

**GEOTECHNICAL INVESTIGATION  
ARROWHEAD VILLAS MUTUAL SERVICE COMPANY  
PROPOSED WATER TANKS SITE  
SYCAMORE DRIVE, LAKE ARROWHEAD  
SAN BERNARDINO COUNTY, CALIFORNIA**

V5  
V2  
V4  
V3  
V1  
ENVIRONMENTAL  
GEOTECHNICAL / CONSTRUCTION MATERIALS TESTING  
INFRASTRUCTURE ENGINEERING  
ASSET MANAGEMENT / CERTIFICATION  
MUNICIPAL SERVICES



August 17, 2015



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NV5 Infrastructure  
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August 17, 2015  
Contract No.: 149745

Attention: Mr. James Owens

Subject: Geotechnical Investigation

Project: Arrowhead Villas Mutual Service Company  
Proposed Water Tanks Site, Sycamore Drive  
Lake Arrowhead, San Bernardino County, California  
NV5 Infrastructure Job No.: SBD098901

Dear Mr. Owens:

As requested, NV5 West, Inc. (NV5) is pleased to submit the results of the geotechnical investigation for the subject project. The purpose of this investigation was to evaluate the subsurface conditions for Arrowhead Villas Mutual Service Company’s proposed water tanks site. We understand that the proposed construction includes two potable water tanks, associated piping and a small pump station. The results of the geotechnical field explorations, laboratory tests, and geotechnical engineering recommendations and conclusions are presented herewith.

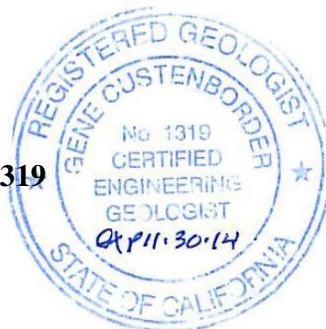
Based on the subsurface exploration, subsequent testing of the subsurface soils, and engineering analyses it was concluded that the construction of the proposed project is geotechnically feasible provided the recommendations contained herein are appropriately incorporated into the design and implemented during construction.

It is recommended that the forthcoming project specifications, in particular the earthwork/compaction sections, be reviewed by NV5 for consistency with our report prior to the bid process in order to avoid possible conflicts, misinterpretations, and inadvertent omissions, etc. It should also be noted that the applicability and final evaluation of recommendations presented herein are contingent upon construction phase field monitoring by NV5 in light of the widely acknowledged importance of geotechnical consultant continuity through the various design, planning and construction stages of a project.

NV5 appreciates the opportunity to provide this geotechnical engineering service for this project and looks forward to continuing our role as your geotechnical engineering consultant.

Respectfully submitted,  
NV5 West, Inc.

**Gene Custenborder, CEG 1319**  
Senior Project Geologist



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## 1.0 INTRODUCTION

This report presents the results of NV5's geotechnical investigation for Arrowhead Villas Mutual Service Company's (AVMSC) proposed water tanks site in Lake Arrowhead, San Bernardino County, California. The approximate location of the project area is shown on Figure 1, Site Location Map. The purpose of this study was to evaluate the subsurface conditions and to provide geotechnical recommendations for the design and construction of the proposed water tanks and associated improvements. This report summarizes the data collected and presents our findings, conclusions, and recommendations.

This report has been prepared for the exclusive use of the client and their consultants in the design of the proposed project. In particular, it should be noted that this report has not been prepared from the perspective of a construction bid preparation instrument and should be considered by prospective construction bidders only as a source of general information subject to interpretation and refinement by their own expertise and experience, particularly with regard to construction feasibility. Contract requirements as set forth by the project plans and specifications will supersede any general observations and specific recommendations presented in this report.

## 2.0 SCOPE OF SERVICES

The scope of services for this project consisted of the following tasks:

- Review of readily available background data, including in-house geotechnical data, geotechnical literature, geologic maps, topographic maps, seismic hazard maps, and literature relevant to the subject site.
- A site reconnaissance to observe the general surficial site conditions and to select boring locations.
- A subsurface investigation, including the excavating, logging, and sampling of two exploratory borings located within the project area to depths up to approximately 50.3 feet below the existing ground surface. Soil samples obtained from the borings were transported to NV5's in-house laboratory for observation and testing.
- Laboratory testing of selected soil samples to evaluate their pertinent geotechnical engineering properties.
- An assessment of faulting, seismicity, slope stability and other geologic hazards affecting the area and possible impacts on the subject project.
- Engineering evaluation of the geotechnical data collected to develop geotechnical recommendations for the design and construction of the proposed project.
- Preparation of this report, including reference maps and graphics, summarizing the data collected and presenting our findings, conclusions, and geotechnical recommendations for the design and construction of the proposed project.



### **3.0 SITE AND PROJECT DESCRIPTION**

The tanks site pad is a relatively level graded pad at an elevation of approximately 5840 feet above mean sea level located adjacent to the north side of Sycamore Drive in the Skyforest Area of Lake Arrowhead (refer to Figure 1, Site Location Map). The pad was apparently constructed by cut-fill grading techniques on a moderately steep north facing slope. The slope gradient ranges from approximately 1 to 1 (horizontal to vertical) to 2 to 1. The site is bounded by Sycamore Drive on the south, single-family residential structures on the east and west and a moderate to steep north facing slope along the north.

Based on preliminary information it is understood that the proposed project includes construction of two potable water tanks, associated underground piping and a small pump station. The capacity of the tanks will be approximately 150,000 gallons each.

### **4.0 FIELD EXPLORATION**

Before starting the field exploration program, a field reconnaissance was conducted to observe site conditions and check locations for the planned subsurface explorations. As required by law, Underground Service Alert was notified of the locations of the exploratory borings prior to drilling.

The subsurface conditions were explored by drilling, logging, and sampling two exploratory borings located within the project area to a maximum depth of approximately 50 feet below the existing ground surface. The borings were drilled using a track-mounted hollow-stem auger drill rig. The drilling subcontractor services were contracted directly by AVMSC, and were provided by 2R Drilling, Inc. of Chino, California. The approximate locations of the exploratory borings are presented on Figure 2, Geotechnical Map. Details of the subsurface exploration and logs of the exploratory borings are presented in Appendix A. Subsequent to logging and sampling, the borings were backfilled.

### **5.0 LABORATORY TESTING**

Laboratory testing was performed on selected representative bulk and relatively undisturbed soil samples obtained from the exploratory borings to aid in the soil classification and to evaluate the engineering properties of the soil materials encountered. The following tests were performed:

- In-situ moisture content (ASTM D2216)
- Sieve analyses (ASTM D422)
- Direct shear (ASTM D3080)
- Maximum density and optimum moisture content (ASTM D1557)
- Corrosivity series including sulfate content, chloride content, pH-value, and resistivity (California Test Methods 417, 422, and 532/643)

Testing was performed in general accordance with applicable ASTM standards or California Test Methods. The laboratory test results and details of the laboratory-testing program are presented in Appendix B.



## 6.0 GEOLOGY

**Geologic Setting** - The site is located in San Bernardino County within the Transverse Ranges geomorphic province. This province is characterized by an east-west trending series of steep mountain ranges and valleys. The east-west structure of the Transverse Ranges is oblique to the normal northwest trend of coastal California, hence the name "Transverse." The province extends offshore to include San Miguel, Santa Rosa, and Santa Cruz islands. Its eastern extension, the San Bernardino Mountains, has been displaced to the south along the San Andreas Fault. Intense north-south compression is squeezing the Transverse Ranges. As a result, this is one of the most rapidly rising regions on earth. Great thicknesses of Cenozoic petroleum-rich sedimentary rocks have been folded and faulted, making this one of the important oil producing areas in the United States. The project site is located on and adjacent to a moderately steep sloping hilly and mountainous terrain. The mountains are underlain by Cretaceous granitic rocks.

**Geologic Materials** - Geologic materials encountered during the subsurface explorations include crystalline granitic basement rocks of the southern California Batholith. Surficial deposits of fill are also present locally. As encountered, the fill was generally less than 8 feet in thickness. However, deeper accumulations may be present. Figure 2, Geotechnical Map presents the general distribution of geologic units at the site and nearby vicinity. Figure 3, Cross Section A-A' depicts the interpreted subsurface conditions. Figure 4, Regional Geologic Map presents the general distribution of geologic units on a regional scale. Detailed descriptions of the earth materials encountered are presented on the exploratory pit logs in Appendix A. Descriptions of the various geologic units are provided below:

- Fill (mapped as Af) - Fill soils were encountered in both of the exploratory borings drilled on the relatively level pad area where the water tanks are proposed. Fill was encountered to a depth of approximately 6 feet and 8 feet in Borings B-1 and B-2, respectively. Fill appears to be derived locally from excavations of the granitic rocks. As encountered these materials generally consisted of light brown, moist, loose to medium dense fine to coarse sand. The fill soils are not considered capable of reliably supporting construction of the proposed water tanks and ancillary structures in their present condition. Recommendations for treatment of the existing fill soils are provided in the Design Recommendations section of this report.
- Granitic Rocks (mapped as Kgr) - The entire project site at depth and the adjacent hillside areas are underlain by Cretaceous-aged "granitic" rocks of the southern California batholith. The granitic rock is generally deeply weathered resulting in a decomposed granitic soil ("DG"). Localized areas of hard granitic rock are exposed in the slopes in the general site area. As encountered in the exploratory borings at the proposed water tank site, the decomposed residual granitic soil is relatively thick and generally comprised of light brown, damp to moist, dense to very dense silty fine to coarse sand. The dense decomposed granitic soil and rock typically exhibit favorable bearing characteristics for proposed fill and/or structural loads.

**Groundwater** - Indications of static, near-surface groundwater table were not observed or encountered during the subsurface exploration to the total depth explored (maximum of approximately 50 feet). It is anticipated that groundwater will not be a constraint during construction. However, experience indicates that near-surface groundwater conditions or localized seepage zones can develop in areas where no such groundwater conditions previously existed, especially in areas where a substantial increase in surface water infiltration results from landscape irrigation, agricultural activity, storage facility leaks or unusually heavy precipitation. Seasonal variations in the groundwater levels should be anticipated.

## 7.0 FAULTING, SEISMICITY AND OTHER GEOLOGIC HAZARDS

The principal seismic considerations for most facilities in Southern California are surface rupturing of fault traces, damage caused by ground shaking or seismically-induced ground settlement or liquefaction. Potential impacts to the project due to faulting, seismicity and other geologic hazards are discussed in the following sections.

**Faulting** - The numerous faults in southern California include active, potentially active, and inactive faults. As used in this report, the definitions of fault terms are based on those developed for the Alquist-Priolo Special Studies Zones Act (AP) of 1972 and published by the California Division of Mines and Geology (Hart and Bryant, 1997). Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within any of the state-designated Earthquake Fault Zones (previously known as Alquist-Priolo Special Studies Zones). Faults are considered potentially active if they exhibit evidence of surface displacement since the beginning of Quaternary time (approximately two million years ago) but not since the beginning of Holocene time. Inactive faults are those that have not had surface movement since the beginning of Quaternary time.

Review of geologic maps and literature pertaining to the general site area indicates that the site is not located within a state-designated Earthquake Fault Zone. In addition, there are no known major or active faults mapped on the project site. Evidence for active faulting at the site was not observed during the subsurface investigation. The relative location of the site to known active faults in the region is depicted on Figure 5, Regional Fault Map. The distance from the site to the projection of traces of surface rupture along major active earthquake fault zones, that could affect the site are listed in the following Table 2.

**Table 2**  
**Distance From the site to Major Active Faults**

Fault	Distance From Site
Waterman fault	1.3 miles
Tunnel Ridge fault	3.3 miles
Cleghorn fault (southern section)	3.9 miles
San Andreas fault (San Bernardino Mountains section) (Mill Creek fault)	6.0 miles
North Frontal Thrust System	13.8 miles
San Jacinto fault (San Bernardo section) (Rialto-Colton fault)	14.3 miles
Sierra Madre fault	15.5 miles
Elsinore fault (Glen Ivey section) (Chino fault)	35 miles

**Seismic Shaking** - The project site is located in southern California which is considered a seismically active area, and as such, the seismic hazard most likely to impact the site is ground shaking resulting from an earthquake along one of the known active faults in the region. The seismic design of the project may be performed using seismic design recommendations in accordance with the 2013 California Building Code (CBC). Recommended seismic design parameters are presented in Section 9.10 of this report.

**Fault Rupture** - The project site is not located within an Earthquake Fault Zone delineated by the State of California for the hazard of fault surface rupture. The surface traces of known active or potentially active faults are not known to pass directly through, or to project toward the site. Therefore, the potential for damage due to surface rupture of faults at the project site is considered low.

**Liquefaction and Seismically-Induced Settlement** - Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking. Liquefaction is generally known to occur in saturated cohesionless soils at depths shallower than approximately 50 feet. Pipes constructed in soils that become liquefied may become buoyant.

The dense to very dense decomposed granitic rock underlying the proposed water tanks are typically not prone to liquefaction. Considering that the underlying materials are not susceptible to liquefaction and the lack of a near-surface groundwater table, it is our opinion that the potential damage to the proposed water tanks due to liquefaction is considered low.

Dynamic settlement due to earthquake shaking can occur in both dry and saturated loose to medium dense sandy soils. These sand particles can become more densely packed and settle when subject to seismic shaking. The dense to very dense decomposed granitic rock underlying the proposed water tanks are typically not prone to dynamic settlement. It is NV5's opinion that the potential for damage to the proposed water tanks due to seismically-induced settlement at the sites is low.

**Landslides and Slope Instability** - The water tank site is located on a relatively flat graded pad constructed on a moderately steep mountain side. However, based on the investigation, there are no known landslides on the project site, and the site is not located in the path of any known landslides. The geologic materials underlying the proposed tank site are not known to be prone to landslides or slope instability in properly engineered slopes.

Slope stability analyses including static and pseudo-static conditions were performed to evaluate the global stability for the tank pad and adjacent slope. Three cases were analyzed. Case 1 evaluated the stability without the proposed tanks. Case 2 was modeled to include loads from the water tanks, and Case 3 included an additional seismic loading. For purposes of modeling the tank loading, a load value of 2,000 psf across the relatively level portion of the tank pad was used in the analysis. The cross sections were analyzed utilizing the computer software program Slide, version 6.0, created by Rocscience®. The Bishop Simplified method of analysis was used to locate the critical slip surface for static and pseudo-static conditions. The subsurface conditions were modeled based upon data from the subsurface exploration program and the earth materials were assigned the strength characteristics based on their shear strength as tested in the laboratory. Based on our analysis the existing slope has a calculated minimum specified safety factor in excess of the currently accepted standard of 1.5 and 1.1 for static and seismic, respectively. A seismic factor (k) equal to a third of the horizontal peak bedrock acceleration (i.e., 0.267g) was added as a horizontal force, while the effects of vertical acceleration were omitted. The results of the stability analyses are presented in Appendix C.

It is our opinion that the potential damage to the proposed water tank project due to landsliding or slope instability is considered low.

**Subsidence** - The site is not located in an area of known ground subsidence due to the withdrawal of subsurface fluids. Accordingly, the potential for subsidence occurring at the site due to the withdrawal of oil, gas, or water is considered low.

**Tsunamis, Inundation Seiches, and Flooding** - The site is located approximately 60 miles inland from the coast at an elevation in excess of 5800 feet above mean sea level. Therefore, tsunamis (seismic sea waves) are not considered a hazard at the site.

The site is not located downslope of any large body of water that could affect the site in the event of an earthquake-induced failure or seiche (oscillation in a body of water due to earthquake shaking). Therefore, earthquake-induced seiches are not considered a hazard at the site.

## 8.0 CONCLUSIONS

Based on the data obtained from the subsurface exploration, the associated laboratory test results, engineering analyses, and experience with similar site conditions, it is NV5's opinion that construction of the proposed water tanks and associated improvements is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and implemented during construction. Geotechnically-related recommendations for the design and construction of the proposed water tanks and associated improvements are presented in the following sections.

## 9.0 DESIGN RECOMMENDATIONS

### 9.1 General

Locally-derived sandy fill soils were encountered at the proposed project site. These materials are considered compressible and not capable of reliably supporting the proposed water tanks and associated improvements in their present condition. Overexcavation and recompaction of these materials are recommended for the proposed structure and fill loads. These materials, when properly moisture-conditioned, are considered suitable for reuse as compacted fill.

### 9.2 Earthwork For Grading of Tank Pad

Site grading should be performed in accordance with the following recommendations and the Typical Earthwork Guidelines provided in Appendix D. In the event of conflict, the recommendations presented herein supersede those of Appendix D.

- Clearing and Grubbing - Prior to grading, the project area should be cleared of all significant surface vegetation, demolition rubble, trash, debris, etc. Any buried organic debris or other unsuitable contaminated material encountered during subsequent excavation and grading work should also be removed. Removed material and debris should be properly disposed of offsite. Holes resulting from removal of buried obstruction which extend below finished site grades should be filled with properly compacted soils. Any utilities within tank footprints should be appropriately abandoned.
- Site Grading –The water tanks should be founded entirely in compacted fill. A cut-fill transition condition should not be allowed underlying the tanks. In order to create a uniform bearing condition for the proposed water tanks, including any adjacent perimeter hardscape features (i.e., walls, walkways, etc.), all areas to receive surface improvements or fill soils should be treated as follows:
  - Tank Pad and Adjacent Slope: Existing fill soils underlying the proposed water tanks and comprising the existing slope bordering the tank pad should be completely excavated, moisture conditioned and uniformly recompacted to at least 95 percent of the soils maximum density (based on ASTM D1557). Excavation should extend laterally a

distance of at least 5 feet outside the tank perimeter. The slope should be keyed and benched in accordance with the detail and recommendations in Appendix D.

- Paved Areas, Flatwork: Excavate to a depth of at least 1.0 feet below the proposed or existing subgrade elevation, whichever is greater and replace with non-expansive compacted fill (Expansion Index not exceeding 20). These excavations should extend a horizontal distance of at least 2.0 feet beyond the outside perimeter.
- Excavatability – Based on our subsurface exploration, it is anticipated that the on-site soils can be excavated by modern conventional heavy-duty excavating equipment in good operating conditions.
- Structural Fill Placement - Areas to receive fill and/or surface improvements should be scarified to a minimum depth of 6 inches, brought to near-optimum moisture conditions, and compacted to at least 95 percent relative compaction, based on laboratory standard ASTM D1557. Fill soils should be brought to near-optimum moisture conditions and compacted in uniform lifts to at least 95 percent relative compaction (ASTM D1557). Rocks with a maximum dimension greater than 4 inches should not be placed in the upper 3 feet of pad grade. The optimum lift thickness to produce a uniformly compacted fill will depend on the size and type of construction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in loose thickness. Placement and compaction of fill should be observed and tested by the geotechnical consultant.
- Graded Slopes – Graded slopes should be constructed at a gradient of 2 to 1 (horizontal to vertical) or flatter. To reduce the potential for surface runoff over slope faces, cut slopes should be provided with brow ditches and berms should be constructed at the top of fill slopes.
- Import Soils - If import soils are needed, proposed import should be sampled and tested for suitability by NV5 prior to delivery to the site. Imported fill materials should consist of clean granular soils free from vegetation, debris, or rocks larger than 3 inches maximum dimension. The Expansion Index value should not exceed a maximum of 20 (i.e., essentially non-expansive).

