FINAL GEOTECHNICAL REPORT
BUNKER & MITCHELL ROADS AND ALEXANDER AVENUE IMPROVEMENTS
CA PRA GOGA 104(1) & 105(2)

GOLDEN GATE NATIONAL RECREATION AREA
CALIFORNIA

February 17, 2012
YA Project No. 210-189A

Prepared for:
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Denver, CO 80237
and
Federal Highway Administration
Central Federal Lands Highway Division
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APPENDICES

EXECUTIVE SUMMARY Yeh and Associates, Inc., as a subconsultant to Atkins, Denver, Colorado, was retained by the Federal Highway Administration (FHWA), Central Federal Lands Highway Division (CFLHD) to provide geotechnical recommendations for the Alexander Avenue Planning Study including preliminary and final design of improvements to the Alexander Avenue and Danes Drive Intersection, and reconstruction of Bunker Road and Mitchell Road in the Golden Gate National Recreation Area, north of San Francisco, California.

Traffic circulation and operational changes are anticipated on Danes Drive and Alexander Avenue due to various reasons affecting local traffic. Roadway widening and intersection improvements are planned to accommodate the traffic. The construction will require expanding an existing rock cut on Alexander Avenue, additional embankment over the Bunker Road Underpass and widening Danes Drive and portions of Alexander Avenue. The Bunker Road underpass structure (tunnel) is considered inadequate to support the additional load imposed by a conventional earth embankment. Widening of the US Highway 101 Underpass and Alexander Avenue near East Road are options considered for future improvements presented in the Alexander Avenue Planning Study.

Reconstruction of Bunker and Mitchell Roads will include drainage improvements, widening and new pavement. The drainage improvements consist of reestablishing the hydraulic link between the north side of Mitchell Road and Rodeo Lagoon. Pipe culverts are proposed for the drainage improvements.

Two borings were drilled in Alexander Avenue on either side of the Bunker Road underpass to evaluate soil and bedrock conditions. Weathered chert bedrock was encountered below about 8.0 feet to 29.0 feet of man-placed fill and 10.0 feet of native sand and clay soils. Hard chert bedrock was encountered at 23.0 feet on the south side of the tunnel and at 47.5 feet on the north side.

Four borings were drilled to evaluate the existing pavement thickness and subgrade soil conditions in Alexander Avenue and Danes Drive. Existing asphalt pavement thickness ranged from 8 inches to 11 inches. About 4 inches to 6 inches of aggregate base course was encountered in two of the borings. Subgrade consisted of granular fill and bedrock. Groundwater was encountered at shallow depth in the boring drilled at the rock cut on Alexander Avenue. The subsoils and bedrock have very good pavement support characteristics with R-values ranging from 70 to 84.

Loose to dense clayey sand and soft to stiff sandy clay were encountered in four borings drilled along Mitchell Road near the proposed culvert locations. The soils are suitable for support of the light loads typical of pipe culverts. Low laboratory soil resistivity values indicate a need for corrosion protection.

A seismic tomograph survey was performed along the top of the rock cut south of the Danes Drive intersection with Alexander Avenue. The results of the survey indicate the bedrock in the proposed cut has seismic velocities that are typical for rock that should be rippable with bulldozers equipped with ripper teeth. Isolated areas of harder rock that require blasting may be encountered in the cut.

A Mechanically Stabilized Earth (MSE) wall supported on a micropile foundation is proposed for the embankment over the east end of the Bunker Road underpass. Micropile foundations that penetrate at least 10 feet into the hard bedrock will have an axial capacity on the order of 175 kips. The micropiles can be installed on either side of the underpass structure to support a MSE wall without adding load to the tunnel.

The rock cut south of the intersection of Danes Drive and Alexander Avenue should be sloped no steeper than 1.33V to 1H. The slope should be draped with rockfall mesh or a barrier provided to reduce the potential for falling rock to enter the traveled way. A catchment ditch is also recommended between the roadway and the toe of the slope.

Pavement in the proposed widening areas should consist of a full depth of 6.5 inches of Hot Asphalt Concrete Pavement (HACP) or 5 inches of HACP over 5 inches of aggregate base course.

1.0 PURPOSE AND SCOPE OF STUDY

Yeh and Associates, Inc., as a subconsultant to Atkins, Denver, Colorado, was retained by the Federal Highway Administration (FHWA), Central Federal Lands Highway Division (CFLHD) to provide geotechnical recommendations for preliminary and final design of improvements to Alexander Avenue, including the Danes Drive Intersection and Bunker & Mitchell Roads in the Golden Gate National Recreation Area (GGNRA), north of San Francisco, California. Alexander Avenue and Danes Drive provide access to the City of Sausalito and GGNRA respectively. Bunker and Mitchell Roads provide access to the western portion of the recreation area, Rodeo Lagoon and the Marine Mammal Research Center. The regional location of the project area is shown on Figure 1.1.

