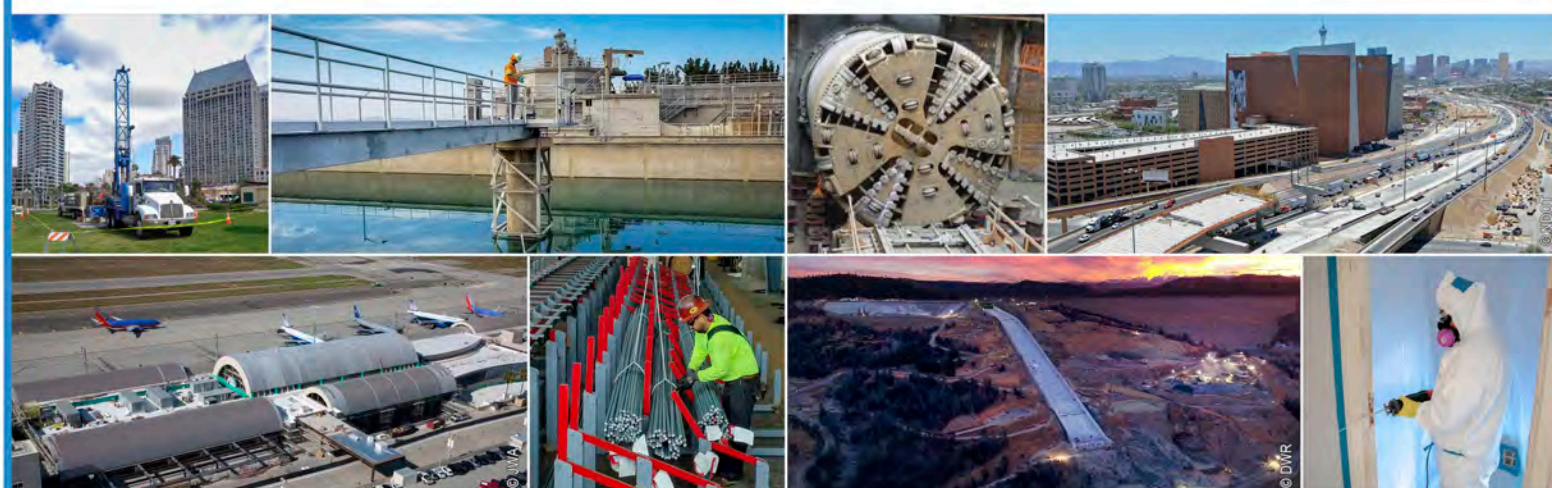
Geotechnical Evaluation
Site 6 Boat Ramp Improvements
591 Beachcomber Boulevard
Lake Havasu City, Arizona

Kimley-Horn

7740 North 16th Street, Suite 300 | Phoenix, Arizona 85020

August 4, 2023 | Project No. 60771101



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August 4, 2023
Project No. 607711001

Mr. Arnoldo Artiles, PE
Kimley-Horn
7740 North 16th Street, Suite 300
Phoenix, Arizona 85020

Subject: Geotechnical Evaluation
Site 6 Boat Ramp Improvements
591 Beachcomber Boulevard
Lake Havasu City, Arizona

Dear Mr. Artiles:

In accordance with our proposal dated April 14, 2023 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

Registered Professional Engineer (Civil)
CERTIFICATE NO.
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1. INTRODUCTION

In accordance with your request, Ninyo & Moore has conducted a geotechnical evaluation for the proposed Site 6 Boat Ramp Improvements project located in Lake Havasu City, Arizona. The purpose of our evaluation was to assess the subsurface conditions at the project site and provide geotechnical recommendations for design and construction. This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

Our scope of services for the project included the following:

- Reviewing available published and in-house geotechnical reports, topographic information, soil surveys, geologic literature, and aerial photographs of the project area.
- Conducting a site visit to conduct a geologic reconnaissance and selecting proposed boring locations.
- Contacting Arizona 811 and subcontracting a private utility locator to evaluate utility locations prior to drilling.
- Coring the existing concrete pavement using an electronic core machine in order to conduct our borings
- Drilling, logging, and sampling two exploratory soil boring to approximate depths of 5.5 and 7.5 feet below ground surface (bgs).
- Collecting soil samples in the borings at 2.5- to 5.0-foot intervals using ASTM International (ASTM) Methods D1586 (Standard Penetration Test) with split-barrel sampling of soils) and D3550 (ring-lined barrel sampling of soils) for laboratory testing and analysis.
- Performing laboratory testing to evaluate the on-site soil's in-situ moisture content and dry density, gradation, Atterberg limits, and chemical testing to evaluate corrosivity (including soil pH, resistivity, sulfate, and chloride).
- 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 site is located at 591 Beachcomber Boulevard in Lake Havasu City, Arizona (Figure 1). The site consists of an existing concrete boat ramp set between a pier and access road. West of pier is a large parking lot paved with asphaltic concrete (AC). Residential properties are located to the north and northeast of the ramp. The boat ramp slopes to the south, toward Lake Havasu.

According to the Lake Havasu City South, Arizona 7.5-Minute United States Geological Survey Topographic Quadrangle Map (2021), the site is at an average elevation of 459 feet relative to mean sea level (MSL).

4. PROJECT DESCRIPTION

The project includes the design and construction of a new ramp and associated retaining walls. The new ramp will have a shallower grade and will extend further into the lake. The new retaining walls will be located along the east side of the ramp and will extend into the lake. The walls will be supported on shallow spread footings. We also assume that little to no grading (+/- 2 feet), other than foundation excavations, will be need to establish the project design grades.