### 9.3 Utility Trenching and Temporary Excavations

Excavation of the on-site soils may be achieved with conventional heavy-duty grading equipment. Temporary, unshored, excavation walls may be sloped back at an inclination of 1:1(H:V) within fill and natural materials. Utility trench excavations should be shored in accordance with guidelines and regulations set forth by CalOSHA. For planning purposes, the alluvial soils may be considered a Type C soil, as defined by the current CalOSHA soil classification. Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1:1(H:V), but no closer than 4 feet. All trench excavations should be made in accordance with CalOSHA requirements.

Temporary, shallow excavations with vertical side slopes less than 4 feet high will generally be stable, although due to the low density of the alluvium, there is a potential for localized sloughing. Vertical excavations greater than 4 feet high should not be attempted without proper shoring to prevent local instabilities. For vertical excavations less than about 15 feet in height, cantilevered shoring may be used. Cantilevered shoring may also be used for deeper excavations; however, the

total deflection at the top of the wall should not exceed one inch. Therefore, shoring of excavations deeper than about 15 feet may need to be accomplished with the aid of tied back earth anchors.

The actual shoring design should be provided by a registered civil engineer in the State of California experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by NV5 for conformance with the design intent and geotechnical recommendations. The shoring system should further satisfy requirements of CalOSHA. Shoring may be accomplished with hydraulic shores and trench plates, and/or trench boxes, soldier piles and lagging. The actual method of a shoring system should be provided and designed by a contractor experienced in installing temporary shoring under similar soil conditions. If soldier piles and lagging are to be used, we should be contacted for additional recommendations.

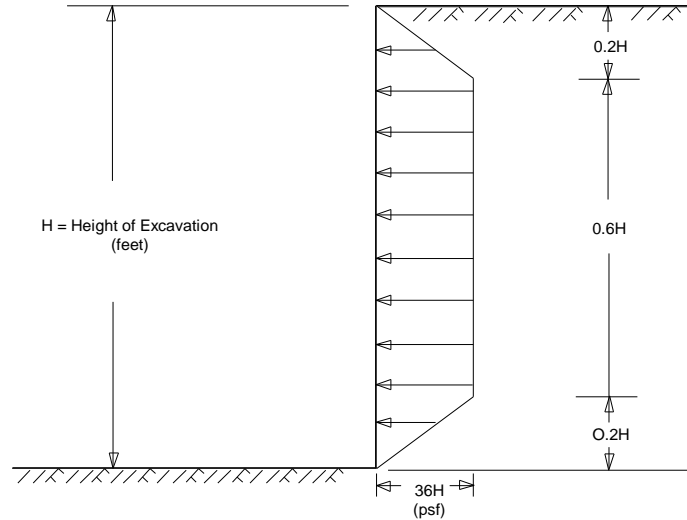
For major excavation or where restrictions do not permit back-sloping, shoring should be utilized in accordance with recommendations for shoring as presented in Section 9.5. Personnel from NV5 should observe the excavation so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met.

Where sloped excavations are used, the tops of the slopes should be barricaded so that vehicles and storage loads are not located within 10 feet of the tops of excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. NV5 should be advised of such heavy loadings so that specific setback requirements may be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended along the tops of the slopes, to prevent runoff water from entering the excavation and eroding the slope faces.

### **9.3.1 Lateral Pressures**

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the drained soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 30 pcf. Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to  $24H$  in psf, where  $H$  is the height of the shored wall in feet.





Any surcharge (live, including traffic, or dead load) located within a 1:1 (H:V) plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The lateral load contribution of a uniform surcharge load located across the 1:1 (H:V) zone behind the excavation walls may be calculated by using Figure 6. Lateral load contributions of surcharges can be provided once the load configurations and layouts are known. As a minimum, a 2-foot equivalent soil surcharge is recommended to account for nominal construction loads.

#### 9.4 Dewatering

Groundwater was not encountered to the maximum depth explored of approximately 16 feet below the existing ground surface. Dewatering is not generally anticipated during the proposed construction. However, any cases of localized seepage or heavy precipitation should be monitored during construction. If necessary, dewatering may be achieved by means of excavating a series of shallow trenches directed by gradient (i.e., gravity) to sumps with pumps. In any case, the actual means and methods of any dewatering scheme should be established by a contractor with local experience. It is important to note that temporary dewatering, if necessary, will require a permit and plan that complies with RWQCB regulations. If excessive water is encountered, NV5 should be contacted to provide additional recommendations for temporary construction dewatering. Based on the subsurface exploration the onsite soils maybe considered to be relatively permeable.

#### 9.5 Trench Bottom Stability

The bottom of onsite excavations will likely expose medium dense to dense silty sand. These soils should provide a suitable base for construction of the pipelines. For the design of flexible conduits, a modulus of soil reaction ( $E'$ ), of 2,500 pounds per square inch is recommended.

While groundwater was not encountered during the geotechnical investigation, if these soils become wet or saturated they may be prone to settlement due to construction activities such as placement and compaction of backfill soils. Buried improvements underlain by these soils could also be damaged or subjected to unacceptable settlement due to subsidence of these soils. If wet or unusually soft conditions are encountered in the trench bottom, the bottom of the excavations will need to be stabilized. A typical stabilization method includes overexcavation of the soft or saturated soil and replacement with properly compacted fill, gravel or lean concrete to form a "mat" or stable working surface in the bottom of the excavation. There are other acceptable methods that can be implemented



to mitigate the presence of compressible soils or unstable trench bottom conditions, and specific recommendations for a particular alternative can be discussed based on the actual construction techniques and conditions encountered.

## **9.6 Pipe Bedding**

It is recommended that pipe bedding materials be placed in the trench to provide uniform support and protection for the pipe. Bedding is defined as that material supporting, surrounding and extending to one foot above the top of the pipe. A cement slurry may not be used as bedding. The bedding materials should be approved by the geotechnical consultant prior to hauling on site. A minimum six-inch layer of pipe bedding should be placed beneath the pipe consisting of sand or other granular material and shall have a minimum sand equivalent of 30. This zone shall be compacted to a minimum of 90 percent relative compaction. Care should be taken by the contractor during placement of the pipe bedding so that uniform contact between the bedding and pipe is attained. There should be sufficient clearance along the side of the utility pipe or line to allow for compaction equipment. The pipe bedding and cover shall be compacted under the haunches and alongside the pipe. Mechanical compaction and hand tamping near the pipe zone should be performed carefully as to not damage the pipe.

## **9.7 Backfill Placement and Compaction**

The majority of the on-site soils should generally be suitable for use as backfill material. Backfill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D1557. Trench backfill should be compacted in uniform lifts (not exceeding 6 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (ASTM D1557). Water jetting should not be used for compaction. Imported backfill should consist of granular, non-expansive soil with an Expansion Index of 20 or less and should not contain any contaminated soil, expansive soil, debris, organic matter, or other deleterious materials. The sand equivalent of the imported material shall be 20 or greater. Import material should be evaluated for suitability by the geotechnical consultant prior to transport to the site.

The upper 12 inches of subgrade soil and all rock base should be compacted to at least 95 percent. The moisture content of the backfill should be maintained within 2 percent of optimum moisture content during compaction. All backfill should be mechanically compacted. Flooding or jetting is not recommended and should not be allowed.

## **9.8 Foundations**

The tank ringwall foundation should be founded entirely in compacted fill prepared in accordance with Section 9.2. Recommendations for the design and construction of foundation system are presented below.

### **9.8.1 Design Parameters**

Ringwall foundations should be designed using the geotechnical design parameters presented in the following Table 3. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the latest edition of the California Building Code.

**Table 3**  
**Geotechnical Design Parameters**  
**Ringwall Footing for Proposed Water Tanks**

<b>Ringwall Foundation Dimensions</b>	Continuous ringwall foundation at least 24 inches in width and at least 36 inches below the lowest adjacent grade.
<b>Allowable Bearing Capacity (dead-plus-live load)</b>	<u>Compacted Fill</u> : 3,000 pounds per square foot (psf) A one-third increase is allowed for transient live loads from wind or seismic forces.
<b>Reinforcement</b>	Reinforce in accordance with requirements as provided by the project Structural Engineer.
<b>Allowable Coefficient of Friction</b>	0.30 0.10 in the event a vapor barrier is used.
<b>Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)</b>	300 pounds per cubic foot (pcf) One third increase in passive value may be used for wind and seismic loads. The total allowable lateral resistance may be taken as the sum of the frictional resistance and the passive resistance, provided that the passive bearing resistance does not exceed two-thirds of the total allowable resistance.

### 9.8.2 Settlement

Estimated settlements will depend on the foundation size and depth, and the loads imposed and the allowable bearing values used for design. For preliminary design purposes, the total static settlement for the continuous ringwall foundation loaded to accordance with the allowable bearing capacities recommended above is estimated to be less than 1 inch.

Differential settlements will depend on the foundation size and depth, and the loads imposed. However, based on our knowledge of the project, differential static settlements are anticipated to be 0.5 inch or less.

### 9.8.3 Lateral Loads

Lateral loads may be resisted by friction and by the passive resistance of the supporting soils. A coefficient of friction of 0.30 may be used between foundations and the compacted fill materials; in the event that a vapor barrier is employed, a reduced coefficient of friction of 0.10 should be used for these the affected areas. The passive resistance of compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot (pcf). A one-third increase in the passive value may be used for wind or seismic loads. The passive resistance of the materials may be combined with the frictional resistance provided the passive component does not exceed two-thirds of the total lateral resistance. For the design of thrust blocks, refer to Figure 7, Thrust Block Lateral Earth Pressure Detail

### 9.8.4 Foundation Observation

To verify the presence of satisfactory materials at design elevations, footing excavations should be observed to be clean of loosened soil and debris before placing steel or concrete and probed for soft areas.

### 9.9 Foundations for Ancillary Structures

A shallow foundation system may be used for support of relatively lightly loaded ancillary structures, such as site screen walls, courtyard shelters, light standards, trash enclosures, etc. The foundations for each feature should be supported entirely on compacted fill prepared in accordance with the recommendations in Section 9.2 of this report. Recommendations for the design and construction of these shallow foundations are presented below.

#### 9.9.1 Design Parameters – Ancillary Structures

Shallow foundations should be designed using the geotechnical design parameters presented in the following Table 4. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the latest edition of the California Building Code.

**Table 4**  
**Geotechnical Design Parameters**  
**Spread Footing Foundations for Ancillary Structures**

<b>Foundation Dimensions</b>	At least 18 inches below the lowest adjacent grade  At least 12 inches in width
<b>Allowable Bearing Capacity (dead-plus-live load)</b>	1,500 pounds per square foot (psf). The allowable bearing value may not be increased by one-third for transient live loads from wind or seismic in view of the potential for liquefaction at shallow depths below foundation elements.
<b>Estimated Static Settlement (Total/Differential)</b>	Less than 1-inch/ less than ½-inch
<b>Allowable Coefficient of Friction</b>	0.35
<b>Allowable Lateral Passive Resistance</b>	250 pounds per cubic foot (pcf; EFP)

The total allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance. The passive resistance values may be increased by one-third when considering wind or seismic loading.

## 9.10 Seismic Design Parameters

Preliminary seismic design parameters for the project site were also developed as per the guidelines outlined in the 2012 IBC (2008 USGS hazard data) and 2010 ASCE 7-10 Standard (with errata as of April 2013). **NV5 should be contacted to provide revisions to these parameters if other codes are specified.** The seismic design parameters for Site Class “C” were developed using a JAVA™ application, Java Ground Motion Parameter Calculator–Version 5.0.9 available on the USGS website (<http://earthquake.usgs.gov>). The preliminary seismic design parameters for the project site are presented in Table 5 below.

**Table 5  
2012 IBC Seismic Design Parameters  
And ASCE 7-10 Standard**

Parameter	Value
Site Class; (Section 11.4.2)	C
Mapped Spectral Accelerations for short periods, $S_S$ ; (Section 11.4.1)	2.569g
Mapped Spectral Accelerations for 1-sec period, $S_1$ ; (Section 11.4.1)	0.853g
Site Coefficient, $F_a$ ; (Table 11.4-1)	1.0
Site Coefficient, $F_v$ ; (Table 11.4-2)	1.3
Maximum considered earthquake spectral response acceleration for short periods, $S_{MS}$ adjusted for Site Class (Equation 11.4-1)	2.569g
Maximum considered earthquake spectral response acceleration at 1-sec period, $S_{M1}$ adjusted for Site Class (Equation 11.4-2)	1.109g
Five-percent damped design spectral response acceleration at short periods, $S_{DS}$ ; (Equation 11.4-3)	1.713g
Five-percent damped design spectral response acceleration at 1-sec period, $S_{D1}$ ; (Equation 11.4-4)	0.739g

## 9.11 Soil Corrosion

Laboratory testing was performed on a representative sample of the on-site soils to evaluate pH, minimum resistivity, and chloride and soluble sulfate content. Table 6 presents the results of the corrosivity testing.

**Table 6**  
**Corrosivity Test Results**

<b>Test Location</b>	Exploratory Boring B-1
<b>Depth (feet)</b>	3 – 4
<b>pH</b>	6.6
<b>Resistivity (ohm-cm)</b>	4800
<b>Chloride Content (ppm)</b>	32
<b>Soluble Sulfate Content (ppm)</b>	36

Based on our experience and various publications including the Caltrans Corrosion Guidelines dated November 2012, the chloride and sulfate content is considered to have a negligible corrosivity potential to steel and concrete. The soil resistivity and pH level reported are considered to be mildly corrosive to concrete. It is our recommendation that a corrosion specialist be contacted to determine if measures are necessary.

## **10.0 CONSTRUCTION OBSERVATION AND TESTING**

Observation and testing of the placement and compaction of backfill, subgrade and base will be important to the performance of the proposed project. Site preparation, removal of unsuitable soils, assessment of imported fill materials, backfill placement, and other earthwork operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the exploratory borings. Continuous observation by a representative of NV5 during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

## **11.0 LIMITATIONS**

The recommendations and opinions expressed in this report are based on NV5's review of background documents and on information obtained from field explorations. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during construction, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of 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 NV5 has no control.

NV5's recommendations for this site are, to a high degree, dependent upon appropriate quality control of construction operations, placement and compaction of backfill, subgrade preparation, etc. Accordingly, the recommendations are made contingent upon the opportunity for NV5 to observe the earthwork operations for the proposed construction. If parties other than NV5 are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

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. NV5 should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

NV5 has endeavored to perform this geotechnical evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions.

## 12.0 REFERENCES

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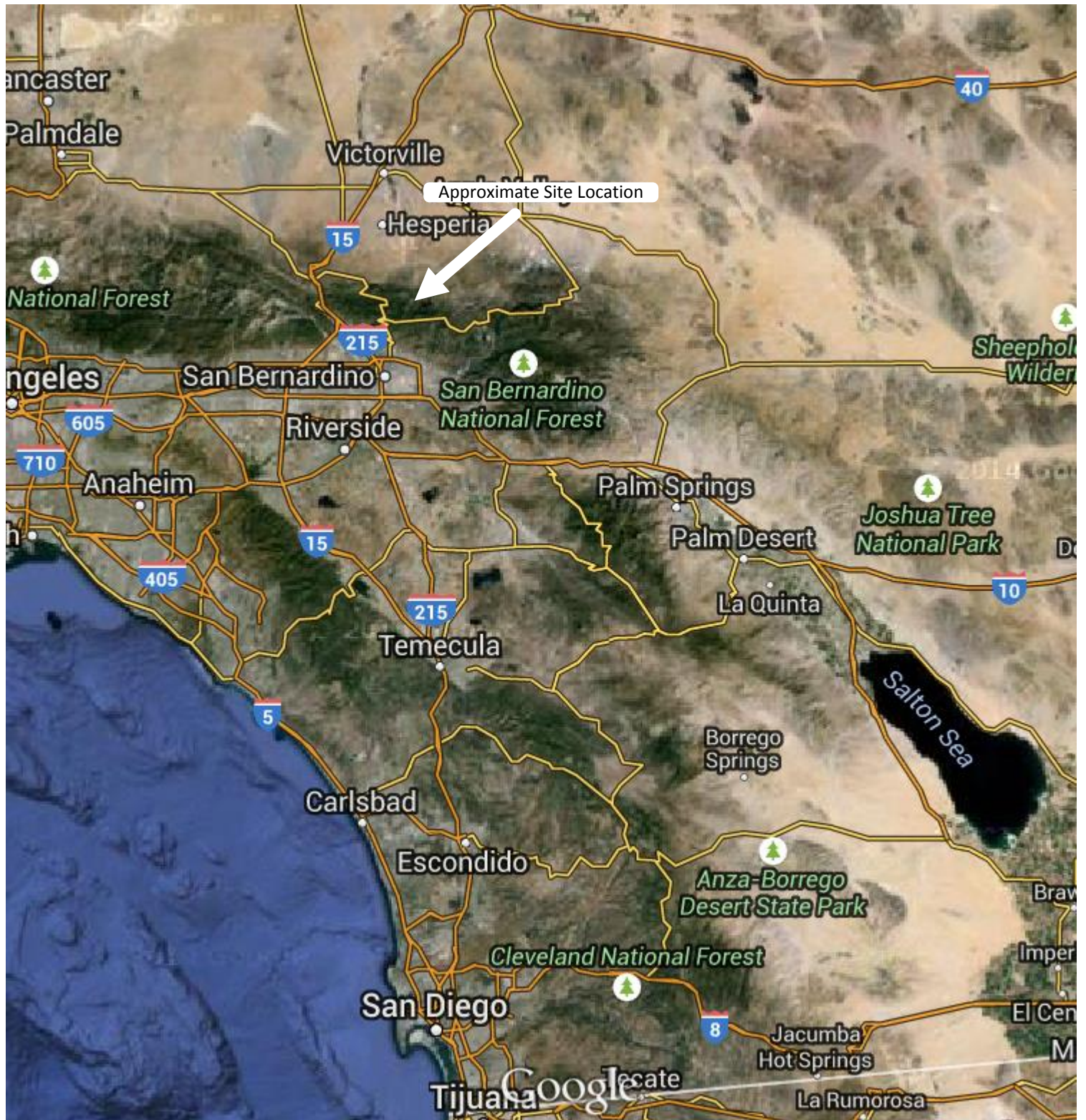
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## Figures

