Geotechnical Evaluation

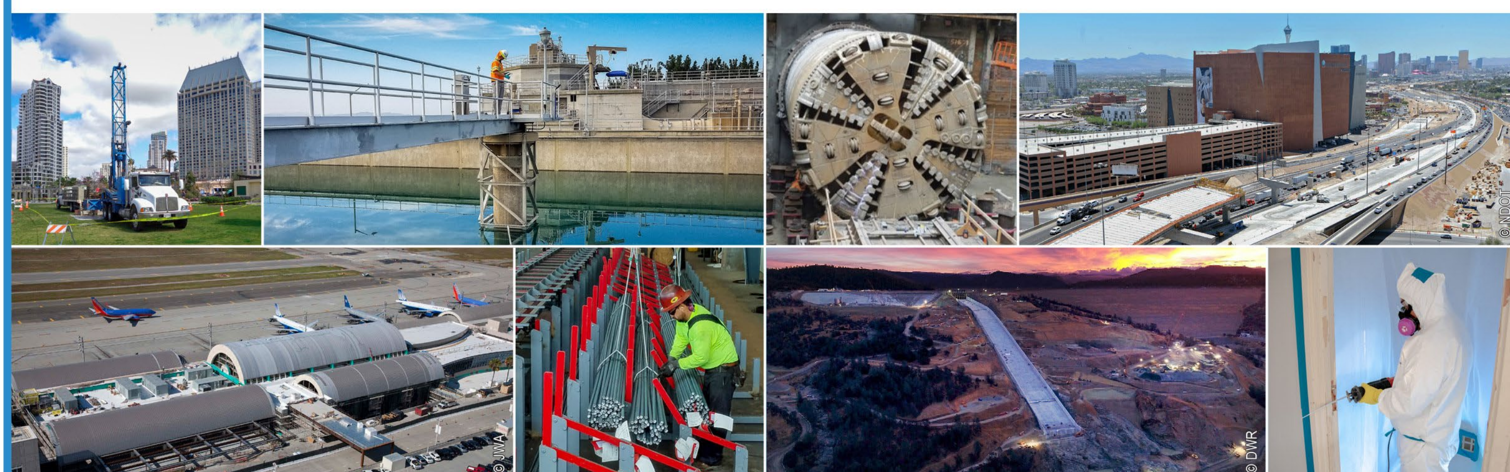
JW Powell Boulevard Extension
Station 200+00 to Station 246+00

Flagstaff, Arizona

Peak Engineering

201 East Birch Avenue, Suite 3 | Flagstaff, Arizona 86001

February 6, 2026 | Project No. 609083001



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Ninyo & Moore

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February 6, 2026
Project No. 609083001

Ms. Julie Leid, PE
Peak Engineering
201 East Birch Avenue, Suite 3
Flagstaff, Arizona 86001

Subject: Geotechnical Evaluation
JW Powell Boulevard Extension
Station 200+00 to Station 246+00
Flagstaff, Arizona

Dear Ms. Leid:

In accordance with our proposal dated May 23, 2025, and your authorization, Ninyo & Moore has performed a geotechnical evaluation for the above-referenced site. The attached report presents our methodology, findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

Dylan C. Walker, PE
Project Engineer

Steven D. Nowaczyk, PE
Managing Principal Engineer

DCW/SDN/amg

Registered Professional Engineer (Civil)
CERTIFICATE NO.
34866
STEVEN D.
NOWACZYK
2/6/2026
Date Signed
ARIZONA U.S.A.

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1. INTRODUCTION

In accordance with our proposal dated May 23, 2025, and your authorization, we have performed a geotechnical evaluation for the proposed JW Powell Boulevard Extension Project to be located in Flagstaff, Arizona (Figure 1). The purpose of our evaluation was to assess the subsurface conditions at the project site in order to provide geotechnical recommendations for design and construction. This report presents the results of our evaluation, and our geotechnical considerations and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

Our scope of services included the following:

- Reviewing readily available geotechnical data, aerial photographs, and published geologic literature, including maps and reports pertaining to the project site and vicinity.
- Conducting a geologic reconnaissance of the site.
- Marking out the boring locations at the project site, securing permission to drill, and notifying Arizona811 of the boring locations prior to drilling.
- Drilling, logging, and sampling 10 exploratory borings to depths of approximately 4 to 30 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Collecting soil samples in the borings at approximately 2.5- and 5.0-foot intervals using ASTM International (ASTM) Methods D1586 (Standard Penetration Test with split-spoon barrel sampling of soils) and D3550 (ring-lined barrel sampling of soils) for laboratory testing and analysis.
- Performing laboratory tests on selected samples obtained from the borings to evaluate in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, and corrosivity characteristics (including pH, minimum electrical resistivity, and soluble sulfate and chloride contents). The in-situ moisture content and dry density results are presented on the boring logs in Appendix A. The remainder of the laboratory test results are presented in Appendix B.
- Performing five seismic refraction surveys at key locations to evaluate the presence of shallow bedrock. The results of the seismic refraction surveys are presented in Appendix C.
- Preparing this report presenting our findings, conclusions, and recommendations regarding the design and construction of the project.

Our scope of services did not include environmental consulting services such as hazardous waste sampling or analytical testing at the site. A detailed scope of services and estimated fee for such services can be provided upon request.

3. SITE DESCRIPTION

The project alignment begins near the northeast end of the existing JW Powell Boulevard alignment; continues to the northeast through existing forest service land; and ends near the southeast end of the existing Flagstaff Urban Trail alignment (Figure 2). At the time of our evaluation, the site consisted of Forest Service land with various utility easements and natural washes and streams. The site was bound by scattered residential properties and Forest Land to the north, south, east, and west.

According to the survey information provided by your office, the site is at an average elevation of roughly 6,811 feet relative to mean sea level at the beginning of the alignment near Station 200+00 and gently slopes upward to the northeast to an average elevation of roughly 6,868 feet relative to mean sea level (MSL) at the end of the alignment near Station 246+00. Based on information from this topographic quadrangle map, the ground surface at the site vicinity gently slopes from the southeast to the northwest.

4. AERIAL PHOTOGRAPH REVIEW

Aerial photographs dated 1964 through 2025 from the Historical Aerials website and Google Earth© were reviewed for this project. A summary of the observations noted for each aerial photograph is presented in Table 1:

Table 1 – Summary of Aerial Photograph Review

Photograph Date(s)	Site	Adjacent Properties	
1964, 1974, 1980, 1981, 1997, 2003, 2005-2007, 2010-2013, 2015-2025	Forest service land with natural washes and utility easements.	North:	Forest service land.
		South:	Forest service land.
		East:	Forest service land. Scattered residential properties depicted as early as 1974.
		West:	Forest service land. Golf course and residential community depicted as early as 2003. JW Boulevard alignment depicted to the south as early as 2005.

5. PROPOSED CONSTRUCTION

The project includes the design and construction of the JW Powell Boulevard Extension project from Station 200+00 to Station 246+00; a distance of about 4,620 linear feet. We understand the improvements will include new pavement, underground utilities, and retaining walls; as well as drainage features such as box culverts. Some site grading (+/- 16 feet vertically) will be needed to support the new roadway. The new underground utilities are assumed to be situated within or near the above-mentioned roadway improvement alignment and will extend 20 or less feet deep. In addition, we assume positive drainage will be established during and after construction at the site.

6. FIELD EXPLORATION AND LABORATORY TESTING

The following sections summarize our field exploration and laboratory testing activities.

6.1. Soil Borings

On September 8 and 9, 2025, Ninyo & Moore conducted a subsurface exploration at the site in order to evaluate the subsurface conditions and to collect soil samples for laboratory testing. Our evaluation consisted of drilling, logging, and sampling of 10 small-diameter borings using a CME-75 truck-mounted drill rig equipped with hollow-stem augers. The borings, denoted as B-1 through B-10, extended to depths of approximately 4 to 30 feet bgs (Figure 3A and 3B). Auger refusal was encountered within several of our borings before achieving their target drill depths. Descriptions of the soils encountered are presented in the boring logs in Appendix A.

Soil samples were collected at selected intervals and were logged in general accordance with the ASTM D2488. Disturbed soil samples were collected during standard penetration testing using a split-spoon sampler. Relatively undisturbed soil samples were collected at regular intervals by using modified ring-lined split tube samplers. Bulk samples were also collected from the drill cuttings and placed in large plastic bags. The selected intervals at which the bulk soil samples were collected are provided on the boring logs. Descriptions of the soils encountered in our borings are presented on the boring logs.

6.2. Laboratory Testing

The soil samples collected from our drilling activities were transported to the Ninyo & Moore laboratory in Phoenix, Arizona. In addition, Ninyo & Moore performed laboratory tests on selected samples obtained from the borings to evaluate the in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, corrosivity characteristics (including pH, minimum electrical resistivity, and soluble sulfate and chloride contents). The in-situ moisture content and dry density results are presented on the boring logs in Appendix A. A description of the laboratory testing as well as the remainder of the laboratory test results are presented in Appendix B.

6.3. Seismic Refraction

On September 8 and 9, 2025, Ninyo & Moore personnel collected refraction data along five seismic lines denoted as SL-1 through SL-5. The refraction data was evaluated in order to better assess the presence of shallow bedrock. Each line was approximately 230 feet long with geophones spaced every 10 feet. Each survey line was oriented roughly parallel to the proposed roadway alignment. The locations of the survey lines are shown on Figures 3A and 3B.

The seismic data were collected using a 24-Channel Geometrics Geode, exploration seismograph coupled with 24 vertical components, and 14 Hertz geophones. A 10-pound hammer and aluminum plate were used as the seismic wave source for the refraction surveys. Field data acquisition included stacking multiple shots at each location in order to increase the quality of the data and reduce noise. The seismic refraction method uses recognition of first-arrival times of refracted waves in units of milliseconds to evaluate the thickness and seismic velocities of subsurface layers. Seismic waves generated by the hammer impacting the ground surface at a given “shot” point are refracted at boundaries separating materials of contrasting material velocities. These refracted seismic waves are then detected by a series of surface geophones and recorded with a seismograph. Each hammer shot is recorded as time zero. The elapsed time, in milliseconds, that the seismic compressional wave (P-wave) signals take to travel to each geophone is recorded through the record length. This information is used in conjunction with the known shot-to-geophone horizontal distances to obtain the approximate thickness and velocity information about the subsurface materials.

The obtained refraction data were processed using SeisImager/2DTM, Version 3.3, which includes a suite of programs by Geometrics, Inc. Initially, data were grouped as shot points in each line and first arrival picks were made manually in Pickwin v. 5.1.1.2. Once data were grouped as first arrival picks, a travel time curve was constructed and calculations were performed to derive approximate velocities. Relative topography data collected in the field were incorporated with travel time data to account for topography effects. An initial velocity model of the surface was developed for each profile using a delay-time technique and velocity inversion in Plotrefa TM v. 3.0.0.6. These models were used for each profile to develop a more detailed tomographic profile depicting approximate lateral and vertical changes (discontinuities) in P-wave velocity across each seismic line. The results of our seismic surveys are presented in Appendix C.

7. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

7.1. Geologic Setting

The project area is located within the High Plateaus Section of the Colorado Plateau physiographic province, which encompasses some parts of northern and central Arizona. In the Paleozoic, Arizona was part of a low-lying platform, which was periodically inundated by seawater. During this time, layers of limestone, shale, and sandstone were deposited. The plateau region was uplifted and there was an increase in volcanic activity during the Cenozoic. Currently, gently warped and faulted sedimentary Paleozoic rocks, Mesozoic rocks, and Cenozoic volcanic rocks characterize this province.

The surficial geology of the site is described as Permian Sedimentary Rock Late and Middle Pleistocene (270 to 280 Ma). The United States Department of Agriculture Web Soil Survey described the site as generally consisting of Jacques clay loam, Lynx loam, Daze fine sandy loam, Tortugas cobbly loam, Tortugas-Daze complex, Telephone-Daze complex, and Amos fine sandy loam. Loam is an agriculture soil classification that refers to a soil comprised of a mixture of clay, silt, and sand. Historical data has also shown that the materials in the upper five feet of soil have primarily comprised of clayey sands, silty sands, lean clays, fat clays, silty clays, and silt.

7.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on the results of our exploratory borings and our understanding of the general geology of the area. The boring logs contains our field test results, as well as our interpretation of the conditions likely to exist between actual samples retrieved. Therefore, the boring logs contains both factual and interpretive information. Lines delineating subsurface strata on the boring logs are intended to group soils having similar engineering properties and characteristics. They should be considered approximate, as the actual transition between soil types may be gradual. Detailed stratigraphic information as well as a key to the soil symbols and terms used on the boring logs are provided in Appendix A.

7.2.1. Alluvium

Native alluvial soils were encountered at the surface of each of our borings and extended to the boring termination depths. In our borings, the alluvium generally consisted of loose to very dense poorly-graded gravels (GP), clayey sands (SC), silty sands (SM), silty, clayey sands (SC-SM), and firm to hard silty clays (CL-ML), lean clays (CL), and silts (ML). Varying quantities of gravel, scattered caliche nodules, cobbles, possible boulders, and bedrock were also observed in our borings. Auger refusal was encountered within this alluvium layer at several of our borings before achieving their target drill depths.

7.2.2. Groundwater

Groundwater was not encountered in our borings. Based on well data from the Arizona Department of Water Resources, the depth to groundwater has historically been recorded as shallow as about 31 feet bgs. Groundwater levels can fluctuate due to seasonal variations, irrigation, groundwater withdrawal or injection, and other factors.

8. GEOLOGIC HAZARDS

The following sections describe regional geologic hazards, including land subsidence, earth fissures, and faults, and collapsible soils.

8.1. Land Subsidence and Earth Fissures

Based on our field reconnaissance and review of the referenced material, there are no known or exposed earth-fissures documented in the Transition Zone province. Land subsidence and earth fissures are not considered to be a constraint to development on this project site.

8.2. Faulting and Seismicity

The site lies within the Northern Arizona Seismic Belt, which is a northwesterly trending area of seismicity, beginning southeast of Flagstaff and trending up through the Grand Canyon to the Arizona-Utah border (Bausch and Brumbaugh, 1997). Seismicity in the vicinity of Flagstaff is generally produced by a group of northwest trending high angle faults comprising the Cataract Creek fault system. This system of faults extends from the Mogollon Plateau to the western Grand Canyon.

The closest documented fault to the site is the Lake Mary Fault Zone, located less than a mile south of the site. This zone is a northwest- to north-trending escarpment formed on Paleozoic bedrock and uppermost Miocene basalt defines the east side of a fairly narrow, asymmetric graben. The trough bottom is covered by late Quaternary alluvium deposited in lakes or marshes, and locally, upper Pliocene basalt. The escarpment slopes are moderately steep suggesting Quaternary fault activity (Pearthree, 1998). This fault zone has exhibited displacement within the past 130,000 years. The slip-rate category of this fault zone is less than 0.2 millimeters per year (Pearthree, 1997). Seismic design considerations are provided in Section 9.2.

9. GEOTECHNICAL CONSIDERATIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations of this report are incorporated into the design and construction of the proposed project, as appropriate. Geotechnical considerations include the following:

- Due to the widely spaced nature of our borings, soil and bedrock conditions that differ from what was encountered in our borings may be encountered during construction.
- The relatively granular and cohesive alluvial soils present on site are prone to consolidation or mat be compressible. These materials are not considered suitable to provide foundation support to the proposed new improvements.

- Earthwork contractors should be made aware of the moisture sensitivity of the near surface clayey and silty soils and potential compaction difficulties.

- The near-surface on-site materials are considered generally excavatable with heavy-duty earthmoving equipment. Varying quantities of gravel, scattered caliche nodules, cobbles, possible boulders, and bedrock were also observed in our borings were encountered in our borings at various depths, which could be more difficult to excavate during construction.

- Conventional earthmoving construction equipment may be used. However, if bedrock is encountered, additional measures may be needed (e.g., blasting, pneumatic hammering, rock teeth for ripping, rock saws, etc.).

- Earthwork cut/fill transitions are expected within the roadway alignment. Settlement sensitive features should be supported solely on either engineered fill soils or competent bedrock, not both.

- Settlement due to the self-weight of deep grade-raised fills can be about 1 percent of the fill height.

- Imported soils and soils generated from on-site excavation activities that exhibit relatively low plasticity and low organic contents can generally be used as engineered fill. On the basis of our limited evaluation, some of the on-site soils are not considered to be suitable for re-use as engineered fill when supporting settlement sensitive features.

- The new structures may be supported on shallow spread footings proportioned for moderate bearing pressures on a zone of engineered fill.

- Groundwater was not observed in our borings. The static groundwater table is anticipated to be approximately 31 feet bgs based on the nearby well data. In general, groundwater is not anticipated to be a constraint to the construction of the project.

- No known or reported geologic hazards are present underlying, or immediately adjacent to the site.

- Corrosivity test results indicate that subgrade soils at the site are corrosive to ferrous metals, and the sulfate content of the soils present a negligible sulfate exposure to concrete.

10. RECOMMENDATIONS

The following sections present our geotechnical recommendations and were developed based on our understanding of the proposed construction (Section 5), the observed subsurface conditions (Section 7), and our experience. Given the project location, recommendations and guidelines outlined by the City of Flagstaff Engineering Design Standards and Specifications for New Infrastructure, Maricopa Association of Governments (MAG), and/or any local requirements should be used unless recommended differently herein. If the proposed construction is changed from that discussed herein or subsurface conditions other than those shown on the boring logs (Appendix A) are observed at the time of construction, Ninyo & Moore should be retained to conduct a review of the new information and to evaluate the need for additional recommendations.

10.1. Earthwork

The following sections provide our earthwork recommendations for this project. If the site grade is planned to change by more than 5 feet vertically, Ninyo & Moore should be contacted for additional recommendations.

10.1.1. Site Preparation

Prior to placing any fill, pavement, or flatwork, the following guidelines should be followed:

- Obstructions that extend below finish grade, if any, should be removed and the resulting holes filled with compacted soil.
- After stripping, clearing, grubbing, and root raking is performed and prior to placement of any fill soils, the exposed subgrade should be evaluated by proof-rolling. Any soft or weak areas observed during the proof-rolling process should be removed and replaced with compacted material as outlined in Section 10.1.14.
- Proof-rolling should be accomplished with a pneumatic-tired roller, a loaded dump truck, or similar equipment weighing approximately 20 tons and observed by the Geotechnical Engineer-of-Record, or the Engineer's designated representative.

Due to the clayey and silty nature of the surficial soils, traffic of heavy equipment (including heavy compaction equipment) may create pumping and general deterioration of shallow soils. Therefore, some construction difficulties should be anticipated, especially during periods when these soils are wet.

10.1.2. Wet Weather Conditions

Earthwork contractors should be made aware of the moisture sensitivity of the near surface clayey and silty soils and potential compaction difficulties. If construction is undertaken during wet weather conditions, the surficial soils may become saturated, soft, and unworkable. Therefore, we recommend that consideration be given to construction during the dryer months and positive drainage be established and maintained during construction.

10.1.3. Subgrade Improvement

We recommend that new shallow foundations (with anticipated loading not in excess of 2,000 pounds per square foot [psf]) and mat foundations (with anticipated loading not in excess of 2,000 psf), needed to support new drainage structures, be supported on a zone of engineered fill that extends 3 feet below the bottom of the foundations. The engineered fill should be placed as discussed in this report. This overexcavation zone should extend a horizontal distance from the edge of the new foundation that is equal to the depth of the overexcavation.

In addition, we recommend that the grade-raised fill segments that are thicker than 5 vertical feet, flatwork, and pavements be supported on 12 inches of moisture-conditioned and compacted engineered fill. This can be achieved by in-place scarification and re-compaction. The fill thickness should be measured from the bottom of the AB layer, where applicable. This subgrade improvement should extend laterally 1 foot beyond the flatwork and pavement footprint.

Furthermore, engineered fill to be placed with 3 vertical feet of the new roadway subgrade should meet the design R-value of 35 as recommended in this report. We recommend that the construction control R-value be not less than the design R-value of 35.

