Appendix B
Geotechnical
Report

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Preliminary Geotechnical Study Report
for San Ramon Canyon Storm Drain System,
City of Rancho Palos Verdes, California

Prepared For
HARRIS & ASSOCIATES

November 10, 2010

GMU Project No. 10-036-00

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TRANSMITTAL

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34 Executive Park, Suite 150
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DATE: November 10, 2010

PROJECT: 10-036-00

ATTENTION: Mr. Randall Berry

SUBJECT: Preliminary Geotechnical Study Report for San Ramon Canyon Storm Drain System, City of Rancho Palos Verdes

WE ARE SENDING THE FOLLOWING:

One (1) wet signature copy of our "Preliminary Geotechnical Study Report for San Ramon Canyon Storm Drain System, City of Rancho Palos Verdes, California," dated November 10, 2010.

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INTRODUCTION

This report is intended to provide a summary of our preliminary geotechnical investigation and evaluation of the feasibility of design alternatives for the proposed San Ramon Canyon Storm Drainage System project within the City of Rancho Palos Verdes, California. Our report is based on our understanding of the design alternatives prepared by Harris & Associates and presented in their conceptual plans, dated September, 2010 (see References).

SCOPE OF WORK

Our scope of work for the preliminary geotechnical investigation and feasibility evaluation of the design alternatives was generally as described in our proposal to Harris & Associates, dated February 23, 2010.

1. Background Review and Technical Study – including review of available geologic and geotechnical data and historical aerial photographs, surface mapping of San Ramon Canyon, preparation of geologic cross-sections, and attendance at team meetings.

2. Evaluation of Palos Verdes Drive East Switchbacks – including drilling two bucket auger borings on the outside of the switchbacks, completion of a laboratory testing program, and revision of the geologic cross-sections to incorporate the collected data.

3. Evaluation of Tunnel Design Alternative – including drilling two continuous core borings along the conceptual tunnel alignment and performing geophysical logging of the borings, completion of a laboratory testing program, and geotechnical analyses of the data collected. In addition to the scope described in our proposal, one additional bucket auger boring was drilled near the launch pit for the southernmost portion of the tunnel alignment, and corresponding additional laboratory testing and analyses were performed.

4. Evaluation of Canyon Design Alternatives – including drilling one hollow-stem auger boring within San Ramon Canyon just upstream from 25th Street, completion of a laboratory testing program, revision of the geologic cross-sections as required, and geotechnical analyses of the data collected.

5. Deliverables and Technical Support – including preparing both a draft and final preliminary geotechnical report (herein), and attendance at meetings as necessary to facilitate understanding and support of the project from governing agencies.

SITE LOCATION

The project site is located within the eastern portion of the City of Rancho Palos Verdes. In general, the site is bounded by Palos Verdes Drive East (PVDE) on the west, Calle Aventura and Tarapaca Road to the north, the City of Los Angeles boundary to the east, and the Pacific Ocean to the south. It should be noted that the site is bisected by Palos Verdes Drive South, which is named 25th Street within the City of LA. For the purposes of this report, this street is referred to as "25th Street". A Location Map showing the specific limits of the project site is included in this report as Plate 1.

EXISTING SITE CONDITIONS AND HISTORY

In general, the project site is in an undeveloped condition, except for the PVDE and 25th Street roadways. Some areas that bound the site are developed, such as the residences upslope of the upper portions of San Ramon Canyon, and the Palos Verdes Shores mobile home park to the south of 25th Street. The location of the park and other major features are shown on Plate 2.

The major topographical feature within the project site is San Ramon Canyon, which trends generally north-south. Prior to development, this canyon extended to the Pacific Ocean. However, during the construction of 25th Street, the lower portion of the canyon was buried and the flow of water was collected into a storm drain system that begins at 25th Street and extends down to the ocean. Currently, the inlet structure that was the upstream beginning of the system has been buried by debris.

The remaining portions of the project site consist of gentle to very steep slopes, with the steeper slopes found within the canyon. The topography of the project site is generally controlled by the large ancient landslide that comprises the majority of the site. This landslide, the South Shores landslide, is considered dormant and is discussed in further detail in subsequent sections of this report.

The need for this storm drain project has been prompted by the episodic flooding of 25th Street that occurs during moderate to heavy rainfall periods and the concern for the stability of PVDE. The flooding of the 25th Street area is primarily due to the clogged and buried storm drain inlet, which does not collect surface water. During a rain event, surface water flowing down the canyon is carried downstream and directly onto the roadway. In addition, erosion of the canyon and slope movement are contributing to the problem, causing mudflows and debris to be washed down the canyon.

Our review of historical aerial photographs (listed in References) indicates that 25th Street and the associated storm drain under the roadway were constructed prior to 1954. Residential construction at the top of the canyon began in the late 1950s and continued intermittently through the 1990s.

Residential construction south of the project site (i.e., south of 25th Street) occurred in the 1970s. In addition, failure of the slope at the head of San Ramon Canyon and the subsequent repair occurred in the early 2000s. This failure, while outside the influence of the project site, is discussed in a subsequent section of this report.

Areas of increased erosion within San Ramon Canyon were noted in the photos from 1970, 1978, 1979, 1988, and 1992. Surficial failures, slumps, and small landslides were noted on the photos within the canyon. The Tarapaca landslide, located on the east side of the canyon, is first noted on the photos in 1978 as increasing erosion, with scarps observed in the 1988 photos. Further detailed discussion of the Tarapaca landslide is included in subsequent sections of this report.

The buildup of silt and debris within the lower portions of the canyon, upstream from 25th Street, is first observed within the 1988 photos and continues through the present. Based on the erosion observed within the photos and the currently observed site conditions, it appears the canyon has widened over time, with increasingly steepened side walls.

PREVIOUS GEOTECHNICAL INVESTIGATIONS

No site-specific investigations for a storm drain within San Ramon Canyon have been completed prior to the current geotechnical investigation by GMU Geotechnical, Inc. (GMU). However, numerous studies by geologists have been done on the South Shores landslide, and several geotechnical reports for single-family residences have been published for properties upslope of San Ramon Canyon. A complete list of reports and publications reviewed as part of this investigation is included in the References section of this report. In addition, where appropriate, data from these previous investigations, as well as data from regional publications, have been incorporated into this preliminary geotechnical report.

In general, the primary focus of previous studies in the area pertains to the South Shores landslide. These studies conclude that the landslide is approximately 16,200 years old (Ray, 1982), and may have failed in one event. Various authors disagree on the eastern limits of the landslide, and different interpretations of the available data have resulted in several opinions as to the depth and toe of the landslide. Dibblee (1999) maps the toe of the landslide within the bluff face above sea level, while Ray (1982) and others map the toe below sea level and the current beach. It appears that the majority of authors on the subject of the South Shores landslide agree that the landslide is dormant and has not moved in historic time.

Site-specific reports for single-family residential developments were also reviewed for this investigation, including reports by Ehlert (1997, 1998), and T.I.N. Engineering Company (2006) for residences upslope of the project site. Geologic data from these reports, including boring and trenching data, were included in our Geotechnical Map and Cross-Sections (Plates 3, 4, and 5). However, much of the data shown on the geologic maps within thee reports references boring or trench logs that were not available for our review. Where available, these logs are included in Appendix A of this report. Logs that were not available are indicated on the legend of our Geotechnical Map, Plate 3.