5. REVIEW OF AERIAL PHOTOGRAPHS

Aerial photographs of the project site dated between dated 1947 through 2020 were reviewed on a historical aerial photograph website (2023) for this project. In 1947, the area was generally native desert. Some small structures were located in close proximity to the future boat ramp site and may have been associated with the large airfield located immediately to the northeast. By 1969, the small structures had been removed and the boat ramp, pier and adjacent parking lot had been constructed. Residential development had also been completed to the northeast of the site and the airfield had been reconfigured. The site has remained generally the same since 2005 with an expansion of the parking lot completed sometime after 1994. The area to the northeast of the site has continued to be developed into residential homes since 2005 with portions of the old airfield being removed and cleared for future development. Since 2015, the surrounding area has remained relatively unchanged.

6. FIELD EXPLORATION AND LABORATORY TESTING

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

6.1. Pavement Core

Ninyo & Moore cored through the existing concrete pavement in prior to conducting our soil borings. The approximate core locations coincide with our boring locations is shown on Figure 2. The cores were advanced using an electronic core machine and allowed for the accurate measurement of the existing pavement thickness.

6.2. Soil Borings

Ninyo & Moore conducted a subsurface exploration of the site on July 10, 2023. The site exploration was needed to evaluate the subsurface conditions and to collect soil samples for laboratory testing. Our exploration consisted of drilling, logging, and sampling two exploratory soil borings denoted as B-1 and B-2. The approximate boring locations are shown on Figure 2. The borings were drilled using a CME-55 truck-mounted drill rig equipped with hollow-stem augers. Our borings were drilled to depths of 5.5 and 7.25 feet bgs. Logs of the borings are included in Appendix A. Both borings experienced auger refusal on cobbles and possibly boulders prior to achieving their target depths.

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.

6.3. Laboratory Testing

Selected samples were visually classified and tested in our laboratory to evaluate their engineering properties as a basis for providing geotechnical recommendations for design and construction considerations. Our geotechnical laboratory testing included in-situ moisture content and dry density, gradation, Atterberg limits, and chemical testing to evaluate corrosivity (including soil pH, resistivity, sulfate, and chloride). Laboratory test results for in-situ moisture content and dry density are shown on the boring logs, the results of the geotechnical laboratory tests are included in Appendix B.

7. GEOLOGY AND SUBSURFACE CONDITIONS

The following sections describe the site geology and subsurface conditions.

7.1. Geologic Setting

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 10 to 18 million years ago during the mid- to late-Tertiary. 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 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 surficial geology at the site consists of unconsolidated to strongly consolidated alluvial and eolian deposits. This unit includes: coarse, poorly sorted alluvial fan and terrace deposits on middle and upper piedmonts and along large drainages; sand, silt and clay on alluvial plains and playas; and wind-blown sand deposits. (Richard, SM, et al., 2000).

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 contain our field test results, as well as our interpretation of the conditions between actual samples retrieved. Therefore, the boring logs contain 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 is provided in Appendix A.

7.2.1. Concrete Pavement

Prior to drilling, the existing concrete pavement was cored. The concrete pavement thickness was measured to be between 7 and 7.25 inches. No underlying aggregate base (AB) material was observed below the concrete at our core locations.

7.2.2. Alluvium

Native alluvial soils were encountered below the concrete pavement in our borings. The alluvium generally consisted of medium dense to very dense coarse-grained soils above the groundwater level. The alluvium was generally characterized as silty sand with gravel (SM) and sand with silt and gravel (SP-SM). Several cobbles and coarse-grained gravels were encountered. Below the groundwater depth the sandy soils were loosely packed between the cobbles which did allow for samples to be collected. However, auger refusal was encounter in both boreholes on cobbles and possibly boulders at 5.5 and 7.25 feet bgs.

7.2.3. Groundwater

Groundwater was encountered in our borings at a depth of 4 feet in boring B-1 and 5.5 feet in boring B-2 Seasonal variations could cause fluctuations in the surrounding groundwater depths. Shallower perched water conditions may be encountered, especially after flood events.

8. GEOLOGIC HAZARDS

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

8.1. Land Subsidence and Earth Fissures

Based on our field reconnaissance and information accessed at Arizona Geologic Survey website (2023), the site is not located in area with documented earth fissures. The project site is not in an area with a measured land subsidence based on information accessed at the ADWR e-Library (2023).

8.2. 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, review of pertinent geologic data, and analysis of aerial photographs, Quaternary faults are not located on or adjacent to the property.

The closest know Quaternary faults to the site are the Needles-Graben Faults, located approximately 23 miles to the north of the site (Pearthree, 1998). Approximately 2 meters of displacement has occurred along this fault within middle to upper Pleistocene deposits

(<750,000 years), but the upper Pleistocene and Holocene deposits (<250,000 years) are not displaced.

9. CONCLUSIONS

Based on the results of our subsurface evaluation and our review of available geotechnical data, 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:

- The near surface site soils can generally be excavated or ripped using heavy-duty earthmoving or excavation equipment. However; very dense soils, varying amounts of gravel, cobbles, and possibly boulders were encounter in our borings and may be more difficult to excavate and/or slow the rate of excavation during construction.
- The proposed retaining walls can be founded on spread footings. Spread footings should be founded on a zone of adequately moisture-conditioned and compacted engineered fill.
- 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.
- Groundwater was observed in our boring and dewatering will be needed to construct a portion of the project. Dewatering operations may call for the construction of a cofferdam. Additional geotechnical information may be needed to design the cofferdam.
- No documented geologic hazards are present underlying or immediately adjacent to the site.
- Corrosivity test results indicate that on-site soils are corrosive to ferrous materials and the sulfate content of the soils presents 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 4), the observed subsurface conditions (Section 7), and our experience. Given the project location, recommendations and guidelines outlined by the Maricopa Association of Governments (MAG), and/or any Lake Havasu City (LHC) Engineering Specifications 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.

10.1.1. Pre-Construction Survey

Prior to construction activities, it may be desirable to recognize the condition of the existing utilities, underground structures, or other features that are near the planned construction and to survey or document (e.g., photographs, video, official documentation, etc.) their pre-construction condition. The findings of the survey could be used to document any damage that might result from this project.