The redevelopment of Fort Baker as a conference center within the GGNRA has resulted in traffic circulation and operational changes at the Danes Drive/Alexander Avenue intersection. The northbound left turn lane on Alexander Avenue will be lengthened to accommodate these changes. Danes Drive at Alexander Avenue will be realigned from a Y-intersection to a T-intersection. The proposed improvements will require widening of Alexander Avenue. The widening of the northbound lane of Alexander Avenue will require excavation of the rock slope in the existing cut south of the intersection and construction of an embankment over the East Bunker Road underpass north of the intersection. The lengthening of the westbound right-turn lane on Danes Drive to East Bunker Road will require additional embankment fill beyond the shoulder of the roadway. Reconstruction and widening of the Alexander Avenue underpass below US Highway 101 and widening of Alexander Avenue south of East Road are also being considered for future construction as discussed in the the Alexander Avenue Planning Study.

The Bunker & Mitchell Roads Rehabilitation Project will include three sections of Bunker Road and Mitchell Road. Old Bunker Road begins at the intersection of Mitchell Road and Bunker Road and extends northwest past the Marine Mammal Center. Bunker Road West begins at the Mitchell Road intersection and extends to the Baker-Barry Tunnel. Bunker Road East begins at Murray Circle and extends to the west to the Baker-Barry Tunnel. Mitchell Road extends from the intersection with old Bunker Road to Rodeo Beach.

The purpose of this project is to rehabilitate the roadways, construct minor roadway widening and improve drainage and safety. The left turn lane on Alexander Avenue at Danes Drive will be extended and new parking areas will be constructed adjacent to Bunker Road West and near the Marine Mammal Center. The rehabilitation is proposed for approximately 3.24 miles of roadway. Drainage improvements will include replacing deteriorated culverts and inlets, restoration of channel flow at the parking lot near the entrance to Rodeo Beach at the west end of Mitchell Road and restoration of the hydraulic connection at Rodeo Lagoon, west of the Bunker/Mitchell Road intersection. The hydraulic connection will consist of a large diameter pipe culvert. However, a concrete box culvert or pedestrian bridge are additional options that have been under consideration for the drainage crossing at the west end of Mitchell Road.

The purpose of the geotechnical investigation is to evaluate geologic and subsurface conditions in the project area and provide recommendations for design of rock cut slopes, embankments, retaining structures, drainage structures and pavements. This report presents the results the current investigation along Bunker and Mitchell Roads and results of previous investigations including a project scoping for the Danes Drive/Alexander Avenue intersection on December 3 and 4, 2008; a Preliminary Geotechnical Investigation Report for the intersection that includes pavement design recommendations dated November 21, 2009; and a report describing existing geotechnical and pavement conditions along the Alexander Avenue alignment dated March, 2010 prepared for the Alexander Avenue Planning Study. The design recommendations consider evaluation of rockfall potential and methods to construct an embankment over the Bunker Road underpass that will result in no additional loading on the tunnel structure. Preliminary recommendations for the embankment alternatives were expanded polystyrene (EPS) fill and a mechanically stabilized earth (MSE) wall supported on micropile foundations. The MSE wall has been selected as the recommended alternative and design recommendations are included herein. Preliminary design recommendations for EPS fill are also included. This report addresses potential geologic hazards and constraints for the proposed improvements, existing pavement conditions along Alexander Avenue and Bunker Road, and includes recommendations for rockfall mitigation, foundations and pavement section thickness design. Discussions of geologic and geotechnical issues related to future improvements that have been presented as options in the Alexander Avenue Planning Study are included.

Figure 1.1: Project Location Map

2.0 GEOLOGIC SETTING

San Francisco and the Golden Gate Headlands are located on the boundary between the North American and Pacific tectonic plates. The plate boundary is observable in the form of a transform fault, the San Andreas Fault Zone. This fault zone has historically produced significant earthquake activity as stresses built up by slow plate movement are released.

Prior to the formation of this transform fault, where the Pacific Plate slowly slips northward past the North American Plate, the Pacific Ocean floor was subducted beneath the North American Plate. The rocks of the Franciscan Complex that underlie much of coastal Northern California were formed in this subduction zone. The Franciscan Complex is composed of a stacked sequence of semi-coherent blocks that were scraped from the subducting ocean plate and thrust against the North American Plate, forming an accretionary wedge. As a result, the structurally highest rocks (to the east) are the oldest. Franciscan rocks form the basement of the Coastal Ranges east of the San Andreas Fault. In the Bay Area the rocks range in age from 200 million to 80 million years old.

The Franciscan Complex primarily consists of greywacke sandstone and argillite, with lesser amounts of greenstone (altered submarine basalt), radiolarian ribbon chert, limestone, serpentine and a variety of metamorphic rocks. These rocks have become fractured, dislocated and blended together on a local scale to form a mixture or mélange. A geologic map showing the approximate locations of the bedrock types within the project limits is shown on Figure A-1 in Appendix A - Geologic Maps.

2.1 Alexander Avenue and Danes Drive

The project site is located in a part of the Franciscan Complex known as the Marin Headlands Terrane. Rocks in the terrane probably originated in the central Pacific near the equator and moved northward with the oceanic plate, colliding with North America and becoming attached to the North American Plate during the subduction process. San Andreas related transform faulting moved the terrane northward to its current location about 40 million years ago. As a result, the rocks in the Marin Headlands Terrane consist of about 20 to 25 percent altered submarine pillow basalt, 50 percent thinly bedded ribbon chert and 25 percent clastic rocks.