After the overexcavation described above is finished and prior to the placement of engineered fill, exposed surfaces from excavations should be carefully evaluated by the geotechnical consultant for the presence of soft, loose, or wet soils that were not removed as part of the improvement process. This evaluation should consist of probing and visual observation of the excavation bottom. Based on this evaluation, additional remediation may be needed. This could include further scarification, moisture-conditioning and compaction of the exposed surface. This additional remediation, if needed, should be addressed by the geotechnical consultant during the earthwork operations.

10.1.4. Excavations and Temporary Slopes

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory boring, site observations, and experience with similar materials. Excavation of the materials can generally be accomplished with heavy-duty earthmoving equipment. However; varying quantities of gravel, scattered caliche nodules, cobbles, possible boulders, and bedrock were also observed in our borings and may be more difficult to excavate and/or slow the rate of excavation and may call for more aggressive rock-removal excavation techniques (i.e., may include blasting, tock sawing, pneumatic hammering, heavy duty hoe ram, etc.).

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system in compliance with OSHA Regulations for employees working in an excavation that may expose them to the danger of moving ground. Based on the soil conditions at the site, we recommend that OSHA Soil “Type C” classification be used for excavations at the site. This corresponds to temporary slopes of 1.5:1 (horizontal: vertical). This side slope is for excavations that are less than 20 feet deep. If material is stored or equipment is operated near an excavation, stronger shoring should be used to resist the extra pressure due to superimposed loads.

If the proposed construction extends deeper than the extent of our test boring in any part of this project, Ninyo & Moore should be contacted for additional consultation and possible further evaluation of the subsurface materials.

10.1.5. Permanent Slopes

Permanent cut slopes and constructed embankment fill slopes should be no steeper than 2:1 (horizontal to vertical). New embankment fills should be benched into existing embankments, where appropriate. Benches should be level and wide enough to for the allow operation of, and compaction by, construction equipment. Benches should be no steeper than 1:1 (horizontal to vertical). Fill slopes should be constructed in a manner (e.g., overfilling and cutting to grade) such that the recommended degree of compaction is achieved to the finished slope face. Cut and fill slopes should be protected from erosion. This should promote re-vegetation and a stable slope. Periodic maintenance of exposed slopes should be anticipated.

Unprotected slopes may rill and erode if exposed to running water. Silty soils and soils containing fine sand are more susceptible in this regard. While 2:1 (horizontal to vertical) slopes are acceptable from a stability standpoint, laying slopes back to 3:1 (horizontal to vertical) will decrease runoff velocity and decrease the likelihood of serious erosion. Steeper slopes will need additional maintenance. Adequate drainage and temporary erosion protection covering could minimize erosion problems and promote post-construction vegetation. Plating the slopes with gravelly material or riprap will reduce the impacts of precipitation and slow the rate of erosion. If riprap is placed in the channel, it should be adequately sized to prevent erosion of the embankment. Along longer slopes, brow ditches should be considered to reduce the amount of surface flow on the slope face. Where feasible, the existing vegetation should be salvaged and replaced.

10.1.6. Temporary Shoring

In some instances, it may be preferable to temporarily brace or shore the excavations rather than using open cuts to the base of the excavations. Temporary earth retaining systems will be subjected to lateral loads resulting from earth pressures. Shored excavations may be designed using the parameters on Figures 3 and 4.

The earth pressure values presented on Figures 3 and 4 assume that spoils from the excavation or other surcharge loads will not be placed above the excavation within a 1:1 (H:V) plane extending up and back from the base of the excavation. If spoil piles are placed closer than this to the braced or shored excavation, the resulting surcharge loads should be considered in the bracing or shoring design. We recommend that an experienced structural engineer design the bracing or shoring system. The bracing and shoring parameters presented in this report should be considered as guidelines.

Trench boxes may also be a suitable alternative to laying back the side walls; however, due to the presence of granular soils, the excavations may not stand open long enough to install the trench boxes. The contractor should be prepared to deal with these soil conditions and plan accordingly. Once installed, some sloughing is possible at the ends of the trench box; therefore, any loose material should be removed prior to backfilling of the trench.

10.1.7. Bottom Stability

Bottom of the excavations should be stable for the purpose of the planned construction. However, if excavations are open during a heavy rain event, the bottom of the excavation may become saturated and unstable. Dewatering as discussed in Section 10.1.8 below may be anticipated in such events.

10.1.8. Construction Dewatering

Excavations that encounter seepage or surface run-off could be dewatered by pumping the water out and away from the excavation. Discharge of water from the excavations to natural drainage channels, if needed, may entail securing a special permit.

10.1.9. Fill Materials

On-site and imported soils that exhibit relatively low plasticity indices are generally suitable for re-use as engineered fill. Relatively low plasticity indices, as evaluated by ASTM D4318, are defined as a plasticity index (PI) of 15 or less for this project.

In addition, suitable fill should not include construction debris, organic material, or other non-soil fill materials. Clay lumps and rock particles should not be larger than 4 inches in dimension. Unsuitable fill material should be disposed of off-site or in non-structural areas.

Imported fill, if used, should consist of soils with a relatively low PI (15 or less). Import material in contact with ferrous metals should preferably have low corrosion potential (minimum resistivity more than 2,000 ohm-cm, chloride content less than 25 parts per million [ppm]). In lieu of this, corrosion protection techniques (e.g., cathodic protection, pipe wrapping, etc.) can be implemented. A corrosion specialist should be consulted for recommendations of an appropriate corrosion protection technique. Imported material in contact with concrete should have a soluble sulfate content of less than 0.1 percent. The geotechnical consultant should evaluate such materials and details of their placement prior to importation.

10.1.10. Re-use of On-Site Soils

The Atterberg limits tests performed on soil samples obtained from our borings resulted in PI values ranging between non-plastic (zero) and 20. Based on our test results, some of the on-site soils are not considered suitable for re-use as engineered fill for this project when situated within 3 feet vertically of settlement sensitive features; however, may be used if deeper than 3 feet vertically from settlement sensitive features or within non-structural areas (e.g., landscaping). Additional field sampling and laboratory testing should be conducted by the contractor during construction to better screen for any unsuitable materials.

10.1.11. Pipe Bedding

We recommend standard utility pipelines be supported on a minimum of 4 inches or more of granular bedding material such as sand and gravel, or crushed rock meeting MAG Section 702 Standard Specifications (pea gravel or crushed chips are not acceptable). Care should be taken not to allow voids to form beneath the pipe (i.e., the pipe haunches should be continuously supported) to avoid damaging the pipeline. Bedding material and compaction requirements should be in accordance with the recommendations in this report, as well as the MAG Standard Specifications. Pipe bedding details are presented on Figure 6.

10.1.12. Trench Backfill

Backfilling should generally be accomplished in a manner consistent with the MAG Section 601. The onsite soils are generally suitable for reuse as trench backfill provided, they are free of organic material, clay lumps, debris, and rocks greater than 4 inches in diameter. Some screening of larger particles may be needed, to meet the MAG specifications. Imported fill, if utilized, should meet the requirements presenting in this report. Trench backfill details are presented on Figure 6.

10.1.13. Controlled Low Strength Material

As an alternative to using soil backfill, some backfill zones may be filled with controlled low strength material (CLSM). CLSM consists of a fluid, workable mixture of aggregate, Portland cement, and water. The use of CLSM has some advantages:

- A narrower backfill zone can be used, thereby minimizing the quantity of soil to be excavated and possibly reducing disturbance to the near-by structures.
- Relatively higher modulus of soil reactions values may be used.
- The support given to the connecting pipes is generally better.
- Because little compaction is needed to place CLSM, there is less risk of damaging the connecting pipes.

- CLSM can be batched to flow into irregularities in the trench bottom and walls.

The CLSM design mix should be in accordance with the MAG specifications. Additional mix design information can be provided upon request.

Buoyant or uplift forces on the piping should be considered when using CLSM and prudent construction techniques.

10.1.14. Engineered Fill Placement and Compaction

Engineered fill soils should be moisture-conditioned within the moisture range shown below in Table 2 and mechanically compacted to the percent compaction shown. Engineered fill should generally be placed in 8-inch-thick loose lifts such that each lift is firm and non-yielding under the weight of construction equipment.

Engineered fill used to raise grade will settle a portion of its height due to its own weight regardless of compaction effort. The magnitude of this settlement will depend on the type of fill used. In general, the engineered fill recommended in this report is expected to settle about 1 percent of its height.

Table 2 – Compaction Recommendations

Engineered Fill Description	Percent Compaction per ASTM D698	Moisture Content
Below raise grade fill, foundations, grade slabs, pavements, and flatwork	95 percent	±2 percent of optimum
Aggregate Base (AB) below areas not subject to traffic	95 percent	±3 percent of optimum
AB below areas subject to traffic	100 percent	±3 percent of optimum
Granular Trench Backfill – Within 2 feet below pavements	100 percent	±3 percent of optimum
Non-Granular* Trench Backfill – Within 2 feet below pavement	95 percent	±3 percent of optimum
Trench Backfill – Deeper than 2 feet below pavement	95 percent	

An earthwork (shrinkage) factor of 10 to 20 percent is estimated. This shrinkage factor range represents an average of the material tested and assumes that materials excavated from the site will be placed as fill. Potential bidders should consider this in preparing estimates and should review the available data to make their own conclusions regarding excavation conditions.

10.1.15. Site Drainage

The long-term performance of the foundation system depends, in part, on maintaining positive surface drainage during the life of the structure. Adequate drainage should be provided to reduce variations in the moisture content of foundation soils. Finished grade within 5 feet of the structure should be adjusted to slope away from the structure at a slope of 2 percent, or more.

Landscaping, including planters, should not be placed in close proximity to the perimeter of the structures. Irrigation associated with landscaping may introduce significant amounts of water to the foundation soils.

Significant moisture variation within expansive soils may induce soil-related movements that can adversely affect the performance of ground-supported foundations or slabs. The following recommendations are intended to aid in maintaining relatively stable moisture conditions for the foundation soils, thus reducing the risk for significant expansive soil-related movement of shallow foundations:

- Post-construction movement of pavement and other flatwork is common and should be anticipated. Normal maintenance should include evaluation of paving and sidewalk joints, etc. and resealing where needed.
- Surface paving of exterior areas adjacent to the structures should be installed to provide an adequate seal to prohibit moisture migration into foundation soils.
- Provide positive drainage away from the planned structures to prohibit water ponding around foundations.
- Downspouts should drain 5 feet or more away from the structures. If possible, they should drain into closed conduits and be routed to suitable discharge facilities.
- A relatively impervious material should be used as backfill for utility trenches within a zone that extends laterally from the inside of the perimeter of the structures, to 5 feet outside of the proposed structures. Relatively pervious materials (i.e., sands and gravels) used as backfill may provide an avenue for water to migrate along utility trenches and should be avoided.
- Landscaping should not be placed in close proximity to the perimeter of the structures. Irrigation associated with landscaping may introduce significant amounts of water to the foundation soils.
- Trees and shrubs should not be placed closer to the foundations than a horizontal distance of about half of their mature height. Depending on their nature, trees are capable of removing significant amounts of water from the foundation soils, resulting in soil movement (shrinkage), particularly during periods of low rainfall.
- If trees are to be placed near the proposed structures, we recommend installing root barriers to aid in keeping the root system away from foundation elements.

- Ponding of water in planters, in unpaved areas, and/or around joints in paving and sidewalks can cause slab movements beyond those predicted in this report. Regular maintenance should include re-grading and/or repair of areas where ponding occurs.

10.2. Seismic Design Parameters

Design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 3 presents the seismic design parameters for the site in accordance with the International Building Code (IBC) guidelines and adjusted maximum considered earthquake spectral response acceleration parameters evaluated using the USGS ground motion calculator (web-based).

Table 3 – IBC Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.55
Site Coefficient, F_v	2.4
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	0.312 g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.096 g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.484 g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.230 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.322 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.153 g

10.3. Seismic Refraction Survey

In order to help evaluate the approximate depth of each significant subsurface layer, Ninyo & Moore performed a seismic refraction survey denoted as SL-1 through SL-5. Each transect was approximately 230 feet in length with geophones spaced 10 feet apart and they gathered data to depths of approximately 40 feet below ground surface. The following sections describe our seismic refraction survey findings and conclusions.

10.3.1. Seismic Velocities and Rippability Correlations

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass for each detected layer. Possible areas of differing composition, texture, or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator. Table 4 outlines the general relationship between P-wave velocity and rippability.

The rippability characteristics in Table 4 are based on our experience with similar materials. It assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that material characteristics can play a significant role in estimating excavation rates and rippability.

Table 4 – P-Wave Velocity and Rippability Correlation

P-Wave Velocity Range Feet/Second	Rippability
0 to 2,000 ft/s	Easy Ripping
2,000 to 4,000 ft/s	Moderate Ripping
4,000 to 5,500 ft/s	Difficult Ripping, Possible Blasting
5,500 to 7,000 ft/s	Very Difficult Ripping, Probable Blasting
Greater than 7,000 ft/s	Blasting Generally Needed

It should be noted that the rippability estimates presented in Table 4 are slightly more conservative than those published in Edition 41 of the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids. It should also be noted that, as a general rule of thumb, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the refraction line.

10.3.2. Seismic Refraction Results

Table 5 below summarizes the results of the seismic refraction survey. Graphical representations of these results are presented in Appendix C.

Table 5 – Seismic Refraction Survey Results

Location	Depth	Estimated Compression Wave Velocities and Materials
SL-1	0-3 feet	1,500 to 2,500 ft/sec, medium dense sands.
	3-6 feet	2,500 to 4,000 ft/sec, very dense soil to moderately hard rock.
	> 6 feet	4,500 to 5,500 ft/sec, moderately hard rock.
SL-2	0-20 feet	1,000 to 3,500 ft/sec, stiff to very stiff soils.
	>20 feet	3,500 to 4,250 ft/sec, very stiff soils to moderately hard limestone.
SL-3	0-3 feet	1,500 to 2,500 ft/sec, very dense soils.
	3-8 feet	3,000 to 4,000 ft/sec very dense soils to moderately hard rock.

	8-12 feet	4,000 to 7,000 ft/sec, moderately hard to hard rock.
	> 12 feet	> 7,000 ft/sec, hard rock.
SL-4	0-3 feet	2,000 to 5,000 ft/sec, hard soils to moderately hard rock.
	3-6 feet	5,000 to 6,500 ft/sec, moderately hard to hard rock.
	> 6 feet	> 6,500 ft/sec, moderately hard to hard rock.
SL-5	0-4 feet	1,500 to 2,500 ft/sec, hard soils.
	> 4 feet	2,500 to 3,500 ft/sec, hard soils to moderately hard rock.

10.4. Foundations

Based on the results of the field and laboratory evaluations, it is our opinion that the proposed drainage structures can be founded on spread footings or mat foundations. Recommendations for these foundation systems are presented in the following sections of this report.

10.4.1. Spread Footings

Spread or continuous footings, if utilized, should be supported at a depth of 30 inches or more below the adjacent finished grade. The footings should be supported on engineered fill, as described in this report. Continuous footings should have a width of 18 or more inches, and isolated spread footings should have a width of 24 or more inches. Spread or continuous footings should be reinforced in accordance with the recommendations of the structural engineer.

Based on the available soil boring information, spread footings associated with the perimeter walls, and light structures supported on engineered fill may be designed using a net allowable bearing capacity of 2,000 psf for static conditions.

Total and differential settlement of up to about 1 inch and ½ inch respectively, may occur.

These settlement estimates are based on the assumption that the foundations act as isolated foundations, that is, the clear spacing between the foundation elements are the width of the largest adjacent foundation or more, and the settlement associated with fill soils has already occurred.

Foundations bearing on moisture-conditioned, compacted engineered fill that are subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.35 (total frictional resistance equivalent to the coefficient of friction multiplied by the dead load). A passive resistance value of 255 psf per foot of depth for drained conditions. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided that the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces. The foundations should preferably be proportioned such that the resultant force from lateral loadings falls within the kern (i.e., middle one-third).

10.4.2. Mat Foundations

A mat foundation may be utilized for proposed box culvert and should bear at a depth of 30 inches or more below the adjacent finished grade, on a layer of compacted engineered fill, as described in this report. Mat foundations should be reinforced in accordance with the recommendations of the structural engineer.

Mat foundations founded on engineered fill may be designed using a net allowable bearing capacity of 2,000 psf for static conditions. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Total and differential settlements beneath mat slab foundations will vary depending on slab configuration and applied load. A modulus of subgrade reaction of 200 kips per cubic foot (kcf) may be used for design of mats supported on compacted engineered fill to estimate the settlement. This value is an estimate based on published typical ranges, results of laboratory testing performed on samples taken from our borings, and our experience with similar materials. If more refined estimates are needed, field plate load testing should be performed in the areas proposed for the new mats.

Mat foundations bearing on compacted engineered fill and subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.40 (total frictional resistance equivalent to the coefficient of friction multiplied by the dead load).

10.5. Retaining Walls

Retaining walls should be supported on shallow spread footings as discussed in Section 10.4. Drainage should consist of free-draining granular material and should be accompanied by weepholes through the wall or corrugated, perforated pipe placed parallel to the wall or abutment bottom, wrapped in a filter fabric, and surrounded by 6 inches or more of granular filter material (e.g., pea gravel). In lieu of the wrapped open-graded gravel, a geocomposite drainage mat attached to the wall and discharging to a drain pipe or weepholes may be considered.

10.5.1. Active Conditions

Active earth pressure occurs when the wall moves away from the soils and the soil mass stretches horizontally, sufficient to mobilize its shear strength, and a condition of plastic equilibrium is reached. For a drained granular backfill, an equivalent fluid active earth pressure of 45 psf per foot (psf/ft) of wall height should be used for design of cantilevered, yielding walls. For undrained conditions, a buoyant active earth pressure of 90 psf per foot (psf/ft) of wall height should be used. An outward lateral movement of about 0.001H (where H is the height of the wall) at the top of the wall is generally needed to mobilized the active earth pressure condition.

Unrestrained retaining walls should also be designed to resist a horizontal earth pressure of 0.33q. The value for “q” represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

10.5.2. At Rest Conditions

A soil mass that is neither stretched nor compressed is said to be in an at-rest state. If the wall is rigidly restrained, so that it does not rotate sufficiently to reach the active earth pressure condition, at-rest earth pressure condition will exist. An equivalent fluid at-rest earth pressure of 67 psf/ft for drained conditions and 102 psf/ft for undrained conditions should be used. Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.5q. The value for “q” represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

10.5.3. Passive Conditions

Passive earth pressure occurs when the wall or foundation moves into the soil and the soil mass is compressed horizontally, mobilizing its shear strength. For below-grade portions of the walls with granular backfill (derived from on-site soils) in front of the toe of the wall, an ultimate equivalent fluid passive earth pressure of 255 psf/ft can be used for drained conditions. An ultimate equivalent fluid passive pressure of 205 psf/ft can be used for undrained conditions. This value assumes that the ground is horizontal for a distance of 10 feet or more in front of the wall or three times the height generating the passive pressure, whichever is more. We recommend that the upper 12 inches of soil not protected by pavement or a concrete slab, or any soil subject to possible future scour or excavation, be neglected when calculating passive resistance.

10.6. Pavements

The pavement sections recommended in this report are generally based on the pavement design procedures outlined in the *ADOT Pavement Design Manual* (2017) and referred to as the Arizona Department of Transportation (ADOT) Manual in the follow sections. In addition, the City of Flagstaff Engineering Standards were considered. The following section provides information on estimated traffic loading, subgrade support, and pavement design parameters.

10.6.1. Traffic Volumes and Growth Rates

One way traffic loading information for the proposed arterial alignment obtained from PEAK Engineering was 5,500 vehicles per day. A truck percentage and a growth rate percentage of 4- and 0.1- percent was also assumed, respectively, over the design life of 20 years.

The configuration of the roadway will generally remain two lanes in each direction. Therefore, a lane distribution factor (D_L) of 90 percent, or 0.90 was selected from Table A-2 in the ADOT Manual.

Based on these assumptions, and using the American Association of State Highway and Transportation Officials procedure for pavement design, the one-way total flexible pavement equivalent single axle load was calculated to be 1,780,000.

