



January 12, 2026

Project No. 19011-12

To: Irvine Company
550 Newport Center Drive
Newport Beach, California 92658-6370

Attention: Mr. Ryan Rasmussen

Subject: Geotechnical Report for Orange Heights Tracts and Associated Improvements, City of Orange, California

In accordance with your authorization, NMG Geotechnical, Inc. (NMG) has prepared this geotechnical report for the Orange Heights Project (previously referred to as the Santiago Hills II Project) ("Project"), which includes the Project site areas north and south of Santiago Canyon Road ("SCR"; the North and South Tracts, respectively),¹ and associated site-adjacent off-site roadway and infrastructure improvements. This report covers the entire Orange Heights Project site that is bounded by Jamboree Road on the west, Irvine Regional Park on the north, State Route (SR) 241/261 on the east, and undeveloped land within the City of Orange to the south.

The Project plans reviewed for this study and/or used for exhibits in this report include:

- "Rough Grading Plan, Santiago Hills II – South, Tract 16199, Delta 1" by Wilson Mikami Corporation (WMC),
- "Rough Grading Plan, Santiago Hills II – North, Tract 17995, Delta 2" by Proactive Engineering Consultants (Proactive),
- "Construction Plans for the Improvement of Jamboree Road from Station 205+92.61 to Station 291+57.21 (Total Length 8,564.60') and Santiago Canyon Road from Stations 76+31.11 to Station 126+58.64 (Total Length 5,027.53')," prepared by Michael Baker International (MBI), and
- "Construction of Santiago Hills Storm Drain Outfall Structure" and the "Line I Storm Drain Plans," prepared by MBI.

NMG has prepared prior reports covering geotechnical review of these plan sets (Appendix A). These prior reports included boring/CPT/trench logs, laboratory data, slope stability analysis, infiltration testing data, etc.

¹ Approved Tracts 16199, 17987, 17988, 17989, 17990 and 17991 are located to the south of SCR, referred to as the South Tract (Plate 1). Tract 17995 is located to the north of SCR and is referred to as the North Tract (Plate 2).

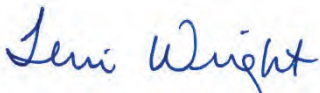
This report presents a summary of the geotechnical conditions and hazards at the Project site as identified in previous geotechnical studies prepared for the Project. This report also includes the updated 2025 seismic design criteria and additional geophysical survey data, some of which was updated in 2024, and confirms the geotechnical recommendations for design and rough grading, updated as necessary based on the updated design criteria and additional data.

We conclude that the proposed rough grading and future development is feasible from a geotechnical standpoint provided the prior geotechnical recommendations, which are included and updated in this report, are implemented during design, grading, and construction.

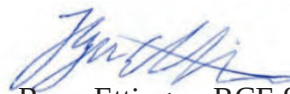
If you have any questions regarding this report, please contact our office. We appreciate the opportunity to provide our services.

Respectfully submitted,

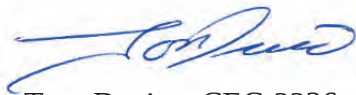
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1.0 INTRODUCTION

1.1 Purpose and Scope

NMG Geotechnical, Inc. (NMG) has prepared this summary geotechnical report for the Orange Heights Project and associated site-adjacent off-site roadway and infrastructure improvements, in the City of Orange, California. The purpose of this report is to present the geotechnical conditions and hazards for the Project, along with the recommendations for design, grading and construction. This report also includes a summary of the applicable mitigation measures for the Project identified in the Santiago Hills II and East Orange Planned Communities Final Supplemental Environmental Impact Report 1278/Environmental Impact Report 1716 certified by the City of Orange in 2005 (2005 SEIR), and the 2016 Addendum to the 2005 SEIR. Collectively, these documents are referred to herein as the "Previous Environmental Documentation."

The scope of services for this geotechnical review included the following tasks.

- **Review of Available Reports:** Both published reports and prior geotechnical investigation reports were reviewed and compiled to provide a summary of the existing conditions; geotechnical constraints; and design, grading and construction recommendations for the Project. A list of these referenced reports is provided in Appendix A.
- **Preparation of Exhibits:** A site location map was prepared to show the Project boundary (Figure 1). The California Geological Survey maps are included as figures, including:
 - 1) Seismic Hazards Map showing state mapped areas of potential liquefaction and earthquake induced landslide (Figure 2),
 - 2) California Fault Activity Map showing the faults and their age/activity (Figure 3),
 - 3) Fault Rupture Hazard Zones Map showing regionally active faults requiring special studies in relation to the site location (Figure 4).

The geologic mapping was compiled onto two figures for the South Tract (Figure 5) and the North Tract (Figure 6).

- **Geotechnical Analysis:** Geologic analysis included compilation and review of the existing data and plans/reports. Two maps are included at 150-scale to present the South Tract (Plate 1) and the North Tract (Plate 2) conditions. These plates show the prior boring/CPT/trench locations, geotechnical conditions and the recommended remedial grading measures. Updated seismic analysis was also performed based on the 2025 California Building Code (Appendix B). The geophysical data is also presented in Appendix C.
- **Report Preparation:** This geotechnical report was prepared with illustrations to support the environmental review of the Project pursuant to the California Environmental Quality Act (CEQA).

1.2 Site Location and Existing Conditions

The Project site consists of approximately 421 acres in the eastern portion of the City of Orange. The Project site is bounded by Jamboree Road on the west, Irvine Regional Park on the north, SR-241/261 on the east, and undeveloped area in the City of Orange to the south (Figure 1).

The high point at the Project site is at the northeast edge of the North Tract at 855 feet above mean sea level (msl). The low point is in the southwestern canyon area in the South Tract just east of Jamboree Road, at 550 feet msl. There is over 300 feet of relief at the site. The majority of the site is in its natural condition. The Project site supports native habitat communities (i.e., riparian, coastal sage scrub, chaparral, and grassland), and non-native habitat communities (i.e., annual grassland and pepper trees).

The roadways bounding or crossing the site include Santiago Canyon Road (SCR), Jamboree Road, Handy Creek Road, State Routes 241/261 and the old Peters Canyon Road. There are existing utilities along these roadways. In addition, the Baker Water Pipeline (Baker Pipeline), owned and operated by IRWD, crosses the western portions of both the North and South Tracts, from Irvine Regional Park to Jamboree Road. There are also two existing water tanks in the IRWD parcel just south of SCR, with an above ground High-Density Polyethylene (HDPE) pipeline that crosses the South Tract from the water reservoirs extending to Jamboree Road near Canyon View Avenue. There are also numerous storm drain culvert outlets that drain onto both tracts from SR241/261 and there are several culverts below SCR that outlet onto the South Tract. There are several water lines east of Jamboree Road, near the planned Peters Canyon Outlet Structure.

1.3 Prior Geotechnical Studies and Site History

Onsite Studies: Geotechnical investigations were undertaken at the Project site in 1988 by John Sayers and in 2000 by GeoSoils, Inc. In 2001, NMG performed a geotechnical investigation at the Zone 6 tank site in the northeast portion of the North Tract. Between 2005 and 2007, Lawson and Associates (LGC) performed geotechnical investigations covering the South Tract and the Peters Canyon Outlet. Later between 2014 and 2017, LGC Geotechnical performed additional investigations to supplement the prior work in both the North and South Tracts and associated improvements. The LGC reports included the data from the prior work.

Adjacent Studies: Leighton and Associates performed observation and testing for the Eastern Transportation Corridor and the realignment of the eastern portion of Santiago Canyon Road in 2001 and 2002. In 2015, Kleinfelder performed an investigation for the planned IRWD reclaimed water tank planned at the northern edge of the South Tract. URS Corporation performed geotechnical investigations in Jamboree Road and SCR in 2016 for IRWD, and NMG performed geotechnical observation and testing during installation of the IRWD reclaimed water lines within these roads in 2018. A complete listing of the references is included in Appendix A.

Prior Grading: There are significant existing fills and buttress slopes around the perimeter and adjacent to the roadways at the Project site. The fill embankments related to grading of SR-241/261 and SCR are up to 120 feet high and were placed under the observation and testing of Leighton and Associates (2001 and 2002). According to the final compaction reports, remedial grading was performed to expose competent soils prior to placement of fill and fill was compacted to a minimum of 90 percent relative compaction to the slope face.

Jamboree Road was graded earlier than SR-261, based on the as-built grading plan prepared by Hunsaker and Associates for Irvine Company dated 1987. The plans show recommended slope

stabilization keys along the east side of Jamboree Road and a deep fill under the southern tip of the South Tract. NMG was not able to obtain the reports for the grading of Jamboree Road. There is an abandoned access road that was graded with Jamboree Road at the southern end of the South Tract which is the prior access road to Cox Communications facility and the East Orange County Water District (EOCWD) water reservoir to the east of SR-261. This access road has since been abandoned and replaced by Handy Creek Road during grading for SR-261.

The historic topographic maps of the Project site show a prior roadway crossing northeast/southwest through the middle of the site. The road was named Peters Canyon Road and extended from Irvine Regional Park to Peters Canyon reservoir. Most of the roadway is no longer visible in the South Tract. There are areas of undocumented fill and the road is paved in the North Tract.

Between 1970 and 1980, the eastern portion of the North Tract, east of the old Peters Canyon Road, was part of Saddleback Motocross Park. There are some concrete remnants and undocumented fills in this area. Irvine Regional Park was in place in the early 1900s, and Peters Canyon Reservoir is first seen on the 1931 aerial photos.

1.4 Proposed Development and Grading

The proposed Project consists of refinements to the previously approved Santiago Hills II Project, evaluated in the Previous Environmental Documentation. The proposed Project would refine previously approved tract maps which would result in an overall reduction in the approved dwelling units (a reduction from 1,180 to 1,058 units), and would include parks, landscaping, trails, streets, utilities, fencing and walls, signage, lighting, and other infrastructure.

The Project site encompasses approximately 421 acres and consists of seven tracts, bisected by SCR, which is located onsite. Tracts 16199, 17987, 17988, 17989, 17990, and 17991 are located to the south of SCR, referred to as the South Tract (Plate 1). Tract 17995 is located to the north of SCR and is referred to as the North Tract (Plate 2). The site-adjacent off-site roadway and infrastructure improvement areas encompass approximately 21 acres.

Consistent with the previously approved Santiago Hills II Project, now referred to as the Orange Heights Project, the proposed grading for the Project will include cuts of up to 101 feet thick and fills of up to 96 feet deep. Cut and fill slopes are designed at 2H:1V or flatter and up to 95 and 125 feet high, respectively.

Following is a description of the anticipated grading and infrastructure construction, which is consistent with the prior reports and plans.

Baker Pipeline Relocation: The Baker pipeline, a 54-inch water transmission line that feeds south Orange County by way of the Baker Treatment Plant, will be realigned with this Project. This existing pipeline crosses the western portion of the Project in the proposed cut areas.

South Tract: The proposed grading on the embankment below SCR would be completed once the power poles are removed, after installation of the underground 12KV power and telecommunication lines in SCR. Also, there are several storm-drain lines that outlet onto the Project site from under SCR and SR-261. The rough grading operations would allow drainage through the Project site until the future storm drain in SCR is installed to provide an outlet for these lines. Grading will include cuts of up to 101 feet deep in the southern portion of the site and fills of up to 60 feet in the central canyon. Cut and fill slopes are designed at 2H:1V or flatter and up to 85 and 110 feet high, respectively.

North Tract: Grading of Street D and the water reservoir (Zone 6 tank) site and access road to the tank in the northeast portion of the North Tract would be completed to allow construction of the tank by IRWD. Grading will include cuts of up to 97 feet deep in the central-northern portion of the site and fills of up to 96 feet in the fill area just east of the deepest cut. Cut and fill slopes are designed at 2H:1V or flatter and up to 95 and 125 feet high, respectively.

Peters Canyon Outlet: Storm drain Line "I" will extend from the 108-inch culvert under SR-261 in the South Tract, down Street A toward the west, where it will cross Jamboree Road and outlet into Peters Canyon Reservoir. The storm drain will be jack-and-bored across Jamboree Road to outlet into the reservoir. The outlet structure will be constructed at the edge of the reservoir, and the Peters Canyon Regional Park trail will be graded over the storm drain to the north and south.

Santiago Canyon Road: An approximately 0.9-mile segment of this roadway between Jamboree Road and SR 241/261 will be re-graded and the road will be widened. The 12kV line will be installed underground in SCR during the tract grading along the roadway so that the existing poles may be removed. Later roadway grading will include cuts and fills on the order of 4 feet or less.

Jamboree Road: Improvements to this roadway will mostly include reconstruction of the pavement and restriping to add a third through lane in both the north and south directions, reconstruction of the median, and installation of left turn lanes and local right turn lanes. Curb and gutter and bus stops will also be added.

2.0 GEOLOGIC SITE CONDITIONS

The following is a compilation of geologic information previously provided in the referenced geotechnical reports and is generally consistent with the prior environmental documentation. Additional Figures 2 through 4 are included in this report to present the seismic and faulting conditions based on mapping by the California Geological Survey. Figures 5 and 6 were also prepared to illustrate the geologic conditions of the North and South Tracts. The geology on these maps is consistent with the Geotechnical Maps (Plates 1 and 2) in this report and included in our prior reports.

2.1 Regional Geologic Setting

The Project site lies in the foothills of the western Santa Ana Mountains, which is in the northern portion of the Peninsular Range Geomorphic Province of California. This northwest trending mountain range is underlain by both sedimentary and volcanic bedrock that has been uplifted, folded and faulted throughout. The geologic conditions at the Project site are complex and consist of bedrock units, including the Eocene- to late Miocene-age Santiago, Vaqueros/Sespe, Topanga, and Puente Formations that were tilted and faulted. There is also a significant amount of the Miocene-age El Modena Volcanics in the northwest portion of the site. Surficial units at the site include large areas of Pleistocene-age terrace and older alluvial deposits in the north and eastern portions of the site, Holocene-age alluvium is mapped in the canyons with local landslide deposits. There are also significant amounts of both undocumented fills and compacted fills onsite. Figures 5 and 6 illustrate the geologic conditions at the site.

The faulting at the site was found in the bedrock units and is considered seismically inactive. Neither the onsite studies nor prior published mapping has encountered evidence of active faults at the site. The closest regionally active faults zoned by the state are the Whittier Fault located approximately 6.6 miles to the northeast and the Newport Inglewood Fault 16 miles to the southwest (Figure 4). The Peralta Hills Buried Thrust Fault is located 4 miles northwest of the site and San Joaquin Hills Blind Thrust Fault is located approximately 10 miles southeast of the site. An earthquake event on these or other regionally active faults could produce strong seismic ground shaking at the site, as with most sites in southern California.

2.2 Earth Units

Figures 5 and 6 show the general geologic conditions at the site. Plates 1 and 2 show the more detailed geology with boring/trench/CPT locations and geologic attitudes. The earth units shown on these maps are anticipated to be exposed during grading and include the following:

Artificial Fill (Map Symbol – Af_L): Fill placed under the observation and testing of LGC (2019) within the canyon in the south-central portion of the North Tract included soils derived from the construction of the reclaimed water reservoir facility across SCR (Figure 6 and Plate 2). The fill reportedly consists of clayey sand, silty sandy clay and silty clay that was compacted to a minimum of 93 percent relative compaction. The maximum fill thickness is on the order of 50 feet thick. The fill is moderately weathered near surface with animal burrows and small erosion rills. Currently, a temporary storm drain system is in place to drain runoff from the top of the fill to near the toe of

the fill slope. A subdrain system was also installed during this grading that is to be extended and outlet through a culvert under SCR during the proposed rough grading operations.

Older Fill (Map Symbol - Afo): The roadways that border the east side of the site and the eastern end of SCR are underlain by compacted fills placed during the grading of SR 241/261 and SCR (Leighton, 2001 and 2002). There are 2H:1V graded fill embankments up to 120 feet high along these roads (Plates 1 and 2). In general, the adjacent fill slopes have experienced some degree of weathering, and local erosion gullies, but appear generally undisturbed and well-vegetated. There are multiple storm-drain outlet structures and subdrains that drain onto the Project site. The fills associated with the grading of Handy Creek Road were observed and documented by Leighton and Associates with the SR 261 grading (2002). These fills generally consist of silty sands with some clayey sands and appear derived from local native deposits. The fills were reportedly compacted to a minimum of 90 percent relative compaction.

Fills placed during the grading and construction of Jamboree Road are also considered older fill (Plate 1). A grading plan for the improvement of Jamboree Road was recovered by NMG. However, NMG was not able to locate the as-graded geotechnical report of observation and testing for the road. The area designated as Lot K, the southernmost portion of the South Tract (Figure 5 and Plate 1), is underlain by fill that appears to have been placed during the grading of Jamboree Road. Based on Borings LGC-20 and LGC-21, these fills were found to be silty and clayey sand to sandy silt that was dense and compacted. The fill is deep with no native geologic units encountered in the upper 60 feet.

In 2024, NMG installed a groundwater observation well (OW-1) along the east side of Jamboree Road, north of Handy Creek Road and found up to 15 feet of the older fill overlying alluvium (Plate 1). This fill consisted of dark brown silty clay and brown clayey fine sand that was moist and medium stiff.

Undocumented Fill (Map Symbol - Afu): In the northwest area of the South Tract there is a stockpile of undocumented fills up to 15 feet high, to the southeast of the intersection of SCR and Jamboree Road (Plate 1). Portions of the existing Baker Pipeline located in the South Tract are reportedly buried by 5 to 15 feet of undocumented backfill overlying alluvium or El Modena Volcanics. Minor undocumented fills appear to have been placed within the canyon in the central portion of the South Tract, which are believed to be associated with some water quality improvements (i.e., berms/check dams) to collect surface waters east of Jamboree Road.

In the North Tract, undocumented fills are primarily associated with old roads that were graded and later abandoned (Plate 2). This includes the original alignment of old SCR and Peters Canyon Road. These fills are reportedly up to 35 feet thick along old SCR. There are areas of undocumented fill from the prior Saddleback Motocross Park east of old Peters Canyon Road. These fills were generally derived from onsite sources and are considered unsuitable to remain in place. The unsuitable fill will be removed prior to placement of compacted fill.

Portions of the Baker Pipeline located in the North Tract are reportedly buried by 5 feet of undocumented backfill overlying terrace deposits or El Modena Volcanics. These undocumented

fills are considered unsuitable and are planned to be completely removed as part of the recommended remedial grading for the Project site.

Colluvium (Map Symbol - Col): Colluvium is found locally on the middle and lower portions of hillsides. The colluvium generally varies from 3 to 15 feet thick and may contain some organic matter. The colluvium is typically grayish brown to dark brown sandy silt and clay with scattered bedrock fragments. Some areas that were investigated by others have colluvium with reported thickness of up to 20 feet thick (locally).

The mapped colluvium locally overlies what was mapped by others as older alluvium in the eastern portion of the South Tract (Figure 5 and Plate 1). The fill slope descending from SR 261 covers the older alluvium where it was deemed generally competent by Leighton (2001 and 2002) during the SR 261 grading. This older alluvium deposit underlies the colluvium locally. It generally consists of clayey sand to sandy clay with gravel that is stiff to dense.

Alluvium (Map Symbol - Qal): The alluvium is generally located within swales and low-lying areas at the site and was deposited by streams that descend from the higher upgradient elevations. The alluvium is anticipated to be exposed during grading within the swales and along the project boundaries where fill has already been placed for the existing roads. Generally, composition of the alluvium ranges from silty sand, sandy silt to silty clay. Deep root systems and other organic matter may exist where mature trees and vegetation occur, typically within major canyon bottoms.

There is an area in the central portion of the South Tract underlying Street A that has deep alluvium up to 60 feet thick (Plate 1). This alluvium generally consists of sandy silt and clayey silt, with a 20- to 30-foot-thick layer of soft clayey sand and clay at depth. This area was previously recommended to be mitigated by wick drains; however, 50-foot-deep removal is now planned to remove the low-density clayey materials. This soft clay material is believed to be ancient pond deposits that accumulated near the bottom of the alluvium and appears to pinch out as it extends up canyon. Below 50 feet, the alluvium becomes lean, stiff clay and sandy clay that is denser and may remain in-place. There is a similar low density clayey material noted below Street A in the southern portion of this roadway. These alluvial materials and the underlying weathered bedrock are planned to be removed and replaced with compacted fill since the groundwater table is deeper in this area.

The proposed Peters Canyon Outlet will be underlain by alluvium along the reservoir edge. The alluvium underlying the outfall structure consists of clayey silt and silty clay.

Older Alluvium (Map Symbol - Qalo): Older alluvium is mapped below the existing compacted fills below both SR-241 and SCR. This material underlies the eastern portion of the South Tract (Plate 1) where it is generally dense and similar to the terrace deposits. Remedial removals of the overlying colluvium and the design cuts in this area are anticipated to expose the older alluvium at the removal bottom.

Landslide Material (Map Symbol - Qls): There are few landslides mapped within the site. They vary from surficial landslides to translational block failures (Figures 5 and 6 and Plates 1 and 2). The slide materials consist of the nearby bedrock and surficial deposits, that are generally more

fractured and weathered. The shallow slides are anticipated to contain more disturbed materials that are unsuitable. The materials of the larger block slides are expected to be denser and less fractured.

Terrace Deposit (Map Symbol - Qt): River or non-marine terrace deposits are mapped in the northern portions of the site, on both the North and South Tracts (Figures 5 and 6 and Plates 1 and 2). The deposits are variable in soil type but generally range from reddish-brown to brown silty/clayey sand with layers of gravel and cobbles, with lesser amounts of clean sands. The terrace deposits are locally visible along the tops of steep canyon walls where colluvium has not yet formed. This material is generally dense and competent, however, much of the material will be removed and recompacted with the design cuts and remedial grading.

El Modena Volcanics (Map Symbol - Tem): The El Modena Volcanics mapped at this site consist of both extrusive and intrusive volcanic rock. The unit is mapped generally within the northwestern portion of the site in both the North and South Tracts (Figures 5 and 6 and Plates 1 and 2). The bedrock consists of pyroclastic and flow rock and intrusive andesite and diabase that form through dikes and sills. The El Modena Volcanics were observed to have highly variable rippability properties but have generally high shear wave velocities. Excavator pits using a Caterpillar 330 with a 48-inch bucket were able to be extended to depths of up to 24 feet in these materials, and borings drilled into this material were able to extend to depths of 40 to 50 feet. Based on the seismic refraction lines (SL-6 and SL-7), there are areas of rock with over 7,000 fps in the cuts deeper than 50 feet in the area of Street A.

Puente Formation, La Vida Member (Map Symbol - Tplv): The Puente Formation, La Vida Member, is a marine sedimentary deposit generally consisting of laminated diatomaceous siltstone, clayey siltstone, and claystone, with minor sandstone interbeds (Figures 5 and 6 and Plates 1 and 2). The formation is known to have occasional bentonitic plastic clays (on the order of 1 to 2 inches thick). LGC reported that this bedrock unit was observed having local "baked" zones, where the intrusive volcanic rocks heated and altered the sedimentary bedrock making it harder and possibly more difficult to excavate locally.

Topanga Formation (Map Symbol - Tt): The Topanga Formation is composed of tan to light gray fine- to medium-grained sandstone with lesser interbedded sandy siltstone that is moderately lithified. This unit may have localized "baked" zones where intrusive volcanic rocks altered them. This unit is mostly present along Jamboree Road within the South Tract (Figure 5 and Plate 1).

Vaqueros and Sespe Formation, Undifferentiated (Map Symbol - Tvs): The Vaqueros and Sespe Formations are known to interfinger and come in and around Orange County. For this reason, prior geotechnical consultants and geologists have mapped them undifferentiated as one assemblage (Figures 5 and 6 and Plates 1 and 2). The Vaqueros and Sespe Formations represent a depositional environment change from marine to non-marine, respectively. The Vaqueros is typically identified as yellow to light brown sandstone and olive gray to light gray siltstone with some shale and occasional conglomerate beds. Marine fossils are common in this formation as well. The Sespe Formation is described as a reddish to olive brown sandstone, with interbeds of silty claystone and clayey siltstones and occasional conglomerate beds. This formation often

includes red claystone beds ("red beds") that are characteristically highly expansive and possibly subject to heave in deep cuts.

Santiago Formation (Map Symbol - Tsa): The Santiago Formation typically consists of yellow to light brown sandstone, siltstone, and cobble conglomerate. The depositional environment of this unit ranges from marine to non-marine. This unit is mapped in the northern corner of the site where the Zone 6 IRWD water reservoir is planned (Figure 6 and Plate 2). In this area, the bedrock is predominantly composed of sandstone.

2.3 Groundwater Conditions

South Tract: Groundwater is anticipated to be encountered during grading in the central portion of the South Tract (Plate 1). There is an area of deep removals recommended below Street A that will extend below the observed groundwater table (Sections 4.3 and 4.6) and dewatering may be necessary. Borings completed by Lawson in 2006 indicated groundwater at 25 feet deep in this area. During a later study by LGC (2016c), the groundwater was encountered at a depth of 40 feet. Groundwater observation wells were reportedly installed by both consultants; however, NMG was not able to locate them during field mapping. As with any groundwater condition, seasonal and annual fluctuations are expected. Fractures within bedrock or contacts with bedrock may also present perched groundwater and/or seepage during grading.

NMG (2024b) completed one groundwater observation well near the southern intersection of Jamboree Road and planned Street A (Plate 1). This well was installed with a hollow-stem-auger drilling rig to a depth of 41.5 feet. Groundwater was first encountered at a depth of 16 feet during drilling on March 4, 2024. After the well was purged and developed, the groundwater level was measured at 10.5 feet on March 21, 2024. Since that high level in the spring of 2024, the groundwater table went down approximately 9 feet by October 2024 to 19.5 feet below ground surface. The latest reading indicates groundwater levels are at 23.1 feet below the ground surface on October 28, 2025. This indicates there is at least 13 feet of seasonal fluctuations.

North Tract: Groundwater is anticipated to be mostly present at depth within some of the swales, in the alluvium above the bedrock contact, locally above the terrace/bedrock contact, above landslide rupture surfaces, or along faults. Seepage was found locally in 10 borings throughout the North Tract drilled during different months/years at depths of 17 to 87 feet below existing ground surface. The shallowest groundwater was encountered in GSB-41 that is located offsite to the northeast in Irvine Regional Park (Plate 2). The seepage encountered in the bedrock and at the terrace bedrock contact was relatively minor. Standing groundwater collected in the bottom of six of these borings at depths of 32 to 89 feet that was mostly generated from the seepage above. The groundwater table is believed to be deep below the site and is not anticipated to be encountered during grading. In general, groundwater is not expected to have an influence on grading operations in the North Tract. Seasonal outflow from existing storm drains along SR-241 may impact the surface and groundwater levels in the eastern portion of the site.

Peters Canyon Outlet Structure: Prior to installation of the storm drain and outlet structure a portion of the existing sewer line needs to be removed and replaced with a steel pipe and casing. One new manhole needs to be constructed as well. The excavation for the sewer will be 20 feet

deep below existing topography and the lower 12 to 13 feet of the excavation will be below the groundwater table and dewatering will be necessary in this area.

2.4 Geologic Structure

The overall geologic structure underlying the Project site can be described as a complex system of faulted sedimentary blocks that have been subjected to regional tectonic forces, which have folded and tilted the layered bedrock units. The mapped faults are generally north- to northwest-trending at the site (Figures 5 and 6 and Plates 1 and 2). In the South Tract, the sedimentary bedrock generally dips between 20 and 60 degrees to the east but is highly variable locally (Plate 1). In the North Tract, the sedimentary bedrock generally dips between 20 and 60 degrees to the south but is locally highly variable (Plate 2). Mapping by the state geologists and US Geological Survey present the geologic conditions at the site differently. The mapping presented in this report is based on LGC's prior site specific investigations and mapping.

Volcanic dikes and sills have intruded into the sedimentary units and formed isolated, irregularly shaped outcrops within the site (Figures 5 and 6 and Plates 1 and 2). Faulting has also brought the El Modena pyroclastic and flow rocks to the surface. Subsequent erosion has dissected the layered sedimentary blocks leaving partially isolated bedrock units at the surface. Older alluvium and terrace deposits have also been locally deposited over the bedrock further masking the underlying bedrock units. Erosion of the uplifted hillsides has deposited alluvium within canyons, which generally flow south to southwest toward the Peters Canyon drainage and reservoir.

3.0 GEOTECHNICAL HAZARDS

The following summarizes the geotechnical hazards within the site based on updated source documents and previous investigations, including investigations conducted for the previously approved Santiago Hills II project. As summarized below, the geotechnical hazards are generally consistent with those presented in the Previous Environmental Documentation. Additional Figures 2 through 4 are included in this report to present the seismic and faulting conditions based on mapping by the California Geological Survey.

3.1 Faulting, Seismicity and Seismic Hazards

Faulting: There are no active faults known to exist at the site, nor are there any fault rupture hazard zones mapped by the state (Figures 3 and 4, CGS, 2018). There are numerous inactive faults mapped at the site that are believed to be old and confined to the bedrock units. The state has mapped one pre-Holocene fault crossing the site (Figure 3) that is considered seismically inactive.

Utilizing the USGS disaggregation program (USGS, 2025) and the site coordinates of 33.7878 degrees north latitude and -117.7551 degrees longitude, the closest major fault to the site is the Peralta Hills Fault approximately 4 miles (6.48km) northwest of the site. However, the Whittier Fault located approximately 6.6 miles (10.65km) north of the site has the largest disaggregation contribution to the site. The seismic design parameters presented in the recommendations section of this report are based on the 2025 California Building Code (CBC) and were obtained for the site utilizing computer programs USGS Earthquake Hazard Toolbox (USGS, 2025) and the Structural Engineers Association/Office of Statewide Health Planning Development U.S Seismic Design Maps (SEA/OSHPD, 2025). The maximum moment magnitude for the Controlling Fault is 7.28 M_w , which would be generated from the Whittier Fault.

Seismicity: Properties in southern California are subject to seismic hazards of varying degrees depending upon the proximity, degree of activity, and capability of nearby faults. These hazards can be primary (i.e., directly related to the energy release of an earthquake, such as surface rupture and ground shaking) or secondary (i.e., related to the effect of earthquake energy on the physical world which can cause phenomena, such as liquefaction and ground lurching).

The primary seismic hazard for this site is ground shaking due to a future earthquake on one of the major regional active faults, such as the Whittier-Elsinore, Chino, Newport-Inglewood, San Andreas, San Jacinto and the Peralta Hills Blind Thrust and San Joquin Hills Blind Thrust Faults.

The seismic design parameters were obtained for the site utilizing the USGS Earthquake Hazard Toolbox (USGS, 2025) and Structural Engineers Association/Office of Statewide Health Planning Development U.S Seismic Design Maps (SEA/OSHPD, 2025), based on the 2025 California Building Code (CBC). The Site Classification used to develop the seismic design parameters is the default site condition in accordance with ASCE 7-22, section 11.4.2.1. It is based on the most critical spectral acceleration at each period of Site Class C, CD, and D. Seismic data is provided in Appendix B and design parameters are presented in Section 4.14.

Secondary Seismic Hazards: Local areas of the site are mapped by the state to be within seismic hazard zones for potential liquefaction or seismically induced landslides (Figure 2 and CGS, 2025). These areas have been investigated and remedial grading is recommended to address these zones. See Sections 3.2 and 3.3 below for further discussions. The site is located away from the coastline at a high elevation and therefore, the potential for a tsunami is considered nil. There is a proposed IRWD Zone 6 water reservoir located at the northeast corner of the northern tract. The proposed reservoir will be located away from the development and there is a canyon between the proposed reservoir and the development, therefore the potential for the residential development to be impacted by seiche (seismically induced wave) is also considered very low.

3.2 Liquefaction and Seismic Settlement

Liquefaction is a phenomenon in which earthquake-induced cyclic stresses generate excess pore water pressure in low density (loose), saturated, sandy soils and soft silts below the water table. This causes a loss of shear strength and, in many cases, ground settlement. For liquefaction to occur, the following four conditions must be present:

- Severe ground shaking, such as those that occur during a strong earthquake,
- Saturated or nearly saturated soil materials, generally below the water table,
- Relatively low corrected normalized standard penetration test (SPT) blow counts ($N_{1,60}$) or the CPT tip resistance (Q), and,
- Granular soil material (usually sands or silts) with low plasticity. Clayey soils and silts of relatively high plasticity are generally not subject to liquefaction.

State mapping indicates that the two main east-west trending canyons in the South Tract (Figure 5 and Plate 1) are within zones of earthquake-induced potential liquefaction. The main canyon south of the north-central hillside is subject to 50-foot remedial removals with deep fills planned to replace the alluvium. The majority of the other main canyon to the south is not a part of the grading and does not have residential units planned within it. Another area also exists in the southern main canyon where 10- to 40-foot removals are recommended (Plate 1). The majority of the alluvium in the North Tract will be removed prior to placement of fill.

Based on the prior LGC study for the South Tract (2016c), the majority of the alluvial soils are considered to be cohesive and not considered to be susceptible to liquefaction based on the saturated moisture content compared to their Liquid Limit. In addition, after the grading the groundwater table in the alluvial areas will be on the order of 55 to 80 feet deep below the roadways and residential areas. In general, based on the proposed design grades, depth of groundwater, the cohesive nature of the alluvial soils left-in-place and the dense nature of the proposed compacted fill, the potential for post-construction liquefaction and liquefaction-induced settlement is considered low to very low.

The Peters Canyon outfall structure is not located in an area of earthquake-induced liquefaction potential as mapped by the state, but close to the mapped zone (Figure 2). The alluvium underlying the outfall structure may be classified as clayey silt and silty clay. Additionally, the recommended remedial removals are designed to remove between 5 to 10 feet of alluvium, leaving between 5 and

15 feet of alluvium underlying the structure. In general, based on the proposed design grades, the cohesive nature of the alluvial soils left-in-place and the dense nature of the proposed compacted fill, the potential for post-construction liquefaction and liquefaction induced settlement is considered low.

3.3 Mass Movements/Slope Stability

Mass Movements: In the South Tract, there is one large ancient landslide mapped in the central portion of the site (Figure 5 and Plate 1). This landslide is a translational block glide that failed in the Vaqueros Sespe Formation bedrock. Prior borings in this slide indicate the majority of the failure is in competent sandstone and siltstone, and the proposed grading will bury the toe of the landslide providing a gravity buttress. There are three other shallow failures mapped in the upper reach of the canyon in the southern portion of the site, just north of Handy Creek Road. These features will be removed during grading.

In the North Tract, there are four mapped landslides. There is a landslide mapped at the base of the hill below the future Zone 6 IRWD water reservoir that is believed to be bedrock translational block failure (Figure 6 and Plate 2). This slide is mostly downhill of the fill slope, which will be provided with a shear key at the toe of slope for stabilization. There is a small landslide mapped along SR-241, where a small portion of this competent slide material was left under the fill embankment. During grading, the remainder of this slide will be covered with compacted fill after removal of unsuitable materials. There is a landslide mapped within the central canyon that was partially removed or filled over during the geotechnical observation and testing of LGC (2019); however, this slide was not mentioned in their final compaction report, and their geotechnical map indicates that removals were made to suitable bedrock of undifferentiated Vaqueros/Sespe Formation. There is also a landslide mapped in the southwest portion of the site above SCR. Based on prior studies, this landslide is believed to toe out above the roadway. During grading, the entire landslide will be removed.

Slope Stability: Slope stability analysis was performed for the project (NMG, 2024c and 2025d) where needed for the landslides and proposed graded slopes. Per the Orange County grading code and prevalent local geotechnical practice, a minimum factors-of-safety of 1.5 (static) was applied in order to determine the necessary remedial measures. The pseudostatic seismic analysis was performed in accordance with the County of Orange Grading Manual and CGS Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California, published in 2008. Per Special Publication 117A, a design seismic peak ground acceleration of 0.48g and a coefficient of seismic horizontal acceleration, k_h , of 0.21 were used for the analysis. A minimum factor-of-safety of 1.0, per Special Publication 117A was the basis for pseudostatic analysis.

In the South Tract, the cut slopes are designed at 2H:1V or flatter and up to 85 feet high. These cut slopes will be provided with stabilization fill keys to provide a uniform fill surface and address potential surficial failures and erosion. There are no designed buttresses on the South Tract. Analysis was also performed for temporary slope stability of the deep excavations for the 50-foot-deep alluvial removal in order to provide recommendations for the grading. The highest fill slope is approximately 110 feet high and was found to have an acceptable factor of safety.

In the North Tract, the cut slopes are designed at 2H:1V or flatter and up to 95 feet high. Slope stability analysis was performed for the cut slopes and landslides. There are several buttresses designed for the stabilization of proposed cut slopes, as shown on Plate 2. The remaining cut slopes will be provided with stabilization fill keys to provide a uniform fill surface and address potential surficial failures and erosion. A shear key is also recommended at the toe of the fill slope above the landslide in the northeast portion of the site. The highest fill slope is approximately 125 feet high and was found to have an acceptable factor of safety.

3.4 Settlement and Collapse Potential

The unsuitable soils subject to settlement and collapse generally consist of alluvium deposits that are believed to be on the order of 60 feet thick in the South Tract and much less in the North Tract. The alluvium throughout the North Tract typically is less than 15 feet deep, except locally near the toe of the fill slope into Irvine Regional Park in the northeast portion of the site, where it is over 35 feet deep (Boring B-22). In general, the clayey alluvium is subject to long term settlement, more than the sandy alluvium. The near surface weathered alluvium subject to collapse will be removed during remedial grading.

The total static settlement for the deeper design fills and fills underlain by alluvium left-in-place is estimated to be several inches. A large portion of the settlement will likely occur during the grading operations. However, the areas where alluvium is left in place and in areas of deeper fills, settlement monitoring should be performed after grading prior to issuance of building permits.

The planned fills are up to 60 feet deep in the South Tract and 97 feet deep in the North Tract. In the South Tract, the alluvium to remain in place after remedial removals will be between 10 and 40 feet deep and will be filled over with between 15 to 60 feet of compacted fill. Much of the deep clayey alluvium will be removed and under the residential lots the alluvium left in place will typically be sandier. In the North Tract, most of the alluvium will be removed under residential lots prior to placement of compacted fills up to 96 feet deep. Up to 20 feet of alluvium will be left in place locally below fill slopes extending into Irvine Regional Park area. Settlement monitoring will be performed in the areas where alluvium will be left in place after grading and in the deeper fill areas (Plates 1 and 2). Enhance compaction is also planned in fill areas deeper than 60 feet.

The storm-drain Line "I" will extend from bedrock to alluvium near the Peters Canyon Outlet Structure. Based on the original study by Lawson (2006), the anticipated differential settlements under the pipe and structure would be on the order of 4 to 6 inches. As a result, MBI designed a RCP encasement that is to be structurally connected to the outfall structure as shown on Sheet 11 of the outfall structure plans. Also, the dewatering for the sewer work should be continued so that remedial removals of the alluvium may be performed under the structure (on the order of 5 to 10 feet).

3.5 Expansive Soils and Soluble Sulfates

The expansion potential of the onsite soil is predominantly medium, as classified by ASTM D4829. Although some claystone beds may be found to be of high to very high expansion

potential and sandstone beds are likely to be of very low expansion potential, grading and lot capping are likely to blend the near-surface soil so that at the completion of grading most of the near surface subgrade soils are anticipated to be within the medium expansion range. However, the potential for some areas with expansion potential in the "high" category cannot be ruled out.

The preliminary soluble sulfate exposure of onsite soils is classified as "S0" per Table 19.3.1.1 of the ACI-318-14.

3.6 Soil Corrosion Potential

The site soils range from moderately to severely corrosive to metals. This is a very common range of corrosivity within Orange County. Where metals will be in contact with onsite soils for a long period of time (such as buried iron or steel pipe), corrosion control measures should be taken to prolong their life.

3.7 Erosion Potential

The erosion potential of the onsite earth unit varies from low to medium. The topsoil, alluvium and terrace deposits would typically have the highest erosion potential if left as cut at finish grade. The bedrock units at the site are less erodible, except when left at steeper slopes. Once the site is graded, compacted fills will be exposed throughout the site. These fills will have a low erosion potential provided they are constructed with surface drainage devices in accordance with the building code.

3.8 Hard Bedrock and Oversize Material Generation

Hard Bedrock Rippability: A total of 17 seismic refraction lines have been performed at the site to assess the hardness and rippability of the deeper cuts into the denser bedrock materials (Appendix C). The tomographic models show generally low velocity rock overlying high velocity bedrock. There are several areas in the North Tract and one area in the South Tract that will encounter hard bedrock. The low velocities represent easily rippable materials (surficial deposits and weathered bedrock) overlying the harder bedrock.

Based on the Caterpillar Handbook (Volume 47, 2017 version), a D-10T bulldozer can rip up to a seismic velocity 8,500 fps in a sedimentary sandstone bedrock and up to 7,500 fps in igneous (trap volcanic) rock. A D9R/D9T can rip up to 7,500 fps in sandstone and 7,000 fps in trap volcanic rock.

Based on prior grading in the Vaqueros Sespe bedrock (interbedded sandstone and siltstone), the sedimentary bedrock could be excavated with relatively good production in materials that were 6,000 fps and below. It wasn't until the velocities were over 6,000 fps that two dozers were needed in the cut to break the rock so the scrapers could easily pick it up. Note that velocities of 8,500 fps or more were only found in SL-8 at 95 feet, SL-9 at 60 feet, SL-10 at 100 feet, and SL-14 at 85 feet. We understand no blasting will be allowed on this project.

The lines with bedrock over 6,000 fps within the limits of grading include the following:

- Seismic Lines 6 and 7 in the north-central portion of the South Tract indicate that the grading will extend into hard El Modena Volcanic bedrock at depths of 50 to 60 feet, with velocities of more than 7,000 fps. The excavations deeper than 50 feet in this area of Street A and the adjacent buttress will encounter this hard rock.
- In the North Tract, seismic lines 8, 9, 10, 14, 15 and 16 were updated by Terra Geosciences with the latest computer program. The depths to hard bedrock with greater than 6,000 fps were found at depths of 42 to 69 feet deep. The top of the hard bedrock was modeled along with the mass excavation and remedial grading bottoms. A series of cross sections were developed, and the quantity of the hard rock was calculated by Proactive.