The Danes Drive/Alexander Avenue intersection is located near a boundary of the basalt and chert rock types. The basalt that can be observed in the rock cut south of the intersection has been weathered and altered to form what is commonly called greenstone. Chert is exposed along the south side of Danes Drive and west of Alexander Avenue, north of the project. The chert is bedded in 3/4 to 4-inch thick layers alternating with thinner, dark red shale layers. Where exposed along Alexander Avenue, the chert is intensely folded. An outcrop of greywacke sandstone is located at the top of the ridge between US 101 and Alexander Avenue, outside the project area. Outcrops of serpentine bedrock, which can contain asbestos minerals, were not observed along the corridor. A geologic map of the Alexander Avenue area is shown in Appendix A, Figure A-2.

Alexander Avenue crosses a shallow east-west trending valley that extends from Highway 101 in the west to Fort Baker and the bay on the east. The valley bottom contains Quaternary deposits of alluvium and colluvium derived from the surrounding hillsides. The road is supported on man-placed fill where it crosses the valley and other shallow drainages.

Clayey soils overly the shallow bedrock on steep slopes west of US 101, South of Danes Drive and at two locations along the west side of Alexander Avenue. These soil deposits, and possibly the underlying bedrock, are susceptible to sliding when weakened by high subsurface moisture conditions. The area west of US 101 is a large landslide complex that has been active in the relatively recent past and currently shows signs of shallow surface slumping failures. Subsurface horizontal drains have been installed near the toe of the slide and water was flowing from the drains during a February 13, 2010 site visit. Figure A-3 in Appendix A shows the locations of the potentially unstable slopes.

During the December 3 and 4, 2008 site visit for project scoping, Yeh & Associates identified adverse geologic conditions including unfavorable rock jointing and unstable soil slopes on the west side of Alexander Avenue and the south side of Danes Drive. Because of these conditions, Yeh and Associates recommended the widening of Alexander Avenue occur along the northbound lane and the widening of Danes Drive occur along the westbound shoulder. Excavation for roadway widening at the toes of these slopes should be avoided. The steep fill slope between Alexander Avenue and East Road shows indications of potential instability including distressed vegetation and shallow failures near the toe. Additional geotechnical investigation, including subsurface exploration, is recommended prior to locating embankment fills or retaining structures above this slope.

2.2 Bunker and Mitchell Roads

West of the Baker-Barry Tunnel, Bunker Road is situated in a broad valley that has become partially filled with Quaternary alluvial deposits. The road generally follows the south side of the valley where the bedrock consists primarily of ribbon chert and basalt greenstone. Shallow cuts in the bedrock for road construction appear to be stable and cut slopes in soil deposits have re-vegetated. Indications of active slope failures were not observed west of the Baker-Barry Tunnel. Mitchell Road is located in a relatively flat Quaternary deposit composed of undifferentiated colluvium, alluvium and slope debris. The area appears to have previously been poorly drained marshy ground associated with the Rodeo Lagoon. The valley surrounding the Rodeo Lagoon is shown to be in a tsunami inundation area on the Point Bonita Quadrangle Tsunami Inundation Map, dated July 1, 2009.

2.3 Seismicity

The project is located at approximate latitude 37.83 and longitude 122.50. The Alexander Avenue/Danes Drive is assumed to be classified as Site Class B based on the subsurface information obtained from the borings near the East Bunker Road Tunnel and the Bunker Road/Mitchell Road site is assumed to be classified as Site Class E based on the subsurface information obtained from the borings near Rodeo Lagoon. Site classifications were determined in accordance with the Method B procedure in Table C3.10.3.1-1 of the AASHTO LRFD Bridge Design Specifications. The Peak Ground Acceleration (PGA), and the short- and long-period spectral acceleration coefficients (SS and S1 respectively) for the area were obtained using the USGS 2007 Seismic Parameters for an event with a 7% Probability of Exceedance (PE) in 75 years and a Site Class B (reference site). An event with the above probability of exceedance has a return period of about 1,000 years. The values were adjusted using Site Factors for Site Class E in accordance with Section 3.10.3.2 of the AASHTO LRFD Bridge Design Specifications. The seismic parameters for this site are shown on Table 2.1 below. Bridge construction at this site will require a site-specific evaluation.

For Site Class B

PGA (0.0 sec)	Ss (0.2 sec)	S1 (1.0 sec)	Seismic Zone
0.715	1.649	0.813	4

For Site Class E

As (0.0 sec)	SDs (0.2 sec)	SD1 (1.0 sec)	Seismic Zone
0.643 g	1.484 g	1.951 g	4

Table 2.1: Seismic Design Parameters

3.0 SITE CONDITIONS

3.1 Alexander Avenue and Danes Drive

The Danes Drive/Alexander Avenue intersection is located within the Golden Gate National Recreation Area, approximately 0.9 miles north of the Golden Gate Bridge. The project area is located on the south side of a broad, shallow east/west trending valley, west of Fort Baker, in the Marin Headlands.