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Reference: Google Maps 2015



No Scale



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Drawn: **GC**

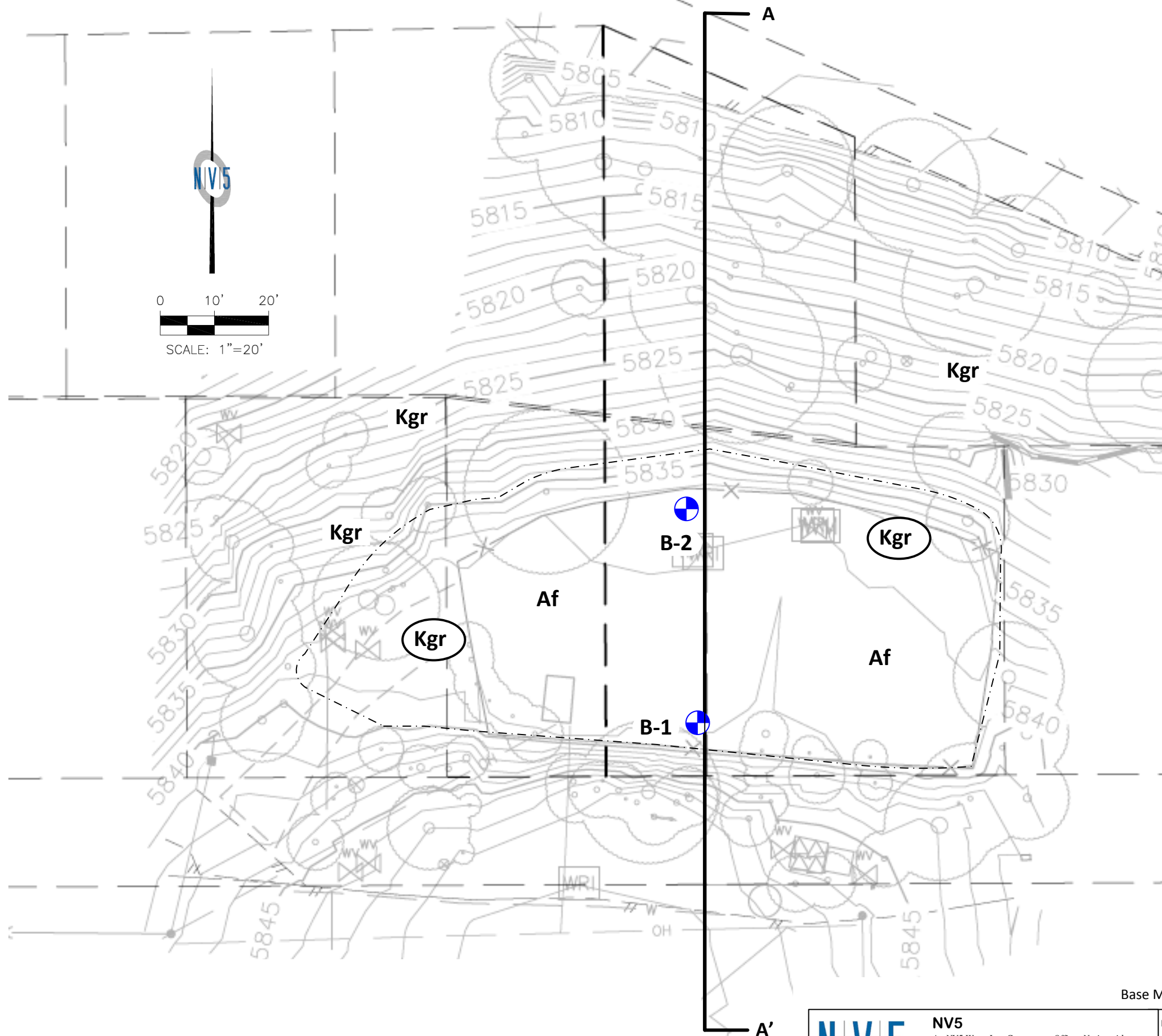
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**Site Location Map**  
**Proposed AVMSC Water Tanks Site**  
**Lake Arrowhead, California**


Figure No. 1







**LEGEND**


 Approximate location of exploratory boring

**B-2**

**Map Symbols**

**Af** Compacted fill soils placed during previous grading of the site

**Kgr** Weathered Cretaceous granitic bedrock (circled where buried)

 Approximate geologic contact

**A A'** Approximate location of geologic cross section

Not a construction drawing.

Base Map: Adapted from an undated topographic map provided by NV5 Infrastructure.

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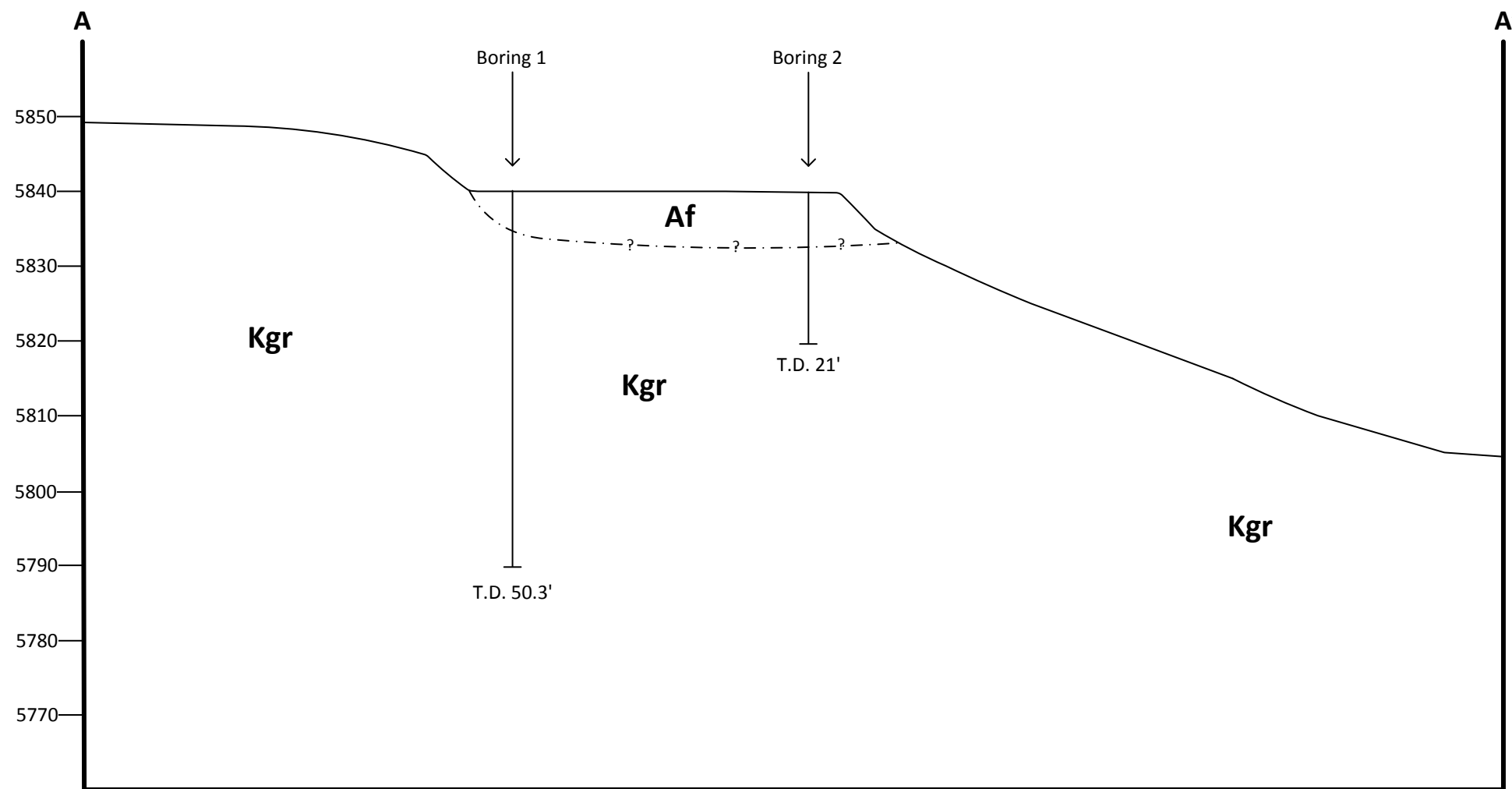
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**Geotechnical Map**  
**Proposed AVMSC Water Tanks Site**  
**Lake Arrowhead, California**



Figure No. 2

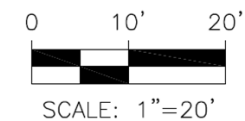






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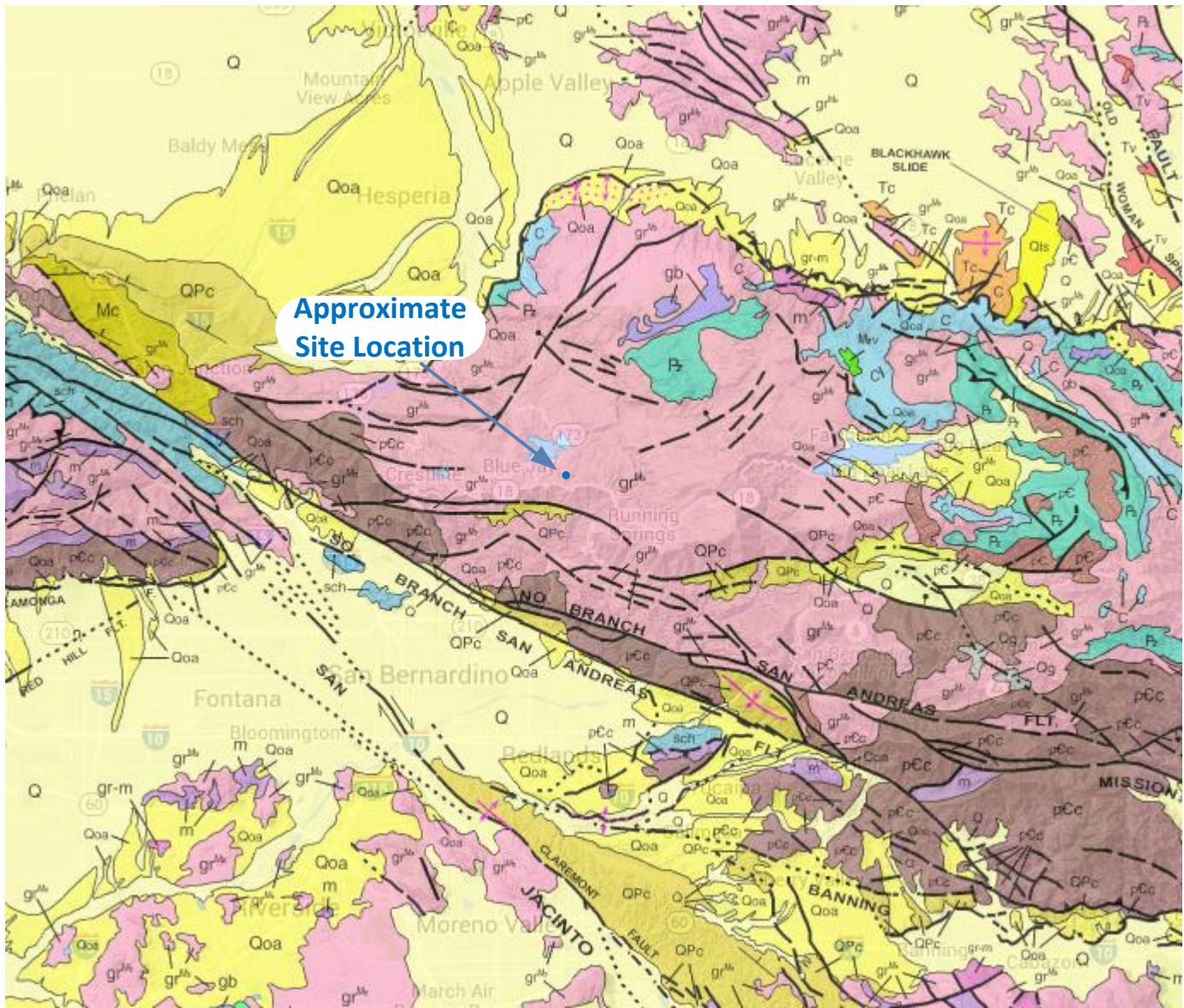
-  Approximate location of exploratory boring
- B-2**
- Map Symbols**
- Af** Compacted fill soils placed during previous grading of the site
- Kgr** Weathered Cretaceous granitic bedrock (circled where buried)
-  Approximate geologic contact



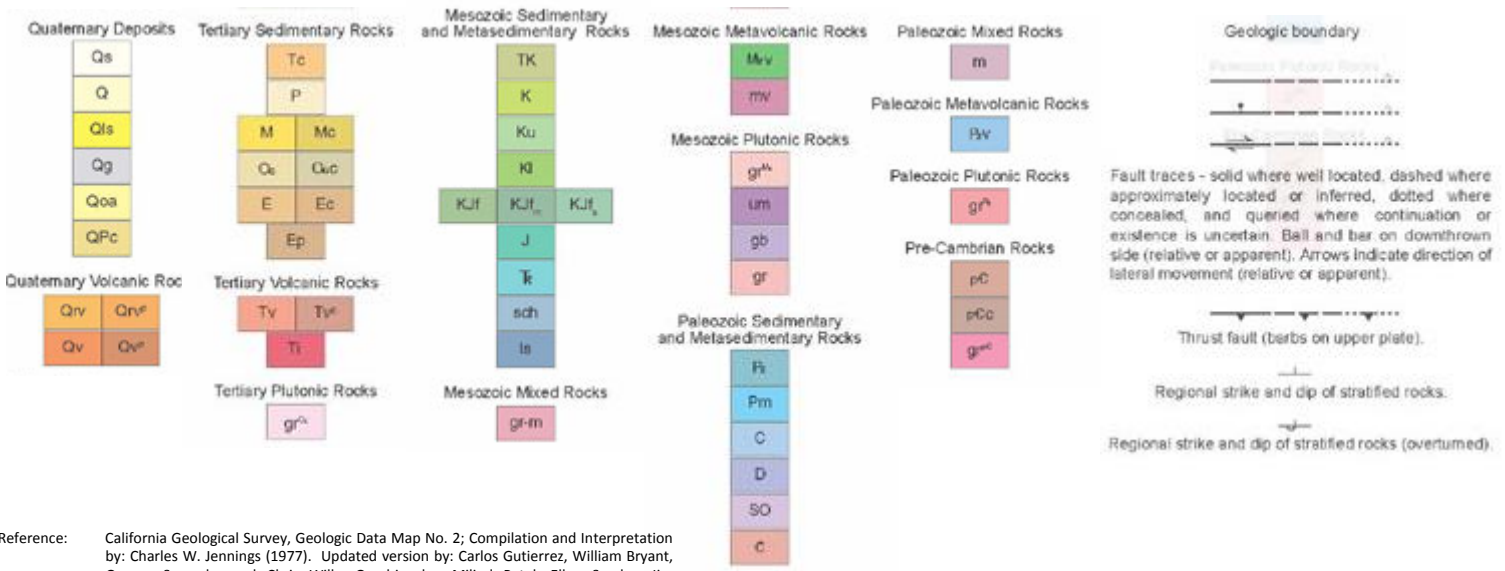
  
 Trend of Section: S – N

Not a construction drawing.





Approximate Site Location



Reference: California Geological Survey, Geologic Data Map No. 2; Compilation and Interpretation by: Charles W. Jennings (1977). Updated version by: Carlos Gutierrez, William Bryant, George Saucedo, and Chris Wills. Graphics by: Milind Patel, Ellen Sander, Jim Thompson, Barbara Wanish and Milton Fonseca Plesch, Andreas et. al., 2007, Community Fault Model (CFM) for Southern California; in the *Bulletin of the Seismological Society of America*, Vol. 97, No. 6. pp. 1793-1802, dated December.

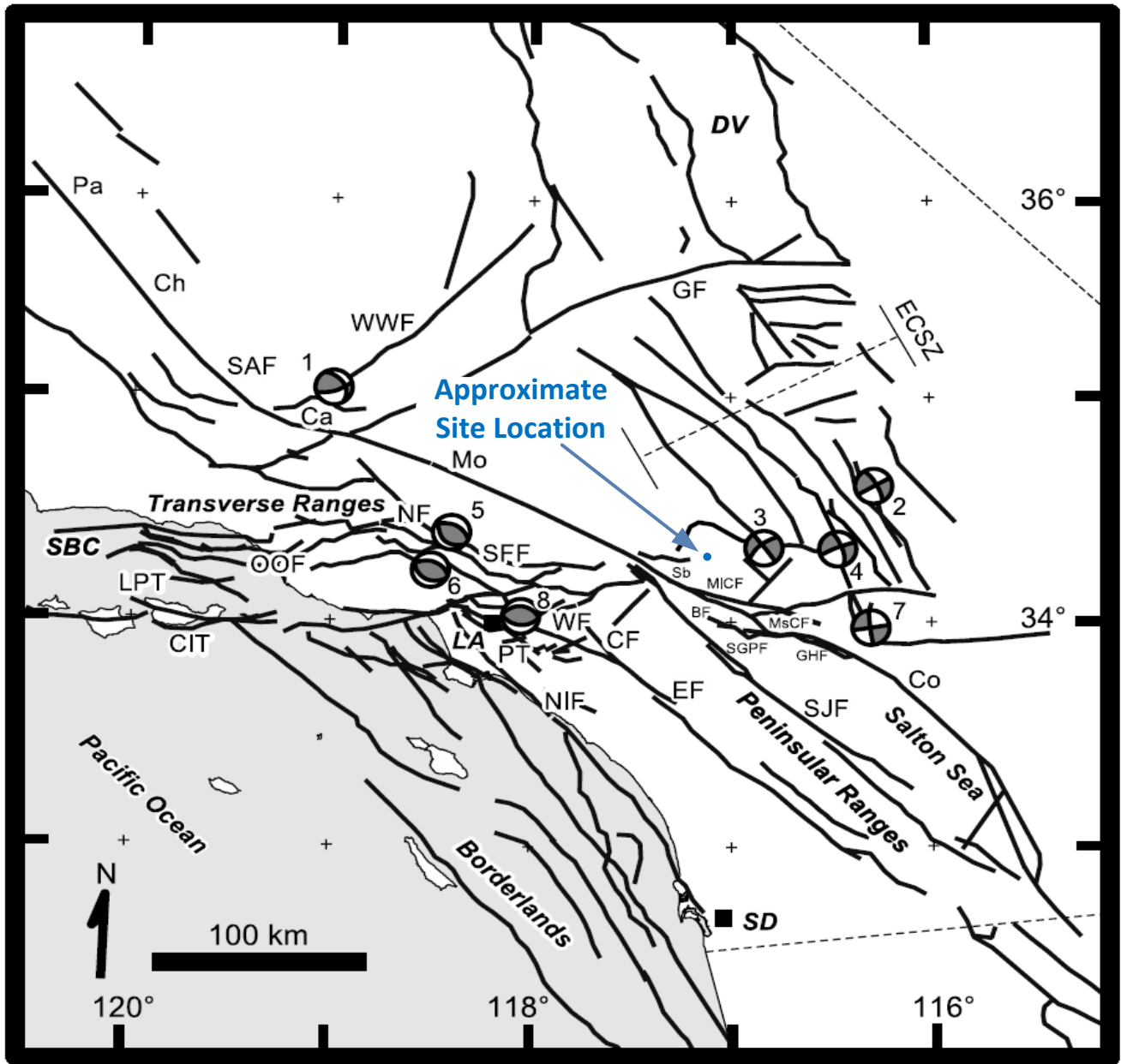
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**Regional Geologic Map**  
**Proposed AVMSC Water Tanks Site**  
**Lake Arrowhead, California**  
 Figure No. 4





Map of southern California showing the geographic regions, faults and focal mechanisms of the more significant earthquakes. **Regions:** Death Valley, DV; Mojave Desert MD; Los Angeles, LA; Santa Barbara Channel, SBC; and San Diego, SD. **Indicated Faults:** Banning fault, BF; Channel Island thrust, CIT; Chino fault, CF; Eastern California Shear Zone, ECSZ; Elsinore fault, EF; Garlock fault, GF; Garnet Hill fault, GHF; Lower Pitas Point thrust, LPT; Mill Creek fault, MICF; Mission Creek fault, MsCF; Northridge fault, NF; Newport Inglewood fault, NIF; offshore Oak Ridge fault, OOF; Puente Hills thrust, PT; San Andreas fault (sections: Parkfield, Pa; Cholame, Ch; Carrizo; Ca; Mojave, Mo; San Bernardino, Sb; and Coachella, Co); San Fernando fault, SFF; San Gorgonio Pass fault, SGPF; San Jacinto fault, SJF; Whittier fault, WF; and White Wolf fault, WWF. **Earthquake Focal Mechanisms:** 1952 Kern County, 1; 1999 Hector Mine, 2; 1992 Big Bear, 3; 1992 Landers, 4; 1971 San Fernando, 5; 1994 Northridge, 6; 1992 Joshua Tree, 7; and 1987 Whittier Narrows, 8.