Oversize Materials: Oversize materials are defined as rocks larger than 12 inches in the greatest dimension, in accordance with the City and County of Orange Grading Manuals. The deeper cuts will generally produce most of the oversized materials. Grading logistics should allow for having some of the deeper fills available for rock disposal in the later stages of grading. This would allow the disposal of the oversized material that might be required near the end of grading.

3.9 Earthwork Shrinkage and Bulking

The loss or gain of volume (shrinkage or bulking, respectively) of excavated natural materials and recompaction as fill varies according to earth material type and location. This volume change is represented as percent shrinkage (volume loss) or as percent bulking (volume gain) after recompaction of a unit volume of cut in this same material in its natural state. Due to the inherent variability of bedrock and soil materials, earthwork volume changes are difficult to accurately quantify. The following estimates are based on our experience with similar materials.

Earthwork Estimates	
<i>Material</i>	<i>Approximate Percent Shrinkage/Bulking</i>
Uncertified Fill, Stockpile Fill	10 to 20 percent shrinkage
Colluvium and Topsoil	10 to 15 percent shrinkage
Alluvium	8 to 10 percent shrinkage
Weathered Compacted Fill	1 to 3 percent shrinkage
Terrace Deposit	2 percent shrinkage to 3 percent bulking
Sandstone and Siltstone Bedrock and Volcanic Bedrock	0 to 5 percent bulking
Hard Bedrock (>6,000 fps)	5 to 10 percent bulking

Ground subsidence at the site is estimated to be on the order of 0.2 foot in the area of natural hillsides, but less in the previously graded areas.

4.0 CONCLUSION AND RECOMMENDATIONS

4.1 General Conclusion and Recommendation

Based on the results of our geotechnical review and analysis, the proposed rough grading and residential development is considered feasible from a geotechnical standpoint provided the geotechnical recommendations, which are included and updated in this report, are implemented during design, grading and construction. The recommendations provided herein are generally consistent with the prior geotechnical studies for the Project site, with updates per the current building codes for seismic analysis.

4.2 General Earthwork and Remedial Removals

General Earthwork: Prior to grading, deleterious material (highly organic topsoil, vegetation, trash, any construction debris, etc.) should be cleared from the site and disposed of offsite. There are several rip-rap and concrete storm drain inlets and outlets throughout both tracts that need to be removed. Oversized rip-rap and concrete may be placed in deeper fills greater than 10 feet below finish grade per the Typical Oversize Rock Placement Detail in Appendix D. All steel reinforcement shall be cut flush to the concrete prior to placement. The existing Baker water line will also need to be removed offsite after the new line is installed and becomes operational.

In the North Tract, there are concrete remnants in the eastern portion of the site that will be encountered during grading. There are several rip-rap and concrete storm drain inlets and outlets around the southern and eastern perimeters of the site that will also be encountered.

Grading and excavations should be performed in accordance with the City of Orange Grading Specifications and Manual and the General Earthwork and Grading Specifications in Appendix D. Prior to placement of fill, removal bottoms should be scarified, moisture conditioned as needed and compacted to a minimum of 90 percent relative compaction. Relative compaction should be based upon ASTM Test Method D1557. Fill material should be placed in loose lifts no greater than 8 inches in thickness and compacted prior to placement of the next lift. The compaction criteria for fill soils at the site should be 90 percent of the maximum dry density for fills that are less than 60 feet deep, and 93 percent relative compaction for fills deeper than 60 feet (see below). Ground sloping greater than 5H:1V should be prepared by benching into firm competent material as fill is placed. Moisture content of compacted fill should be over optimum moisture content (± 2 percent).

Native materials that are relatively free of deleterious and organic materials should be suitable for use as compacted fill. If import soils are required in order to achieve design grades, the import soils should be evaluated and accepted by the geotechnical consultant prior to transport to the site.

Deep Fills: Areas where fills are deeper than 60 feet thick (including remedial removals) below finish grade should be compacted to a minimum of 93 percent relative compaction based on ASTM Test Method D1557. This increased compaction should help in reducing the settlement waiting periods.

4.3 Remedial Removals

Unsuitable earth materials should be removed prior to placement of compacted fill. Unsuitable materials at the site include topsoil, colluvium, degraded and/or undocumented fill, unsuitable alluvium, weathered landslide material and weathered bedrock. In general, estimated removal depths vary from 3 to 15 feet in the South Tract, except for two areas within the central canyon where the remedial removals are anticipated to be in the range of 30- to 50-feet below the existing surface (Plate 1). Estimated removal depths vary from 3 to 35 feet in the North Tract. On the hillsides the removals are estimated to be between 3 to 8 feet deep and in the canyon areas the removals are recommended between 10 and 15 feet deep. In areas of landslides, the removals are estimated to be on the order of 8 to 15 feet deep, except in the north where the complete landslide removal will extend to 35 feet deep. Existing undocumented fill should be removed to expose competent native materials. Deeper removals may be necessary as determined by the geotechnical consultant based on exposed conditions during grading.

Removal bottoms should expose competent material and/or saturated alluvium, as reviewed, and accepted by the geotechnical consultant prior to placement of fill. Removal bottoms should be scarified, moisture-conditioned, and recompacted to a minimum of 90 percent relative compaction prior to placement of fill. If removal bottoms expose saturated alluvium, some additional measures may be required for workability and to achieve the proper compaction (i.e., mixing or bridging with gravels, sands and/or placement of a geotextile/geofabric).

One of the areas in which landslide material is mapped within design fill areas is located on the southern slope of the IRWD Zone C+ facility in the South Tract. Remedial removals are anticipated to remove 8 to 10 feet of this landslide material to competent material (Plate 1). The remedial removals south of the landslide are recommended to be on the order of 50 feet deep to remove saturated clayey alluvium. The removals will encounter groundwater and wet alluvium material at depth. The 50-foot-deep wet removals at the toe of the landslide should be performed and filled back in prior to the removals in the side canyon to the east. Removal bottoms are to be mapped and approved by the geotechnical consultant prior to backfilling. These deep removal areas are to be backfilled prior to the next removals along the toe of the landslide. Canyon-type subdrains are recommended along the side walls of the deep remedial removals. Remedial removals along the eastern side canyon of this same landslide, below the proposed Street E, should be performed in maximum 100-foot sections. Removals are anticipated to be on the order of 10 to 15 feet deep to exposed competent alluvium. The geotechnical consultant should review, test as necessary, and accept the removal bottom prior to fill placement. A canyon-type subdrain is recommended to be installed prior to fill placement. Each section is to be backfilled at least 10 feet prior to excavation of the adjacent section. Consideration should be given to performing the removals in alternating 100-foot sections (skipping a 100-foot section).

Removals should be performed under the future outfall structure to a depth of 5 to 10 feet below the bottom of the structure. The exposed subgrade soils should be observed by the geotechnical consultant to verify that the soil materials are considered suitable, prior to placement of fill materials or installation of the structure.

4.4 Capping/Overexcavation

In general, pads and streets exposing bedrock should be overexcavated and provided with a minimum 5-foot-thick fill cap measured below finish grade of the lots and streets in both tracts. Where hard bedrock is anticipated to exist (seismic velocities of near 6,000 fps or greater) below the future Baker Pipeline and Street A, deeper street overexcavation and re-compaction is recommended to go at least 2 feet below the deepest utilities. This occurs mostly in the deeper cuts into the El Modena Volcanics in the northern portion of the South Tract and in the Vaqueros/Sespe Formation bedrock in the northern portion of the North Tract.

In the remainder of the North Tract, there is an area on Street C where the overexcavation in the street should be 7 feet deep to remove some of the hard bedrock for the shallower utilities. Please note that after these overexcavations of 5 to 7 feet, there will be hard bedrock left in place below that will require special trenching equipment (i.e. large excavators, Vermeers, rock breaking equipment, etc.) to install the deep sewer and storm drain lines. This is done to reduce the excavations into and/or the extra removals of hard bedrock during grading.

If expansive claystone and siltstone beds are exposed at finish grade, they may pose a differential expansion/heave potential when abutting low- to non-expansive sandstones. As a result, the pads and streets exposing these materials should be overexcavated to a depth of 10 feet below finish grade to provide a uniform blanket of fill over the lots and streets. This will be determined by the geotechnical consultant during grading where and if the thicker "red" clay beds are encountered.

4.5 Slope Stabilization

The recommended slope stabilization keys are shown on the Remedial Measures Map (Plates 1 and 2) and a series of cross-sections presented in NMG, 2024c and 2025d. These consist mostly of stabilization fills, with a side-hill key in the South Tract and several buttresses and shear key in the North Tract. These should be constructed in accordance with the Earthwork and Grading Specifications presented in Appendix D.

The stabilization fill keys vary between 15 and 30 feet in width and to depths of 5 to 10 feet as shown on Plates 1 and 2. The backcuts should be excavated at inclinations of 1.5H:1V or flatter. The backcuts for the stabilization fills should start approximately 15 feet behind the top of the cut slope as shown on the detail in Appendix D. However, locally the top of backcut will need to stay within the property line, possibly creating a condition of overfilling and trimming back near the top-of-slope.

Cut slope stabilization measures in the South Tract consist of recommended stabilization fill keys; no buttresses are needed for adverse geologic conditions. There is also a side-hill key with a width of 20 feet and a bottom elevation of 620 feet msl recommended at the rear of lots in the northeast portion of the South Tract (Plate 1). The landslides in the South Tract will be stabilized either by filling over the toe (partial burial) or complete removal of the landslide.

In the North Tract, buttress keys varying between 40 and 100 feet in width and 5 to 15 feet in depth are recommended for slopes with adverse bedding conditions. A shear key with a width of 25 feet

and a depth of 20 feet is recommended in the northeast portion below the proposed IRWD Zone 6 reservoir site fill slope to stabilize the toe of fill slope and separate the slope from the landslide below. The backcuts should be excavated at 1.5H:1V or flatter (Plate 2).

The landslide north of SCR in the southwest portion of the North Tract should be completely removed (Plate 2). This removal should start near the upper portion of the landslide in order to reduce the potential for landslide reactivation. There are two other shallow landslides that were encountered in the central and eastern portions of the North Tract. The upper unsuitable materials in these slides will be removed and replaced. After remedial removals and fill placement, the competent landslide materials will remain buried by deep compacted fill.

In both tracts, fill keys are recommended for the fill-over-natural slopes (Plates 1 and 2). The fill keys vary from 15 to 30 feet wide and should extend at least 2 feet into competent native soils and tilted back per our details in Appendix D. These perimeter keys should be excavated to expose competent material, with remedial removals behind the keys continuing up the canyons/hillsides.

The backcuts and key excavations should be mapped by an engineering geologist as these keys are excavated to evaluate the stability and to confirm the anticipated conditions. If more adverse conditions are encountered, then the geotechnical consultant should provide updated design recommendations. The key bottoms should be reviewed and accepted by the geotechnical consultant prior to placement of fill and subdrains.

4.6 Groundwater and Excavations

NMG anticipates that during grading, removals in canyons and drainage channels may encounter groundwater and/or perched groundwater. The storm-drains that outlet onto the site below SCR and SR 261 may flow during and after storm events. Depending on the time of year that these removals are performed, the removals in these areas may expose saturated soils shallower than expected. Minor seepage along the terrace and bedrock contact or in the fractured bedrock may also be present during grading.

In the South Tract, wet removals are anticipated within the main (central) canyon at the site (Plate 1). The recommended 50-foot-deep removals are expected to extend approximately 10 to 20 feet below the groundwater table. A groundwater dewatering plan may be utilized in this canyon to reduce adverse impacts that saturated soils could present to grading operations. Groundwater will also be encountered during the sewer replacement at Peters Canyon Outlet and dewatering will be necessary in this area. These wet materials will need to be spread out, dried back and/or mixed with other materials to be placed as compacted fill.

4.7 Onsite Faulting

Based on prior studies, there are no seismically active faults at the site. There were numerous inactive faults, fractures and shears encountered in the subsurface investigation. Several of these faults may act as conduits for groundwater migration where shallow groundwater is trapped behind the uphill sides of the faults. Where seepage is encountered along faults and fractures during grading, deeper overexcavation and/or additional subdrains will likely be necessary. If significant

fault gouge (greater than 1-inch wide) is encountered the geotechnical consultant should evaluate the extent of the gouge and additional overexcavation may be recommended. Also, these features may increase the potential for instability of the temporary excavations and backcuts.

4.8 Settlement Monitoring

The total static settlement for the deeper design fills underlain by alluvium left-in-place is estimated to be several inches. A large portion of the settlement will likely occur during the grading operations. Deeper fills should have settlement monitoring in order to confirm the long-term settlement estimates.

NMG recommends monitoring of settlement upon completion of grading in eight locations in the South Tract and six locations in the North Tract where fills will be deeper than 60 feet after remedial removals and/or where alluvium is left in place. Settlement monuments should be installed at finish grade at or near completion of grading. Preliminary locations of settlement monuments are shown on Plates 1 and 2. Additional locations may be required based on the conditions observed during grading and the anticipated construction sequence for future development.

The monuments should be surveyed every two weeks for three months and monthly thereafter to initialize and monitor settlement trends. NMG does not expect the settlement monitoring to exceed a total of nine to twelve months. Survey data for settlement monuments should be forwarded to the geotechnical consultant after each reading. The monuments should be protected to ensure integrity of the data collection.

The settlement estimates and monitoring duration may be subject to revision based upon the collected monitoring data within settlement-prone areas. In general, the long-term settlement is anticipated to be on the order of 1 to 2 inches once an area is released from a geotechnical standpoint. Also, the differential settlement should not exceed 1 inch over a 40-foot span.

4.9 Rippability and Placement of Oversize Material

Geophysical surveys (seismic refraction) were conducted along ridge-tops with large amounts of planned cuts up to 110 feet deep with remedial grading. Based on the recorded shear wave velocities of over 6,000 feet per second, the majority of the bedrock at the site should be rippable with some difficulty using D-10T bulldozers. Bedrock cuts deeper than 50 feet below existing topography are anticipated to be difficult to very difficult to rip with D-10T bulldozers and might result in slower production rates. There may be some local cemented layers in the upper 50 feet that will also be difficult to rip. The geophysical results are discussed in Section 3.8 above and provided in Appendix C.

Cuts into the denser and more cemented bedrock will produce significant amounts of oversize rock (greater than 12 inches in size) that will require special placement in the deeper fills.

Under residential areas, oversize materials may be placed as rock rows in fills deeper than 10 feet or 2 feet below the deepest utilities in structural areas. Rocks exceeding 6 inches in the largest dimension

should not be placed within fills in the upper 5 feet of residential pads. Rocks/boulders over 4 feet in diameter should be isolated and buried separately, generally deeper than 20 feet below finish grade. Rocks over 7 feet in diameter should be broken down prior to placement in the fills. The oversize materials may be placed in accordance with the Typical Oversize Rock Placement Detail in Appendix D.

Because of the amount of the oversize material anticipated to be generated in the Nouth Tract during the deeper excavations and during the later stages of grading, an alternative placement method may be performed in non-structural landscape and fill slope areas. This alternative allows placement of oversize rocks in layers as described in the Alternative Oversize Rock Placement Detail in Appendix D. These layers should be at least 20 feet behind the slope face and at least 10 feet deep below finish grade. This alternative method should result in satisfactory fill for the stability and support of the proposed non-structural improvements at the site.

4.10 Subdrainage

Canyon-type subdrains should be placed after remedial removals in the canyon bottoms and in some of the perimeter keys prior to placement of fill (Plates 1 and 2). These canyon-type subdrains should consist of a perforated Schedule 40 PVC pipe inserted into a minimum of 9 cubic feet per linear foot of $\frac{3}{4}$ - to 1-inch-diameter gravel and wrapped in geotextile filter fabric (Mirafi 140N or equivalent). In general, the canyon-type subdrains may be placed as follows:

1. 4-inch pipe may be used for drains 20 to 500 feet (upper most portions) of the subdrain.
2. 6-inch pipe may be used between 500 feet and 1,000 feet of the subdrain.
3. 8-inch pipe should be used in portions of the subdrain over 1,000 feet long to the outlet location.

However, in areas of shallow groundwater or heavy seepage, the minimum pipe size should be 6 inches in diameter.

There are several subdrain outlets mapped at the edge of the fills for both the SR 261 and SCR fill slopes. These outlets will need to be extended and outlet down canyon to an appropriate location and should be determined in the field during grading by the geotechnical consultant. There are two areas of previously mapped subdrain outlets along the eastern portion of the site from the SR 261 to where it meets with SCR that will also need to be extended and outlet. Due to the deep canyons north of SCR, there are three existing culverts where subdrain pipe (non-perforated, 8-inch PVC pipe) will need to be extended through the culvert in order to provide an outlet for the deep subdrains on the north (Plates 1 and 2).

Backdrains (3 cubic feet of gravel per linear foot of subdrain) should also be provided for all buttresses, stabilization fills and shear key backcuts at 30-foot-vertical intervals with outlets every 100 feet through the slope face (Appendix D). The lowest subdrain should start along the bottom of the key where possible with an acceptable outlet or at the lowest possible part of the backcut allowing for an outlet to a future storm drain or slope face. Outlet spacing can be increased up to a maximum of 500 feet if the backdrain is constructed with 5 cubic feet per foot of gravel. Additional backdrains may be necessary for areas if seepage is encountered. These backdrain locations should be determined in the field during grading by the geotechnical consultant.

During grading, additional subdrains may be necessary for areas where seepage is encountered (i.e., along highly fractured bedrock zones). Canyon-type subdrain and backdrain details are included in the General Earthwork and Grading Specifications (Appendix D).

Protection of Subdrain Outlets: Once the outlets are videoed and found to be acceptable, the outlet pipe should be protected by installation of devices per exhibit labeled "Subdrain Outlet Marker Detail" in the Grading and Earthwork Specifications (Appendix D). This will allow the pipe outlets to be protected in the future during landscaping and make them easier to find, if necessary.

4.11 Expansion Potential and Soluble Sulfate

The expansion potential of the onsite soil is predominantly medium, as classified by ASTM D4829. Although some claystone beds may be found to be of high to very high expansion potential and some sandstone beds are likely to be of very low expansion potential, grading and lot capping are likely to blend the near-surface soil so that at the completion of grading most of the near surface subgrade soils are anticipated to be within the medium expansion range. However, the potential for some areas with expansion potential in the "high" category cannot be ruled out.

The preliminary soluble sulfate exposure of onsite soils is classified as "S0" per Table 19.3.1.1 of the ACI-318-14. However, soil with abundant gypsum crystal may increase the sulfate exposure and could be classified as "S1" or "S2." We recommend that additional sulfate testing be performed on near-surface soils after completion of grading.

4.12 Soil Corrosivity to Metal

The site soils range from moderately to severely corrosive to metals. This is a very common range of corrosivity within Orange County. Where metals will be in contact with onsite soils for a long period of time (such as buried iron or steel pipe), corrosion control measures should be taken to prolong their life. Site-specific testing should be considered following grading.

4.13 Preliminary Lateral Earth Pressures

Lateral earth pressures for retaining structures may vary for different earth materials across the subject site. With approved onsite soils in drained conditions, the following design criteria are provided for preliminary purposes. These preliminary recommendations should be verified and/or revised based upon the as-graded conditions. The two tables below provide the recommended equivalent fluid pressures for the North and South Tracts.

SOUTH TRACT

Equivalent Fluid Pressure (psf/ft.)		
<i>Conditions</i>	<i>Level</i>	<i>2:1 Slope</i>
Active	45	90
At Rest	65	95
Passive	320	130 (sloping down)

NORTH TRACT AND PETERS CANYON OUTFALL

Equivalent Fluid Pressure (psf/ft.)		
<i>Conditions</i>	<i>Level</i>	<i>2:1 Slope</i>
Active	40	65
At Rest	60	85
Passive	360	130 (sloping down)

Soils that have high expansion potential and oversize rocks should not be used in the wall backfill. To design an unrestrained retaining wall, such as a cantilever wall, the active earth pressure may be used. For a restrained retaining wall, such as a vault or at restrained wall corners, the at-rest pressure should be used. Passive pressure is used to compute lateral soils resistance developed against lateral structural movement. The passive pressures provided above may be increased by one-third for wind and seismic loads. Further, for sliding resistance the friction coefficient of 0.35 may be used at the concrete and soil interface. In addition, the passive resistance is taken into account only if it is ensured that the soil against embedded structure will remain intact with time. The retaining walls may also need to be designed for additional lateral loads if other structures or walls are planned within a 1H:1V projection.

The seismic lateral earth pressure for walls retaining more than 6 feet of soil and level backfill conditions may be estimated to be an additional 22 pcf for active and at-rest conditions. The earthquake soil pressure has a triangular distribution and is added to the static pressure. For the active and at-rest conditions, the additional earthquake loading is zero at the top and maximum at the base. The seismic lateral earth pressures do not apply to walls retaining less than, or equal to, 6 feet of soil (2025 CBC Section 1803.5.12).

Typical retaining wall drainage detail is included as Figure 7 (rear of text). Proper surface drainage, such as a concrete V-ditch, should also be provided along the top of walls. Downdrains (outlets) for surface drainage should not be tied into the subdrain system for walls, they should be outlet separately.

4.14 Seismic Design Parameters

The following table summarizes the seismic design criteria for the subject site. The seismic design parameters are developed in accordance with 2025 CBC and ASCE 7-22. Due to the variable depths to bedrock across the overall site, the seismic parameters were determined using the Default Site Class per ASCE 7-22. Once individual tract boundaries are determined, additional seismic

analysis will be provided to better represent the individual tract geologic conditions. This analysis was performed for a central location to the combined project limits (Figure 1).

<i>Selected Seismic Design Parameters from 2025 CBC/ASCE 7-22</i>	<i>Seismic Design Values</i>	<i>Reference</i>
Latitude	33.7878 North	
Longitude	117.7551 West	
Controlling Seismic Source	Whittier (2)	USGS, 2025
Distance to Controlling Seismic Source	6.6 Miles (10.65 km)	USGS, 2025
Site Class per Section 20.3 and 11.4.2 of ASCE 7-22	Default	
S_s , Spectral Acceleration for Short Periods	1.65 g	SEA/OSHPD, 2025
S₁ , Spectral Accelerations for 1-Second Periods	0.52 g	SEA/OSHPD, 2025
S_{DS} , Design Spectral Response Acceleration at Short Periods from Equation 11.4-1 of ASCE 7-22	1.25 g	SEA/OSHPD, 2025
S_{DI} , Design Spectral Response Acceleration at 1-Second Period from Equation 11.4-2 of ASCE 7-22	0.86 g	SEA/OSHPD, 2025
T_s , S_{DI} / S_{DS} , Section 11.4.5.2 of ASCE 7-22	0.68 sec	SEA/OSHPD, 2025
T_L , Long-Period Transition Period	8 sec	SEA/OSHPD, 2025
PGA_M , Peak Ground Acceleration Corrected for Site Class Effects from Section 11.8.3 of ASCE 7-22	0.72 g	SEA/OSHPD, 2025
Seismic Design Category , Section 11.6 of ASCE 7-22	D	SEA/OSHPD, 2025

4.15 Structural Setbacks

The footings of structures (including retaining walls) located above descending slopes should be setback from the slope face in accordance with the minimum requirements of the City of Orange Grading Manual, whichever is greater. The setback distance is measured from the outside edge of the footing bottom along a horizontal line to the face of the slope. The table provided below summarizes the minimum setback criteria for structures above descending slopes.

Structural Setback Requirements for Footings Above Descending Slopes	
<i>Slope Height [H] (feet)</i>	<i>Minimum Setback from Slope Face (feet)</i>
Less than 6	5
6 to 14	H/2; minimum 5
14 to 30	H/2; maximum 10
More than 30	H/3; maximum 40

4.16 Utility Installation and Trench Backfill

Geologic Conditions: Trench excavations are anticipated to expose competent artificial fill overlying alluvium and/or bedrock. The sedimentary bedrock is known to have interlayered weak clay and silt beds, faults, fractures and shears that could adversely impact the trench wall stability.

The alluvium is known to have sandy zones and soft clay zones that could be subject to trench failure.

Trench Excavations: Trench excavations should conform to applicable safety requirements and at minimum, trenches should be shored or sloped in accordance with the CalOSHA excavation requirements for trenches less than 20 feet deep. Trenches deeper than 20 feet require specific evaluation/analysis by a geotechnical consultant. Trench excavations adjacent to structures should conform to any clearance requirements on the grading and foundation plans.

In general, trenches exposing compacted fill are anticipated to expose Type B soils (CalOSHA classification). Trenches exposing alluvium may be classified as either Type B or C, depending upon the density, friable sandy layers and groundwater. The bedrock is generally considered Type C soil. As described above, trenches may encounter adverse geologic conditions and/or potential seepage where simply adhering to CalOSHA requirements for Type C soil may not be sufficient to prevent trench failures. Where bedrock is exposed in a trench, the contractor should notify the project geotechnical consultant to evaluate the trench conditions and provide recommendations as needed. Long portions of the trench should not be left open without shoring or shields to minimize the trench wall failures.

Temporary spoil piles should not be placed on the sides of trenches where bedding is dipping adversely into the trench (geologically referred to as the up-dip side). The spoil pile should be placed on the other side and set back from the edge of the trench a minimum distance equal to the depth of the trench.

Trench Backfill: The excavated soils should be suitable for use as trench backfill, with the exception of oversized rock that may be encountered. Backfill materials should be compacted to a minimum of 90 percent relative compaction, as indicated in Section 3.2, and the General Earthwork and Grading Specifications (Appendix D).

Trenches excavated on a graded slope-face should be properly backfilled and compacted in order to obtain a minimum 90 percent relative compaction to the slope face.

4.17 Jack and Bore Pits

There are three planned jack and bore crossings (tunneling under existing roads and or utilities) at the Project site. The drilling for the Baker Pipeline under SCR is anticipated to encounter fill over volcanic and sandstone bedrock that were excavatable with both hollow-stem drilling rig and Caterpillar 330 Excavator trench on both sides of the roadway. These excavations did not encounter groundwater.

During drilling for the jack and bore for storm drain Line "I" both under Jamboree Road and to the west to the outlet, the drilling will encounter the Topanga Formation. There were at least three hollow-stem borings drilled to the depths of the storm drain that indicate the sandstone will be excavatable. Based on the groundwater well installed at the South Tract, it is anticipated that the groundwater will be just below the jack/bore pit on the east side of Jamboree Road and 3 to 6 feet below the pits to the west. However, local seepage may be encountered higher in the excavation.

The jack/bore pits and drilling to the west of Jamboree Road will need to be carefully shored and/or controlled to protect the existing water lines in the area.

In general, micro-tunneling and/or drilling through the bedrock can be locally problematic due to bedding and fractures causing the drill to deflect in direction. Additional effort may be necessary during excavation to keep the boring in the correct alignment.

4.18 Select Material and Grading for Retaining Wall Backfill

Select materials should be placed behind the proposed walls on the South Tract, located along the east side of the tract and below SR 241, to the north and south of Handy Creek Road. This includes the two masonry walls and the cast-in-place wall. The rough grading contractor should first construct the recommended stabilization fills with onsite fill materials (Plate 1). Then the walls should be constructed, with select backfill placed behind the walls up to finish grade.

The select backfill materials are anticipated to be mined onsite from the El Modena Volcanics in the northwest portion of the tract or in the Topanga Formation Sandstone in the southwest portion of the tract. The select backfill will be provided by the rough grade contractor and stockpiled near the proposed walls in a location agreed upon by the developer, grading contractor, and wall contractor.

Select materials should be soil type SM, SP, GP, or GW, of low plasticity with a minimum friction angle (Φ) of 30°. NMG should review and test this material to confirm it meets the above criteria.

4.19 Soil Cement for Baker Pipeline

Soil cement is recommended along the northern tie-in of the Baker Pipeline (NMG, 2022a). In order to construct the tie-in, an excavation up to 30 feet deep will be made through approximately 10 feet of terrace material overlying sandstone bedrock. These materials appear to be mostly massive and friable; however, the bedrock dips to the south, which is into the hillside and favorable for slope stability. Some of the bedrock may be difficult to excavate and require a larger excavator. The pipeline contractor will also be responsible for removing the interfering portion of the existing pipeline as shown on Connection Detail Station 42+15 on Sheet C4 of the plan. We anticipate that the excavations made in order to perform this work may impact a large portion of the upper hillside and remove some of the existing drainage devices and slope protection on the hillside.

As part of reconstruction of the existing slope face over the pipeline, the excavation is recommended to be backfilled with a soil-cement mixture and finished near the existing slope face configuration. The required soil-cement mixture will depend on the soil materials available at the site. Based on our review of available information related to the site, the excavated materials are anticipated to consist of silty sand materials (SM). The preliminary required cement content for the soil materials is 8 percent by dry weight. The design of the soil-cement mixture will ultimately be based on laboratory testing of the available soils and the compressive strength of the mixture.

The approximate area of soil cement is shown in the referenced project memorandum (NMG, 2022a) and on Plate 2, with a minimum of 10 feet behind the top of the permanent slope face and

20 feet at the bottom of the excavation (extending back to the slope anchor south of the tie in). The top of the soil cement should slope at least 2 percent to the south for drainage purposes.

The soil-cement mixture should be prepared at 2 to 3 percent over optimum moisture content per ASTM D1557. The mixture must be placed and compacted within two hours of its preparation. The soil-cement fill should be compacted to a minimum of 95 percent relative compaction. The embankment slope should be overbuilt 2 to 4 feet (depending on the steepness of the slope and compaction equipment) and trimmed back to design slope face grades.

4.20 Surface Drainage

Surface drainage should be carefully taken into consideration during grading, landscaping, and building construction. Positive surface drainage should be provided to direct surface water away from structures and slopes and toward the street or suitable drainage devices. Ponding of water adjacent to the structures or tops of slopes should not be allowed. Paved areas should be provided with adequate drainage devices, gradients, and curbing to reduce run-off flowing from paved areas onto adjacent unpaved areas.

4.21 Maintenance of Graded Slopes

To reduce the erosion and surficial slumping potential of the graded slopes, permanent manufactured slopes should be protected from erosion by planting with appropriate ground cover or by placing suitable erosion protection. These measures should be applied as soon as is practical. Proper drainage should be designed and maintained to collect surface waters and direct them away from slopes. In addition, the design and construction of improvements and landscaping should also provide appropriate drainage measures.

4.22 Future Geotechnical Reviews

The geotechnical consultant should review future grading/construction plans in light of the site-specific geotechnical conditions. Remedial grading measures consistent with the recommendations in this report will be provided as needed. In addition, future precise grading, improvement, building foundation and retaining wall plans will need to be reviewed by the geotechnical consultant as they become available.

4.23 Geotechnical Observation and Testing during Grading

The findings, conclusions and recommendations in this report are based upon interpretation of the existing data. Verification and refinement of actual geotechnical conditions during grading is essential, especially where slope stabilization is involved. Geotechnical observation and testing should be conducted during the following stages:

- During and following clearing and grubbing, prior to site processing;
- During and following remedial removals to evaluate and accept the removal bottom;
- During and following cutting of slopes and excavation of slope-stabilization measures;

- During installation of subdrains;
- During placement of compacted fill and soil cement;
- During and upon completion of excavations for storm-drain structures and during trench backfill;
- During construction and backfill of utility lines and/or structures;
- During pavement subgrade and aggregate-base preparation for street pavements; and
- When any unusual or unexpected geotechnical conditions are encountered during grading and construction.

4.24 Limitations

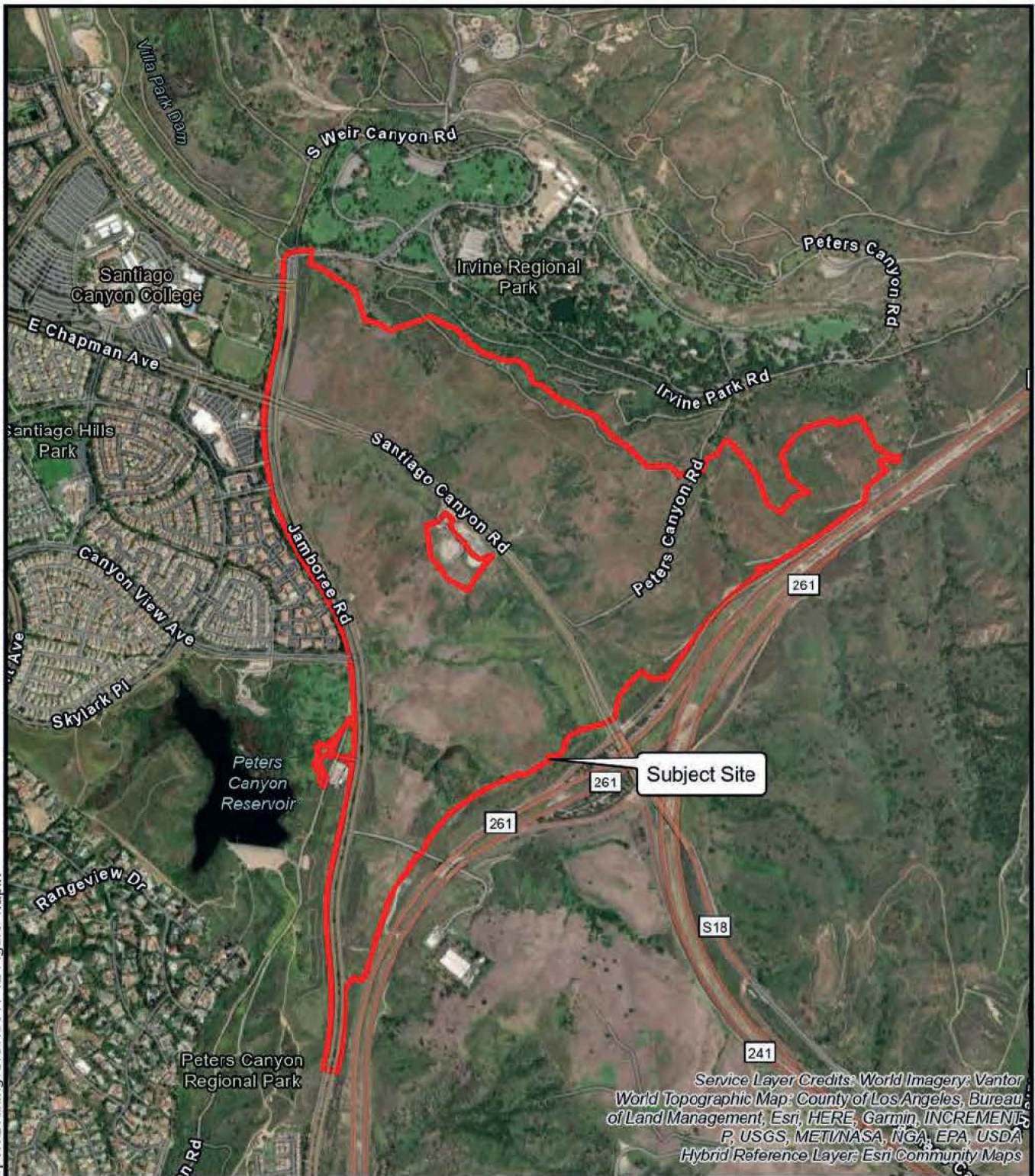
Our findings, conclusions, and recommendations are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in between points, and can also change over time. Standards of practice and codes also change with time. Our conclusions and recommendations are subject to verification and/or modification with more exploration and/or during grading and construction when more subsurface conditions are exposed.

5.0 MITIGATION MEASURES

As previously discussed, this report is a summary of the prior reports prepared by NMG and others pertaining to the grading and development of the planned Orange Heights Project (Appendix A). The Santiago Hills II and East Orange Planned Communities Final Supplemental Environmental Impact Report 1278/Environmental Impact Report 1716 were certified by the City of Orange in 2005 (2005 SEIR), and the 2016 Addendum to the 2005 SEIR (Previous Environmental Documentation) identify mitigation measures applicable to development at the Project site.

Table 1 includes a list of the adopted mitigation measures and identification of their current status. As identified many of the required reports have been prepared, most of the required investigations have been performed, and remedial recommendations for grading have been established. Grading plans were approved by the City for the previously approved tract maps and grading permits were issued in 2022. The Project proposes minor revisions to the previously approved grading plans.

Consistent with the conclusion of the Previous Environmental Documentation, implementation of the previously adopted mitigation measures would reduce impacts related to geology and soils to less than significant levels.