10.6.2. Design R-Value

Correlated R-values were estimated using the procedure described in the Arizona Department of Transportation (ADOT) Preliminary Engineering and Design Manual (PE&DM). Based on the procedure outlined in the PE&DM, we estimate an R-Value of 35. Imported fills, if used, as described above in this report, should exhibit an R-value that is equivalent or more.

10.6.3. Resilient Modulus

A design R-value of 35 was utilized in our analysis of the new pavement section to be constructed over prepared subgrade. A seasonal variation factor (SVF) of 3.5 was selected for the design from Figure 2-1 in the ADOT Manual. Using these inputs, a resilient modulus of 9,927 pounds per square inch was calculated. In addition, shallow bedrock may be encountered as outlined in Section 7.2. In these areas, a resilient modulus of 26,000 pounds per square inch was used.

10.6.4. Drainage Coefficient

A drainage coefficient of 0.84 was selected for use in our design from Table 2-7 in the ADOT Manual based on the anticipated quality of drainage and the SVF.

10.6.5. Standard Deviation and Level of Reliability

ADOT recommends a combine standard error value (S_0) of 0.45 for flexible pavement design. Based on ADOT standards, a level of reliability of 90 percent was used for roadways with a traffic volume between 2,001 to 10,000 ADT. A standard normal deviate (ZR) value of -1.282 was selected from Table 2-1 in the ADOT Manual and utilized in our design.

10.6.6. Serviceability Index

An initial serviceability of 4.1 and a terminal serviceability of 2.6 were used for roadways with a traffic volume between 2,001 to 10,000 ADT. The resulting serviceability index loss is 1.5.

10.6.7. Flexible Pavement - Structural Coefficient and Minimum Thickness

Table 2-6 in the ADOT Manual provides a list of structural coefficients based on material type. For our design, a structural coefficient of 0.44 was used for AC, a structural coefficient of 0.14 was used for AB, and a structural coefficient of 0.11 was used for subbase material. Based on Section 2.1.7 of the ADOT Design Manual, subbase material is defined as having a tested or correlated R-value of 73 or greater.

Table 2-10 in the ADOT Manual provides minimum structural numbers (SNs) and minimum AC thicknesses base on roadway functional classifications. As a roadway with a traffic volume between 2,001 to 10,000 ADT, JW Boulevard pavements should have a SN of 2.75 or more with a minimum AC thickness of 5.0 inches.

10.6.8. Recommendations

Utilizing a design R-Value of 35 and the anticipated traffic over 20 years, a minimum SN of 3.52 was calculated. In situations where shallow bedrock is encountered, a minimum SN of 2.39 was calculated; however, for design purposes, we elected to use a SN of 2.75.

Four alternative asphalt pavement sections were developed for the project using this minimum SN. Alternative No. 1 was developed to meet the City of Flagstaff's minimum required AC thickness per Detail 10-09-010. Alternative No. 2 was developed for scenarios in which a minimum of 36-inches of subbase material is used below the new AB layer. Alternative No. 3 was developed for scenarios where shallow bedrock is encountered. Alternative No. 4 was developed to reduce the effects of freeze-thaw. Based on the ADOT manual, Flagstaff has a freezing index of 700 which recommends a minimum thickness of 27 inches of non-frost susceptible materials. While this final alternative may not be feasible from a cost standpoint it was provided as a reference point. The thinner pavements sections (if used) may call for additional life-cycle maintenance due to the effects of freeze-thaw. The recommended pavement sections are provided below win Table 6.

Table 6 – Calculated Pavement Sections						
Roadway Segment	Subgrade Improvement	AC (in)	AB (in)	Subbase (in)	Pavement Section Thickness (in)	SN
JW Boulevard Alternative No. 1	Engineered Fill[1]	5.0	12.0	--	17.0	3.61
JW Boulevard Alternative No. 2	Engineered Fill[1]	5.0	6.0	36	47.0	6.23
JW Boulevard Alternative No. 3	Bedrock (Limestone)	5.0	6.0	--	11.0	2.91
JW Boulevard Alternative No. 4	Engineered Fill[1]	5.0	22.0	--	27.0	4.79

Note:
[1] Includes minimum of 12-inches of improvement.

The pavement sections presented in Table 6 are based on the expected traffic and the existing / improved subgrade soil conditions. The full design life of 20 years is expected to be achieved with these pavement sections with periodic maintenance to include possible overlays, and as long as good drainage is provided and maintained. In Arizona, pavement sections often become brittle and experience cracking before the design life is attained, therefore, the asphalt surface needs to be maintained and sealed periodically over its life. If the subgrade soils experience a significant increase in moisture content or freeze-thaw cycles, that accelerated pavement deterioration an increased maintenance cycle should be anticipated. Pavement and base course materials should not be placed when the subgrade is wet. Good surface drainage should be provided away from the edges of paved areas to minimize lateral moisture transmission into the subgrade soils.

10.6.9. Structural Number Check

In accordance with the procedure for flexible pavement design noted in the ADOT Manual, and using the design parameters presented in this report, we have checked that the SN for the proposed AC roadway sections are adequate with respect to Table 2-10 (minimum required SN). This evaluation is presented in Table 7.

Table 7 – SN Summary			
Roadway Segment	Minimum Required SN[1]	Required by Design SN	Calculated SN of New Pavement
JW Boulevard Alternative No. 1	2.75	3.52[2]	3.61
JW Boulevard Alternative No. 2	2.75	3.52[2]	6.23
JW Boulevard Alternative No. 3	2.75	2.75[3]	2.91

Table 7 – SN Summary			
Roadway Segment	Minimum Required SN[1]	Required by Design SN	Calculated SN of New Pavement
JW Boulevard Alternative No. 4	2.75	3.52[2]	4.79

Notes:
[1] Required by Table 2-10 in the ADOT Manual.
[2] Required based on traffic loading and subgrade conditions. For 12 inches of improved subgrade, Mr = 9,927 psi.
[3] Required based on traffic loading and bedrock subgrade conditions. Mr = 26,000 psi.

10.6.1. Rigid Pavement Design

In addition to flexible pavement, a rigid pavement design alternative was considered. The same traffic loading, level of service, and serviceability index were utilized to evaluate a Portland cement concrete pavement (PCCP). ADOT recommends a combine standard error value (S0) of 0.25 for rigid pavement design.

In addition, the ADOT Manual recommends an average modulus of rupture for PCCP of 670 pounds per square inch (psi). An assumed average concrete compressive strength of 5,000 psi which yields a modulus of elasticity for PCCP of 4,000,000 psi.

Based on Table 2-8, a load transfer coefficient (J) of 3.9 was assigned for plain-jointed concrete without dowels.

A modulus of subgrade reaction (k) value of 512 pounds per cubic inch (pci) was evaluated based on the design resilient modulus of 9,927.

From Table 2-9 in the ADOT manual, a loss of soil support value of 1.5 was utilized based on the SVF of 3.5 and base material of AB. Using the loss of soil support value and resilient modulus values, a corrected, effective modulus of subgrade reaction (k) of 91 pci was selected.

Based on these inputs, Ninyo & Moore recommends a PCCP be not less than 12 inches thick. The PCCP should be support on a minimum of 4 inches of compacted AB material placed over engineered fill as summarized in this report. If freeze-thaw conditions are a concern an additional 15 inches of non-frost suspectable material should be used.

10.7. Exterior Concrete Flatwork

Exterior concrete flatwork should be supported on a zone of moisture-conditioned and compacted engineered fill as described in this report and compacted to reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer.

10.8. Corrosion

The corrosion potential of the on-site materials was tested to evaluate its potential effect on the foundations and structures. Our corrosion evaluation of the on-site soils is based on the results of our field and laboratory testing done for this project. A corrosion specialist should perform their own analysis.

Laboratory testing consisted of pH, minimum electrical resistivity, and chloride and soluble sulfate contents. The pH and minimum electrical resistivity tests were performed in general accordance with Arizona Test 236c, while sulfate and chloride tests were performed in accordance with Arizona Test 733 and 736, respectively. The results of these corrosivity tests are presented in Appendix B.

The soil pH values of the selected samples tested from our borings ranged from 6.8 to 8.1, which is considered to be neutral. The minimum electrical resistivity of the samples tested ranged from 1,356 to 2,034 ohm-cm, which is considered corrosive to ferrous materials. The chloride content of the samples tested ranged from 5 to 16 ppm, which indicates a non-corrosive environment for ferrous materials. The soluble sulfate content of the soil samples tested ranged between 0.011 to 0.015, which is considered to represent negligible sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials are considered to be corrosive to ferrous materials. To reduce the corrosion potential of buried metallic utilities, we recommend that topsoil, organic soils, soils, and mixtures of sand and clay not be placed adjacent to buried metallic utilities. Rather, we suggest that sand or gravel be placed around buried metal piping. Also, buried utilities of different metallic construction or operating temperatures should be electrically isolated from each other to minimize galvanic corrosion problems. In addition, new piping should be electrically isolated from old piping, if any, so that the old metal will not increase the corrosion rate of the new metal. A corrosion specialist should be consulted for further recommendations.

10.9. Concrete

Laboratory chemical tests performed on on-site soil samples indicated a sulfate content ranging between 0.011 and 0.015 percent by weight, which represents a negligible sulfate exposure for concrete. Based on the following American Concrete Institute (ACI) table (Table 8), the on-site soils should be considered to have negligible sulfate exposure to concrete. Based on the sulfate test results, and based on our experience with similar soil conditions, the specific use of the facility, and nearby practice, we however recommend the use of sulfate resistant cement (Type II or similar) for construction of concrete structures at this site. Due to potential uncertainties as to the use of reclaimed irrigation water, or topsoil that may contain higher sulfate contents, pozzolan or admixtures designed to increase sulfate resistance may be considered.

Table 8 – ACI Requirements for Concrete Exposed to Sulfate-Containing Soil

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Cement Type	Water-Cementitious Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	f'c, Normal-Weight and Lightweight Aggregate Concrete, psi x 0.00689 for MPa
Negligible	0.00 - 0.10	--	--	--
Moderate[2]	0.10 - 0.20	II, IP(MS), IS (MS)	0.50 or less	4,000 or more
Severe	0.20 - 2.00	V	0.45 or less	4,500 or more
Very severe	Over 2.00	V plus pozzolan[3]	0.45 or less	4,500 or more

Notes:

1 A lower water-cementitious materials ratio or higher strength may be needed for low permeability or for protection against corrosion of embedded items or freezing and thawing (ACI Table 4.2.2).

2 Seawater.

3 Pozzolan that has been evaluated by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

We recommend that the structural concrete have a water-cementitious materials ratio no more than 0.50 by weight for normal weight aggregate concrete. The structural engineer should ultimately select the concrete design strength based on the project specific loading conditions. Higher strength concrete may be selected for increased durability and resistance to slab curling and shrinkage cracking.

10.10. Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the project plans and schedule. Our office should be notified if the project description included herein is incorrect, or if the project characteristics are significantly changed.

10.11. Construction Observation and Testing

During construction operations, we recommend that a qualified geotechnical consultant perform observation and testing services for the project. These services should be performed to evaluate exposed subgrade conditions, including the extent and depth of overexcavation, to evaluate the suitability of the on-site materials for use as fill and to observe placement and test compaction of fill soils. If another geotechnical consultant is selected to perform observation and testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our recommendations and they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

11. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

12. REFERENCES

Aerial Photographs Reviewed, Historical Aerials and Google Earth©, dated 1964, 1974, 1980, 1981, 1997, 2003, 2005, 2006, 2007, 2010, 2011, 2012, 2013, 2015, 2016, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025.

American Concrete Institute (ACI), 1991a, Guidelines for Concrete Floor and Slab Construction (ACI 302.1R).

American Concrete Institute (ACI), 2005, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).

Arizona Department of Transportation, 2017. Pavement Design Manual. September 29.

Arizona Department of Water Resources (ADWR), "GWSI Groundwater Maps" https://gisweb.azwater.gov/waterresourcedata/.

ASTM International (ASTM), Annual Book of ASTM Standards.

City of Flagstaff, Chapter 13-23 Standard Drawings.

Euge, K.M., Schell, B.A., and Lam, I.P., 1992, Development of Seismic Acceleration Contour Maps for Arizona: Arizona Department of Transportation Report No. AZ 92-344: dated September.

International Code Council (ICC), International Building Code (IBC).

Maricopa Association of Governments (MAG), 2025, Uniform Standard Specifications and Details for Public Works Construction.

Ninyo & Moore, In-house proprietary information.

Occupational Safety and Health Administration (OSHA), Title 29 of the Code of Federal Regulations (CFR), Part No. 1926 - Safety and Health Regulations for Construction, Subpart P - Excavations.

Pearthree, P.A., 1998, Quaternary Fault Data and Map for Arizona: Arizona Geological Survey, Open-File Report 98-24.

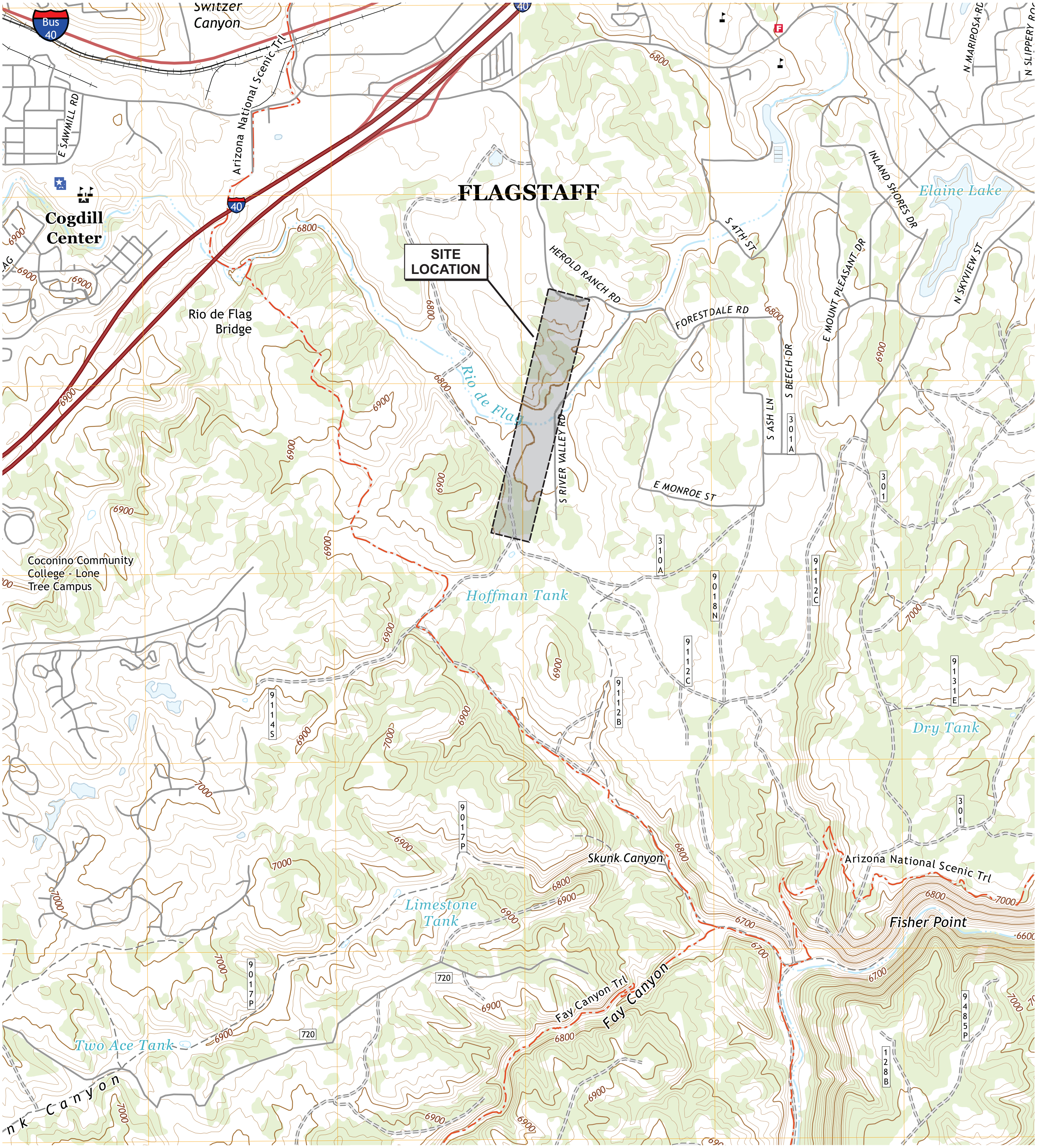
Richard, S.M., Reynolds, S.J., Spencer, J.E., and Pearthree, P.A., 2000, Geologic Map of Arizona: Arizona Geological Survey Map 35, 1 sheet, scale 1:1,000,000.

Schumann, H.H. and Genualdi, R., 1986, Land Subsidence, Earth Fissures, and Water-level Changes in Southern Arizona: Arizona Geological Survey OFR 86-14: scale 1:500,000.

United States Department of Agriculture (USDA), Soil Survey website. http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx.

United States Geological Survey (USGS), 2018, U.S. Seismic Design Maps, page URL: http://geohazards.usgs.gov/designmaps/us/application.php.

FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 1

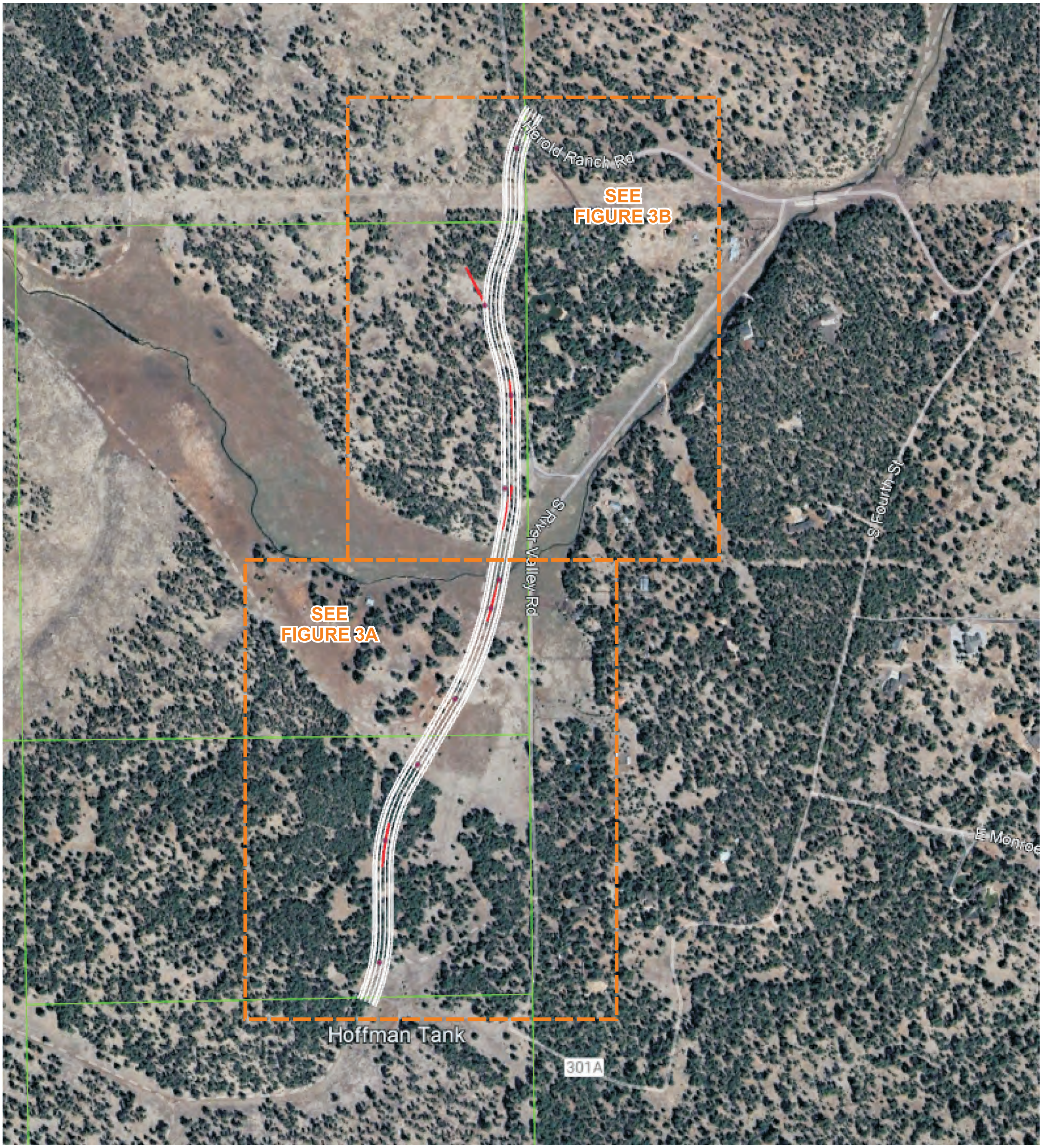
SITE LOCATION

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

609083001 | 2/26

Ninyo & Moore
A SOCOTEC COMPANY

bsm file no: 9083vmap0925



Source: NAVTEQ, 04/09/25.