Reference: Plesch, Andreas et. al., 2007, Community Fault Model (CFM) for Southern California; in the *Bulletin of the Seismological Society of America*, Vol. 97, No. 6. pp. 1793-1802, dated December.

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**Regional Fault Map**  
**Proposed AVMSC Water Tanks Site**  
**Lake Arrowhead, California**

Figure No. 5

## **Appendix A**









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Exploratory Boring Logs





**LOG SYMBOLS:**

	Bulk/Bag sample		Water level (level after completion)
	California sampler (2-1/2 inch outside diameter)		Water level (level where first encountered)
	Modified California sampler (3 inch outside diameter)	<b>Abbreviations:</b>	
	Standard penetration Split spoon sampler (2 inch outside diameter)	SA - Sieve Analysis	
	NX size core barrel (2-5/8 inch outside diameter)	P200 - Percent passing #200 sieve	
	Shelby tube	AL - Atterberg Limits	
		LL - Liquid limit	
		DS - Direct shear test	
		'R' - R-value test	
		CS - Corrosivity test	
		EI - UBC expansion index	
		MD - Laboratory compaction test	
		CN - Consolidation test	

**General Notes:**

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, unified soil classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

**Consistency criteria based on field tests**

Relative density	SPT* (# blows/ft)	Relative density (%)	Consistency	SPT (# blows/ft)	Torvane	Pocket** penetrometer
					Undrained shear strength (tsf)	Unconfined compressive strength
Very Loose	<4	0 - 15	Very soft	<2	<0.13	<0.25
Loose	4 - 10	15 - 35	Soft	2 - 4	0.13 - 0.25	0.25 - 0.5
Medium Dense	10 - 30	35 - 65	Medium stiff	4 - 8	0.25 - 0.5	0.5 - 1.0
Dense	30 - 50	65 - 85	Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0
Very dense	>50	85 - 100	Very stiff	15 - 30	1.0 - 2.0	2.0 - 4.0
			Hard	>30	>2.0	>4.0

\* Number of blows of 140 pounds hammer falling 30 inches to drive a 2 inch C.D. (1 3/8" I.D.) split barrel sampler (ASTM - 1386 standard penetration test)

\*\* Unconfined compressive strength in Tons/ft<sup>2</sup>. Read from pocket penetrometer

**Moisture content**

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

**Cementation**

Description	Field test
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure



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 Date: **July 2015**

Title:  
 Project:

**Log Legend**  
**AVMSC Water Tanks Site**  
**Lake Arrowhead, California**

### Soil Classification Chart

Major Divisions			Symbols		Typical Descriptions	
			Graph	Letter		
Coarse Grained Soils	Gravel and Gravelly soils	Clean Gravels (Little or no fines)		GW	Well-Graded Gravel, Gravel SAND mixtures, little or no fines	
		Gravels with fines (Appreciable amount of fines)		GP	Poorly-Graded Gravels, Gravel - SAND mixtures, little or no fines	
		more than 50% of coarse fraction retained on No. 4 sieve	Clean SANDS (Little or no fines)		SW	Well-Graded SANDS, Gravelly, SANDS, little or no fines
			Sands with Fines (Appreciable amount of fines)		SM	Silty SANDS, SAND-Silt mixtures
More than 50% of material is larger than No. 200 sieve size	Sand and Sandy Soils	SANDS with Fines (Appreciable amount of fines)		SC	Clayey SANDS, SAND - Clay mixtures	
	More than 50% of coarse fraction passing on No.4 sieve					
Fine grained soils	Silt and Clays	Liquid Limit less than 50		ML	Inorganic Silts and very fine SANDS, rock flour, Silty or Clayey fine SANDS or clayey Silts with slight Plasticity	
				CL	Inorganic Clays of low to medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays	
				OL	Organic Silts and organic Silty Clays of low Plasticity	
	More than 50% of material is smaller than No. 200 sieve size	Silt and Clays	Liquid Limit Greater than 50		MH	Inorganic Silts, micaceous or diatomaceous fine SAND or Silty Soils
					CH	Inorganic Clays of high Plasticity
					OH	Organic Clays of medium to High Plasticity, organic Silts
Highly organic soils				PT	Peat, Humus, swamp soils with High organic contents	

NOTE: Dual symbols are used to indicate borderline soil classifications.



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 Date: **July 2015**

Title: **Log Legend – Soil Classification**  
 Project: **AVMSC Water Tanks Site**  
**Lake Arrowhead, California**

Chart No. 2



Project: AVMSC Water Tanks Site  
 Project Location: Lake Arrowhead  
 Project Number: 149745

**Boring B-1**

Sheet 1 of 1

Date(s) Drilled	June 8, 2015	Logged By	G. Custenborder	Checked By	G. Gau
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	±5839 feet
Drilling Contractor	2R Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain
Drill Rig Type:	LAR 75 Hollow Stem Auger	Location:	South Side, Middle of Tank Pad	Lat. - Long.:	

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION <small>This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.</small>	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0					<b>Gravel Surfacing</b> - 6 inches of 1½-inch diameter angular crushed rock			
1				SM	<b>@ 6" Fill</b> - light brown, moist, loose to medium dense fine to coarse SAND			
2								
3								
4			Bag 1					
5								
6		6						
6		6	SPT 1			12.3		
8		8		SM	<b>@ 6" Decomposed Granite</b> - light brown, moist, very dense silty fine to coarse SAND			
10					becomes damp, very hard drilling			
10		12						
10		21	Cal 1			8.8	111.5	
10		46						
10			Bag 2					
15								
15		32						
15		50	SPT 2			2.6		
20								
20		14						
20		21	CAL 2			6.6	107.5	
20		32						
25								
25		9						
25		10						
25		11	SPT 3			6.8		
30								

Sample Type

Cal. Mod.  
 SPT  
 Bulk  
 Other  
 No Recovery



Project: AVMSC Water Tanks Site  
 Project Location: Lake Arrowhead  
 Project Number: 149745

**Boring B-1**

Sheet 2 of 2

Date(s) Drilled	June 8, 2015	Logged By	G. Custenborder	Checked By	G. Gau
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	±5839 feet
Drilling Contractor	2R Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain
Drill Rig Type:	LAR 75 Hollow Stem Auger	Location:	South Side, Middle of Tank Pad	Lat. - Long.:	

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION		Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
					<p>This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.</p>				
30		31 50/5'	Cal 3	SM	Boring 1 continued: Decomposed Granite - light brown, damp, very dense silty fine to coarse SAND		4.8	112.8	
35		21 32 37	SPT 4				5.3		
40		57/6'	Cal 4				3.9	108.5	
45		39 50/5'	SPT 5				3.5		
50		50/4'	Cal 4				6.7		
55					Total depth: 50.3 feet No refusal Groundwater not encountered Boring backfilled 6-8-15				
60									

Sample Type

Cal. Mod.   
 SPT   
 Bulk   
 Other   
 No Recovery



Project: AVMSC Water Tanks Site  
 Project Location: Lake Arrowhead  
 Project Number: 149745






**Boring B-2**

Sheet 1 of 1

Date(s) Drilled	June 8, 2015	Logged By	G. Custenborder	Checked By	G. Gau
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	±5838 feet
Drilling Contractor	2R Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain
Drill Rig Type:	LAR 75 Hollow Stem Auger	Location:	South Side, Middle of Tank Pad	Lat. - Long.:	

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0					<b>Gravel Surfacing</b> - 6 inches of 1½-inch diameter angular crushed rock			
1				SM	<b>@ 6" Fill</b> - light brown, moist, loose to medium dense fine to coarse SAND			
2								
3								
4	Bag 1							
5		5	SPT 1	SM		6.7		
6		2						
7		1						
8								
9	Bag 2				<b>@ 8" Decomposed Granite</b> - light brown, moist, very dense silty fine to coarse SAND			
10		43	Cal 1			3.4	107.3	
11		50/3"						
12								
13								
14								
15		7	SPT 2			5		
16		21						
17		28						
18								
19								
20		46	CAL 2		becomes damp, harder drilling	2.7	106.8	
21		50/3.5						
22								
23								
24								
25								
26								
27								
28								
29								
30								

Total depth: 21.0 feet  
 No refusal  
 Groundwater not encountered  
 Boring backfilled 6-8-15

	Cal. Mod.		SPT		Bulk		Other		No Recovery
---	-----------	---	-----	---	------	---	-------	---	-------------

Sample Type

## **Logs of Exploratory Borings**

Bulk and relatively undisturbed drive samples were obtained in the field during our subsurface evaluation. The samples were tagged in the field and transported to our laboratory for observation and testing. The drive samples were obtained using the Standard Penetration Test (SPT) samplers as described below.

### **California Modified Split Spoon Sampler**

The split barrel drive sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil sample within the rings is removed, sealed, and transported to the laboratory for observation and testing.

### **Standard Penetration Test (SPT) Sampler**

The split barrel sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of 2.0 and 1.5 inches, respectively. The soil sample obtained in the interior of the barrel is measured, removed, sealed and transported to the laboratory for observation and testing.

## **Appendix B**

---

Laboratory Test Results





## SUMMARY OF LABORATORY TEST RESULTS

### In-situ Moisture and Density Tests

The in-situ moisture contents and dry densities of selected samples obtained from the test borings were evaluated in general accordance with the latest version of D-2216 and D2937 laboratory test methods. The method involves obtaining the moist weight of the sample and then drying the sample to obtain its dry weight. The moisture content is calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample and expressing the result as a percentage. The results of the in-situ moisture content and density tests are presented in the following table and on the logs of exploratory borings in Appendix A.

### RESULTS OF MOISTURE CONTENT AND DENSITY TESTS (ASTM D2216)

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring 1 @ 5 - 6.5' feet	12.3	density not determined
Boring 1 @ 11 - 11.5 feet	8.8	111.5
Boring 1 @ 15 - 16.5 feet	2.6	density not determined
Boring 1 @ 21 - 21.5 feet	6.6	107.5
Boring 1 @ 25 - 26.5 feet	6.8	density not determined
Boring 1 @ 30 - 31 feet	4.8	112.8
Boring 1 @ 35 - 36.5 feet	5.3	density not determined
Boring 1 @ 40 - 40.5 feet	3.9	108.5
Boring 1 @ 45 - 46 feet	3.5	density not determined
Boring 2 @ 5 - 6.5 feet	6.7	density not determined
Boring 2 @ 10 - 11 feet	3.4	107.3
Boring 2 @ 15 - 16.5 feet	5.0	density not determined
Boring 2 @ 20 - 20.5 feet	2.7	106.8

### Classification

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

## Particle-size Distribution Tests

An evaluation of the grain-size distribution of selected soil samples was performed in general accordance with the latest version of ASTM D-422 (including –200 wash). These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System. Particle size distribution test results are presented on the laboratory test sheets attached in this appendix.

## Direct shear

A direct shear test was performed on a representative undisturbed sample in accordance with ASTM D3080 to evaluate the shear strength characteristics of the on-site materials. The test method consists of placing the soil sample in the direct shear device, applying a series of normal stresses, and then shearing the sample at the constant rate of shearing deformation. The shearing force and horizontal displacements are measured and recorded as the soil specimen is sheared. The shearing is continued well beyond the point of maximum stress until the stress reaches a constant or residual value. The results of the tests are presented in the following table and attached in this appendix.

RESULTS OF DIRECT SHEAR TEST  
(ASTM D3080)

Location	Angle of Internal Friction (degrees)	Cohesion Intercept (psf)	Notes
Boring 1 @ 11-11.5 ft.	34.5	127	undisturbed

## Sand Equivalent Test

A sand equivalent test was performed on a sample of the on-site soils. The test was performed in General accordance with California Test Method 217. The result of the test is presented below and attached in this appendix.

Sample Location	B-2 @ 3-4 ft
Sand Equivalent Value	21

## Soil Corrosivity Tests

Soluble sulfate, chloride, resistivity and pH tests were performed in accordance with California Test Methods 643, 417 and 422 to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel.

### RESULTS OF CORROSIVITY TESTS (CTM 417, CTM 422)

Sample Location	B-1 @ 3-4 ft
pH	6.6
Resistivity (Ohm-cm)	4800
Sulfates (ppm)	36
Chlorides (ppm)	32



NV5  
15070 Avenue of Science, suite 100  
San Diego, CA 92128

July 7, 2015

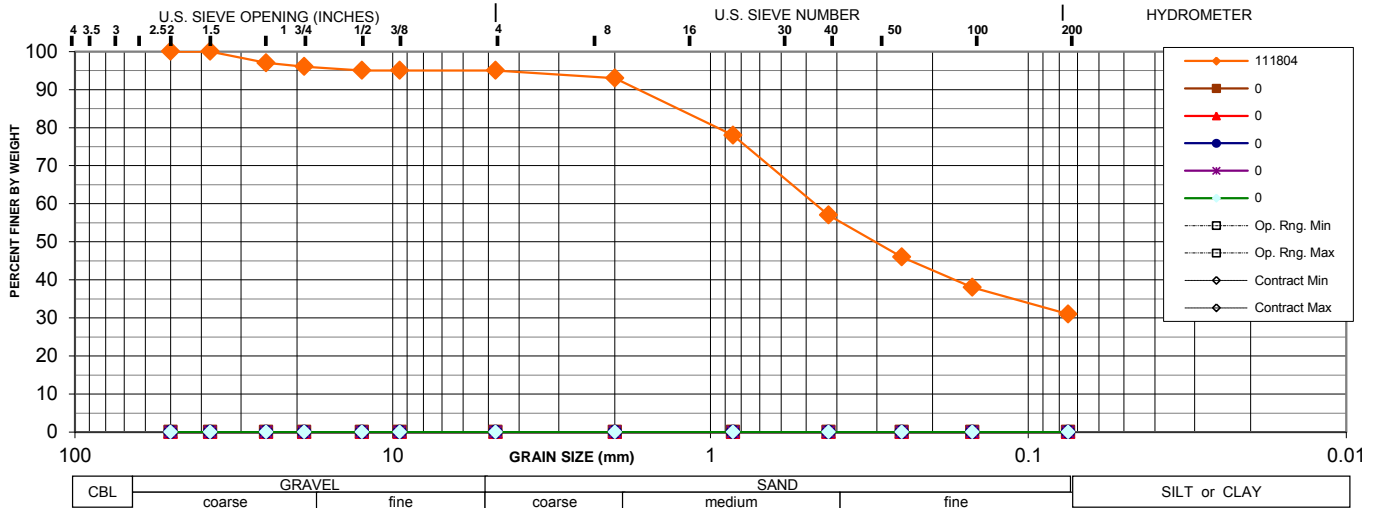
JOB No: 149745  
Report No: 3781

**Project:** Arrowhead Villas MSC Water Tanks

**REPORT OF SIEVE ANALYSIS TEST**  
ASTM D422 - Soil

SAMPLE INFO: 111804

Material	SM				
Color	--				
Sample Location	B-1 @ 3-4 ft.				
Date Sampled	6/8/2015				
Sampled By	G. Custenborder				
Date Tested	6/11/2015				
Tested By	Darrel Delgado				



Sieve Size	% Passing
63mm (2 1/2")	
50mm (2")	100
37.5mm (1 1/2")	100
25mm (1")	97
19mm (3/4")	96
12.5mm (1/2")	95
9.5mm (3/8")	95
4.75mm (#4)	95
2mm (#10)	93
850µm (#20)	78
425µm (#40)	57
250µm (#60)	46
150 µm (#100)	38
75 µm (#200) wash	31.0
Fineness Modulus	1.5
Shape (sand & gravel)	N.R.
Hardness (sand & gravel)	N.R.
Specific Gravity	2.65
Coef. of Curvature (C <sub>c</sub> )	6.6
Coef. of Uniformity (C <sub>u</sub> )	781.0
% Gravel	5
% Sand	64
% Fines	31.0
USCS Class:	SM

Notes: Hardness: H&D = Hard & Durable; W&F = Weathered & Friable  
N.R.: Not Recorded; N/A: Not Available.

Reviewed By: *[Signature]*

As mutual protection to clients, the public, and BVNA, all reports are the confidential property of clients. Authorization for publication of statements, conclusions, or extracts from our reports is reserved pending written approval.