PROPOSED DESIGN ALTERNATIVES

Based on our review of the conceptual plans by Harris & Associates (2010), we understand there are two main alternatives for this project, with a third alternative that is considered to be an option for the first two alternatives. These alternatives are described below, and shown generally on Plate 6.

ALTERNATIVE 1

In order to significantly reduce the erosion and flooding within San Ramon Canyon from surface water, Alternative 1 proposes to construct an inlet structure within the canyon which will carry runoff water through a subsurface storm drain system to the west of the canyon, exiting through an outlet structure at the toe of the bluffs at the ocean. The storm drain system would be constructed with tunnel and open trench methods, and would be located entirely within the City of Rancho Palos Verdes. The general location of the storm drain system proposed for Alternative 1 is detailed in conceptual plans prepared by Harris & Associates. In addition to the construction of the storm drain system, a gravity-type buttress would be constructed within the canyon in order to reduce the potential for future deep-seated movement within the actively failing portion of the canyon (i.e., the Tarapaca landslide).

ALTERNATIVE 2

This alternative includes construction of a subsurface drainage system within the canyon, with an inlet structure in approximately the same location as Alternative 1. This structure will collect surface water into a subsurface storm drain system consisting of a 48-inch-diameter pipe system with a 12-inch-diameter underlying subdrain system. Construction of the storm drain system would include placing fill within the majority of the canyon in order to restore the ground surface to "pre-erosion" conditions and to mitigate the over-steepened canyon walls and failing areas. This storm drain system would tie into the existing system that underlies 25th Street. The general location and details of this alternative are presented in the conceptual plans prepared by Harris & Associates.

ALTERNATIVE B (OPTION FOR ALTERNATIVES 1 AND 2)

The third alternative consists of an optional upstream extension of the storm drain system that would apply to either Alternative 1 or 2. This option would consist of connecting the proposed storm drain systems (either within the canyon or to the west of the canyon) to the existing storm drain system for the residences at the head of the canyon. The existing storm drain system currently outlets into the canyon at the toe of the graded cut slope at the head of San Ramon Canyon.

GEOLOGIC EXPLORATION AND SURFACE MAPPING

FIELD EXPLORATION

Our field exploration program was designed to provide preliminary data for evaluation of the feasibility of the design alternatives described above. In order to accomplish this, our investigation consisted of multiple types of subsurface exploration as well as surface geologic mapping. All aspects of our field investigation were performed by a Certified Engineering Geologist.

Surface Mapping

Geologic mapping of exposed materials was performed across the project site. Exposures of ancient and recent landslide debris, alluvium, and bedrock were observed and geologic structure recorded where observed, generally within San Ramon Canyon and at the bluff descending to the ocean at the southern edge of the project site. Our geologic observations and mapping were incorporated into our data and analyses and are shown as mapped on the Geotechnical Map, Plate 3, and where appropriate on our Cross-Sections, Plates 4 and 5.

Subsurface Exploration

Our subsurface exploration program included drilling, sampling, and logging small-diameter hollow-stem, large-diameter bucket auger, and continuous diamond core borings. Each boring type was selected in order to provide the optimal data retrieval methods for the soil and bedrock conditions anticipated, and to provide the appropriate data type for the proposed alternative. Each method of subsurface exploration is discussed below, and geologic discussion of materials encountered is presented in a subsequent section of this report. Detailed logs of each boring are presented within Appendix A of this report. The approximate location of each of these borings is shown on our Geotechnical Map, Plate 3, and the associated geologic structure is shown on the Cross-Sections, Plates 4 and 5, where appropriate. Backfill of all exploratory borings was completed immediately after logging, and consisted of backfilling and tamping with native materials or backfilling with concrete slurry, where appropriate.

Small-diameter Hollow-Stem Auger Boring

One hollow-stem auger boring was drilled within San Ramon Canyon, just north of 25th Street, within the City of Los Angeles easement. This boring was drilled to a maximum depth of 46.5 feet, and was intended to evaluate the recently deposited alluvial materials at the upstream intersection of San Ramon Canyon and 25th Street. Drive and bulk samples were collected and Standard Penetration Tests (SPTs) were performed in order to geotechnically evaluate the materials encountered.

Large-diameter Bucket Auger Borings

Three bucket auger borings were drilled as part of our investigation. Boring DH-1 was located in the southern portion of the project site, near the proposed launch pit area for the storm drain alignment in Alternative 1, and was intended to evaluate the materials to be encountered at the launch pit as well as geologic structure of the materials exposed in the bluff face. This boring was drilled to approximately 103 feet, where refusal was encountered on very hard material. In addition, the boring was downhole logged by a Certified Engineering Geologist to about 93 feet.

Boring DH-2 was located just east of the lower switchback of PVDE, adjacent to the descending slope to San Ramon Canyon, and was intended to evaluate the geologic structure of the materials underlying the switchback. This boring was drilled to 63 feet, where refusal was encountered due to severe caving of highly fractured material. In addition, the boring was downhole logged by a Certified Engineering Geologist to about 55 feet.

Boring DH-3 was located adjacent to the upper switchback, on the east side of the roadway, and was intended to evaluate the geologic structure of the materials underlying the switchback. This boring was drilled to 60 feet, where refusal was encountered due to severe caving of highly fractured material. In addition, the boring was downhole logged by a Certified Engineering Geologist to about 22 feet, where severe caving precluded further logging of the boring. Drive and bulk samples were collected in each of these bucket auger borings in order to geotechnically evaluate the materials encountered. These borings were backfilled with native materials and tamped in place to properly backfill the borings to minimize settlement potential.

Continuous Diamond Core Borings

Two continuous core borings were drilled as part of our investigation. Boring C-1 was drilled on the "inside" of the lower switchback of PVDE, near the conceptual storm drain alignment of Alternative 1. This boring was intended to primarily evaluate the materials within the South Shores landslide at the storm drain location and to evaluate the geologic structure of this area. This boring was drilled to approximately 149 feet. The continuous core samples collected during drilling were logged by a Certified Engineering Geologist, and geophysical testing of the borings was performed.

Boring C-2 was drilled on the southern shoulder of 25th Street adjacent to the City boundary, near the conceptual storm drain alignment of Alternative 1. This boring was also intended to primarily evaluate the materials within the South Shores landslide at the storm drain location and to evaluate the geologic structure of this area. This boring was drilled to approximately 104 feet. The continuous core samples collected during drilling were logged by a Certified Engineering Geologist, and geophysical testing of the borings was performed.

Geophysical Testing

The geophysical testing performed for Borings C-1 and C-2 included caliper measurements, optical televiewer, and Suspension P- and S-wave velocities. It should be noted that during the logging of Boring C-1, water levels would not rise above 103 feet despite water added to the boring by the drillers via a gravity hose from a water truck. Therefore, Suspension logging could not be performed above 103 feet. The results of the geophysical testing, performed by Norcal Geophysical Consultants, Inc., are attached to this report as Appendix D, and are incorporated into our findings, analyses, conclusions, and recommendations as discussed in this report.

