Geotechnical Evaluation

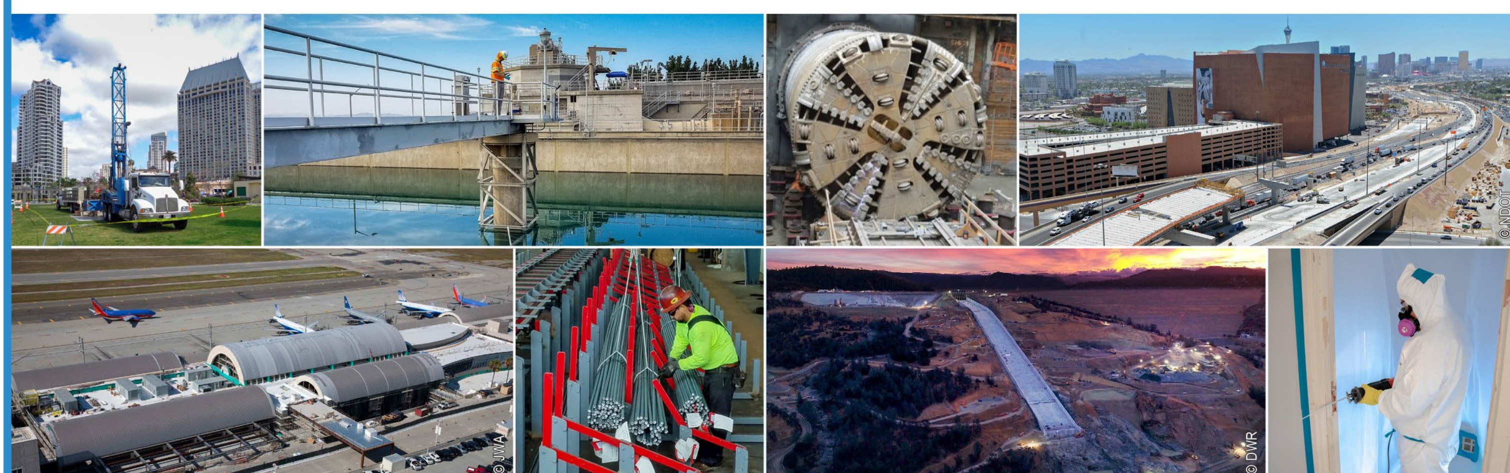
Cañada del Oro and Sutherland Wash Crossing Catalina State Park

Tucson, Arizona

Jacobs

1501 West Fountainhead Parkway, Suite 401 | Tempe, Arizona 85282

November 6, 2025 | Project No. 607775001



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Ninyo & Moore

A SOCOTEC COMPANY

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1. INTRODUCTION

In accordance with our proposal dated December 8, 2023, and your authorization, we have performed a geotechnical evaluation for the design and construction of a new multi-box drainage crossing located at the confluence of the Cañada Del Oro Wash and the Sutherland Wash within Catalina State Park, in Tucson, Arizona (Figure 1). This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

The scope of our services for this project generally included:

- Reviewing available published and in-house geotechnical reports, topographic information, soil surveys, geologic literature, and aerial photographs of the project area.
- Conducting a visual reconnaissance of the project area and marking out the boring locations.
- Notifying Arizona 811 of the proposed boring locations prior to conducting our field work.
- Drilling, logging, and sampling two exploratory soil borings to approximate depths of 30 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Performing laboratory tests on selected samples collected from our borings to evaluate the 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 results of the laboratory tests are included in Appendix B.
- Preparing this geotechnical report presenting our findings, conclusions, and recommendations regarding the proposed design and construction.

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 site is located near where the main entrance to the Catalina State Park in Tucson, Arizona crosses the confluence between the Cañada Del Oro and Sutherland Wash. The current crossing (a low water crossing) is approximately 200 feet in length and 34 feet in width. The Catalina State Park is located at the base of the Santa Catalina Mountains and within Pima County. The Park is maintained by Arizona State Parks and Trails.

At the time of our evaluation, the area surrounding the site was primarily undeveloped with sparse to dense desert vegetation, traversed with paved roadways to enter the Park and access campgrounds, unpaved paths and trails, and recreational ramadas. Oracle Road (State Highway 77) formed the Park's western boundary and the Pusch Ridge Wilderness formed the boundary on the south and east. Scattered residential developments were observed to the north of the site.

4. TOPOGRAPHIC MAP AND AERIAL PHOTOGRAPH REVIEW

According to the Oro Valley, Pima County, 7.5-Minute United States Geological Survey Topographic Quadrangle Map (2023) the average site elevation is approximately 2,690 feet relative to mean sea level. The topography of the site is relatively flat and slopes gently from northeast to southwest.

Several historical aerial photographs from Historicaerials.com (Nationwide Environmental Title Research) and from Google Earth™ were reviewed for this project. Aerial images dated 1975 depicted no development within the project area. An image dated 1983 depicted the Park's unpaved entrance road and an area west of the project site appeared to be cleared and grubbed for construction of a maintenance building. An image dated 1990 depicted the residential development north of the site. A photograph dated 1997 depicted the entrance road as being paved. An aerial image from 2003 depicted the construction of campgrounds within the Park and the campgrounds also appeared on an image dated 2004. An aerial image dated 2007 depicted an improved low water crossing where the Canada del Oro wash intersects the entrance road. Aerial photographs dated 2007 through 2023 depicted the project site and the surrounding areas as being generally similar to their current condition.

5. PROPOSED CONSTRUCTION

The project generally includes the design and construction of improvements to the current low water crossing along the entrance road of Catalina State Park. The anticipated traffic over the culvert will predominantly consist of passenger cars and light pickup trucks, with an occasional light pickup truck pulling trailers. No grading and drainage plans were available at the time of the study. We assume that the project will include moderate excavation and earthwork for the installation of a new multi-cell box culvert.

6. FIELD EXPLORATION AND LABORATORY TESTING

On October 2, 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 two exploratory borings (denoted as B-1 and B-2) using a CME 75 truck mounted drill rig equipped with hollow-stem augers (Figure 2). The borings were drilled near the footprint of the proposed box culver location to depths of approximately 30 feet below ground surface (bgs). Bulk and relatively undisturbed soil samples were collected at selected depth intervals in our borings.

Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) test method D2488 by observing cuttings and drive samples. Collected ring samples were trimmed in the field, wrapped in plastic bags, and placed in cylindrical plastic containers to retain in-place moisture conditions. Similarly, Standard Penetration Test (SPT) and bulk samples were sealed in plastic bags to retain their approximate in-place moisture. Detailed descriptions of the soils encountered are presented on the boring log in Appendix A.

The soil samples collected from our exploratory activities were transported to the Ninyo & Moore laboratory in Tucson, Arizona for geotechnical laboratory testing. The testing included 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 results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A and a description of each laboratory test method and the remainder of the test results is presented in Appendix B.

7. REVIEWED HISTORIC GEOTECHNICAL REPORTS

To evaluate the geology at the project site, we have reviewed several reports prepared by Ninyo & Moore from other projects in the general project area. The following sections summarize our review of these historic geotechnical reports.

- Oro Valley Marketplace Multi Use Path (April 6, 2020) – Ninyo & Moore conducted a geotechnical evaluation for the design and reconstruction of 1-mile long pavement multi use path that consisted of eight exploratory borings to depths of approximately 5 feet below ground surface (bgs) and laboratory testing. Field and laboratory testing indicate the fill materials encounter in some of the borings to be sandy lean clay, clayey sand and silty clayey sand. The laboratory test results also show that many subgrade soils along the project alignment are sensitive to moisture changes and exhibit volumetric expansion upon saturation.

- Quail Ridge Estates Manufactured Home Community (April 28, 2021) – Ninyo & Moore conducted a geotechnical evaluation for the design and construction of an expansion to the manufactured home community that consisted of four borings to approximate depths of 15 feet bgs and laboratory testing. Based on field exploration and laboratory testing, the site’s soils consist of native alluvial soils such as clayey sand and silty clayey sands with varying percentages of gravel. These soils ranged in density from loose to dense in the upper 10 feet and were very dense below approximately 10 feet bgs. The on-site soils also exhibited a sudden reduction in volume (collapse) on the order of 11 percent when inundated with water.

- Extra Space Storage Expansion (September 21, 2022) – Ninyo & Moore conducted a geotechnical evaluation consisting of 11 borings from 5 to 24 feet bgs and laboratory testing. These soils consisted of native alluvial soils such as silty sand and silty clayey sand with vary percentages of gravel, ranging in relative density from loose to very dense. Corrosivity test results indicate that these on-site soils are corrosive to ferrous materials. Additionally, these soils may be compressible upon saturation.

8. GEOLOGY AND SUBSURFACE CONDITIONS

The project site is located in the Sonoran Desert Section of the Basin and Range physiographic province, which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 18 million years ago during the mid- to late-Tertiary age. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins were filled with alluvium from the erosion of the surrounding mountains as well as from deposition from rivers. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains.

The surface geology at the site is described as Quaternary-age (0 to 10,000 years) active stream channels, relatively dissected alluvial fans and low stream terrace deposits. The youngest deposits are active or recently abandoned stream channels or fans, these deposits have essentially no soil development. Older deposits include low stream terraces and abandoned alluvial fans that have been stable long enough to permit modest soil development and are manifested as thin accumulations of calcium carbonate on bottoms of gravel clasts or as fine filaments. The oldest deposits show some evidence of clay accumulation in cambic or weak argillic horizons. (Pearthree, et al, 1998). According to the Natural Resources Conservation Service Web Soil Survey by the United States Department of Agriculture, various soil types are located within the project area. The predominant soil types are described in Table 1 below.