Project No. **149745.00**  
 Client: **NV5**  
 Proj. Name: **Arrowhead Villas MSC Water Tanks**  
 Location: **Lake Arrowhead, CA**  
 Sample date **6/8/2015**      Sample Location: **B-1**

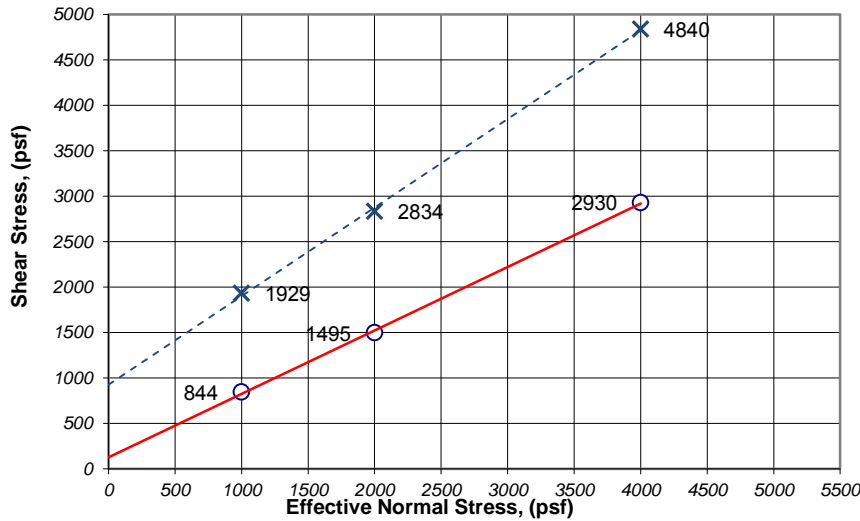
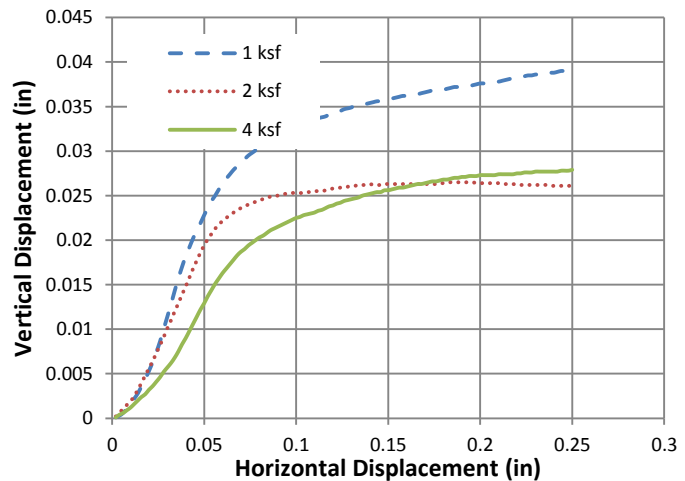
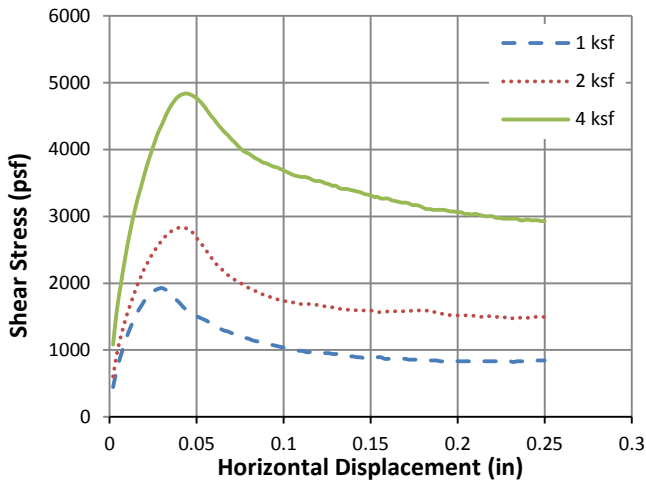
Report No.: **3871**  
 Lab No.: **111806**  
 Test Date: **6/29/2015**

Depth: **11-11.5 ft.**

**TEST DATA:**

Sample ID:		1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	9.1	8.4	8.3
	Dry Density (%)	115.4	114.6	118.5
	Saturation (%)	58.4	52.5	58.6
Final	Water Content (%)	19.6	19.1	17.9
	Dry Density (%)	115.4	114.6	118.5
	Saturation (%)	125.4	119.0	125.8
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		844	1495	2930
Peak Shear Stress (psf)		1929	2834	4840

Sample Type: Undisturbed  
 Description: Tan / Brown Fine SAND



— Linear (Ultimate Strength Envelope)  
 - - - Linear (Peak Strength Envelope)

Peak  
 $y = 0.975x + 926$

Ultimate  
 $y = 0.6985x + 126.5$

Peak Cohesion, C'(psf): **926**      Ultimate Cohesion, C'(psf): **127**  
 Peak Friction,  $\Phi'$  (deg): **44.3**      Ultimate Friction,  $\Phi'$  (deg): **34.5**

**DIRECT SHEAR TEST (ASTM D3080)**

Reviewed By: *[Signature]*



**NV5 West, Inc.**  
 7895 Convoy Court, Suite 18  
 San Diego CA 92111  
 p. 858 715 5800 f. 858 715 5810



**SAND EQUIVALENT TEST**

Date: July 7, 2015  
Job No: 149745  
Client: NV5  
Address: 15070 Avenue of Science #100  
San Diego, CA 92128  
Attention: James Owens

Report No: **3781**  
Project: Arrowhead Villas MSC Water Tanks  
Sampled By: Gene Custenborder  
Date Received: 6/9/15

<b>SAND EQUIVALENT VALUE</b> (CTM 217)	
Sample ID	111814
Location	B-2
Depth	3 - 4 ft.
<b>Sand Equivalent Value</b>	<b>21</b>

Gene Custenborder  
Engineering Geologist



L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993

Fax 425-7917

Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.  
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com  
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: June 12, 2015  
Purchase Order Number: 15-0332  
Sales Order Number: 27333  
Account Number: TESE.R1

To:  
\*-----\*

NV5 West Inc  
7895 Convoy Court, Suite 18  
San Diego, CA 92111  
Attention: Guillaume Gau

Laboratory Number: S05709      Customers Phone: 858-715-5800  
Fax: 858-715-5810

Sample Designation:  
\*-----\*

One soil sample received on 06/10/15 at 12:35pm,  
taken on 06/10/15 from Job# 149745.00 Arrowhead Villas MSC Water Tanks  
marked as B1 @ 3-4' Lab# 111804 Report# 3781.

Analysis By California Test 643, 1999, Department of Transportation  
Division of Construction, Method for Estimating the Service Life of  
Steel Culverts.

pH 6.6

Water Added (ml)	Resistivity (ohm-cm)
10	12000
5	7800
5	5700
5	5100
5	4800
5	5000
5	5300

27 years to perforation for a 16 gauge metal culvert.  
36 years to perforation for a 14 gauge metal culvert.  
49 years to perforation for a 12 gauge metal culvert.  
63 years to perforation for a 10 gauge metal culvert.  
77 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417      0.004%      36 ppm

Water Soluble Chloride Calif. Test 422      0.003%      32 ppm



\_\_\_\_\_  
Rosa M. Bernal  
RMB/arr



## **Appendix C**

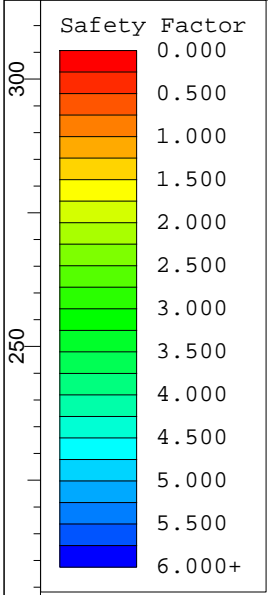
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Slope Stability Analyses

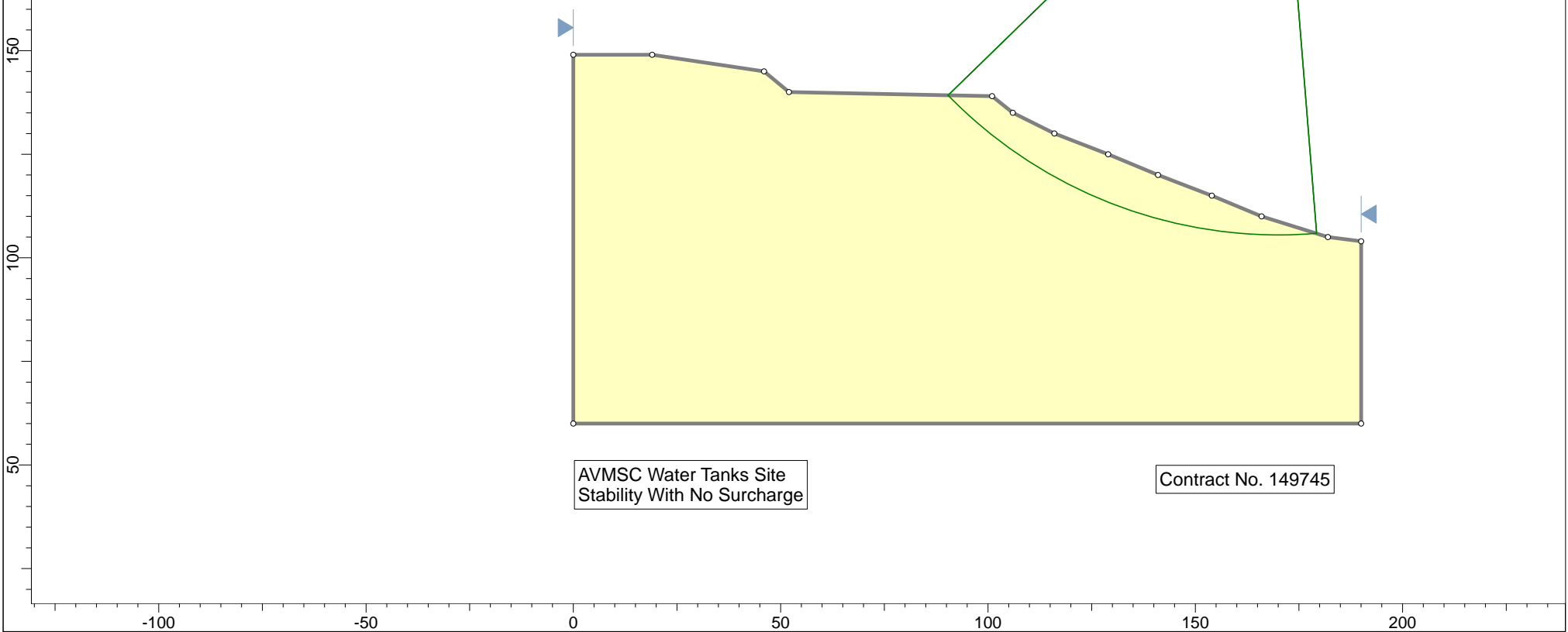
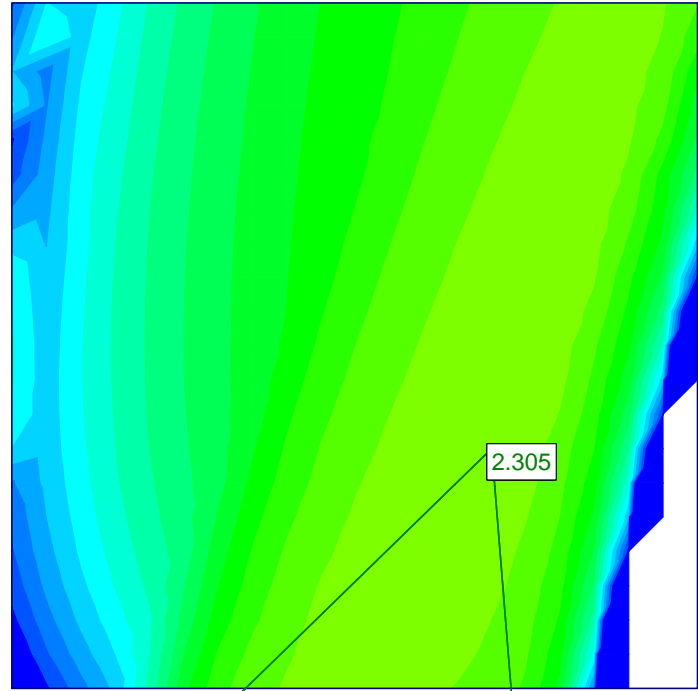


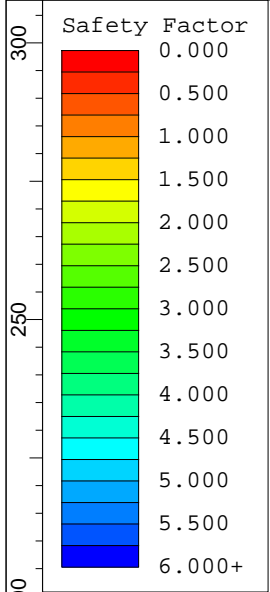
## **Slope Stability Calculations**

Slope stability analyses including static and pseudo-static conditions were performed to evaluate the global stability for the tank pad and adjacent slope. Three cases were analyzed. Case 1 evaluated the stability without the proposed tanks. Case 2 was modeled to include loads from the water tanks, and Case 3 included an additional seismic loading. For purposes of modeling the tank loading, a load value of 2,000 psf across the relatively level portion of the tank pad was used in the analysis. The cross sections were analyzed utilizing the computer software program Slide, version 6.0, created by Rocscience®. The Bishop Simplified method of analysis was used to locate the critical slip surface for static and pseudo-static conditions. The subsurface conditions were modeled based upon data from the subsurface exploration program and the earth materials were assigned the strength characteristics based on their shear strength as tested in the laboratory. Based on our analysis the existing slope has a calculated minimum specified safety factor in excess of the currently accepted standard of 1.5 and 1.1 for static and seismic, respectively. A seismic factor (k) equal to a third of the horizontal peak bedrock acceleration (i.e., 0.267g) was added as a horizontal force, while the effects of vertical acceleration were omitted. The results of the stability analyses are presented in the following pages.

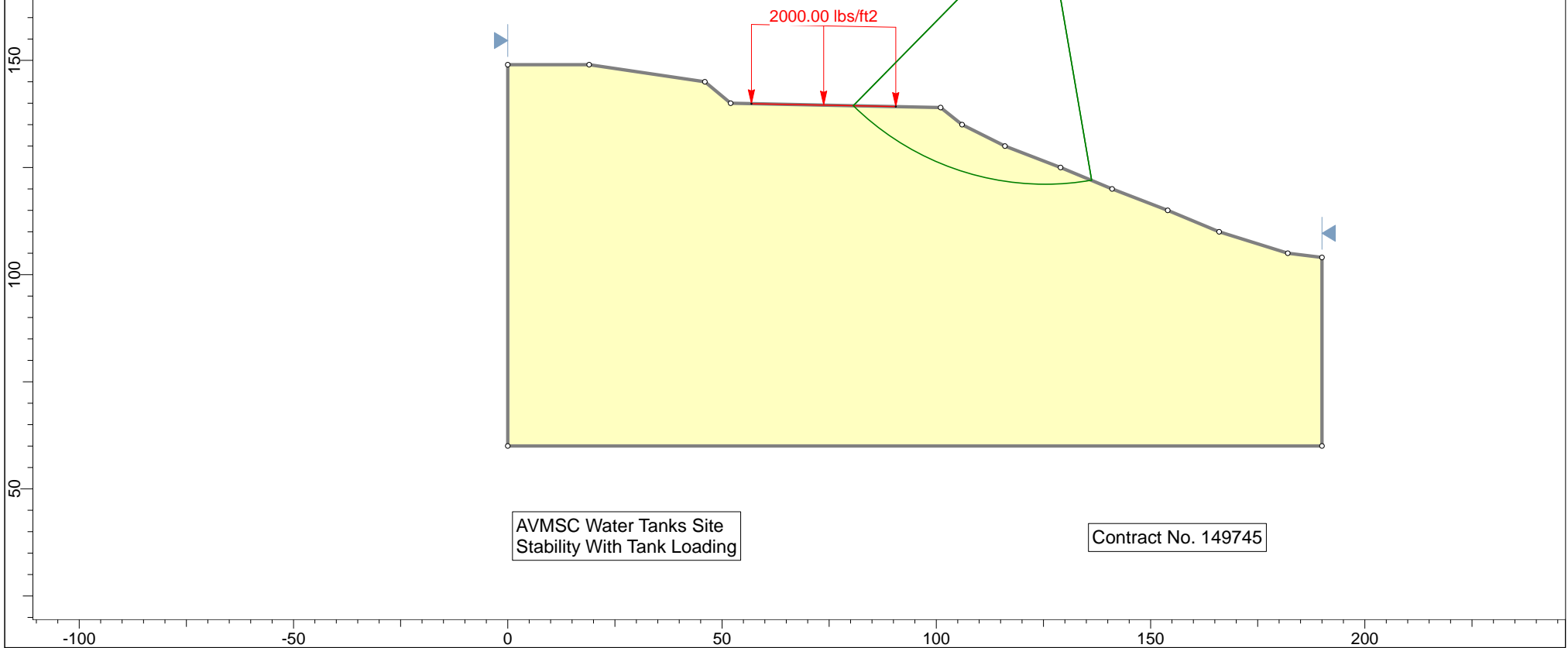
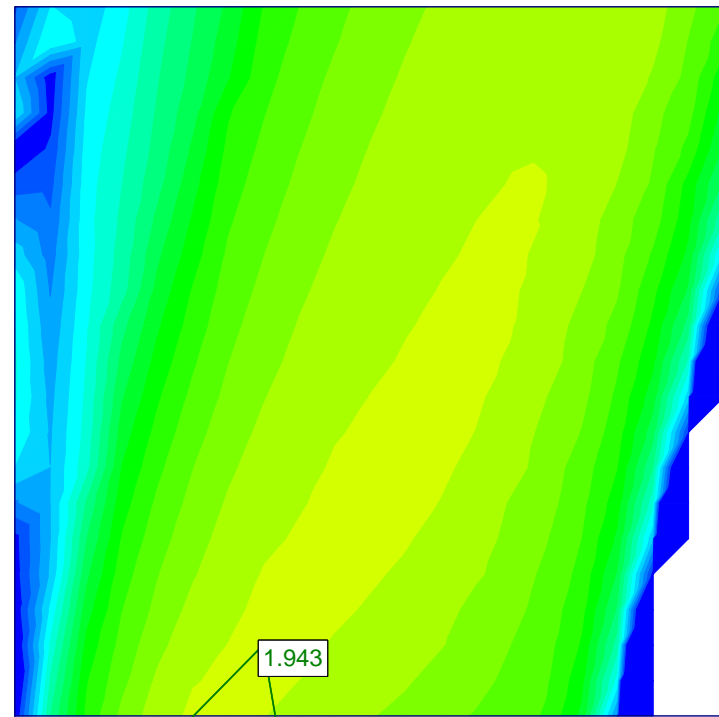


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Decomposed Granite		110	Mohr-Coulomb	127	34.5	None	0