Alexander Avenue is a north-south two-lane thoroughfare connecting the City of Sausalito with US Highway 101. Danes Drive provides access to the west portion of the recreation area and intersects with Alexander Avenue at a “Y” intersection. East Bunker Road is located north of the intersection and roughly parallels Danes Drive. East Bunker Road passes under Alexander Avenue through a reinforced concrete tunnel approximately 120 feet in length. The tunnel was constructed in the late 1930s using the cut and cover method. It has a maximum interior height at the crown of about 21 feet and is covered with about 10 feet of fill.

Alexander Avenue passes through a steep rock cut immediately south of the intersection. The maximum height of the cut is about 115 feet and basalt rock is exposed in the cut face. The road is supported on embankment fill on either side of the East Bunker Road underpass. Danes Drive is partially in cut with the westbound lane mainly on fill. Near the East Bunker Road underpass, the embankment fill heights range to about 35 feet and the existing fill slopes are graded at approximately 1V to 1.5H.

Vegetation on the site consists of native grasses and shrubs with scattered evergreen and deciduous trees.

3.2 Bunker and Mitchell Roads

Old Bunker Road traverses the south facing slope of a low hill above the Rodeo Lagoon. Beginning at the entrance to the Marine Mammal Center, the road continues down the slope to the intersection with Bunker Road and Mitchell Road near the east end of the lagoon. The grade of the two-lane road is generally about 3 to 6 percent. Chert bedrock is exposed in a cut slope along the northeast side of the road, above the intersection.

Mitchell Road begins at a gravel parking lot that serves visitors to Rodeo Beach. The road follows the north side of Rodeo Lagoon for about ½ mile to the intersection with Bunker Road. The profile grade is relatively flat. Several cross culverts provide drainage between the toe of the shallow slope on the north side of the road and the lagoon.

Bunker Road follows the floor of a broad valley to connect the facilities at Fort Cronkhite with Fort Baker on the east side of the recreation area. The road is located along the south side of the valley, adjacent a relatively steep north facing slope. The terrain is relatively flat to gently rolling and climbs from west to east. Bunker Road passes through a residential subdivision before entering the Baker-Barry Tunnel. Chert and basalt bedrock formations are exposed in cuts along the south side of the road.

West of the Baker-Barry Tunnel vegetation consists mainly of grass and deciduous shrubs in the valley floor. Evergreen and deciduous trees cover the slopes on the sides of the valley.

4.0 SUBSURFACE INVESTIGATIONS

Yeh and Associates contracted with Precision Sampling of Stockton, California to drill exploratory borings for geotechnical investigations at the Alexander Avenue/Danes Drive intersection and along Mitchell Road, west of Bunker Road. Traffic control during drilling was provided by Road Safety, Inc. of Rocklin, California. The borings at Alexander Avenue/Danes Drive were drilled on February 17 and 18, 2009 and the borings on Mitchell Road were drilled on February 9, 2011. The locations of the borings are shown on the Boring Location Plans in Appendix B. Logs of the borings are shown in Appendix C.

4.1 Exploratory Borings

Borings T-1 and T-2 were drilled in the northbound lane of Alexander Avenue, on the south and north sides of the East Bunker Road underpass, respectively. These borings were drilled to evaluate subsurface conditions for design of embankment over the East Bunker Road Tunnel. Borings P-1 through P-4 were drilled in the northbound lane of Alexander Avenue and the westbound lane of Danes Drive at approximately 300-foot intervals. These shallow borings were drilled to evaluate subgrade conditions for pavement design.

Boring YA-01 was drilled near the southeast corner of a gravel-surfaced parking lot on the north side of Mitchell road at approximate Station 12+00. Boring YA-02 was drilled in the eastbound lane of Mitchell Road at approximate Station 12+20. Boring YA-04 was drilled in the westbound lane of Mitchell Road and YA-03 was drilled in the eastbound lane at approximate Stations 29+50 and 30+50 respectively. The borings were drilled near locations of proposed drainage structures and culverts to evaluate foundation support characteristics and corrosivity of the soils.

The borings were drilled with a truck-mounted CME 75 drilling rig using 8-inch O.D. (4.25-inch I.D.) hollow stem auger. Samples were obtained at selected intervals using a 1.5-inch I.D. split-spoon sampler, a 2-inch I.D. California barrel sampler and a thin-walled tube sampler (Shelby tube). The split-spoon and California samplers were driven into the subsoils with a 140-pound automatic hammer falling 30 inches. The number of blows needed to drive the sampler 12 inches constitutes the blow count, N, reported on the Boring Logs (Appendix C). The blow count can be used as a relative measure of the material stiffness or density. Bulk samples of auger cuttings were also obtained from the borings at selected intervals. Upon completion, the shallow borings (P-1 thru P-4) were backfilled with auger cuttings and the deep borings (T-1, T-2 and YA-01 through YA-04) were backfilled with cement grout. The pavement was patched with at least 6 inches of Portland cement concrete that was stained to resemble asphalt or with cold asphalt patch mix.

