	$\epsilon_v$ (%)	Height (in)
Initial		0.0070	0.00	0.9934
Seating (20)		0.0074	0.04	0.9930
250		0.0129	0.58	0.9875
500		0.0163	0.92	0.9841
1000		0.0201	1.31	0.9803
500		0.0196	1.25	0.9808
250		0.0190	1.20	0.9814
500		0.0194	1.23	0.9810
1000		0.0203	1.33	0.9801
1000		0.0252	1.81	0.9752
2000		0.0292	2.22	0.9712
4000		0.0757	6.86	0.9247
8000		0.1179	11.08	0.8825

Innundation Stress (psf):	1000
Innundation Collapse (%):	0.5

	Initial	Final
Sample Mass (g):	140.74	148.10
Water Content (%):	15.01	20.3
Sample Height (h, in):	1.000	0.982
Sample Diameter (D, in):	2.504	2.504
Dry Unit Weight ( $\rho_d$ , pcf):	94.619	96.474
Saturation (%):	53.36	75.58
Void Ratio:	0.745	0.714

Test Notes:



Standard Test Methods for 1-D Swell or Collapse of Soils  
ASTM D4546



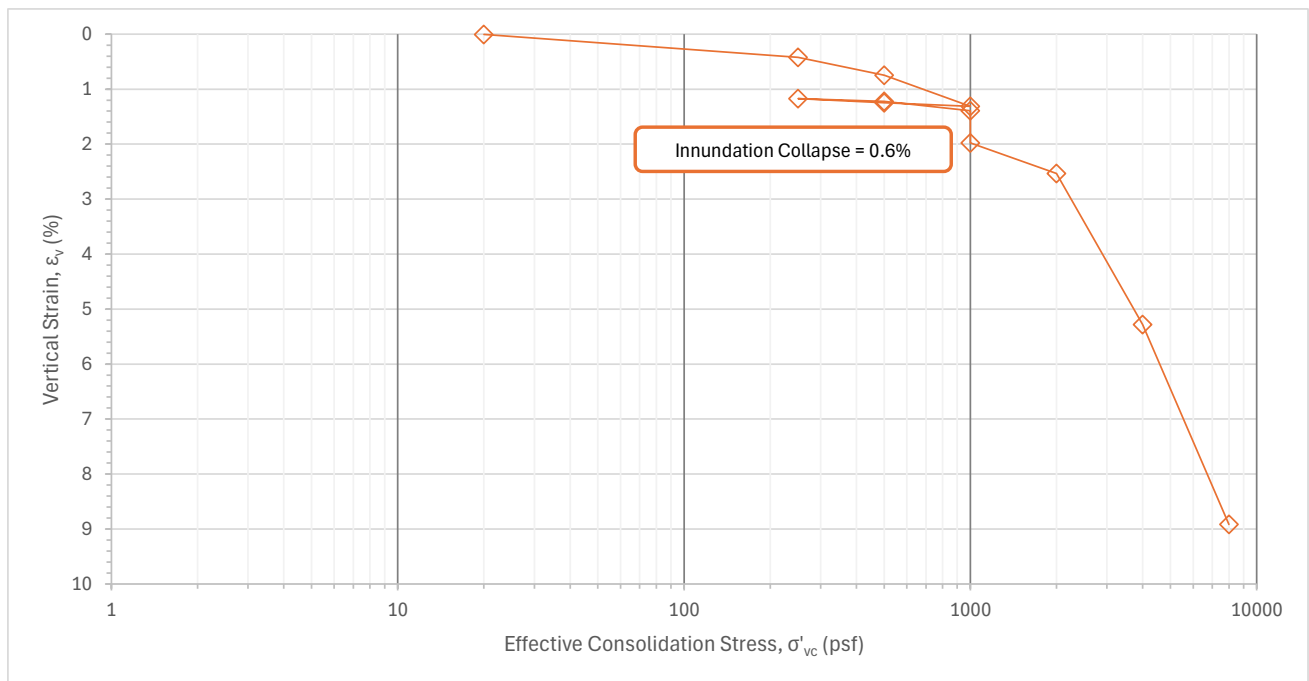
Project:	Wellington City Building
Project No:	2025-098
Location:	Wellington, UT
Exploration:	B-03
Sample ID:	ST-02
Depth (ft):	4-6
Sample Date:	11/3/2025
Description:	Brown clayey silt
Tested By:	DP
Test Date:	11/24/2025
Test Method:	C
Sample Type:	Intact
Intact Sample Disturbed:	No
Sample Specific Gravity:	2.65 Assumed
Innundation Water Used:	Tap
Mass of Oversized (g):	0.00

	Stress (psf)	Dial Reading	$\epsilon_v$ (%)	Height (in)
Initial		0.0090	0.00	0.9914
Seating (20)		0.0091	0.00	0.9913
250	250	0.0132	0.42	0.9872
500	500	0.0165	0.74	0.9839
1000	1000	0.0222	1.31	0.9782
500	500	0.0215	1.25	0.9789
250	250	0.0208	1.17	0.9796
500	500	0.0213	1.22	0.9791
1000	1000	0.0230	1.39	0.9774
1000	1000	0.0288	1.98	0.9716
2000	2000	0.0344	2.53	0.9660
4000	4000	0.0619	5.28	0.9385
8000	8000	0.0982	8.92	0.9022

Innundation Stress (psf):	1000
Innundation Collapse (%):	0.6

	Initial	Final
Sample Mass (g):	145.88	151.72
Water Content (%):	15.78	21.3
Sample Height (h, in):	1.000	0.981
Sample Diameter (D, in):	2.504	2.504
Dry Unit Weight ( $\rho_d$ , pcf):	97.406	97.697
Saturation (%):	60.15	81.79
Void Ratio:	0.695	0.662