LABORATORY TESTING PROGRAM

Our laboratory program was designed to include testing on representative samples of all geologic materials encountered in order to provide a preliminary geotechnical database for the project site in light of the three alternatives. Our testing program included geotechnical index testing of typical onsite soils as well as direct shear testing of a variety of materials at critical locations to provide a compilation of strength values for the onsite materials. Detailed discussion of each type of testing performed as well as testing results are presented in Appendix B of this report. In addition, further discussion of testing results is included in subsequent sections of this report.

GEOLOGIC FINDINGS

REGIONAL GEOLOGY

Published regional data and our experience in the Palos Verdes Peninsula indicate the peninsula is underlain by Tertiary sedimentary units over basement rock of the Catalina Schist. These geologic materials have been uplifted over time through folding and faulting to create a large-scale anticline that comprises the peninsula, generally trending northwest-southeast. Tectonic uplift in the area may be primarily due to movement on the Cabrillo and Palos Verdes fault zones. Quaternary sediments overlie the Tertiary materials in much of the lower portions of the peninsula due to deposition of sediments by wave action during uplift and through sediment deposition due to gravity, erosion, or in situ weathering.

The geologic features of primary interest within the Palos Verdes Peninsula are the numerous landslides that exist mainly on the ocean (southwesterly) side of the peninsula, generally coincident with southwesterly dipping regional bedding. The two most significant landslide features in the Palos Verdes area are the Portuguese Bend landslide, located approximately 2 miles west of the project site, and the South Shores landslide, located partially within the project site. Further discussion of landsliding and impacts to the project alternatives are discussed in a subsequent section of this report.

MATERIALS ENCOUNTERED

Geologic soil and bedrock materials encountered during our field investigation are described below and within our boring logs. In addition, the lateral extent of these materials is shown on the Geotechnical Map, Plate 3. The geologic structure of these materials is shown on the Cross-Sections, Plates 4 and 5.

Topsoil

Topsoil was observed during our field investigation as a thin veneer across much of the project site. While topsoil was not encountered within our borings, it was observed within drill pad and access road excavations, as well as during surface mapping of the site. Where observed, the topsoil consisted of dark brown silty clay, dry to damp, with no soil structure. Due to the thin nature of the topsoil (i.e., less than three feet in thickness), this geologic unit is not shown on our Geotechnical Map or Cross-Sections.

Recent Alluvium (Qal)

These materials are generally located within San Ramon Canyon, on the canyon floor and in a relatively thick deposit on the northern side of the intersection of the canyon and 25th Street. Where encountered in Boring DH-4, and during surface mapping, the recent alluvium generally consisted of dark brown clay with fine- to medium-grained sand. These materials were generally moist, very soft to soft, with scattered to abundant bedrock fragments and organic materials (i.e., plant debris). The thickest deposit of these recent alluvial materials was found to be about 31 feet thick.

Artificial Fill (Qaf)

These materials are generally located underlying and adjacent to the paved roadways, PVDE, and 25th Street and, as such, were likely placed during grading of these roads pre-1950s. Where observed, these fill materials were dark brown, dry to damp, soft to firm silty clay and sandy silt with fragments of bedrock. Visible lifts were not observed within the downhole logged boring, DH-3, nor were they observed within the core samples recovered within C-2. The maximum thickness of these fill soils was observed to be about 18 feet. However, there may be deeper fill soil deposits within the project site, particularly adjacent to or underlying roadway areas.

Older Alluvium (Qoal)

Deposits of older alluvium were observed within the upper portions of San Ramon Canyon during surface mapping. Where observed within the canyon bottom and sidewalls, these materials consisted on dark brown clayey silt with scattered to abundant bedrock fragments and rare charcoal fragments. These soils were moderately well-developed, with a blocky to columnar structure and local porosity. Structure within these soils was difficult to identify, with some local areas showing subtle textural layers. Given that no borings were drilled within the older alluvium, the maximum thickness of these deposits is unknown; however, it is estimated to be less than 50 feet.

Recent Landslide Debris (Qlsr)

Recently failed materials derived from bedrock and ancient landslide debris were observed during our field exploration on the east wall of San Ramon Canyon. These materials are referred to in geologic publications as the Tarapaca landslide. This landslide is considered to be actively moving. Where observed during surface mapping, the materials of the Tarapaca landslide consist of loose bedrock fragments up to cobble-sized with a soil matrix. Pockets of topsoil with organic debris were observed within the landslide mass. Further discussion of this recent landslide is provided within the "Landslides and Geologic Structure" section of this report.

Ancient Landslide Debris (Qols)

These materials, known as the South Shores landslide, underlie the majority of the project site, and were encountered within all of our borings. Where observed, these materials consisted of remnant blocks of bedrock up to 10 feet thick within a silty clay matrix. These materials are varicolored, soft to hard, dry to moist, and contain blocks of siliceous siltstone that can be very hard. Further discussion of the South Shores landslide is provided within the "Landslides and Geologic Structure" section of this report.

Altamira Shale Member, Monterey Formation (Tma)

Bedrock of the Altamira Shale member of the Monterey Formation underlies the project site at depth, and is exposed within portions of San Ramon Canyon. Where observed, the Altamira Shale member consisted of interbedded siltstone and siliceous siltstone with tuffaceous siltstone, bentonitic tuff, and bentonite. These beds are generally thinly to thickly bedded, planar, with some local soft sediment deformation. The materials are generally gray to olive brown, damp to moist, firm to very hard, with scattered fracturing and jointing. Further discussion of the geologic structure of the bedrock underlying the project site is included in the section below.

LANDSLIDES AND GEOLOGIC STRUCTURE

Regional geologic publications and site-specific geotechnical reports for properties adjacent to the project site indicate the area around the project site may form a geologic "bowl" structure. Bedding inclinations to the north of the site are generally oriented towards the south. Bedding inclinations to the east of the site are generally oriented to the west, and inclinations west of the site are generally oriented to the east. This synclinal geologic structure likely contributed to and controlled the lateral extent of the failure of the South Shores landslide, which dominates the project site.

The South Shores landslide is considered to be approximately 16,200 years old, and failed as a block glide type failure (Ray, 1982). While the authors of publications on the South Shores landslide agree the landside is dormant, there is some disagreement on the limits of the landslide, in particular the eastern edge of the landslide. Some geologists include the currently active Tarapaca landslide as part of the South Shores landslide, while others map the active landslide as originating upslope of the limits of the dormant landslide mass.

Our interpretation of the limits of the South Shores landslide are based on our review of existing geologic data, our observations during our field investigation, and our review of historic aerial photographs. All of our borings encountered the ancient landslide debris, with all three of the large-diameter borings excavated entirely within the landslide. Observations made during downhole logging suggest the South Shores landslide has variable composition, depending on location within the landslide mass. Borings drilled in the upper middle of the landslide (Borings DH-2 and DH-3) encountered large remnant blocks of siltstone and siliceous siltstone that appeared to be highly fractured, sheared, and laterally discontinuous. Continuous beds of bentonite or bentonitic tuff were not observed within either of these two borings. The geologic structure within the landslide mass in the area of the lower switchback of PVDE generally consists of discontinuous remnant fragments and small blocks of bedrock within the debris matrix. The geologic structure of the landslide mass in the area of the upper switchback appears to be more continuous, but moderately to severely folded with some faulting and discontinuities.