10.1.2. Site Preparation

Vegetation, unsuitable materials, or debris from the clearing operation should be removed from the site and disposed of or placed in non-structural areas (e.g., landscaping). Obstructions that extend below finish grade, if present, should be removed and the resulting voids filled with moisture-conditioned and compacted engineered fill.

After rough grade has been achieved and prior to further earthwork, the exposed subgrade should be proof-rolled and visually observed for the presence of debris, organic matter and other unsuitable materials. If unsuitable soils are encountered at subgrade level during earthwork operations, these soils should be removed to their full depth, and be replaced with engineered fill.

The geotechnical consultant should carefully evaluate any areas of loose, soft, or wet soils prior to placement of fill or other construction. Drying or over-excavation of some materials may be appropriate.

10.1.3. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on our review of the results of the exploratory borings, site observations, and experience with similar materials. Excavation of the near surface materials can generally be accomplished with heavy-duty earthmoving equipment. However; very dense soils, varying amounts of gravel, cobbles, and possibly boulders were encounter in our borings and may be more difficult to excavate and/or slow the rate of excavation during construction.

10.1.4. Temporary Slopes

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. Based on the soil conditions at the site, we recommend that OSHA Soil “Type C” classification be used for excavations within the alluvium soils 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 3:1 (horizontal to vertical). New embankment fills should be benched into existing embankments, where appropriate. Benches should be level and wide enough to allow operation of and compaction by, construction equipment. 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. 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 will reduce precipitation impact and slow the rate of erosion. 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. Subgrade Preparation

We recommend that the new shallow foundations (with anticipated loading not in excess of 1,500 pounds psf) be supported on a zone of engineered material that extends 2 feet below the bottom of the foundations or to the surface of the underlying gravel, cobbles and possible boulders layer; whichever is shallower. 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 new flatwork, and asphalt/concrete 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 or 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 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.

10.1.7. Dewatering

It is anticipated that a portion of the site will call for dewatered so that subgrade preparation and foundation excavation can be performed. A dewatering subcontractor may be needed to conduct this task. The discharge of water from the dewatering zone or excavations to natural drainage channels or the lake, if needed, may entail securing a special permit.

In order to conduct the dewatering operations a cofferdam may be needed. This cofferdam should be designed by an engineer with experience in cofferdam design. In general, there are several types of cofferdams that may be utilized. Rock or earthen fill, precast concrete blocks or water filled inflatable barriers could be utilized as cofferdams for this project. The size, geometry, and construction sequencing should drive this decision. Given the conditions observed in our borings, a sheet pile or driven pile type cofferdam may not be feasible without additional geotechnical exploration of the planned dewatering zone.

The exposed surface of the dewatered zone should be stable for the purpose of the planned construction. If the areas or associated excavations are open during a heavy rain event, the bottom may become saturated and unstable. The dewatered area should be inspected by the geotechnical consultant for the presence of soft, loose, or wet soils prior the placement construction materials.

10.1.8. Engineered Fill

On-site and imported soils that exhibit low plasticity indices and very low expansive potential are suitable for use as engineered fill. Low plasticity indices, as evaluated by ASTM D4318, are defined as a PI of 15 or less for this project. Low expansive potential soils are defined as having a swell potential of less than 1.5 percent when evaluated in accordance with ASTM D4546 with a load of 144 psf.

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 3 inches in dimension within the upper 5 feet of finished grade. Engineered fill should contain less than 20 percent fines (material passing the No. 200 sieve). 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). 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.

10.1.9. Re-use of On-Site Soils

The Atterberg limits tests performed on soil samples obtained from the borings resulted in PI values of NP (0). Based on our test results, many of the on-site soils are considered suitable for re-use as engineered fill, but may need to be screened to remove larger particles. However, the effects of corrosion will need to be considered in design and larger particles will need to be to be screened. Additional field sampling and laboratory testing should be conducted by the contractor either prior to or during construction to better screen for any unsuitable materials.

10.1.10. Fill Placement and Compaction

Engineered fill, as described in this report, and AB should be moisture-conditioned within the moisture range shown below in Table 1 and mechanically compacted to the percentage shown. Engineered fill and AB 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 prior to construction of the foundation systems. 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 1 – Compaction Recommendations

Engineered Fill Description	Minimum Compaction per ASTM D698	Moisture Content
Native surfaces prior to fill placement	95 percent	±2 percent of optimum
Pavement subgrade and beneath flatwork	95 percent	±2 percent of optimum
AB course	100 percent	±3 percent of optimum
Below footings	95 percent	±2 percent of optimum

An earthwork (shrinkage) factor of 10 to 20 percent is estimated for the alluvium soils. 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.

10.1.11. 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.

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 2 presents the seismic design parameters for the site in accordance with the American Society of Civil Engineers (ASCE 7-22) guidelines and adjusted maximum considered earthquake spectral response acceleration parameters evaluated using the ASCE Hazard Tool (web-based):

Table 2 – Seismic Design Criteria (ASCE 7-22)

Seismic Design Factors	Value
Site Soil Class	D
Seismic Design Category	D
Site Coefficient, F_a	1.6
Site Coefficient, F_v	2.325
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.25g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.12g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	0.44g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.34g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.30g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.23g
Site Modified Peak Ground Acceleration, PGA_M	0.15g

10.3. Foundations

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

Spread or continuous footings, if utilized, should bear at a depth of 24 inches or more below the exiting grade or adjacent finished grade whichever is deeper. 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 supported on 24 inches of engineered fill may be designed using a net allowable bearing capacity of 1,500 pounds 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. As described in this report, fill used to raise grade will settle a portion of its height due to its own weight prior to construction of the foundation systems. The magnitude of this settlement will depend on the type of fill used. In general, the fill recommended in this report is expected to settle about 1 percent of its height.