Table 1 – NRCS Soil Units

Soil Map Unit Name	Description of Soil Units
Comoro sandy loam	Sandy loam, fine sandy loam
Keysto extremely gravelly fine sandy loam	Extremely gravelly fine sandy loam, extremely gravelly loam, very gravelly loam, extremely gravelly sandy clay loam

Note:
Loam is an agricultural soil classification that refers to a soil comprised of a mixture of clay, silt, and sand

8.1. Subsurface Conditions

The boring logs contain our field and laboratory test results, as well as our interpretation of conditions believed to exist between actual samples retrieved. Therefore, these 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 (strata) 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.

Native alluvial soils were encountered at the surface of our borings and extended to boring termination depths of 30 feet bgs. This material generally consisted of medium dense to dense well-graded sand, silty sand, clayey sand, and poorly graded sand with varying percentages of gravel and cementation in our borings.

8.2. Groundwater

No groundwater was encountered in our borings. Based on well data provided by the Arizona Department of Water Resources, groundwater has been historically measured at a depth on the order of 90 feet bgs. However, it should be noted that groundwater levels near the site can fluctuate due to seasonal variations, flows in nearby washes, irrigation, groundwater withdrawal or injection, and other factors. Because of the close proximity of the Cañada Del Oro and Sutherland Wash to the project site, transient groundwater may be at or near the ground surface after certain storm events.

9. GEOLOGIC HAZARDS

The following section provides a discussion regarding potential geologic hazards such as land subsidence, earth fissures, faulting, and seismicity.

9.1. Land Subsidence and Earth Fissures

Groundwater depletion, due to groundwater pumping, has caused land subsidence and earth fissures in numerous alluvial basins in Arizona. It has been estimated that subsidence has affected more than 3,000 square miles and has caused damage to a variety of engineered structures and agricultural land. From 1948 to 1983, excessive groundwater withdrawal has been documented in several alluvial valleys where groundwater levels have been reportedly lowered by up to about 500 feet. With such large depletions of groundwater, the alluvium has undergone consolidation resulting in large areas of land subsidence (Schumann and Genualdi, 1986).

In Arizona, earth fissures are generally associated with land subsidence and pose an on-going geologic hazard. Earth fissures generally form near the margins of geomorphic basins where significant amounts of groundwater depletion have occurred. Reportedly, earth fissures have also formed due to tensional stress caused by differential subsidence of the unconsolidated alluvial materials over buried bedrock ridges and irregular bedrock surfaces (Schumann and Genualdi, 1986).

Based on our field reconnaissance and review of the referenced material, there are no known earth fissures at the surface of the subject site. Based on fissure maps published by the Arizona Geological Survey (AZGS, 2014), the closest reported unconfirmed earth fissures to the site are located approximately 17 miles to the west of the project site near the Town of Marana.

Continued groundwater withdrawal in the area may result in subsidence and the formation of new fissures or the extension of existing fissures. While the future occurrence of land subsidence and earth fissures cannot accurately be predicted, these phenomena are not expected to be a constraint to the construction of this project.

9.2. Faulting and Seismicity

The site lies within the Sonoran zone, which is a relatively stable tectonic region located in southwestern Arizona, southeastern California, southern Nevada, and northern Mexico (Euge et al., 1992). This zone is characterized by sparse seismicity and few Quaternary faults. Based on our field observations and on our review of readily available published geologic maps and literature, there are no known active faults underlying the subject site or adjacent areas. The closest known Quaternary fault to the site is the Santa Rita Fault Zone, located approximately 32 miles southeast of the site. The Santa Rita Fault Zone is situated along the western piedmont of the Santa Rita Mountains. The fault zone is a series of northeast-striking normal faults that dip to the northwest. The most recent movement along this fault was approximately 130,000 years ago during the Middle to Late Pleistocene epoch. The slip-rate category of this fault is less than 0.2 millimeters per year (Pearthree, 1998). Seismic parameters recommended for the design of the proposed improvements are presented in Section 11.2.

10. CONCLUSIONS

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

- The near-surface site soils can generally be excavated or ripped using heavy-duty earthmoving or excavation equipment.
- Soils of variable relative densities encountered in our borings may be sensitive to moisture upon saturation.
- Imported soils and soils generated from on-site excavation activities, that exhibit a relatively low plasticity index (PI) can generally be used for engineered fill. Based on the results of our study, many of the on-site soils are suitable for re-use as engineered fill.

- The proposed culvert can be founded on mat and spread / continuous footings. These footings should be founded on a zone of adequately moisture-conditioned and compacted engineered fill.
- Slabs on grade, pavements and flatwork should be founded on compacted engineered fill.
- Groundwater was not observed in our borings. Based on well data from USGS, the regional groundwater table has been historically measured at depths as shallow as 90 feet bgs. Because of the close proximity of the Cañada Del Oro and Sutherland Wash to the project site, transient groundwater may be at or near the ground surface after certain storm events.
- No documented geologic hazards are present underlying or immediately adjacent to the site.

11. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the project's design and construction. If the proposed construction is changed from that discussed in this report, Ninyo & Moore should be contacted for additional recommendations.

11.1. Earthwork

The following sections provide our earthwork recommendations for this project. In general, the earthwork specifications contained in the Pima Association of Governments Standard Specifications for Public Improvements (Standard Specifications) are expected to apply unless specifically noted.

11.1.1. Site Preparation

Construction areas should be cleared of deleterious materials, if any are present, construction debris, vegetation, and any other material that might interfere with the performance or progress of the work. These materials should be disposed of at a legal dumpsite. Existing features that are expected on this site call for relocation or removal and extend below finish grade should be removed, and the resulting excavations backfilled with compacted engineered fill as discussed in this report.

11.1.2. Excavations

Our evaluation of the excavation characteristics of the on-site soils is based on site observations and experience with similar soils. The on-site soils include medium dense to dense well-graded sand, poorly graded sand, clayey sand, and silty sand with varying percentages of gravel and cementation; which can generally be excavated or ripped using heavy-duty earthmoving or excavation equipment.

Equipment and procedures should be used that do not cause significant disturbance to the excavation bottoms. Excavators and backhoes with buckets having large claws to loosen the soil should be avoided when excavating the last 6 to 12 inches. Such equipment will probably disturb the excavation bases. If wet or saturated soils are found at the excavation bases, these soils may soften under the action of light equipment and foot traffic. If the subgrade becomes disturbed, it should be compacted before placing the backfill material.

11.1.3. Temporary Slopes

Sidewalls for temporary excavations should not be anticipated to stand near-vertical without sloughing. Therefore, the contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration (OSHA) regulations, for employees working in an excavation that may expose them to the danger of moving ground. For planning purposes and according to OSHA soil classifications, a "Type C" soil should be considered for this project. This corresponds to a temporary slope inclination no steeper than 1.5:1 (horizontal to vertical [H:V]). During excavation, soil classification and excavation performance should be evaluated in the field by Ninyo & Moore in accordance with the OSHA regulations.

11.1.4. Permanent Slopes

We recommend that permanent cut slopes be constructed no steeper than 2.5:1 (H:V). Permanent fill embankment slopes should be no steeper than 3:1 (H:V). 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. It should be noted that slopes steeper than 3:1 (H:V) may be subject to erosion. Erosion control measures such as deep-rooted perennial vegetation, gravel mulch, riprap, geotextiles, gabion mats, concrete lining, or bio-reinforcement may be considered.

11.1.5. Temporary Shoring

It may be preferable to shore or brace the trenches and pits rather than utilize open cuts to the base of the excavations. Temporary earth retention systems may include braced systems, such as trench boxes or shields with internal supports or cantilevered systems like soldier piles and lagging; however, the risk of excessive lateral deflections/movements should be considered when designing a cantilevered shoring system for the project.

Braced temporary earth retention systems may be designed using the parameters presented on Figure 3. The earth pressure values provided on Figure 3 assume the shoring system will be constructed without raising the ground surface elevation behind the shoring system, and the 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. For earth retention systems subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the design lateral earth pressures. We recommend that an experienced structural engineer design the shoring system. The shoring parameters presented in this report should be considered as guidelines.

We anticipate that settlement of the ground surface will occur behind shoring systems during excavation. The amount of settlement depends heavily on the type of shoring system used, the contractor's workmanship, and soil conditions. We recommend that roadways, utilities, and structures in the vicinity of the planned shoring installation be reviewed with regard to foundation support and tolerance to settlement. To reduce the potential for distress to adjacent structures, we recommend that the retaining system be designed to limit the ground settlement behind the shoring to ½-inch or less. Possible causes of settlement that should be addressed include settlement during excavation for construction of new structures, construction vibrations, de-watering, and removal of the support system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction.

11.1.6. Fill Materials and Reuse of On-site Soils

On-site and imported soils that exhibit relatively low plasticity indices and very low to low expansive potential are generally suitable for re-use as engineered fill. Relatively low plasticity indices are defined as a PI value of 15, or less, as evaluated by ASTM D4318. Very low to low expansive potential soils are defined as having an Expansion Index (evaluated in accordance with ASTM D4829) of 50 or less. Based on laboratory test results, the on-site soils are characterized by PI values of 0 (non-plastic). As such, we anticipate that many of the on-site soils will be suitable for re-use as engineered fill during construction. The Contractor should perform additional testing prior to or during construction to better understand the site's soil conditions. Soils not suitable for re-use as engineered fill can be re-used in non-structural areas (e.g., landscaping, etc.).