Our large-diameter boring DH-1 was drilled near the mapped toe of the South Shores landslide, at the top of the bluff above the beach. Observations made during downhole logging of this boring suggest this area is comprised of generally intact bedrock materials with continuous, planar bedding. The materials were moderately to rarely fractured, with little to no shearing observed. Bentonite or bentonitic tuff beds were not observed within this boring. Surface mapping along the bluff below indicates this continuous, intact bedrock material continues along the bluff face within the project area. Based on this limited subsurface data and the surface geologic mapping, it appears that this lower area may be either a separate, older landslide that failed as a generally intact block, or intact bedrock that has not failed as previously thought. However, very limited data was collected during our investigation, and the presence of a deep-seated landslide rupture surface as previously published cannot be ruled out.

Our Cross-Section 7-7' illustrates the general structure of the older landslide and the postulated rupture surfaces. At least two significant rupture surfaces may exist; one rupture surface at the base of the upper, chaotic debris, and a basal rupture surface at the postulated landslide/bedrock contact below the relatively continuous material noted in DH-1 and the bluff face.

The Tarapaca landslide appears to have failed on a continuous, planar bedding plane surface within the Altamira Shale bedrock east of the South Shores landslide. It is our opinion that the Tarapaca landslide is not a part of the South Shores landslide, as discussed above. As discussed in the "Slope Stability" section of this report, this bedding plane surface does not appear to be a bentonite bed, based on back-calculations performed of the landslide. The most likely scenario for the failure of the Tarapaca landslide is an over-steepening of the canyon walls, resulting in a "daylighted" adverse bedding condition. Given the steep nature of the failure plane, and the continuous erosion of the toe of the landslide by surface water flow down the canyon, movement of the Tarapaca landslide is expected to continue. This episodic movement and failure of the landslide material into the canyon bottom is causing increased erosion of the opposite canyon walls, as discussed below.

The failure that occurred at the head of the San Ramon Canyon area in the early 2000s occurred due to undercutting of oversteepened bedrock, and was subsequently repaired by grading a buttress fill and installing a new storm drain system outlet. It should be noted that this failure area, while shown on our Geotechnical Map, is not within the project site. None of the proposed design alternatives will adversely impact this repaired area.

EROSION

While erosion due to wind and water is a common geologic phenomenon over all of southern California, the impacts of water-driven erosion are significant within the project site in the area of San Ramon Canyon. Erosion within this canyon ranges from moderate to severe. The areas of severe erosion are generally in the area of the Tarapaca landslide and downstream.

The episodic and active downslope movement of the Tarapaca landslide is forcing the flowline of the canyon to shift westerly, causing increased erosion of the western walls of the canyon. These areas are directly downslope of the switchbacks of PVDE, in particular the lower switchback. Erosional scars can be seen on the topographic map used as the base for our Geotechnical Map, Plate 3. Based on our review of historical aerial photographs and our experience in the Palos Verdes area, it appears that these areas of the canyon are eroding at an average rate of about 5 feet per year. Continued annual erosion of these areas may cause stability issues with PVDE. Further discussion of the current and future stability of PVDE is discussed in subsequent sections of this report.

Moderate to severe erosion of the canyon walls and floor due to heavy flow of surface water and flash flooding during rains has caused deep cutting of the canyon, in some areas generating vertical cuts up to 30 feet in height. Instability of these cuts is triggering surficial failures and topple of the vertical walls. Further discussion of the impact of erosion on the Tarapaca landslide and the PVDE switchbacks is provided in other sections of this report.

GROUNDWATER

In general, groundwater was not observed during our investigation. Boring C-1 encountered water at 103 feet; however, this water appeared to be seepage or a perched zone, as samples collected at lower depths were not saturated. The hollow-stem and bucket auger borings did not encounter significant seepage or groundwater. Surface mapping during our investigation did not encounter surface water within San Ramon Canyon; however, it is likely water flows in this canyon during the winter months. Further exploration will likely be required to evaluate the impact of groundwater on the proposed Alternative 1 storm drain alignment. The impact of ground and surface water on Alternatives 2 and 3 will be greatly dependent on the time of year work was performed and the rainfall patterns at the time of work.

SEISMIC HAZARDS

Faulting, Ground Rupture, and Seismic Shaking

The site is not within an Alquist-Priolo Earthquake Fault Zone, and no known active faults are shown on current geologic maps as crossing the site. The nearest known active fault is the Palos Verdes fault, which is located approximately 5.4 kilometers from the site and is capable of generating a maximum earthquake magnitude (M_w) of 7.3. The site is also located within 15.6 kilometers of the Newport-Inglewood fault, which is capable of generating a maximum earthquake magnitude (M_w) of 7.1. Given the proximity of the site to these and numerous other active and potentially active faults, the site will likely be subject to earthquake ground motions in the future.

In order to evaluate to evaluate the likelihood of future earthquake ground motions occurring at the site, a probabilistic seismic hazard analysis (PSHA) of horizontal ground shaking was performed using the commercial computer program EZ-FRISK ver. 7.43. The PSHA utilized seismic sources and next generation attenuation (NGA) equations consistent with the 2008 USGS National Seismic

Hazard Mapping Project. Assuming a risk level of 10 percent probability of exceedance in 50 years (i.e., ~475 year ARP), the PHGA is 0.35g.

Seismically-Induced Landsliding

Given that the site is predominately underlain by a large, dormant landslide, and that the existing walls and slopes of San Ramon Canyon are generally over-steepened due to erosion, the potential for further landsliding due to a large seismic event is high. However, the three design alternatives proposed are intended to reduce the rate of erosion within the canyon, reduce the flow of water and debris down canyon, and reduce the movement of the Tarapaca landslide. Therefore, construction of any of the design alternatives would likely reduce the potential impact of seismically-induced landsliding.

Liquefaction and Lateral Spreading

Given the depth to groundwater and the well-consolidated nature of the landslide and bedrock materials on site, the potential for liquefaction and lateral spreading of these materials is low. However, localized areas where the canyon is underlain by recent alluvium or colluvium may be subject to these seismic hazards should these surficial soils be saturated at the time of the seismic event.

Tsunami

Based on our review of the Torrance/San Pedro Quadrangle of the Tsunami Inundation Map for Emergency Planning prepared by the California Geological Survey (CGS, 2009), the area at the toe of the bluff within the project site may be susceptible to tsunami inundation. Therefore, the storm drain outlet structure of Alternative 1 would be susceptible to impact by tsunami during a seismic event.

Seiche

Given that a seiche, by definition, is restricted to a confined body of water, and no confined or semi-confined bodies of water are found on the project site or upstream of the project site, the probability of impact from a seiche is considered to be nil.

Seismic Design

No active or potentially active faults are known to cross the site; therefore, the potential for primary ground rupture due to faulting on-site is very low to negligible. However, the site will likely be subject to seismic shaking at some time in the future.

Wall and Tunnel Design

The above PSHA derived PGA should be considered in the design of tunnel and retaining elements at the site.