0 750 1,500 Feet
1 inch = 1,500 feet

SITE LOCATION MAP

ORANGE HEIGHTS DEVELOPMENT
CITY OF ORANGE, CALIFORNIA

Project Number: 19011-12 By: RTE/TW
Project Name: ICDC/Orange Heights SEIR
Date: 1/12/2026 Figure No. 1





Legend

- Liquefaction Zones
- Landslide Zones

0 1,000 2,000
 Feet
 1 inch = 2,000 feet

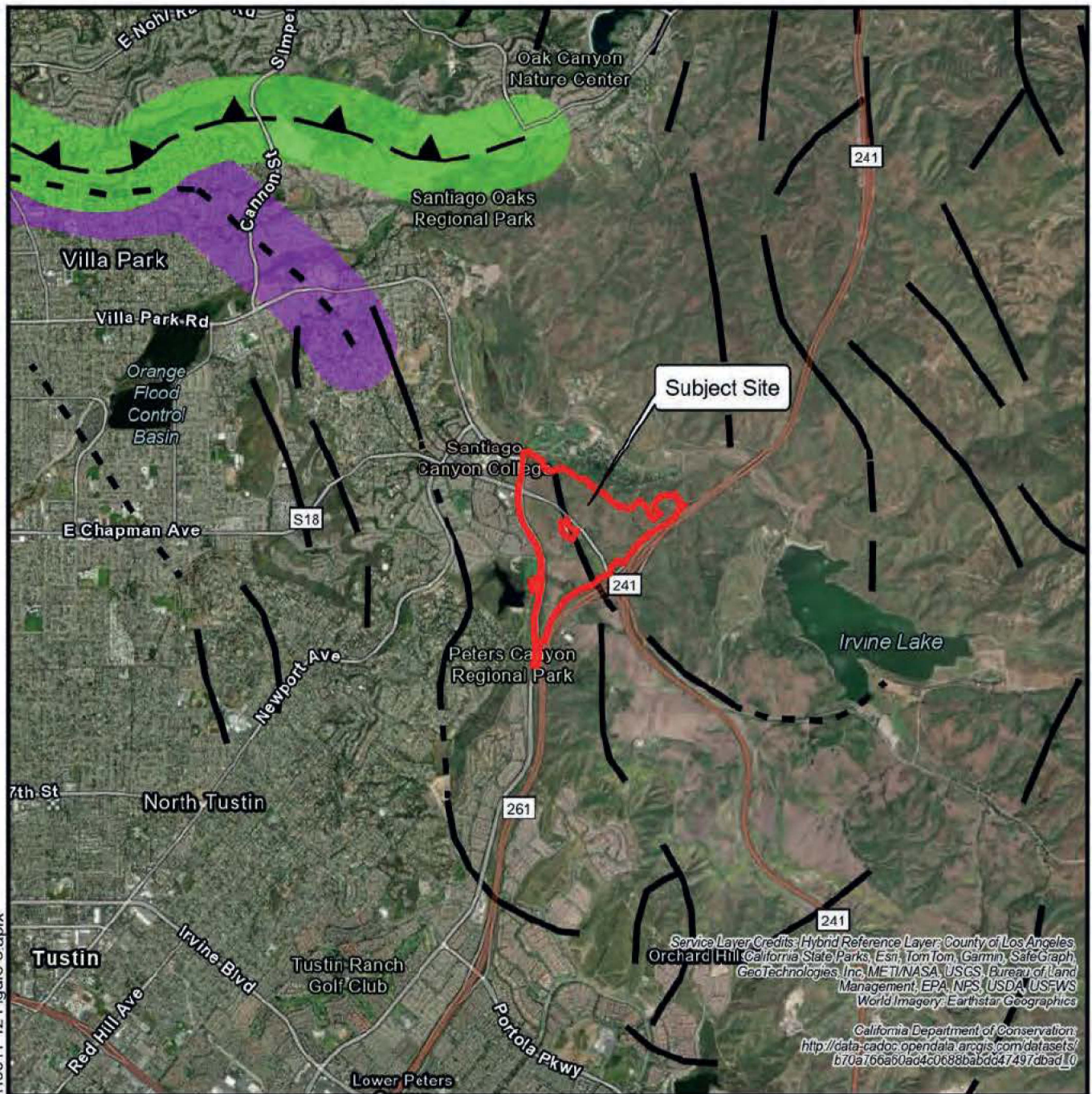


SEISMIC HAZARDS MAP

ORANGE HEIGHTS DEVELOPMENT
 CITY OF ORANGE, CALIFORNIA

Project Number: 19011-12 By: RTE/TW
 Project Name: ICDC/Orange Heights SEIR
 Date: 1/12/2026 Figure 2





- Legend**
- fault, certain
 - - - fault, approx. located
 - · · fault, concealed
 - ▲ thrust fault, certain
 - ▲ thrust fault, approx. located

- Regency of Movement**
- Orange Holocene
 - Green Late Quaternary
 - Purple Quaternary
 - Black Pre-Quaternary

0 3,000 6,000 Feet
 1 inch = 6,000 Feet

CALIFORNIA FAULT ACTIVITY MAP

Base: California Geological Survey, Fault Activity Map of California, 2010

ORANGE HEIGHTS DEVELOPMENT
 CITY OF ORANGE, CALIFORNIA

Project Number: 19011-12 By: RTE/TW
 Project Name: ICDC/Orange Heights SEIR
 Date: 1/12/2026 Figure 3





Legend

- AP Fault Zones
- AP Faults

0 10,000 20,000
Feet
1 inch = 20,000 feet



FAULT RUPTURE HAZARD ZONE MAP

Source: <https://maps.conservacion.ca.gov/cgs/informationwarehouse/eqzapp/>

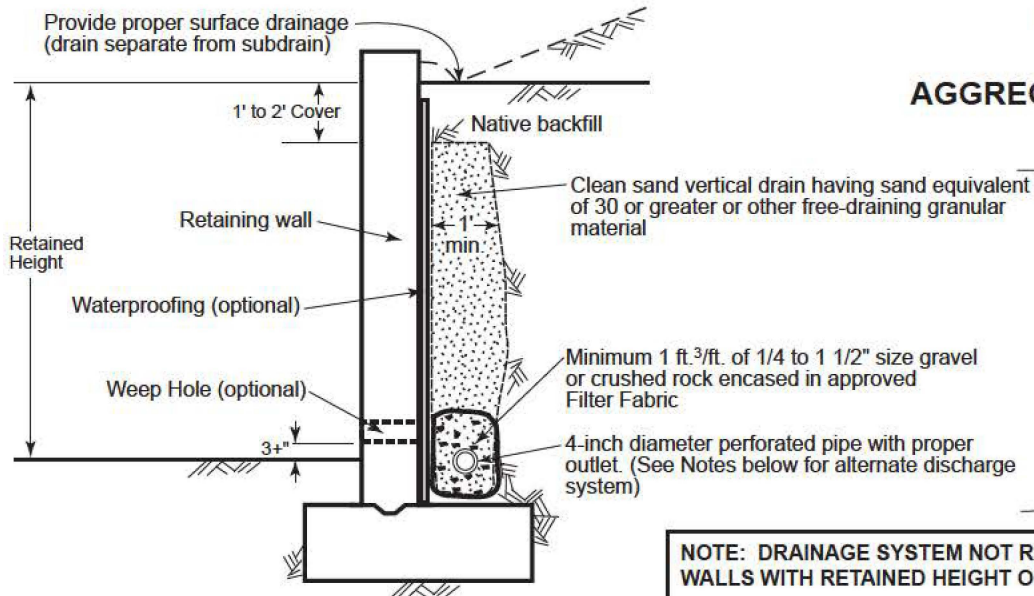
ORANGE HEIGHTS DEVELOPMENT
CITY OF ORANGE, CALIFORNIA

Project Number: 19011-12 By: RTE/TW
Project Name: ICDC/Orange Heights SEIR
Date: 1/12/2026 Figure 4



OPTION 1:

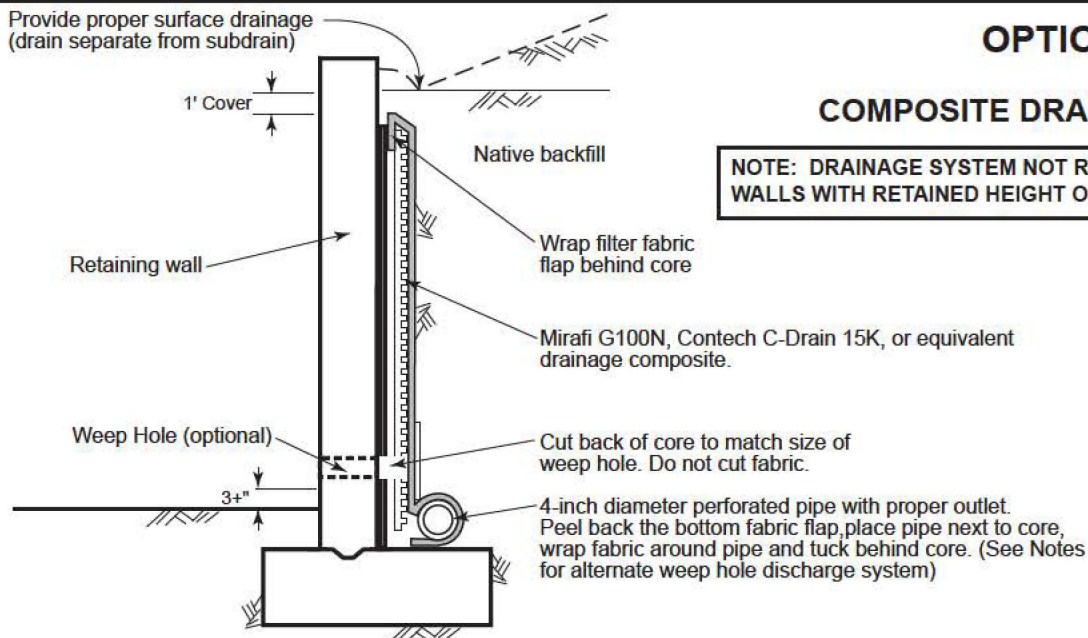
AGGREGATE SYSTEM DRAIN



Alternative: Class 2 permeable filter material (Per Caltrans specifications) may be used for vertical drain and around perforated pipe (without filter fabric)

OPTION 2:

COMPOSITE DRAINAGE SYSTEM



NOTES:

1. PIPE TYPE SHOULD BE PVC OR ABS, SCHEDULE 40 OR SDR35 SATISFYING THE REQUIREMENTS OF ASTM TEST STANDARD D1527, D1785, D2751, OR D3034.
2. FILTER FABRIC SHALL BE APPROVED PERMEABLE NON-WOVEN POLYESTER, NYLON, OR POLYPROPYLENE MATERIAL.
3. DRAIN PIPE SHOULD HAVE A GRADIENT OF 1 PERCENT MINIMUM.
4. WATERPROOFING MEMBRANE MAY BE REQUIRED FOR A SPECIFIC RETAINING WALL (SUCH AS A STUCCO OR BASEMENT WALL).
5. WEEP HOLES MAY BE PROVIDED FOR LOW RETAINING WALLS (LESS THAN 3 FEET IN HEIGHT) IN LIEU OF A VERTICAL DRAIN AND PIPE AND WHERE POTENTIAL WATER FROM BEHIND THE RETAINING WALL WILL NOT CREATE A NUISANCE WATER CONDITION. IF EXPOSURE IS NOT PERMITTED, A PROPER SUBDRAIN OUTLET SYSTEM SHOULD BE PROVIDED.
6. IF EXPOSURE IS PERMITTED, WEEP HOLES SHOULD BE 2-INCH MINIMUM DIAMETER AND PROVIDED AT 25-FOOT MAXIMUM SPACING ALONG WALL. WEEP HOLES SHOULD BE LOCATED 3+ INCHES ABOVE FINISHED GRADE.
7. SCREENING SUCH AS WITH A FILTER FABRIC SHOULD BE PROVIDED FOR WEEP HOLES/OPEN JOINTS TO PREVENT EARTH MATERIALS FROM ENTERING THE HOLES/JOINTS.
8. OPEN VERTICAL MASONRY JOINTS (I.E., OMIT MORTAR FROM JOINTS OF FIRST COURSE ABOVE FINISHED GRADE) AT 32-INCH MAXIMUM INTERVALS MAY BE SUBSTITUTED FOR WEEP HOLES.
9. THE GEOTECHNICAL CONSULTANT MAY PROVIDE ADDITIONAL RECOMMENDATIONS FOR RETAINING WALLS DESIGNED FOR SELECT SAND BACKFILL.

RETAINING WALL DRAINAGE DETAIL

NMG
Geotechnical, Inc.

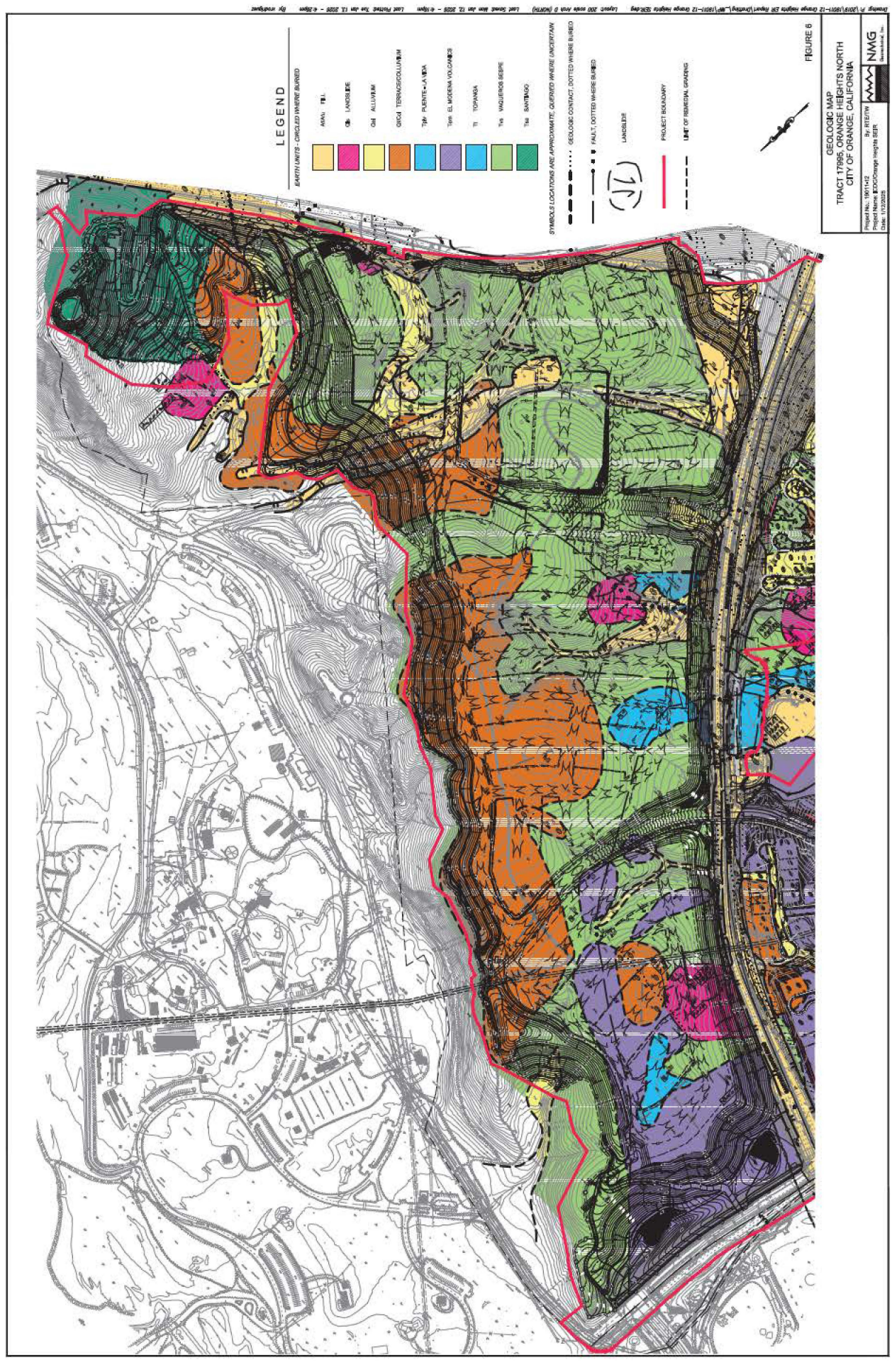


TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
MM G-1. All grading for development areas within the project site will be subject to the regulations included in the City Grading Manual.	1. Grading plans.	1. Prior to approval of final grading plans.	1. City Public Works Department	Grading plans were previously approved by the City.
MM G-2. Removal of colluvium, alluvium, topsoil, landslide debris, and uncompacted artificial fill down to an elevation where suitable foundation earth materials exist will be required prior to placement of fill in areas where these deposits occur. Specific grading recommendations for removal depths will be determined as part of future, more detailed geotechnical studies and will be subject to the approval of the City.	1. Geotechnical studies. 2. Report by geologist of compliance.	1. Prior to approval of final grading plans. 2. Following grading but prior to issuance of building permits.	1. City Public Works Department 2. City Public Works Department	Grading plans were previously approved by the City.
MM G-4. Detailed geotechnical and soil engineering reports will be prepared subsequent to development of preliminary design layouts and final grading plans (e.g., at the tentative tract map preparation stages). The reports will provide more detailed measures for treatment of excavation (ripping) difficulties, surficial material removals, cut and fill slopes, expansive soils, and liquefaction hazards, as well as measures to avoid rockfall and mudflows, as appropriate. The geotechnical analyses will be subject to technical review and approval of the City or other agencies, as appropriate.	1. Detailed geotechnical and soil engineering reports.	1. Prior to approval of tentative tract maps.	1. City Community Development Director	Grading plans and certain tract maps were previously approved by the City.

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
MM G-5. All structures within the project site shall be designed and constructed in accordance with seismic safety design criteria specified in the latest building code or as otherwise recommended by a qualified, registered structural engineer. Building plans will be subject to review and approval by the City Building Department to provide an additional level of safety.	1. Detailed geotechnical and soil engineering reports.	1. Prior to issuance of a building permit approval of tentative tract maps.	1. City Community Development Director	The timing of the verification has been corrected consistent with the wording of the mitigation measure.
MM 3C-1. Slopes Will Be Limited to 2:1. As part of the grading plans and during grading activities, all cut and fill slopes will not exceed slope ratios of 2:1 (horizontal to vertical).	1. Grading plans.	1. Prior to approval of final grading plans.	1. City Public Works Department	Grading plans were previously approved by the City. There are no proposed slopes exceeding a 2:1 slope ratio.
MM 3C-2. Slopes Will Be Stabilized. As part of the grading plans and during grading activities, cut slopes along the northern boundary of Santiago Hills II Planned Community will be stabilized (through stabilization and/or buttress fills) to avoid the effects of adverse geologic conditions (e.g., out-of-slope bedding or highly erosive materials and landslides).	1. Geologist report verifying required post-grading conditions.	1. Prior to issuance of building permits.	1. City Building Department	Grading plans were previously approved by the City.

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
<p>MM 3C-3. Standard Grading Codes Will Be Applied. The grading plans and grading activities will comply with all standard grading codes, including:</p> <ul style="list-style-type: none"> • All graded and natural slopes will have a minimum static factor of safety of 1.5 and a minimum pseudostatic (seismic) factor of safety of 1.1. • All temporary construction cut slopes will only be left exposed for as short a time as practical prior to filling to reduce the potential for slope failure. • All cut slopes will be evaluated and mapped, as they are graded, by a qualified geologist and geotechnical engineer to identify adverse bedding, weak zones, or other features that could affect the stability. If cut slopes are found to expose adverse geologic conditions, then buttresses will be constructed or the slope geometry modified. If cut slopes are found to expose fractured/faulted bedrock and surficial instability, then stabilization fills will be required. • All fill slopes will be evaluated for stability by a qualified engineer and or geologist. 	<ol style="list-style-type: none"> 1. Grading plan. 2. Geologist report verifying required post-grading conditions. 	<ol style="list-style-type: none"> 1. Prior to approval of final grading plans. 2. Prior to issuance of building permits. 	<ol style="list-style-type: none"> 1. City Public Works Department 2. City Building Department 	Grading plans were previously approved by the City.

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
<p>MM 3C-4. Compressible Soils Will Be Identified. Before submission of final grading plans, the developer will retain a qualified geotechnical engineer and engineering geologist. The geologist/engineer will conduct field observations and testing of onsite soils and formations to identify and define the limits of compressible soils. A final report will be prepared and submitted to the City. This report will identify:</p> <ul style="list-style-type: none"> • thickness and distribution of the compressible materials, • anticipated depth of moisture variation, • anticipated amounts of total and differential settlement, • tolerance of the structure for displacement, and • confirmation or modification of mitigation measures for compressible materials. 	1. Geologist report.	1. Prior to approval of final grading plans.	1. City Public Works Department	Grading plans were previously approved by the City.

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
<p>MM 3C-5. Compressible Soils Will Be Mitigated. Following identification and delineation of compressible materials, the engineers will identify the most appropriate methods of mitigation. Mitigation measures will include one or more of the following.</p> <ul style="list-style-type: none"> • removal and/or compaction of compressible natural soils (e.g., alluvium, colluvium, and deeply weathered bedrock) and non-compacted fills before placing fill; • provision of gradual transitions between cuts and fills; • location of structures away from abrupt transitions; • location of structures on soils that are relatively uniform; • compaction of deep fills to at least 93-95 percent relative compaction, and at high moisture content; • delay of construction for site improvements and buildings until after until settlement has essentially stopped; • construction of a drainage system that provides complete control of flows and control landscape watering to preclude water from ponding and/or penetrating the deep fills; and • where required, foundations designed to span across areas of potential differential settlement, or utilize deep foundations that gain support in similar materials below the settlement zones. 	<ol style="list-style-type: none"> 1. Grading plans with required measures. 2. Building plans with required measures, if necessary. 	<ol style="list-style-type: none"> 1. Prior to approval of tentative tract maps. 2. Prior to issuance of building permits. 	<ol style="list-style-type: none"> 1. City Community Development Director 2. City Building Department 	<p>Grading plans were previously approved by the City. Consistent with the grading plans, deep soils compaction requirements apply to fills deeper than 60 feet. The mitigation measure has been modified to reflect current industry standards relative to compaction, and as incorporated into the revised grading plans.</p>

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
MM 3C-6. Implement Sediment Transport Minimization Measures. Grading and construction activities will utilize standard practices—such as dust control, impoundment dikes, erosion control, and revegetation—to minimize potential for increases in sediment transport. Such measures will be subject to approval of a notice of intent and SWWPP consistent with SWRCB criteria for construction sites.	1. Grading plans with required measures.	1. Prior to approval of final grading plans.	1. City Public Works Department	Grading plans with the identified requirements were previously approved by the City.
MM 3C-7. Revegetation of Exposed Graded Slopes. All graded slopes (cut and fill), including roadsides, will undergo permanent revegetation as feasible and appropriate to minimize the potential for erosion and siltation.	1. Grading plans with required measures. 2. Geologist report verifying compliance.	1. Prior to approval of final grading plans. 2. Prior to permit of occupancy for affected sector.	1. City Public Works Department 2. City Building Department	Grading plans with the identified requirements were previously approved by the City

TABLE 1 – CEQA TABLE

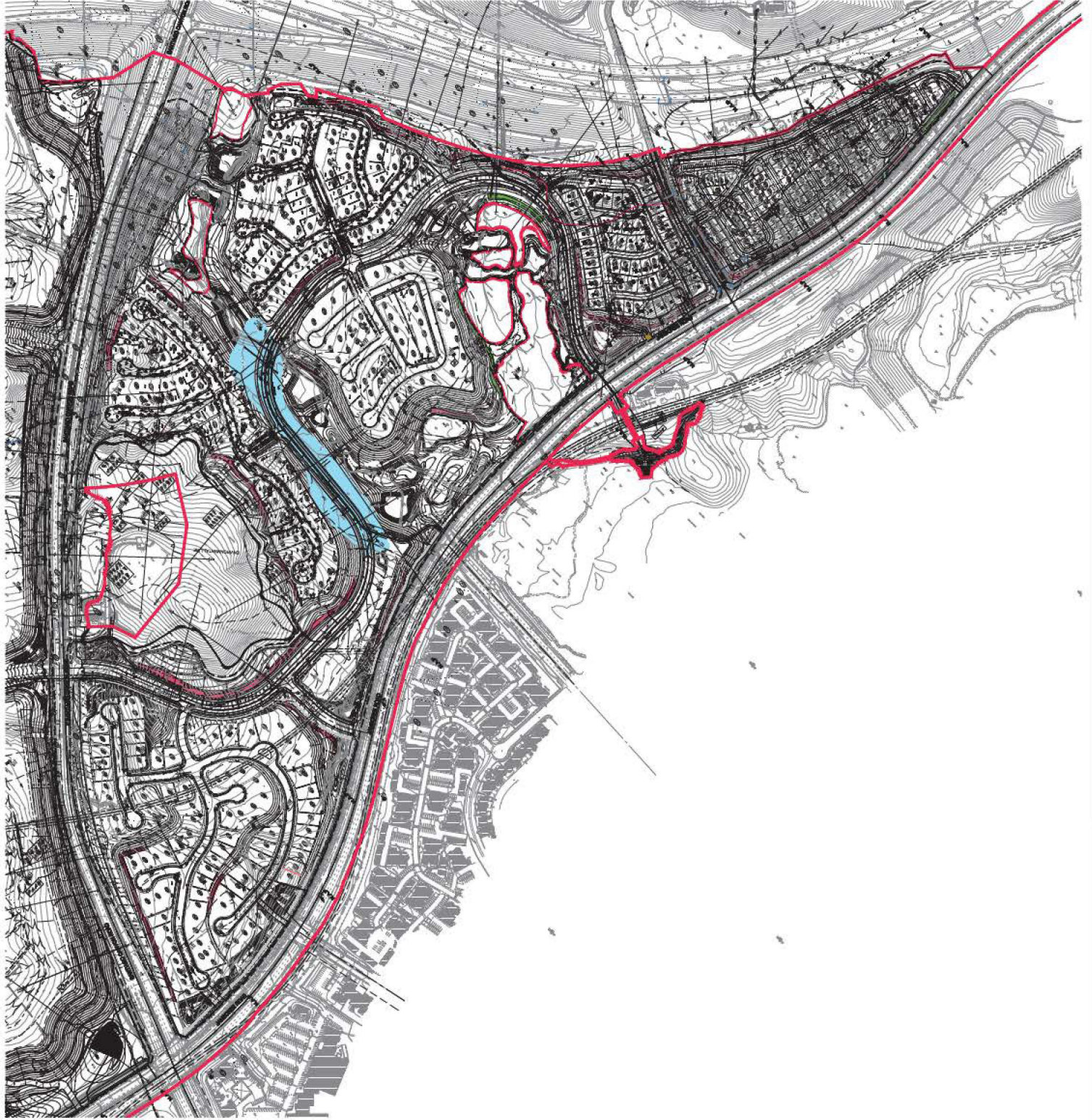
Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
<p>MM 3C-9. Field Observations and Testing Will Be Conducted. Before submission of final grading plans, the developer will retain a qualified geotechnical engineer and engineering geologist. The geologist/engineer will conduct field observations and testing of onsite soils and formations to identify and define the limits of expansive materials. A final report will be prepared and submitted to the City. This report will include identification of:</p> <ul style="list-style-type: none"> • thickness and distribution of the expansive materials, • anticipated depth of moisture variation, • expansiveness of the earth materials, • the tolerance of the structure for displacement, and • confirmation or modification of mitigation measures for expansive materials. 	1. Geotechnical engineer/engineering geologist report.	1. Prior to approval of final grading plans.	1. City Public Works Department	Grading plans were previously approved by the City.
<p>MM 3C-10. Fill Materials Will Be Tested. Upon completion of rough grading and before final grading begins, a qualified geotechnical engineer and/or engineering geologist will:</p> <ul style="list-style-type: none"> • obtain representative samples of near-surface soils and formations exposed in cut areas, • identify expansion potential, and • locate and define the limits of the expansive soils. 	1. Geotechnical engineer/engineering geologist report.	1. Prior to approval of final grading plans.	1. City Public Works Department	

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
<p>MM 3C-11. Expansive Soils Will Be Mitigated. Following identification and delineation of expansive materials, the engineers will identify the most appropriate methods of mitigation. Mitigation measures will include one or more of the following.</p> <ul style="list-style-type: none"> • excavation and replacement with non-expansive fill materials; • moisture conditioning of the expansive material to a high moisture content to cause preswelling, then capping with non-expansive fill; • compliance with the most recent Uniform Building Code, which specifies special foundation/slab design for residential construction on soils having an expansion potential of "low" or greater; • designing foundations to span across areas of potential differential expansion; and • submission of special foundation and slab design by the geotechnical and structural engineers and approval of this design by the building official. 	<ol style="list-style-type: none"> 1. Grading plans with required measures. 2. Building plans with required measures, if necessary. 	<ol style="list-style-type: none"> 1. Prior to approval of tentative tract maps. 2. Prior to issuance of building permits. 	<ol style="list-style-type: none"> 1. City Community Development Director 2. City Building Department 	<p>Grading plans with the identified requirements were previously approved by the City.</p> <p>Note that the Uniform Building Code is now referred to as the California Building Code.</p>

TABLE 1 – CEQA TABLE

Santiago Hills II/Orange Heights Geology/Soils Mitigation Measures				
Mitigation Measures (MM) and Project Design Features (PDF)	Method of Verification	Timing of Verification	Responsible for Verification	Notes
<p>MM 3C-20. Further Slope Stability Investigations. Further slope stability investigations will be conducted pursuant to required future geotechnical studies for the areas of potential slope instability within the proposed limits of development. The level of detail will vary with the local geologic conditions and planned grading. In most cases, a subsurface geologic investigation will be required to evaluate critical lithologic and structural geological interpretations. In general, conclusions pertaining to slope stability in these preliminary studies should be clearly presented and supported by adequate geologic maps, cross sections, and supporting engineering data. Technical review for adequacy of all such reports should be accomplished in accordance with current practices. Reviewing agencies have found that compliance with Chapter 70 of the Uniform Building Code, which regulates earthwork and grading, is important in the mitigation of slope instability during the actual grading phase of development. Should unfavorable slide conditions be encountered, they may be removed during grading or stabilized by means of buttressing or modification of slope orientation, geometry, or inclination.</p>	<p>1. Grading plans with required measures.</p>	<p>1. Prior to approval of tentative tract maps.</p>	<p>1. City Community Development Director</p>	<p>Grading plans with the identified requirements were previously approved by the City.</p>



APPENDIX A

APPENDIX A

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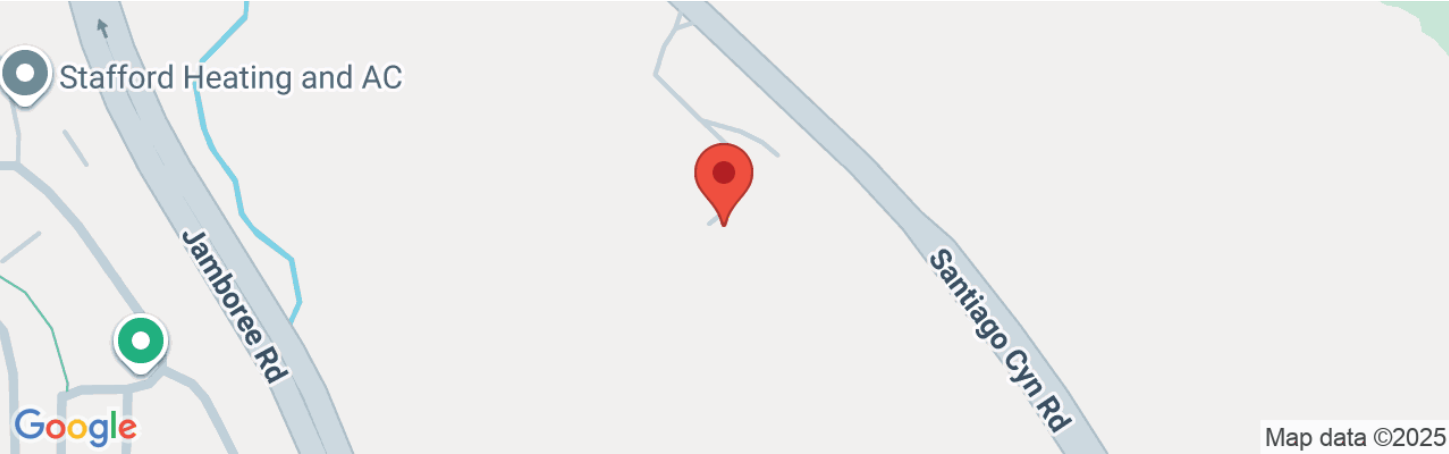
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APPENDIX B

Announcement
ASCE 7-22 is now available.



Latitude, Longitude: 33.7878, -117.7551



Date	10/28/2025, 4:10:52 PM
Design Code Reference Document	ASCE7-22
Risk Category	II
Site Class	Default

Type	Value	Description (Data)
S_S	1.65	The MCE_R spectral response acceleration at 0.2 seconds for Site Class BC, in units of g.
S_1	0.52	The MCE_R spectral response acceleration at 1 second for Site Class BC, in units of g.
S_{MS}	1.88	$S_{MS} = 1.5 \times S_{DS}$, the Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class.
S_{M1}	1.28	$S_{M1} = 1.5 \times S_{D1}$, the MCE_R spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class.
S_{DS}	1.25	The design spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class, in units of g.
S_{D1}	0.86	The design spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class, in units of g

Type	Value	Description (Data Contd.)
SDC	D	Seismic design category
PGA_M	0.72	PGA_M , the Geometric-Mean Maximum Considered Earthquake (MCE_G) peak ground acceleration for the user-specified Site Class, in units of g
T_S	0.684	$T_S = S_{D1}/S_{DS}$, in seconds, for construction of the two-period design spectrum
T_0	0.137	$T_0 = 0.2 \times T_S$, in seconds, for construction of the two-period design response spectrum
T_L	8	T_L , the long-period transition period, in seconds, for construction of the two-period design response spectrum

Type	Value	Description (Underlying Data and Metadata)
PGA_{uh}	See underlying data for Site Classes C, CD, and D	Probabilistic uniform-hazard (2%-in-50-years), geometric-mean peak ground acceleration, in units of g.
PGA_{84th}	See underlying data for Site Classes C, CD, and D	Deterministic 84th-percentile, geometric-mean peak ground acceleration (without deterministic lower limit), in units of g.
V_{S30}	260	The shear-wave velocity used for the user-specified Site Class, in units of m/s
Spatial Interpolation Method	linearloglinear	Identifier for spatial interpolation method used to obtain values for location of interest from underlying gridded values: "linearloglinear" for bilinear of natural logarithm of values.
PGA_{dFloor}		Deterministic lower limit peak ground acceleration (PGA_G) for the user-specified Site Class, in units of g.
riskTargetedSpectrum		Probabilistic risk-targeted, maximum direction response spectrum (for 1%-in-50-years collapse risk)
eightyFourthSpectrum		Deterministic 84 th -percentile, maximum-direction response spectrum (without deterministic lower limit)

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Multi Period

Period T(s)	Design Spectrum Ordinates (g)	MCE _R Spectrum Ordinates (g)
0	0.55	0.83
0.01	0.56	0.84
0.02	0.56	0.84
0.03	0.59	0.88
0.05	0.72	1.09
0.075	0.94	1.4
0.1	1.09	1.64
0.15	1.26	1.88
0.2	1.36	2.04
0.25	1.39	2.08
0.3	1.38	2.07
0.4	1.39	2.08
0.5	1.31	1.97
0.75	1.05	1.57
1	0.86	1.28
1.5	0.59	0.89
2	0.44	0.66
3	0.28	0.42
4	0.2	0.3
5	0.15	0.22
7.5	0.089	0.13
10	0.059	0.088

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Two Period

Period T(s)	Design Spectrum Ordinates (g)	MCE _R Spectrum Ordinates (g)
0	0.5	0.75
0.025	0.64	0.96
0.05	0.77	1.16
0.1	1.05	1.57
0.14	1.25	1.88
0.15	1.25	1.88
0.2	1.25	1.88
0.25	1.25	1.88
0.3	1.25	1.88
0.35	1.25	1.88
0.4	1.25	1.88
0.45	1.25	1.88
0.5	1.25	1.88
0.55	1.25	1.88
0.6	1.25	1.88
0.65	1.25	1.88
0.68	1.25	1.88
0.7	1.22	1.83
0.75	1.14	1.71
0.8	1.07	1.6
0.85	1.01	1.51
0.9	0.95	1.43
0.95	0.9	1.35

Period T(s)	Design Spectrum Ordinates (g)	MCE _R Spectrum Ordinates (g)
1	0.86	1.28
1.05	0.82	1.22
1.1	0.78	1.17
1.15	0.74	1.12
1.2	0.71	1.07
1.25	0.68	1.03
1.3	0.66	0.99
1.35	0.63	0.95
1.4	0.61	0.92
1.45	0.59	0.89
1.5	0.57	0.86
1.55	0.55	0.83
1.6	0.53	0.8
1.65	0.52	0.78
1.7	0.5	0.76
1.75	0.49	0.73
1.8	0.48	0.71
1.85	0.46	0.69
1.9	0.45	0.68
1.95	0.44	0.66
2	0.43	0.64
2.05	0.42	0.63
2.1	0.41	0.61
2.15	0.4	0.6
2.2	0.39	0.58
2.25	0.38	0.57
2.3	0.37	0.56
2.35	0.36	0.55
2.4	0.36	0.53
2.45	0.35	0.52
2.5	0.34	0.51
2.55	0.34	0.5
2.6	0.33	0.49
2.65	0.32	0.48
2.7	0.32	0.48
2.75	0.31	0.47
2.8	0.31	0.46
2.85	0.3	0.45
2.9	0.3	0.44
2.95	0.29	0.44
3	0.29	0.43
3.05	0.28	0.42
3.1	0.28	0.41
3.15	0.27	0.41
3.2	0.27	0.4
3.25	0.26	0.39
3.3	0.26	0.39
3.35	0.26	0.38
3.4	0.25	0.38
3.45	0.25	0.37

Period T(s)	Design Spectrum Ordinates (g)	MCE _R Spectrum Ordinates (g)
3.5	0.24	0.37
3.55	0.24	0.36
3.6	0.24	0.36
3.65	0.23	0.35
3.7	0.23	0.35
3.75	0.23	0.34
3.8	0.23	0.34
3.85	0.22	0.33
3.9	0.22	0.33
3.95	0.22	0.32
4	0.21	0.32
4.05	0.21	0.32
4.1	0.21	0.31
4.15	0.21	0.31
4.2	0.2	0.31
4.25	0.2	0.3
4.3	0.2	0.3
4.35	0.2	0.3
4.4	0.19	0.29
4.45	0.19	0.29
4.5	0.19	0.29
4.55	0.19	0.28
4.6	0.19	0.28
4.65	0.18	0.28
4.7	0.18	0.27
4.75	0.18	0.27
4.8	0.18	0.27
4.85	0.18	0.26
4.9	0.17	0.26
4.95	0.17	0.26
5	0.17	0.26
5.05	0.17	0.25
5.1	0.17	0.25
5.15	0.17	0.25
5.2	0.16	0.25
5.25	0.16	0.24
5.3	0.16	0.24
5.35	0.16	0.24
5.4	0.16	0.24
5.45	0.16	0.24
5.5	0.16	0.23
5.55	0.15	0.23
5.6	0.15	0.23
5.65	0.15	0.23
5.7	0.15	0.23
5.75	0.15	0.22
5.8	0.15	0.22
5.85	0.15	0.22
5.9	0.15	0.22
5.95	0.14	0.22

Period T(s)	Design Spectrum Ordinates (g)	MCE _R Spectrum Ordinates (g)
6	0.14	0.21
6.05	0.14	0.21
6.1	0.14	0.21
6.15	0.14	0.21
6.2	0.14	0.21
6.25	0.14	0.21
6.3	0.14	0.2
6.35	0.13	0.2
6.4	0.13	0.2
6.45	0.13	0.2
6.5	0.13	0.2
6.55	0.13	0.2
6.6	0.13	0.19
6.65	0.13	0.19
6.7	0.13	0.19
6.75	0.13	0.19
6.8	0.13	0.19
6.85	0.12	0.19
6.9	0.12	0.19
6.95	0.12	0.18
7	0.12	0.18
7.05	0.12	0.18
7.1	0.12	0.18
7.15	0.12	0.18
7.2	0.12	0.18
7.25	0.12	0.18
7.3	0.12	0.18
7.35	0.12	0.17
7.4	0.12	0.17
7.45	0.11	0.17
7.5	0.11	0.17
7.55	0.11	0.17
7.6	0.11	0.17
7.65	0.11	0.17
7.7	0.11	0.17
7.75	0.11	0.17
7.8	0.11	0.16
7.85	0.11	0.16
7.9	0.11	0.16
7.95	0.11	0.16
8	0.11	0.16
8.05	0.11	0.16
8.1	0.1	0.16
8.15	0.1	0.15
8.2	0.1	0.15
8.25	0.1	0.15
8.3	0.099	0.15
8.35	0.098	0.15
8.4	0.097	0.15
8.45	0.096	0.14

Period T(s)	Design Spectrum Ordinates (g)	MCE _R Spectrum Ordinates (g)
8.5	0.095	0.14
8.55	0.094	0.14
8.6	0.093	0.14
8.65	0.091	0.14
8.7	0.09	0.14
8.75	0.089	0.13
8.8	0.088	0.13
8.85	0.087	0.13
8.9	0.086	0.13
8.95	0.085	0.13
9	0.085	0.13

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Probabilistic & Deterministic

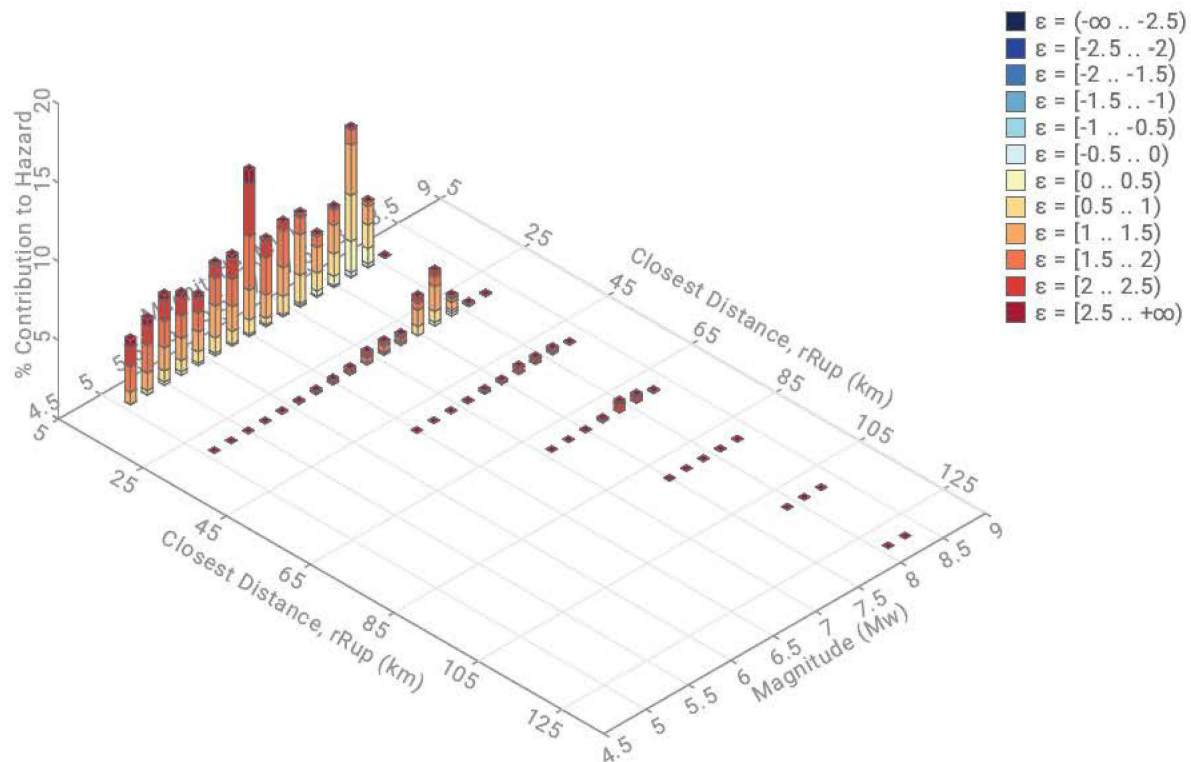
Period T(s)	Risk Targeted Spectrum Ordinates (g)	84 th Spectrum Ordinates(g)
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DISCLAIMER

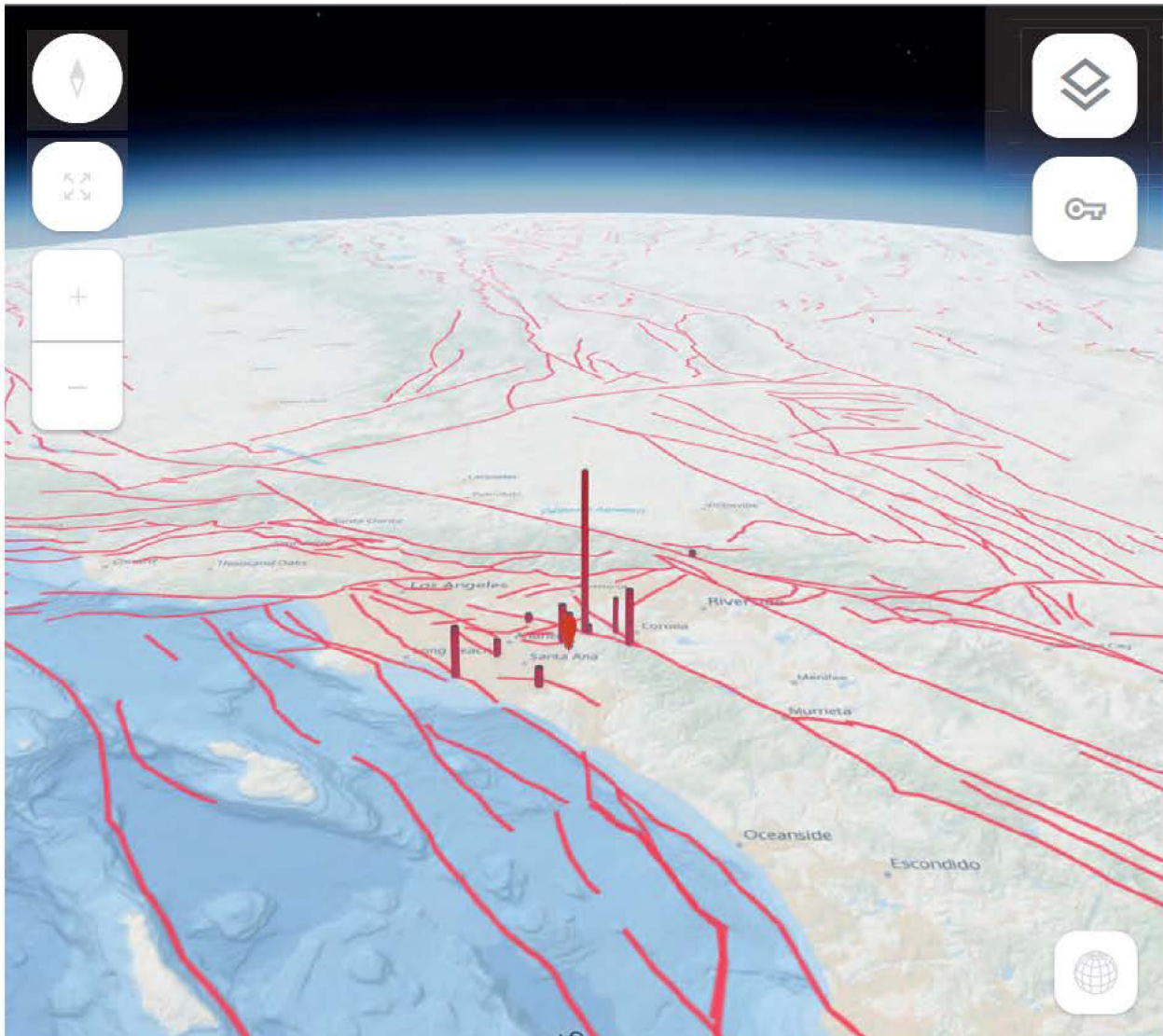
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Disaggregation Report