4.2 Laboratory Testing

Samples retrieved during the field exploration were returned to our laboratory for observation by the project geotechnical engineer. Selected bulk samples and Shelby tube samples were submitted to Cooper Testing Labs in Palo Alto, California for testing. An applicable program of laboratory testing was developed to determine engineering properties of the subsurface materials. Following the completion of the laboratory testing, the field descriptions were confirmed or modified as necessary and boring logs were prepared.

Laboratory tests performed included gradation (ASTM D 421, C 136 and AASHTO T 27), Atterberg limits (AASHTO T 89/T 90), one-dimensional consolidation (ASTM D 2435), moisture content (AASHTO T 265), in-situ dry density (ASTM D 2937), moisture density relations (AASHTO T 99), R-value (Caltrans 301), sulfate content (AASHTO T290), pH (ASTM D 4972/AASHTO T 289) and resistivity (AASHTO T 288). Gradation and Atterberg limits test results were used to classify the soils in accordance with the AASHTO classification system and the

Unified Soil Classification System (USCS). The swell and consolidation tests were used to evaluate potential settlement or expansion of the on-site soils when wetted under the anticipated loading conditions resulting from the proposed construction. Moisture content and in-situ dry density, when compared to the results of the moisture-density relations test, provide an estimate of the expected shrink or swell and moisture conditioning requirements if the on-site soils are used as compacted fill. Soil R-value is a measure of soil subgrade strength use for pavement design. Tests for soil sulfate content, pH and resistivity are used to evaluate the potential of the soil to be aggressive to concrete and to corrode buried metal. The laboratory test results are presented in Appendix D and on the boring logs in Appendix C. Photos of the boring locations are in Appendix E.

5.0 SUBSURFACE CONDITIONS

The project at the Alexander Avenue /Danes Drive intersection was originally designed using SI (metric) units of measurement and the Preliminary Geotechnical Investigation was reported in metric units. The boring logs, laboratory test results and discussion of subsurface conditions for modifications to the East Bunker Road Underpass and pavement at the intersection are presented in metric units and English units for consistency. The design for Bunker and Mitchell Road is in English units as are the results of the geotechnical investigation.

5.1 East Bunker Road Underpass

Boring T-1 was drilled in Alexander Avenue, south of the East Bunker Road Underpass. The locations are shown in Appendix B on Figure B-1. The boring was advanced with hollow-stem auger. Samples were obtained using a split-spoon sampler and from auger cuttings. Below about 10 inches of asphalt pavement, the soils encountered in the boring consisted of about 7 feet of man placed sandy gravel fill over 10 feet of soft, medium to high plasticity sandy clay. Weathered chert bedrock interlayered with clay was encountered at a depth of 18 feet. Hard to very hard chert bedrock was encountered at about 23 feet to the bottom of the boring at 36 feet.

Boring T-2 was drilled using hollow-stem auger in Alexander Avenue on the north side of the underpass and encountered 10.5 inches of asphalt pavement over about 28.5 feet of man-placed clayey, sandy gravel fill. Below the fill the boring encountered 10 feet of dense silty sand. Weathered chert bedrock interlayered with claystone was encountered at a depth of 29 feet and hard chert bedrock was encountered from 47.5 feet to the bottom of the boring at 56 feet. Samples were obtained from auger cuttings and using a split-spoon sampler.

The laboratory test results indicate the granular fill materials are non-plastic to low plasticity and contain 7 to 20 percent clayey fines. The claystone layers in the weathered and hard chert bedrock have medium to high plasticity. The bedrock materials have pH ranging from 6.5 to 7.9, 0 percent water soluble sulfates and resistivity ranging from 200 to 4500 ohm-cm. The gravel fill encountered at shallow depth in Boring T-2 has an R-value of 84.

5.2 Pavement Borings

Pavement borings P-1 and P-2 were drilled in the northbound lane of Alexander Avenue, south and north of the intersection, respectively. Borings P-3 and P-4 were drilled in the westbound lane of Danes Drive. The borings were advanced with hollow-stem augers. Samples were obtained from auger cuttings and by driving a split-spoon sampler. The locations of the pavement borings are included on Figure B-1. The thickness of asphalt pavement encountered in the borings ranged from 8 to 11 inches. Aggregate base course was encountered below the asphalt in Borings P-1 and P-4. The base course thickness was about 4 inches in P-1 and 6 inches in P-4. Chert bedrock was encountered below the base course in these borings. Clayey gravel with sand was encountered below the asphalt in Borings P-2 and P-3. The pavement borings were drilled to a depth of 5 feet. Groundwater was encountered in Boring P-1 at a depth of 4 feet at the time of drilling.

The natural moisture content of the tested samples ranged from 1.1 percent to 4.2 percent. Sieve analyses and Atterberg limits tests indicate the man placed fill materials are generally granular and have low plasticity and the clayey materials associated with the chert bedrock have medium to high plasticity. R-values of existing subgrade soils, including the sample from Boring T-2, ranged from 70 to 84. The subgrade soils have AASHTO classifications of A-2-6 and A-2-7.