Building Structure Design

Site-specific seismic design parameters were determined using the USGS computer program titled "Seismic Hazard Curves and Uniform Hazard Response Spectra, Version 5.0.9a." The site coordinates used in the analysis were 33.7295° North Latitude and 118.3297° West Longitude. On-site structures should be designed in accordance with the following 2007 CBC criteria:

Parameter	Factor	Value
Mapped Spectral Response Acceleration (0.2 sec Period)	S_S	1.61g
Mapped Spectral Response Acceleration (1.0 sec Period)	S_1	0.66g
Site Class	Site Class	D
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.5
Maximum Considered Earthquake Spectral Response Acceleration(0.2 sec Period)	S_{MS}	1.61g
Maximum Considered Earthquake Spectral Response Acceleration(1.0 sec Period)	S_{M1}	0.98g
Design Spectral Response Acceleration (0.2 sec Period)	S_{DS}	1.07g
Design Spectral Response Acceleration (1.0 sec Period)	S_{D1}	0.66g

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2007 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

GEOTECHNICAL ENGINEERING CHARACTERISTICS

GEOPHYSICAL TESTING

P-wave velocities from the geophysical testing performed for Borings C-1 and C-2 ranged from 3458 fps to 5860 fps (Appendix D). These velocities are characteristic of weathered shale. Structural data obtained from geologic logging of the core samples and the OPTV Image logs was utilized in the determination of a Rock Mass Rating (RMR).

GEOMECHANICS CLASSIFICATION (RMR)

The geomechanics classification of the rock mass rating (RMR) system was developed on the basis of experience in shallow tunnels in sedimentary rocks. The purpose of the RMR is to classify the rock into groups with specific characteristics relative to tunnel performance and support requirements. RMR values are based on six parameters: 1) uniaxial compressive strength of the intact rock material, 2) rock quality designation, 3) joint or discontinuity spacing, 4) joint condition, 5) groundwater condition, and 6) joint orientation (i.e., relative to the tunnel alignment). Points are assigned for each category and added numerically to obtain an overall RMR for the rock mass that can be correlated with several tunneling characteristics.

Uniaxial Compressive Strength

Based on Schmidt hammer readings taken from the rock cores, unconfined compressive strengths for the remnant blocks of siltstone within the South Shores landslide and intact siltstone of the Altamira Shale member of the Monterey Formation is expected to range from about 100 to about 7000 psi depending on the degree of weathering and disturbance. This corresponds to RMR ratings ranging from 0 to 7.

The above uniaxial compressive strengths are not representative of hard siliceous zones within the South Shores landslide which may be encountered during tunneling and excavation. These zones may have unconfined compressive strengths of in the range of 7,500 to 15,000 psi.

Rock Quality Designation (RQD)

Calculated values of RQD are contained on the logs for the continuous core borings. RQD values range from 0 to 86. In general, the upper 40 to 60 feet indicates RQD values of zero with a range of higher values below. This is summarized in the following table.

Core Hole	Tunnel Invert Depth	Depth Range	RQD Range	Weighted RQD Average
C-1	-	0-39 feet	0-13	1
C-1	88	39-149	0-86	33
C-2	50	0-58	0	0
C-2	-	58-104	0-44	19

Based on the above table, RMR ratings for RQD would range from 3 to 8 (i.e., corresponding to weighted RQD values ranging from 0 to 33).

Joint or Discontinuity Spacing

Joint spacing ranges from about 6 inches to about 2 feet in the area of the proposed tunnel for Alternative 1. Consequently an RMR rating of 8-10 can be assumed.

Joint Condition

The joint condition ranges from "slightly rough and moderately to highly weathered, wall rock surface separation < 1mm" to "slickensided wall rock surface or 1-5mm thick gouge or 1-5mm wide continuous discontinuity". These conditions correspond to RMR ratings of 10-20.

Groundwater Condition

Boring and core logs indicate the landslide debris and bedrock materials are dry in the area of the proposed tunnel (Alternative 1). Consequently, a groundwater rating of 15 may be applied.

Joint Orientation

For the purposes of joint orientation, all geologic structural data (i.e., from bedding, joints, fractures, etc.) were treated the same. Based on a comparison of structural attitudes determined from the OPTV logs, the geologic structure is considered favorable. This results in an RMR rating of (-)2.

Overall RMR

Based on the above individual ratings, the overall RMR for the older landslide debris of the South Shores landslide ranges from 34 to 58. This corresponds to a classification of poor to fair rock. Additional continuous core borings should be performed to better define the range of RMR values along the final tunnel alignment should this design alternative be selected.

GENERAL GEOTECHNICAL ENGINEERING CHARACTERISTICS

Based on the results of our field investigation and laboratory testing, preliminary geotechnical properties of the onsite soils are anticipated to be as discussed below. Additional exploration along the selected design alternative is recommended to further evaluate these construction conditions.

- In general, we anticipate all onsite soil and bedrock materials can be excavated with conventional trenching and tunneling methods. Hard to very hard and oversize materials may be encountered in local areas.

- Based on our preliminary laboratory testing, we anticipate the onsite soils will be highly expansive. These materials include recent and older alluvium, existing artificial fill, recent and ancient landslide debris, and bedrock.

- Based on our preliminary laboratory testing (See appendix B), we anticipate the onsite soils and rock will have the following corrosion potential:

    - Potential Soil Corrosion to Concrete
        - Recent/Older Alluvium/Topsoil – negligible
        - Existing Artificial Fill - negligible
        - Recent/Ancient Landslide Debris and bedrock – negligible

    - Potential Soil Corrosion to Ferrous Metals
        - Recent/Older Alluvium – severe
        - Existing artificial Fill - severe
        - Recent/Ancient Landslide Debris and bedrock – severe

- Based on our preliminary laboratory testing, we anticipate the onsite surficial soils (i.e., alluvium, older alluvium, topsoil, etc.) will be moderately to highly compressible. Removal and re-compaction of these materials will likely be required in local areas, depending on the design alternative selected. It is anticipated the landslide debris and bedrock will be generally slightly to non-compressible.

SUMMARY OF SLOPE STABILITY ANALYSES

The following project areas were analyzed for slope stability: Tarapaca landslide, descending slope below the switchbacks of PVDE, and the bluff area along the beach at the proposed outlet structure location for Alternative 1. Slope stability results along with details of the strength model used at each section are contained in Appendix C. A summary of the results of the analyses is contained below.

Tarapaca Landslide

To evaluate how much fill is required in the canyon bottom to act as a gravity buttress, Cross-Sections 2-2' and 3-3' (drawn through the Tarapaca landslide) were analyzed with various gravity buttress heights. The results of these analyses indicate that approximately 10 to 20 feet of fill (10 feet at the upper end and 20 feet at the lower end) will be required to obtain a safety factor of approximately 1.25. Approximately 20 to 30 feet of fill placed in the canyon at the toe of the landslide would be required to obtain a safety factor of approximately 1.5. Given the relatively small fill height differential required to obtain a 1.5 safety factor, it is recommended that strong consideration be given to designing a buttress that achieves the 1.5 safety factor.

The fill could be placed in various configurations to obtain the required safety factors. The exact configuration of the buttress fill will be developed at the design stage of the project and once the final safety factor is decided upon.