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

Fill materials in contact with ferrous metals should also have low corrosion potential (minimum resistivity more than 2,000 ohm-cm, chloride content less than 25 parts per million). Fill material in contact with concrete should have a soluble sulfate content of less than 0.1 percent.

11.1.7. Subgrade Preparation

Our borings disclosed near-surface alluvial deposits generally consisted of well-graded sand, poorly graded sand, clayey sand and silty sand with varying percentages of gravel. Some of the soil encountered in our borings are compressible and/or exhibited a potential for collapse upon inundation with water. Accordingly, we recommend that new foundations be supported on a zone of engineered fill that extends 3 feet below the footing base. The engineered fill should be moisture-conditioned and compacted as discussed in Section 11.1.8. This overexcavation zone should extend a horizontal distance from the edge of the foundation footprint that is equal to the depth of the overexcavation.

In addition, we recommend that grade slabs, pavements and flatwork be supported on 12 inches of moisture-conditioned and compacted engineered fill as discussed in Section 11.1.8. This can be achieved by overexcavation or in-place scarification. The fill thickness should be measured from the bottom of the aggregate base (AB) layer, where applicable. This subgrade improvement should extend laterally 1 foot beyond the slab/pavement footprint.

After the overexcavation described above is finished and prior to the placement of engineered fill, exposed surfaces from excavations should be carefully evaluated by Ninyo & Moore 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 of the exposed surface. This additional remediation, if needed, should be addressed by the geotechnical consultant during the earthwork operations.

11.1.8. Fill Placement and Compaction

Fill soils should be moisture-conditioned within the moisture range shown below in Table 2 and mechanically compacted to the percent compaction shown. 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.

Table 2 – Compaction Recommendations		
Description	**Percent Compaction per ASTM D698**	**Moisture Content**
Below foundations, box culverts, grade slabs, exterior flatwork, and pavement	95 percent	±2 percent of optimum
AB	100 percent	-1 to +3 percent of optimum
Trench Backfill – within 2 feet below pavements	100 percent	±2 percent of optimum
Trench Backfill – deeper than 2 feet below pavement	95 percent	±2 percent of optimum
Pipe Bedding/Pipe Zone	90 percent	±2 percent of optimum

An earthwork (shrinkage) factor of 5 to 15 percent is estimated. This shrinkage factor range represents an average of the materials encountered on nearby projects 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.

11.2. Seismic Design Considerations

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 International Building Code (IBC) guidelines and adjusted maximum considered earthquake spectral response acceleration parameters evaluated using the California's Office of Statewide Health Planning and Development Seismic Design Maps (web-based).

Table 3 – IBC Seismic Design Criteria	
Site Coefficients and Spectral Response Acceleration Parameters	**Values**
Site Class	D
Site Coefficient, F_a	1.585
Site Coefficient, F_v	2.40
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	0.268 g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.083 g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.425 g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.199 g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.284 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.133 g

11.3. Foundations

Based upon our background review we are providing recommendations for spread or continuous footings and mat foundations. We understand that the foundation system used will be selected by others based on loading conditions, settlement tolerances of the structures, as well as associated costs and constructability.

11.3.1. Spread or Continuous Footings

Spread or continuous footings may be used to support ancillary wing wall structures and should be supported at a depth of 18 inches or more below finished grade, bearing on engineered fill in accordance with recommendations presented in Section 11.1.7. Continuous footings should have a width of 16 inches or more, and isolated column footings should have a width of 24 inches or more. Footings may be designed using the allowable net bearing pressures for static conditions of 2,000 to 3,000 pounds per square foot. The allowable soil bearing pressure may be increased by one-third when considering total loads including loads of short duration such as wind or seismic forces.

Total and differential settlement of 1-inch and 1/2-inch over a horizontal distance of 40 feet, respectively, may occur. These settlement estimates are based on the estimated loading conditions, the available soil boring information, and our experience with similar soils. These settlements are contingent on the preparation of soils underlying the foundations in accordance with the recommendations contained in Section 11.1.7 of this report and in compliance with the drainage recommendations presented in Section 11.9 of this report.

Foundations bearing on engineered fill and subject to lateral loadings may be designed using an ultimate coefficient of friction of approximately 0.35 to 0.40 (total frictional resistance equals the coefficient of friction multiplied by the dead load).

An ultimate passive resistance value of 370 psf per foot of depth may be used up to a value of 3,700 psf. The ultimate 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 its kern (i.e., middle one-third).

11.3.2. Mat Foundation

Mat foundation (double reinforced slab) may be used to support the box culvert structure and is recommended for any foundation loads more than about 500 pounds per square foot (psf), while conventional concrete slabs may be considered for more lightly loaded foundations. For mat foundations more than 5 feet in width, the average bearing pressure should not exceed an allowable equivalent uniform bearing pressure of 3,000 psf. However, peak edge stresses may exceed this value as long as the resultant passes through the middle third of the foundation base. The allowable soil bearing pressure may be increased by one-third when considering total loads including loads of short duration such as wind or seismic forces. Ninyo & Moore should be contacted in the event that higher loadings or special situations are needed.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design modulus of subgrade reaction, K, of 150 tons per cubic foot (tcf) in engineered fill soils may be used for evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large-sized mats. Adjusted values of the modulus of subgrade reaction, K, can be obtained from the following equation for mats of various widths:

$$K = 150\left[\frac{(B+1)}{2B}\right]^2 \text{ (tcf)};$$

where B is the width of mat measured in feet

11.4. Lateral Earth Pressures

Earth pressures are calculated to compute the lateral forces acting on retaining structures such as basement walls, and foundations. These pressures can be classified as at-rest, active, and passive. The active pressures are exerted when the wall moves out or rotates slightly (typical for cantilever retaining walls) and the soil moves toward the wall away from the mass, thereby mobilizing the shear strength of the soil. The active pressures are typically mobilized at movements of less than about 0.1 percent of the wall height for granular soils. Passive pressures occur when the wall moves toward the soil mass. At-rest conditions exist when there is insufficient movement to fully mobilize active or passive pressures, such as a braced or rigid wall.

Retaining walls that are not restrained from movement at the top and have a level granular structure backfill behind the wall may be designed using an “active” equivalent fluid unit weight of 39 pounds per cubic foot (pcf). This value assumes compaction within about 5 feet of the wall will be accomplished with relatively light compaction equipment, and that engineered granular fill will be placed behind the wall. Unrestrained retaining walls should also be designed to resist a surcharge pressure of 0.33q. The value for “q” represents the pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

The “at-rest” earth pressure against walls that are restrained at the top or braced so that they cannot yield, and with level backfill, may be taken as equivalent to the pressure exerted by a fluid weighing 55 pcf. Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.50q. The value of q represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

For “passive” resistance to lateral loads, we recommend that an ultimate equivalent fluid pressure of 330 psf per foot of depth be used. This value assumes that the ground is horizontal for a distance of 10 feet or more in front 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 be neglected when calculating passive resistance. If passive and frictional resistances are to be used in combination, we recommend that the passive resistance be limited to one-half of the ultimate lateral resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Measures should be taken so that moisture does not build up behind the basement walls. The basement walls should be backfilled with structure fill and provided with a drain. Drainpipes should outlet away from structures, and the walls should be waterproofed in accordance with the recommendations of the project civil engineer or architect. To reduce the potential for water- and sulfate/salt-related damage to the walls, particular care should be taken in selection of the appropriate type of waterproofing material to be utilized and in the application of this material.

11.5. Pavement Section

This section presents our recommendations for the new pavement section anticipated for the project that includes the entrance road for Catalina State Park. Detailed traffic information for the planned roadway and drive was not available. It is assumed that the roadway will be subjected mostly to passenger cars, light pick-up trucks, and only occasional traffic of delivery or trash collection trucks. Based on our experience with similar projects, the new AC pavement section is proposed as a result of our background review, which does not include any traffic analysis or laboratory testing. Subgrade preparation recommendations outlined in this report in Section 11.1.7 should be employed. The recommended pavement section is summarized in Table 4.

Table 4 – Recommended Structural Pavement Section

Pavement Section	Aggregate Base (inches)	Asphalt Concrete (inches)
Light-duty	4.0	3.0

11.6. Corrosion

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on the foundations and structures. Corrosion potential was evaluated using the results of laboratory testing of soil samples obtained during our subsurface evaluation that were considered representative of soils at the subject site.

Laboratory testing consisted of pH, minimum electrical resistivity, and chloride and soluble sulfate contents. The results of the corrosivity tests are presented in Appendix B.

The soil pH value of the tested samples varied between 7.3 and 7.5, which is considered to be alkaline. The minimum electrical resistivity measured in the laboratory ranged between 13,400 and 17,420 ohm-cm, which is considered to be negligible to ferrous metals. The chloride content of the samples tested was not detectible at a practical quantitation limit of 10 parts per million (ppm), which is considered to be negligible environment to ferrous metals. The soluble sulfate content of the soil samples tested was not detectible at a practical quantitation limit of 0.005 percent by weight or 50 ppm, which is considered to represent negligible to sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials are neglible to ferrous materials. However, it is possible that soils with variable corrosivity characteristics may be encountered at the site and should be evaluated during construction. As such, special consideration should be given to the use of heavy-gauge, corrosion-protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. To minimize corrosion of buried metallic utilities, we recommend that topsoil, organic soils, existing fill 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.