Test Notes:



**Minimum Laboratory Soil Resistivity, pH of Soil for Use in Corrosion Testing, Mercuric Thiocyanate**



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**Method for WSC, and USEPA SulfaVer 4 Method for WSS** (AASHTO T 288, T 289, HACH 8113, and 8051)

**Project: Moab Geotechnical Group LLC**

**No: M03898-091 (2025-098)**

Location: **Wellington**

Date: **11/17/2025**

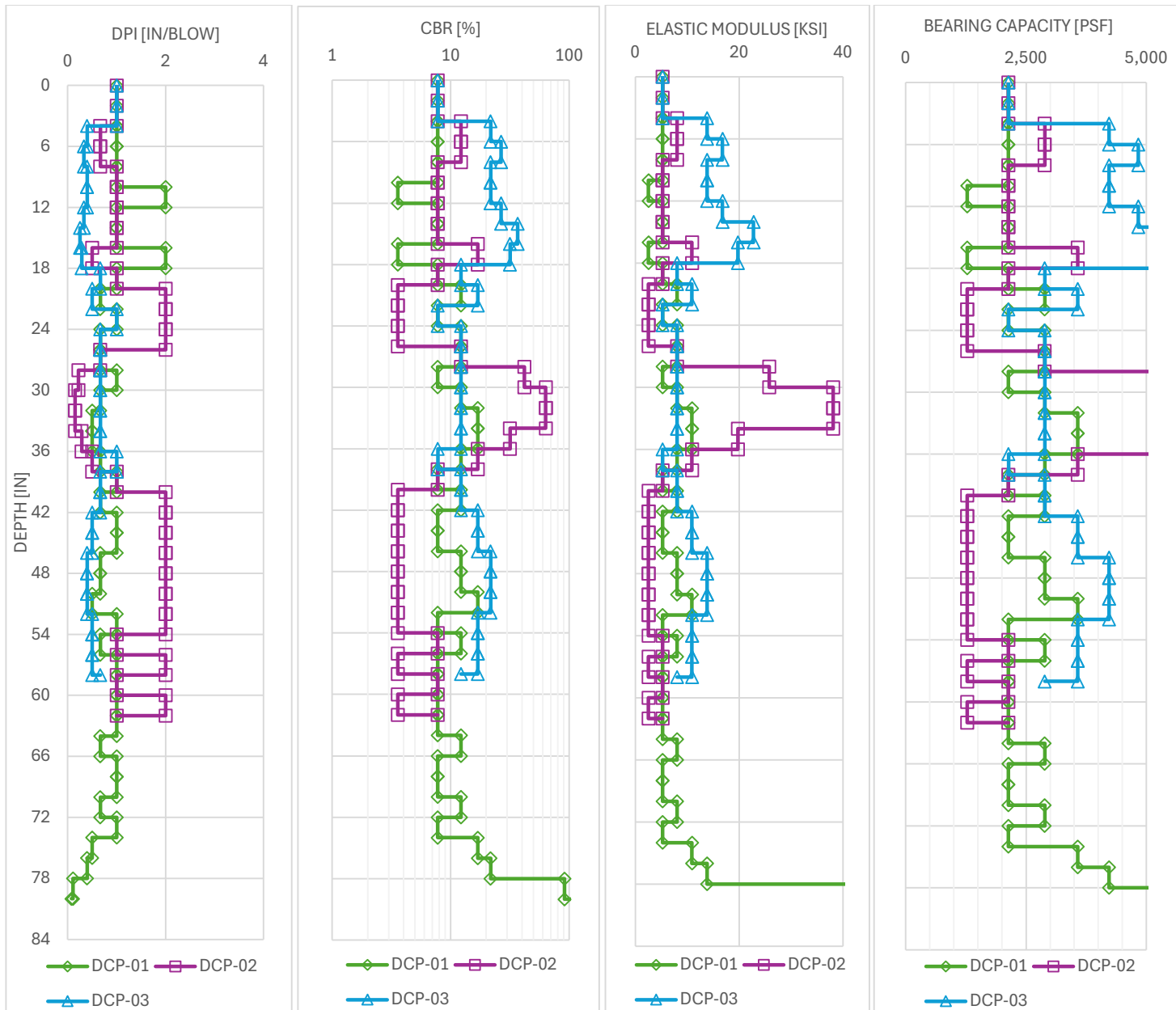
By: **LM**

Sample info.	Boring No.	B-03							
	Sample	Bulk-03							
	Depth	2-4'							
Water content data	Wet soil + tare (g)	39.32							
	Dry soil + tare (g)	37.73							
	Tare (g)	23.59							
	Water content (%)	11.2							
Chem. data	pH	8.3							
	Soluble chloride (ppm)	355							
	Soluble sulfate (ppm)	320							
Resistivity data	Pin method	2							
	Soil box	Miller Small							
		Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)	Approximate Soil condition (%)	Resistance Reading (Ω)	Soil Box Multiplier (cm)	Resistivity (Ω-cm)
		As is	3532	0.67	2366				
		+3	1483	0.67	994				
		+6	1056	0.67	708				
		+9	1021	0.67	684				
		+12	993	0.67	665				
		+15	1008	0.67	675				
		<b>Minimum resistivity (Ω-cm)</b>	<b>665</b>						

Entered by: \_\_\_\_\_

Reviewed: TQH

# APPENDIX C – FIELD TEST RESULTS



WELLINGTON CITY BUILDING  
150 W MAIN STREET  
WELLINGTON, UTAH

**DROP CONE PENETROMETER  
TEST RESULTS &  
CORRELATIONS**

Project No: 2025-098

May 2026

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