Lower Switchback PVDE

Cross-Section 3-3' was analyzed to evaluate the existing slope stability safety factor at the lower switchback of PVDE and to estimate how much additional erosion would be required to impact the existing roadway (i.e., a safety factor of 1.0). Based on the strength model and assumptions provided in Appendix C, a safety factor of approximately 1.4 was obtained for existing conditions. Parametric analyses were performed by progressively moving the existing canyon wall and slope face back until a safety factor of 1.0 was achieved. These analyses indicate that the existing slope face would have to be eroded back approximately 35 feet before the roadway would be in a state of imminent failure. The probability that the existing slope face would be eroded back 35 feet should be evaluated by the project civil engineer to determine if any remediation is warranted. Based on our preliminary analyses and erosion rate assumptions, it appears that it would take approximately 7 years for the roadway to be impacted by erosion. It should be noted that the existing sewer line and utility easement would be impacted prior to this distance and time. Given the poor quality of the as-built sewer plans on file at the City, the exact location of the sewer line is not known; however, it appears this line is located between 5 and 10 feet closer to the canyon than the roadway.

Upper Switchback PVDE

Cross-Section 1-1' was analyzed to evaluate the existing slope stability safety factor at the upper switchback of PVDE and to estimate how much additional erosion would be required to impact the existing roadway (i.e., a safety factor of 1.0). Based on the strength model and assumptions provided in Appendix C, a safety factor of approximately 1.3 was obtained for existing conditions. Parametric analyses were performed by progressively moving the existing canyon wall and slope face back until a safety factor of 1.0 was achieved. These analyses indicate that the existing slope face would have to be eroded back approximately 40 feet before the roadway would be in a state of imminent failure. The probability that the existing slope face would be eroded back 40 feet should be evaluated by the project civil engineer to determine if any remediation is warranted. Based on our preliminary analyses and erosion rate assumptions, it appears that it would take approximately 8 years for the roadway to be impacted by erosion. The existing sewer line and easement are located very close to the currently eroding areas, as shown on our Geotechnical Map, Plate 3. We strongly recommend this active line be protected from damage as soon as possible by the pipeline owner to prevent damage to the line.

Bluff Stability

The existing bluff conditions were analyzed for deep seated stability. The results of the analyses indicate that the bluff likely has a safety factor ranging from about 1.2 to over 1.4. Given that a slope failure would have to cut across large blocks of intact siltstone and siliceous siltstone and that the strength of these materials was not considered in the analyses, the actual safety factor is likely closer to the higher end of the range. Slope instability of the bluff face is not considered to be a geotechnical constraint for the project. However, surficial instability in the form of local slumps or "pop outs" may occur and will require further evaluation should this design alternative be selected.

PVDE SWITCHBACKS STABILITY DISCUSSION

As discussed in previous sections of this report, our analyses indicates an additional lateral erosion of the canyon walls of 35 to 40 feet will result in a reduction of stability of the switchbacks to the point of incipient failure. In addition, lesser erosion is necessary to impact the existing 8-inch sewer line that is located between the canyon and the switchbacks. Regardless of the anticipated rate of erosion in this area, the existing sewer line should be protected by the owner as soon as possible in order to prevent failure of the line, particularly in the area of the lower switchback, where the line appears to be very close to the top of the eroded canyon wall.

If hydraulic calculations indicate the switchbacks will require stabilization prior to construction of the chosen design alternative for the storm drain system, three potential repair solutions are discussed below:

- Installation of caissons (CIDH piles) on the outside of the switchbacks – this option would be the most costly due to the construction materials, staging and construction area grading, and the difficulty of drilling in the ancient landslide debris. The bucket-auger borings in these areas encountered very difficult drilling due to hard zones and severe caving, and ultimately were abandoned due to refusal by the drilling equipment. However, this work would be within the City's property, and environmental constraints may be minimal.

- Installation of riprap or similar type of revetment in the canyon bottom – this option would be significantly less costly than caissons; however, the riprap would require grouting, and the flow velocities in the canyon may cause damage to the riprap in a major storm. In addition, continued failure of the Tarapaca landslide would likely bury the riprap, causing additional maintenance, repairs, or replacement. Finally, access to the area of the canyon with equipment to install the riprap would be difficult.

- Installation of a gravity buttress at the toe of the Tarapaca landslide with a flexible pipe system – this option would include installation of a flexible pipe (i.e., butt-fused, high strength HDPE) along the canyon bottom between the upper switchback and the boundary with the City of Los Angeles, with fill placement above the pipe to reduce movement of the Tarapaca landslide and erosion of the western canyon wall. This option would likely reduce the erosion in this area, significantly reduce (and potentially ultimately stop) the movement of the Tarapaca landslide, and significantly reduce the debris moving down canyon towards 25th street. It should be noted that this option differs from Alternative 2 in that there is no permanent storm drain relocation solution.

Should the City wish to pursue one of these options, additional geotechnical analyses would likely be required in order to provide detailed recommendations for construction.

CANYON WALL STABILITY DISCUSSION

The results of the stability analyses for the slopes below the switchbacks indicate that where continuous adversely oriented bedding planes are not exposed in the bluff face, the in situ safety factor of the canyon walls is likely in the range of 1.3 to 1.4. Where adversely oriented bedrock exists relative to the canyon wall – such as in the area of the Tarapaca landslide – failure has either already occurred or the current safety factor is in the range of just above 1.0 to 1.2. In addition, local occurrences of adversely oriented planar bedrock surfaces may also result in local small failures.

CONCLUSIONS

Based on the results of our investigation and analyses, we present the following conclusions:

1. Design Alternatives 1, 2, and 3 are considered to be feasible, provided the design considerations and recommendations for additional work presented in this report are followed.

2. The site is predominately underlain by the South Shores landslide, an ancient, dormant landslide complex.

3. The site includes the Tarapaca landslide, a currently failing mass that appears to have failed along continuous adversely oriented bedding due to erosion of the canyon wall.

4. None of the design alternatives will adversely impact the repaired San Ramon Canyon failure area, located offsite to the north.

5. Groundwater should not be a significant impact to any of the design alternatives for the project.

6. The site will be subject to seismic hazards in the future; however, none of the design alternatives will increase the likelihood or magnitude of these impacts.

7. The Tarapaca landslide can be stabilized with a reduction of erosion at the toe and construction of a gravity-type buttress.

8. The switchbacks of PVDE are currently considered to have safety factors at or greater than 1.3. Approximately 35 to 40 feet of lateral erosion/failure would occur before the factor of safety is reduced to 1.0 (imminent failure).

9. The existing 8-inch sewer line should be relocated as soon as possible in order to avoid damage to the line from canyon wall erosion.

10. The conceptual access road will require significant corrective grading and/or stabilization of the cuts. Should these construction constraints become cost prohibitive, consideration should be given to relocation of the road to a more favorable site.

DESIGN ALTERNATIVE GEOTECHNICAL CONSIDERATIONS

ALTERNATIVE 1

As discussed in a previous section of this report, Alternative 1 consists of constructing a storm drain system to divert runoff water to the west of San Ramon Canyon, as shown on Plate 6 and in the Harris (2010) plans. It is our understanding this storm drain system would be constructed with a combination of open trench and tunneling methods.

In addition, excavation spoils and any local import would be utilized to grade a "gravity-type" buttress at the toe of the Tarapaca landslide. As discussed in our "Slope Stability" section, it is anticipated that up to 30 vertical feet of engineered fill would be required in order to reduce the movement of the landslide to static levels and bring the factor of safety up to 1.5.