11.7. Concrete

Laboratory chemical tests performed on soil samples indicated sulfate content of approximately 0.005 percent by weight or less. Based on American Concrete Institute (ACI), the on-site soils should be considered to represent negligible sulfate exposure to concrete.

We recommend the use of Type II cement 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.

The concrete should have a water-cementitious materials ratio of no more than 0.50 by weight for normal weight aggregate concrete. The structural engineer should 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.

We recommend that concrete cover over reinforcing steel for foundations be in accordance with the recommendations of the structural engineer. The structural engineer should be consulted for additional concrete specifications.

11.8. 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; however, drought tolerant planting design as well as low-pressure, drip irrigation utilizing a master valve and flow sensor may be used within 5 feet of the building foundation. Nevertheless, irrigation associated with landscaping may introduce significant amounts of water to the foundation soils.

12. LIMITATIONS

The evaluations 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.

13. REFERENCES

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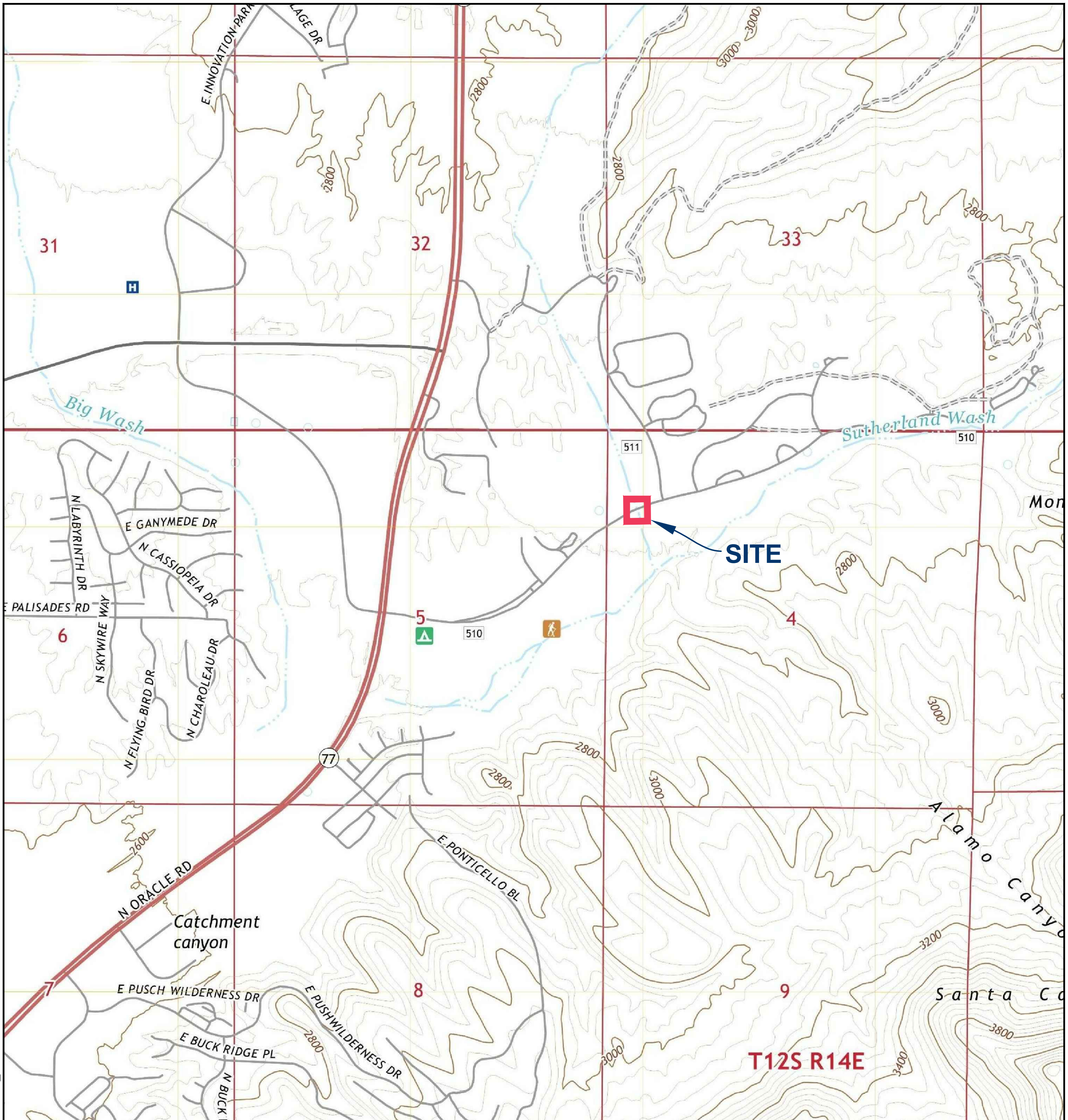
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS,

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FIGURE 1

SITE LOCATION

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CANADA DEL ORO AND SUTHERLAND WASH CROSSING
CATALINA STATE PARK
TUCSON, ARIZONA

607775001 I 11/25

LEGEND

B-2 ⊕ BORING

N

FEET

0 150 300

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2025.

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FIGURE 2

BORING LOCATIONS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING
CATALINA STATE PARK
TUCSON, ARIZONA

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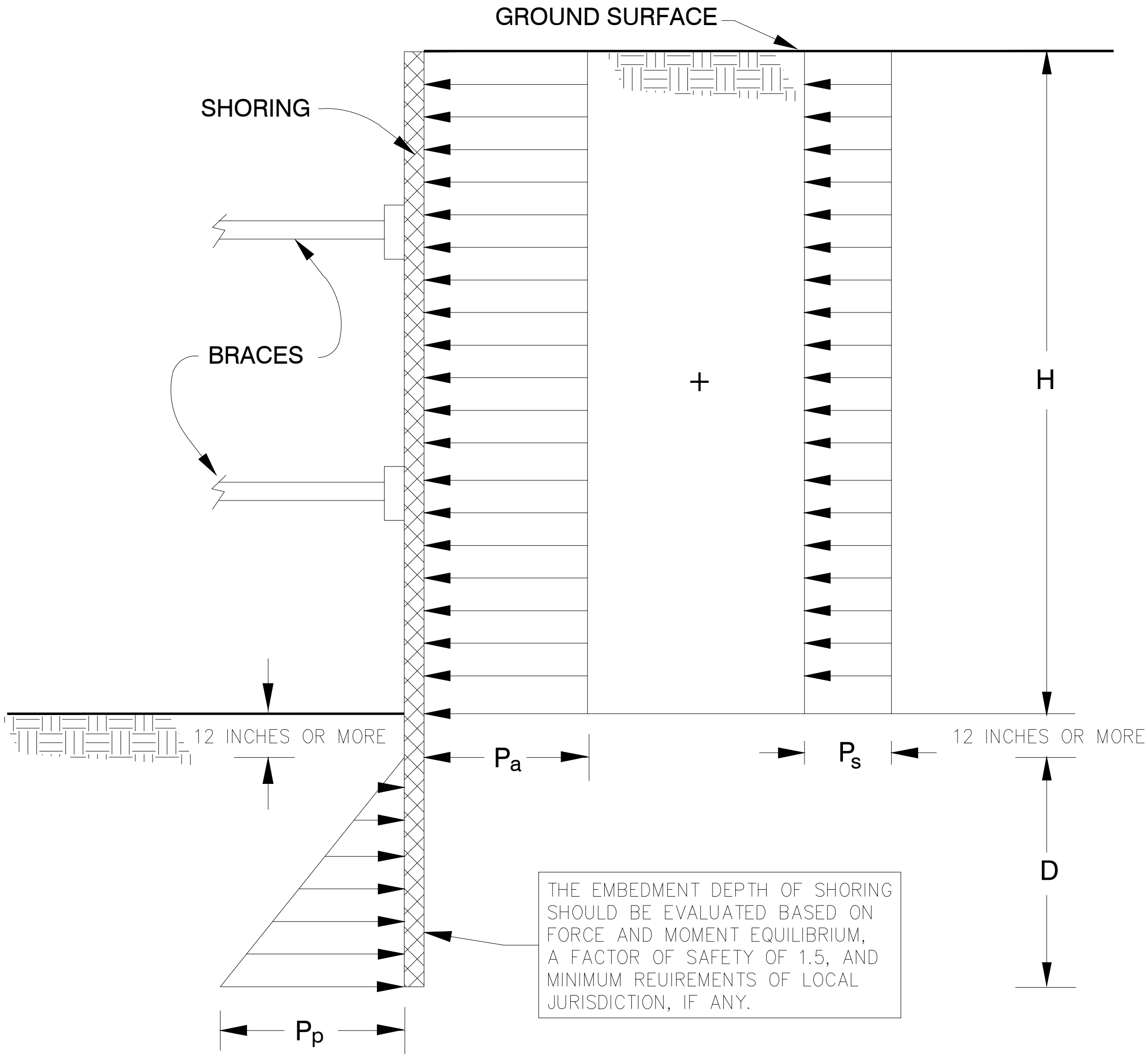

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 23H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 330 D 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

FIGURE 3

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (GRANULAR SOIL)