5.3 Bunker and Mitchell Roads

Borings YA-01 and YA-02 were drilled near the west end of Mitchell Road near the site of an existing drainage structure. The borings were advanced with hollow-stem auger and samples were obtained using split-spoon, California and Shelby tube samplers. Bulk samples were obtained from auger cuttings. The locations of the borings are shown in Appendix B on Figure B-2. Boring YA-01 was drilled in a gravel-surfaced parking lot, near the existing drainage inlet on the north side of the road. The subsoils encountered for the full 31.5-foot depth of the boring consisted of loose to medium dense clayey sand with trace amounts of gravel. Groundwater was encountered during drilling at a depth of 25 feet. Boring YA-02 was drilled in the eastbound lane of Mitchell Road. During the initial attempt to drill the boring auger refusal was encountered at a depth of about 3 feet. The obstruction is probably a large boulder placed during construction of the road. The boring location was off-set to the east. Subsoils encountered in YA-02 below the pavement consisted of about 8 feet of very loose clayey sand with gravelly and sandy clay lenses over 11 feet of soft sandy clay with trace amounts of gravel. Below 19 feet to the full depth of drilling, 31.5 feet, the subsoils consisted of interlayered, very loose to dense, clayey sand and clayey, sandy gravel. The groundwater level was at a depth of 22 feet at the time of drilling.

Boring YA-03 was drilled in the westbound lane of Mitchell Road, about 750 feet west of Old Bunker Road. The subsoils encountered below pavement consist of about 13 feet of very soft to stiff sandy clay over about 5 feet of medium dense clayey sand and 3.5 feet of stiff sandy clay. The bottom of the boring was at a depth of 21.5 feet. Groundwater was encountered at a depth of 15 feet during drilling. Boring YA-04 was drilled in the eastbound gravel shoulder of Mitchell Road, about 650 feet west of Old Bunker Road. The boring encountered about 8 feet of loose clayey sand over soft to stiff sandy clay with trace amounts of gravel. The sandy clay was encountered from a depth of 8 feet to 21.5 feet, the bottom of the boring. Groundwater was encountered at about 14.5 feet below the ground surface during drilling of YA-04.

The laboratory test results indicate the clayey sand soils encountered in the borings along Mitchell Road have low to medium plasticity and contain 11 to 39 percent clayey fines. A sample of the clayey sand soil from YA-01 has a pH of 6.5, 0.01 percent water soluble sulfates and resistivity of 1737 ohm-cm. The sandy clay soils have medium to high plasticity and 51 to 57 percent fines. A relatively undisturbed sample of the sandy clay from Boring YA-02 was tested in one-dimensional consolidation. The results indicate the soil is normally consolidated and is moderately compressible under light loading conditions. A sample of the sandy clay soil from Boring YA-03 has a pH of 7.5, 0 percent water soluble sulfates and resistivity of 1431 ohm-cm. The near surface soils classify as A-6 and A-7-6 in accordance with AASHTO.

6.0 SEISMIC TOMOGRAPH SURVEY

Seismic tomography involves placing a line of regularly spaced sensors (geophones) on the surface and measuring the relative arrival time of seismic energy transmitted from a specified source location. The data are recorded in the field using a portable tomograph, multiple geophones (generally 12 per line), a repeatable seismic source, and a power source. Seismic sources generate both compression (P) and shear (S) waves and, although either may be used for subsurface imaging, P waves are preferred since they are not absorbed by saturated soil units (shear waves cannot transmit through water). Seismic energy travels with a compression velocity that is characteristic of the density, porosity, structure, and water content of each geologic layer.

The seismic tomograph survey of the northern end of the rock cut on the east side of the intersection utilized a 100 foot long array of 12 geophones deployed in a roughly north-south line along the top of the existing cut, just outside the boundary fence. The survey location is shown on Figure B-1. The energy source for this study consisted of a sledgehammer striking a metal plate.

Tomography data was refined, analyzed, and interpreted using the tomographic inversion method. The subsurface profile was resolved into distinct layers with average velocities and portrayed as color-coded gradient plot (tomogram). This method is useful for identifying key subsurface units and their distribution along the survey. Seismic velocities can be interpreted to evaluate the rippability of subsurface materials.

The color-coded sectional plot shown below represents the subsurface velocity distribution. The velocities were measured in meters per second and were converted to feet per second. Typically, seismic velocities provide an indication of the properties of the material through which the seismic waves are traveling. In this case, the thinly laminated and weathered chert bedrock, to a depth of about 15 feet, has relatively low wave velocities due to extensive fracturing and clay inclusions within the rock mass. The tomograph data indicated a P-wave velocity in the chert between about 1000 feet per second and 2500 feet per second. The velocity in the underlying "greenstone" basalt appears to be between about 2600 feet per second and 4300 feet per second. The tomogram shows indications of isolated areas of rock with velocities of 5000 to 6000 feet per second that may be encountered in the cut.