Anticipated Construction Methods

We understand the open trench method would be utilized in the portion of the system adjacent to the inlet structure in the canyon, and in the portion south of 25th Street. Tunneling methods would be utilized in the remaining portions of the system, including near the bluff, connecting to the outlet structure. In addition, the system will require construction of an inlet structure within the upper San Ramon Canyon and an outlet structure at the toe of the bluff ascending from the ocean. It is anticipated the entire system will encounter ancient landslide debris of the South Shores landslide.

Pipe Design Considerations

The South Shores landslide is considered to be dormant, and has not shown signs of movement in historic time. However, design of the pipe and appurtenant structures (i.e., manholes, etc.) should take into consideration potential movement of the landslide mass, particularly during a seismic event. Minor movement along internal rupture or shear surfaces within the landslide mass may occur during the life of the storm drain system. Consideration should be given to choosing a high-strength flexible pipe material without joints (such as Butt-fused High Density Poly Ethylene Pipe) that can accommodate these possible minor movements.

Preliminary Geotechnical Considerations for Open Trench Segments

Trench Excavation

Based on our preliminary evaluation and the results of our field exploration, variable stability conditions will be encountered in the trench walls during construction. Some local areas may be temporarily unstable, particularly within the deeper areas of the trench; therefore, shoring or trench wall lay-back will likely be required. Further exploration and analyses will be required in order to provide detailed shoring and temporary stability recommendations. However, for preliminary design alternative evaluation purposes, trench walls excavated at 1:1 (horizontal to vertical) should be anticipated to be temporarily stable.

<u>Backfill and Pipe Bedding</u>

We anticipate the onsite soils will be suitable for backfill of the trench above the pipe bedding zone. Some oversize materials will likely be encountered, and will not be suitable for placement within the backfill.

Preliminary Geotechnical Considerations for Tunnel Segments

Based on the RMR, visual observations and geophysical logging of cores, downhole geologic logging, and geologic observations of older landslide debris exposed in the canyon walls, the following preliminary observations can be made. Further evaluation of these considerations may be warranted, if this alternative is selected.

- Currently, the geologic structure is oriented favorably with respect to the tunnel alignment. Any modifications to the tunnel alignment should consider the geologic structure. Further core holes are recommended to further define the geologic structure along the tunnel alignment.

- Groundwater is not anticipated to be encountered during tunneling. However future borings drilled below the tunnel invert elevation are recommended to further evaluate this condition.

- The overall RMR ratings do not consider the presence of hard siliceous zones which may be encountered during tunneling. Hard siliceous zones or blocks of materials should be expected to be encountered during tunneling.

- Some of the Altamira Shale member bedrock cores swelled after being exposed to the air for several days. The swelling is attributed to air drying and potentially secondary mineral crystal growth. The swelling will create pressure on the ground supports installed for the tunnel construction.

- Tunneling may also encounter local zones of adversely oriented geologic discontinuities that may be lined with bentonite. These zones may produce local stability problems during tunneling.

- Based on the RMR ratings:
  - The proposed tunnel will be excavated through Fair to Poor rock.
  - Approximate stand-up time during tunneling is expected to range from:
    - 10 hours to 1 week for an 8-foot to 15-foot span.

Preliminary Geotechnical Considerations for Canyon Inlet Structure

Design Considerations

Based on our understanding of Alternative 1 and our review of the conceptual plans by Harris & Associates (2010), the conceptual location of the inlet structure is anticipated to be founded on bedrock or ancient landslide debris. Design of this structure should take into consideration the geotechnical characteristics of these materials (i.e., high expansion, etc.). In addition, further investigation at the location of the structure may be required in order to evaluate temporary stability of excavations and to provide site-specific design values for the structure walls as well as recommendations for foundation design and wall drainage.

Temporary Stability and Shoring

While the canyon slopes in the area of the possible inlet structure location may be grossly stable during construction (see "Upper Switchback" portion of the "Slope Stability" section, above), surficial slumping or localized "pop outs" are likely to occur. Further investigation of the inlet structure location will be required in order to provide specific recommendations in regards to temporary stability. However, for the purposes of this study of Alternative 1, it can be assumed that shoring or other stability methods (i.e., caissons, sheet piles, etc.) will likely be required for temporary stability.

Maintenance

Regular maintenance of the inlet structure will be critical to keep the drain system clear. A maintenance schedule should be established and followed regularly with, at a minimum, annual inspection, repair, and cleanout of the structure. Additional inspections should be considered after heavy rain events.

Access Road and Retaining Wall Construction

Based on our review of the Harris (2010) conceptual access road plans, it is our understanding that an access road would be constructed from the cul-de-sac of Tarapaca Road to the inlet structure in order to provide maintenance access. The Harris (2010) plans indicate this road would be mostly in cut bedrock materials, and would require a retaining wall against the ascending cut slope. Cross-Sections 8-8' and 9-9' were drawn to illustrate the conceptual road location and the subsurface geologic structure. Based on our review of previous geotechnical reports for adjacent properties (see Reference list), bedding orientations in this area will result in adversely oriented bedrock exposed during grading of this road. In addition, this road would be located upslope of the currently moving Tarapaca landslide, and directly downslope of existing residential development.

Wall design will need to accommodate adverse structure, and temporary instability will require corrective grading, shoring, and structural support such as tiebacks. In addition, alternative paving may be considered, including concrete, pavers, or other designs that may accommodate the expansive soils and slope creep. Further field investigation and analyses of this area will be required for this possible access road location in order to obtain site-specific geologic and geotechnical data to evaluate these potential issues.

Should these construction measures become cost prohibitive, consideration should be given to relocating this road to an area with more favorable geologic conditions, such as the western side of the canyon near the existing PVDE switchbacks.

Preliminary Geotechnical Considerations for Bluff Face Outlet Structure

Design Considerations

Based on our understanding of Alternative 1 and our review of the conceptual plans by Harris & Associates (2010), the conceptual location of the outlet structure at the toe of the bluff is anticipated to be founded on bedrock or ancient landslide debris (Cross-Section 6-6'). Design of this structure should take into consideration the geotechnical characteristics of these materials (i.e., high expansion, etc.). In addition, further investigation at the location of the structure will likely be required in order to evaluate temporary stability of excavations and to provide site-specific design values for the structure walls as well as recommendations for foundation design and wall drainage.

Temporary Stability and Shoring

While the bluff in the area of the possible outlet structure location may be grossly stable during construction (see "Slope Stability" section, above), minor surficial slumping or localized "pop outs" may potentially occur. These local instabilities are anticipated to be less significant than those of the canyon inlet structure due to the generally intact bedrock materials exposed in the bluff face. Further investigation of the outlet structure location may be required in order to provide specific recommendations in regards to temporary stability. However, for the purposes of this study of Alternative 1, it can be assumed that shoring or other stability methods (i.e., caissons, sheet piles, etc.) may be required for temporary stability of the bluff face during construction.