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 2

SITE PLAN

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

609083001 | 2/26

Ninyo & Moore
A SOCOTEC COMPANY

bsm file no: 9083sp0925

Source: NAVTEQ, 04/09/25.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 3A

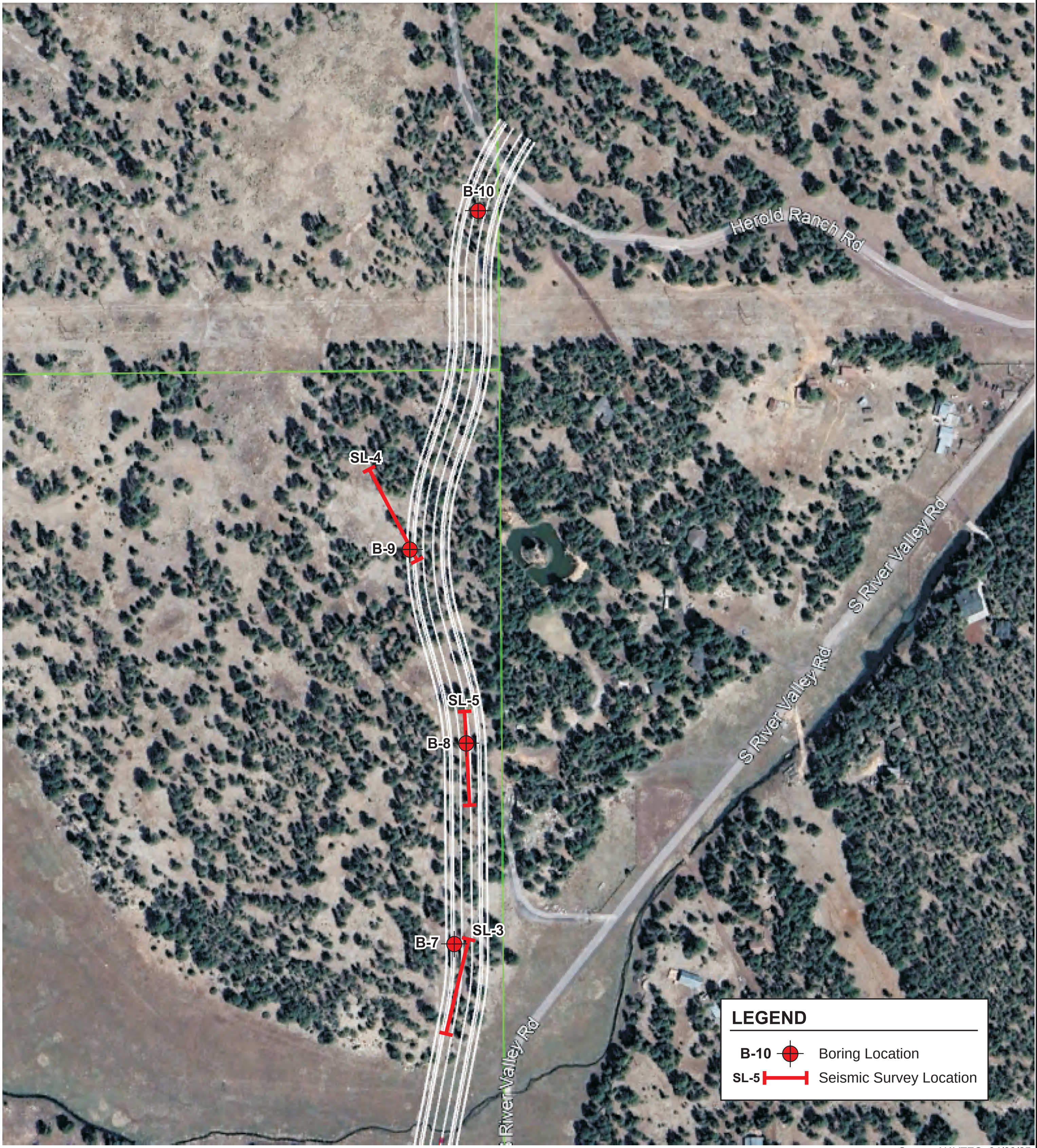
EXPLORATION LOCATIONS

Ninyo & Moore
A SOCOTEC COMPANY

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

609083001 | 2/26

bsm file no: 9083blm0925a



Source: NAVTEQ, 04/09/25.

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

bsm file no: 9083blm0925b

FIGURE 3B

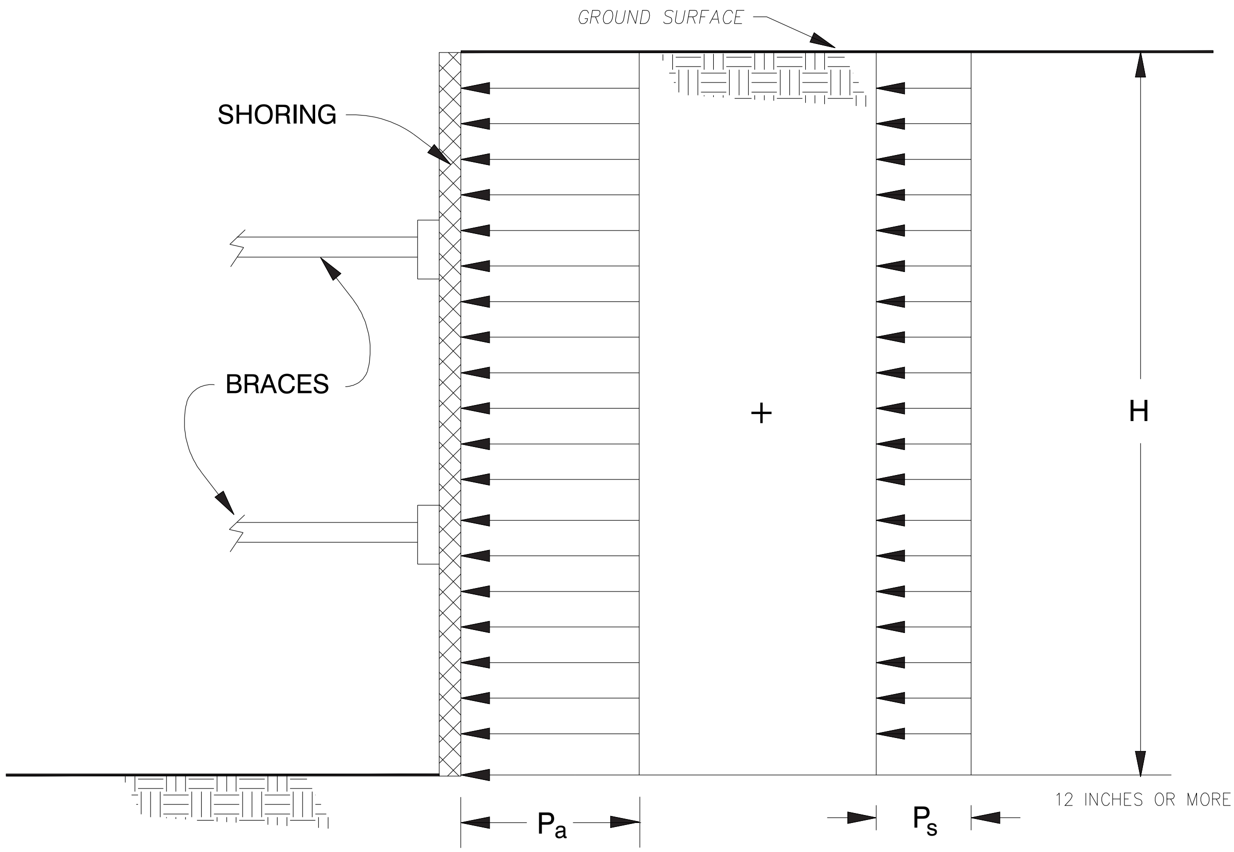
Ninyo & Moore

A SOCOTEC COMPANY

EXPLORATION LOCATIONS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

609083001 | 2/26



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a

   P_a = 31H psf

2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s

   P_s = 120 psf

3. ASSUMES GROUNDWATER IS NOT PRESENT

4. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED

5. H IS IN FEET

bsm file no: 5986lep1219

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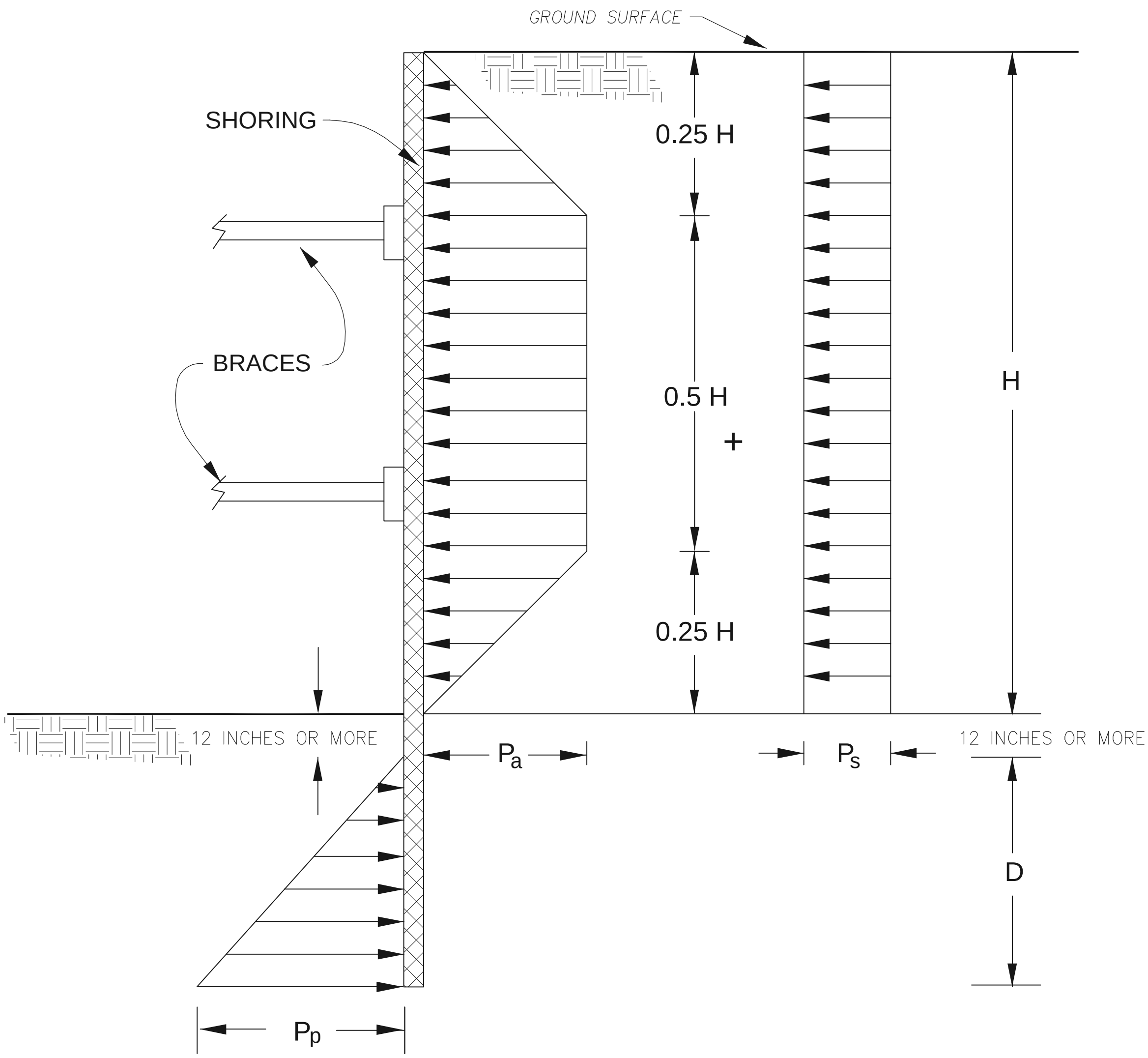
FIGURE 4

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION IN GRANULAR SOILS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 56 H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 255D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

bsm file no: 6809lep0721b

FIGURE 5

LATERAL EARTH PRESSURES FOR BRACED EXCAVATIONS IN COHESIVE SOILS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

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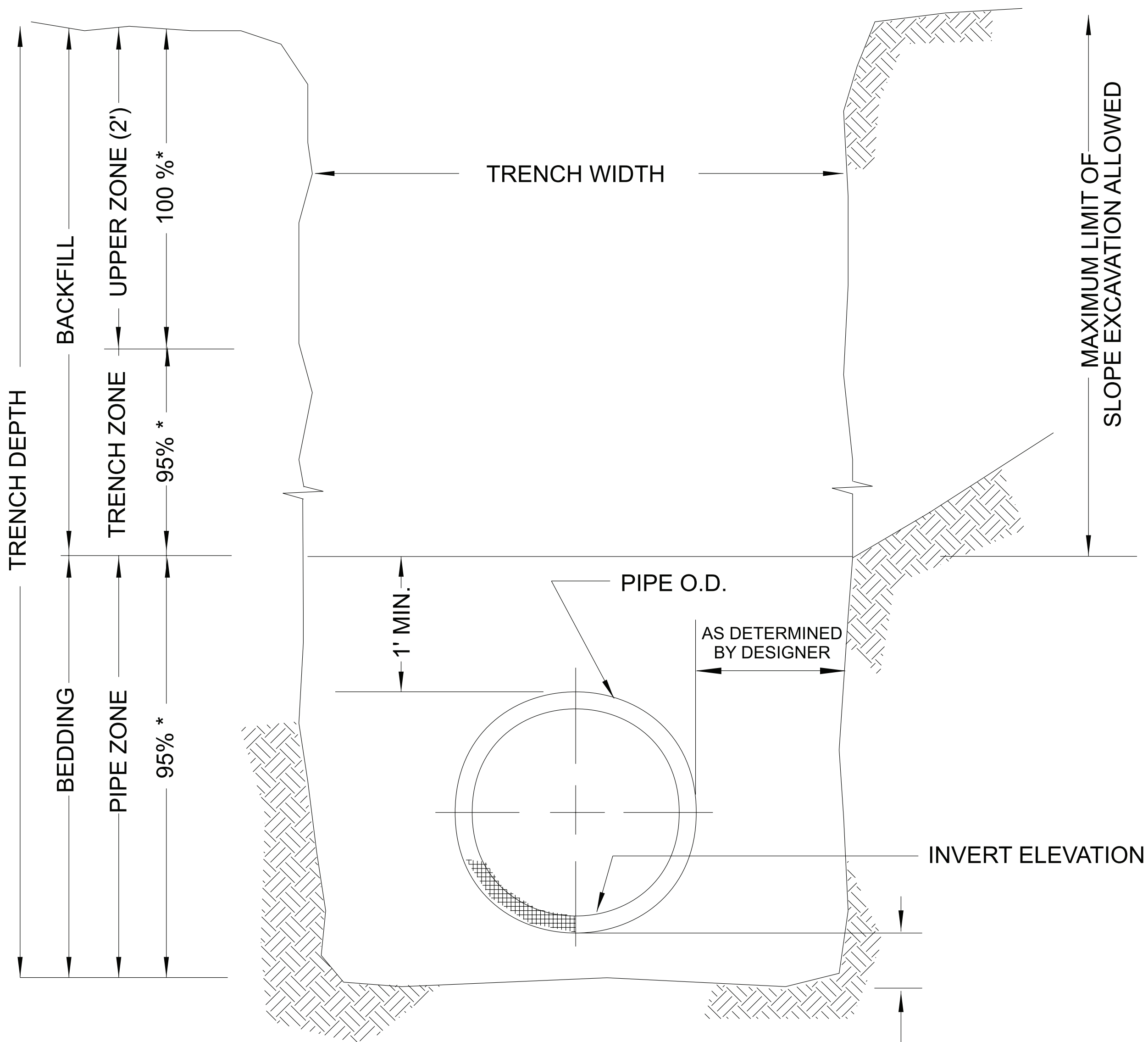

SECTION

NOTE

* Indicates minimum relative compaction (see report for details).

Upper zone required for pavement areas only.

NOT TO SCALE

FIGURE 6

TRENCH BACKFILL DETAIL

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a SPT sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven up to 18 inches into the ground with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D3550. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

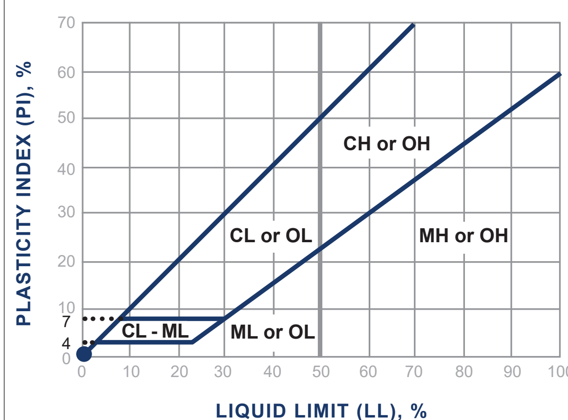
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	<u>MAJOR MATERIAL TYPE (SOIL)</u>: Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

DATE DRILLED 9/8/25 BORING NO. B-1

GROUND ELEVATION 6,811' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						CL-ML	ALLUVIUM: Brown, dry, hard, silty CLAY.
		34	7.0	97.5			
		30					Scattered caliche nodules.
5							
		50/4"				GP	Pink, dry, very dense, poorly graded GRAVEL; sandstone.
		50/5"					Reddish orange.
10							
							@ 11 feet: Total refusal; bedrock.
15							
20							

Total Depth = 11 feet. (Refusal)
Groundwater not encountered during drilling.
Backfilled on 9/8/25 shortly after completion of drilling.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED	9/8/25
BORING NO.	B-2
GROUND ELEVATION	6,815' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (GSI)
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)
DROP	30"
SAMPLED BY	DM
LOGGED BY	DM
REVIEWED BY	SDN

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					SM	ALLUVIUM: Brown, dry, medium dense, silty SAND.
	Driven	16					
	Driven	50/4"					Very dense; scattered roots.
5							
	Driven	50/2"				GP	Brown, dry, very dense, poorly graded GRAVEL; bedrock. @ 6.2 feet: Auger refusal.
							Total Depth = 6.2 feet. (Refusal) Groundwater not encountered during drilling. Backfilled on 9/8/25 shortly after completion of drilling.
10							
15							
20							

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

FIGURE A- 2

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JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 9/9/25 BORING NO. B-3

GROUND ELEVATION 6,796' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0				CL-ML	ALLUVIUM: Reddish brown, dry, hard, sandy, silty CLAY.
	66/11"	6.5	114.6		
	50/3"				
5					
	50/4"				
	50/1"				@ 8.1 feet: Total refusal; possible bedrock.
					Total Depth = 8.1 feet. (Refusal) Groundwater not encountered during drilling. Backfilled on 9/9/25 shortly after completion of drilling.
10					Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15					
20					

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ALLUVIUM: Brownish/red, dry, very dense, clayey SAND; trace roots.
			46					
			50/1"					No ring recovery; possible bedrock. @ 4 feet: Total refusal.
5								Total Depth = 4 feet. (Refusal) Groundwater not encountered during drilling. Backfilled on 9/9/25 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