Disaggregation



Geographical Disaggregation



Esri, GEBCO, Garmin, NaturalVue County of Los Angeles, California State Parks, Esri... Powered by Esri

Parameter Summary

Model: NSHM Conterminous U.S. 2023	Intensity Measure Type PGA
Latitude: 33.788 °	Return Period: 2475 (2% in 50)
Longitude: -117.755 °	Component Total
Site Class: CD (Vs30 365)	

Disaggregation Summary: Total

Disaggregation targets

Return period : 2475 yrs
Exceedance rate : 4.040e-4 yr⁻¹
PGA ground motion : 7.532e-1 g

Totals

Binned : 100 %
Residual : 0 %
Trace : 0.12 %

Mode (largest m-r bin)

m : 6.5
r : 12.73 km
ε₀ : 1.78 σ
Contribution : 10.51 %

Discretization

r : min = 0.0, max = 1000.0, Δ = 20.0 km
m : min = 4.4, max = 9.4, Δ = 0.2
ε : min = -3.0, max = 3.0, Δ = 0.5 σ

Recovered targets

Return period : 2403.1213 yrs
Exceedance rate : 4.161e-4 yr⁻¹

Mean (over all sources)

m : 6.69
r : 13.65 km
ε₀ : 1.47 σ

Mode (largest m-r-ε₀ bin)

m : 6.5
r : 13.51 km
ε₀ : 1.77 σ
Contribution : 3.4 %

Epsilon keys

ε0 : [-∞ .. -2.5)
ε1 : [-2.5 .. -2.0)
ε2 : [-2.0 .. -1.5)
ε3 : [-1.5 .. -1.0)
ε4 : [-1.0 .. -0.5)
ε5 : [-0.5 .. 0.0)
ε6 : [0.0 .. 0.5)
ε7 : [0.5 .. 1.0)
ε8 : [1.0 .. 1.5)
ε9 : [1.5 .. 2.0)
ε10 : [2.0 .. 2.5)
ε11 : [2.5 .. +∞]



Disaggregation Contributions: Total

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
WUS Branch Average		FaultSystem							59.4
	Whittier (2)		10.65	7.28	1.20	117.712°W	33.876°N	21.86	17.94
	Elsinore (Glen Ivy) (0)		15.96	6.60	2.09	117.590°W	33.829°N	73.34	7.46
	Compton (0)		21.63	7.57	1.17	118.043°W	33.703°N	250.47	6.48
	Peralta Hills (0)		6.48	7.37	0.73	117.772°W	33.843°N	345.64	5.86
	Chino (2)		14.45	7.13	1.33	117.628°W	33.876°N	50.09	5.02
	San Joaquin Hills (1)		15.55	7.37	1.41	117.826°W	33.664°N	205.72	3.02
	Anaheim (0)		18.03	7.47	1.08	117.943°W	33.780°N	267.18	2.6
	Puente Hills (Coyote Hills) (0)		18.41	7.21	1.63	117.868°W	33.919°N	324.46	1.32
	Whittier (1)		10.75	6.71	1.57	117.702°W	33.874°N	27.03	1.19
	San Andreas (San Bernardino, north) (2)		60.25	7.98	2.38	117.385°W	34.234°N	34.42	1.09
WUS Branch Average (opt)		Grid							40.6
	PointSourceFinite: -117.755, 33.855		8.13	5.78	1.52	117.755°W	33.855°N	0.00	1.95
	PointSourceFinite: -117.755, 33.864		8.83	5.81	1.60	117.755°W	33.864°N	0.00	1.93
	PointSourceFinite: -117.755, 33.828		6.19	5.73	1.25	117.755°W	33.828°N	0.00	1.88
	PointSourceFinite: -117.755, 33.819		5.70	5.70	1.18	117.755°W	33.819°N	0.00	1.73
	PointSourceFinite: -117.755, 33.873		9.54	5.84	1.67	117.755°W	33.873°N	0.00	1.71
	PointSourceFinite: -117.755, 33.855		9.17	5.79	1.56	117.755°W	33.855°N	0.00	1.66
	PointSourceFinite: -117.755, 33.828		7.18	5.75	1.29	117.755°W	33.828°N	0.00	1.64
	PointSourceFinite: -117.755, 33.864		9.88	5.82	1.64	117.755°W	33.864°N	0.00	1.63
	PointSourceFinite: -117.755, 33.846		7.28	5.82	1.39	117.755°W	33.846°N	0.00	1.59
	PointSourceFinite: -117.755, 33.837		6.71	5.78	1.32	117.755°W	33.837°N	0.00	1.56
	PointSourceFinite: -117.755, 33.819		6.59	5.71	1.21	117.755°W	33.819°N	0.00	1.54
	PointSourceFinite: -117.755, 33.873		10.60	5.85	1.71	117.755°W	33.873°N	0.00	1.44
	PointSourceFinite: -117.755, 33.810		5.27	5.67	1.10	117.755°W	33.810°N	0.00	1.38
	PointSourceFinite: -117.755, 33.846		8.42	5.84	1.43	117.755°W	33.846°N	0.00	1.37
	PointSourceFinite: -117.755, 33.837		7.79	5.80	1.36	117.755°W	33.837°N	0.00	1.35
	PointSourceFinite: -117.755, 33.882		10.09	5.92	1.80	117.755°W	33.882°N	0.00	1.31
	PointSourceFinite: -117.755, 33.810		6.03	5.68	1.13	117.755°W	33.810°N	0.00	1.26
	PointSourceFinite: -117.755, 33.882		11.22	5.93	1.83	117.755°W	33.882°N	0.00	1.09
	PointSourceFinite: -117.755, 33.891		10.86	5.94	1.87	117.755°W	33.891°N	0.00	1.06



Application Metadata

Application: Disaggregation

URL: <https://earthquake.usgs.gov/nshmp/hazard/disagg>

Repository: nshmp-apps

Version: 20.8.0

URL: <https://code.usgs.gov/ghsc/nshmp/nshmp-apps>

Repository : nshmp-haz

Version: 2.7.0

URL: <https://code.usgs.gov/ghsc/nshmp/nshmp-haz>

Repository : nshmp-lib

Version: 1.7.15

URL: <https://code.usgs.gov/ghsc/nshmp/nshmp-lib>

Repository : nshm-conus

Version: 6.1.1

URL: <https://code.usgs.gov/ghsc/nshmp/nshms/nshm-conus>

November 04, 2025, 09:50 AM

APPENDIX C



**SEISMIC REFRACTION SURVEY
TRACTS 16199 & 16201 AND BAKER PIPELINE
COUNTY OF ORANGE, CALIFORNIA**

Project No. 262030-1

January 24, 2006

Prepared for:

LGC, Inc.
1319 Calle Avanzado
San Clemente, CA 92673-6351

LGC, Inc.
1319 Calle Avanzado
San Clemente, CA 92673-6351

Attention: Mr. Kevin B. Colson

Regarding: Seismic Refraction Survey
Tracts 16199 & 16201 and Baker Pipeline
County of Orange, California
LGC Job Number 041151-01

INTRODUCTION

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site along selected areas as delineated by you. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to aid in evaluating whether high velocity earth materials (non-rippable) are present along local areas which could possibly indicate areas of potential excavation difficulties. Numerous bedrock materials have been locally mapped within our study area by Tan (1995), to include Pleistocene age terrace deposits consisting of semi-consolidated sand with high clay and silt content and locally high gravel content; the middle Miocene age El Modeno Volcanics consisting of interbedded andesite and basalt flows, tuff and breccia; and sedimentary bedrock of the lower Miocene age Vaqueros and Sespe Formations consisting of undifferentiated sandstone, claystone, siltstone, and conglomerate.

Representative Layer Velocity Profiles for each seismic line have been prepared which indicates their respective "weighted average" subsurface velocities in generalized layers. In addition, associated Tomographic Models have also been prepared for comparative purposes, which generally indicate the relative structure and velocity distribution for each seismic line. These comparative survey line images are presented within Appendix A. We understand that this report will be included as a technical appendix to your report, therefore as requested, the locations of our geophysical survey lines were transferred onto your field map for inclusion onto your final map. As authorized by you, the following services were performed during this study:

- Review of available published and unpublished geologic/geophysical data in our files pertinent to the site,
- Performing a seismic refraction survey by a State of California Registered Geophysicist, to include ten traverses along selected portions of the subject property.
- Preparation of this report, presenting our findings and conclusions with respect to the velocity characteristics and the expected rippability potentials of the subsurface earth materials.

Accompanying Appendices

Appendix A - Seismic Survey Lines
Appendix B - Excavation Considerations
Appendix C - References

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Ten seismic refraction survey lines were performed, varying from 195 to 300 feet in length, with a target depth ranging between 60- to 100±-feet. A 16-pound sledgehammer was used as an energy source to produce the seismic waves and both 12 or 24, 14-Hz geophones (with 70% damping), were spaced at 10- to 15-foot intervals along the traverse lines (depending on target depth) to detect both the direct and refracted waves. The seismic wave arrivals were recorded on a 24-channel Geometrics SmartSeis® SE model signal enhancement refraction seismograph. Seven shot points were utilized along each seismic line spread using offset, forward, reverse, and intermediate locations, in order to obtain sufficient data for velocity analysis and depth modeling purposes. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrival, of which are digitally recorded on the in-board seismograph computer and subsequently transferred to a disk and downloaded into our office computer for further processing, analyzing, and printing purposes.

Data Reduction

The data on the paper record and/or display screen were used to analyze the arrival time of the seismic waves at each geophone station, in the form of a wiggle trace, or wave travel-time curve, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes, using the computer programs **SIP** (Seismic refraction Interpretation Program) developed by Rimrock Geophysics, Inc. (1995), and **Rayfract™** (Intelligent Resources, Inc., 1996-2005). **SIP** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). In addition, **Rayfract™** was also used for comparative purposes. **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin our discussion, it should be understood that the velocity data obtained during this survey represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders/dikes or local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics.

It is also important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (bedding, fracturing, etc.). It is assumed that for the most part the sedimentary bedrock materials located within the subject property are well-bedded. Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time, as the structure of the bedrock locally is not known.

In general, the predominance of the site, where locally surveyed, was noted to be characterized by three major subsurface layers with respect to seismic velocities. The following velocity layer summaries have been prepared using the **SIP** analysis, as presented in Appendix A on the Layer Velocity Profiles.

□ **Velocity Layer V1:**

This uppermost velocity layer (V1) is most likely comprised of topsoil, colluvium, and/or very highly-weathered or intensely fractured volcanic and/or sedimentary bedrock materials. This layer has an average weighted velocity ranging from 1,185 to 1,576 fps, which is typical for these types of surficially-mantling materials.

□ **Velocity Layer V2:**

The second velocity layer (V2) yielded a range of 2,058 to 3,438 fps, indicating moderate degrees of induration, weathering, and/or fracturing of the underlying sedimentary bedrock, volcanics, and/or terrace deposits. These velocities are also typical for the near-surface weathered zone commonly found in both sedimentary and igneous rocks.

□ **Velocity Layer V3:**

The third layer (V3) indicates less-weathered volcanic bedrock or more lithified sedimentary bedrock materials, with average weighted velocities of 4,382 to 7,626 fps. This wide range of seismic velocities most likely indicates the variability of earth materials such as surficially mapped by Tan (1995) and as locally exposed across the site. At this time, it is assumed that the V3 velocities obtained from Seismic Lines S-1 through S-5 (4,382 to 5,963 fps) are most likely well-lithified sedimentary bedrock materials, while the relatively higher V3 velocities within Seismic Lines S-6 through S-10 (7,140 to 7,626 fps) most likely represent less-weathered volcanic rock at depth.

The following table summarizes the results of the survey lines with respect to the "weighted average" seismic velocities for each layer and their approximate average depths, based on the Layer Velocity Profiles presented in Appendix A.

TABLE 1: SUMMARY OF SEISMIC SURVEY LINES

<u>Seismic Line No.</u>	<u>Average Weighted Velocity (ft/sec)</u>	<u>Approximate Average Depth Range (feet) *</u>
S-1	1,249	0-5
	2,390	5-60
	4,382	60-100±**
S-2	1,185	0-8
	2,581	8-40
	4,608	40-75±**
S-3	1,240	0-7
	2,307	7-45
	4,871	45-70±**
S-4	1,192	0-18
	2,931	18-50
	5,158	50-65±**
S-5	1,297	0-8
	2,548	8-45
	5,963	45-65±**
S-6	1,576	0-8
	3,438	8-60
	7,552	60-70±**
S-7	1,293	0-5
	3,008	5-50
	7,626	50-65±**
S-8	1,219	0-7
	2,058	7-40
	7,469	40-100±**
S-9	1,250	0-8
	2,144	8-35
	7,140	35-100±**
S-10	1,366	0-7
	2,691	7-40
	7,239	40-100±**

NOTES: * See Layer Velocity Profiles in Appendix A for layer depth variability.

** Minimum anticipated depth of V3 layer based on length of survey line.

It was observed that some of the raw arrival times of the primary “first-arrival” seismic waves were generally curvilinear in shape, indicating that the velocity of the subsurface earth materials most likely increases gradually with depth rather than creating a sharp and clear boundary, as discussed further below. In order to create a subsurface geometric model for the **SIP** analysis, these curvilinear lines must be approximated as linear features, and are somewhat subjective. The contact boundaries, as depicted on the Layer Velocity Profiles are approximate as they should generally be considered to be gradational in nature.

Using **Rayfract™**, a tomographic model for each seismic line was also prepared and analyzed for comparative purposes, as also presented in Appendix A, which generally indicates the relative structure and velocity distribution. Although no discrete velocity layers or boundaries are created, these models generally resemble the **SIP** analysis. **Rayfract™** allows imaging of subsurface velocity using first break energy propagation modeling. An initial 1D gradient model is created using the Delta-t-V method which gives a good initial fit between modeled and picked first breaks. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion. WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break.

It can be seen in these models that the seismic velocity (i.e., hardness) of the bedrock gradually increases with depth which is most likely the representative condition of the subsurface materials. It was also noted that for the most part, the seismic velocities on the Layer Velocity Profiles appears to generally correlate with the average of the velocity gradients as shown on the Tomographic Models as presented on the Seismic Survey Line images presented within Appendix A.

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

Due to the highly variable earth materials that are present within the subject property, excavations characteristics could vary significantly across the site even for similar seismic velocity ranges. Although not known to be present at the site, the following summary is generally considered for granitic-type bedrock and has been slightly modified for this project in an effort to help aid in evaluating potential excavation difficulties with respect to the seismic velocities. The velocity ranges described below are approximate and assume typical, good-working, heavy excavation equipment, such as single shank or D9R dozer, such as described by Caterpillar, Inc. (2000 and 2004); however, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges. Caterpillar indicates that the non-rippable velocity limits for sedimentary and volcanic rocks are somewhat higher than for granitic bedrock and therefore the discussion below should be considered at least a minimum.

For reference purposes, a summary of excavation considerations has been included within Appendix B in order to provide the client and contractor with a better understanding of the complexities of excavation commonly found within bedrock materials. Although these conditions may not exist at the site, this information may still be of some use since there are some relatively high seismic velocities that were encountered locally.

□ **Rippable Condition (0 - 4,000 ft/sec):**

This velocity range indicates rippable materials which may consist of alluvial-type deposits, sedimentary bedrock, and decomposed igneous and metamorphic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

□ **Marginally Rippable Condition (4,000 - 8,000 ft/sec):**

This range of velocities indicates materials which may consist of moderately weathered bedrock or large areas of fresh bedrock materials separated by weathered fractured zones. These materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

□ **Non-Rippable Condition (8,000 ft/sec or greater):**

This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of fair to good quality due to the introduction of significant "noise" that was locally encountered during our field survey originating from vehicular traffic along the nearby roads, including the 241 Freeway and Santiago Canyon and Jamboree Roads. Analysis of the data and picking of the primary "P"-wave arrivals was performed with some difficulty and some interpolation of the data was necessary. The anticipated maximum depth of velocity information appears to be on the order of 65- to 100±-feet in accordance with the depths you specified respectively for each seismic line.

Based on the results of our comparative seismic analyses of both **SIP** and **Rayfract™**, the seismic refraction survey lines appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data. For this reason, both seismic model types for each survey line have been included on the same sheet for comparative purposes as presented within Appendix A.

The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

□ **Velocity Layer V1:**

No major excavating difficulties are expected within the uppermost, low-velocity layer V1 (velocity range of 1,185 to 1,576 fps). This layer is expected to be comprised of topsoil, colluvium, and/or very highly-weathered or intensely fractured volcanic and/or sedimentary bedrock materials. However, some fresher boulders should be anticipated to be encountered based on surface exposures of bedrock, locally.

□ **Velocity Layer V2:**

The second velocity layer V2, is believed to be comprised of moderate degrees of induration, weathering, and/or fracturing of the underlying sedimentary bedrock, volcanics, and/or terrace deposits (velocity range of 2,058 to 3,438 fps), and should excavate with slight to moderate difficulty, assuming appropriate good-working equipment for the proposed type of excavation. Isolated floaters (i.e., boulders, corestones, high velocity bedding layers, etc.) are presumably present within this weathered layer and could produce difficult conditions locally. Placement of infrastructures in this material may also be difficult. Localized blasting cannot be completely ruled out, due to the presence of any localized fresh bedrock and/or buried boulders and dikes, although this condition appears very unlikely.

□ **Velocity Layer V3:**

Excavation difficulties within the lower V3 velocity layer (velocity range of 4,382 to 7,626 fps) are anticipated. Solely based on surficial geologic mapping and the seismic velocities obtained, Seismic Lines S-1 through S-5 are most likely well-lithified sedimentary bedrock materials (4,382 to 5,963 fps) and are expected to excavate with varying degrees of difficulty based on the lithologic composition, bedding and fracture orientations, and degree of induration. Minor blasting could be anticipated in areas of well-lithified massive materials with wide spaced fracturing that approach the higher end of the velocity range.

Seismic Lines S-6 through S-10 (7,140 to 7,626 fps) most likely represent less-weathered volcanic rock at depth also based upon geologic mapping and the very narrow range of higher seismic velocities obtained suggesting similar materials. Since the nature and composition of the volcanic rock is not locally known, hard excavating areas consisting of boulders, dikes, and/or relatively fresher intact rock matrix with wide-spaced fracturing could be encountered during both remedial grading and placement of infrastructures, which may require minor to significant blasting to achieve desired grade.

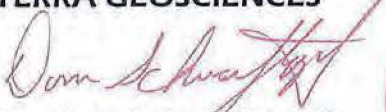
Based on the Tomographic Models and typical excavation characteristics observed within bedrock materials of the southern California region, anticipation of gradual increasing hardness with depth and lateral variations, with respect to excavation characteristics, should be anticipated across the site. The Excavation Considerations included within Appendix B, should be understood so that proper planning and excavation techniques can be employed by the selected grading contractor.

CLOSURE

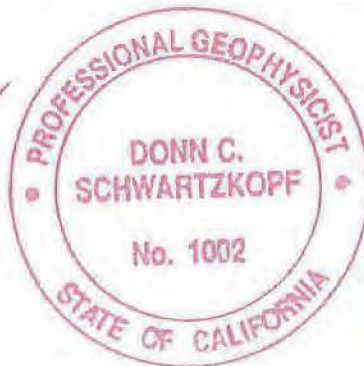
This survey was performed using "state of the art" geophysical techniques, computer processing, and equipment, in the localized areas delineated by you. We make no warranty, either expressed or implied. It should be noted that our data was obtained along only ten specific areas; therefore, other local areas at the site beyond the limits of our seismic lines may contain different velocity layers and depths not encountered during our field survey. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. In summary, the results of this survey are to be considered as an aid to assessing the rippability potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative "test" excavations should be considered, so that they may be correlated with the data presented within this report.

If you should have any questions regarding this report or do not understand the limitations of this survey, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

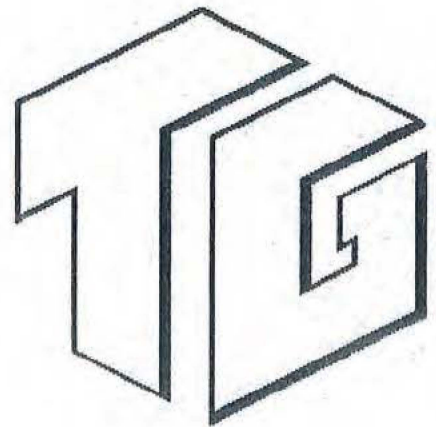


Donn C. Schwartzkopf
Principal Geophysicist
GP 1002



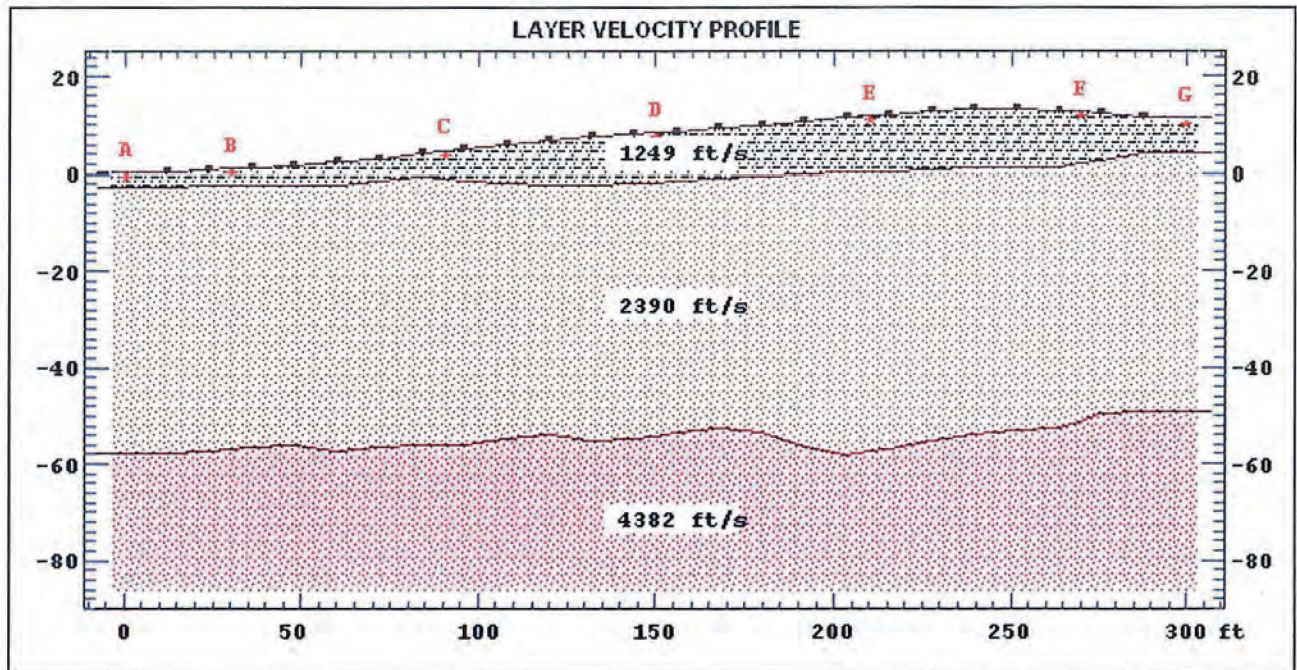
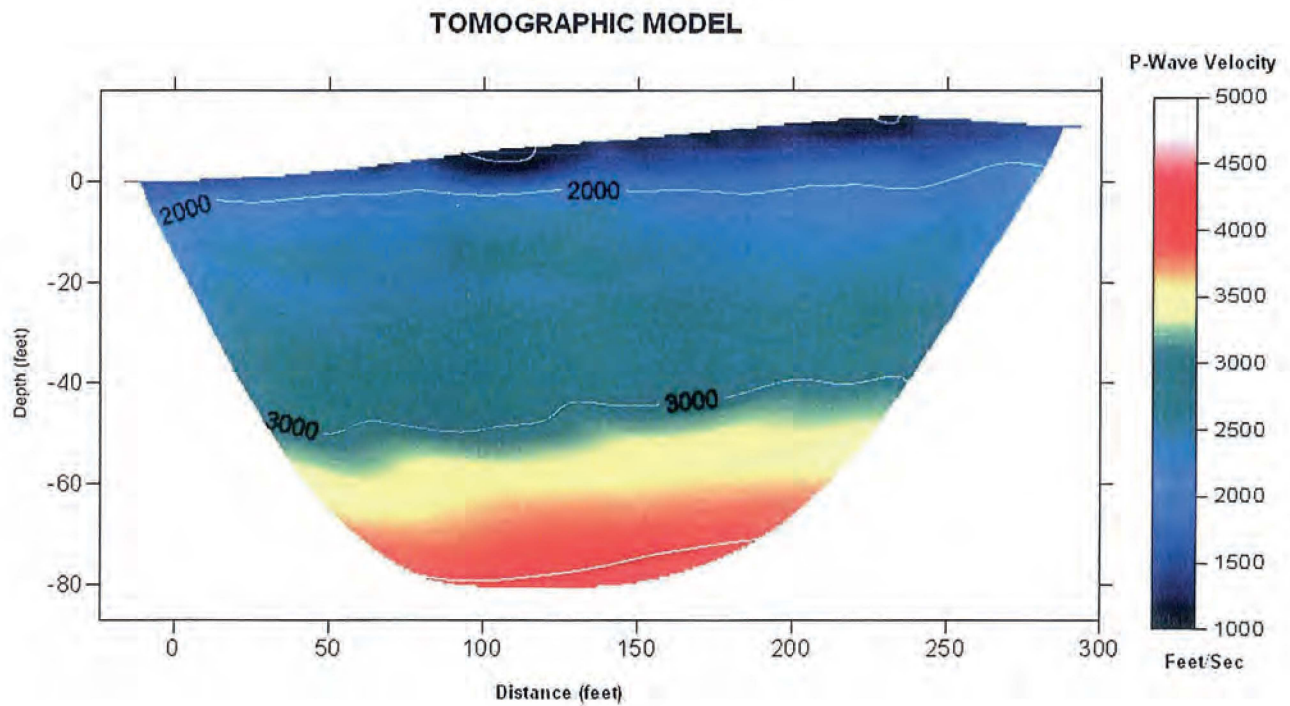
APPENDIX A

SEISMIC SURVEY LINES



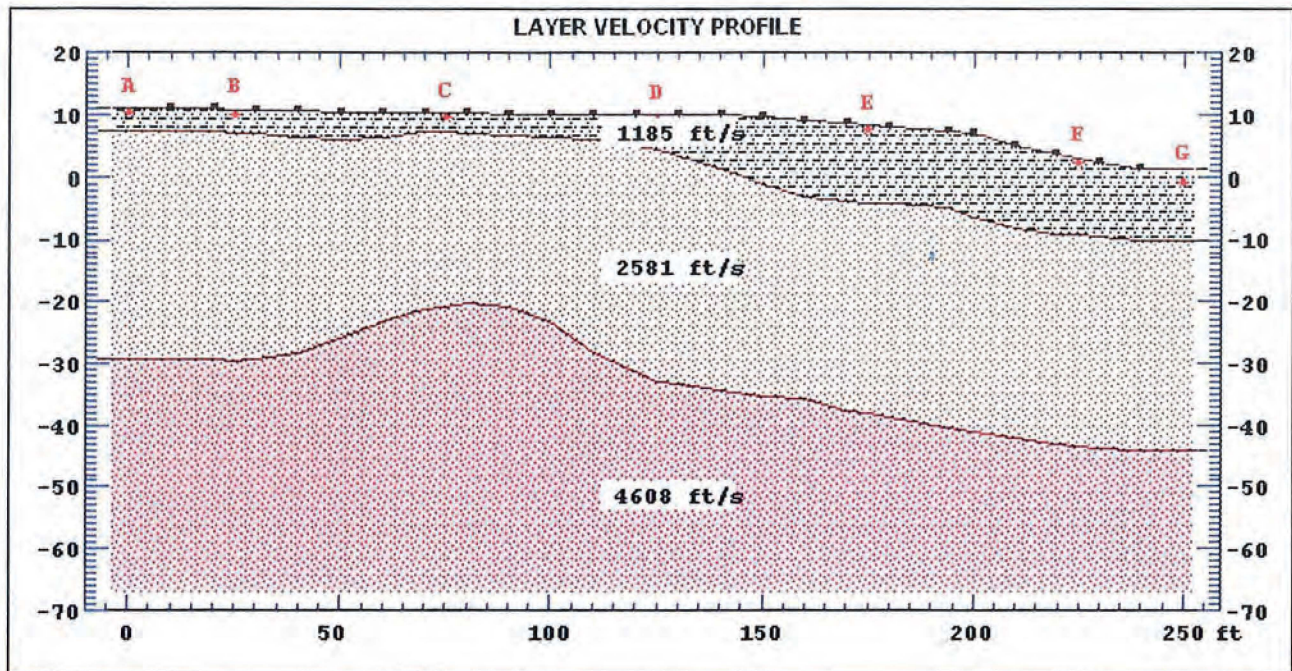
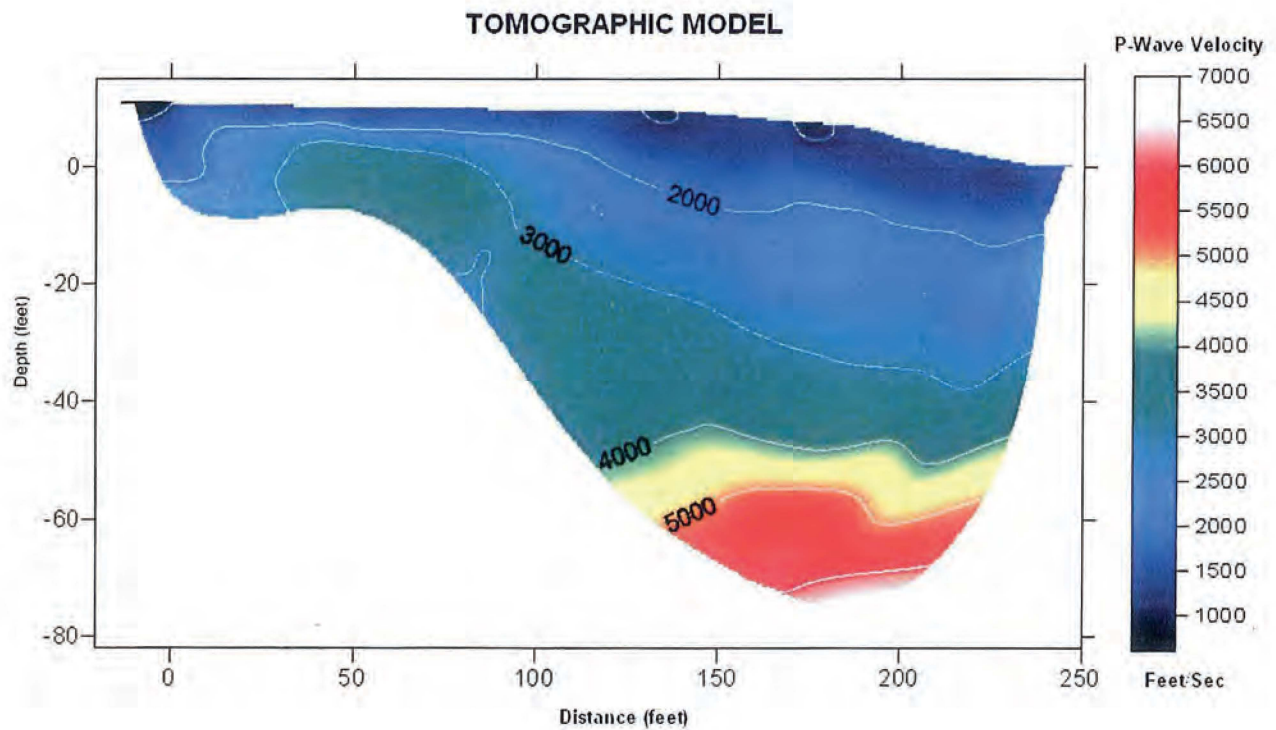
SEISMIC LINE S-1

North 10° West →



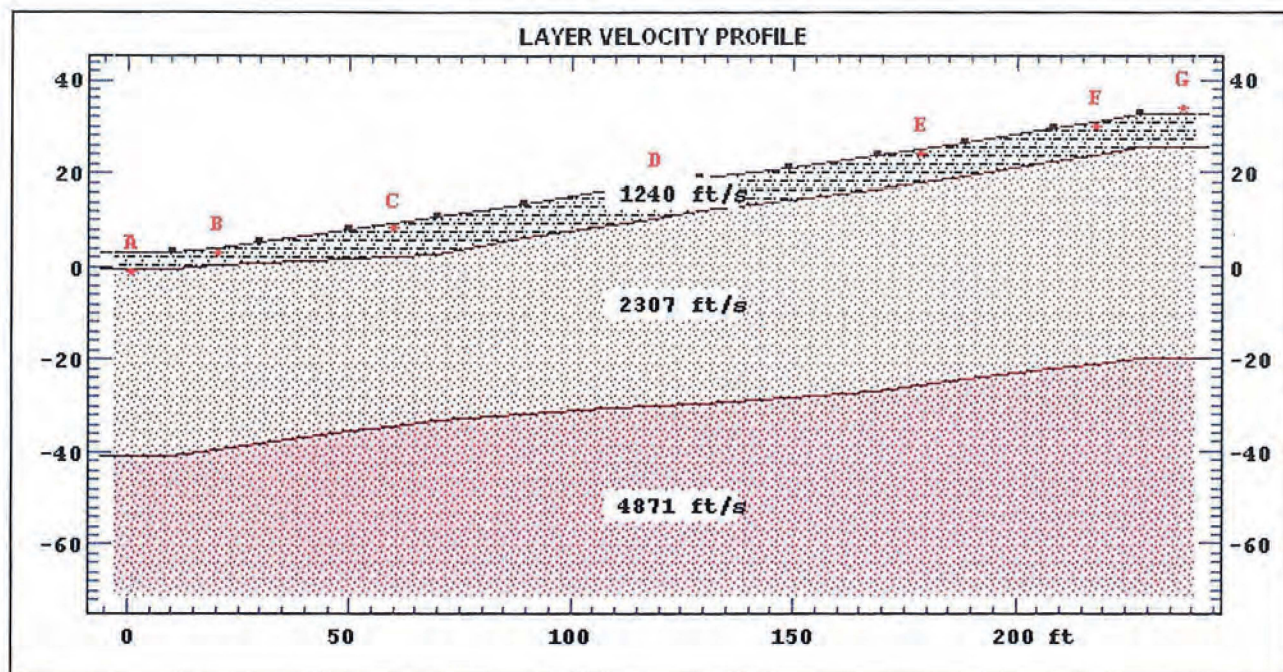
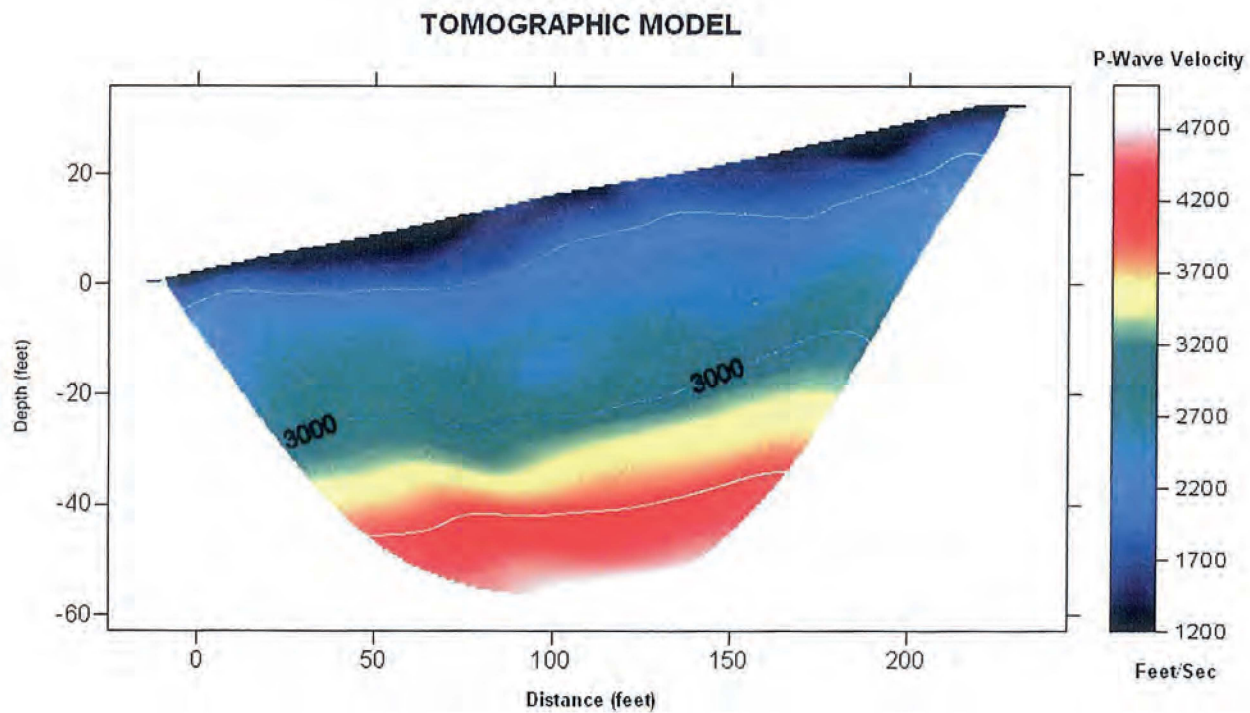
SEISMIC LINE S-2

North 85° East →



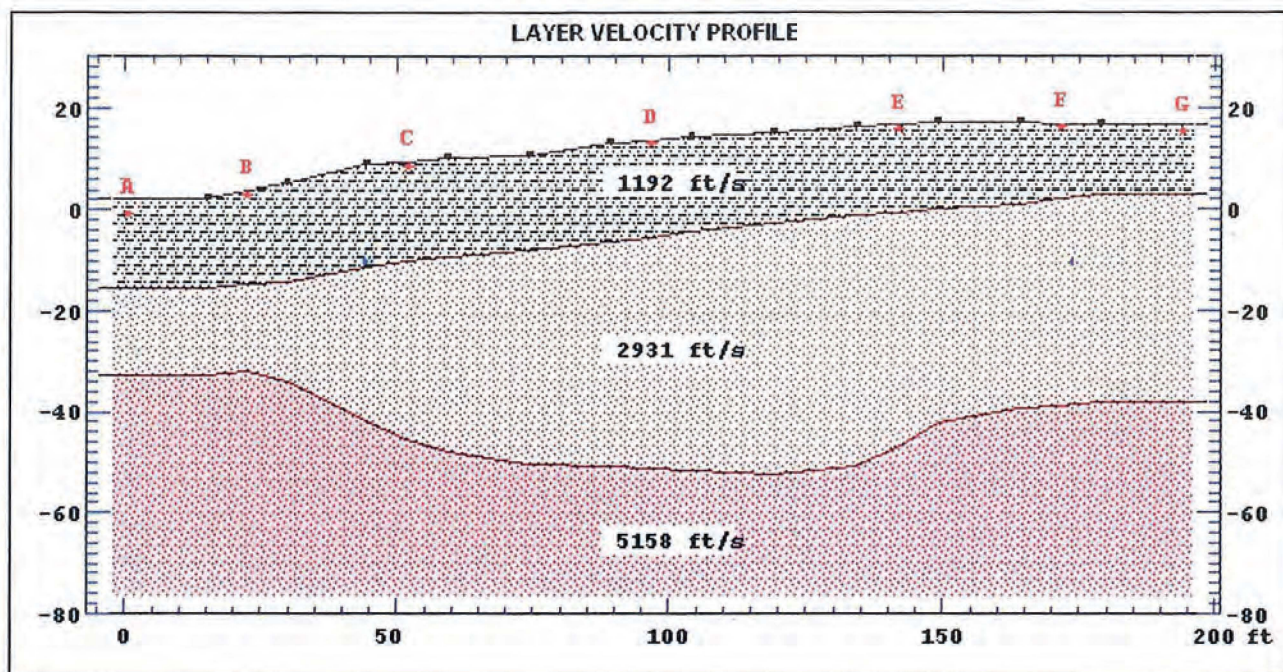
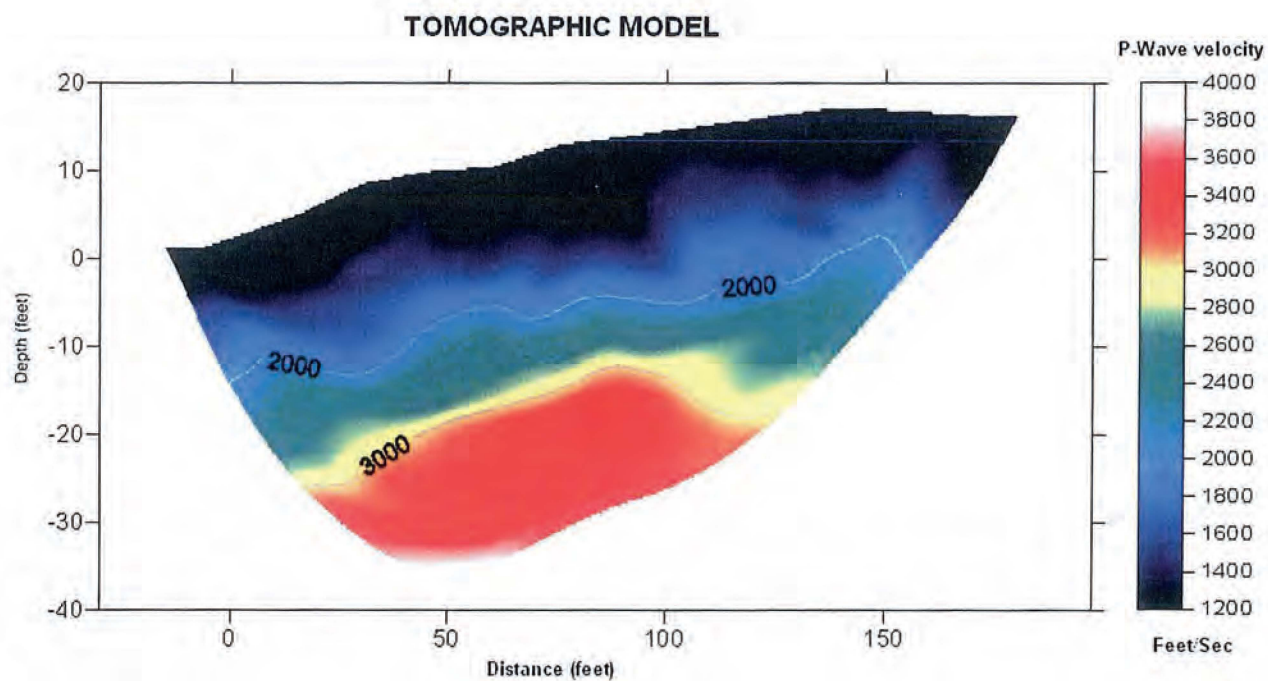
SEISMIC LINE S-3