Foundations bearing on moisture-conditioned, compacted engineered fill that are subject to lateral loadings may be designed using an ultimate coefficient of friction of 0.40 (total frictional resistance equivalent to the coefficient of friction multiplied by the dead load). A passive resistance value of 340 psf per foot of depth can be used for soils above the high-water level. Soils that may experience saturation due to the increase in the water elevation should use a passive resistance value of 150 psf per foot of depth. 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. Retaining Walls

Retaining walls should be supported on shallow spread footings as discussed in Section 10.3. Wall backfill 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. Retaining walls should be designed using the applicable lateral earth pressures. The earth pressures provided assumes level backfill at the top of the wall. It is anticipated that some retaining walls may be with a zone of fluctuating water levels and should consider the impacts of hydrostatic pressure.

10.4.1. 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 drained, level backfill, an equivalent fluid active earth pressure of 40 psf per foot (psf/ft) of wall height should be used for design of cantilevered, yielding walls. 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.

10.4.2. 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 60 psf/ft for drained level backfill should be used.

Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.50q. The value for "q" represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

10.4.3. 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 340 psf/ft can be used for drained conditions. For undrained conditions, an ultimate equivalent fluid passive earth pressure of 150 psf/ft can be used. 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.

10.5. Pavements

For the new paved areas both AC and (Portland cement concrete) PCC sections may be considered. The design parameters for these pavement sections include a 20-year design life and a traffic load of 100,000 or less Equivalent Single-Axle Loads. The pavement sections are assumed to bear on imported or on-site soils with an average soil correlated R-value of 35 or more.

An AC pavement section consisting of 3 inches or more of plant-mix asphalt (per LHC Section 02630) over 6 inches or more of graded AB (per LHC Section 02610) can be considered in the standard duty parking areas. For heavier traveled areas or main driveways, an asphalt pavement section consisting of 4 inches or more of plant-mix asphalt over 8 inches or more of graded AB (per LHC Section 02610) can be utilized.

PCC pavements are recommended for areas that will experience regular truck traffic, main ingress and egress areas, and in areas where vehicles will be turning or loading (e.g., the ramp extension into the lake). PCC in heavy traffic areas should have a thickness of 8 inches or more, with edges thickened to 10 inches. In parking areas not subject to truck traffic, the concrete pavement thickness can be reduced to 6 inches, with edges thickened to 8 inches.

Concrete pavements should have longitudinal and transverse joints that meet the applicable requirements of the MAG Uniform Standard Specification and/or any Lake Havasu City requirements. The need for reinforcement within the concrete pavement should be evaluated by the structural engineer based on the anticipated traffic loading. Concrete pavements should be underlain by 4 inches or more of AB that meets LHC Section 02610.

For both the PCC and AC pavements given above, we recommend that the underlying subgrade soils be moisture-conditioned and compacted as described in this report.

Table 3 – Pavement Structural Sections

Pavement Description[1,]	PCC Thickness (Inches)	AC Thickness (inches)	AB Thickness (inches)	Total Pavements Section Thickness (inches)
Standard Duty AC (Parking Areas)	---	3.0	6.0	9.0
Heavy Duty AC (Main Driveways)	---	4.0	8.0	12.0
Standard Duty PCC (Parking Areas)	6.0	---	4.0	10.0

Table 3 – Pavement Structural Sections				
Pavement Description[1]	PCC Thickness (Inches)	AC Thickness (inches)	AB Thickness (inches)	Total Pavements Section Thickness (inches)
Heavy Duty PCC (Ingress/Egress)	8.0	---	4.0	12.0

Note:
[1] These pavements should be supported on improved subgrade soils per this report.

10.6. Concrete Flatwork

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

10.7. 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 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 value of the selected sample tested from our borings was 9.4, which is considered to be alkaline. The minimum electrical resistivity of the tested sample was 3,283 ohm-cm, which is considered to have non-corrosive potential with respect to ferrous materials. The chloride content of the sample tested was 48 ppm, which indicates a corrosive environment for ferrous materials. The soluble sulfate content of the soil sample tested was 0.0003 percent by weight of soil, which is considered to represent negligible sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials are considered corrosive to ferrous materials. As such, 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.

10.8. Concrete

Laboratory chemical tests performed on an on-site soil sample indicated a sulfates content of 0.0003 percent by dry weight of soil. Based on American Concrete Institute (ACI) criteria (Table 4), the on-site soils should be considered to present a negligible sulfate exposure to concrete.

Due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and regional practice, we recommend that “Type II” cement be used for the 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. Calcium chloride should not be used as an admixture in concrete.

Table 4 – 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.

10.9. 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.

10.10. 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 over-excavation, 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, 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