DATE DRILLED 9/9/25 BORING NO. B-4

GROUND ELEVATION 6,792' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						CL	ALLUVIUM: Brown, dry, very stiff, lean CLAY.
		17	11.8	76.7			
		11					
5							
		17	22.2	84.5		SC	Dark brown, dry to moist, medium dense, clayey SAND.
		5					Moist; loose.
10							
		7	28.3	71.5			
15							
		7					Medium dense; trace fine gravel.
20							

DATE DRILLED 9/9/25 BORING NO. B-5

GROUND ELEVATION 6,788' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DATE DRILLED 9/9/25 BORING NO. B-5

GROUND ELEVATION 6,788' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							GP	White, dry, medium dense, poorly graded GRAVEL; limestone.
							SC	Brown, moist, medium dense, clayey SAND; trace fine to coarse gravel.
			16	13.0	110.0			
25								
			11					
30								Total Depth = 30 feet. Groundwater not encountered during drilling. Backfilled on 9/9/25 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
35								
40								

DATE DRILLED 9/9/25 BORING NO. B-6

GROUND ELEVATION 6,787' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ALLUVIUM: Brown, dry, stiff, lean CLAY.
			9					
			17	15.9	78.9			Very stiff.
5								
			9					Stiff.
			7	28.6	91.4			Moist.
10								
			4					Firm.
15								
			9	32.6	86.1			Wet; stiff.
20								

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JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/9/25 BORING NO. B-6 GROUND ELEVATION 6,787' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN **DESCRIPTION/INTERPRETATION**
20							CL	ALLUVIUM: (Continued) Brown, wet, stiff, lean CLAY.
			14					Very stiff; trace fine gravel.
25								
			50/5"				GP	Hard. White, dry to moist, very dense, poorly graded GRAVEL; bedrock.
30								Total Depth = 29 feet. Groundwater not encountered during drilling. Backfilled on 9/9/25 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
35								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						SM	ALLUVIUM: Brownish yellow, dry, very dense, silty SAND; trace clay; scattered caliche nodules.
	X	X	50/5"					
	X	X	50/5"					
5								
		X	50/4"					No recovery.
		X	50/5"					
10								
		X	50/0"					@ 11 feet: Total refusal; coarse gravel; possible bedrock.
								Total Depth = 11.1 feet. (Refusal) Groundwater not encountered during drilling. Backfilled on 9/8/25 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15								
20								

DATE DRILLED 9/8/25 BORING NO. B-7

GROUND ELEVATION 6,799' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DATE DRILLED	9/8/25
BORING NO.	B-8
GROUND ELEVATION	6,832' ± (MSL)
SHEET	1 OF 2
METHOD OF DRILLING	CME-75, 8" Diameter Hollow-Stem Auger (GSI)
DRIVE WEIGHT	140 lbs. (Automatic Trip Hammer)
DROP	30"
SAMPLED BY	DM
LOGGED BY	DM
REVIEWED BY	SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ALLUVIUM: Reddish brown, dry, hard, lean CLAY.
			51					
			50/4"	4.6	116.2		CL-ML	Reddish brown, dry, hard, silty CLAY.
							CL	Reddish brown, dry, hard, lean CLAY.
5								
			50/5"					
			40	5.9	126.3			
10								
			24					No recovery.
15								
								@ 17 feet: Total refusal; coarse gravel; cobbles and possible boulders.
20								

Total Depth = 17 feet. (Refusal)
Groundwater not encountered during drilling.
Backfilled on 9/8/25 shortly after completion of drilling.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/8/25 BORING NO. B-8 GROUND ELEVATION 6,832' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI) DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30" SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN DESCRIPTION/INTERPRETATION
20								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
25								
30								
35								
40								

FIGURE A- 11

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JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	ALLUVIUM: Brown, dry, hard, sandy SILT.
			50/5"	3.8	103.9			Brownish yellow; few fine to coarse gravel; possible sandstone.
5			24					
			50/5"	9.4	103.2			
			50				SC	Reddish brown, moist, very dense, clayey SAND; few to fine fin to coarse gravel.
10								@ 11 feet: Total refusal; coarse gravel; cobbles and possible boulders.
								Total Depth = 11 feet. (Refusal) Groundwater not encountered during drilling. Backfilled on 9/8/25 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
15								
20								

DATE DRILLED 9/8/25 BORING NO. B-9

GROUND ELEVATION 6,844' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC-SM	ALLUVIUM: Brown, dry, very dense, silty, clayey SAND; scattered caliche nodules; possible sandstone.
			50/5"					
			50/3"					No recovery.
5								
			50/4"					
			50/2"					No recovery.
10								
			50/1"				SC	Light brown, dry, very dense, clayey SAND; caliche nodules; possible bedrock.
15								Total Depth = 15 feet. (Refusal) Groundwater not encountered during drilling. Backfilled on 9/8/25 shortly after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								

DATE DRILLED 9/8/25 BORING NO. B-10

GROUND ELEVATION 6,868' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 8" Diameter Hollow-Stem Auger (GSI)

DRIVE WEIGHT 140 lbs. (Automatic Trip Hammer) DROP 30"

SAMPLED BY DM LOGGED BY DM REVIEWED BY SDN

FIGURE A- 13

Ninyo & Moore
A SOCOTEC COMPANY

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D2937. The test results are presented on the logs of the exploratory borings in Appendix A.

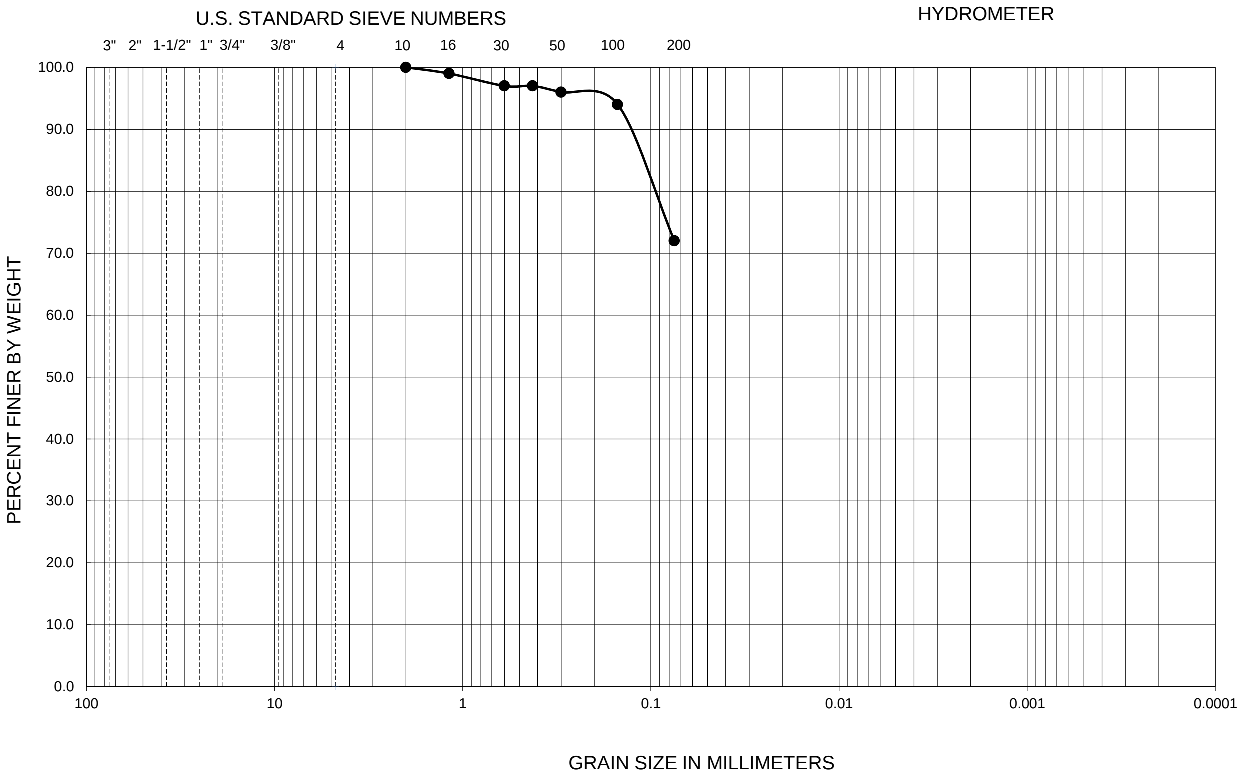
Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain-size distribution curves are shown on Figures B-1 through B-12. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Atterberg Limits Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figures B-13 and B-14.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-15 through B-18.

Soil Corrosivity Tests
Soil pH and minimum resistivity tests were performed on a representative sample in general accordance with Arizona test method, ARIZ 236c. The chloride content of the selected sample was evaluated in general accordance with ARIZ 736. The sulfate content of the selected sample was evaluated in general accordance with ARIZ 733. The test results are shown on Figure B-19.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	0.0-5.0	22	17	5	--	--	--	--	--	72.0	CL-ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-1

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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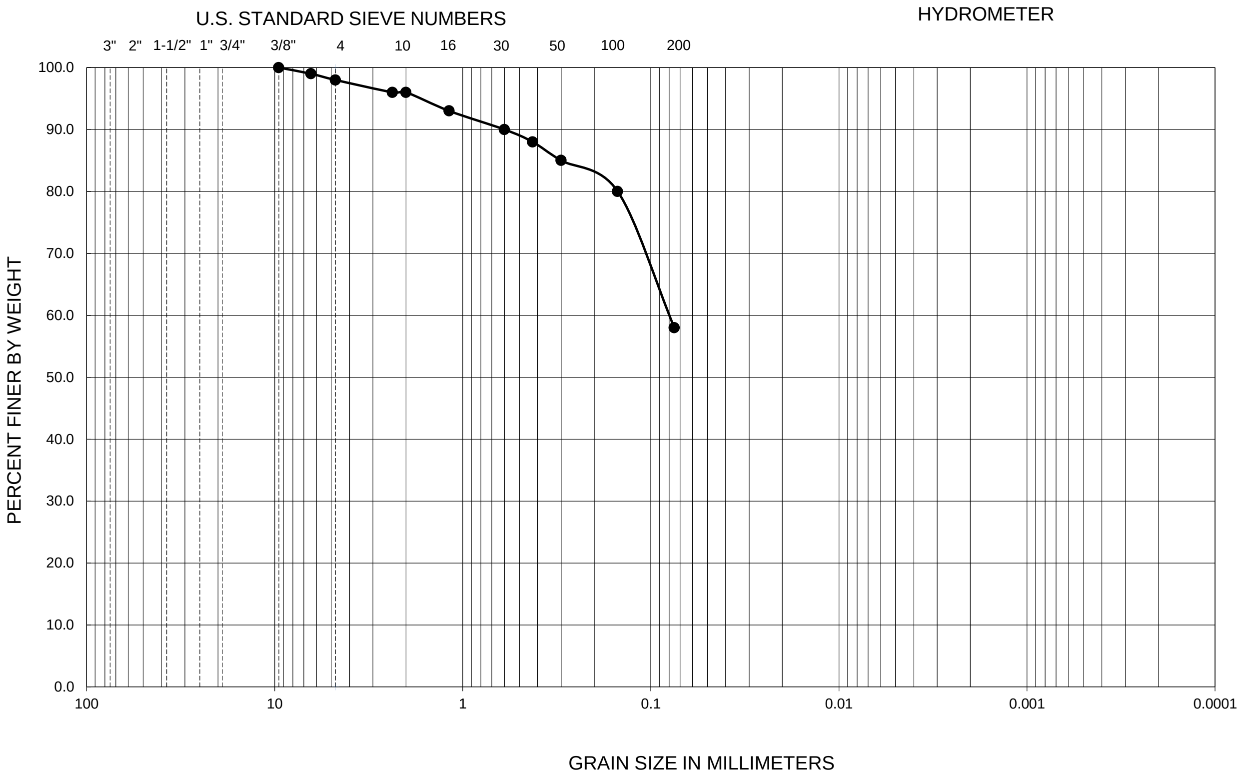
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	0.0-5.0	21	17	4	--	--	0.08	--	--	58.0	CL-ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-2

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

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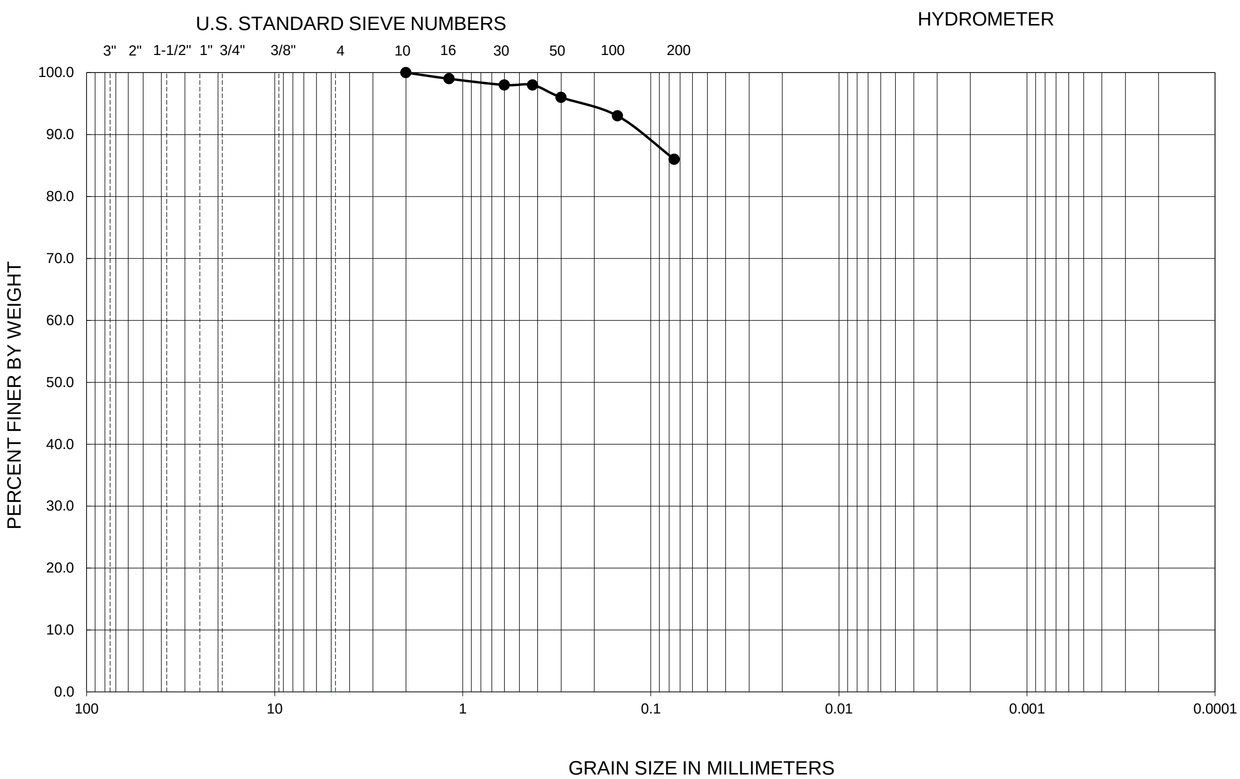
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-5	0.0-5.0	37	23	14	--	--	--	--	--	86.0	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-3

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GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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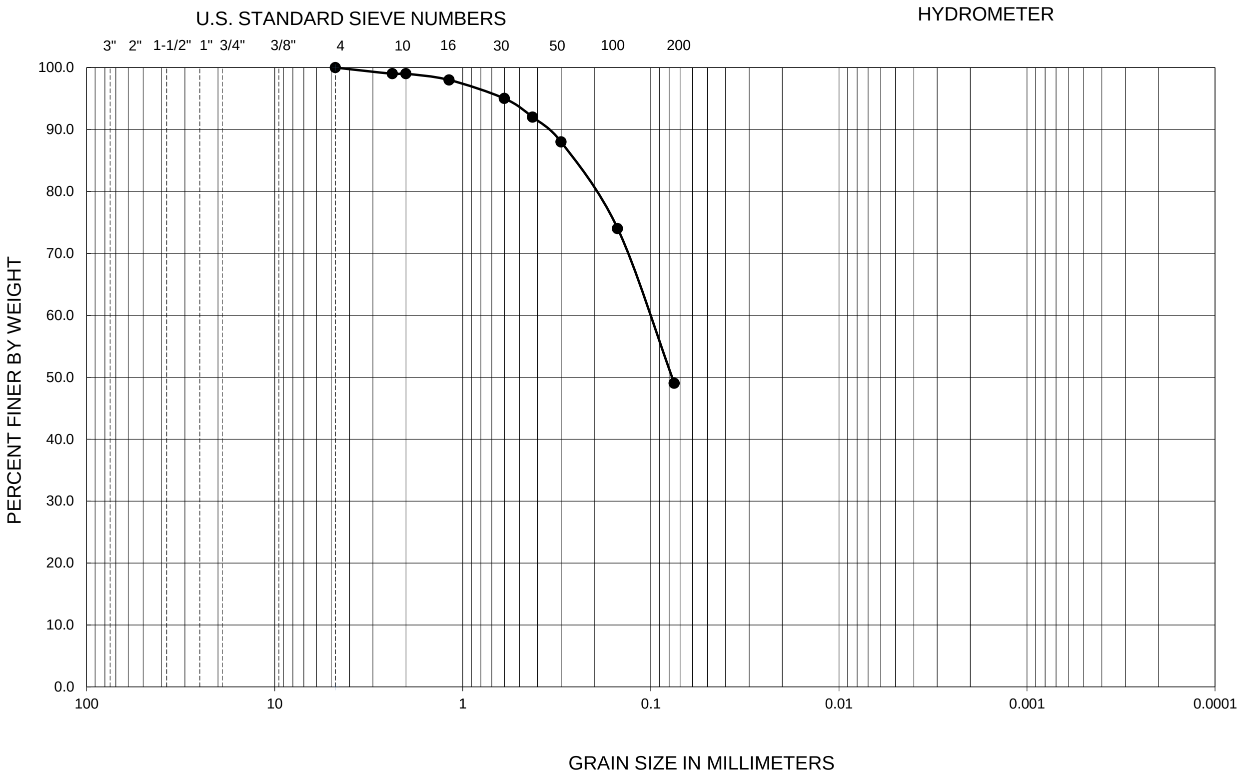
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-5	13.5-15.0	37	17	20	--	--	0.10	--	--	49.0	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-4