South 15° West →



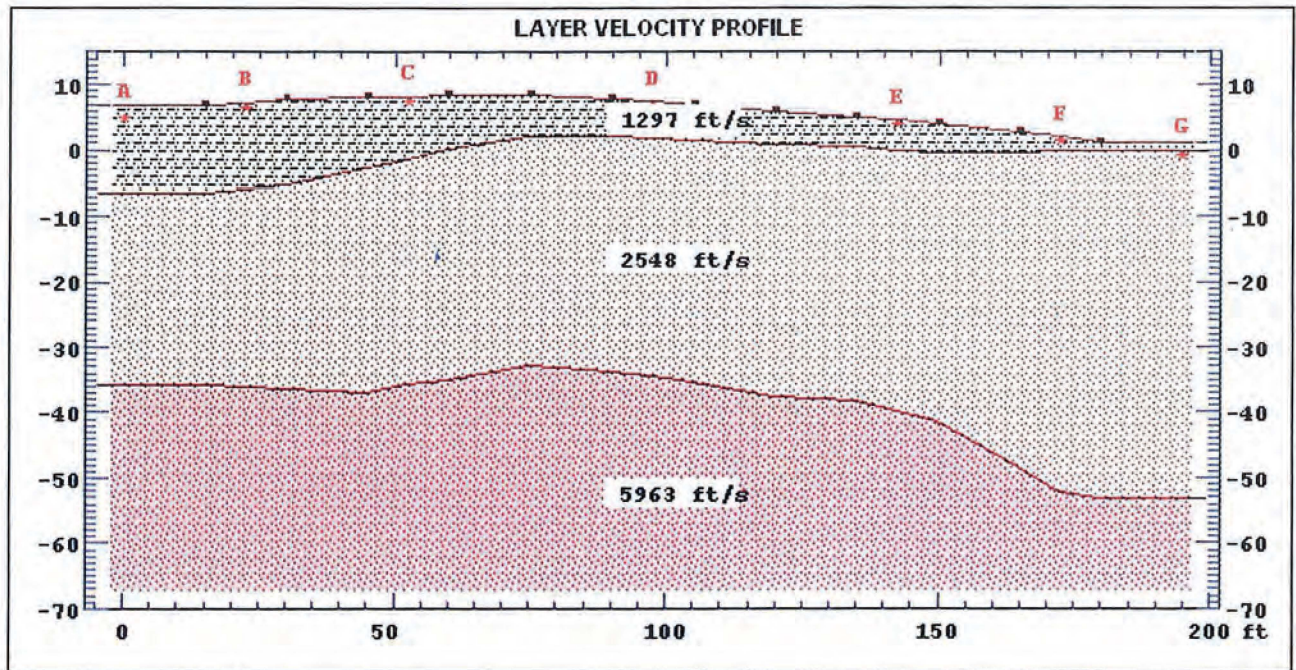
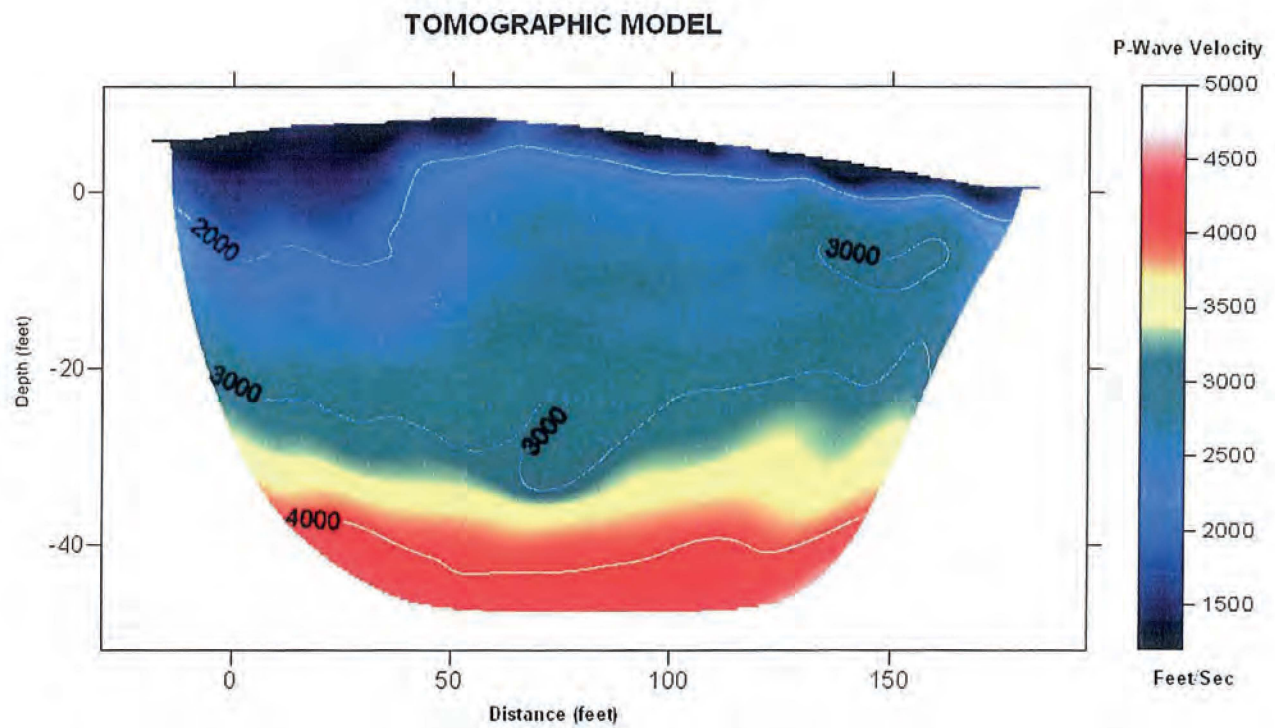
SEISMIC LINE S-4

North 50° West →



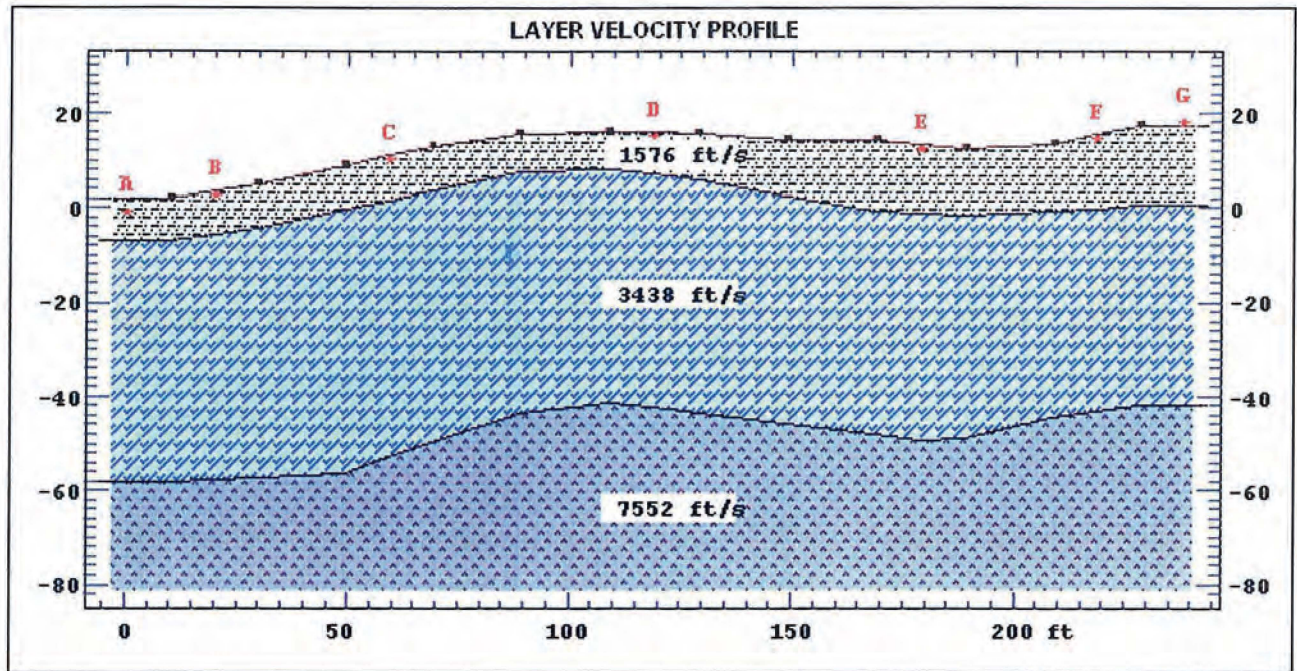
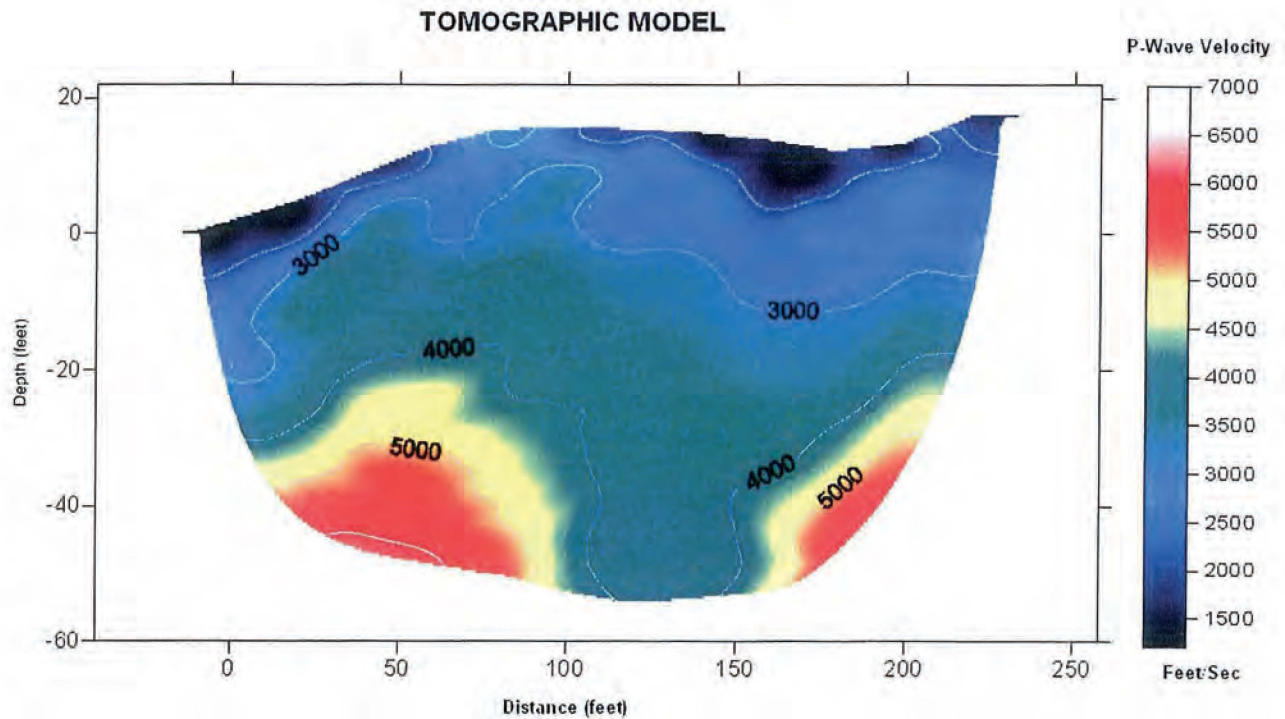
SEISMIC LINE S-5

North 65° East →



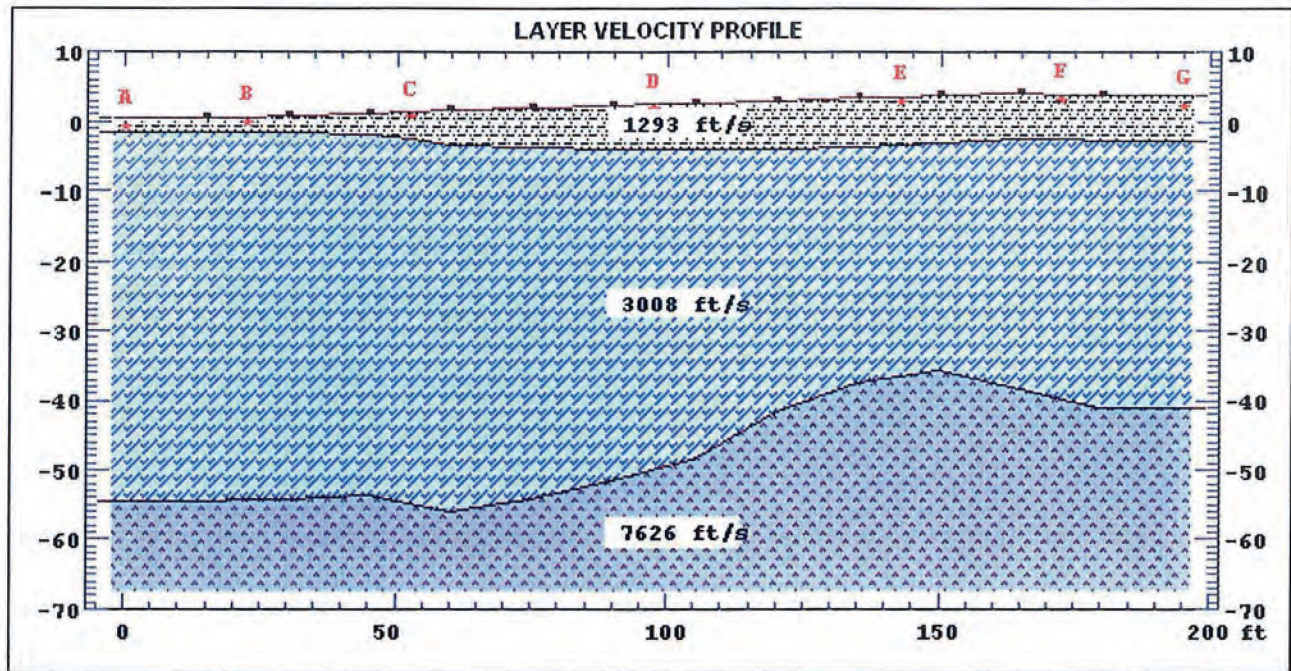
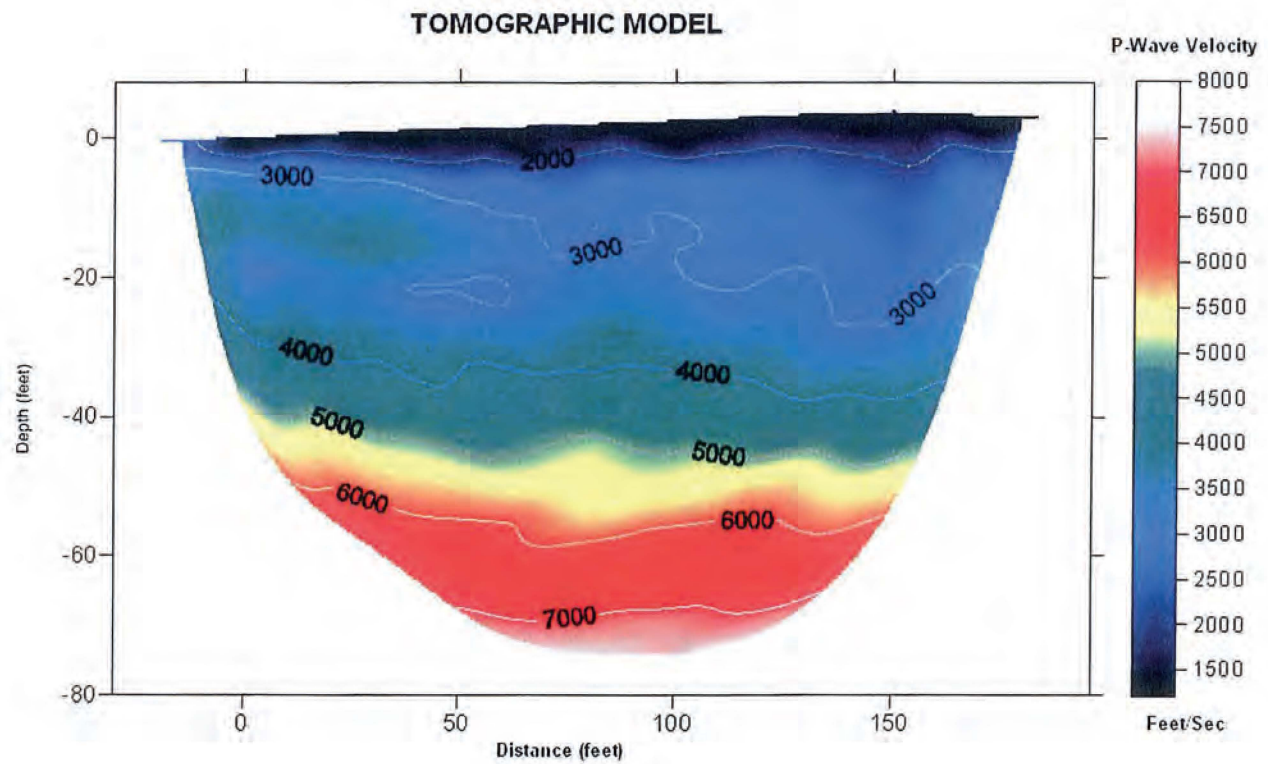
SEISMIC LINE S-6

South 15° East →



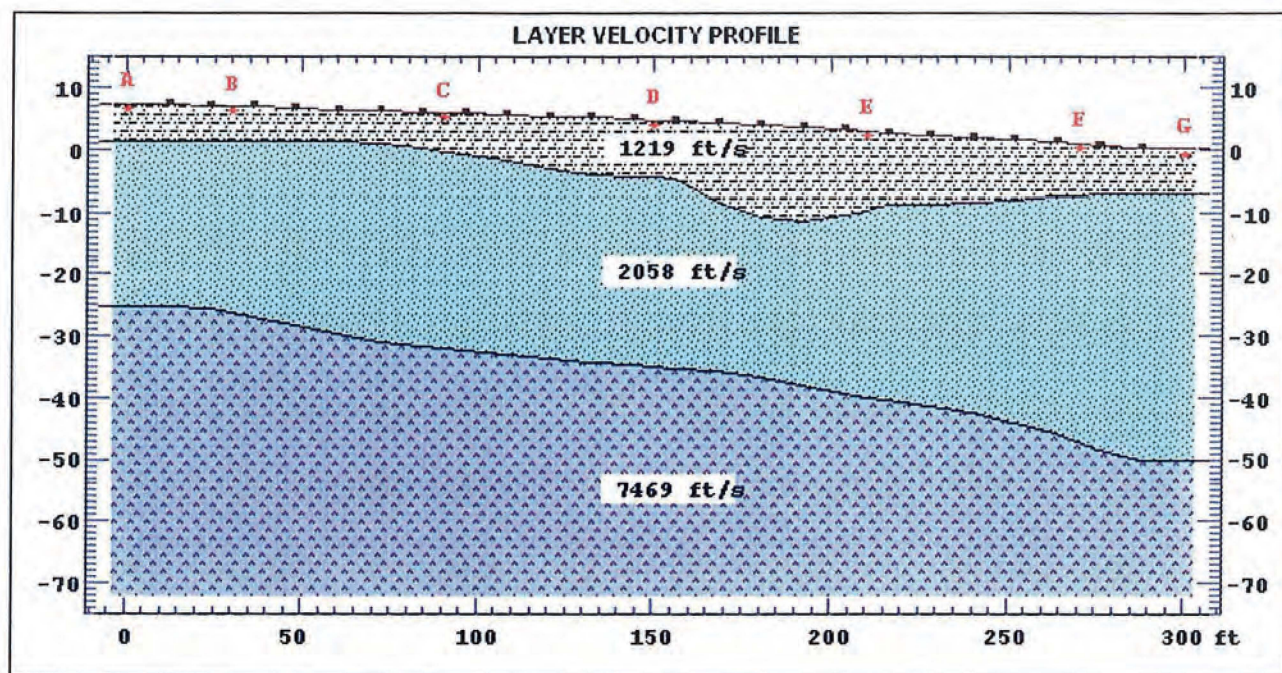
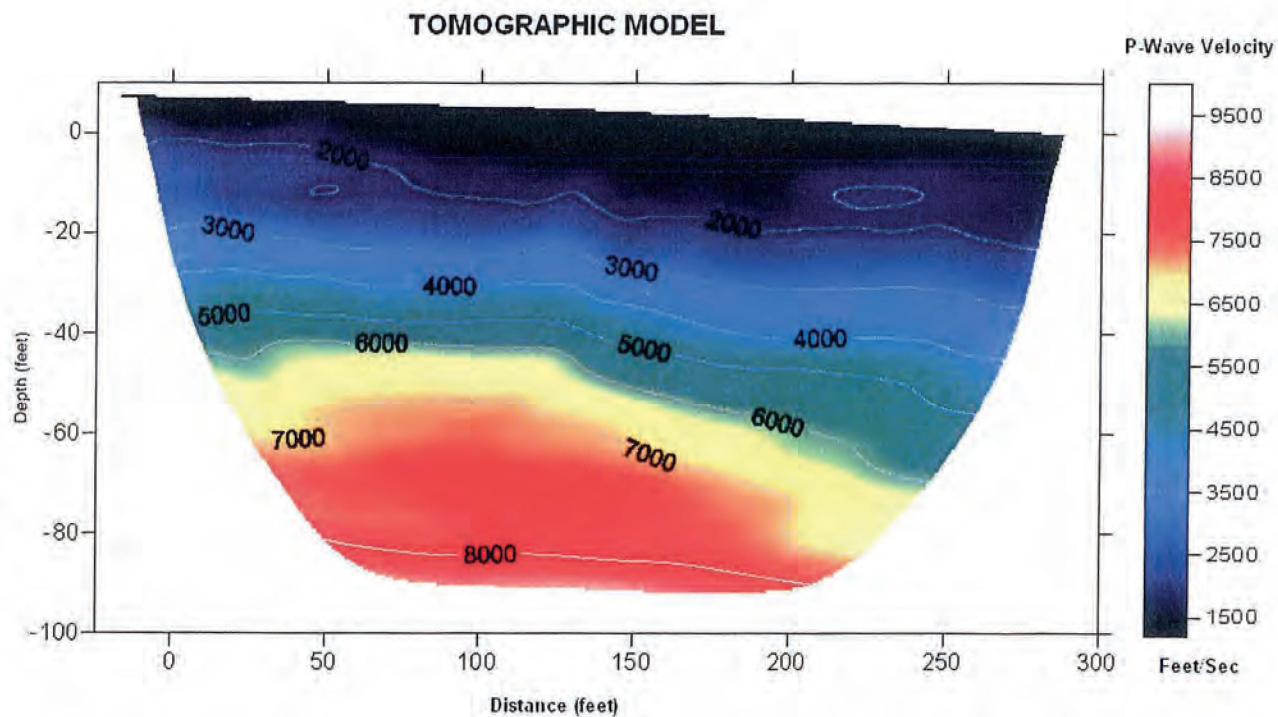
SEISMIC LINE S-7

South 30° West →



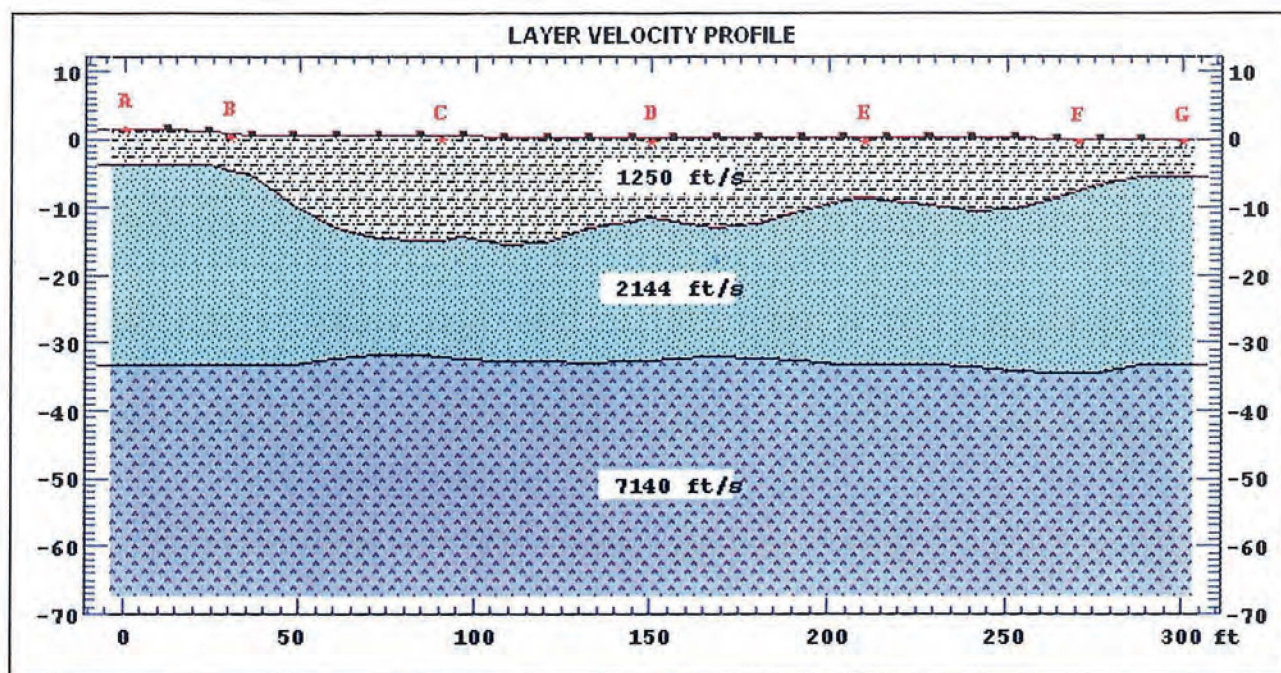
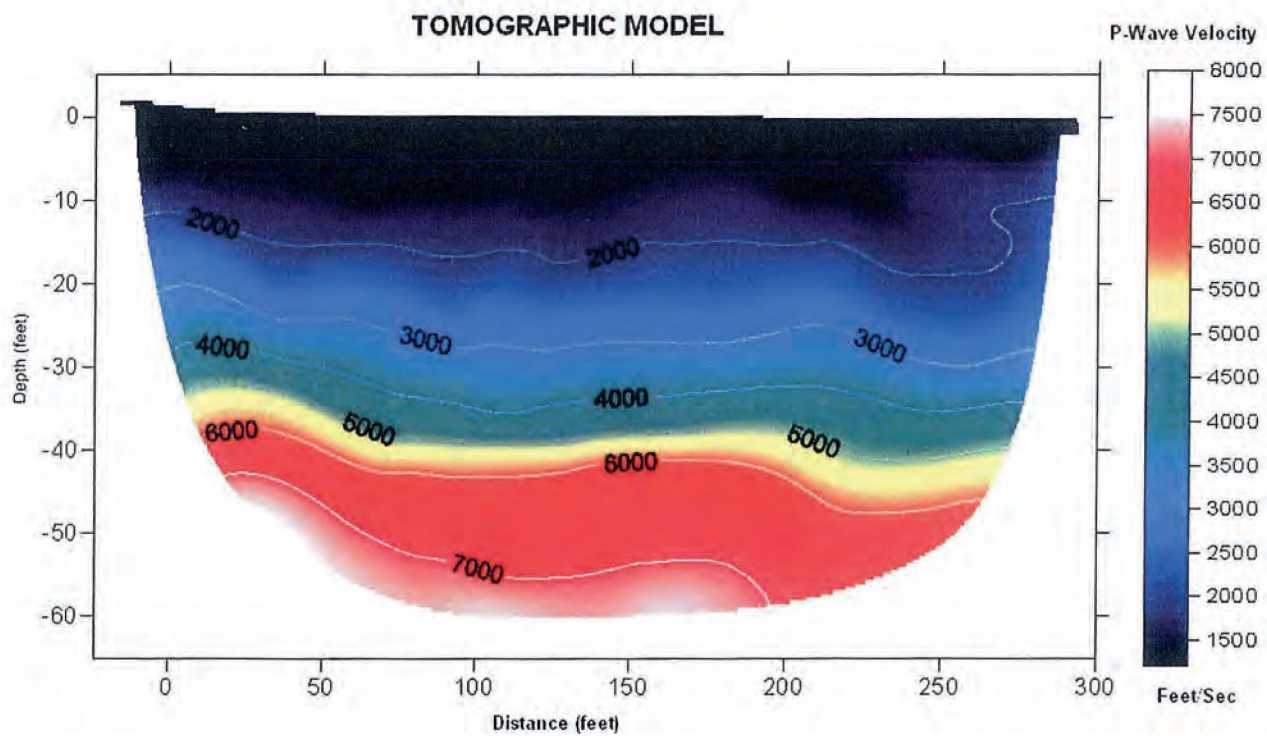
SEISMIC LINE S-8

South 10° West →



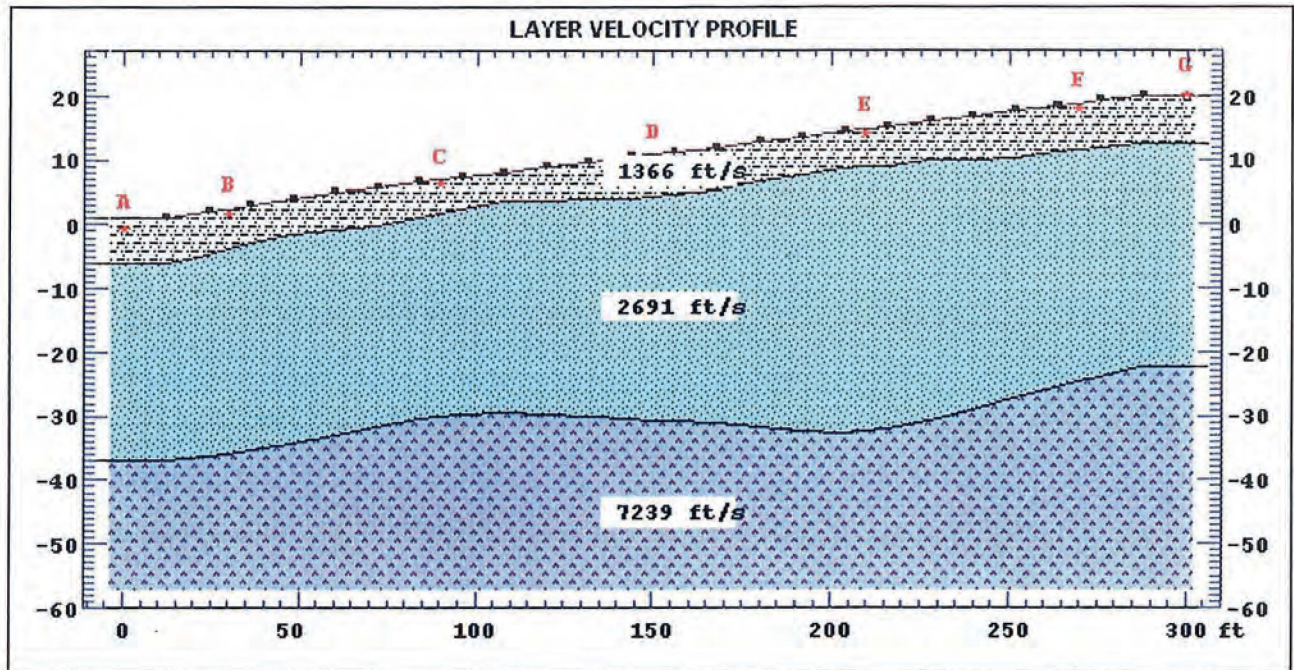
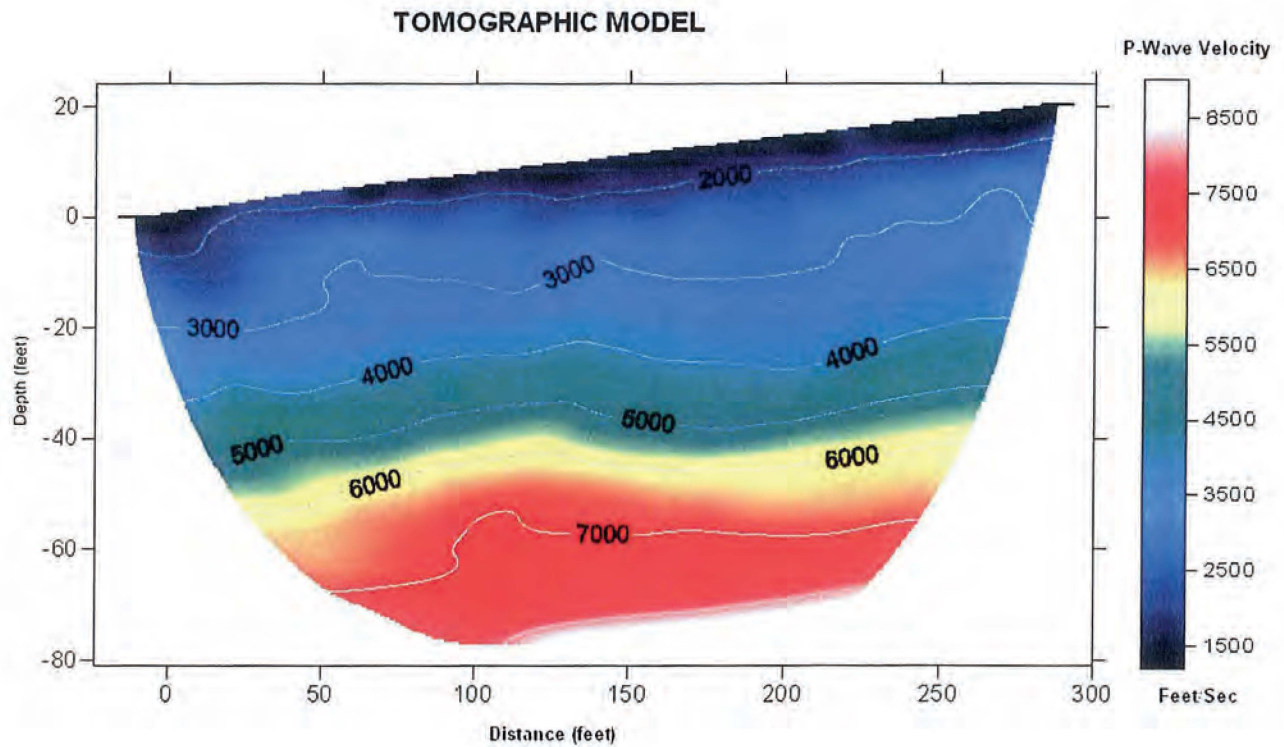
SEISMIC LINE S-9

South 10° East →



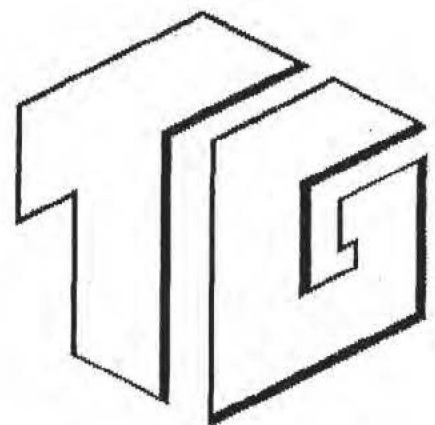
SEISMIC LINE S-10

South 75° East →



APPENDIX B

EXCAVATION CONSIDERATIONS



EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the clients responsibility to insure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification or lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ◆ **Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,**
- ◆ **Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,**
- ◆ **Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.**

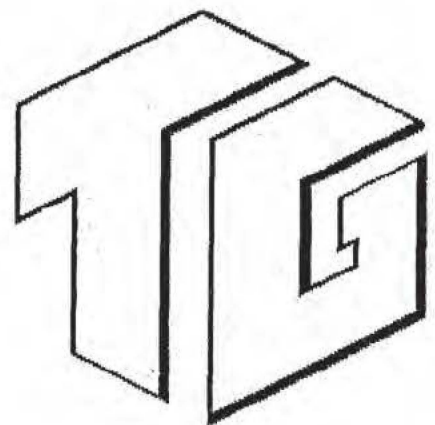
In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to insure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX C

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REFERENCES

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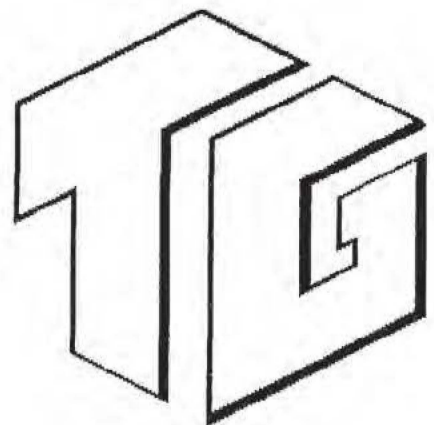
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APPENDIX A

SEISMIC SURVEY LINES





SEISMIC REFRACTION SURVEY

TRACT 16201

CITY OF ORANGE, CALIFORNIA

Project No. 262030-2

February 24, 2006

Prepared for:

LGC, Inc.
1319 Calle Avanzado
San Clemente, CA 92673-6351

LGC, Inc.
1319 Calle Avanzado
San Clemente, CA 92673-6351

Attention: Mr. Kevin B. Colson

Regarding: Seismic Refraction Survey
Tract 16201
City of Orange, California
LGC Job Number 051004-01

INTRODUCTION

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site along selected areas as delineated by you. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to aid in evaluating whether high velocity earth materials (non-rippable) are present along local areas which could possibly indicate areas of potential excavation difficulties. Numerous bedrock materials have been locally mapped within our study area by Tan (1995), to include Pleistocene age terrace deposits consisting of semi-consolidated sand with high clay and silt content and locally high gravel content; the middle Miocene age El Modeno Volcanics consisting of interbedded andesite and basalt flows, tuff and breccia; and sedimentary bedrock of the lower Miocene age Vaqueros and Sespe Formations consisting of undifferentiated sandstone, claystone, siltstone, and conglomerate.

Representative Layer Velocity Profiles for each seismic line have been prepared which indicates their respective "weighted average" subsurface velocities in generalized layers. In addition, associated Tomographic Models have also been prepared for comparative purposes, which generally indicate the relative structure and velocity distribution for each seismic line. These comparative survey line images are presented within Appendix A. We understand that this report will be included as a technical appendix to your report, therefore as requested, the locations of our geophysical survey lines were transferred onto your field map for inclusion onto your final map. As authorized by you, the following services were performed during this study:

- Review of available published and unpublished geologic/geophysical data in our files pertinent to the site,
- Performing a seismic refraction survey by a State of California Registered Geophysicist, to include seven traverses along selected portions of the subject property.
- Preparation of this report, presenting our findings and conclusions with respect to the velocity characteristics and the expected rippability potentials of the subsurface earth materials.

Accompanying Appendices

- Appendix A - Seismic Survey Lines
- Appendix B - Excavation Considerations
- Appendix C - References

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Seven seismic refraction survey lines were performed, varying from 195 to 300 feet in length, with a target depth ranging between 65- to 100±-feet. A 16-pound sledgehammer was used as an energy source to produce the seismic waves and both 12 or 24, 14-Hz geophones (with 70% damping), were spaced at 12- to 15-foot intervals along the traverse lines (depending on target depth) to detect both the direct and refracted waves. The seismic wave arrivals were recorded on a 24-channel Geometrics SmartSeis® SE model signal enhancement refraction seismograph. Seven shot points were utilized along each seismic line spread using offset, forward, reverse, and intermediate locations, in order to obtain sufficient data for velocity analysis and depth modeling purposes. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrival, of which are digitally recorded on the in-board seismograph computer and subsequently transferred to a disk and downloaded into our office computer for further processing, analyzing, and printing purposes.

Data Reduction

The data on the paper record and/or display screen were used to analyze the arrival time of the seismic waves at each geophone station, in the form of a wiggle trace, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes, using the computer programs **SIPwin** (Seismic refraction Interpretation Program for Windows) developed by Rimrock Geophysics, Inc. (2003), and **Rayfract™** (Intelligent Resources, Inc., 1996-2005). **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). In addition, **Rayfract™** was also used for comparative purposes. **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin our discussion, it should be understood that the velocity data obtained during this survey represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders/dikes or local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics.

It is also important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (bedding, fracturing, etc.). It is assumed that for the most part the sedimentary bedrock materials located within the site are well-bedded. Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time, as the structure of the bedrock locally is not known.

In general, the predominance of the site, where locally surveyed, was noted to be characterized by three major subsurface layers with respect to seismic velocities. The following velocity layer summaries have been prepared using the **SIPwin** analysis, as presented in Appendix A on the Layer Velocity Profiles.