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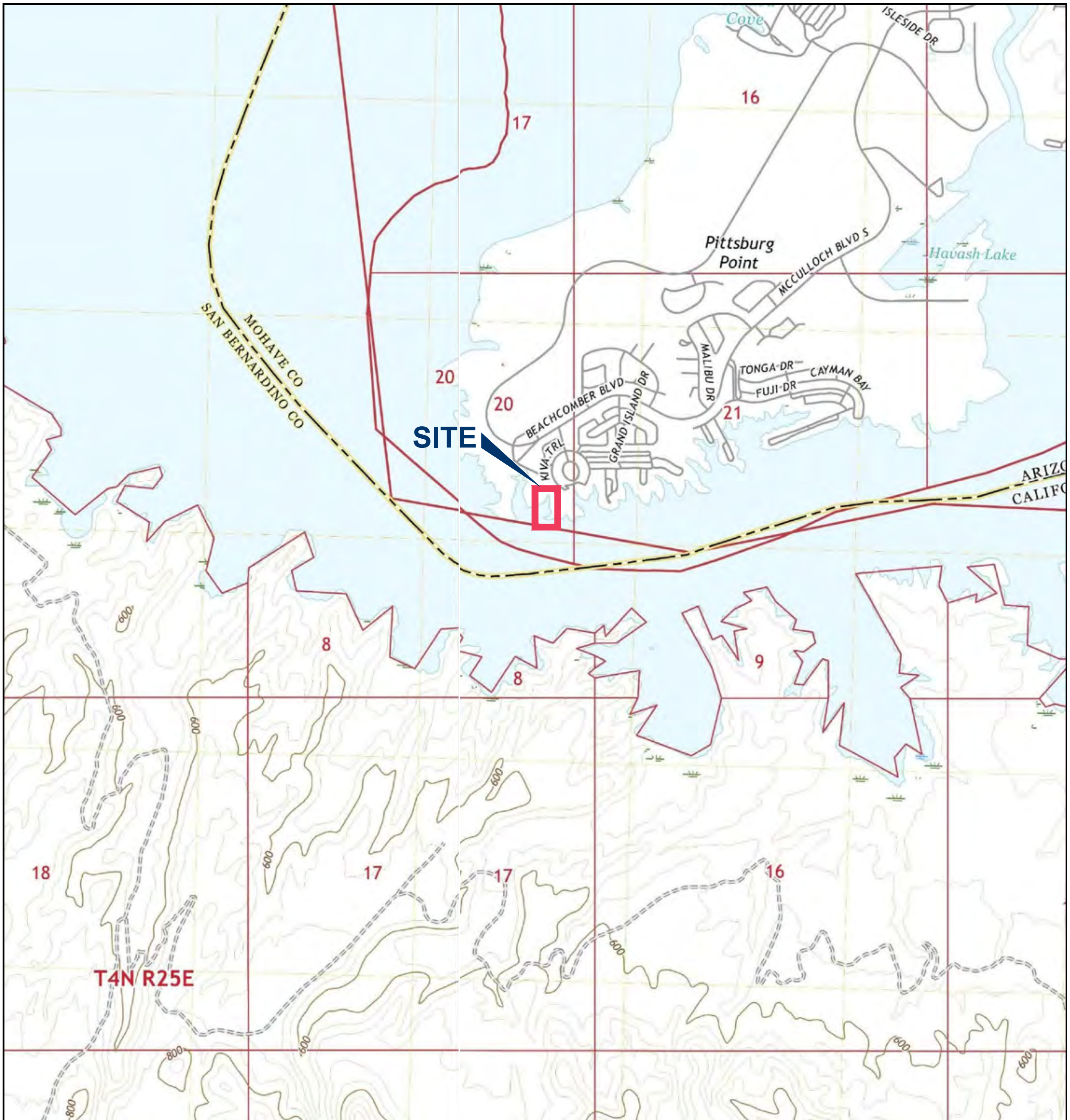
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2021.

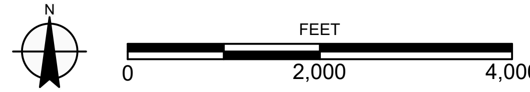


FIGURE 1

SITE LOCATION

SITE 6 BOAT RAMP IMPROVEMENTS
LAKE HAVASU CITY, ARIZONA

Ninyo & Moore
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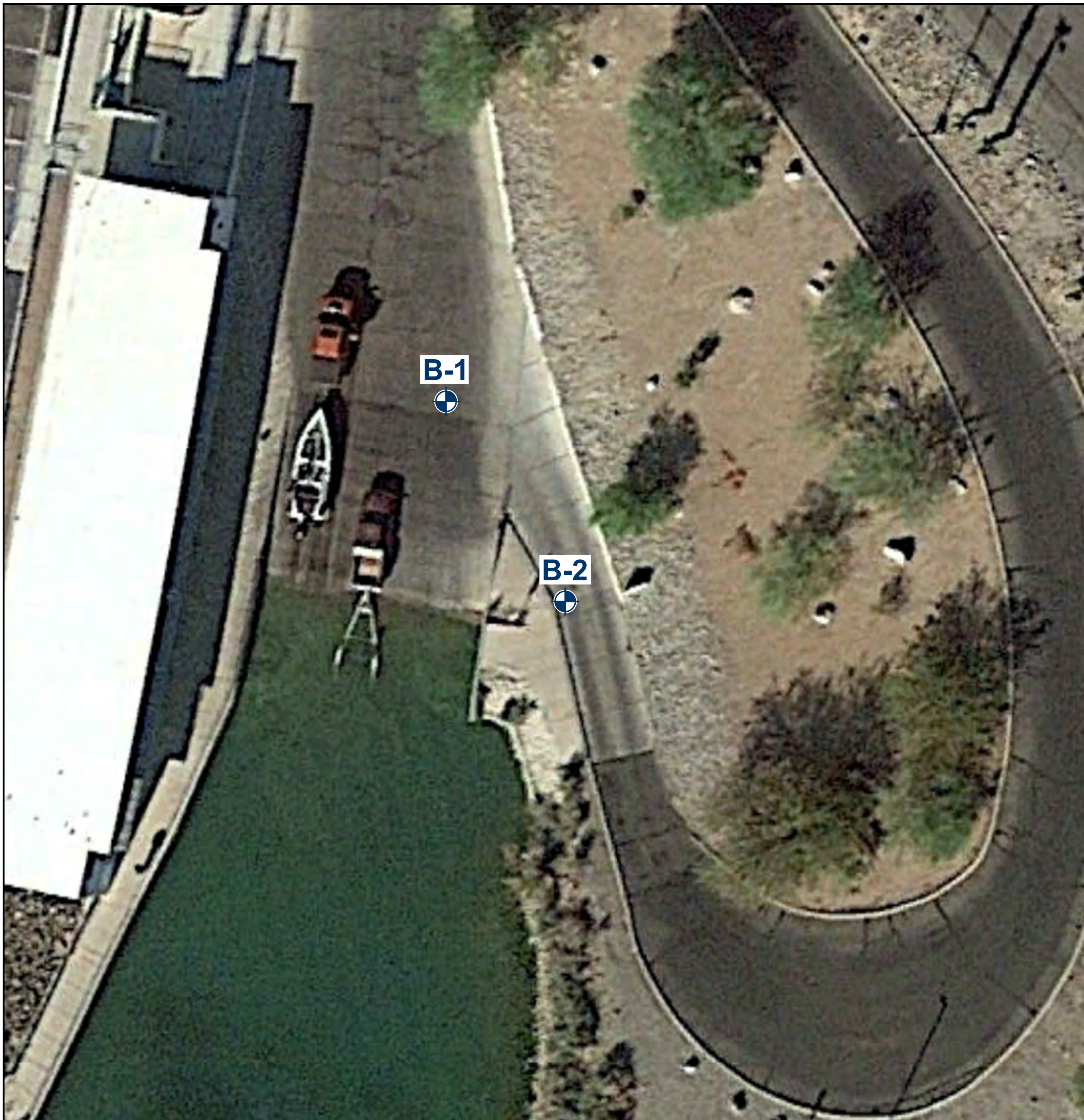
607711001 I 8/23