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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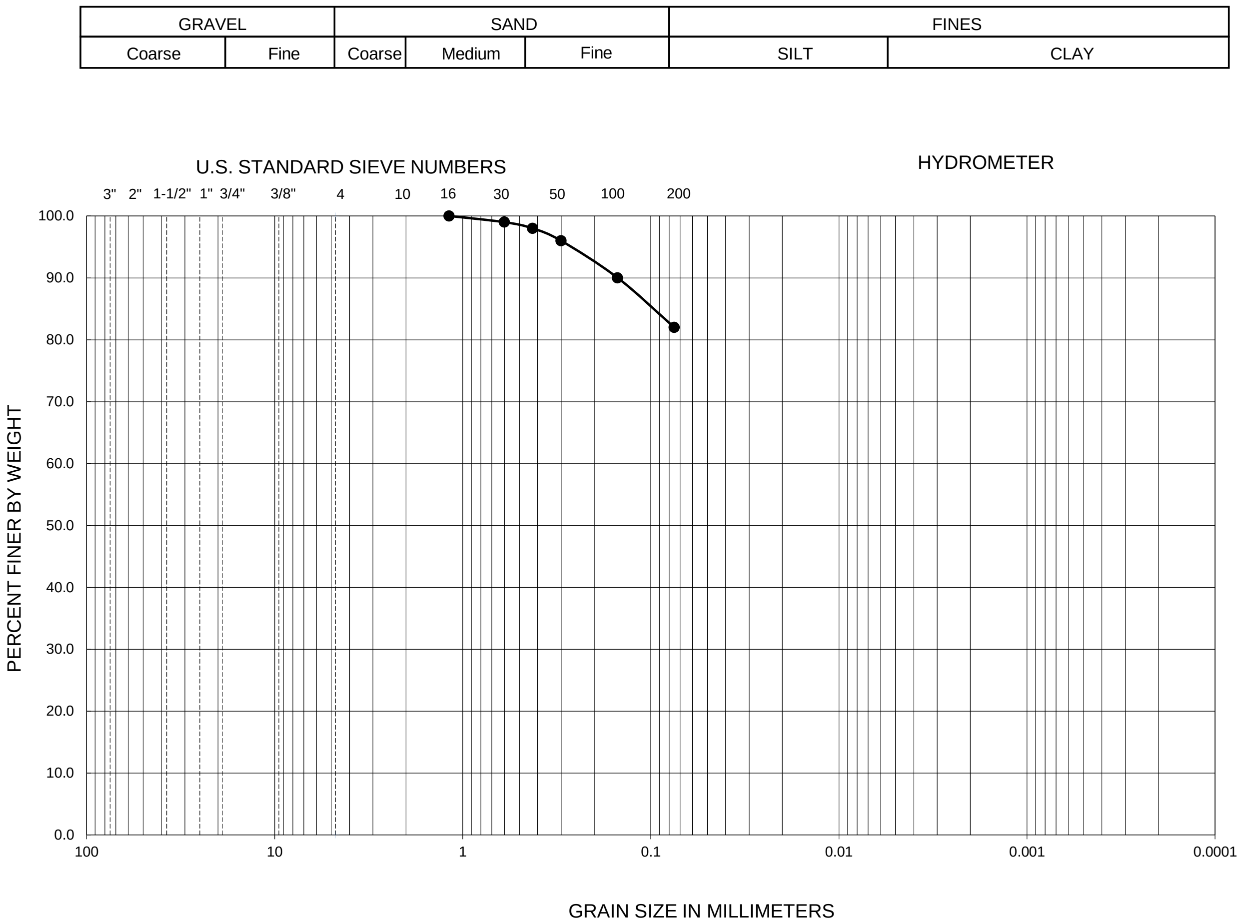

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	0.0-5.0	39	23	16	--	--	--	--	--	82.0	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-5

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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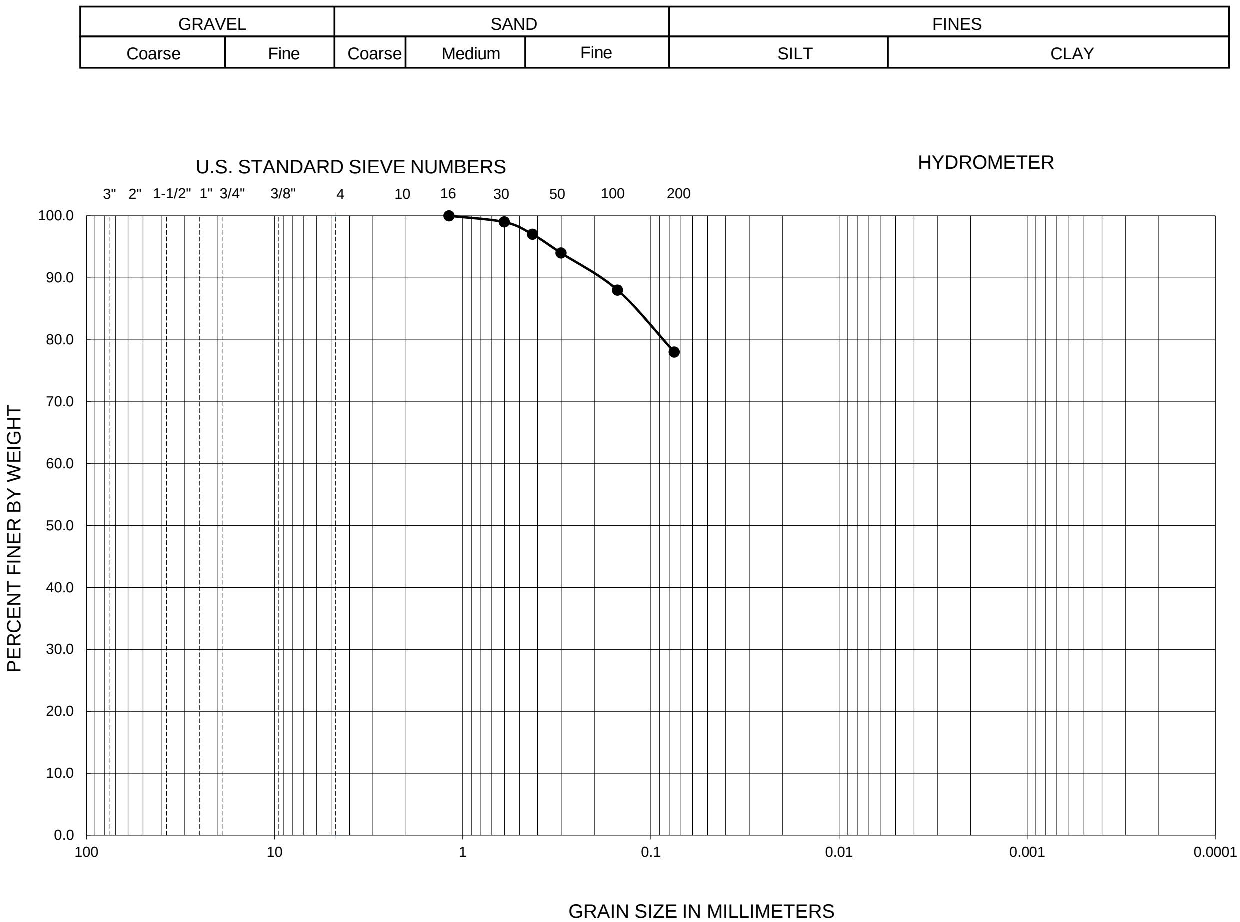

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	8.5-10.0	38	21	17	--	--	--	--	--	78.0	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-6

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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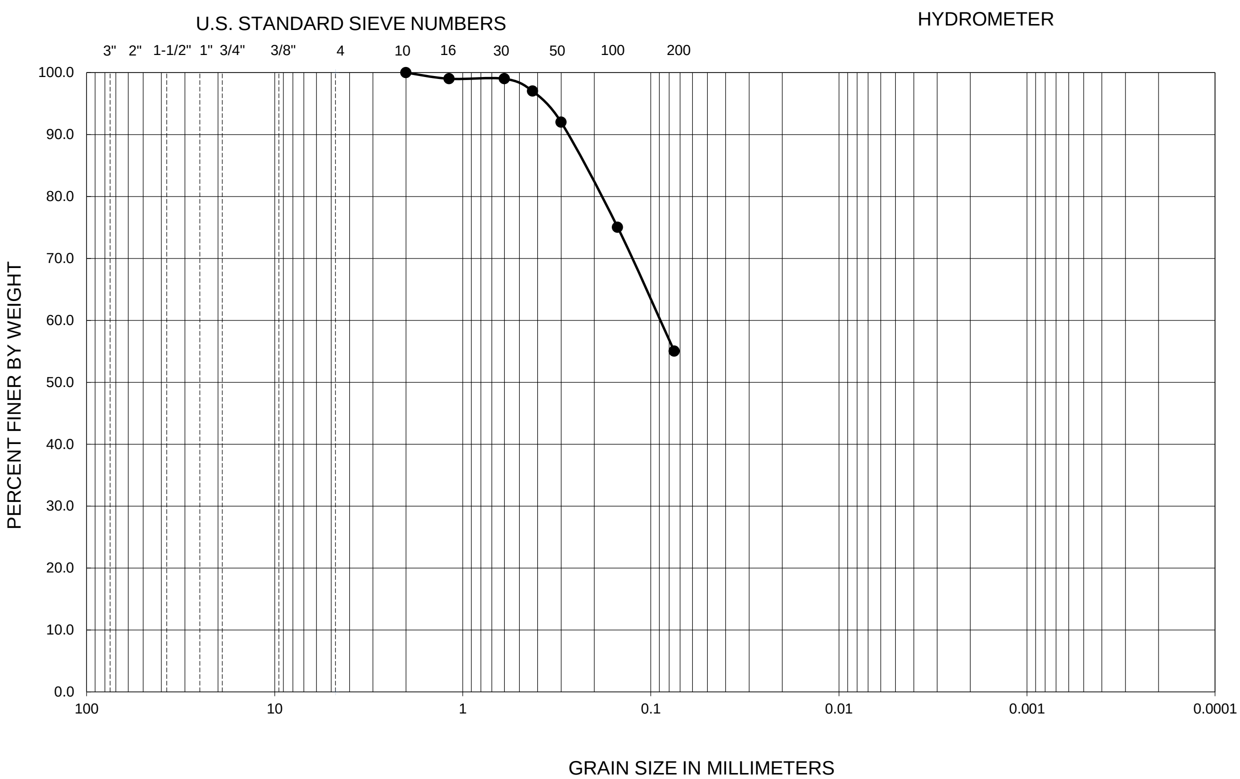
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-6	18.5-20.0	41	23	18	--	--	0.09	--	--	55.0	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-7

Ninyo & Moore
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GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

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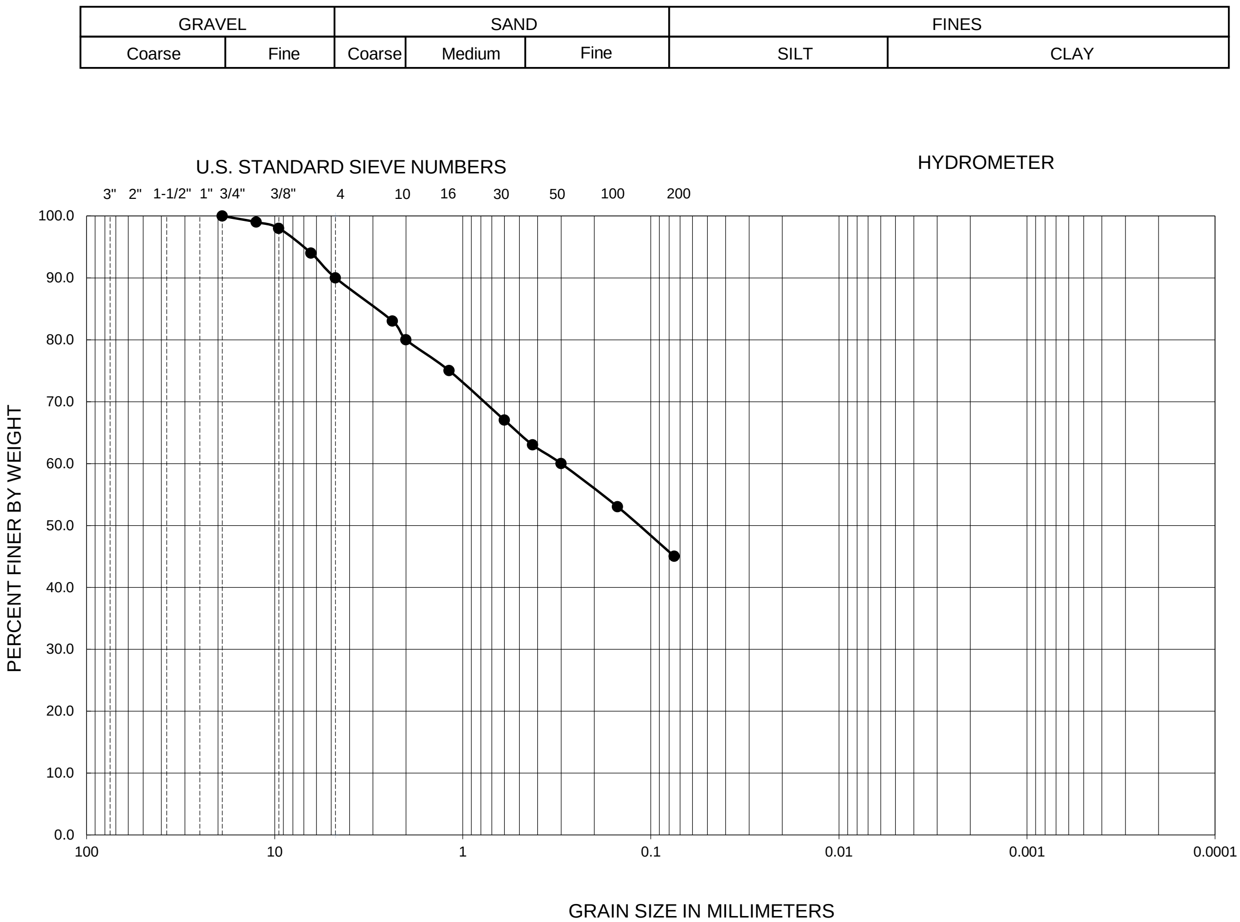

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-7	1.0-1.4	--	--	NP	--	--	0.31	--	--	45.0	SM

NP - INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-8

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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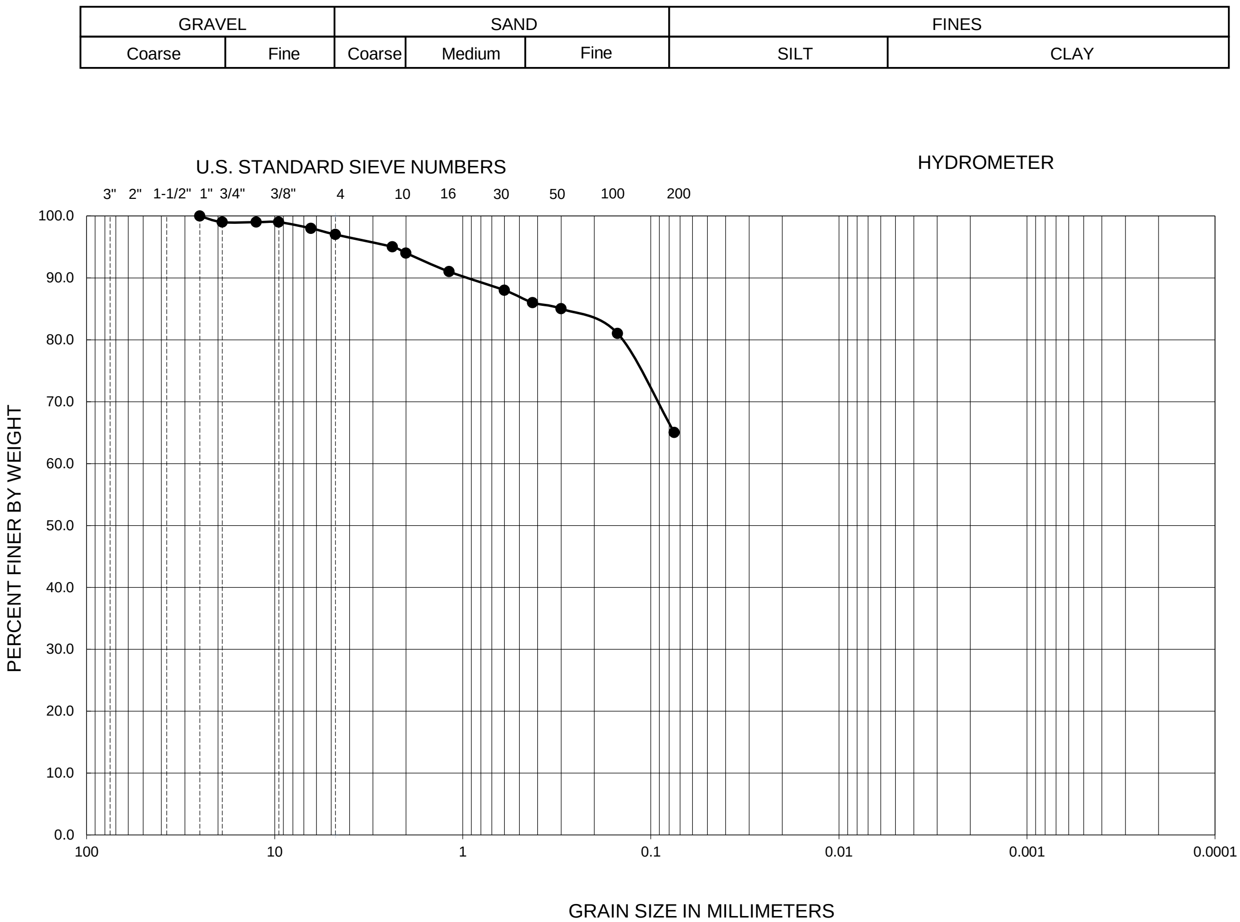

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-8	0.0-5.0	26	17	9	--	--	--	--	--	65.0	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-9

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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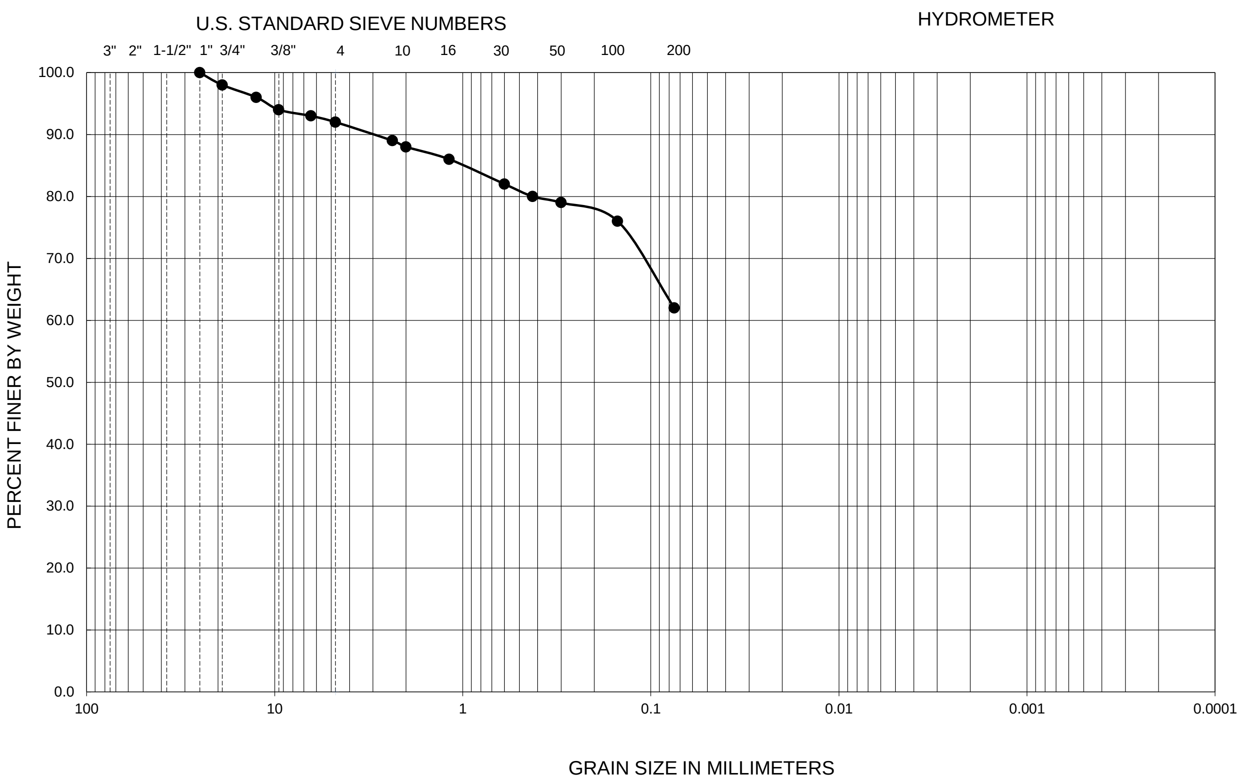
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-8	3.5-4.3	21	16	5	--	--	--	--	--	62.0	CL-ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