CANADA DEL ORO AND SUTHERLAND WASH CROSSING
CATALINA STATE PARK
TUCSON, ARIZONA
607775001 I 11/25

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Figure 4
Factored Bearing Resistance Chart
Retaining Wall Footings at Elevation 2,684

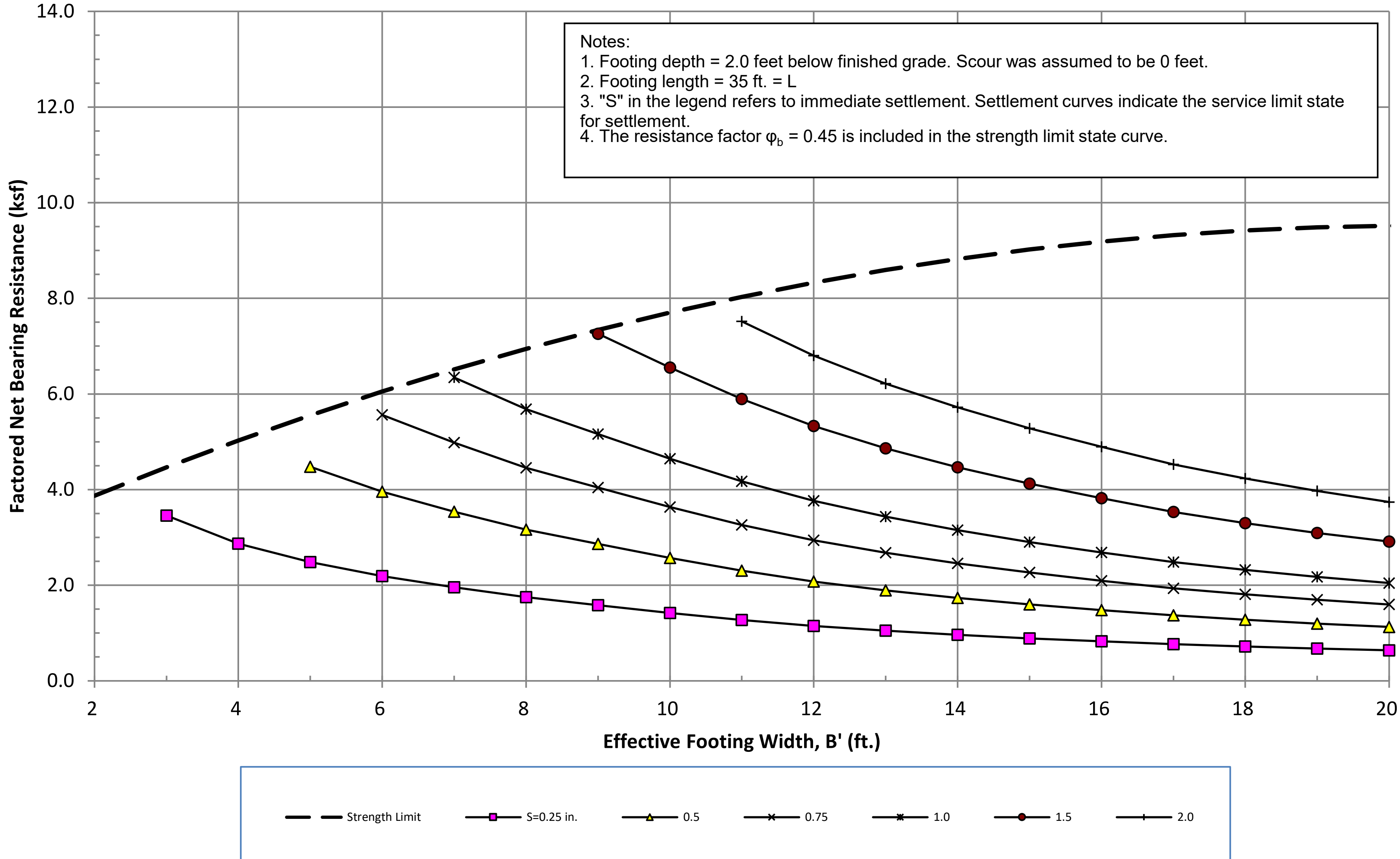


FIGURE 4

FACTORED BEARING RESISTANCE CHART
CANADA DEL ORO AND SUTHERLAND WASH CROSSING
CATALINA STATE PARK
TUCSON, ARIZONA

607775001 I 11/25

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 Standard Penetration Test 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 into the ground 12 to 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the log 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 methods.

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 the weight of a hammer or the Kelly bar of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. 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 log 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.

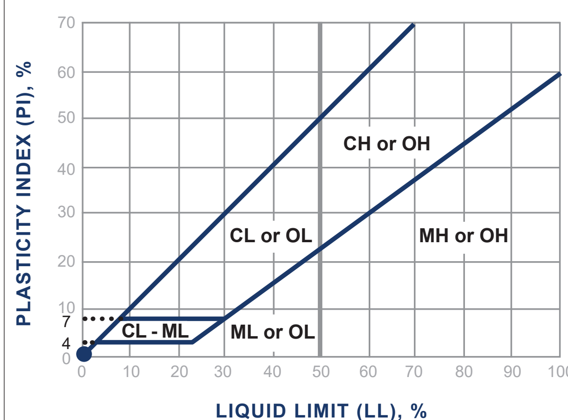
Soil Classification Chart Per ASTM D 2488

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 clay
		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

Consis-tency	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

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USCS METHOD OF SOIL CLASSIFICATION

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	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								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
20								The total depth line is a solid line that is drawn at the bottom of the boring.

DATE DRILLED 10/2/2025 BORING NO. B-1

GROUND ELEVATION 2,694' ± (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 ACM LOGGED BY ACM REVIEWED BY SDN

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SW-SM	ALLUVIUM: Brown, dry, medium dense, well-graded SAND with silt; few gravel.
			14					
			42	1.6	104.8			
5								
			24				SP-SC	Brown, dry, dense, poorly graded SAND with clay.
			56					
10							SW-SM	Brown, dry, medium dense, well-graded SAND with silt; few gravel.
			11					
15								
			36	3.3	110.6			Little gravel.
20								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 10/2/2025 BORING NO. B-1 GROUND ELEVATION 2,694' ± (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 ACM LOGGED BY ACM REVIEWED BY SDN **DESCRIPTION/INTERPRETATION**
20							SC	ALLUVIUM: (Continued) Brown, dry, medium dense, clayey SAND.
			17				SP-SC	Brown, dry, medium dense, poorly graded SAND with clay.
25								
			19					Weakly cemented.
30								Total Depth = 30 feet. Groundwater not encountered during drilling. Backfilled with soil on 10/2/25 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 10/2/2025	BORING NO. B-2	
GROUND ELEVATION 2,690' ± (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 ACM	LOGGED BY ACM	REVIEWED BY SDN

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SW-SM	ALLUVIUM: Light brown, dry, dense, well-graded SAND with silt; few gravel.
		44	2.2	101.1			
		13					Brown; medium dense.
5							
		26	0.9	97.7			Dry to moist.
		20				SP-SM	Brown, dry, medium dense, poorly graded SAND with silt.
10							
		27				SC	Brown, dry, medium dense, clayey SAND; little gravel.
15							
		17					
20							

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							SM	ALLUVIUM: (Continued) Brown, dry, medium dense, silty SAND; few gravel.
			19					Weakly cemented.
25								
			14				SP-SC	Brown, dry, medium dense, poorly graded SAND with clay.
30								Total Depth = 30 feet. Groundwater not encountered during drilling. Backfilled with soil on 10/2/25 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 10/2/2025 BORING NO. B-2

GROUND ELEVATION 2,690' ± (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 ACM LOGGED BY ACM REVIEWED BY SDN

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 D 2488. Soil classifications are indicated on the log of the exploratory borings in Appendix A.