LEGEND

B-2 BORING

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH 2022.

N

FEET
0 30 60

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FIGURE 2

EXPLORATION LOCATIONS

SITE 6 BOAT RAMP IMPROVEMENTS
LAKE HAVASU CITY, ARIZONA

607711001 I 8/23

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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 consisting of auger cuttings of representative earth materials were obtained from selected 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 (California) 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 free-falling from a height of 30 inches in general accordance with ASTM D1586. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

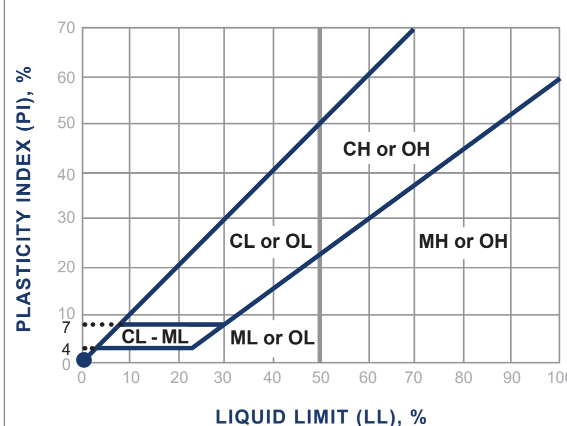
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	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		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	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		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	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. 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 7/10/23 BORING NO. B-1

GROUND ELEVATION 457' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)

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								CONCRETE: Approximately 7 inches thick.
			50/5"				SP-SM	ALLUVIUM: Brown, moist, very dense, poorly graded SAND with silt and gravel.
								@ 3 feet: Coarse gravel; cobbles.
								@ 4 feet: Encountered groundwater.
5			6				SM	Brown, wet to saturated, loose, silty SAND with gravel.
								@ 5.5 feet: Auger refusal; cobbles and possible boulders.
								Total Depth = 5.5 feet. (Refusal) Groundwater was measured at a depth of approximately 4 feet in borehole during drilling and at approximately 3 feet after completion of drilling on 7/10/23. Backfilled and concrete patched on 7/10/23 shortly after completion of drilling.
								Notes: Groundwater may rise to a level higher than that measured in borehole 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								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								CONCRETE: Approximately 7 1/4 inches thick.
							SP-SM	ALLUVIUM: Brown, moist, very dense, poorly graded SAND with silt and gravel.
			51					
								@ 3 feet: Coarse gravel; cobbles.
			50/5"	5.0	125.8			
5								@ 5.5 feet: Encountered groundwater.
							SM	Brown, moist to wet, medium dense, silty SAND; few coarse gravel; trace clay.
			9					@ 7.5 feet: Auger refusal; gravel; cobbles and possible boulders.
								Total Depth = 7.5 feet. (Refusal) Groundwater was measured at a depth of approximately 5.5 feet in borehole during drilling and at approximately 3 feet after completion of drilling on 7/10/23. Backfilled and concrete patched on 7/10/23 shortly after completion of drilling. Notes: Groundwater may rise to a level higher than that measured in borehole 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 7/10/23 BORING NO. B-2

GROUND ELEVATION 458' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-55, 8" Diameter Hollow-Stem Auger (Wildcat)

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

SAMPLED BY DM LOGGED BY DM 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 D2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