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A SOCOTEC COMPANY

FIGURE B-10

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

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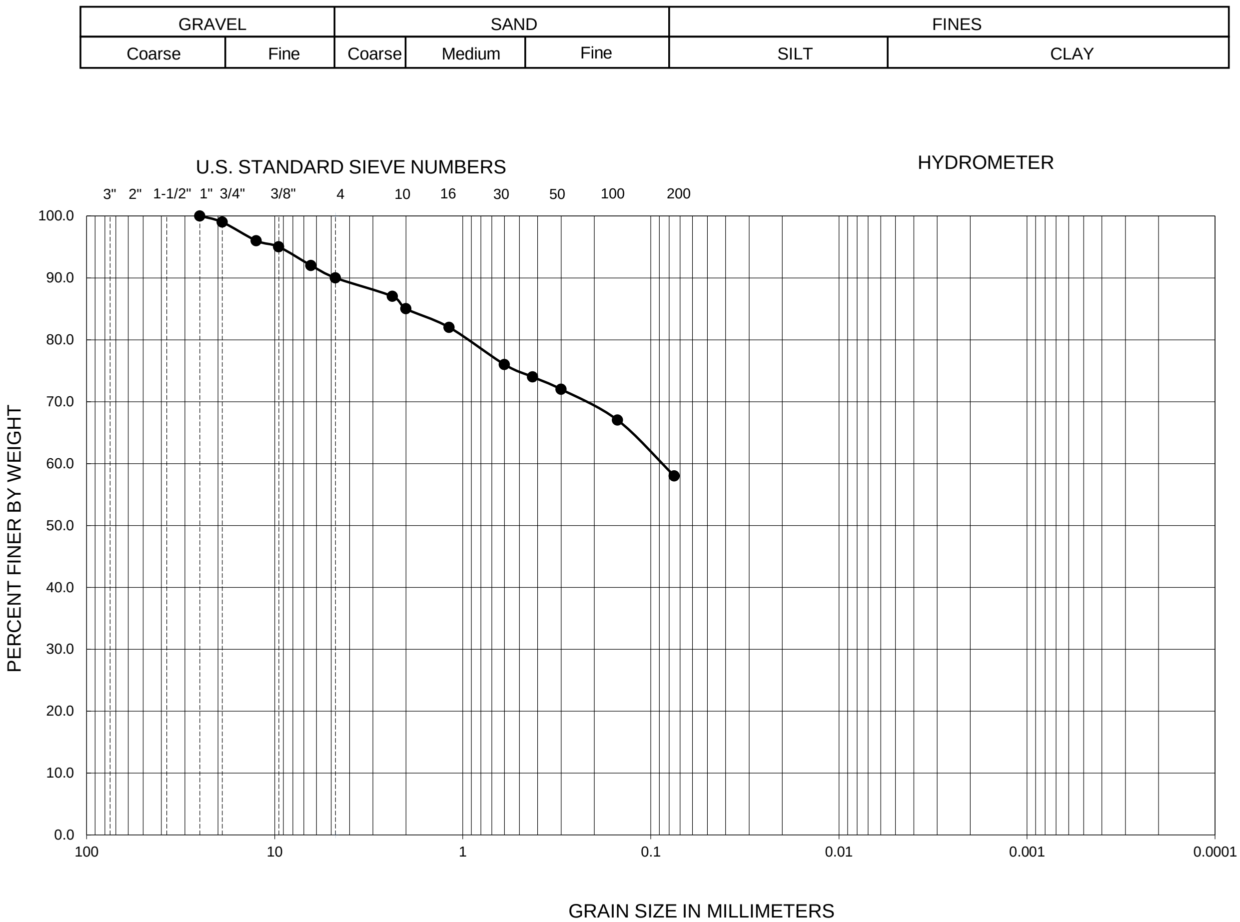
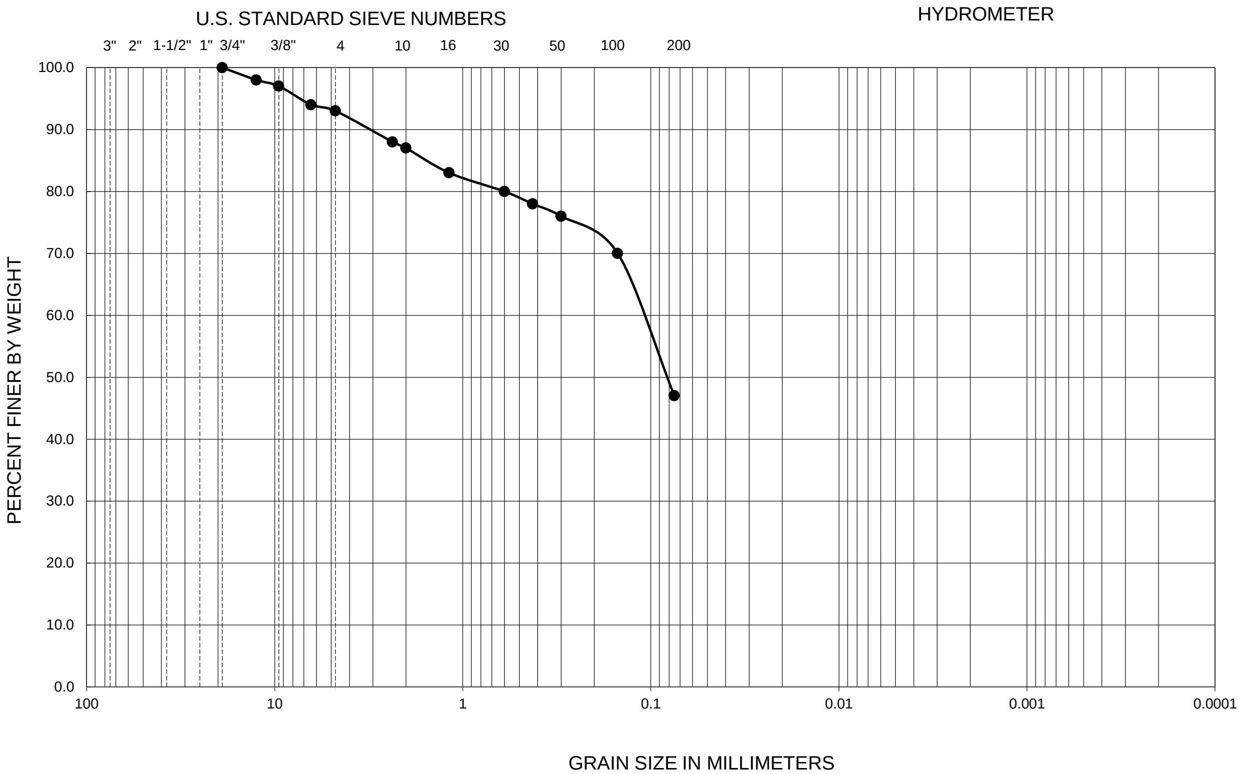


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-9	0.0-5.0	--	--	NP	--	--	0.09	--	--	58.0	ML

NP - INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-10	0.0-5.0	26	19	7	--	--	0.11	--	--	47.0	SC-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-12

GRADATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

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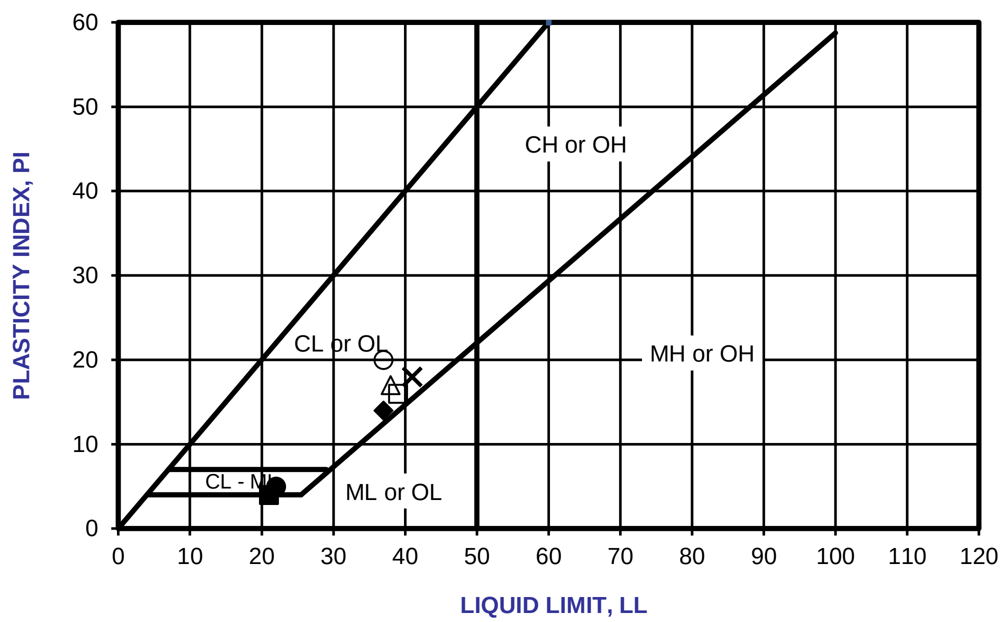
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	0.0-5.0	22	17	5	CL-ML	CL-ML
■	B-3	0.0-5.0	21	17	4	CL-ML	CL-ML
◆	B-5	0.0-5.0	37	23	14	CL	CL
○	B-5	13.5-15.0	37	17	20	CL	SC
□	B-6	0.0-5.0	39	23	16	CL	CL
△	B-6	8.5-10.0	38	21	17	CL	CL
X	B-6	18.5-20.0	41	23	18	CL	CL
-	B-7	1.0-1.4	--	--	NP	ML	SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-13

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ATTERBERG LIMITS TEST RESULTS Method B

JW POWELL BOULEVARD EXTENSION

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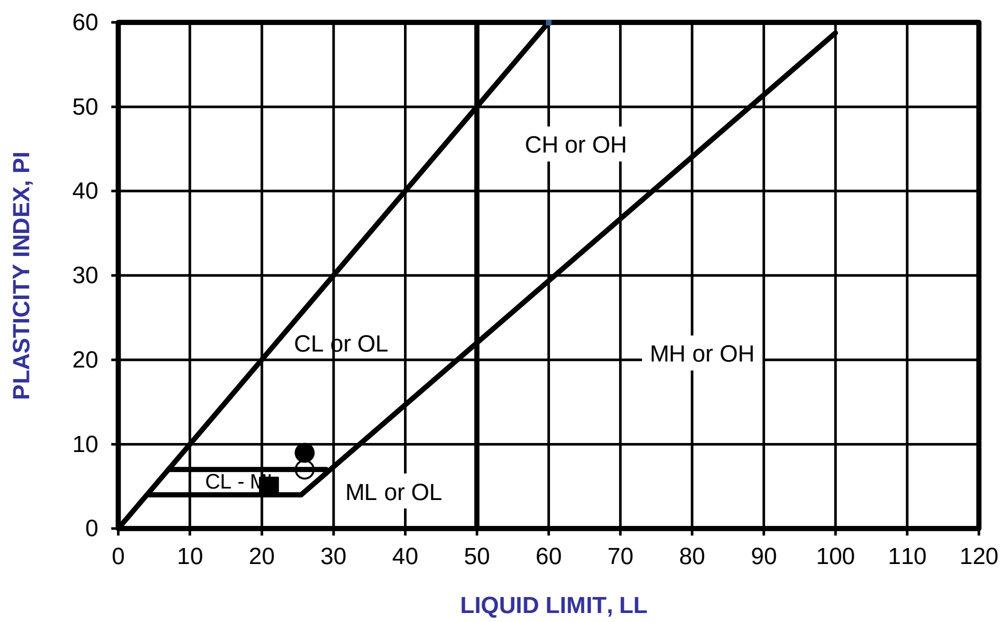
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-8	0.0-5.0	26	17	9	CL	CL
■	B-8	3.5-4.3	21	16	5	CL-ML	CL-ML
◆	B-9	0.0-5.0	--	--	NP	ML	ML
○	B-10	0.0-5.0	26	19	7	CL-ML	SC-SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-14

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ATTERBERG LIMITS TEST RESULTS Method B

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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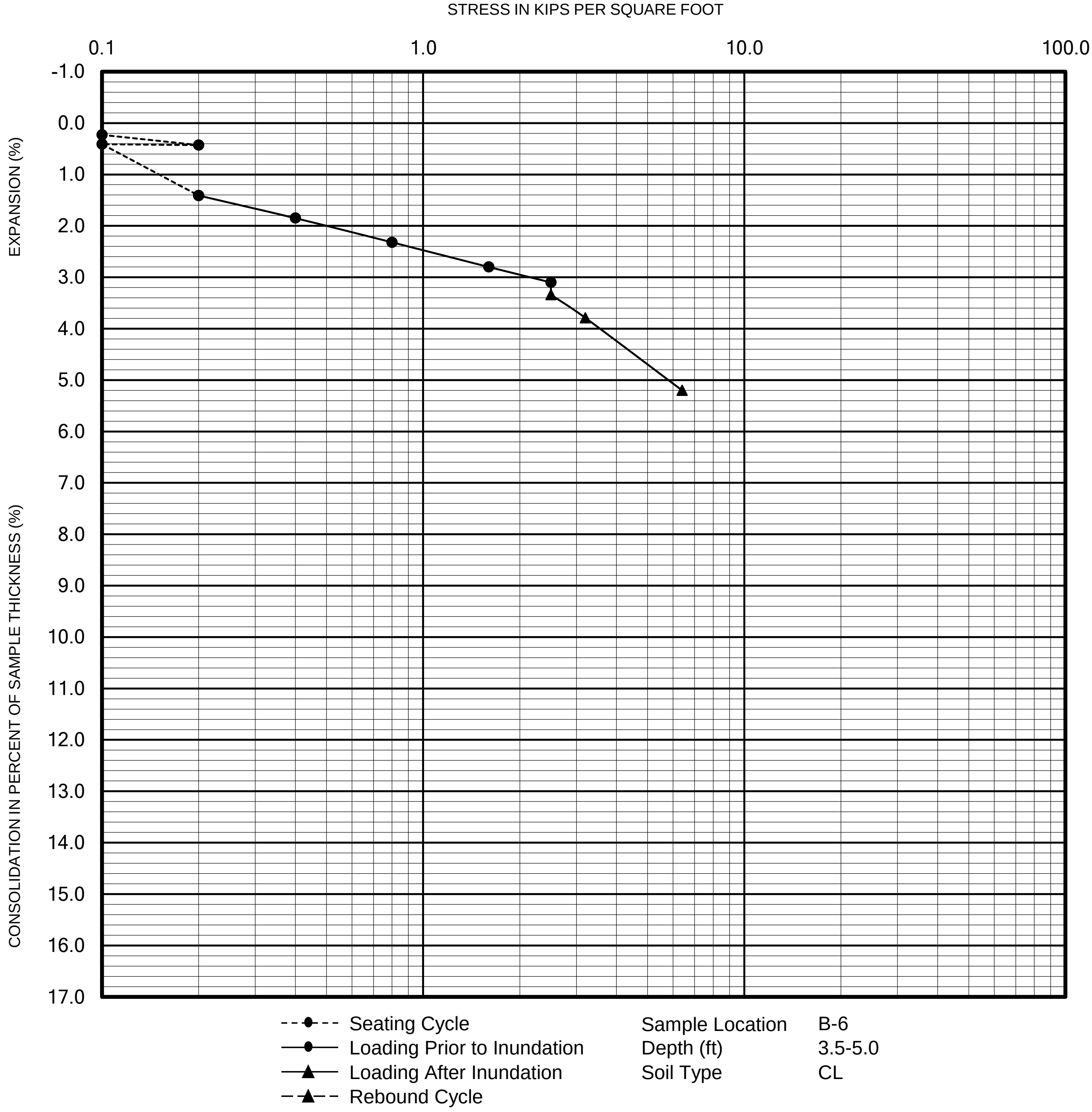

Sample Location	B-6
Depth (ft)	3.5-5.0
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435/4546

FIGURE B-15

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CONSOLIDATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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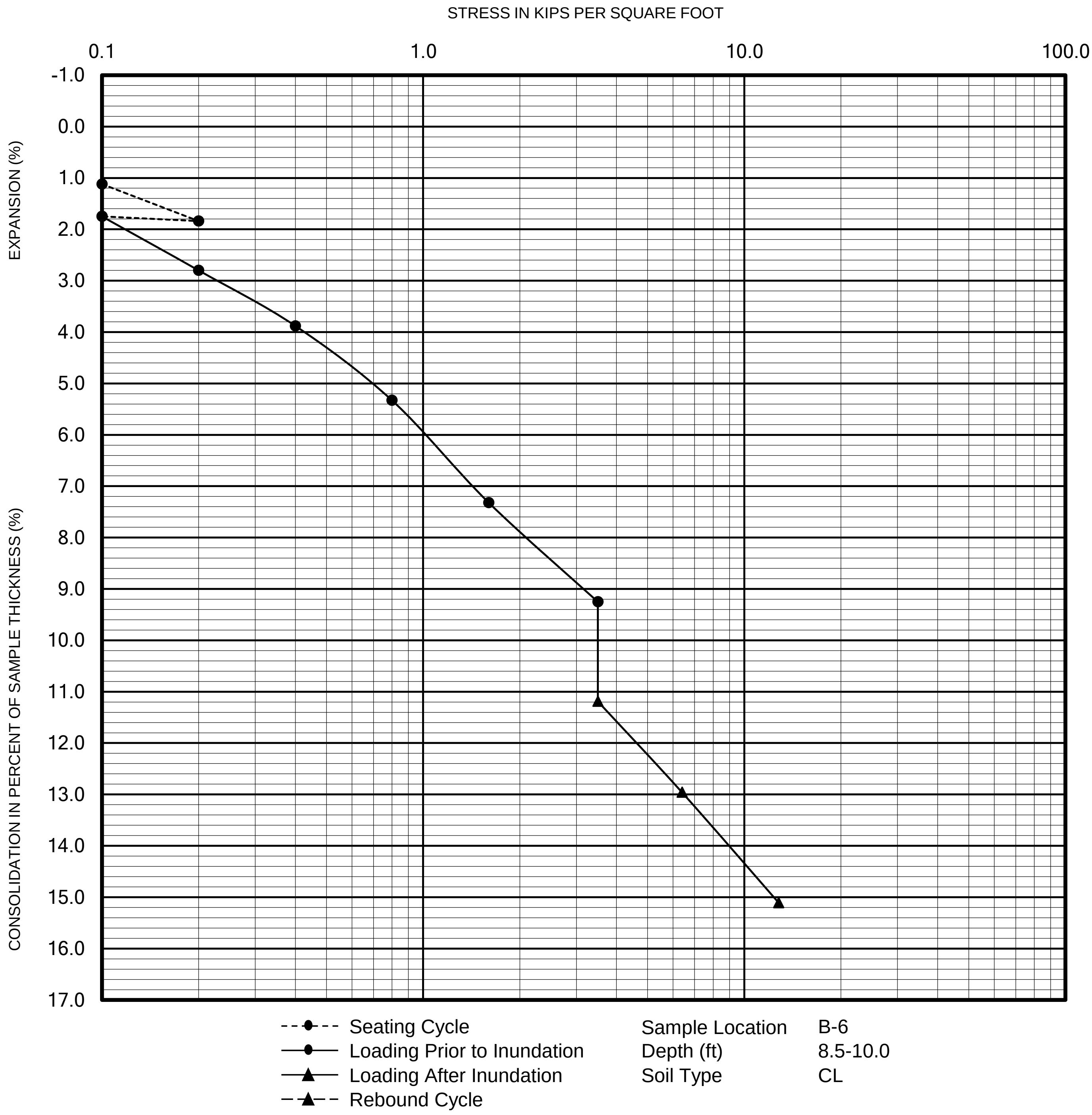

Sample Location	B-6
Depth (ft)	8.5-10.0
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435/4546