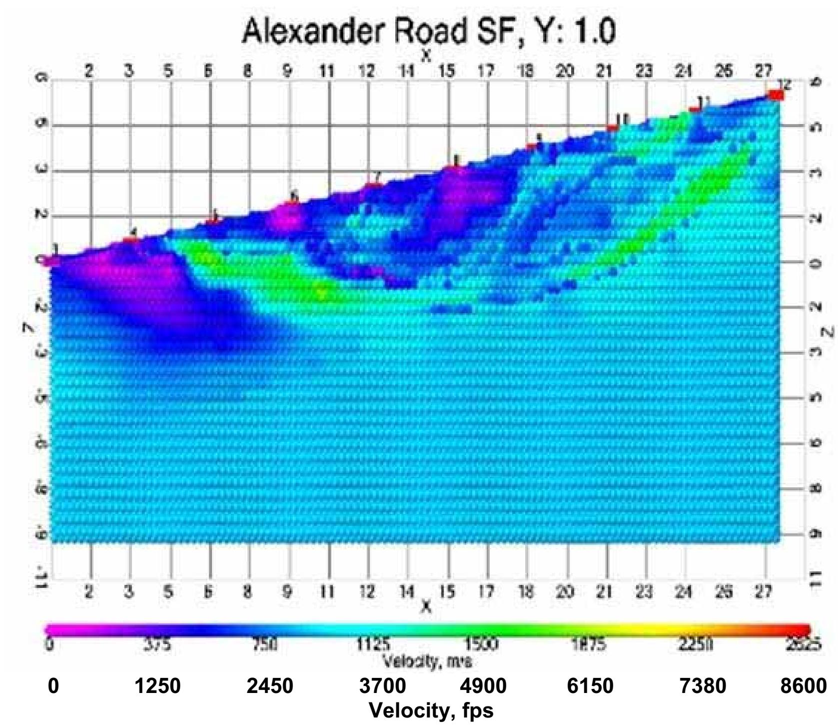

Figure 3: Subsurface Velocity Distribution

7.0 RECOMMENDATIONS

7.1 Alexander Avenue Embankment

The embankment required for the proposed widening beyond the existing northbound shoulder of Alexander Avenue will be constructed over the existing Bunker Road underpass. Because the tunnel structure is considered inadequate to support the additional load of a conventional earthen fill, alternative methods to support the roadway have been evaluated. Presented below are recommendations for a micropile foundation to support a retaining structure such as a Mechanically Stabilized Earth (MSE) wall and preliminary recommendations for lightweight fill consisting of Expanded Polystyrene (EPS) Blocks.

7.1.1 Micropile Foundation

A retaining structure supported on a micropile foundation can be used to allow construction of the fill above the tunnel. By transferring the foundation loads down to the competent bedrock, micropile foundations on either side of the tunnel can support a structure designed to span the tunnel. Design and construction will be similar to that for a narrow bridge. An MSE wall or reinforced slope constructed on top of the "bridge" would allow the widening of Alexander Avenue without increasing the load on the tunnel. The temporary excavation for construction of the MSE wall will require shoring to support traffic loads on Alexander Avenue during construction. Shoring at the tunnel location should consist of horizontal support elements such as ground nails or ground anchors to avoid possible damage to the tunnel caused by driving or drilling vertical support elements.

Design recommendations for micropile foundations are presented below.

1. Recommendations for the micropile support assume Type A Composite Reinforced Micropiles constructed with gravity grouting methods as described in Section 10.9.1 of the AASHTO 2008 LRFD Bridge Design Specifications. Other micropile construction methods may be used at the discretion of the Engineer.
2. The Type A micropiles should have permanent steel casing installed to prevent caving during drilling and to provide structural capacity. The cased section of the micropile should penetrate a minimum of 5 feet into the chert bedrock encountered in the borings at 18 and 40 feet below the existing pavement surface in Borings T-1 and T-2, respectively. The casing should have a minimum diameter of 7.25 inches and should consist of steel pipe. The interior of the casing should be fully grouted and the annulus between the casing and the soil/bedrock should be filled with grout.

3. Below the casing, the micropile should extend a minimum of 10 feet into chert bedrock. The rock socket portion of the micropile should have a diameter of at least 6 inches for the full depth in the chert bedrock.
4. Reinforcing in each micropile should consist of a single No. 13 high strength thread bar.
5. Micropile capacity in the rock-socket zone can be calculated using grout-to ground bond resistance in the chert bedrock and neglecting pile tip resistance. The nominal grout-to-ground bond strength is estimated to be 12 ksf based on AASTHO Table C10.9.3.5.2-1. A resistance factor of 0.55 can be used for that portion of the micropile in the chert bedrock.

Laboratory resistivity tests indicate the bedrock will not be aggressive toward buried steel. Sulfate content and pH are in the non-aggressive range. Reinforcing and hardware for the micropiles should be epoxy-coated to provide corrosion resistance in the coastal environment.

Vertical micropiles of the lengths anticipated for this project will not have significant capacity to support lateral loads. Recommendations for micropiles to support lateral loads can be provided on request.

7.1.2 MSE Wall

The proposed MSE wall should be designed with the geotechnical parameters shown below.