□ Velocity Layer V1:

This uppermost velocity layer (V1) is most likely comprised of topsoil, colluvium, and/or very highly-weathered and/or intensely fractured sedimentary bedrock materials. This layer has an average weighted velocity ranging from 1,196 to 1,431 fps, which is typical for these types of surficially-mantling materials.

□ Velocity Layer V2:

The second velocity layer (V2) yielded a range of 1,826 to 2,886 fps, indicating slight to moderate degrees of induration, weathering, and/or fracturing of the underlying sedimentary bedrock, possible volcanics, and/or terrace deposits. These velocities are also typical for the near-surface weathered zone commonly found in both sedimentary and igneous rocks.

□ Velocity Layer V3:

The third layer (V3) indicates suspected volcanic bedrock at depth or possibly more lithified sedimentary bedrock materials, with average weighted velocities of 6,078 to 7,280 fps. At this time, it is assumed that the V3 velocities most likely represent volcanic rock at depth and may actually be present in the form of relatively thin dikes or sills, which are in turn underlain by lower velocity sedimentary bedrock. However, if this condition is present, it is not possible to detect or model any such possible lower velocity materials at depth as well as estimate the thickness of any such thin overlying volcanic structure, due to seismic velocity reversals. This causes the refracted waves to propagate downwards rather than upwards toward the surface where they can be detected.

The following table summarizes the results of the survey lines with respect to the "weighted average" seismic velocities for each layer and their approximate average depths, based on the Layer Velocity Profiles presented in Appendix A. The survey lines begin with S-11 as Seismic Lines S-1 through S-10 were performed during our previous study (Terra Geosciences, 2006).

TABLE 1: SUMMARY OF SEISMIC SURVEY LINES

<u>Seismic Line No.</u>	<u>Average Weighted Velocity (ft/sec)</u>	<u>Approximate Average Depth Range (feet) *</u>
S-11	1,339	0-4
	2,886	4-50
	6,264	50-100±**
S-12	1,276	0-7
	2,421	7-63
	6,144	63-100±**
S-13	1,349	0-7
	2,547	7-40
	6,078	40-100±**
S-14	1,196	0-11
	1,826	11-50
	7,207	50-100±**
S-15	1,336	0-5
	2,182	5-45
	7,280	45-100±**
S-16	1,391	0-5
	2,356	5-40
	6,102	40-70±**
S-17	1,431	0-4
	2,150	4-70±**

NOTES: * See Layer Velocity Profiles in Appendix A for layer depth variability.

** Minimum anticipated depth of lowest layer based on length of survey line.

It was observed that some of the raw arrival times of the primary "first-arrival" seismic waves were generally curvilinear in shape, indicating that the velocity of some of the subsurface earth materials most likely increases gradually with depth rather than creating a sharp and clear boundary, as discussed further below. In order to create a subsurface geometric model for the SIPwin analysis, these curvilinear lines must be approximated as linear features, and are somewhat subjective. Additionally, it should be noted that in the event that the higher velocity layer (V3) may consist of thin volcanic dikes and sills that are in turn underlain by lower velocity sedimentary bedrock, it is not possible to detect any such materials or estimate the thickness of these structures due to seismic velocity reversals.

Using **Rayfract™**, a tomographic model for each seismic line was also prepared and analyzed for comparative purposes, as also presented in Appendix A, which generally indicates the relative structure and velocity distribution. Although no discrete velocity layers or boundaries are created, these models generally resemble the **SIPwin** analysis. **Rayfract™** allows imaging of subsurface velocity using first break energy propagation modeling. An initial 1D gradient model is created using the Delta-t-V method which gives a good initial fit between modeled and picked first breaks. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion. WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break.

It can be seen in these models that the seismic velocity (i.e., hardness) of the bedrock gradually increases with depth which could possibly be the representative condition of the subsurface materials. However, in the event that there are relatively thin high velocity structures (i.e., volcanic dikes, sills, etc.) underlain by lower velocity sediments at depth, these models will not portray the true velocity distribution and gradients due to velocity reversals that would occur along the boundary of any such high velocity volcanic structures. It was noted that for the most part, the seismic velocities on the Layer Velocity Profiles appears to generally correlate with the average of the velocity gradients as shown on the Tomographic Models as presented on the Seismic Survey Line images presented within Appendix A.

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

Due to the highly variable earth materials that are present within the subject property, excavations characteristics could vary significantly across the site even for similar seismic velocity ranges. Although not known to be present at the site, the following summary is generally considered for granitic-type bedrock and has been slightly modified for this project in an effort to help aid in evaluating potential excavation difficulties with respect to the seismic velocities. The velocity ranges described below are approximate and assume typical, good-working, heavy excavation equipment, such as single shank or D9R dozer, such as described by Caterpillar, Inc. (2000 and 2004); however, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges. Caterpillar indicates that the non-rippable velocity limits for sedimentary and volcanic rocks are somewhat higher than for granitic bedrock and therefore the discussion below should be considered at least a minimum.

For reference purposes, a summary of excavation considerations has been included within Appendix B in order to provide the client and contractor with a better understanding of the complexities of excavation commonly found within bedrock materials. Although these conditions may not exist at the site, this information may still be of some use since there are some relatively high seismic velocities that were encountered locally.

□ Rippable Condition (0 - 4,000 ft/sec):

This velocity range indicates rippable materials which may consist of alluvial-type deposits, sedimentary bedrock, and decomposed igneous and metamorphic bedrock, with random hardrock floaters. These materials typically break down into

silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

□ **Marginally Rippable Condition (4,000 - 8,000 ft/sec):**

This range of velocities indicates materials which may consist of moderately weathered bedrock or large areas of fresh bedrock materials separated by weathered fractured zones. These materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

□ **Non-Rippable Condition (8,000 ft/sec or greater):**

This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of fair to good quality due to the introduction of significant "noise" that was locally encountered during our field survey originating from vehicular traffic along the nearby roads, including the 241 Freeway and Santiago Canyon and Jamboree Roads. Analysis of the data and picking of the primary "P"-wave arrivals was performed with some difficulty and some interpolation of the data was necessary. The anticipated maximum depth of velocity information appears to be on the order of 65- to 100±-feet in accordance with the depths you specified respectively for each seismic line.

Based on the results of our comparative seismic analyses of both **SIPwin** and **Rayfract™**, the seismic refraction survey lines appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data. For this reason, both seismic model types for each survey line have been included on the same sheet for comparative purposes as presented within Appendix A.

The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

□ Velocity Layer V1:

No major excavating difficulties are expected within the uppermost, low-velocity layer V1 (velocity range of 1,196 to 1,431 fps). This layer is expected to be comprised of topsoil, colluvium, and/or very highly-weathered or intensely fractured sedimentary bedrock materials.

□ Velocity Layer V2:

The second velocity layer V2, is believed to be comprised of slight to moderate degrees of induration, weathering, and/or fracturing of the underlying sedimentary bedrock and/or terrace deposits (velocity range of 1,826 to 2,886 fps), and should excavate with slight to moderate difficulty, assuming appropriate good-working equipment for the proposed type of excavation. Isolated floaters (i.e., boulders, high velocity bedding layers, etc.) are presumably present within this weathered layer and could produce difficult conditions locally. Placement of infrastructures in this material may also be difficult. Localized blasting cannot be completely ruled out, due to the presence of any localized fresh volcanic bedrock and/or buried boulders and dikes, although this condition appears very unlikely.

□ Velocity Layer V3:

Excavation difficulties within the lower V3 velocity layer (velocity range of 6,078 to 7,280 fps) are anticipated. These materials most likely represent less-weathered volcanic rock at depth which is also based upon local geologic mapping, nearby exposures, and exploratory subsurface boring data as discussed with your firm (personal communication). Since the nature and composition of the suspected volcanic rock is not locally known, hard excavating areas consisting of boulders, dikes, and/or relatively fresher intact rock matrix with wide-spaced fracturing could be encountered during both remedial grading and placement of infrastructures, which could require minor to significant blasting to achieve desired grade.

Based on the Tomographic Models and typical excavation characteristics observed within bedrock materials of the southern California region, anticipation of gradual increasing hardness with depth and lateral variations, with respect to excavation characteristics, should be anticipated across the site. The Excavation Considerations included within Appendix B, should be understood so that proper planning and excavation techniques can be employed by the selected grading contractor.

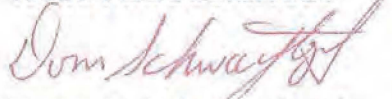
CLOSURE

This survey was performed using "state of the art" geophysical techniques, computer processing, and equipment, in the localized areas delineated by you. We make no warranty, either expressed or implied. It should be noted that our data was obtained along only seven specific areas; therefore, other local areas at the site beyond the limits of our seismic lines may contain different velocity layers and depths not encountered during our field survey. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data

obtained and in the interpretation. In summary, the results of this survey are to be considered as an aid to assessing the rippability potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative "test" excavations should be considered, so that they may be correlated with the data presented within this report.

If you should have any questions regarding this report or do not understand the limitations of this survey, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

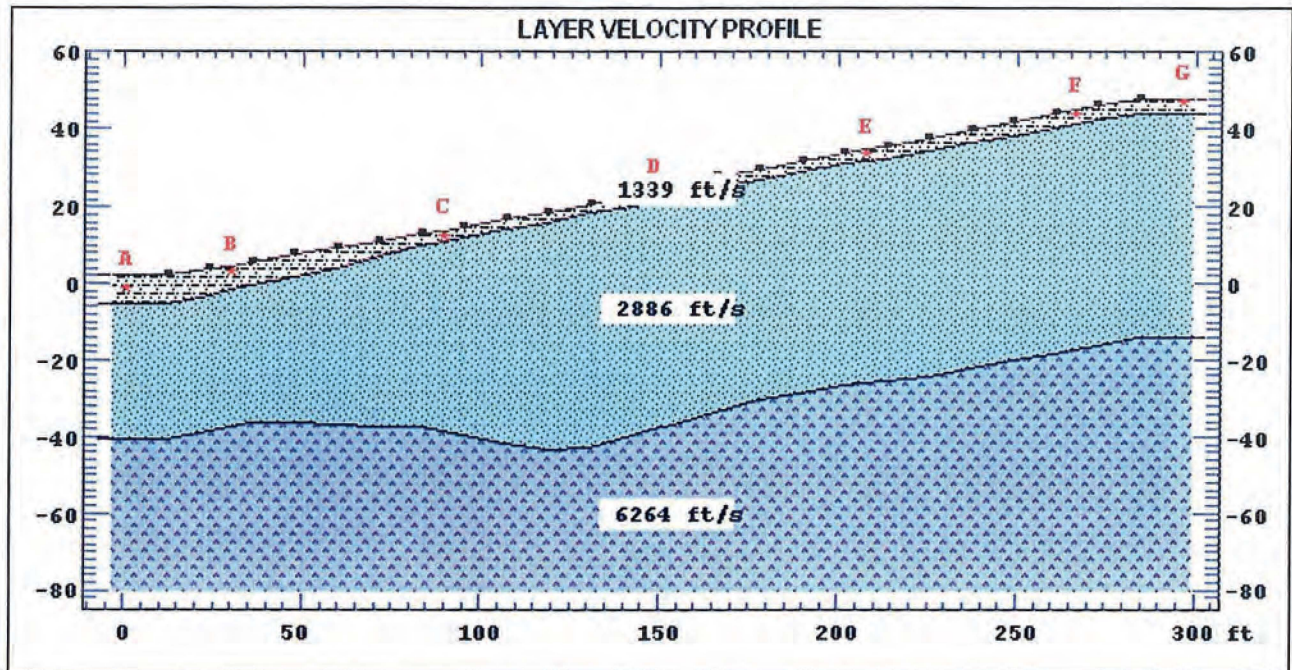
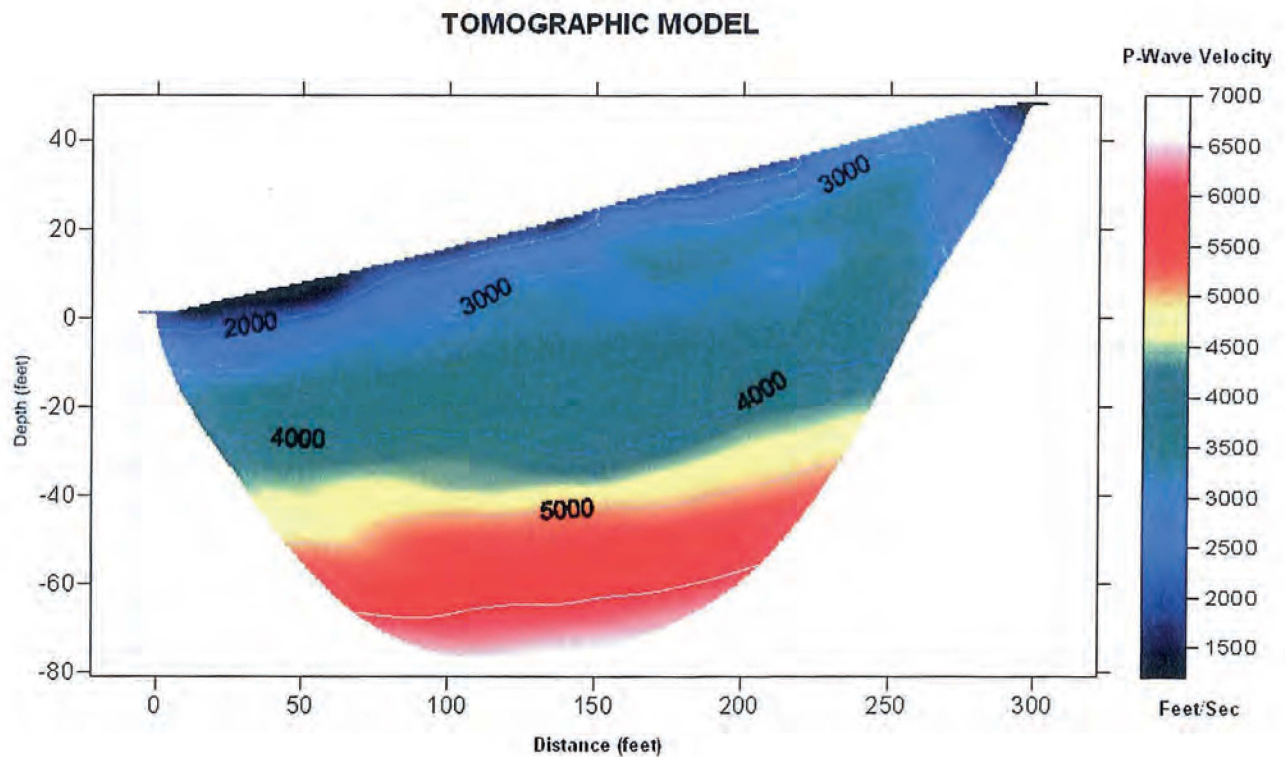


Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002



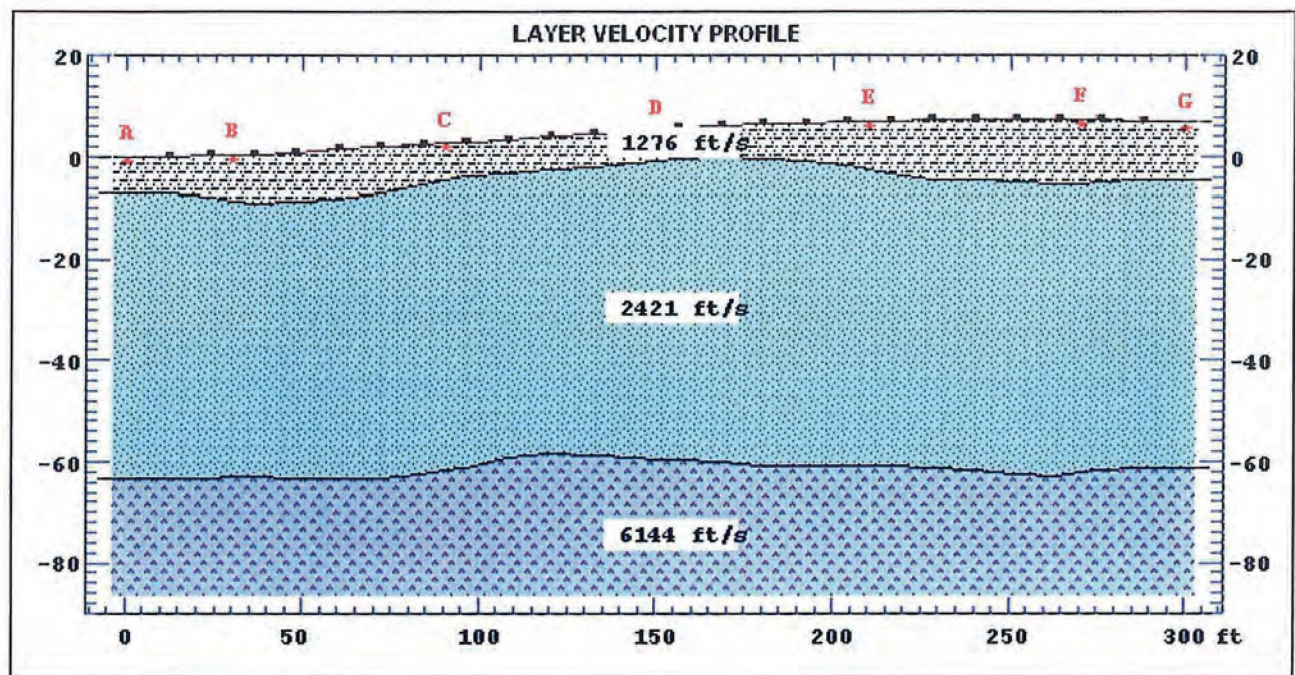
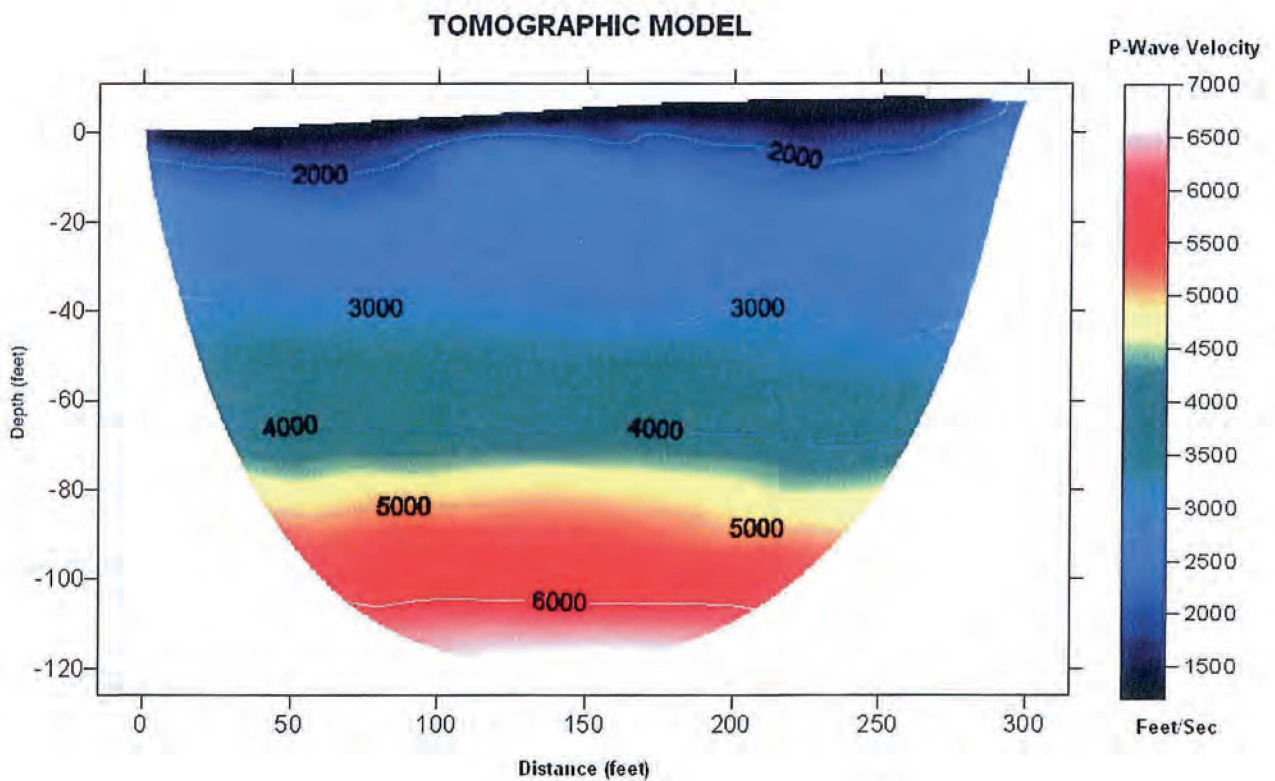
SEISMIC LINE S-11

South 85° East →



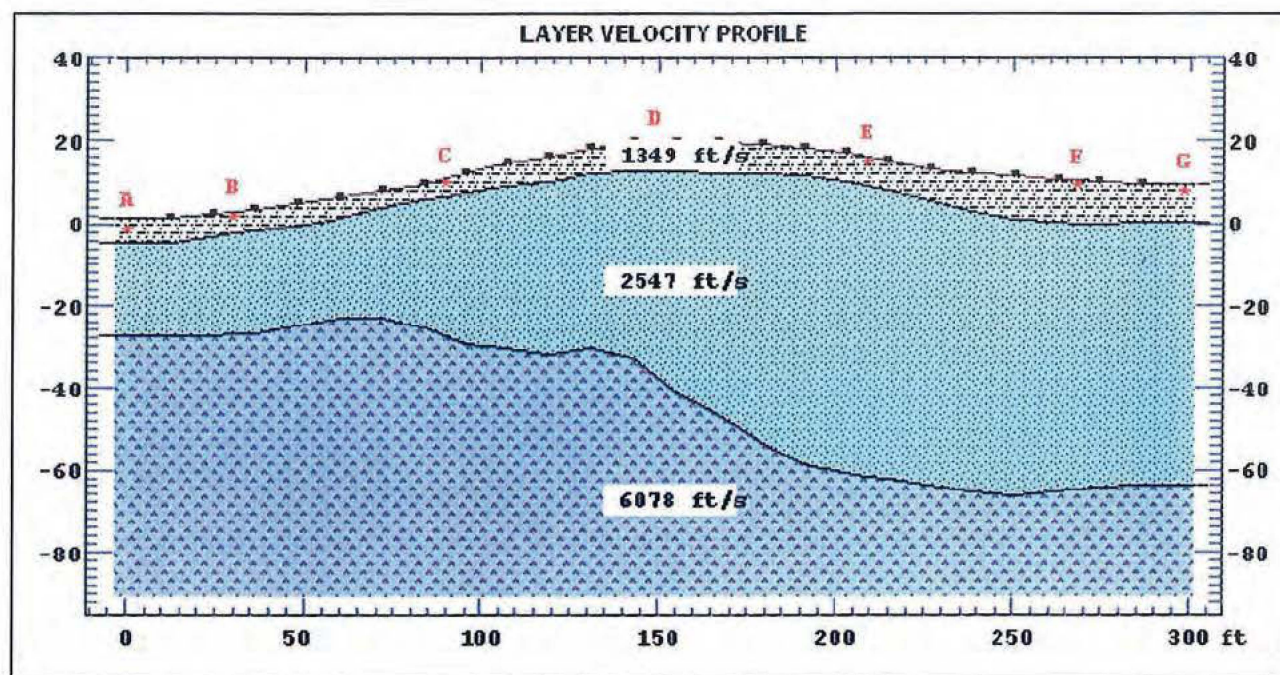
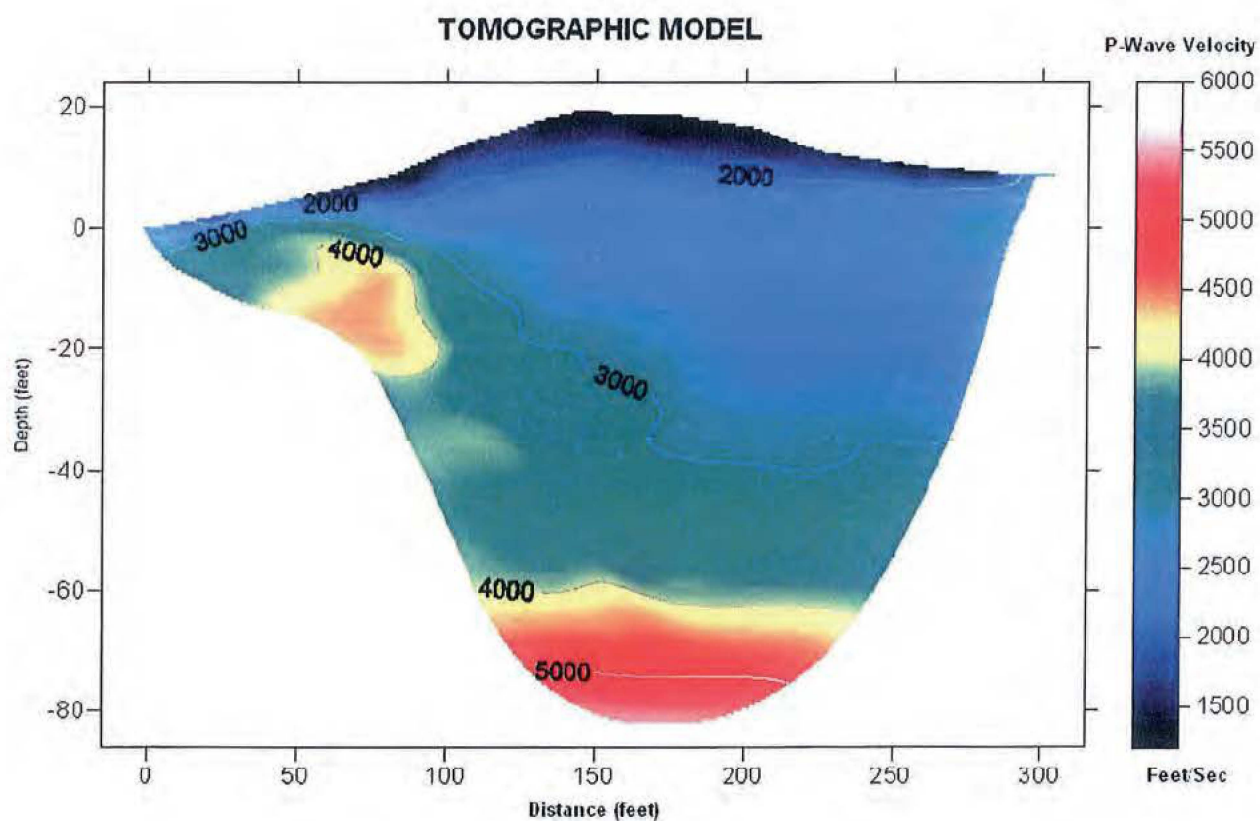
SEISMIC LINE S-12

← North – South →



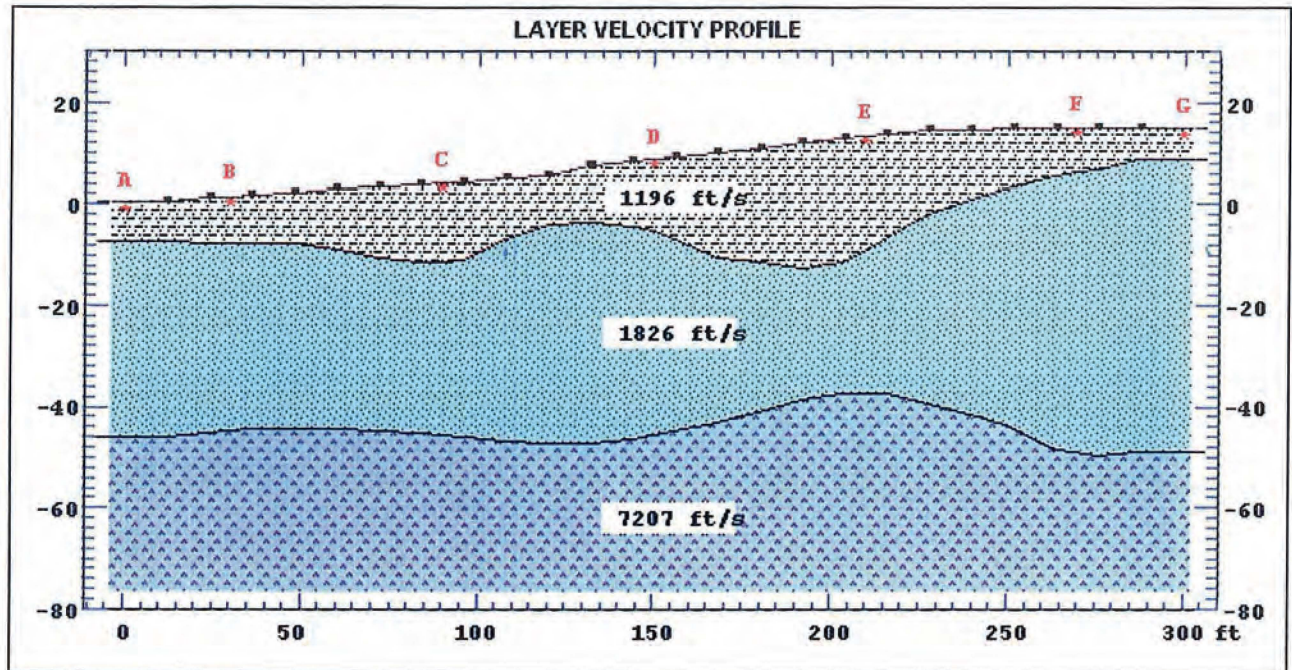
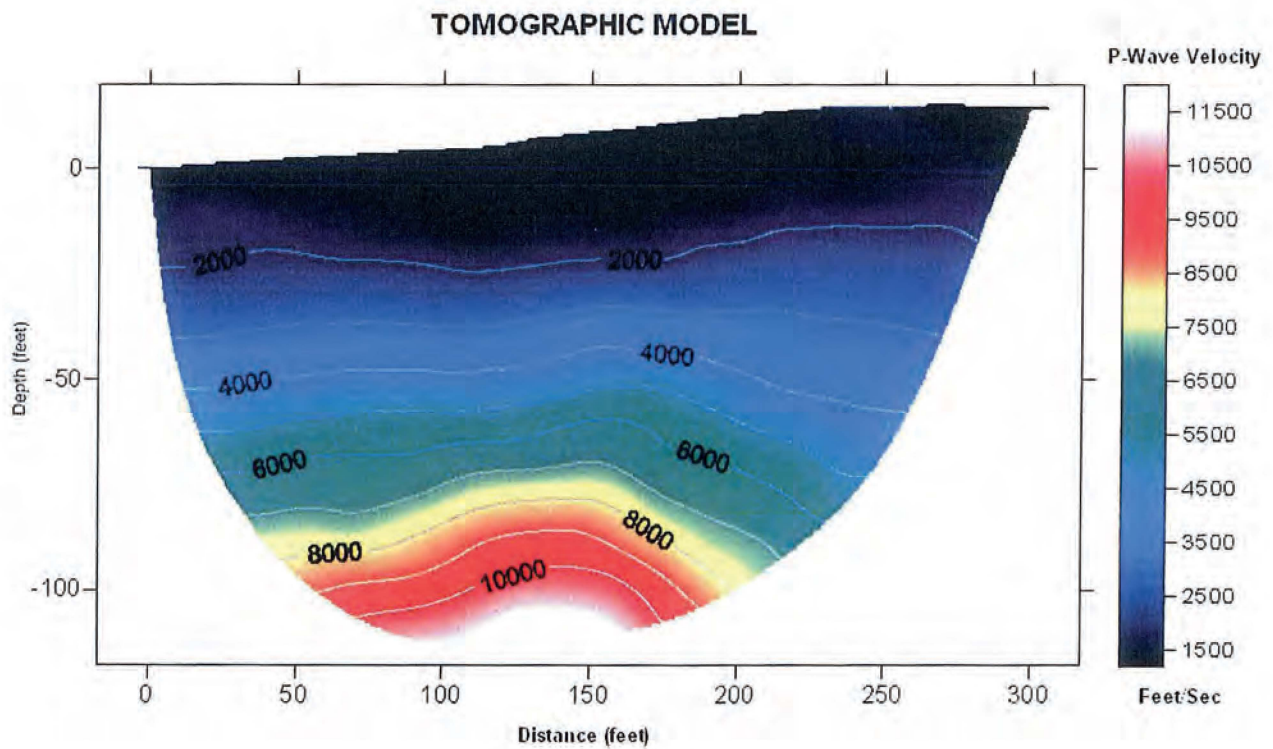
SEISMIC LINE S-13

South 45° East →



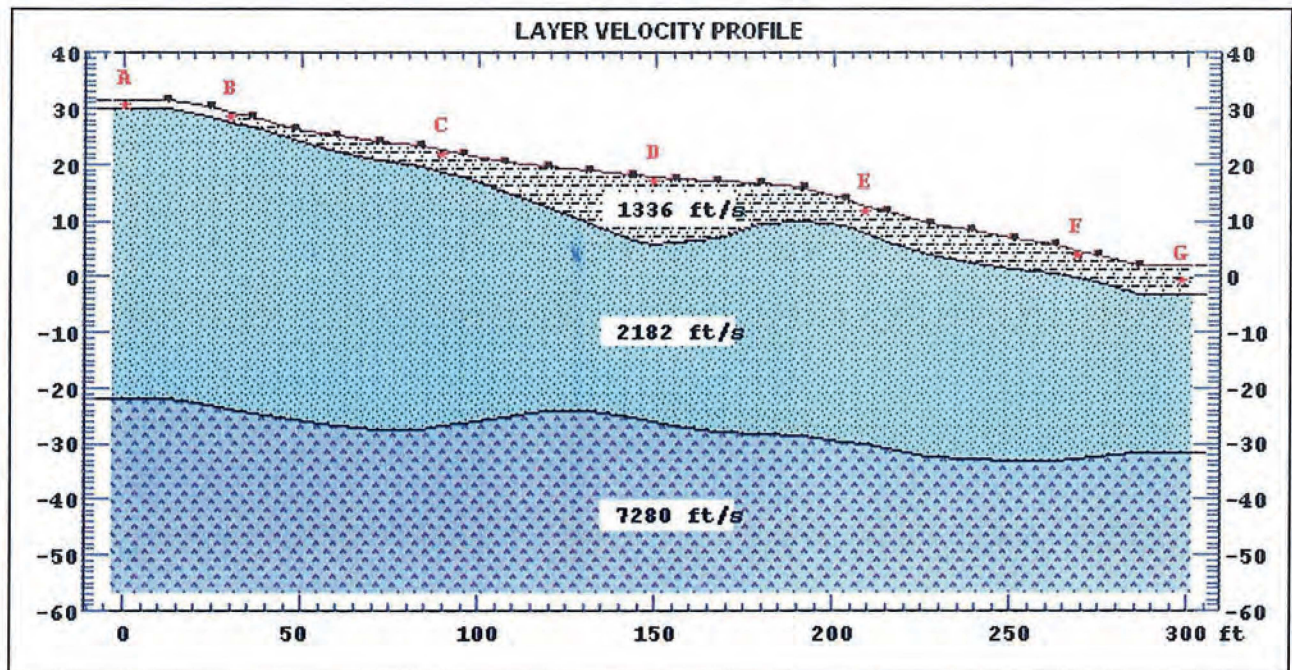
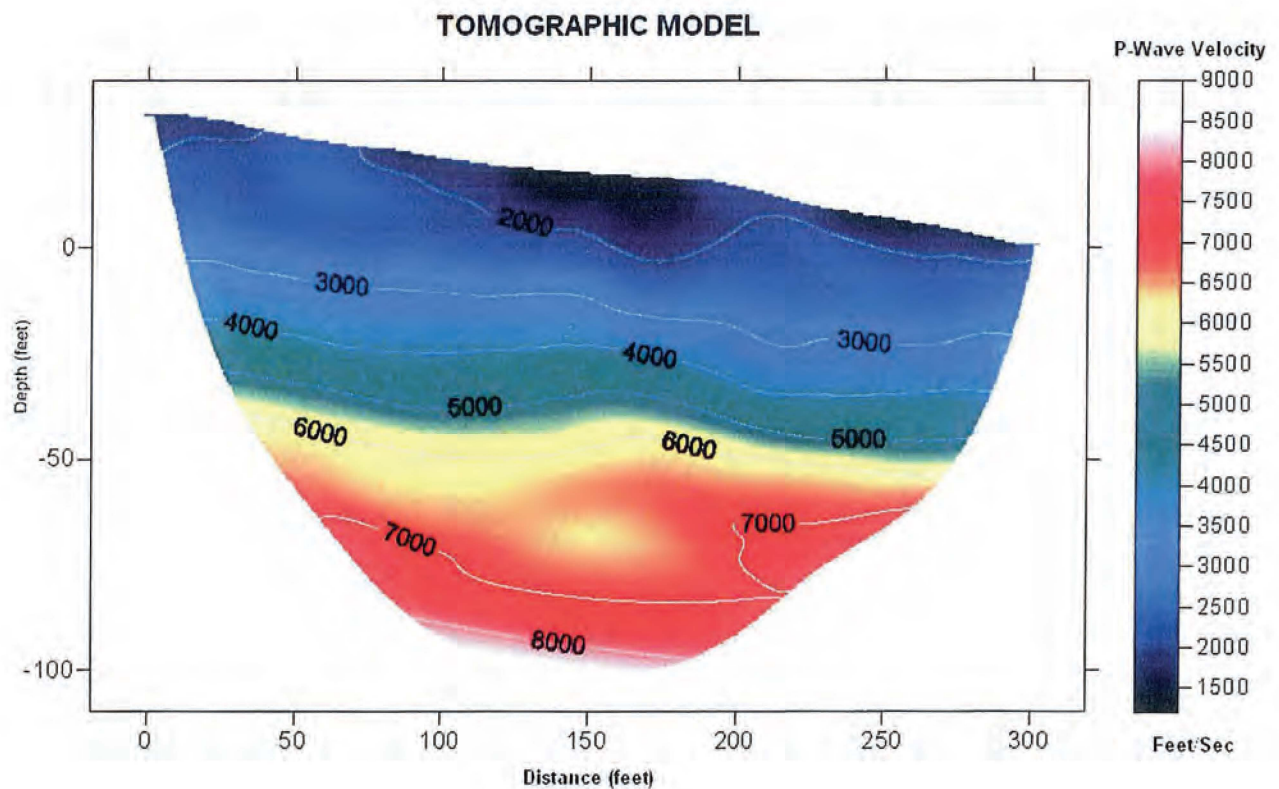
SEISMIC LINE S-14

South 20° West →



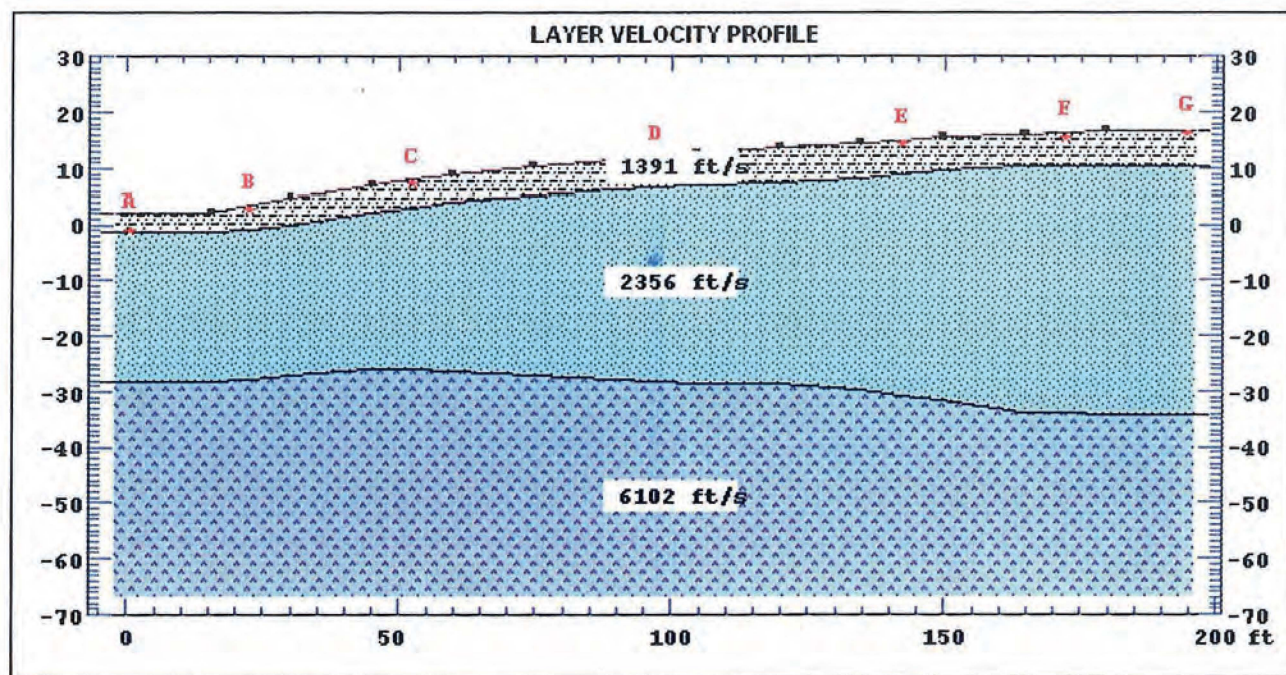
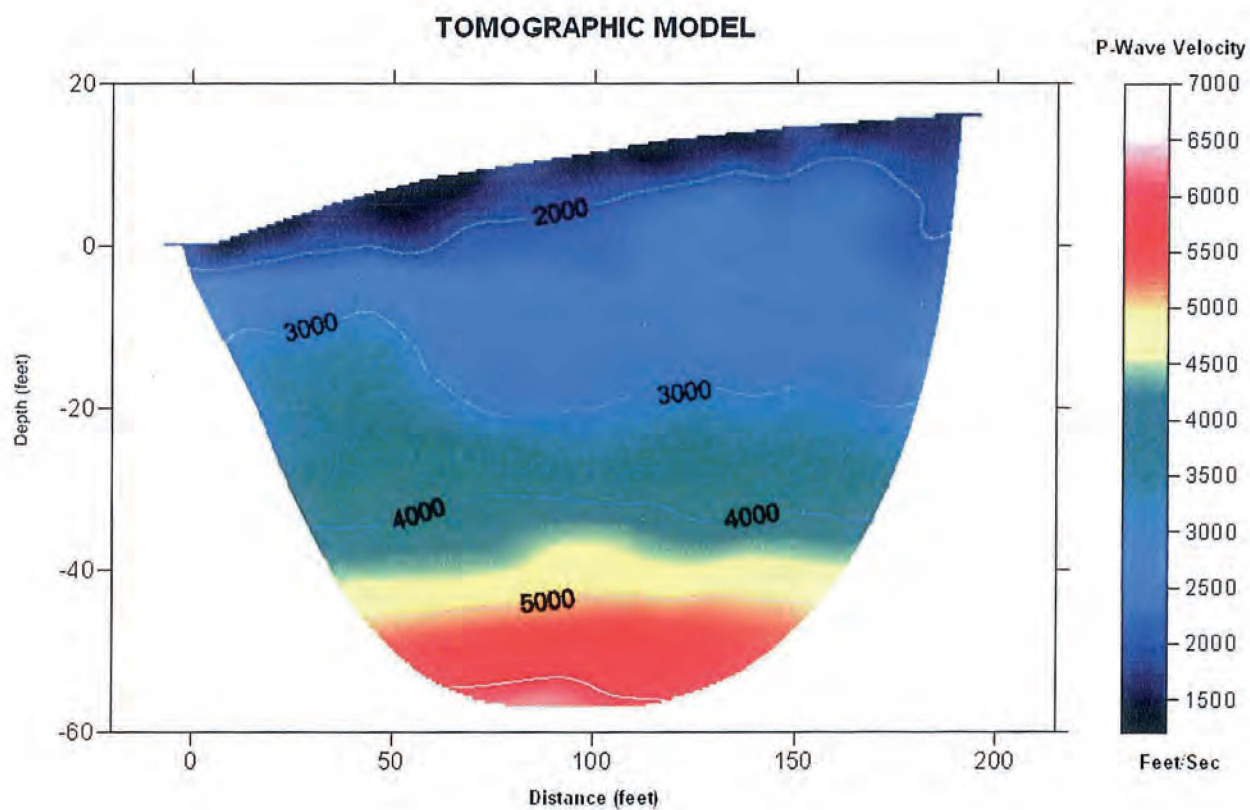
SEISMIC LINE S-15