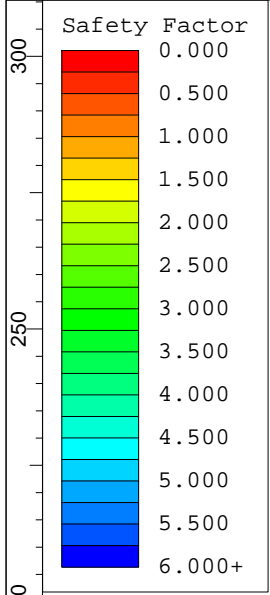




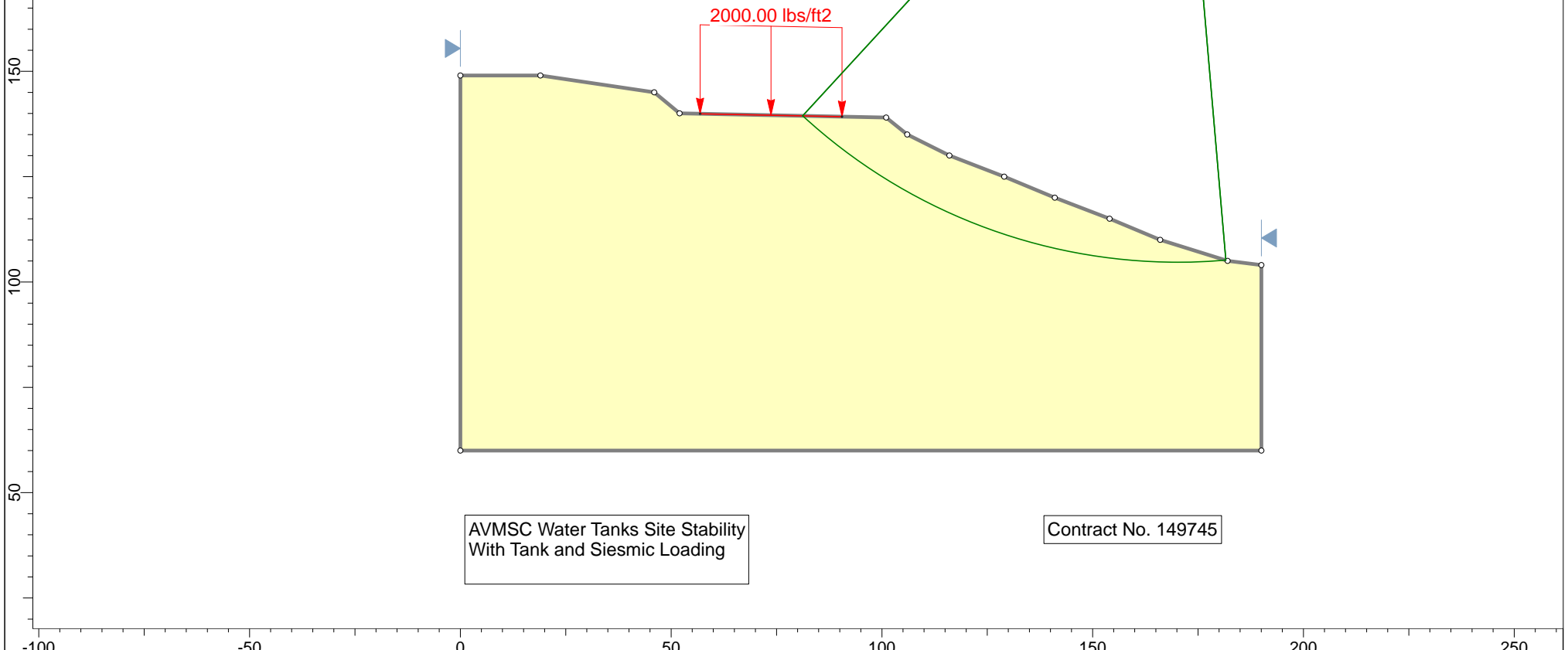
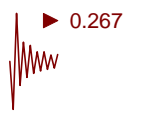
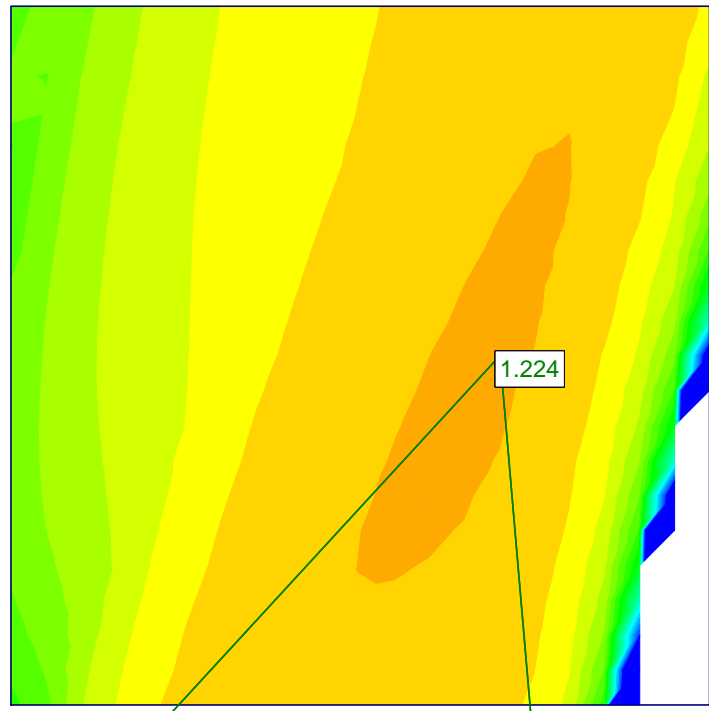
Material Name	Color	Unit Weight (lbs/ft <sup>3</sup> )	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Decomposed Granite		110	Mohr-Coulomb	127	34.5	None	0







Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Decomposed Granite		110	Mohr-Coulomb	127	34.5	None	0



## **Appendix D**

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Typical Earthwork Guidelines



## TYPICAL EARTHWORK GUIDELINES

### 1. GENERAL

These guidelines and the standard details attached hereto are presented as general procedures for earthwork construction for sites having slopes less than 10 feet high. They are to be utilized in conjunction with the project grading plans. These guidelines are considered a part of the geotechnical report, but are superseded by recommendations in the geotechnical report in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new recommendations which could supersede these specifications and/or the recommendations of the geotechnical report. It is the responsibility of the contractor to read and understand these guidelines as well as the geotechnical report and project grading plans.

- 1.1. The contractor shall not vary from these guidelines without prior recommendations by the geotechnical consultant and the approval of the client or the client's authorized representative. Recommendations by the geotechnical consultant and/or client shall not be considered to preclude requirements for approval by the jurisdictional agency prior to the execution of any changes.
- 1.2. The contractor shall perform the grading operations in accordance with these specifications, and shall be responsible for the quality of the finished product notwithstanding the fact that grading work will be observed and tested by the geotechnical consultant.
- 1.3. It is the responsibility of the grading contractor to notify the geotechnical consultant and the jurisdictional agencies, as needed, prior to the start of work at the site and at any time that grading resumes after interruption. Each step of the grading operations shall be observed and documented by the geotechnical consultant and, where needed, reviewed by the appropriate jurisdictional agency prior to proceeding with subsequent work.
- 1.4. If, during the grading operations, geotechnical conditions are encountered which were not anticipated or described in the geotechnical report, the geotechnical consultant shall be notified immediately and additional recommendations, if applicable, may be provided.
- 1.5. An as-graded report shall be prepared by the geotechnical consultant and signed by a registered engineer and registered engineering geologist. The report documents the geotechnical consultants' observations, and field and laboratory test results, and provides conclusions regarding whether or not earthwork construction was performed in accordance with the geotechnical recommendations and the grading plans. Recommendations for foundation design, pavement design, subgrade treatment, etc., may also be included in the as-graded report.
- 1.6. For the purpose of evaluating quantities of materials excavated during grading and/or locating the limits of excavations, a licensed land surveyor or civil engineer shall be retained.

## **2. SITE PREPARATION**

Site preparation shall be performed in accordance with the recommendations presented in the following sections.

- 2.1. The client, prior to any site preparation or grading, shall arrange and attend a pre-grading meeting between the grading contractor, the design engineer, the geotechnical consultant, and representatives of appropriate governing authorities, as well as any other involved parties. The parties shall be given two working days notice.
- 2.2. Clearing and grubbing shall consist of the substantial removal of vegetation, brush, grass, wood, stumps, trees, tree roots greater than 1/2-inch in diameter, and other deleterious materials from the areas to be graded. Clearing and grubbing shall extend to the outside of the proposed excavation and fill areas.
- 2.3. Demolition in the areas to be graded shall include removal of building structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, etc.), and other manmade surface and subsurface improvements, and the backfilling of mining shafts, tunnels and surface depressions. Demolition of utilities shall include capping or rerouting of pipelines at the project perimeter, and abandonment of wells in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of demolition.
- 2.4. The debris generated during clearing, grubbing and/or demolition operations shall be removed from areas to be graded and disposed of off site at a legal dump site. Clearing, grubbing, and demolition operations shall be performed under the observation of the geotechnical consultant.
- 2.5. The ground surface beneath proposed fill areas shall be stripped of loose or unsuitable soil. These soils may be used as compacted fill provided they are generally free of organic or other deleterious materials and evaluated for use by the geotechnical consultant. The resulting surface shall be evaluated by the geotechnical consultant prior to proceeding. The cleared, natural ground surface shall be scarified to a depth of approximately 8 inches, moisture conditioned, and compacted in accordance with the specifications presented in Section 5 of these guidelines.

## **3. REMOVALS AND EXCAVATIONS**

Removals and excavations shall be performed as recommended in the following sections.

- 3.1. Removals
  - 3.1.1. Materials which are considered unsuitable shall be excavated under the observation of the geotechnical consultant in accordance with the recommendations contained herein. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic, compressible natural soils, fractured, weathered, soft bedrock, and undocumented or otherwise deleterious fill materials.

- 3.1.2. Materials deemed by the geotechnical consultant to be unsatisfactory due to moisture conditions shall be excavated in accordance with the recommendations of the geotechnical consultant, watered or dried as needed, and mixed to generally uniform moisture content in accordance with the specifications presented in Section 5 of this document.

### 3.2. Excavations

- 3.2.1. Temporary excavations no deeper than 4 feet in firm fill or natural materials may be made with vertical side slopes. To satisfy California Occupational Safety and Health Administration (CAL OSHA) requirements, any excavation deeper than 4 feet shall be shored or laid back at a 1:1 inclination or flatter, depending on material type, if construction workers are to enter the excavation.

## 4. COMPACTED FILL

Fill shall be constructed as specified below or by other methods recommended by the geotechnical consultant. Unless otherwise specified, fill soils shall be compacted to 90 percent relative compaction, as evaluated in accordance with ASTM Test Method D 1557.

- 4.1. Prior to placement of compacted fill, the contractor shall request an evaluation of the exposed ground surface by the geotechnical consultant. Unless otherwise recommended, the exposed ground surface shall then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve a generally uniform moisture content at or near the optimum moisture content. The scarified materials shall then be compacted to 90 percent relative compaction. The evaluation of compaction by the geotechnical consultant shall not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.
- 4.2. Excavated on-site materials which are in general compliance with the recommendations of the geotechnical consultant may be utilized as compacted fill provided they are generally free of organic or other deleterious materials and do not contain rock fragments greater than 6 inches in dimension. During grading, the contractor may encounter soil types other than those analyzed during the preliminary geotechnical study. The geotechnical consultant shall be consulted to evaluate the suitability of any such soils for use as compacted fill.
- 4.3. Where imported materials are to be used on site, the geotechnical consultant shall be notified three working days in advance of importation in order that it may sample and test the materials from the proposed borrow sites. No imported materials shall be delivered for use on site without prior sampling, testing, and evaluation by the geotechnical consultant.
- 4.4. Soils imported for on-site use shall preferably have very low to low expansion potential (based on UBC Standard 18-2 test procedures). Lots on which expansive soils may be exposed at grade shall be undercut 3 feet or more and capped with very low to low

- expansion potential fill. In the event expansive soils are present near the ground surface, special design and construction considerations shall be utilized in general accordance with the recommendations of the geotechnical consultant.
- 4.5. Fill materials shall be moisture conditioned to near optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils shall be generally uniform in the soil mass.
  - 4.6. Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill shall be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.
  - 4.7. Compacted fill shall be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift shall be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to the specified relative compaction. Successive lifts shall be treated in a like manner until the desired finished grades are achieved.
  - 4.8. Fill shall be tested in the field by the geotechnical consultant for evaluation of general compliance with the recommended relative compaction and moisture conditions. Field density testing shall conform to ASTM D 1556-00 (Sand Cone method), D 2937-00 (Drive-Cylinder method), and/or D 2922-96 and D 3017-96 (Nuclear Gauge method). Generally, one test shall be provided for approximately every 2 vertical feet of fill placed, or for approximately every 1000 cubic yards of fill placed. In addition, on slope faces one or more tests shall be taken for approximately every 10,000 square feet of slope face and/or approximately every 10 vertical feet of slope height. Actual test intervals may vary as field conditions dictate. Fill found to be out of conformance with the grading recommendations shall be removed, moisture conditioned, and compacted or otherwise handled to accomplish general compliance with the grading recommendations.
  - 4.9. The contractor shall assist the geotechnical consultant by excavating suitable test pits for removal evaluation and/or for testing of compacted fill.
  - 4.10. At the request of the geotechnical consultant, the contractor shall "shut down" or restrict grading equipment from operating in the area being tested to provide adequate testing time and safety for the field technician.
  - 4.11. The geotechnical consultant shall maintain a map with the approximate locations of field density tests. Unless the client provides for surveying of the test locations, the locations shown by the geotechnical consultant will be estimated. The geotechnical consultant shall not be held responsible for the accuracy of the horizontal or vertical locations or elevations.
  - 4.12. Grading operations shall be performed under the observation of the geotechnical consultant. Testing and evaluation by the geotechnical consultant does not preclude the need for approval by or other requirements of the jurisdictional agencies.

- 4.13. Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When work is interrupted by heavy rains, the filling operation shall not be resumed until tests indicate that moisture content and density of the fill meet the project specifications. Regrading of the near-surface soil may be needed to achieve the specified moisture content and density.
- 4.14. Upon completion of grading and termination of observation by the geotechnical consultant, no further filling or excavating, including that planned for footings, foundations, retaining walls or other features, shall be performed without the involvement of the geotechnical consultant.
- 4.15. Fill placed in areas not previously viewed and evaluated by the geotechnical consultant may have to be removed and recompacted at the contractor's expense. The depth and extent of removal of the unobserved and undocumented fill will be decided based upon review of the field conditions by the geotechnical consultant.
- 4.16. Off-site fill shall be treated in the same manner as recommended in these specifications for on-site fills. Off-site fill subdrains temporarily terminated (up gradient) shall be surveyed for future locating and connection.

## **5. OVERSIZED MATERIAL**

Oversized material shall be placed in accordance with the following recommendations.

- 5.1. During the course of grading operations, rocks or similar irreducible materials greater than 6 inches in dimension (oversized material) may be generated. These materials shall not be placed within the compacted fill unless placed in general accordance with the recommendations of the geotechnical consultant.
- 5.2. Where oversized rock (greater than 6 inches in dimension) or similar irreducible material is generated during grading, it is recommended, where practical, to waste such material off site, or on site in areas designated as "nonstructural rock disposal areas." Rock designated for disposal areas shall be placed with sufficient sandy soil to generally fill voids. The disposal area shall be capped with a 5-foot thickness of fill which is generally free of oversized material.
- 5.3. Rocks 6 inches in dimension and smaller may be utilized within the compacted fill, provided they are placed in such a manner that nesting of rock is not permitted. Fill shall be placed and compacted over and around the rock. The amount of rock greater than 3/4-inch in dimension shall generally not exceed 40 percent of the total dry weight of the fill mass, unless the fill is specially designed and constructed as a "rock fill."
- 5.4. Rocks or similar irreducible materials greater than 6 inches but less than 4 feet in dimension generated during grading may be placed in windrows and capped with finer materials in accordance with the recommendations of the geotechnical consultant and the approval of the governing agencies. Selected native or imported granular soil (Sand Equivalent of 30 or higher) shall be placed and flooded over and around the windrowed rock such that voids are filled. Windrows of oversized materials shall be staggered so that successive windrows of oversized materials are not in the same vertical plane. Rocks



greater than 4 feet in dimension shall be broken down to 4 feet or smaller before placement, or they shall be disposed of off site.

## 6. SLOPES

The following sections provide recommendations for cut and fill slopes.

### 6.1. Cut Slopes

- 6.1.1. The geotechnical consultant shall observe cut slopes during excavation. The geotechnical consultant shall be notified by the contractor prior to beginning slope excavations.
- 6.1.2. If, during the course of grading, adverse or potentially adverse geotechnical conditions are encountered in the slope which were not anticipated in the preliminary evaluation report, the geotechnical consultant shall evaluate the conditions and provide appropriate recommendations.

### 6.2. Fill Slopes

- 6.2.1. When placing fill on slopes steeper than 5:1 (horizontal:vertical), topsoil, slope wash, colluvium, and other materials deemed unsuitable shall be removed. Near-horizontal keys and near-vertical benches shall be excavated into sound bedrock or fine fill material, in accordance with the recommendation of the geotechnical consultant. Keying and benching shall be accomplished. Compacted fill shall not be placed in an area subsequent to keying and benching until the area has been observed by the geotechnical consultant. Where the natural gradient of a slope is less than 5:1, benching is generally not recommended. However, fill shall not be placed on compressible or otherwise unsuitable materials left on the slope face.
- 6.2.2. Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a temporary slope, benching shall be conducted in the manner described in Section 7.2. A 3-foot or higher near-vertical bench shall be excavated into the documented fill prior to placement of additional fill.
- 6.2.3. Unless otherwise recommended by the geotechnical consultant and accepted by the Building Official, permanent fill slopes shall not be steeper than 2:1 (horizontal:vertical). The height of a fill slope shall be evaluated by the geotechnical consultant.
- 6.2.4. Unless specifically recommended otherwise, compacted fill slopes shall be overbuilt and cut back to grade, exposing firm compacted fill. The actual amount of overbuilding may vary as field conditions dictate. If the desired results are not achieved, the existing slopes shall be overexcavated and reconstructed in accordance with the recommendations of the geotechnical consultant. The degree of overbuilding may be increased until the desired compacted slope face condition is achieved. Care shall be taken by the contractor to provide

mechanical compaction as close to the outer edge of the overbuilt slope surface as practical.

- 6.2.5. If access restrictions, property line location, or other constraints limit overbuilding and cutting back of the slope face, an alternative method for compaction of the slope face may be attempted by conventional construction procedures including backrolling at intervals of 4 feet or less in vertical slope height, or as dictated by the capability of the available equipment, whichever is less. Fill slopes shall be backrolled utilizing a conventional sheepsfoot-type roller. Care shall be taken to maintain the specified moisture conditions and/or reestablish the same, as needed, prior to backrolling.
  - 6.2.6. The placement, moisture conditioning and compaction of fill slope materials shall be done in accordance with the recommendations presented in Section 5 of these guidelines.
  - 6.2.7. The contractor shall be ultimately responsible for placing and compacting the soil out to the slope face to obtain a relative compaction of 90 percent as evaluated by ASTM D 1557 and a moisture content in accordance with Section 5. The geotechnical consultant shall perform field moisture and density tests at intervals of one test for approximately every 10,000 square feet of slope.
  - 6.2.8. Backdrains shall be provided in fill as recommended by the geotechnical consultant.
- 6.3. Top-of-Slope Drainage
- 6.3.1. For pad areas above slopes, positive drainage shall be established away from the top of slope. This may be accomplished utilizing a berm and pad gradient of 2 percent or steeper at the top-of-slope areas. Site runoff shall not be permitted to flow over the tops of slopes.
  - 6.3.2. Gunitelined brow ditches shall be placed at the top of cut slopes to redirect surface runoff away from the slope face where drainage devices are not otherwise provided.
- 6.4. Slope Maintenance
- 6.4.1. In order to enhance surficial slope stability, slope planting shall be accomplished at the completion of grading. Slope plants shall consist of deep-rooting, variable root depth, drought-tolerant vegetation. Native vegetation is generally desirable. Plants native to semiarid and mid areas may also be appropriate. Large-leafed ice plant should not be used on slopes. A landscape architect shall be consulted regarding the actual types of plants and planting configuration to be used.
  - 6.4.2. Irrigation pipes shall be anchored to slope faces and not placed in trenches excavated into slope faces. Slope irrigation shall be maintained at a level just sufficient to support plant growth. Property owners shall be made aware that over watering of slopes is detrimental to slope stability. Slopes shall be

monitored regularly and broken sprinkler heads and/or pipes shall be repaired immediately.