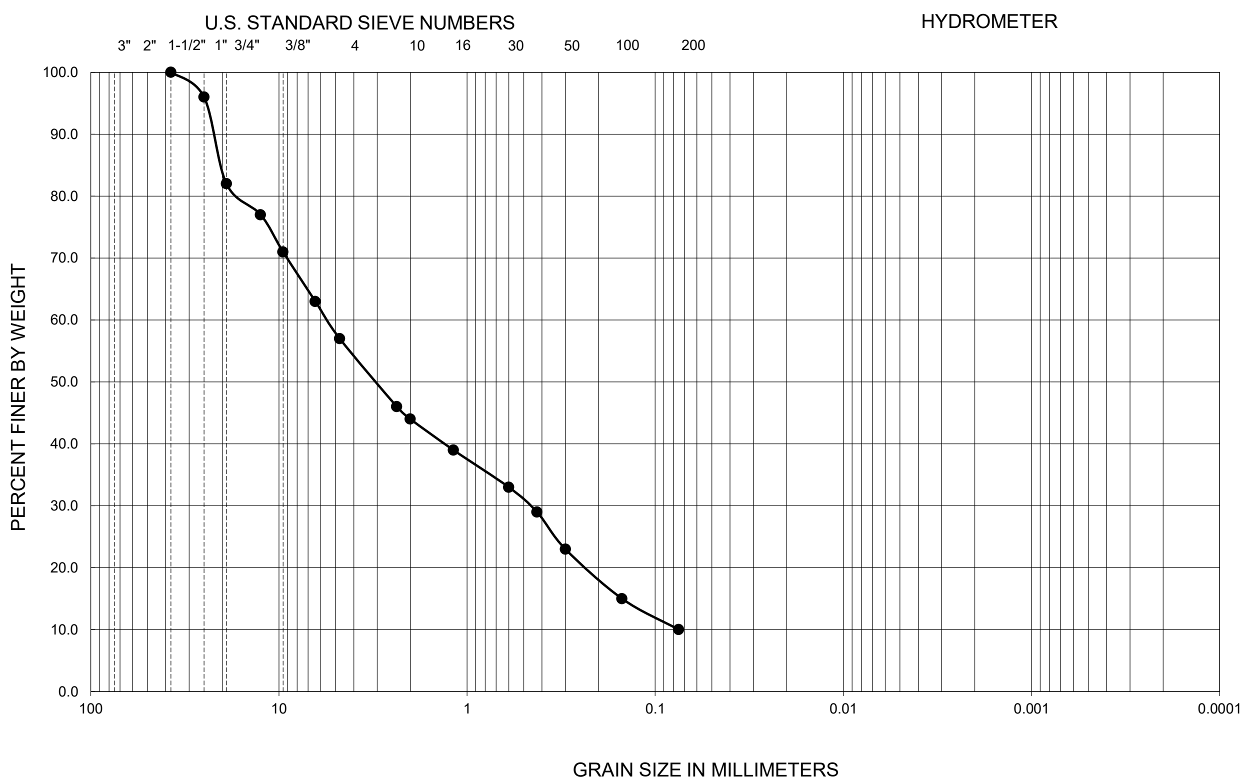
In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM C136 and D422. The grain-size distribution curves are shown on Figures B-1 and B-2. 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 D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results and classifications are shown on Figure B-3.

Soil Corrosivity Tests
Soil pH, and resistivity tests were performed on representative samples in general accordance with Arizona Test Method 236b. The soluble sulfate and chloride content of selected samples were evaluated in general accordance with Arizona Test Method 733 Arizona Test Method 736, respectively. The test results are presented on Figure B-4.

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	0.7-1.5	--	--	NP	0.074	0.466	5.54	74.8	0.5	10.0	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-1

GRADATION TEST RESULTS

SITE 6 BOAT RAMP IMPROVEMENTS

LAKE HAVASU CITY, ARIZONA

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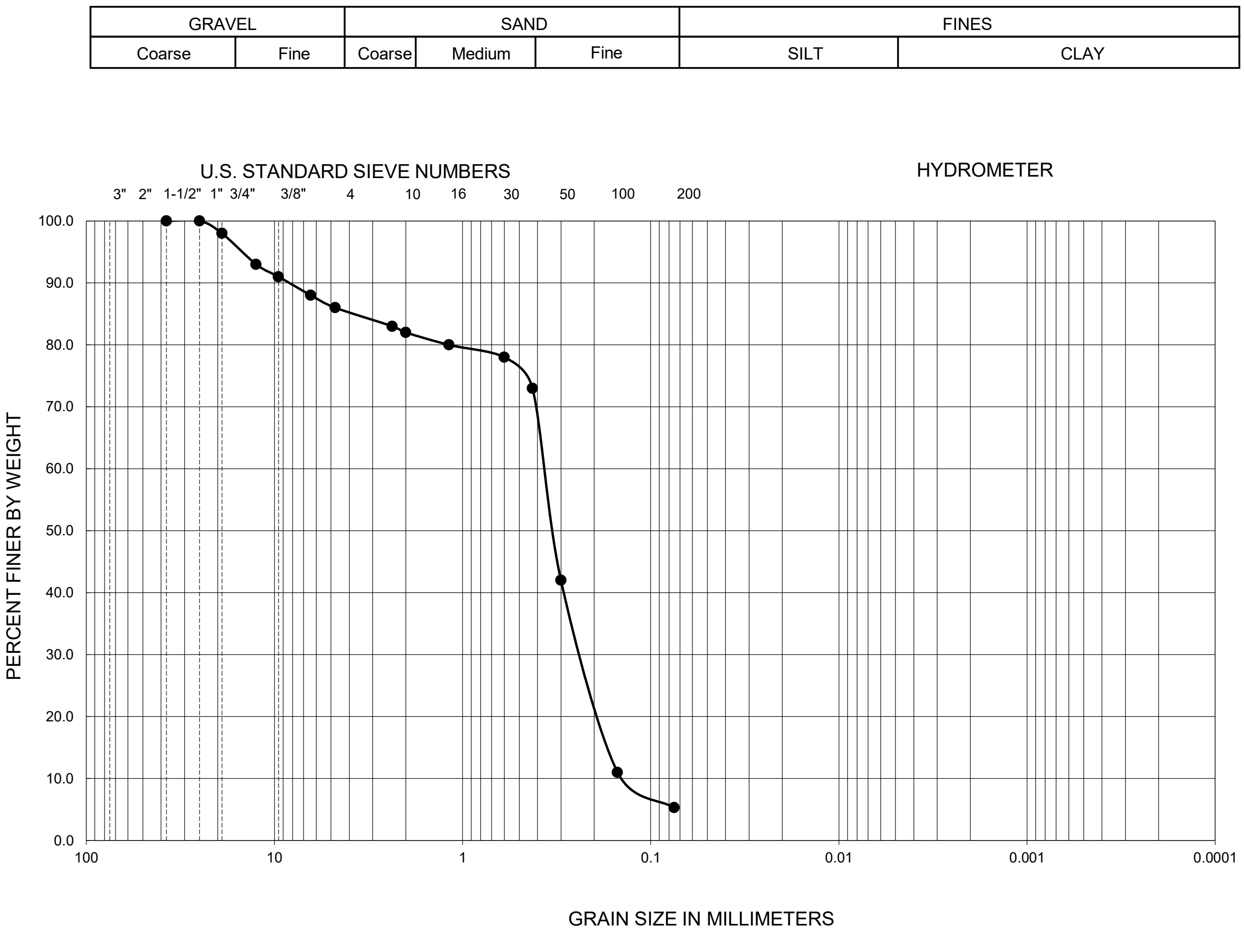