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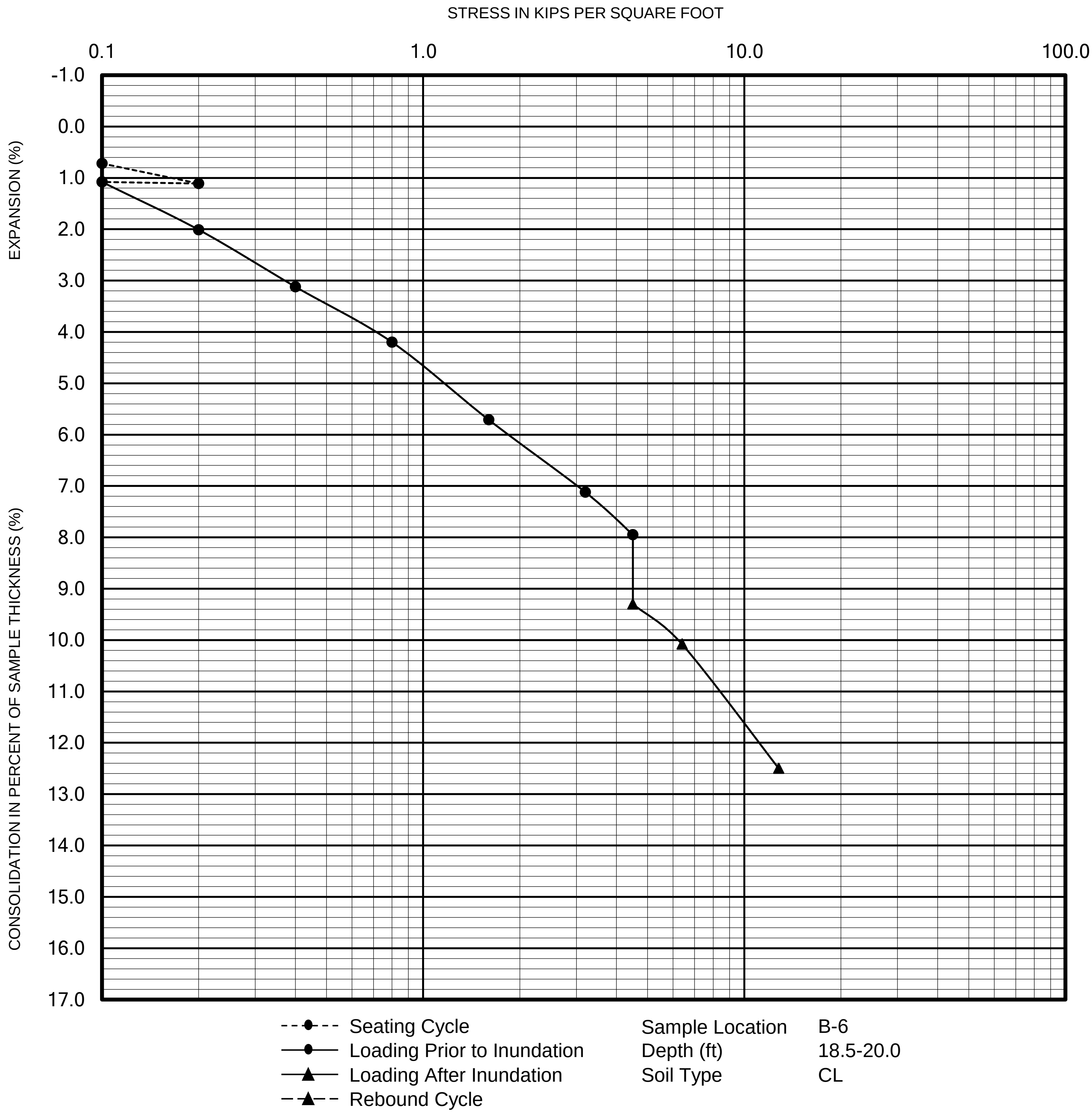
FIGURE B-16

CONSOLIDATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA



Sample Location	B-6
Depth (ft)	18.5-20.0
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435/4546

FIGURE B-17

CONSOLIDATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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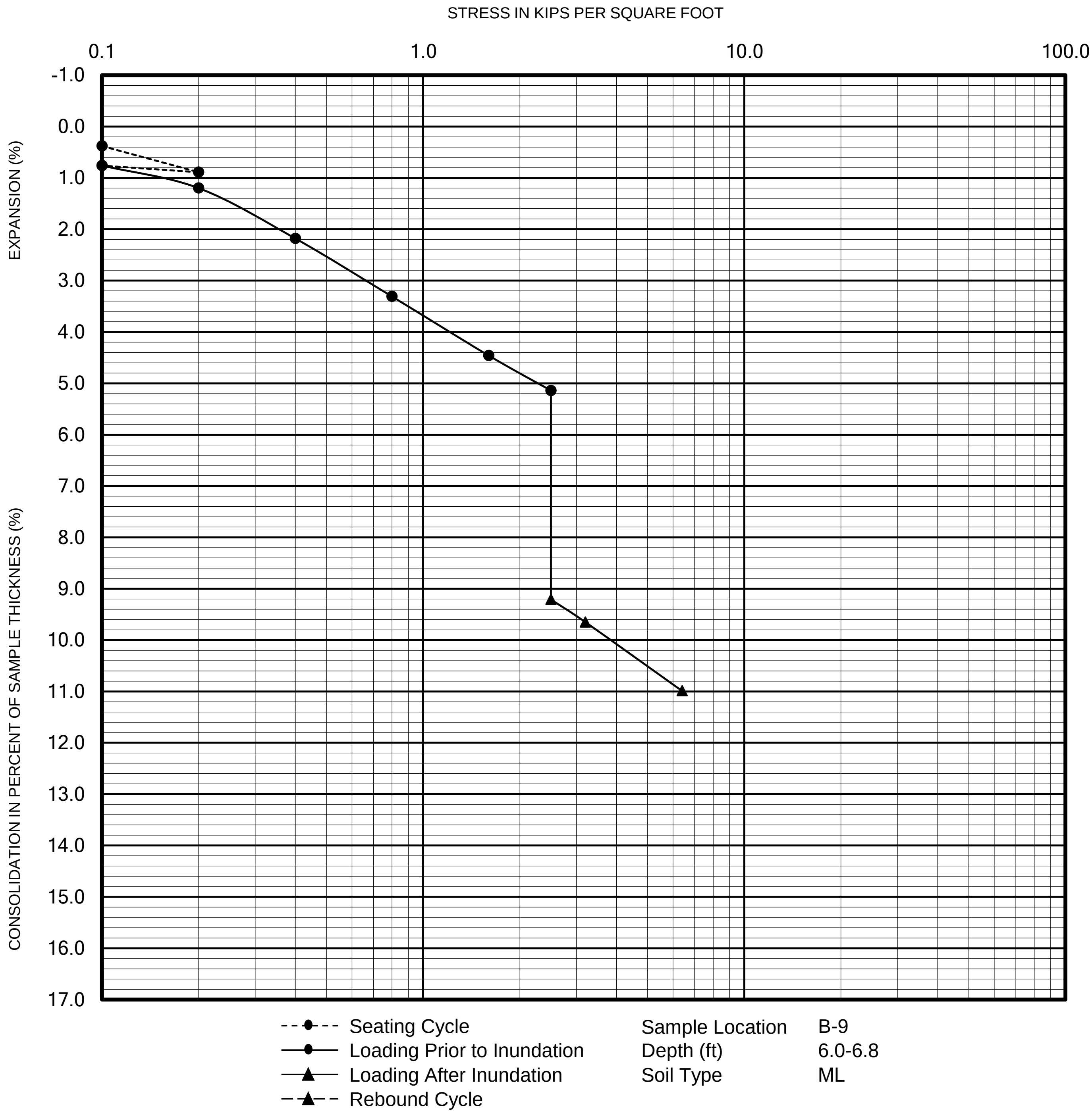

Sample Location	B-9
Depth (ft)	6.0-6.8
Soil Type	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435/4546

FIGURE B-18

CONSOLIDATION TEST RESULTS

JW POWELL BOULEVARD EXTENSION

FLAGSTAFF, ARIZONA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2] (ppm)	(%)	CHLORIDE CONTENT [3] (ppm)
B-1	0.0-5.0	6.8	1,559	154	0.015	6
B-5	0.0-5.0	7.3	1,356	112	0.011	16
B-8	0.0-5.0	8.1	2,034	117	0.012	5

[1] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 236e
[2] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 733b
[3] PERFORMED IN GENERAL ACCORDANCE WITH ARIZONA TEST METHOD 736b

APPENDIX C

Seismic Refraction Results

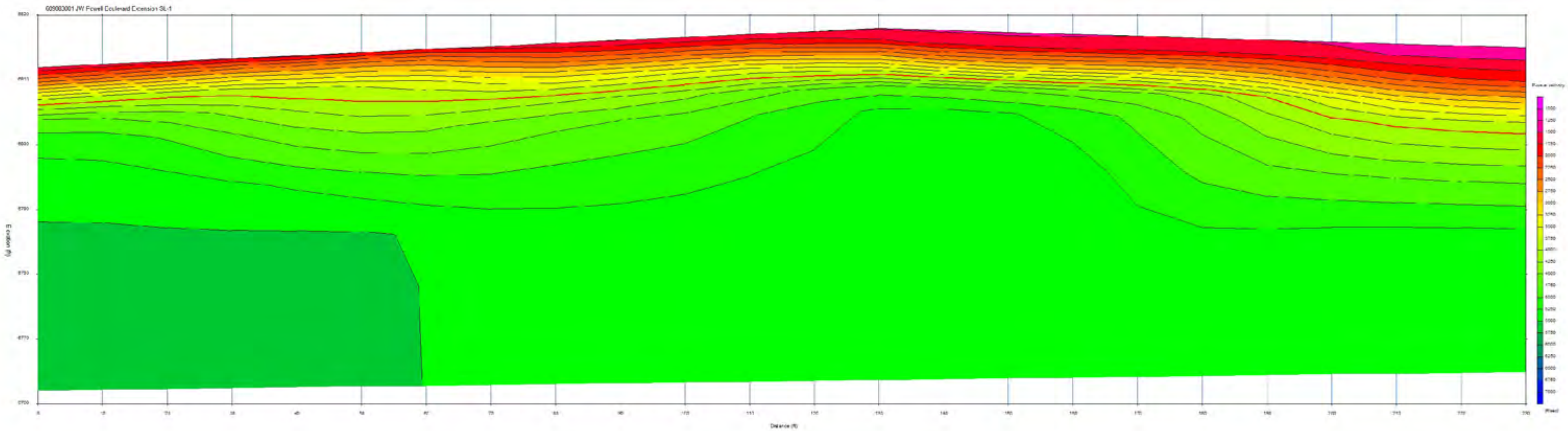

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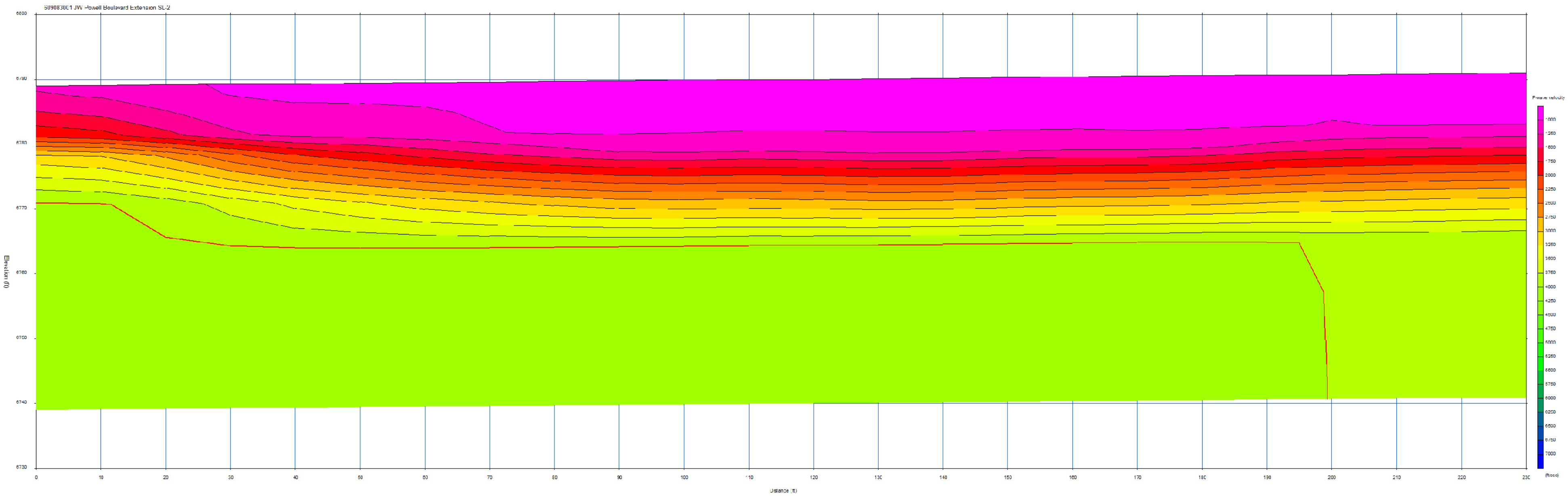
FIGURE C-1

SL-1 SEISMIC REFRACTION RESULTS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA



bsm file no: 9083sei1025b

FIGURE C-2

SL-2 SEISMIC REFRACTION RESULTS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

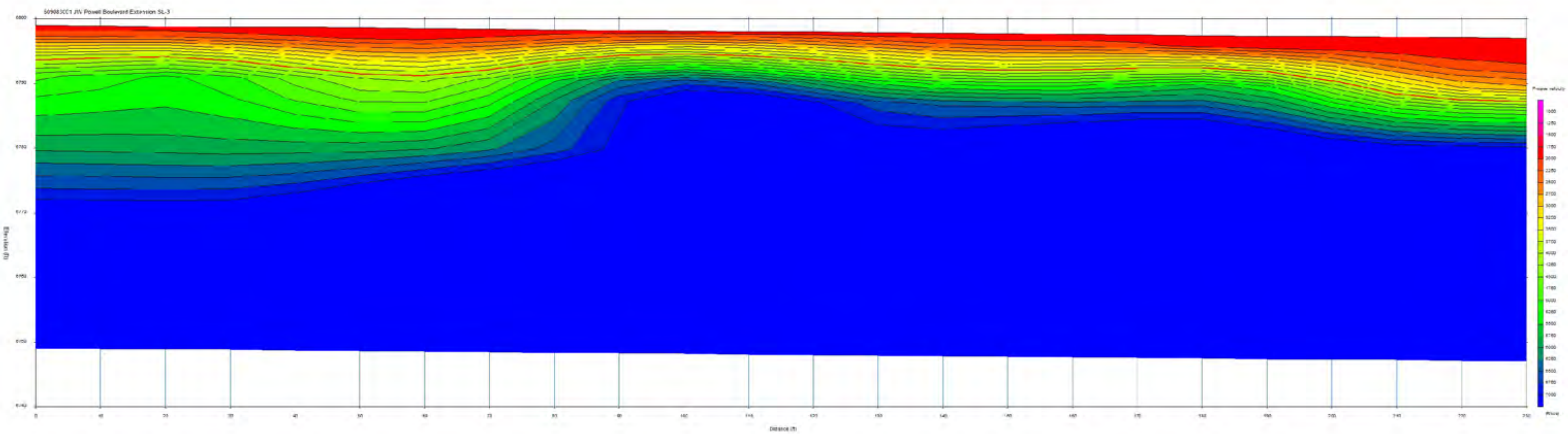


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FIGURE C-3

SL-3 SEISMIC REFRACTION RESULTS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

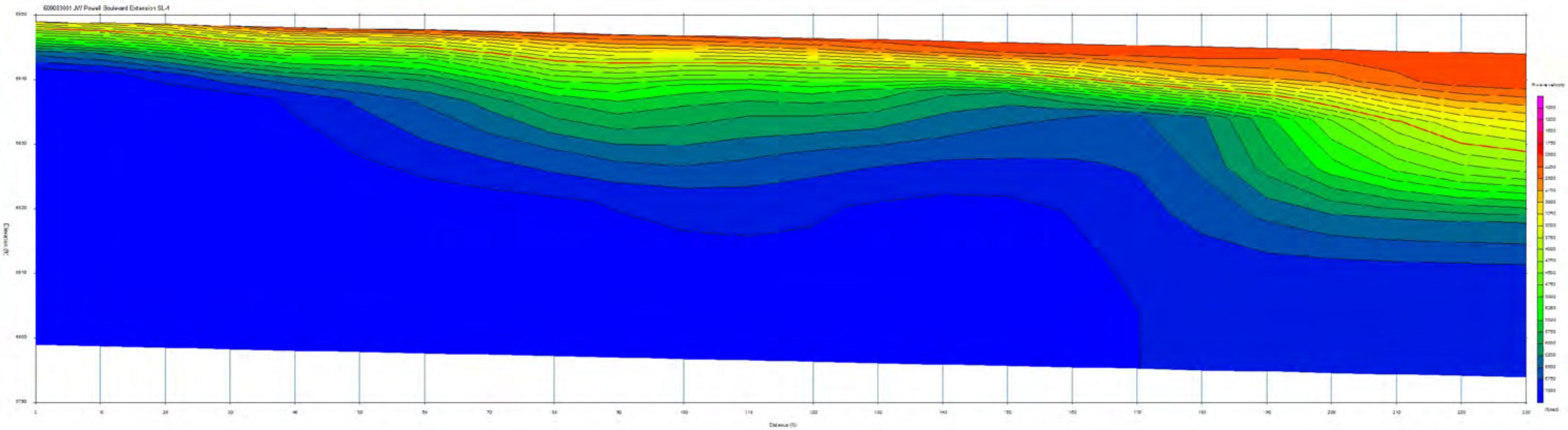

FIGURE C-4

SL-4 SEISMIC REFRACTION RESULTS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

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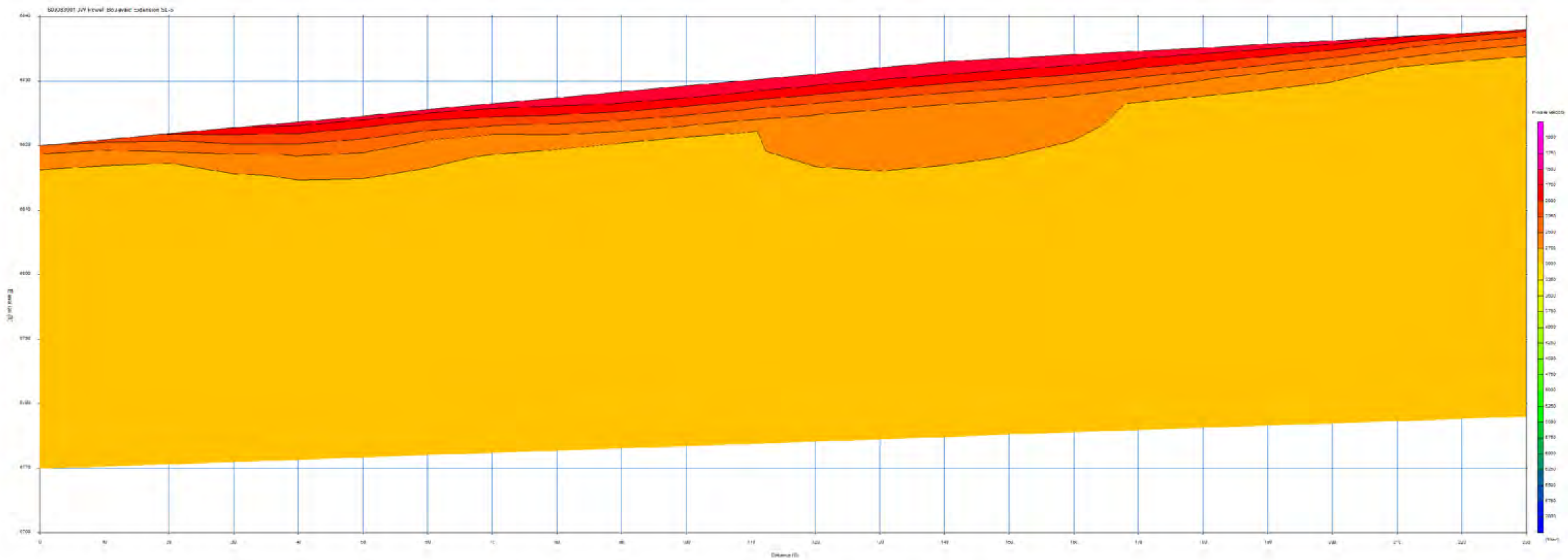

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FIGURE C-5

SL-5 SEISMIC REFRACTION RESULTS

JW POWELL BOULEVARD EXTENSION
FLAGSTAFF, ARIZONA

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3202 East Harbour Drive | Phoenix, Arizona 85034 | p. 602.243.1600

ninyoandmoore.com