6.4.3. Periodic observation of landscaped slope areas shall be planned and appropriate measures taken to enhance growth of landscape plants.

6.4.4. Graded swales at the top of slopes and terrace drains shall be installed and the property owners notified that the drains shall be periodically checked so that they may be kept clear. Damage to drainage improvements shall be repaired immediately. To reduce siltation, terrace drains shall be constructed at a gradient of 3 percent or steeper, in accordance with the recommendations of the project civil engineer.

6.4.5. If slope failures occur, the geotechnical consultant shall be contacted immediately for field review of site conditions and development of recommendations for evaluation and repair.

## **7. TRENCH BACKFILL**

The following sections provide recommendations for backfilling of trenches.

- 7.1. Trench backfill shall consist of granular soils (bedding) extending from the trench bottom to 1 foot or more above the pipe. On-site or imported fill which has been evaluated by the geotechnical consultant may be used above the granular backfill. The cover soils directly in contact with the pipe shall be classified as having a very low expansion potential, in accordance with UBC Standard 18-2, and shall contain no rocks or chunks of hard soil larger than 3/4-inch in diameter.
- 7.2. Trench backfill shall, unless otherwise recommended, be compacted by mechanical means to 90 percent relative compaction as evaluated by ASTM D 1557. Backfill soils shall be placed in loose lifts 8-inches thick or thinner, moisture conditioned, and compacted in accordance with the recommendations of Section 5 of these guidelines. The backfill shall be tested by the geotechnical consultant at vertical intervals of approximately 2 feet of backfill placed and at spacings along the trench of approximately 100 feet in the same lift.
- 7.3. Jetting of trench backfill materials is generally not a recommended method of densification, unless the on-site soils are sufficiently free-draining and provisions have been made for adequate dissipation of the water utilized in the jetting process.
- 7.4. If it is decided that jetting may be utilized, granular material with a sand equivalent greater than 30 shall be used for backfilling in the areas to be jetted. Jetting shall generally be considered for trenches 2 feet or narrower in width and 4 feet or shallower in depth. Following jetting operations, trench backfill shall be mechanically compacted to the specified compaction to finish grade.
- 7.5. Trench backfill which underlies the zone of influence of foundations shall be mechanically compacted to 90 percent or greater relative compaction, as evaluated by ASTM D 1557-02. The zone of influence of the foundations is generally defined as the

- roughly triangular area within the limits of a 1:1 (horizontal:vertical) projection from the inner and outer edges of the foundation, projected down and out from both edges.
- 7.6. Trench backfill within slab areas shall be compacted by mechanical means to a relative compaction of 90 percent, as evaluated by ASTM D 1557. For minor interior trenches, density testing may be omitted or spot testing may be performed, as deemed appropriate by the geotechnical consultant.
  - 7.7. When compacting soil in close proximity to utilities, care shall be taken by the grading contractor so that mechanical methods used to compact the soils do not damage the utilities. If the utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, then the grading contractor may elect to use light mechanical compaction equipment or, with the approval of the geotechnical consultant, cover the conduit with clean granular material. These granular materials shall be jetted in place to the top of the conduit in accordance with the recommendations of Section 8.4 prior to initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review by the geotechnical consultant and the utility contractor, at the time of construction.
  - 7.8. Clean granular backfill and/or bedding materials are not recommended for use in slope areas unless provisions are made for a drainage system to mitigate the potential for buildup of seepage forces or piping of backfill materials.
  - 7.9. The contractor shall exercise the specified safety precautions, in accordance with OSHA Trench Safety Regulations, while conducting trenching operations. Such precautions include shoring or laying back trench excavations at 1:1 or flatter, depending on material type, for trenches in excess of 5 feet in depth. The geotechnical consultant is not responsible for the safety of trench operations or stability of the trenches.

## **8. DRAINAGE**

The following sections provide recommendations pertaining to site drainage.

- 8.1. Roof, pad, and slope drainage shall be such that it is away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.).
- 8.2. Positive drainage adjacent to structures shall be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside the building perimeter, further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.
- 8.3. Surface drainage on the site shall be provided so that water is not permitted to pond. A gradient of 2 percent or steeper shall be maintained over the pad area and drainage patterns shall be established to remove water from the site to an appropriate outlet.

- 8.4. Care shall be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of finish grading shall be maintained for the life of the project. Property owners shall be made very clearly aware that altering drainage patterns may be detrimental to slope stability and foundation performance.

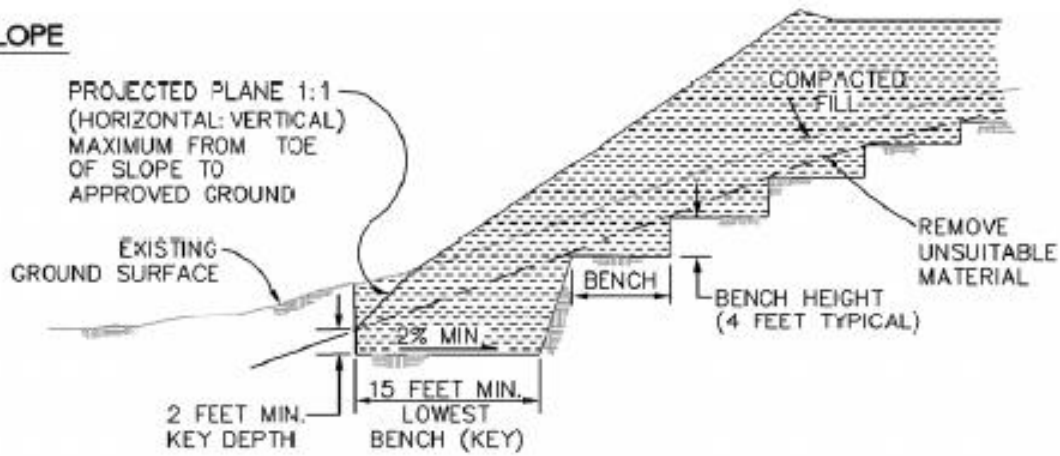
## **9. SITE PROTECTION**

The site shall be protected as outlined in the following sections.

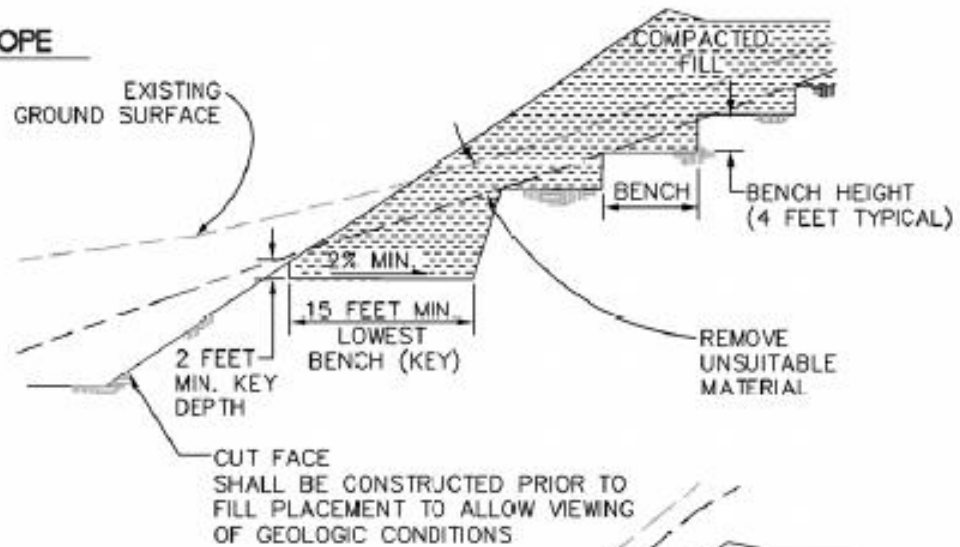
- 9.1. Protection of the site during the period of grading shall be the responsibility of the contractor unless other provisions are made in writing and agreed upon among the concerned parties. Completion of a portion of the project shall not be considered to preclude that portion or adjacent areas from the need for site protection, until such time as the project is finished as agreed upon by the geotechnical consultant, the client, and the regulatory agency.
- 9.2. The contractor is responsible for the stability of temporary excavations. Recommendations by the geotechnical consultant pertaining to temporary excavations are made in consideration of stability of the finished project and, therefore, shall not be considered to preclude the responsibilities of the contractor. Recommendations by the geotechnical consultant shall also not be considered to preclude more restrictive requirements by the applicable regulatory agencies.
- 9.3. Precautions shall be taken during the performance of site clearing, excavation, and grading to protect the site from flooding, ponding, or inundation by surface runoff. Temporary provisions shall be made during the rainy season so that surface runoff is away from and off the working site. Where low areas cannot be avoided, pumps shall be provided to remove water as needed during periods of rainfall.
- 9.4. During periods of rainfall, plastic sheeting shall be used as needed to reduce the potential for unprotected slopes to become saturated. Where needed, the contractor shall install check dams, desilting basins, riprap, sandbags or other appropriate devices or methods to reduce erosion and provide recommended conditions during inclement weather.
- 9.5. During periods of rainfall, the geotechnical consultant shall be kept informed by the contractor of the nature of remedial or precautionary work being performed on site (e.g., pumping, placement of sandbags or plastic sheeting, other labor, dozing, etc.).
- 9.6. Following periods of rainfall, the contractor shall contact the geotechnical consultant and arrange a walk-over of the site in order to visually assess rain-related damage. The geotechnical consultant may also recommend excavation and testing in order to aid in the evaluation. At the request of the geotechnical consultant, the contractor shall make excavations in order to aid in evaluation of the extent of rain-related damage.
- 9.7. Rain or irrigation related damage shall be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress, and other adverse conditions noted by the geotechnical consultant. Soil adversely affected shall be classified as

- "Unsuitable Material" and shall be subject to overexcavation and replacement with compacted fill or to other remedial grading as recommended by the geotechnical consultant.
- 9.8. Relatively level areas where saturated soils and/or erosion gullies exist to depths greater than 1 foot shall be overexcavated to competent materials as evaluated by the geotechnical consultant. Where adverse conditions extend to less than 1 foot in depth, saturated and/or eroded materials may be processed in-place. Overexcavated or in-place processed materials shall be moisture conditioned and compacted in accordance with the recommendations provided in Section 5. If the desired results are not achieved, the affected materials shall be overexcavated, moisture conditioned, and compacted until the specifications are met.
  - 9.9. Slope areas where saturated soil and/or erosion gullies exist to depths greater than 1 foot shall be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where adversely affected materials exist to depths of 1 foot or less below proposed finished grade, remedial grading by moisture conditioning in-place and compaction in accordance with the appropriate specifications may be attempted. If the desired results are not achieved, the affected materials shall be overexcavated, moisture conditioned, and compacted until the specifications are met. As conditions dictate, other slope repair procedures may also be recommended by the geotechnical consultant.
  - 9.10. During construction, the contractor shall grade the site to provide positive drainage away from structures and to keep water from ponding adjacent to structures. Water shall not be allowed to damage adjacent properties. Positive drainage shall be maintained by the contractor until permanent drainage and erosion reducing devices are installed in accordance with project plans.

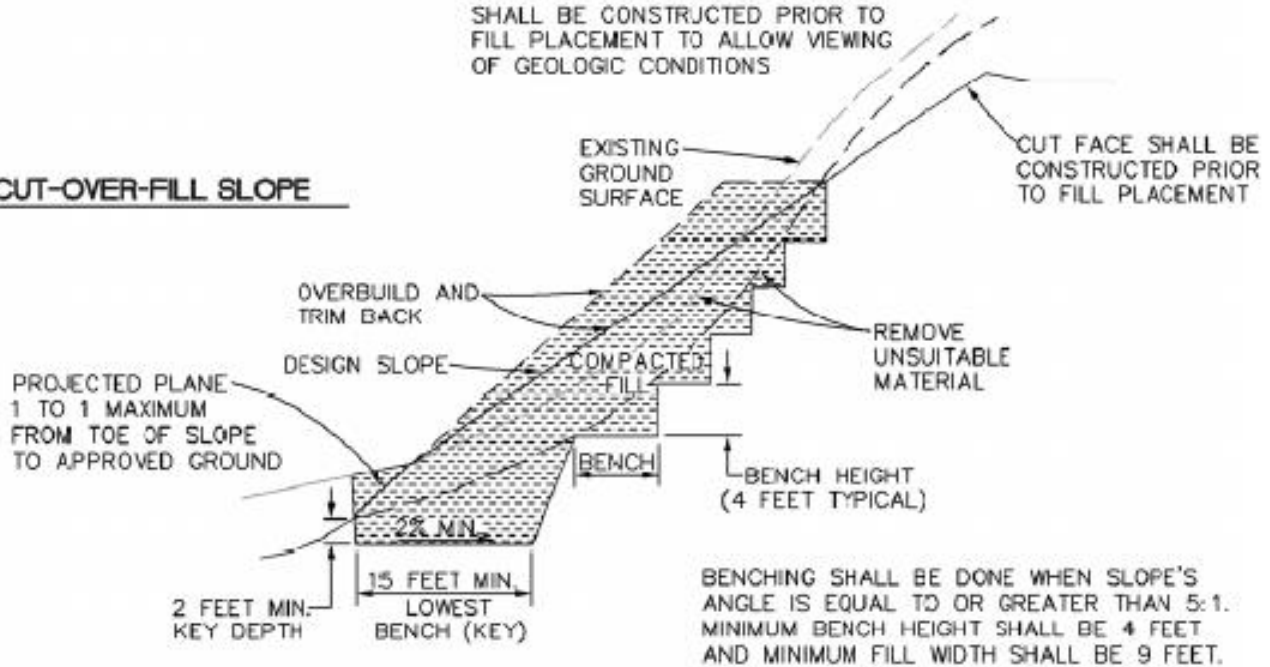
**FILL SLOPE**



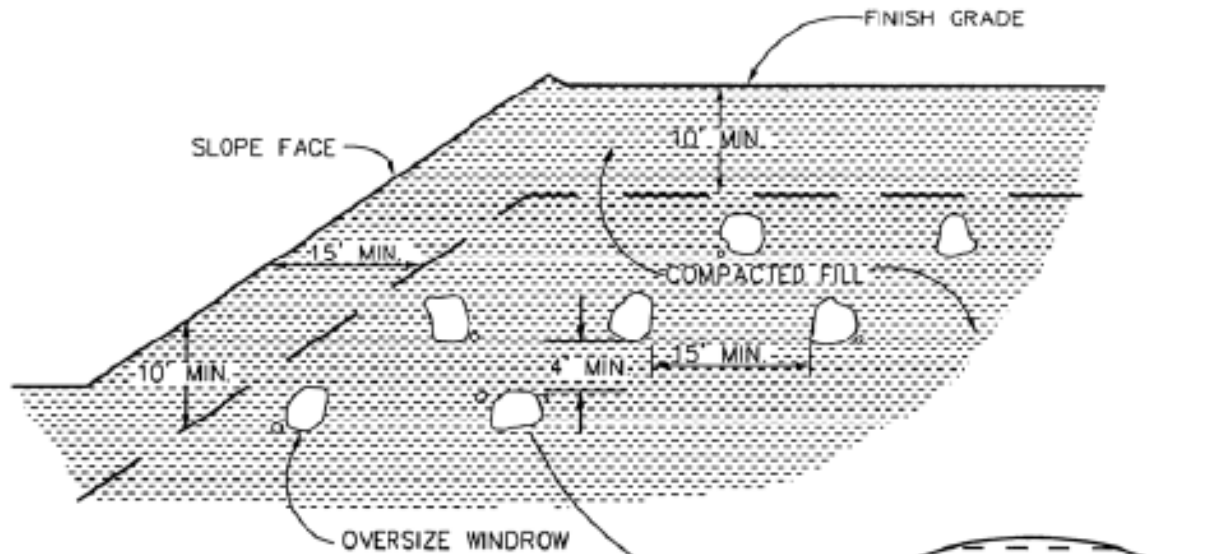
**FILL-OVER-CUT SLOPE**



**CUT-OVER-FILL SLOPE**

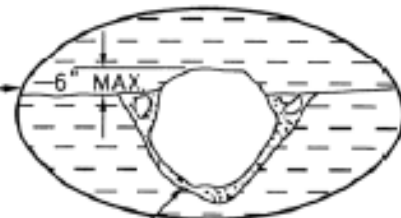




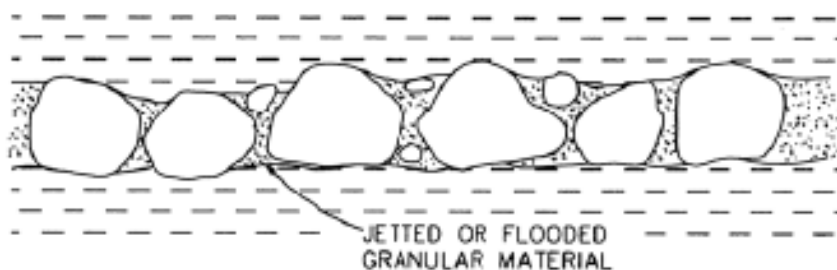


- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE Voids.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