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-2	1.0-5.0	--	--	NP	0.132	0.232	0.37	2.8	1.1	5.3	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM C136 / D422

FIGURE B-2

GRADATION TEST RESULTS

SITE 6 BOAT RAMP IMPROVEMENTS

LAKE HAVASU CITY, ARIZONA

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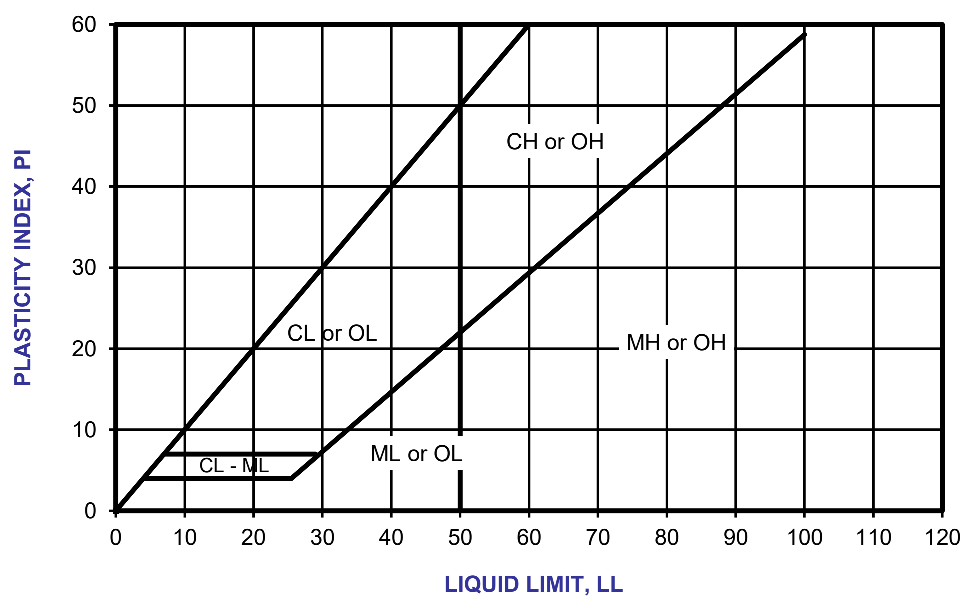
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	0.7-1.5	--	--	NP	ML	SP-SM
■	B-2	1.0-5.0	--	--	NP	ML	SP-SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-3

ATTERBERG TEST RESULTS

SITE 6 BOAT RAMP IMPROVEMENTS

LAKE HAVASU CITY, 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	1.0-4.0	9.4	3,283	3	0.0003	48

[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

CORROSIVITY TEST RESULTS

SITE 6 BOAT RAMP IMPROVEMENTS
LAKE HAVASU CITY, ARIZONA

607711001 | 8/23

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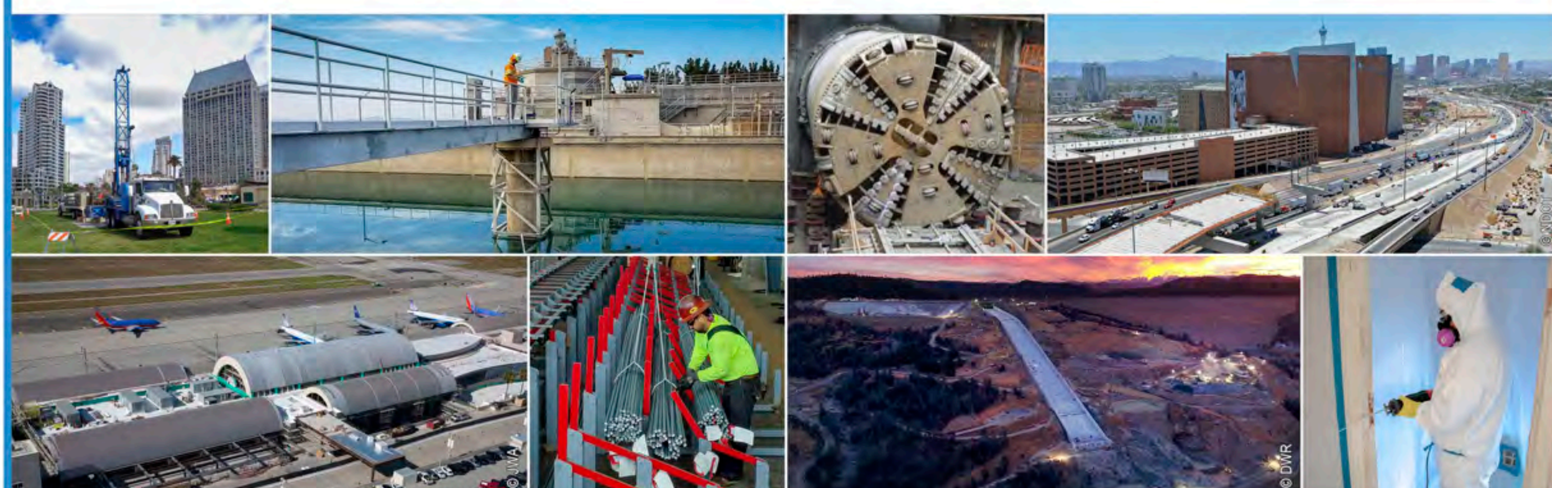

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