ALTERNATIVE 2

As discussed in a previous section of this report, Alternative 2 consists of constructing a storm drain system to collect runoff water in upper San Ramon Canyon, as shown on Plate 6 and in the reference (1) plans, and carry it via storm drain piping to the existing inlet structure under 25th Street. It is our understanding this storm drain system would likely be constructed using open trench and supporting grading methods. It is anticipated this construction would encounter ancient landslide, bedrock, and alluvial soils. Based on our preliminary data, the majority of the pipe trench would be founded on bedrock or ancient landslide debris. It should be noted that significant trenching within the canyon bottom is not anticipated for this alternative.

In addition, construction of the storm drain system within the canyon would include construction of a "gravity-type" buttress at the toe of the Tarapaca landslide, similar to Alternative 1.

Pipe Design Considerations

Design of the pipe and appurtenant structures (i.e., manholes, etc.) should take into consideration potential movement of the South Shores landslide mass and the adjacent Tarapaca landslide, particularly during a seismic event. Consideration should be given to choosing a flexible pipe material without joints (such as butt-fused HDPE pipe) that can accommodate these possible minor movements.

Preliminary Geotechnical Considerations for Open Trench Segments

Trench Excavation

Based on our preliminary evaluation and the results of our field exploration, variable stability conditions will be encountered in the trench walls during construction. Some local areas may be temporarily unstable, particularly within the deeper areas of the trench; therefore, shoring will likely be required. In addition, our slope stability analyses indicate the canyon walls are likely to be grossly stable; however, surficial slumps and local failures may occur during construction. Efforts should be made to design the pipe and trench such that excavation into the bedrock within the canyon bottom is kept to a minimum. Further exploration and analyses will be required in order to provide detailed shoring and temporary stability recommendations.

Backfill and Pipe Bedding

We anticipate the onsite soils will be suitable for backfill of the trench above the pipe bedding zone. Some oversize materials will likely be encountered, and will not be suitable for placement within the backfill.

Preliminary Geotechnical Considerations for Canyon Inlet Structure

Design Considerations

Based on our understanding of Alternative 2 and our review of the conceptual plans by Harris & Associates (reference (1)), the conceptual location of the inlet structure is the same as Alternative 1, and is anticipated to be founded on bedrock or ancient landslide debris. Design of this structure should take into consideration the geotechnical characteristics of the soil (i.e., high expansion, etc.). In addition, further investigation at the location of the structure may be required in order to evaluate temporary stability of excavations and to provide site-specific design values for the structure walls as well as recommendations for foundation design and wall drainage.

Temporary Stability and Shoring

While the canyon slopes in the area of the possible inlet structure location may be grossly stable during construction (see "Slope Stability" section, above), surficial slumping or localized "pop outs" may potentially occur. Further investigation of the inlet structure location will be required in order to provide specific recommendations in regards to temporary stability. However, for the purposes of this study of Alternative 1, it can be assumed that shoring or other stability methods (i.e., caissons, sheet piles, etc.) will likely be required for temporary stability.

Access Road and Retaining Wall Construction

Based on our review of the Harris (2010) conceptual access road plans, it is our understanding that an access road would be constructed from the cul-de-sac of Tarapaca Road to the inlet structure in order to provide maintenance access. The Harris (2010) plans indicate this road would be mostly in cut bedrock materials, and would require a retaining wall against the ascending cut slope. Cross-Sections 8-8' and 9-9' were drawn to illustrate the conceptual road location and the subsurface geologic structure. Based on our review of previous geotechnical reports for adjacent properties (see Reference list), bedding orientations in this area will result in adversely oriented bedrock exposed during grading of this road. In addition, this road would be located upslope of the currently moving Tarapaca landslide, and directly downslope of existing residential development.

Wall design will need to accommodate adverse structure, and temporary instability will require corrective grading, shoring, and structural support such as tiebacks. In addition, alternative paving may be considered, including concrete, pavers, or other designs that may accommodate the expansive soils and slope creep. Further field investigation and analyses of this area will be required for this possible access road location in order to obtain site-specific geologic and geotechnical data to evaluate these potential issues.

Should these construction measures become cost prohibitive, consideration should be given to relocating this road to an area with more favorable geologic conditions, such as the western side of the canyon near the existing PVDE switchbacks.

Preliminary Geotechnical Considerations for Canyon Outlet Structure/Tie-In to City Inlet

The area of the canyon where the Alternative 2 storm drain system connects to the existing City system is underlain by recent alluvium over landslide debris of the South Shores landslide. Should this alternative be selected, it is likely the alluvial soils underlying the pipe trench and the connection area will require corrective grading to remove compressible alluvial soils. However, further detailed investigation of this area may be required to fully evaluate the alluvial soils below the pipe depth.

ALTERNATIVE B (OPTION TO ALTERNATIVES 1 AND 2)

As discussed in a previous section of this report, Alternative B consists of extending the storm drain systems described within Alternatives 1 and 2 up the canyon to connect to the existing storm drain system that outlets at the head of the canyon. Geotechnical considerations for this alternative are similar to that of Alternative 2. However, for ease of evaluation, these considerations are reproduced below.

<u>Pipe Design Considerations</u>

Design of the pipe and appurtenant structures (i.e., manholes, etc.) should take into consideration potential movement of the South Shores landslide mass and the adjacent Tarapaca landslide, particularly during a seismic event. Consideration should be given to choosing a flexible pipe material without joints (such as butt-fused HDPE pipe) that can accommodate these possible minor movements.

Preliminary Geotechnical Considerations for Open Trench Segments

<u>Trench Excavation</u>

Based on our preliminary evaluation and the results of our field exploration, variable stability conditions will be encountered in the trench walls during construction. Some local areas may be temporarily unstable, particularly within the deeper areas of the trench; therefore, shoring will likely be required. In addition, our slope stability analyses indicate the canyon walls are likely to be grossly stable; however, surficial slumps and local failures may occur during construction. Efforts should be made to design the pipe and trench such that excavation into the bedrock within the canyon bottom is kept to a minimum. Further exploration and analyses will be required in order to provide detailed shoring and temporary stability recommendations.

Backfill and Pipe Bedding

We anticipate the onsite soils will be suitable for backfill of the trench above the pipe bedding zone. Some oversize materials will likely be encountered, and will not be suitable for placement within the backfill.

Preliminary Geotechnical Considerations for Tie-In to Existing Outlet

The area of the canyon where this alternative system would tie into the existing outlet is anticipated to be underlain by bedrock of the Altamira Shale. Should this alternative be selected, field investigation at this location may be required to evaluate the underlying materials and any temporary construction slopes and/or trench walls.

FUTURE TASKS

Once the design alternative is selected, we recommend our office be retained to perform future geotechnical investigations to provide design-level geotechnical recommendations for final design and construction of the chosen alternative. These future tasks will include:

- Additional field exploration at the chosen storm drain system alignment, including drilling additional borings and performing additional laboratory testing;
- Further specific quantitative analyses of foundation and retaining wall design, slope stability, surficial stability, temporary stability, and shoring design;
- Preparation of a final design report to support the chosen design alternative final plans.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgements. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading will be identical to those observed and sampled during our study or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property. We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

SUPPORTING DATA

The following Plates and Appendices that complete this report are listed in the Table of Contents.

Respectfully submitted,

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AERIAL PHOTOGRAPHS

Date	Flight No.	Frame No.	Date	Flight No.	Frame No.
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