South 25° West →



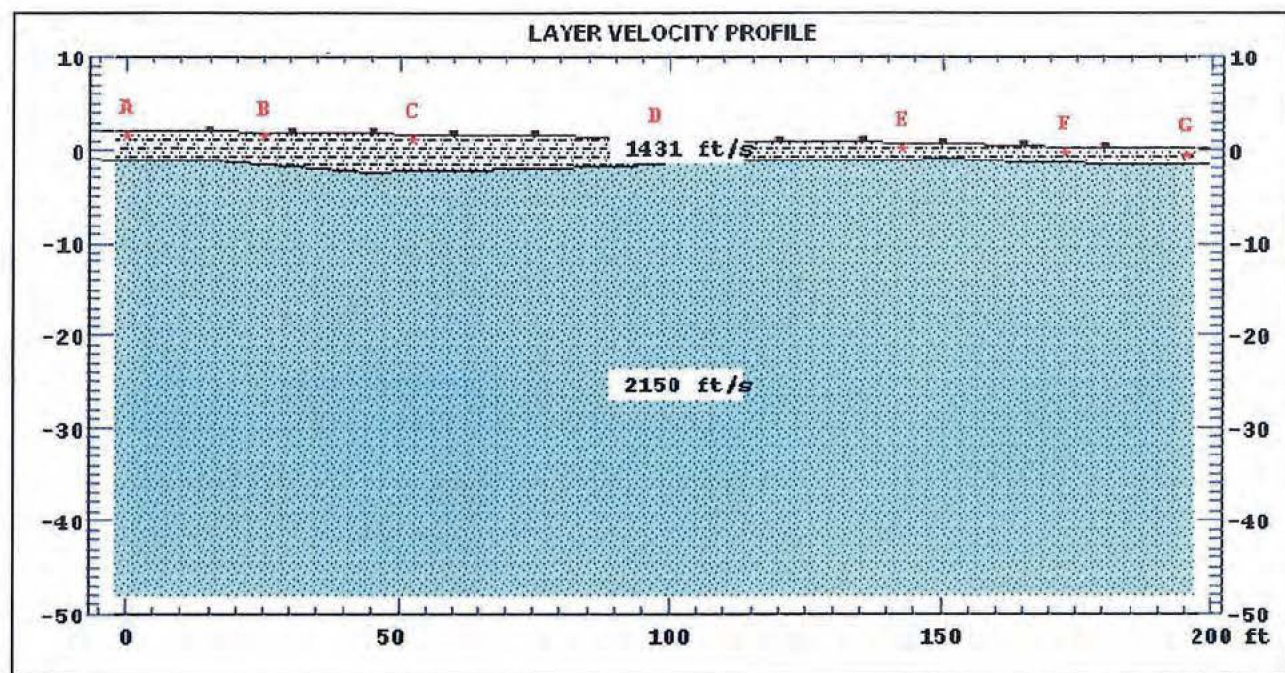
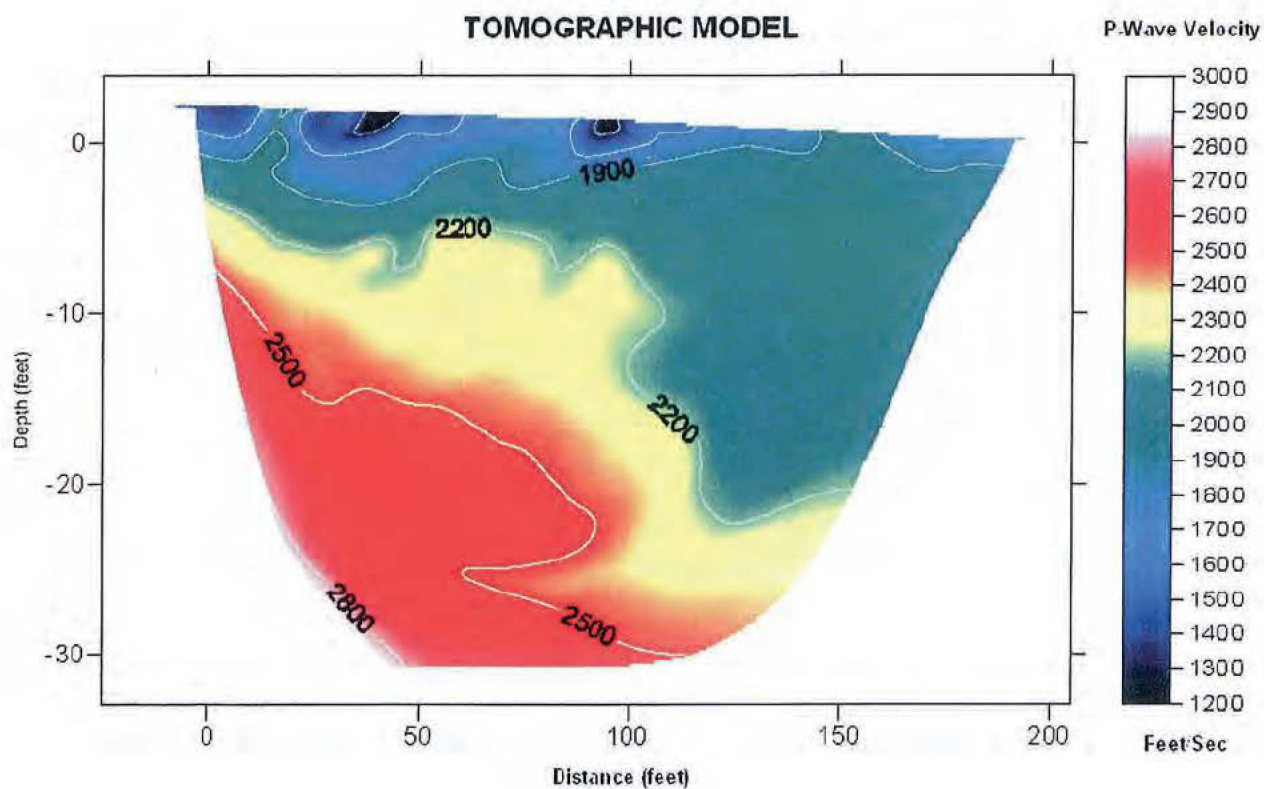
SEISMIC LINE S-16

South 85° West →



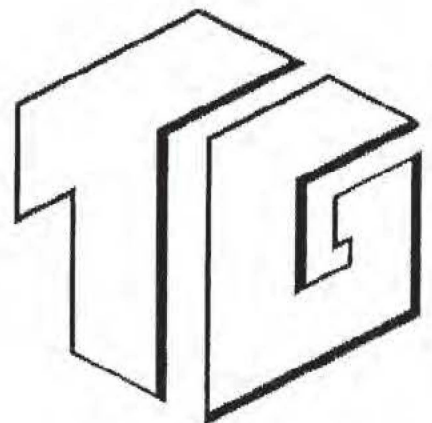
SEISMIC LINE S-17

← West - East →



APPENDIX B

EXCAVATION CONSIDERATIONS



EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the clients responsibility to insure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ♦ **Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,**
- ♦ **Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,**
- ♦ **Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.**

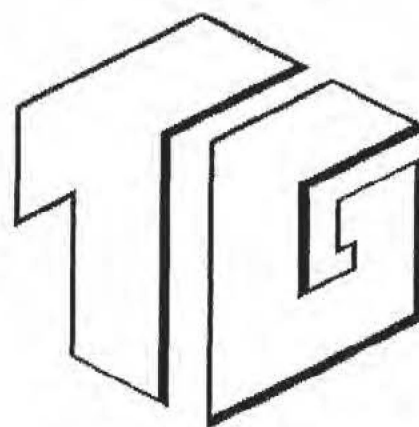
In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to insure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX C

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REFERENCES

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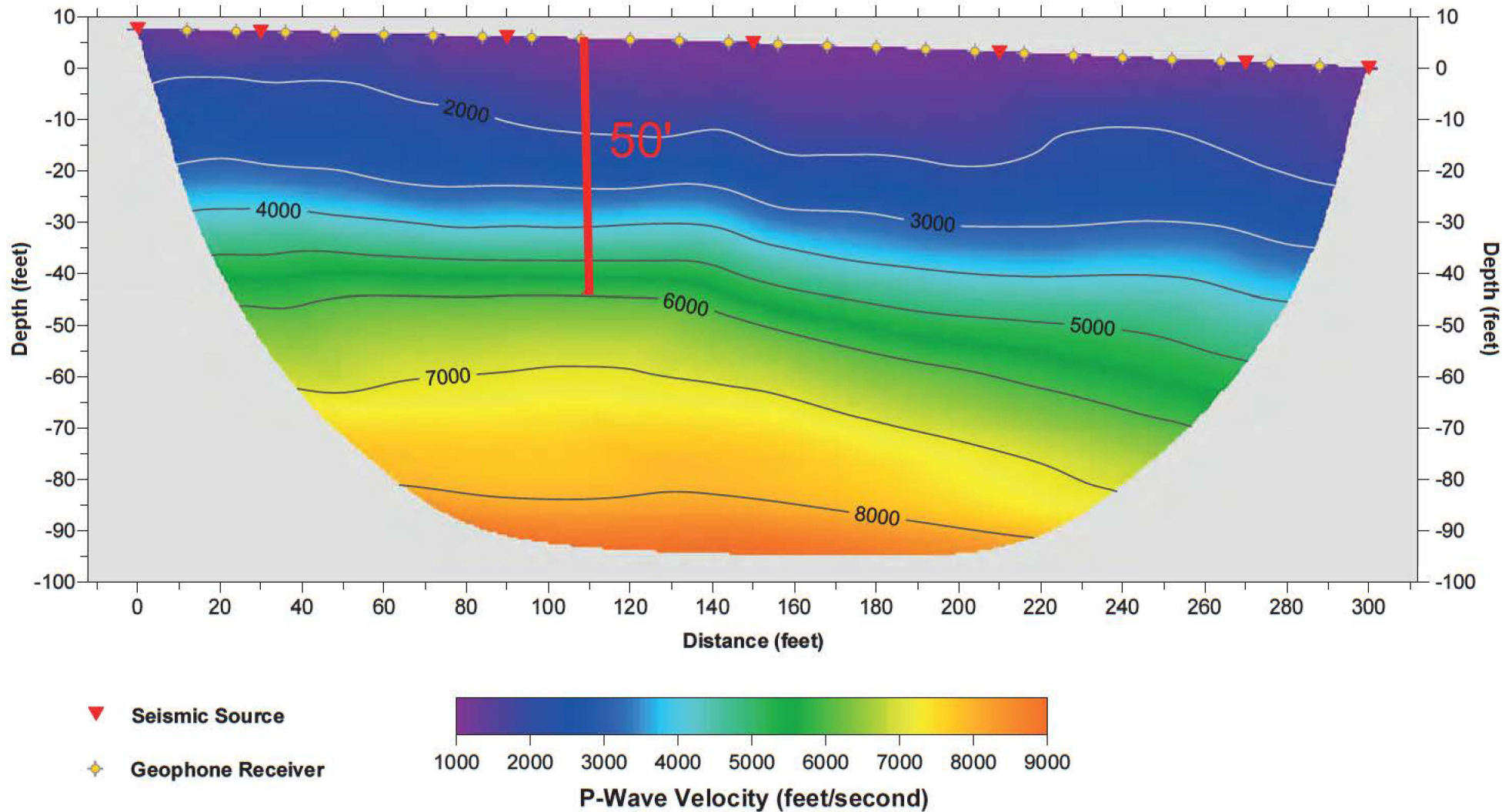
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**SEISMIC REFRACTION LINES IN NORTH TRACT
UPDATED BY TERRA GEOSCIENCE
IN 2024**

SEISMIC LINE S-8

South 10° West →

REFRACTION TOMOGRAPHIC MODEL



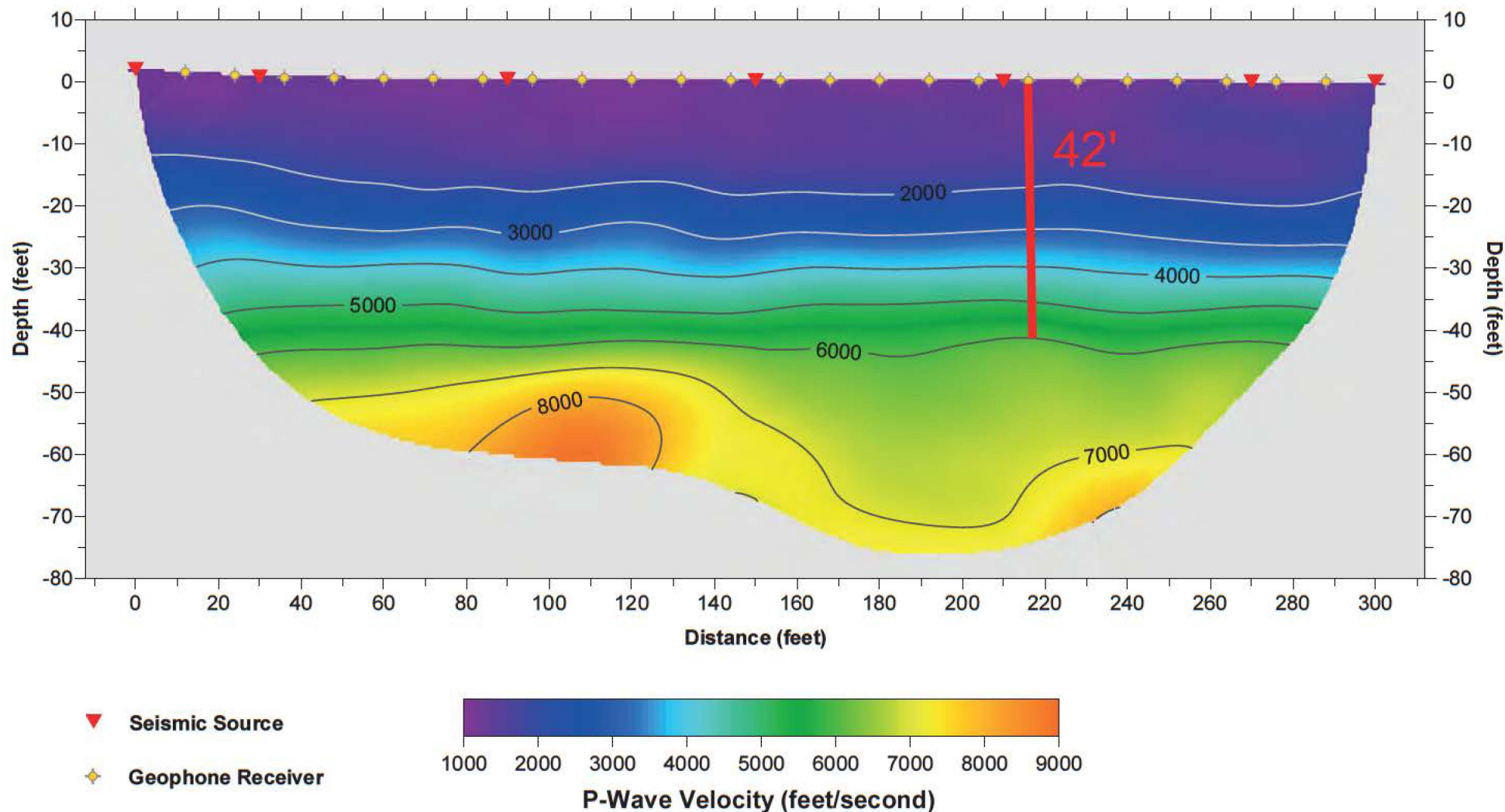
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RMS error 1.2; Rayfract Version 4.06

SEISMIC LINE S-9

South 10° East →

REFRACTION TOMOGRAPHIC MODEL



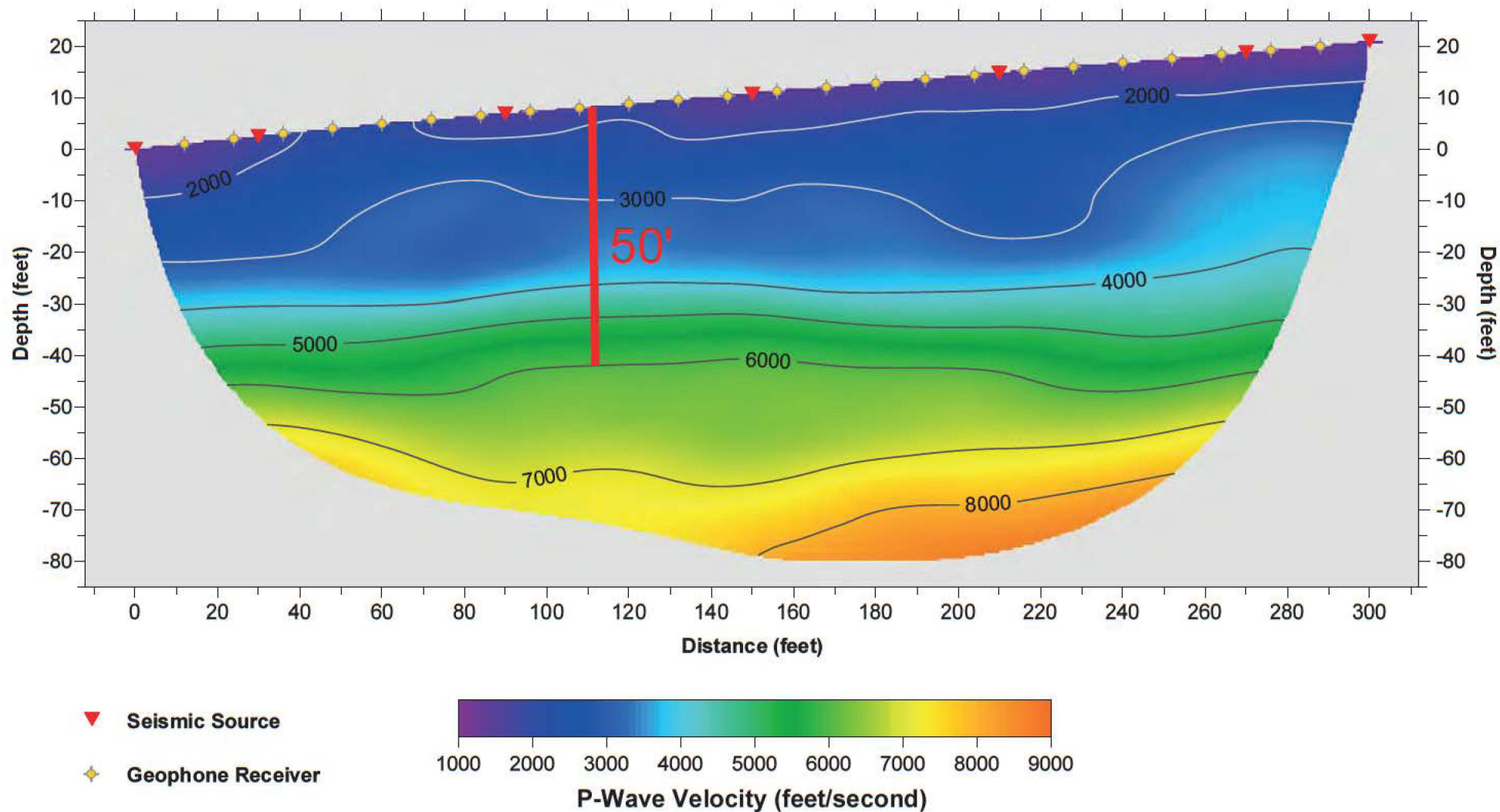
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RMS error 1.0; Rayfract Version 4.06

SEISMIC LINE S-10

South 75° East →

REFRACTION TOMOGRAPHIC MODEL



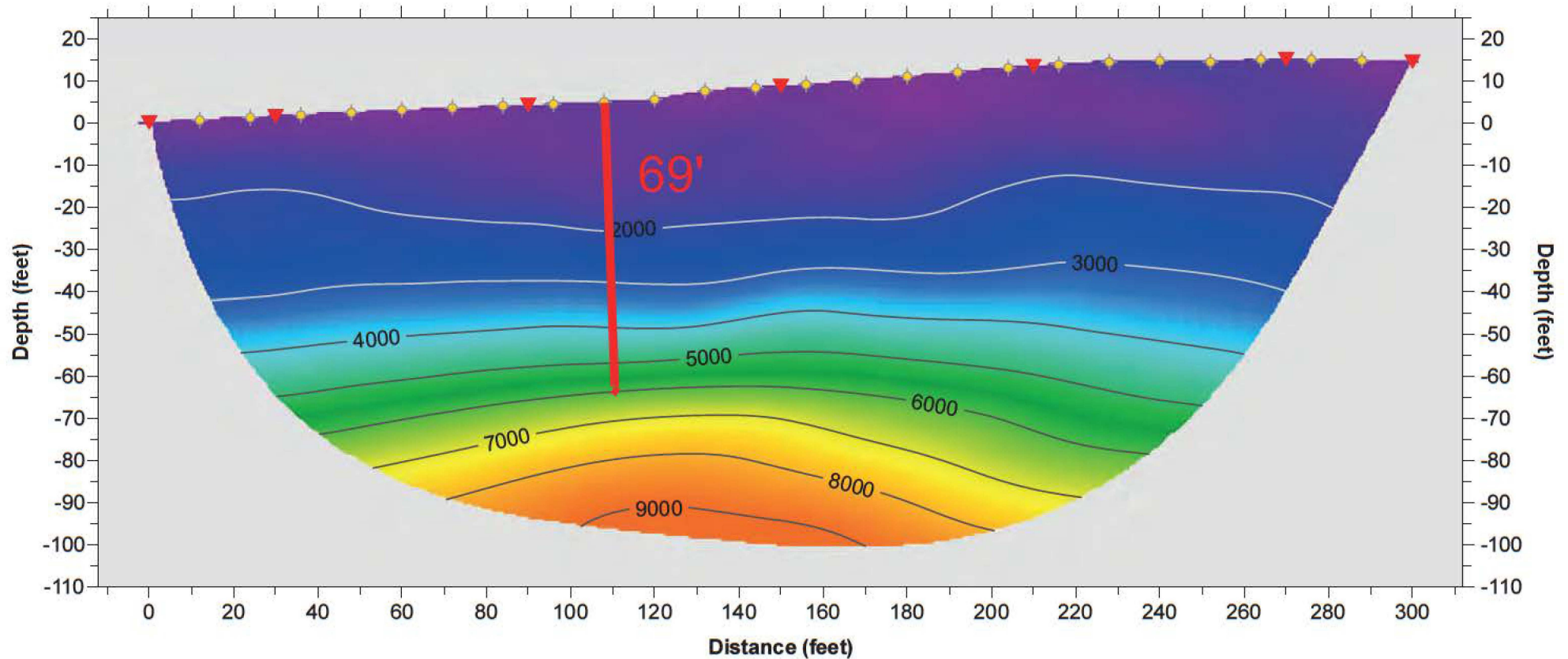
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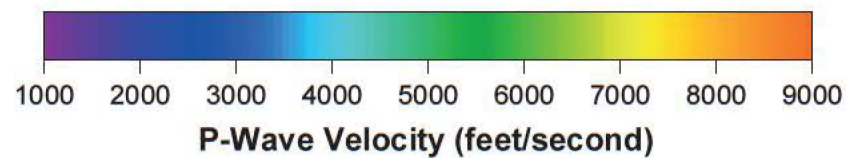
SEISMIC LINE S-14

South 20° West →

REFRACTION TOMOGRAPHIC MODEL



- ▼ Seismic Source
- ◆ Geophone Receiver



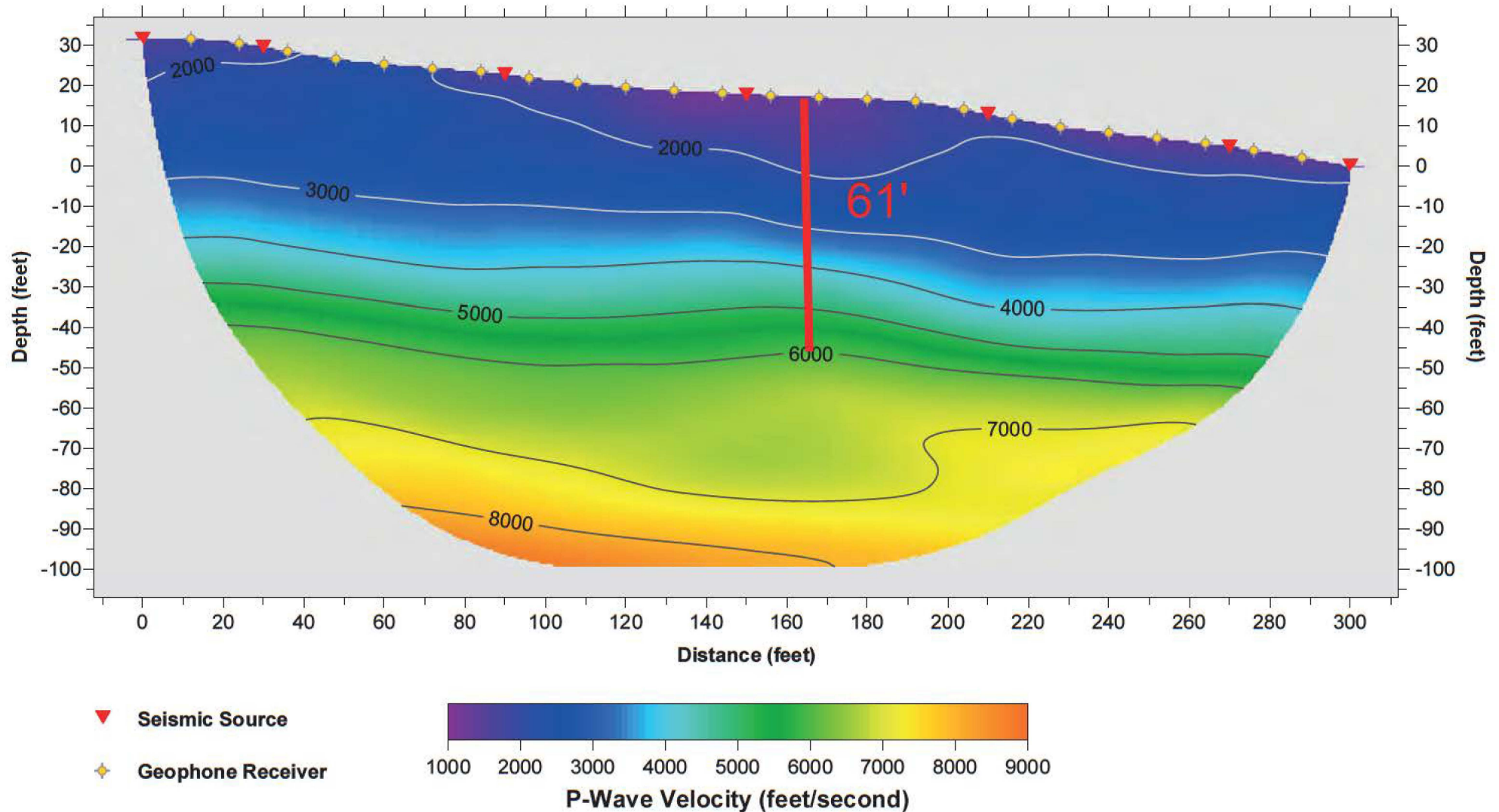
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SEISMIC LINE S-15

South 25° West →

REFRACTION TOMOGRAPHIC MODEL



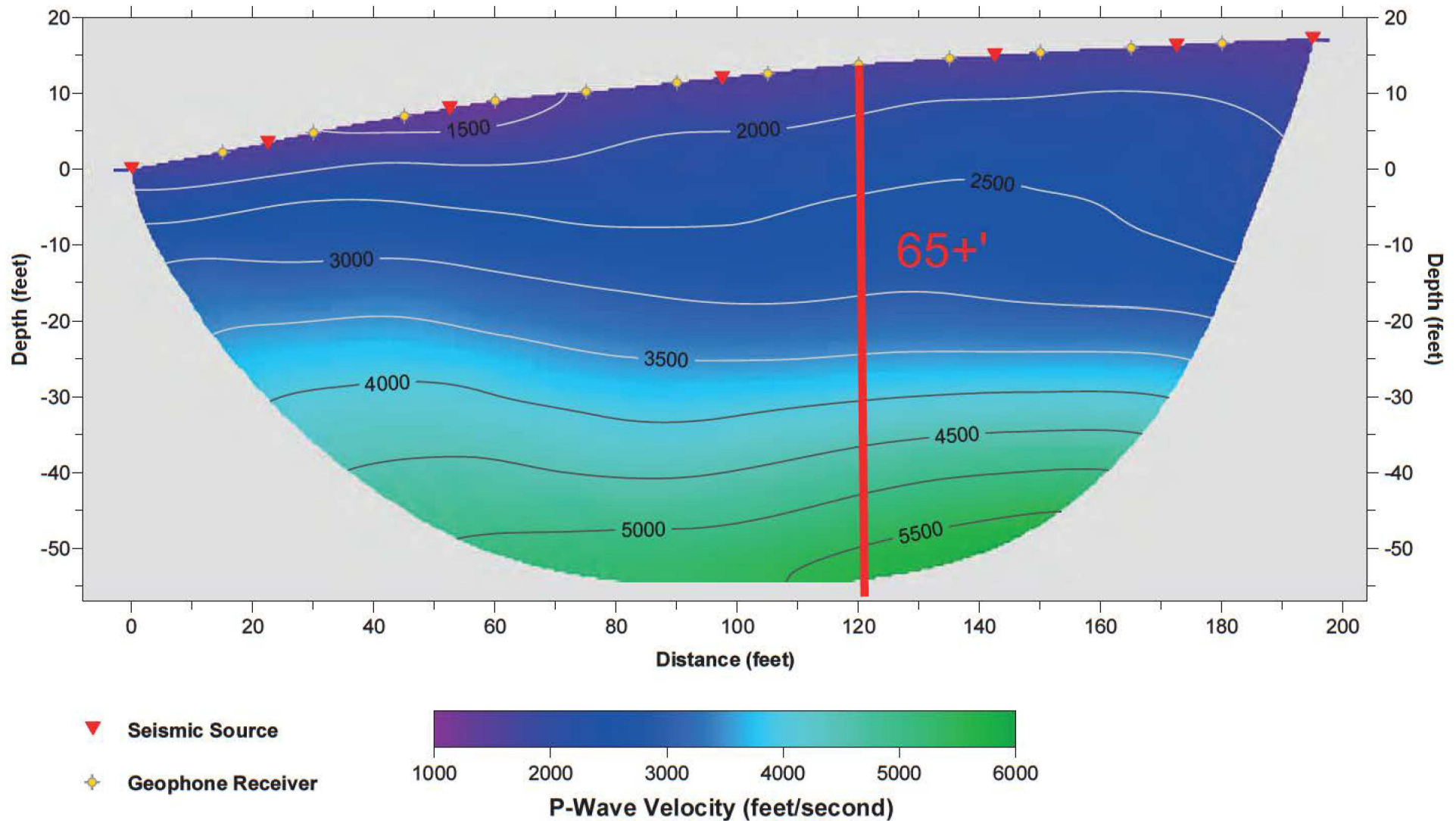
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RMS error 1.1%; Rayfract Version 4.06

SEISMIC LINE S-16

South 85° West →

REFRACTION TOMOGRAPHIC MODEL



SCALE: Vertical Exaggeration 1.25X

RMS error 1.4; Rayfract Version 4.06

APPENDIX D

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 General

- 1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).
- 1.2 Geotechnical Consultant: Prior to commencement of work, the owner shall employ a geotechnical consultant. The geotechnical consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

- 1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

- 2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

- 2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.
- 2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.
- 2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.
- 2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

- 3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.
- 3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.
- 3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

- 4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.
- 4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).
- 4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

- 4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.
- 4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).
- 4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.
- 4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

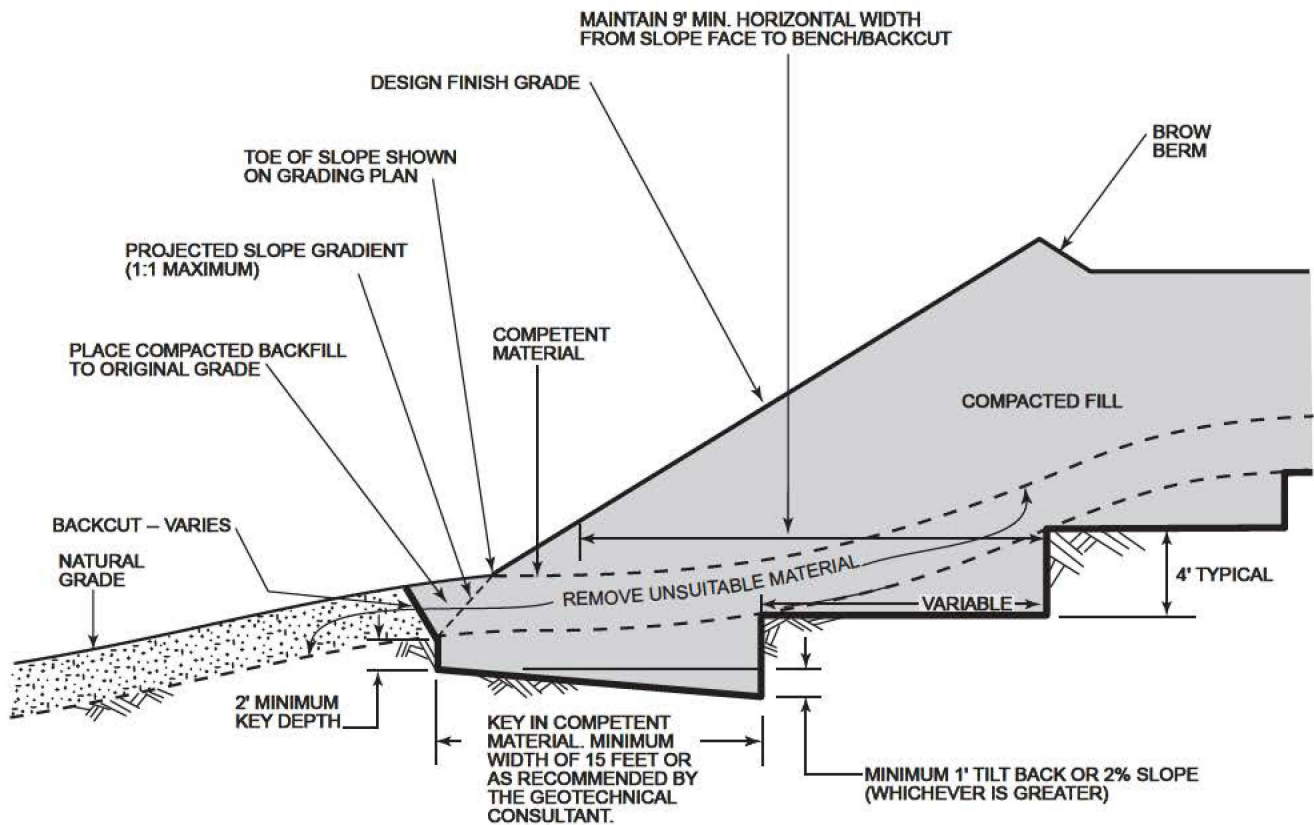
Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

- 7.1 Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.
- 7.2 Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 ($SE > 30$). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum 90 percent of maximum from 1 foot above the top of the conduit to the surface, except in traveled ways (see Section 7.6 below).
- 7.3 Jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.
- 7.4 Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.
- 7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.
- 7.6 Trench backfill in the upper foot measured from finish grade/subgrade within existing or future traveled way, shoulder, and other paved areas (or areas to receive pavement) should be placed to a minimum 95 percent relative compaction unless specified differently by the governing agency.

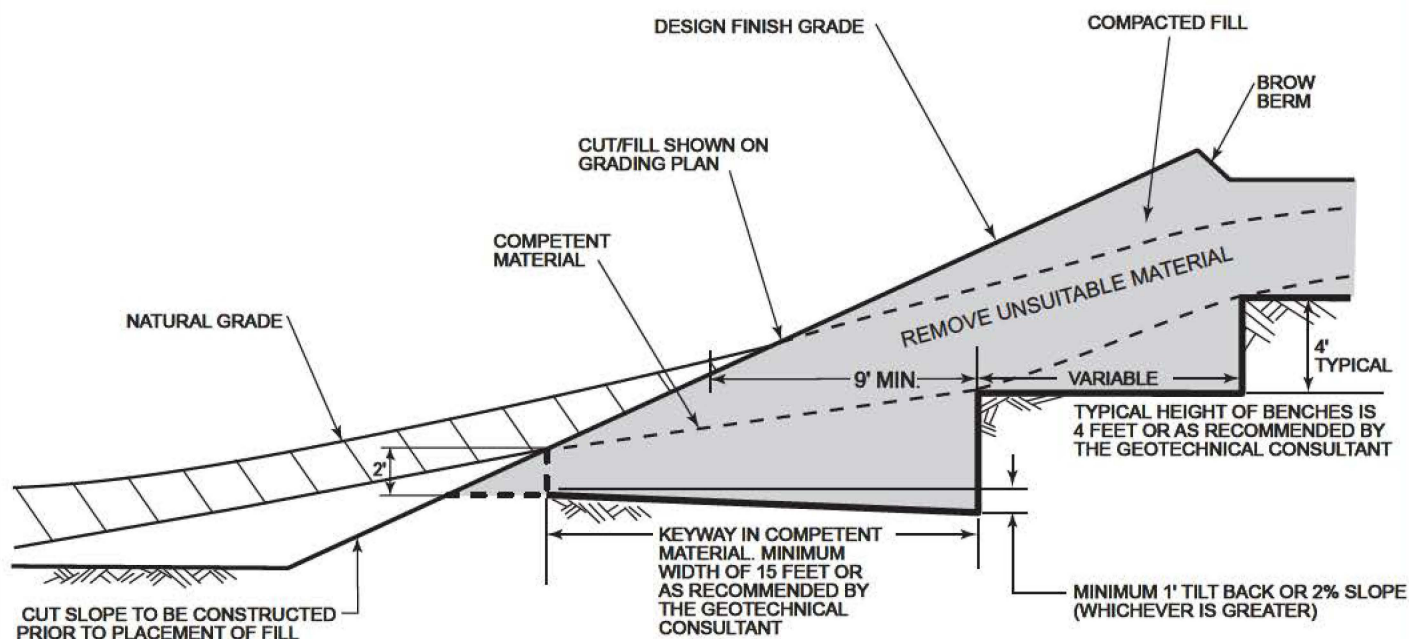


NOTE: BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE SOIL ENGINEER. WHERE THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN SLOPE RATIO, SPECIAL RECOMMENDATIONS WILL BE PROVIDED BY THE GEOTECHNICAL ENGINEER.