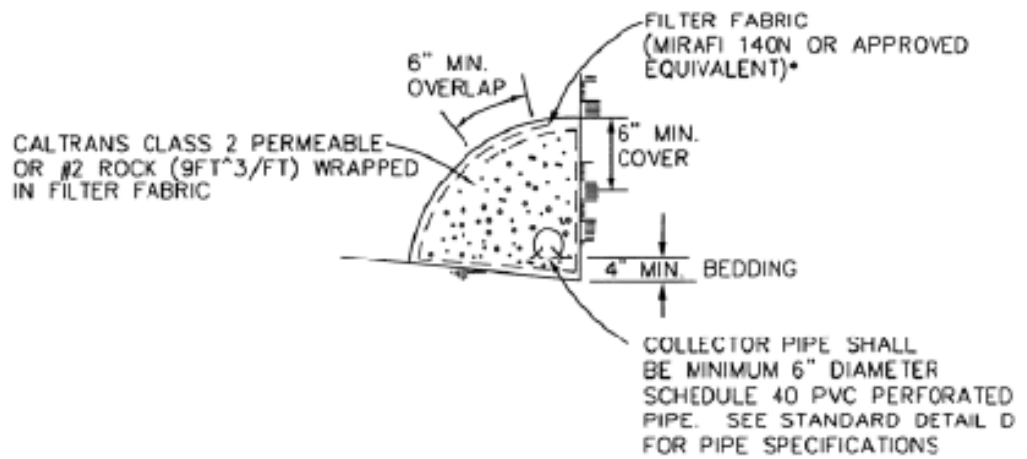
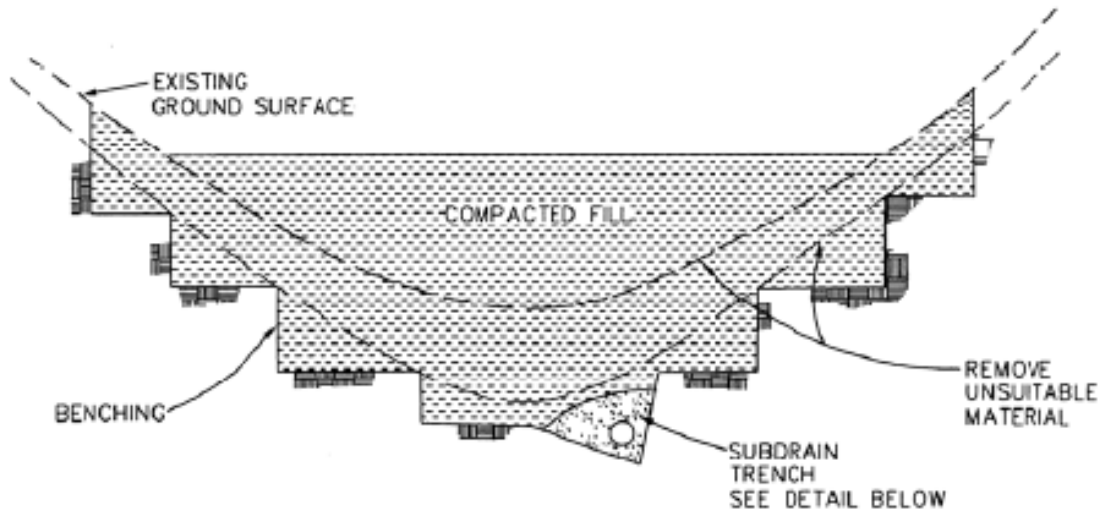


DETAIL

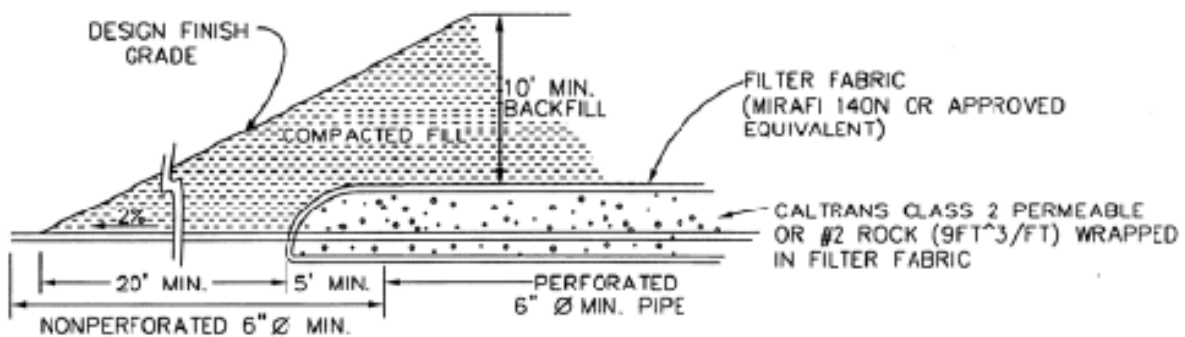


TYPICAL PROFILE ALONG WINDROW

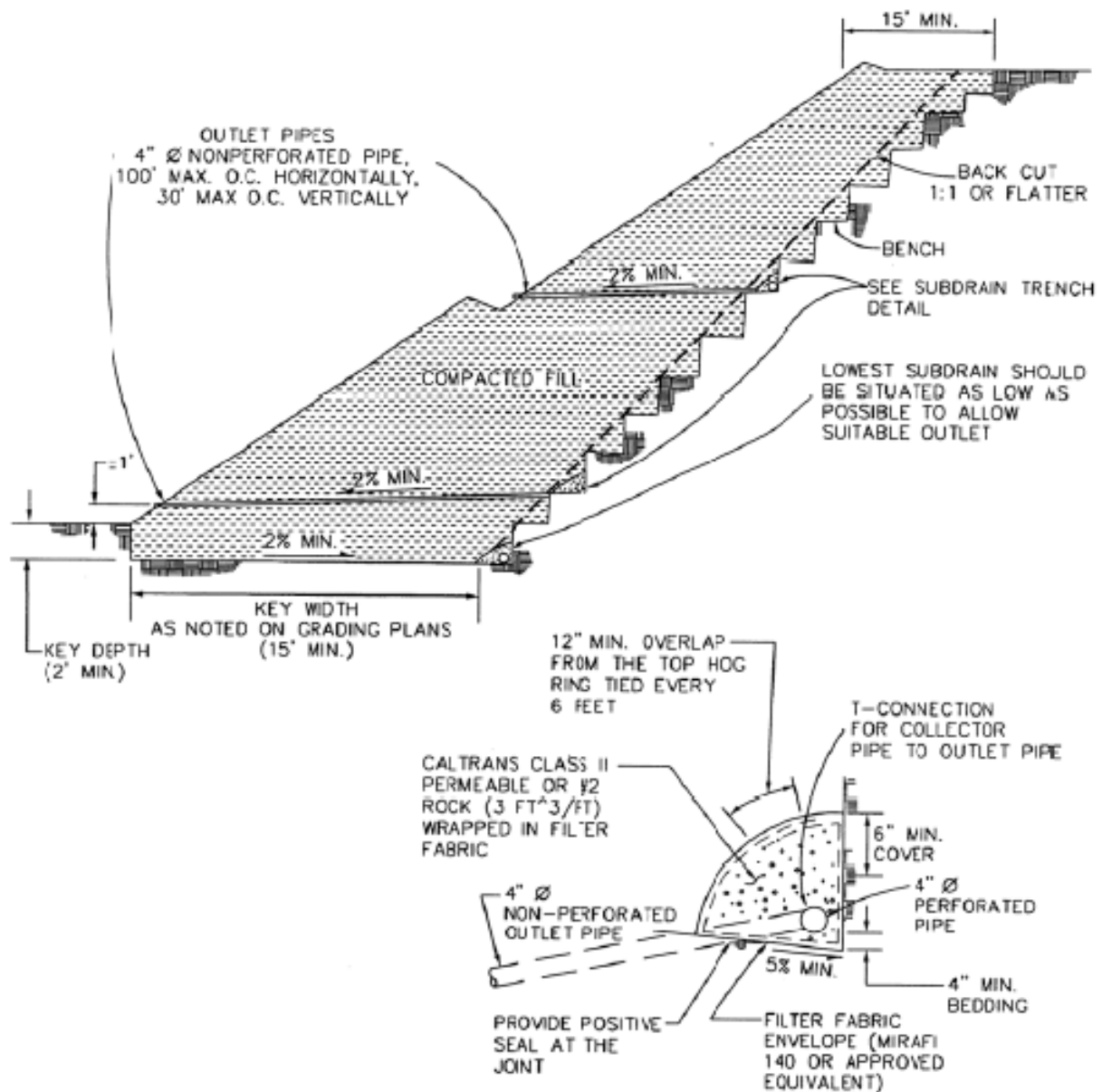




**SUBDRAIN DETAIL**



**DETAIL OF CANYON SUBDRAIN OUTLET**



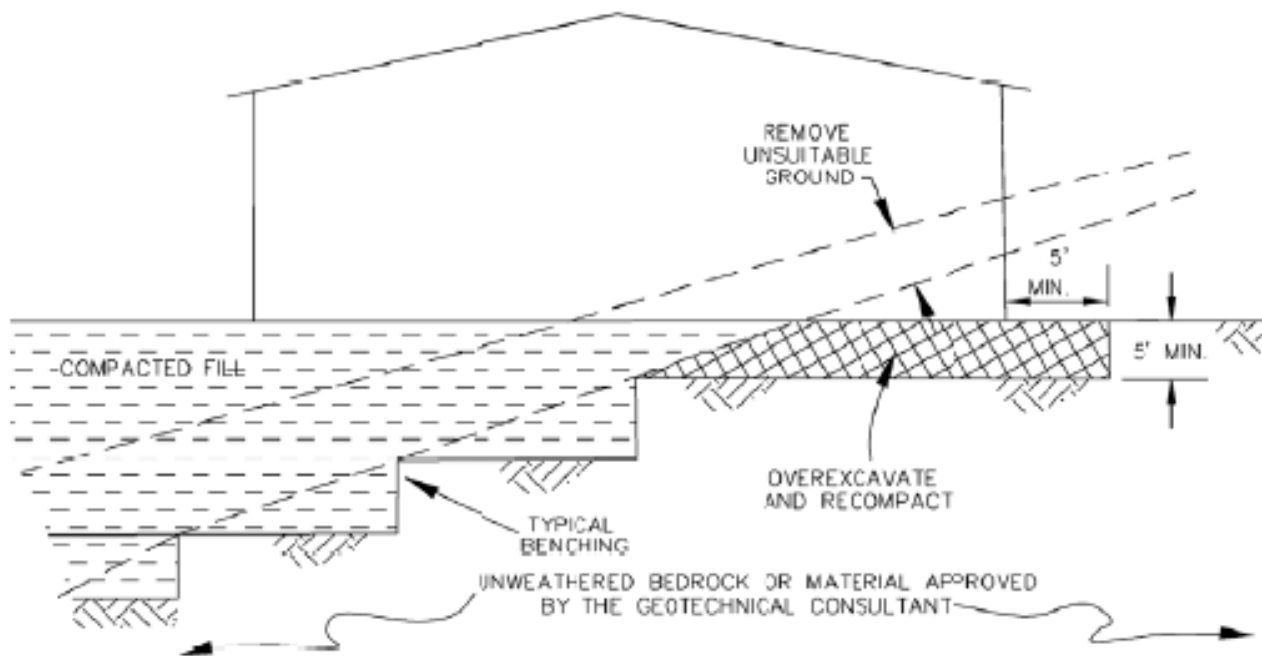
**SUBDRAIN TRENCH DETAIL**

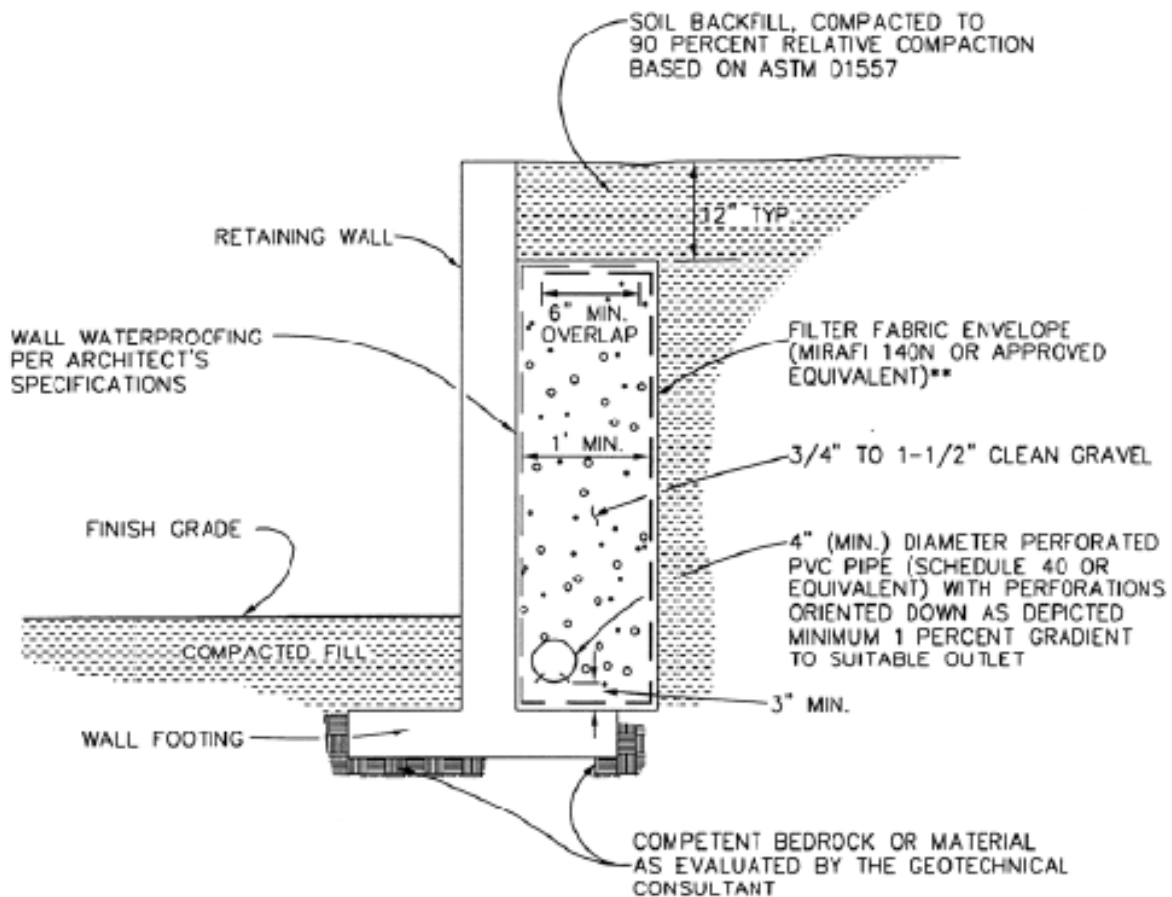
**SUBDRAIN INSTALLATION** – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

**SUBDRAIN PIPE** – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

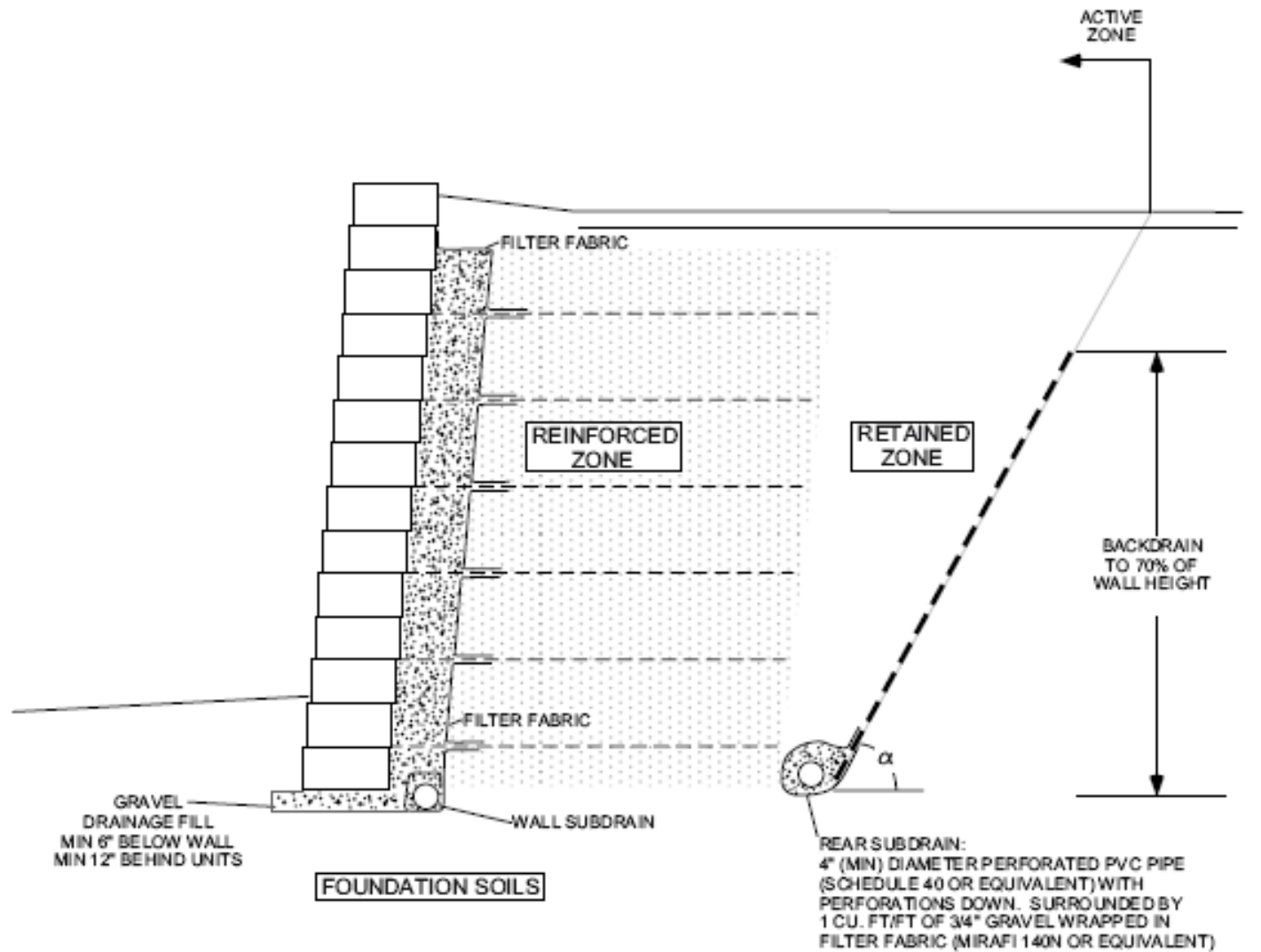
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

### CUT-FILL TRANSITION LOT OVEREXCAVATION





NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.



OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER, BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

**NOTES:**

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20  
 FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10  
 FOR TIERED WALLS, USE COMBINED WALL HEIGHTS  
 WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

- CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.
- GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.
- GEOGRID TO BE PRETENSIONED DURING INSTALLATION.
- IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE  $\alpha = 45 + \phi/2$ , WHERE  $\phi$  IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.
- BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

## **Appendix E**

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ASFE Important Information about Your Geotechnical Engineering Report





# Important Information About Your Geotechnical Engineering Report

*Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.*

*The following information is provided to help you manage your risks.*

## **Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects**

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the report for any purpose or project except the one originally contemplated.*

## **Read the Full Report**

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

## **A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors**

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

## **Subsurface Conditions Can Change**

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

## **Most Geotechnical Findings Are Professional Opinions**

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

## **A Report's Recommendations Are *Not* Final**

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual



subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

## **A Geotechnical Engineering Report Is Subject to Misinterpretation**

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

## **Do Not Redraw the Engineer's Logs**

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

## **Give Contractors a Complete Report and Guidance**

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time to perform additional study.* Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

## **Read Responsibility Provisions Closely**

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that

have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

## **Geoenvironmental Concerns Are Not Covered**

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

## **Obtain Professional Assistance To Deal with Mold**

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

## **Rely, on Your ASFE-Member Geotechnical Engineer for Additional Assistance**

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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