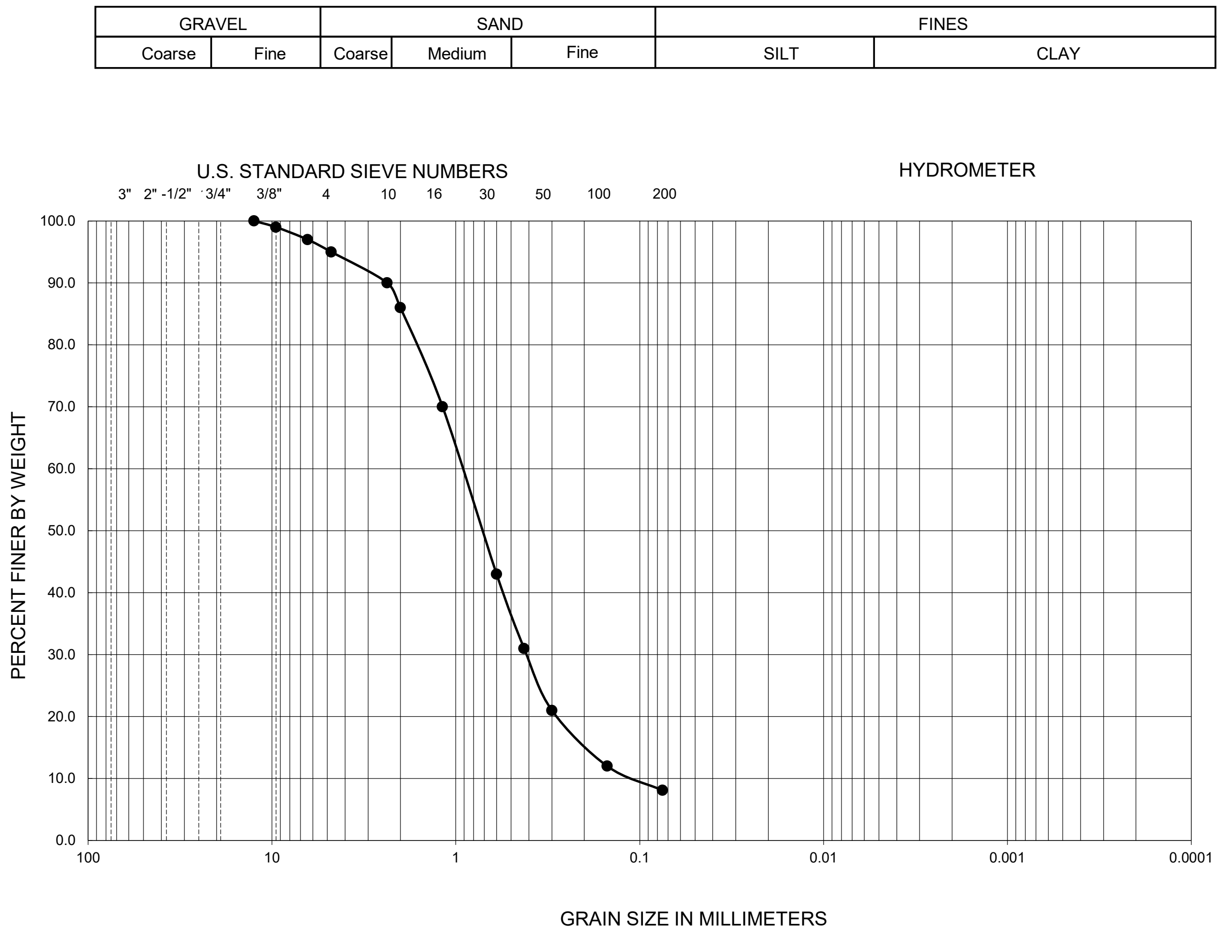
In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-1 through B-4. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

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 D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-5.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. 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 samples. The results of the tests are summarized on Figures B-6 through B-8.

Soil Corrosivity Tests
Soil pH and resistivity tests were performed on representative samples in general accordance with Arizona Test Method 236c. The soluble sulfate and chloride content of the samples were evaluated in general accordance with Arizona Test Method 733 and Arizona Test Method 736, respectively. The test results are presented on Figure B-9.



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	--	--	NP	0.104	0.417	0.92	8.8	1.8	8.1	SW-SM

NP - INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-1

GRADATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

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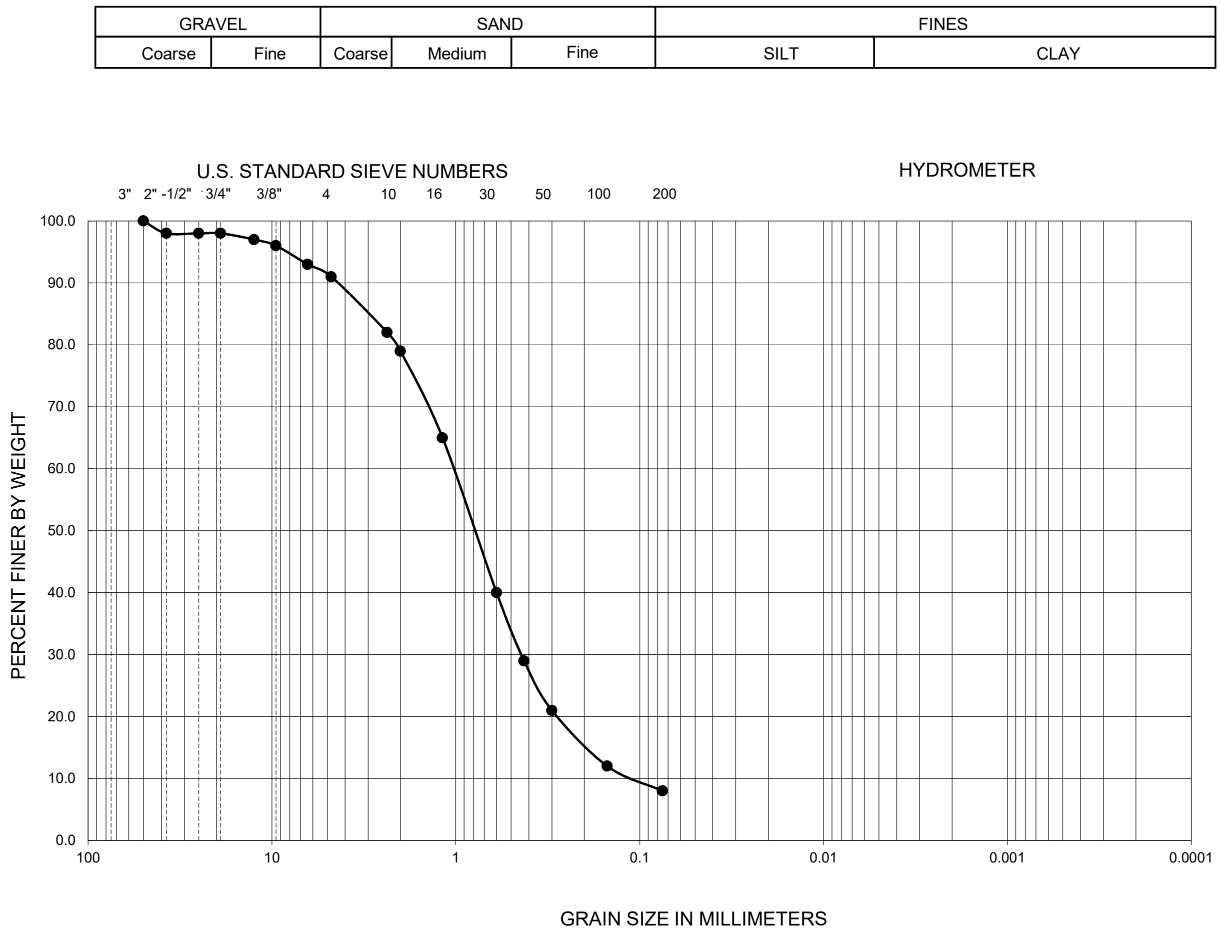

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	10.0-15.0	--	--	NP	0.105	0.444	1.04	9.9	1.8	8.0	SW-SM

NP - INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-2

GRADATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

TUCSON, ARIZONA

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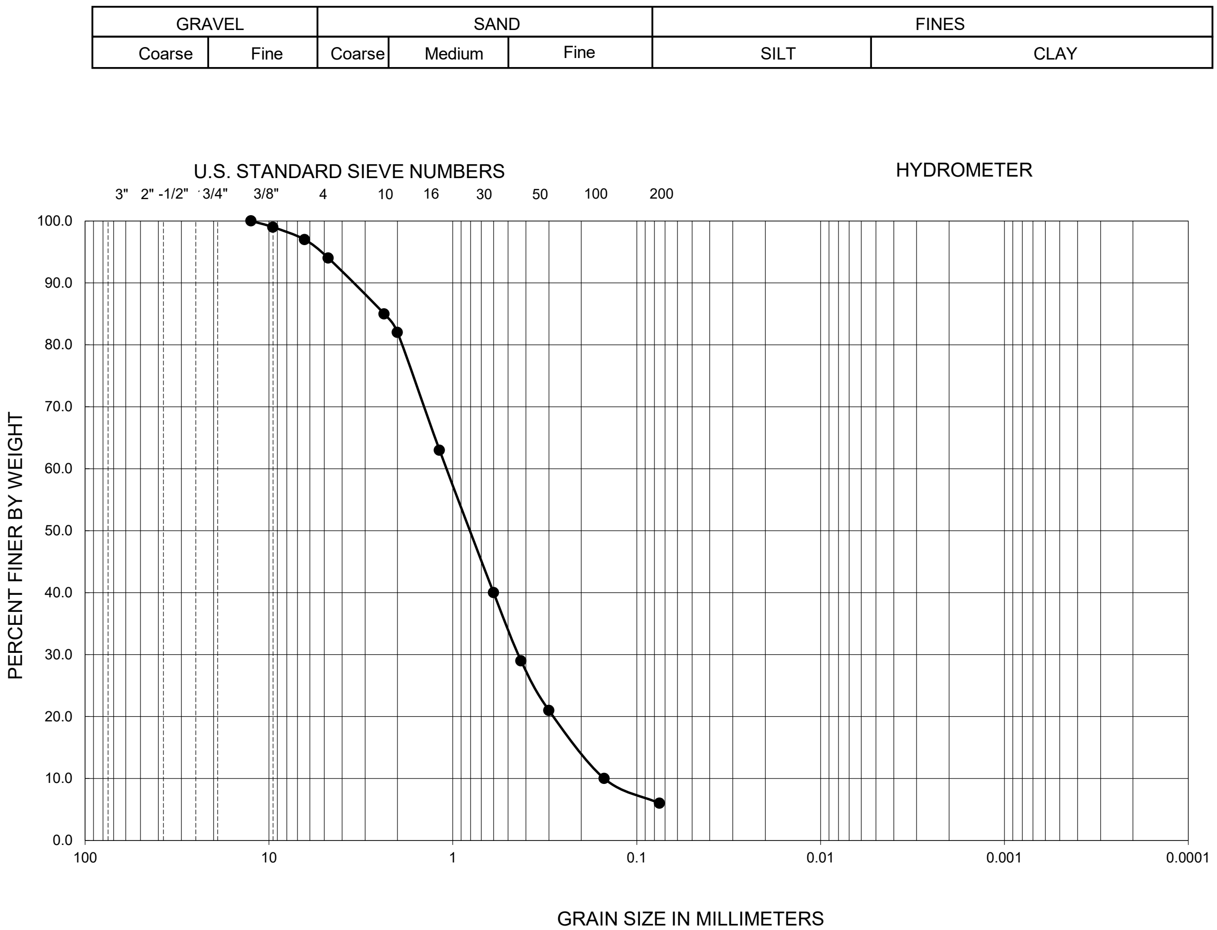