FIGURE 1

TYPICAL FILL KEY ABOVE NATURAL SLOPE MINIMUM STANDARD GRADING DETAILS

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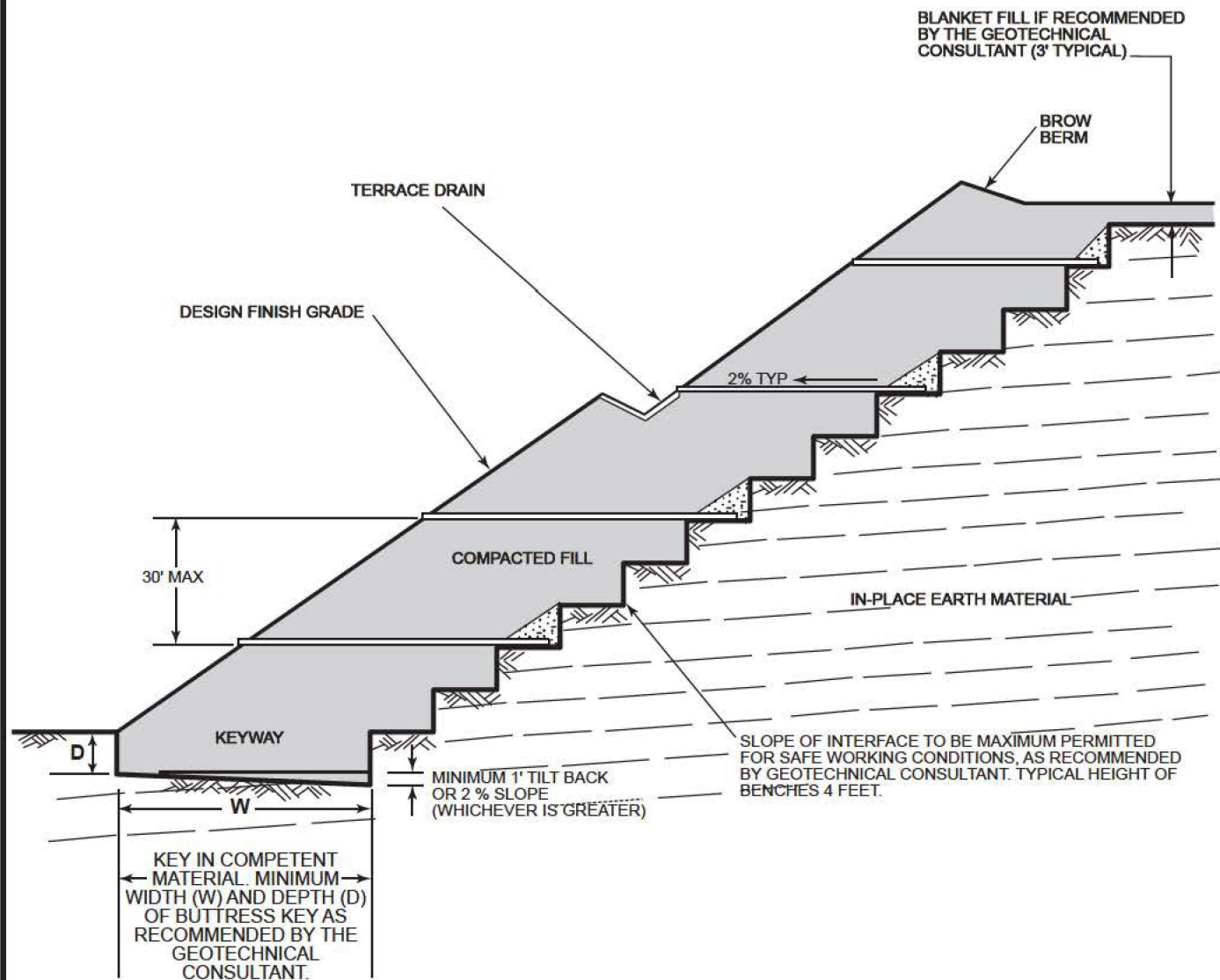


NOTE: THE FILL PORTION OF THE SLOPE SHALL BE COMPACTED AS STATED IN THE PROJECT SPECIFICATIONS.

FIGURE 2

TYPICAL FILL ABOVE CUT SLOPE MINIMUM STANDARD GRADING DETAILS

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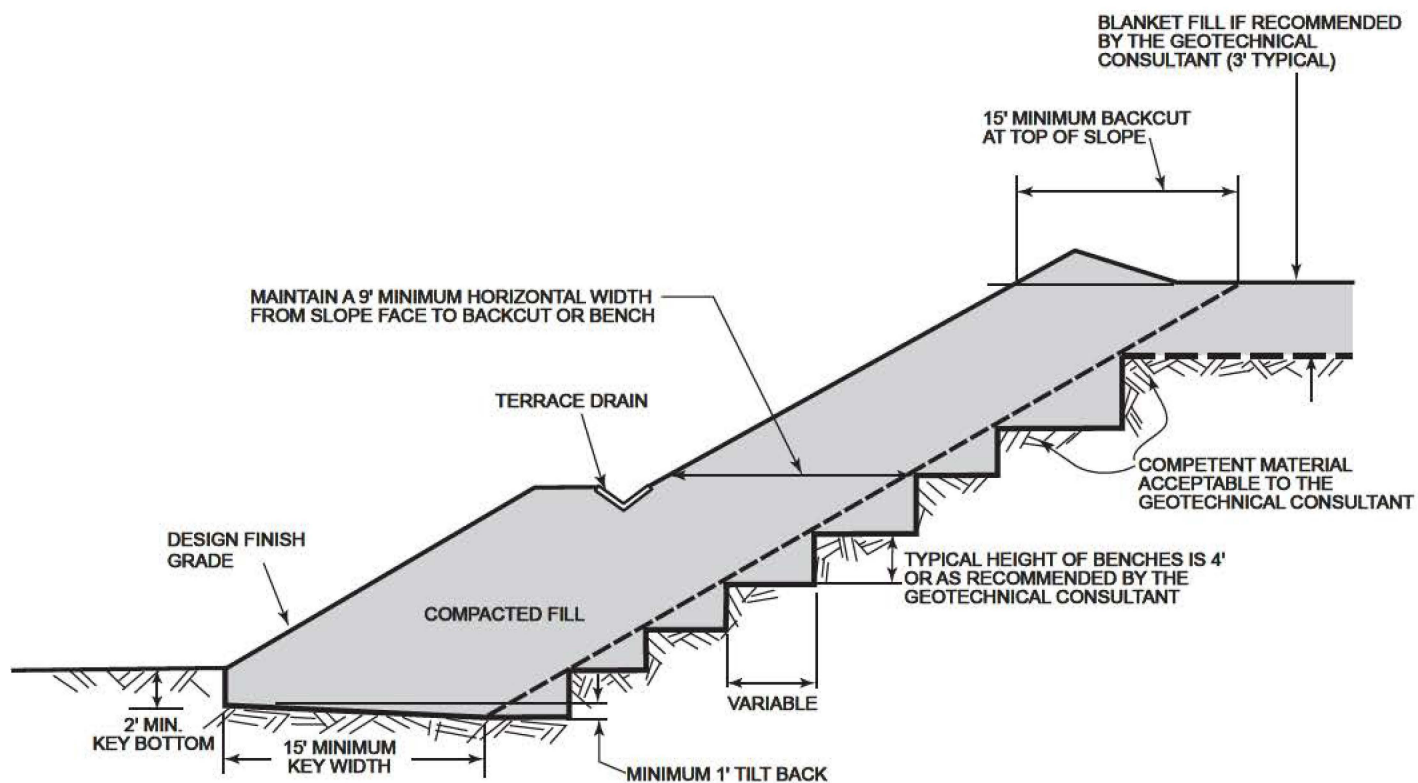


NOTE: SUBDRAIN DETAILS, SEE FIGURE 5.

FIGURE 3

TYPICAL BUTTRESS FILL MINIMUM STANDARD GRADING DETAILS

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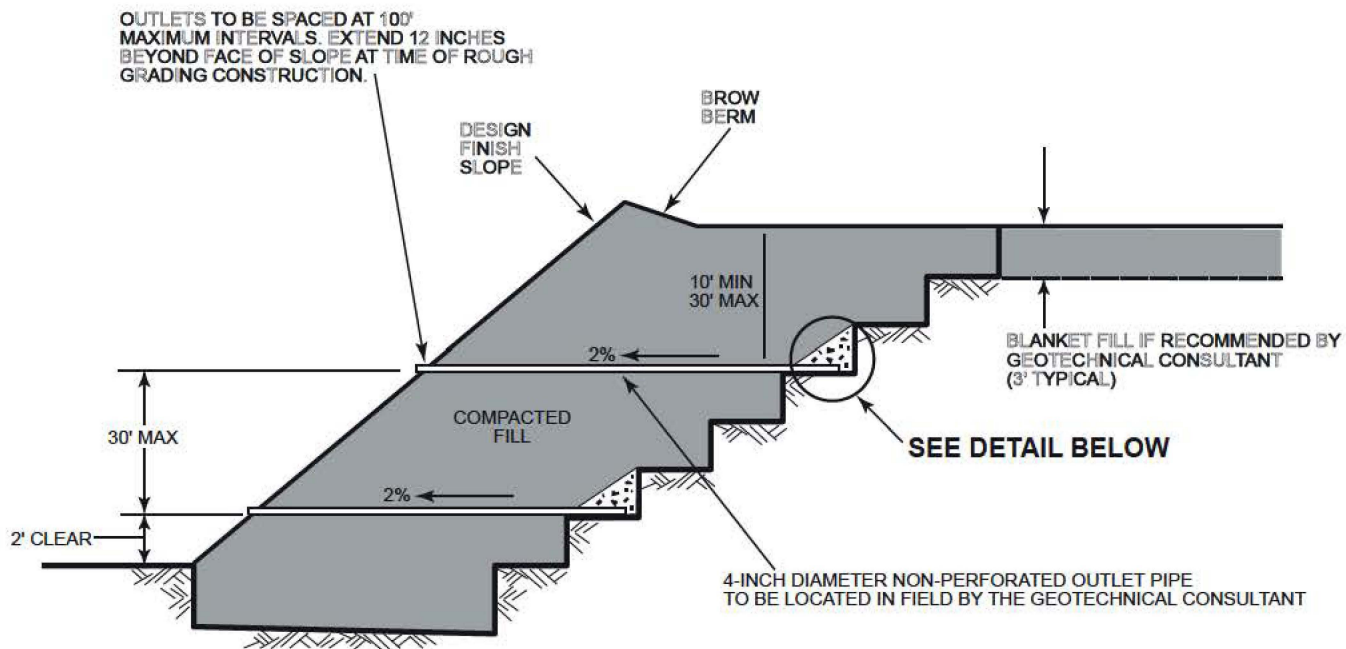


NOTE:
SEE FIGURE 5 FOR TYPICAL SUBDRAIN DETAILS FOR STABILIZATION FILLS

FIGURE 4

TYPICAL STABILIZATION FILL MINIMUM STANDARD GRADING DETAILS

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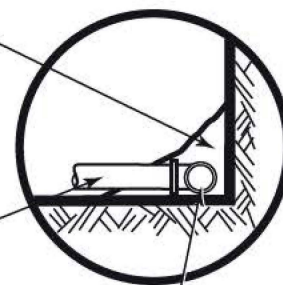
FILTER MATERIAL - MINIMUM OF THREE CUBIC FEET PER FOOT OF PIPE. SEE FILTER MATERIAL SPECIFICATION.

ALTERNATE: IN LIEU OF FILTER MATERIAL, THREE CUBIC FEET OF GRAVEL PER FOOT OF SUBDRAIN (WITHOUT PIPE) MAY BE ENCASED IN FILTER FABRIC. SEE GRAVEL SPECIFICATION, AND FIGURE 6 FOR FILTER FABRIC SPECIFICATION

"GRAVEL" TO CONSIST OF 1/2" TO 1" CRUSHED ROCK PER STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION.

FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

DETAIL



OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW

MINIMUM 4-INCH DIAMETER SCHEDULE 40 ASTM D1527 OR D1785 OR SDR 35 ASTM D2751 OR D 3034. FOR FILL DEPTH OF 90 FEET OR GREATER, USE ONLY SCHEDULE 40 OR EQUIVALENT. THERE SHALL BE A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT.

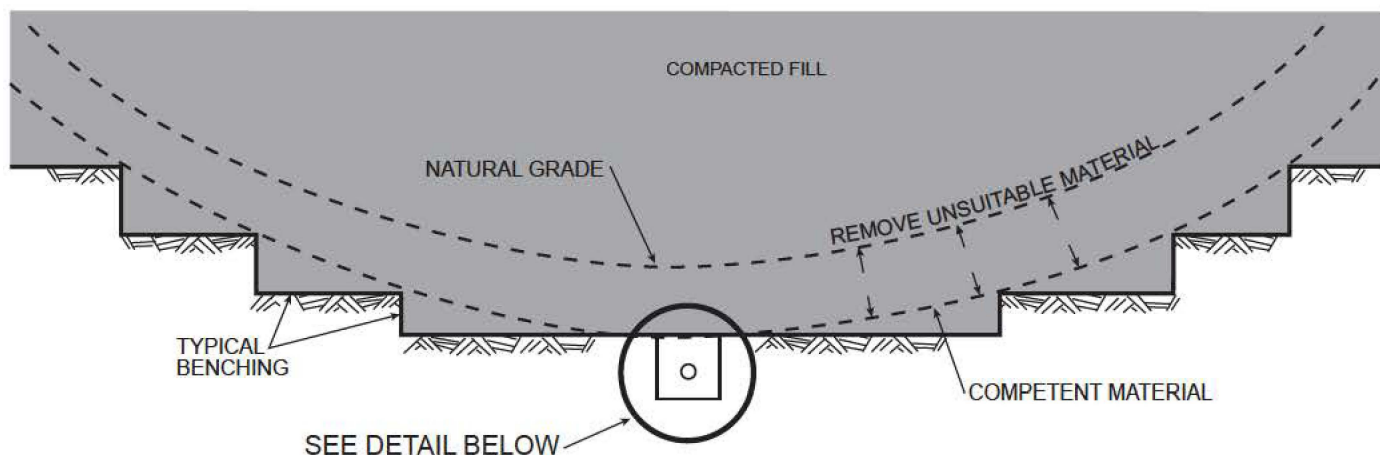
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOTE:
TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

FIGURE 5

TYPICAL STABILIZATION AND BUTTRESS FILL SUBDRAINS MINIMUM STANDARD GRADING DETAILS

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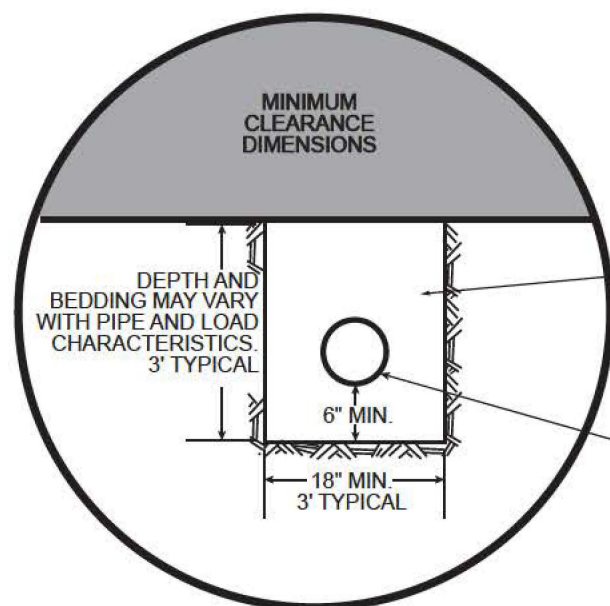
FILTER FABRICS SHALL BE PERMEABLE NON-WOVEN POLYESTER, NYLON, OR POLYPROPYLENE MATERIAL CONFORMING TO THE FOLLOWING:

- | | |
|---|------|
| 1) GRAB TENSILE STRENGTH, POUNDS, MIN. ASTM D 4632..... | 90 |
| 2) ELONGATION, AT PEAK LOAD, PERCENT, MIN. ASTM D 4632..... | 50 |
| 3) PUNCTURE STRENGTH, LBS., MIN. ASTM D 3787..... | 45 |
| 4) COEFFICIENT OF WATER PERMITTIVITY, 1/SEC. ASTM D 4491..... | >0.7 |
| 5) BURST STRENGTH, P.S.I., MIN. ASTM D 3786..... | 180 |

NOTES: DOWNSTREAM 20' OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL

PIPE SHALL BE A MINIMUM OF 4-INCH DIAMETER. FOR RUNS OF 500 FEET OR MORE, USE 6-INCH DIAMETER PIPE, OR AS RECOMMENDED BY THE GEOTECHNICAL CONSULTANT

DETAIL



FILTER MATERIAL - MINIMUM OF NINE CUBIC FEET PER FOOT OF PIPE. SEE FIGURE 5 FOR FILTER MATERIAL SPECIFICATIONS.

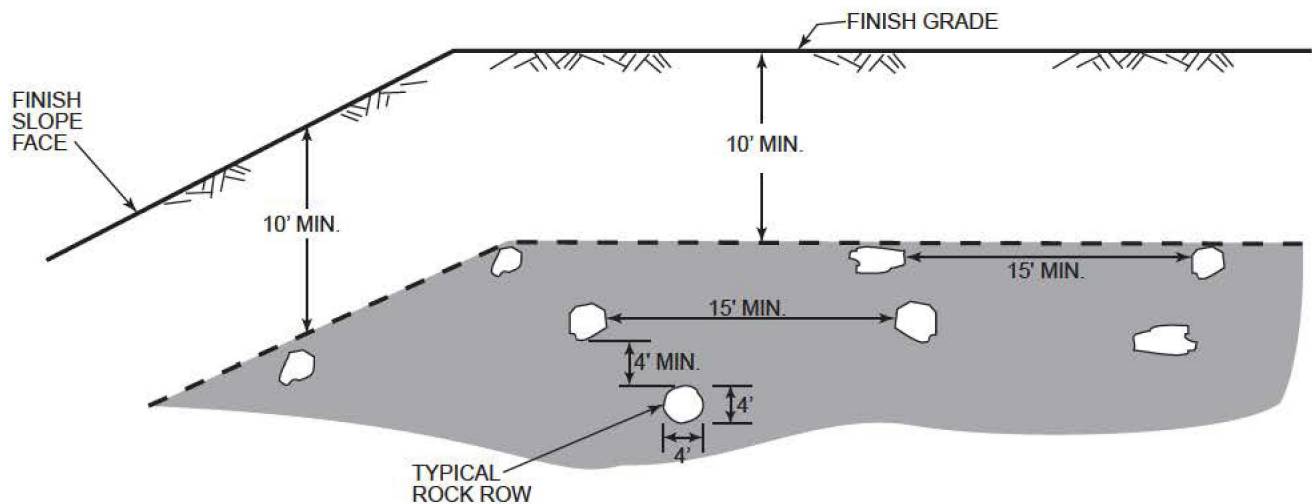
ALTERNATE: IN LIEU OF FILTER MATERIAL, NINE CUBIC FEET OF GRAVEL PER FOOT OF SUBDRAIN (WITHOUT PIPE) MAY BE ENCASED IN FILTER FABRIC. SEE FIGURE 5 TO GRAVEL SPECIFICATION. SEE ABOVE FOR FILTER FABRIC SPECIFICATION. FILTER FABRIC SHALL BE LAPPED MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4 INCH DIAMETER SCHEDULE 40 ASTM D 1527, OR D 1785, OR SDR 35 ASTM 2751 OR D 3034. FOR FILL DEPTH OF 90 FEET OR GREATER, USE ONLY SCHEDULE 40 OR APPROVED EQUIVALENT. THERE SHALL BE A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE.

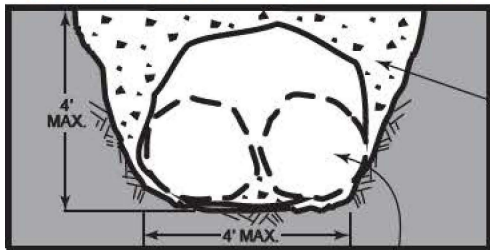
FIGURE 6

TYPICAL CANYON SUBDRAIN MINIMUM STANDARD GRADING DETAILS

NMG
Geotechnical, Inc.

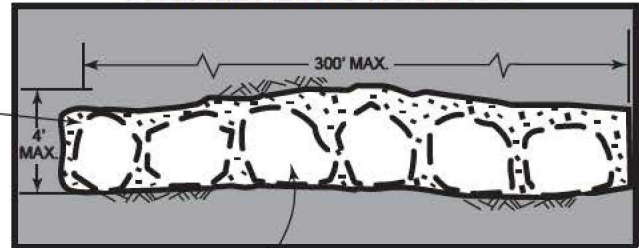


SECTION THROUGH ROCKROW



FILL VOIDS WITH SELECT GRANULAR SOIL PLACED BY WATER DENSIFICATION AND MECHANICAL COMPACTION. NESTING OR STACKING OF OVERSIZE MATERIAL IS NOT ACCEPTABLE.

PROFILE ALONG ROCKROW



PLACE OVERSIZE MATERIAL IN TRENCH. FALSE SLOPE OR CUT SLOT INTO APPROVED MATERIAL. OVERSIZE MATERIAL MAY BE PLACED SIDE BY SIDE IF SIZE PERMITS. (NOT TO EXCEED A WIDTH OF 4 FEET)

NOTES:

- A) OVERSIZE ROCK IS DEFINED AS LARGER THAN 12" IN SIZE (IN GREATEST DIMENSION).
- B) SPACE BETWEEN ROCKROWS SHOULD BE ONE EQUIPMENT WIDTH OR A MINIMUM OF 15 FEET.
- C) THE WIDTH AND HEIGHT OF THE ROCKROW SHALL BE LIMITED TO FOUR FEET AND THE LENGTH LIMITED TO 300 FEET UNLESS APPROVED OTHERWISE BY THE GEOTECHNICAL CONSULTANT. OVERSIZE SHOULD BE PLACED WITH FLATTEST SIDE ON THE BOTTOM.
- D) OVERSIZE MATERIAL EXCEEDING FOUR FEET MAY BE PLACED ON AN INDIVIDUAL BASIS IF APPROVED BY THE GEOTECHNICAL CONSULTANT.
- E) FILLING OF VOIDS WILL REQUIRE SELECT GRANULAR SOIL (SE > 20, OR LESS THAN 20 PERCENT FINES) AS APPROVED BY THE GEOTECHNICAL CONSULTANT. VOIDS IN THE ROCKROW TO BE FILLED BY WATER DENSIFYING GRANULAR SOIL INTO PLACE ALONG WITH MECHANICAL COMPACTION EFFORT.
- F) IF APPROVED BY THE GEOTECHNICAL CONSULTANT, ROCKROWS MAY BE PLACED DIRECTLY ON COMPETENT MATERIALS OR BEDROCK, PROVIDED ADEQUATE SPACE IS AVAILABLE FOR COMPACTION.
- G) THE FIRST LIFT OF MATERIAL ABOVE THE ROCKROW SHALL CONSIST OF GRANULAR MATERIAL AND SHALL BE PROOF-ROLLED WITH A D-8 OR LARGER DOZER OR EQUIVALENT.
- H) ROCKROWS NEAR SLOPES SHOULD BE ORIENTED PARALLEL TO SLOPE FACE.
- I) NESTING OR STACKING OF ROCKS IS NOT ACCEPTABLE.

FIGURE 7

TYPICAL OVERSIZE ROCK PLACEMENT METHOD MINIMUM STANDARD GRADING DETAIL FOR STRUCTURAL FILL

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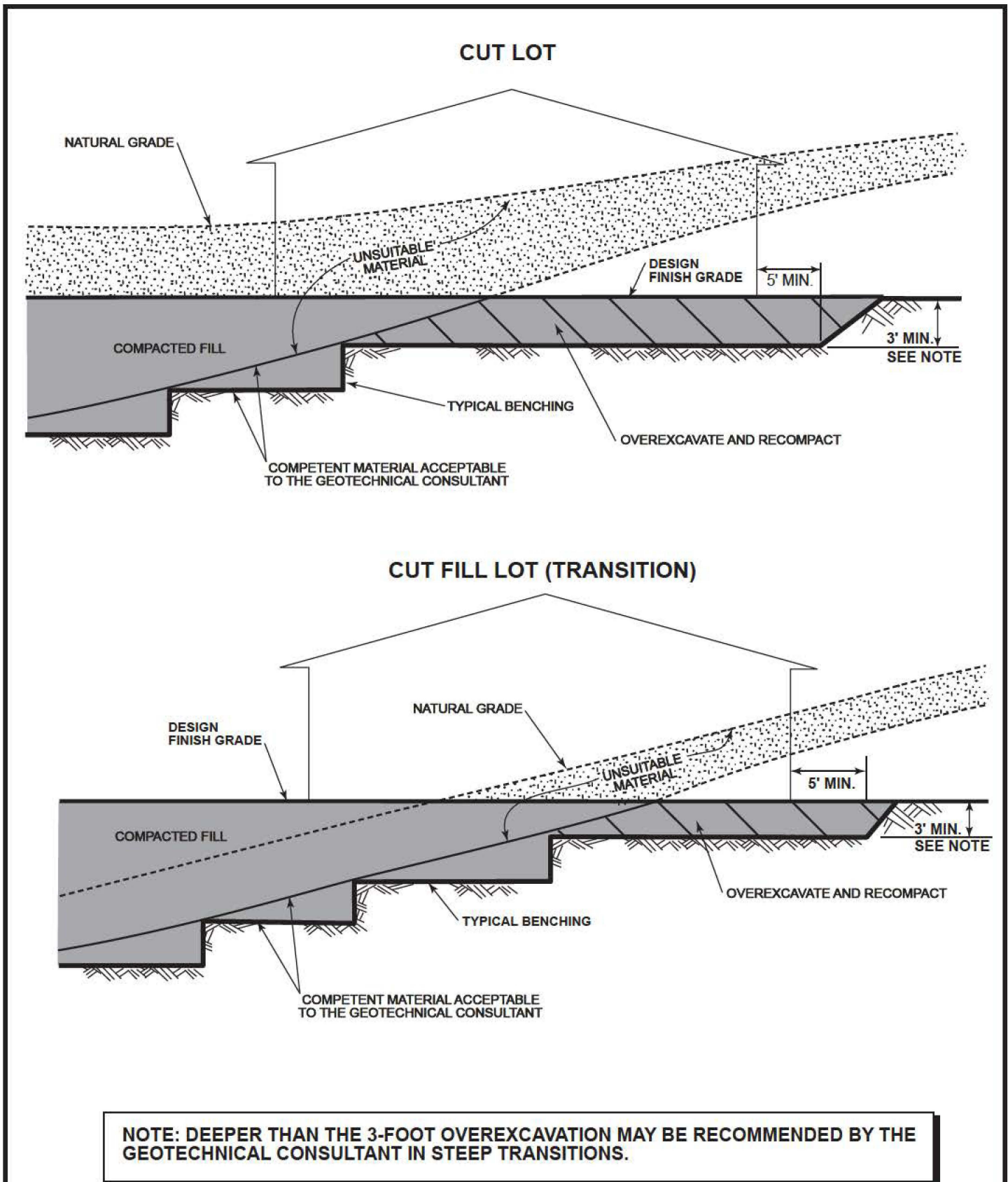
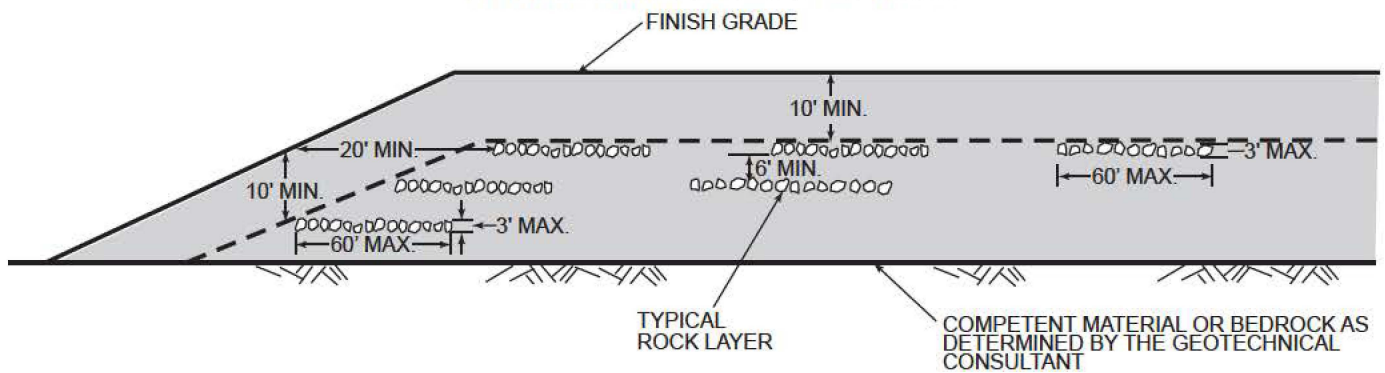


FIGURE 8

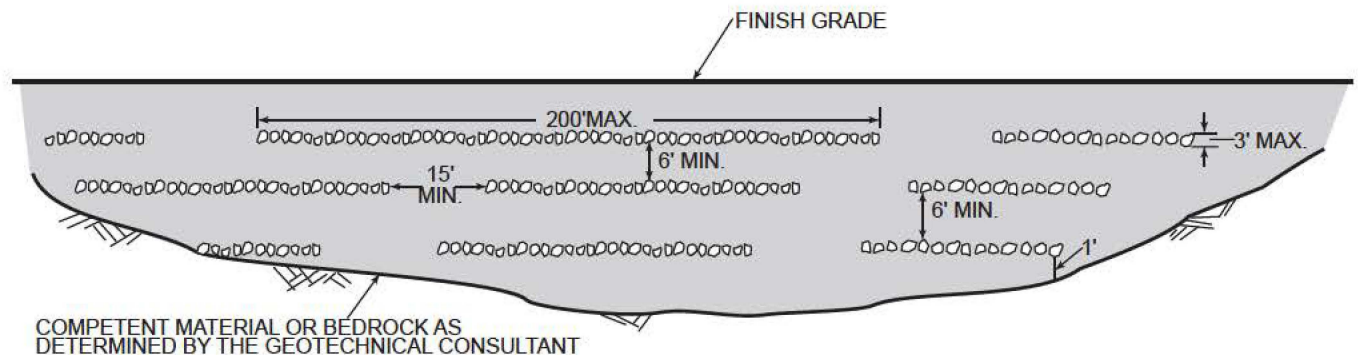
TYPICAL OVEREXCAVATION OF DAYLIGHT LINE MINIMUM STANDARD GRADING DETAILS

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VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

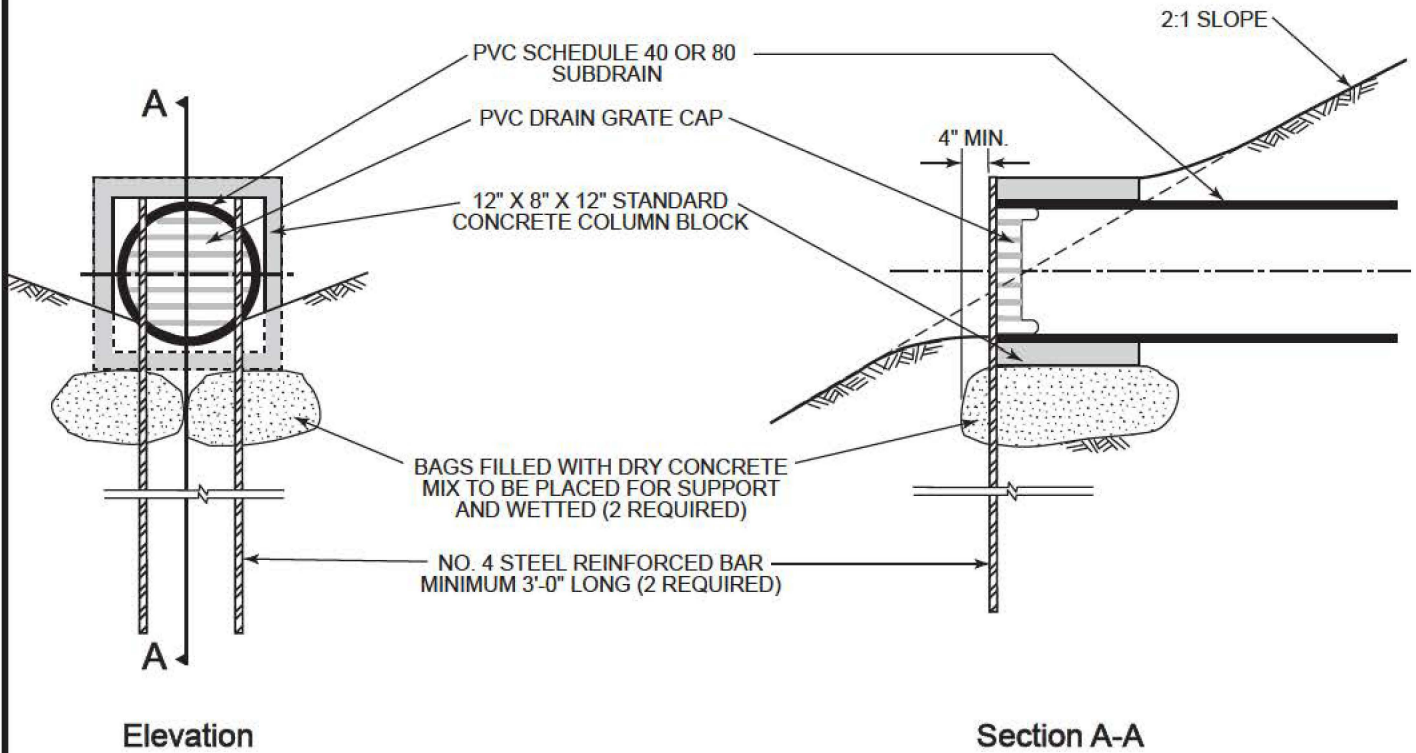
- 1) LAYER METHOD MAY ONLY BE USED IF OVERALL QUANTITY OF ROCK EXCEEDS THE CAPACITY FOR DISPOSAL BY WINDROW METHOD.
- 2) ORIENTATION AND STAGGERING MAY VARY, BUT SHALL BE AS RECOMMENDED BY THE GEOTECHNICAL CONSULTANT.
- 3) THE ROCK LAYER SHALL NOT BE MORE THAN 3 FEET IN HEIGHT. ROCK SHALL NOT BE STACKED OR NESTED.
- 4) EACH ROCK CELL TO BE COVERED WITH A GRANULAR SOIL WITH A MINIMUM SAND EQUIVALENT (SE) OF 30 OR WITH LESS THAN 20% PASSING THE NO. 200 SIEVE AS DETERMINED BY THE GEOTECHNICAL CONSULTANT. THE GRANULAR SOIL IS TO BE FLUSHED, FLOODED, AND JETTED IN THE ROCK LAYER IN ORDER TO FILL THE VOIDS. ADDITIONAL LIFTS OF GRANULAR SOIL SHALL BE PLACED AS NECESSARY TO ALLOW FOR THE FILLING OF VOIDS BY FLOODING, AND JETTING. THE ROCK LAYER SHALL THEN BE COMPACTED WITH A MINIMUM 12-TON VIBRATORY SHEEPSFOOT ROLLER.
- 5) ADEQUACY OF ABOVE PROCEDURE SHALL BE VERIFIED BY THE GEOTECHNICAL CONSULTANT.
- 6) THIS PROCEDURE NOT TO BE COMBINED WITH OTHER METHODS OF ROCK PLACEMENT.

FIGURE 7D

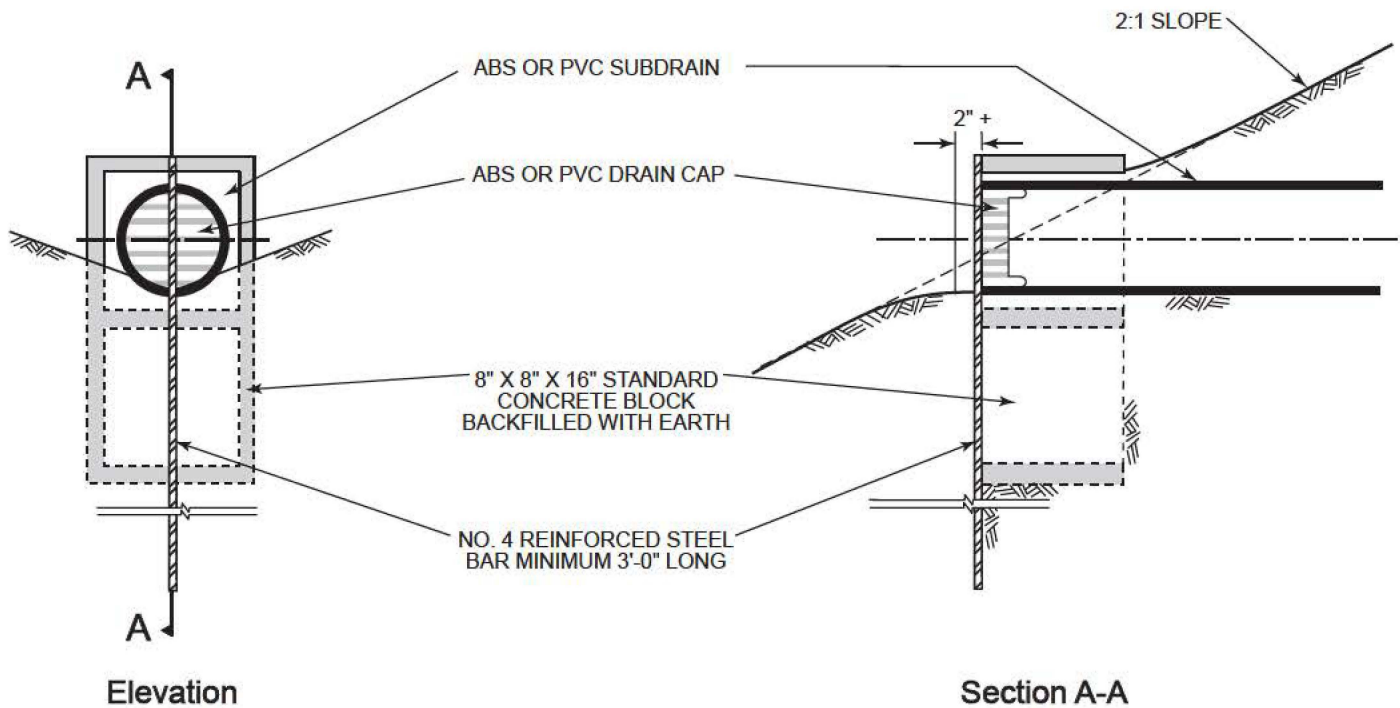
ALTERNATIVE OVERSIZE ROCK PLACEMENT MINIMUM STANDARD GRADING DETAILS FOR STRUCTURAL FILL

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SUBDRAIN OUTLET MARKER - 6" AND 8" PIPE

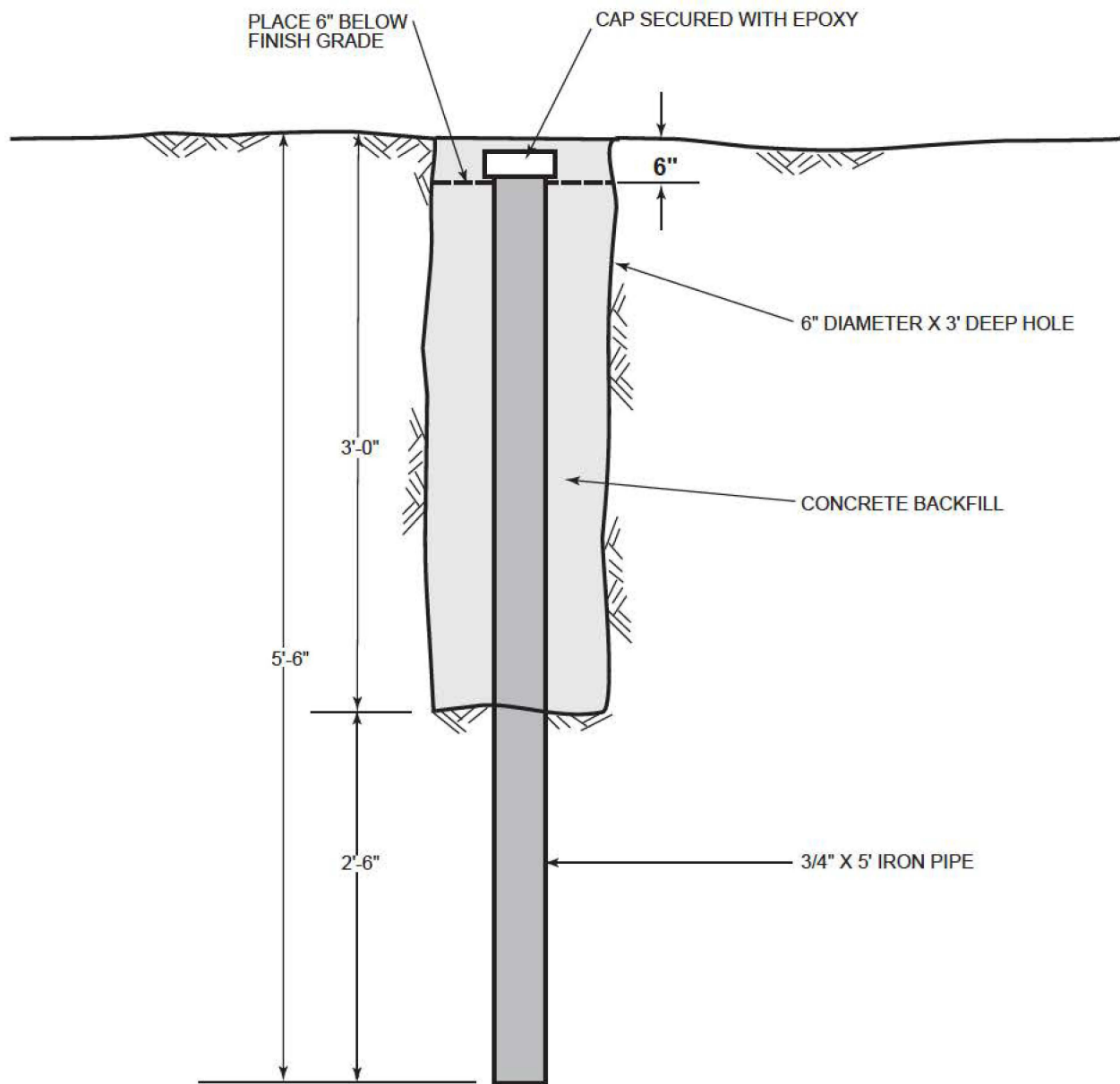


SUBDRAIN OUTLET MARKER - 4" PIPE



**SUBDRAIN OUTLET MARKER
MINIMUM STANDARD GRADING DETAILS**

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TYPICAL SURFACE SETTLEMENT MONUMENT MINIMUM STANDARD GRADING DETAILS

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