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	0.0-5.0	--	--	NP	0.150	0.444	1.09	7.3	1.2	6.0	SW-SM

NP - INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-3

GRADATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

TUCSON, ARIZONA

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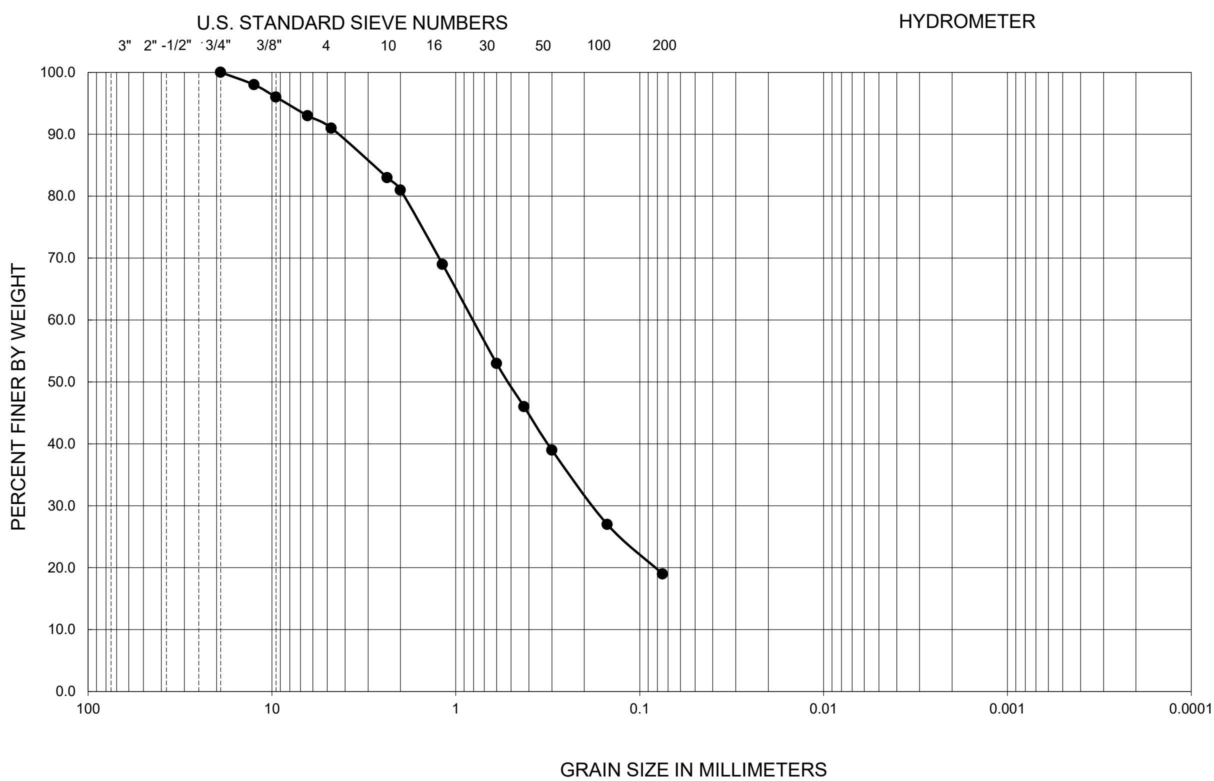
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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-2	25.0-30.0	--	--	NP	--	0.179	0.80	--	--	19.0	SM

NP - INDICATES NON-PLASTIC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-4

GRADATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

TUCSON, ARIZONA

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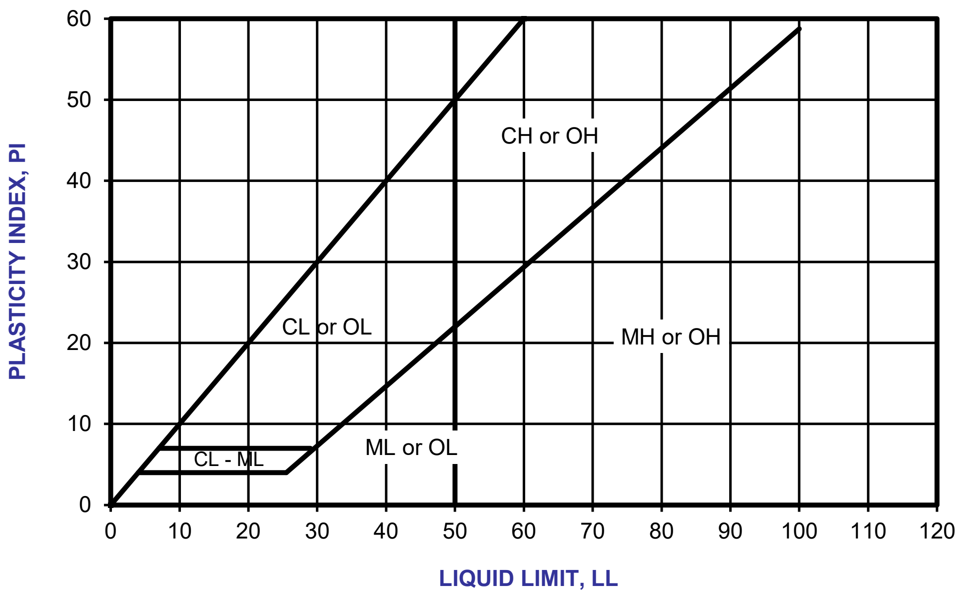
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	0.0-5.0	--	--	NP	ML	SW-SM
■	B-1	10.0-15.0	--	--	NP	ML	SW-SM
◆	B-2	0.0-5.0	--	--	NP	ML	SW-SM
○	B-2	25.0-30.0	--	--	NP	ML	SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-5

ATTERBERG LIMITS TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

TUCSON, ARIZONA

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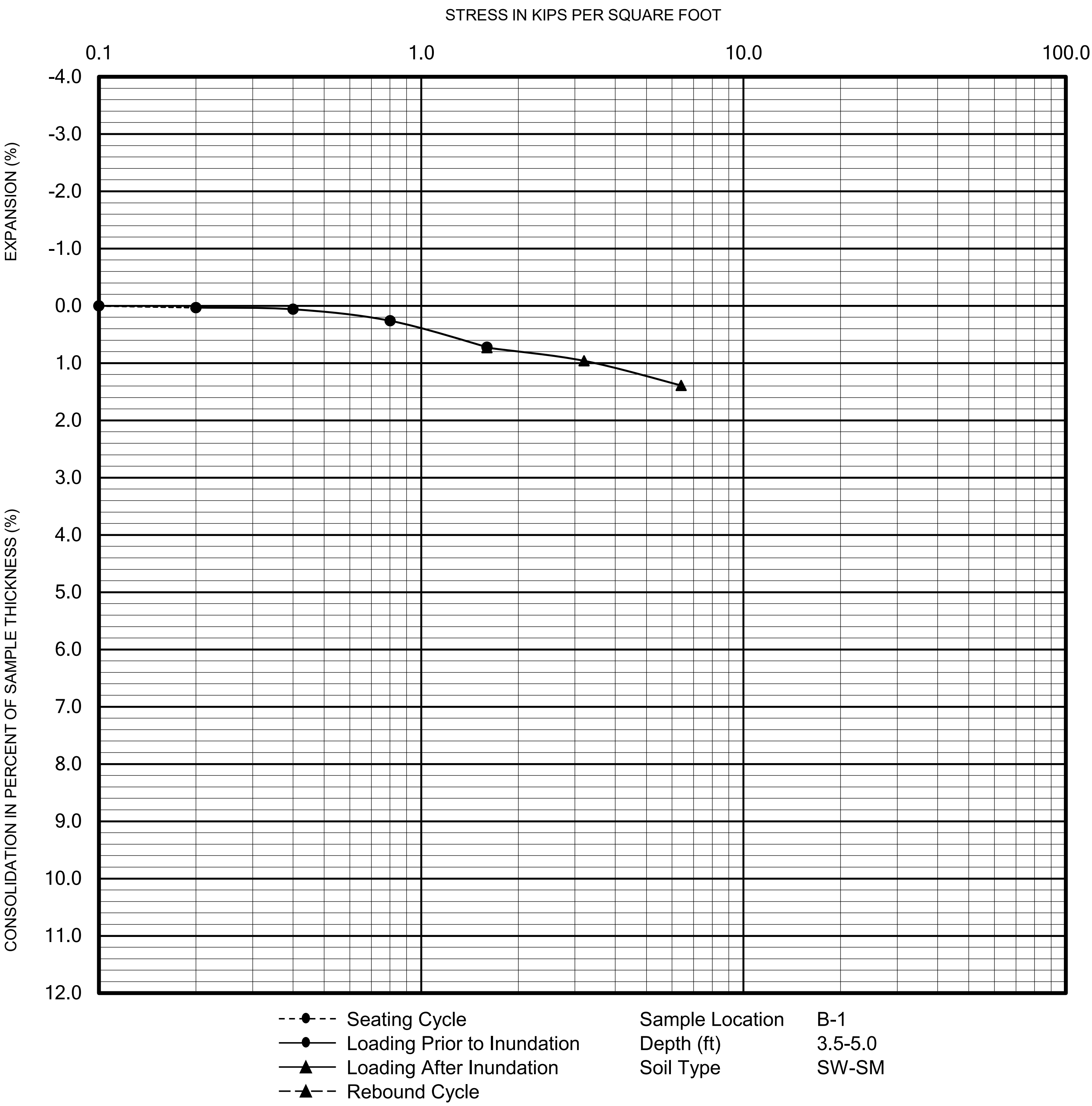

Sample Location	B-1
Depth (ft)	3.5-5.0
Soil Type	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-6

CONSOLIDATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

TUCSON, ARIZONA

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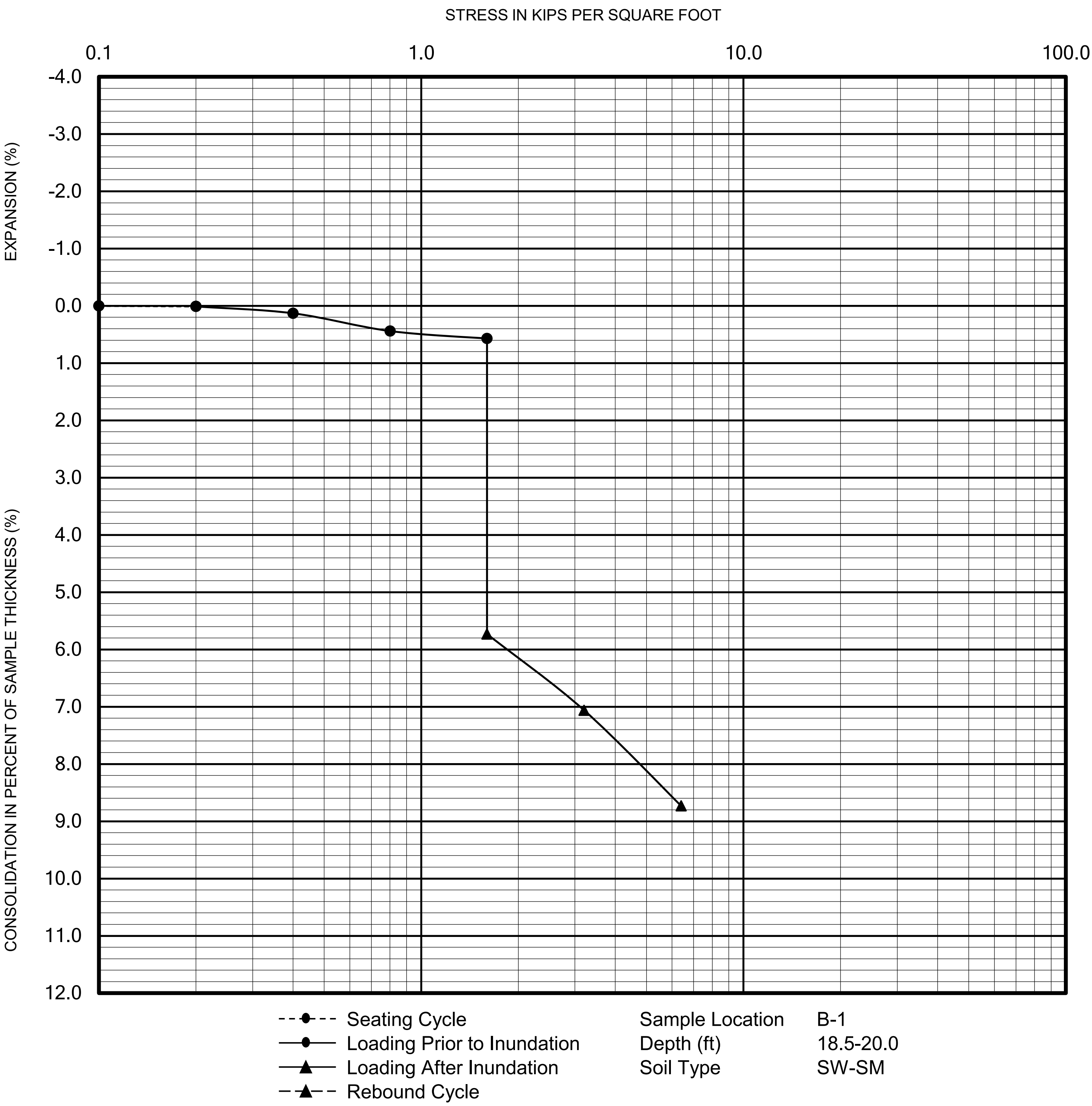

Sample Location	B-1
Depth (ft)	18.5-20.0
Soil Type	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-7

CONSOLIDATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

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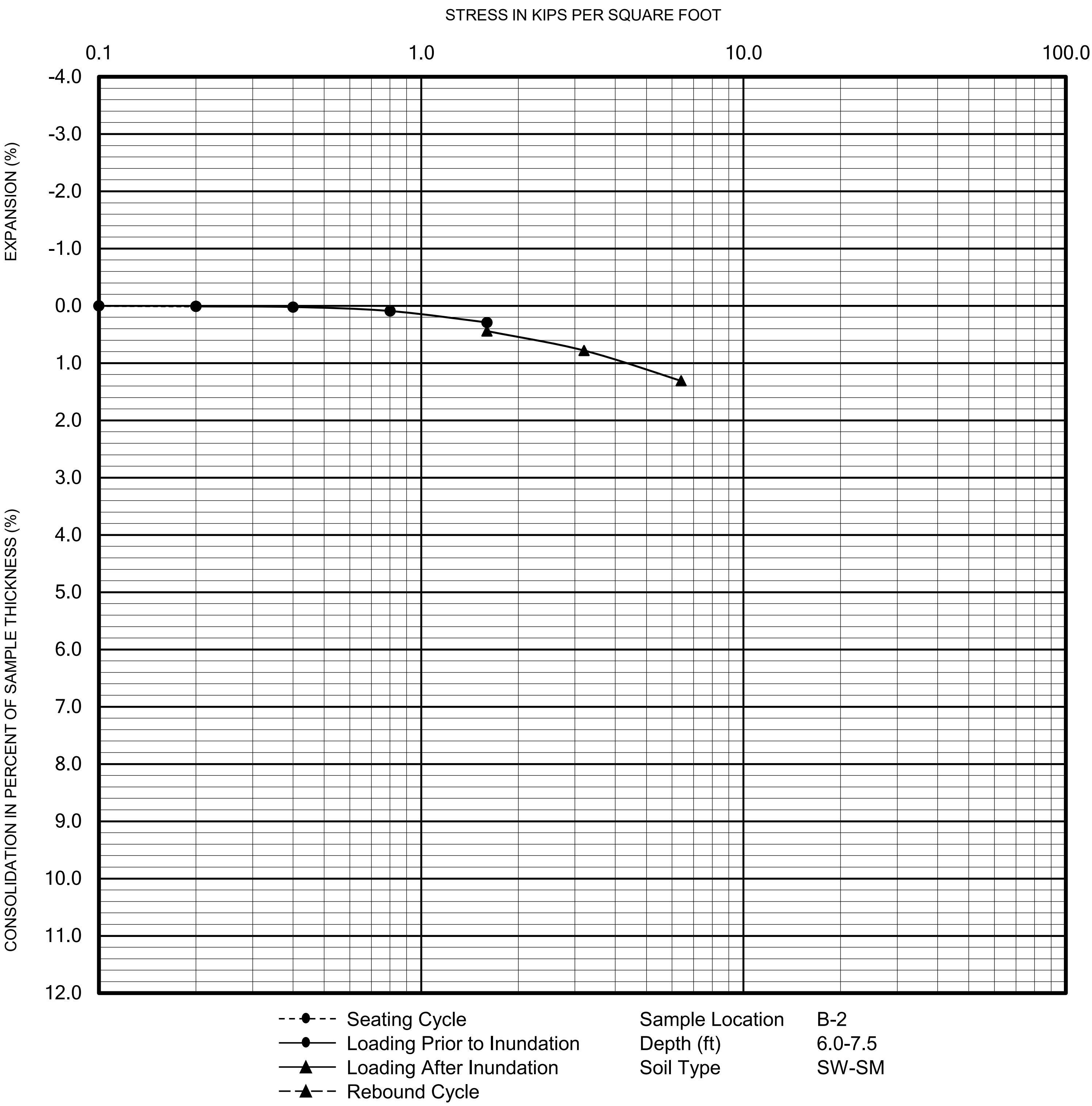

Sample Location	B-2
Depth (ft)	6.0-7.5
Soil Type	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-8

CONSOLIDATION TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

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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	10.0-15.0	7.5	17,420	ND	--	ND
B-2	0.0-5.0	7.3	13,400	ND	--	ND

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

ND = NOT DETECTED AT OR ABOVE THE PRACTICAL QUANTITATION LIMIT

FIGURE B-9

CORROSIVITY TEST RESULTS

CANADA DEL ORO AND SUTHERLAND WASH CROSSING

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