Lateral Earth Pressures:

Coefficient of Active Earth Pressure, ka = 0.33 and unit weight of 120 pcf for the existing clayey gravel embankment fill soils.

Coefficient of Passive Earth Pressure, kp = 3.00 and unit weight of 135 pcf for imported structural backfill. A resistance factor of 0.50 should be used to calculate passive earth pressure that resists wall movement.

Coefficient of Friction:

The wall should be constructed using select granular backfill. A coefficient of friction of 0.55 should be used to calculate sliding resistance of the MSE wall on the concrete slab. A resistance factor of 0.80 should be used when calculating sliding resistance due to friction.

7.1.3 Expanded Polystyrene Fill Alternative

Lightweight fill constructed of expanded polystyrene (EPS) blocks has been successfully used in transportation applications to support roadways, bridge abutments and for landslide mitigation. The high quality EPS has sufficient strength to bear traffic loads but weighs much less than compacted soil. An EPS fill can be constructed above the East Bunker Road Tunnel without imposing additional load on the structure.

The lightweight fill is constructed by placing large blocks of EPS in alternating directions, similar to brick masonry construction. The blocks are available from many manufacturers in the U.S. and can be produced with dimensions of 2 feet thick by 4 feet wide by 8 feet long. Construction of EPS fill over the tunnel will require removal of some of the existing embankment to allow the blocks to key into the slope. The end result could be a reduction of the load on the tunnel, depending on the amount of soil removed. The EPS fill should be covered with at least 3 feet of soil or pavement structure to protect the blocks from weather, burrowing animals and petroleum fuel spills that can severely degrade or destroy the material. The EPS fill can be designed to include a facing such as a reinforced slope that would allow vegetation to be restored over the embankment.

The preliminary design recommendations for the EPS fill are as follows:

1. The EPS should be a virgin material with a density of at least 1.15 pcf (a nominal density of 1.24 pcf), minimum compressive strengths of 48 psf and 130 psf at 1% and 10% deformation respectively, and maximum water absorption of 3%.
2. The footprint of the excavation can either be laid-back as a temporary cut or be supported with shoring prior to EPS placement. If shoring support is required the shoring can be designed as a permanent application to resist the active earth pressure against the EPS.
3. The EPS should be treated to prevent damage from insects including termites and carpenter ants.
4. A concrete cap and impervious liner should be placed on top and around the EPS to prevent petroleum products, especially diesel fuel, from dissolving the EPS material.
5. A drainage system is required behind and underneath the EPS fill.
6. The EPS should be keyed into both sides of the excavation cut to reduce the potential for differential settlements that can occur in the roadway between the EPS and the existing fill.
7. The design of the EPS fill should include global stability analysis.

7.2 Rock Excavation

Seismic tomography was used during the geotechnical investigation for the Danes Drive intersection to obtain average seismic velocities of the chert and basalt bedrock at the rock cut on the east side of the intersection. The recorded seismic velocities provide an indication of the properties of the material through which the seismic waves are traveling. In this case, the thinly laminated and weathered chert bedrock has relatively low wave velocities due to extensive fracturing and clay inclusions within the rock mass. The tomograph data for the Danes Drive site indicated a P-wave velocity in the chert between about 1000 feet per second and 2400 feet per second. The velocity in the underlying "greenstone" basalt appears to be between about 2600 feet per second and 4300 feet per second. Materials with these seismic velocities should be rippable per the Caterpillar Handbook of Ripping, 8th Edition. Based on the results of the seismic tomography, it appears that most of the rock excavation will be rippable. Isolated areas of rock with P-wave velocities in the range of 5000 to 6000 feet per second may be encountered in rock cuts and could require blasting. Pre-blast and post-blast surveys of nearby structures should be performed if blasting is required. Damage to structures can be prevented by requiring limits to blast related vibrations and monitoring small trial blasts to establish safe blasting procedures during excavation.

Generally, rock cuts for roadway widening should be located on the east side of Alexander Avenue. Because the terrain slopes down toward the east, rock slope heights will be less on the east side. Cuts on the east side will avoid potentially unstable slopes identified on Figure A-3.

Crushed basalt rock removed from the excavation may be suitable for re-use as embankment or backfill. An average bulking factor of 1.3 can be used to estimate the increased volume of excavated rock verses in-place rock.

7.3 Rockfall Mitigation

Traffic below the proposed rock cut includes more than 9,400 vehicles per day, numerous bicycles and frequent pedestrians. Anecdotal reports from maintenance personnel indicate that the area requires occasional removal of fallen rock from the traffic lanes. Rocks ranging in size from gravel to small boulders were observed in the shoulder and at the edge of the pavement during the geotechnical investigation. These rocks could cause vehicle accidents and pose a hazard to bicyclist safety if present in the paved roadway.

Rockfall hazard mitigation alternatives were evaluated with the aid of the Colorado Rockfall Simulation Program (CRSP) computer software (Version 4.0). The software uses input parameters including slope geometry, slope material properties, rock geometry and rock material properties to model rockfall and predict rock velocity, bounce height and percent of falling rocks passing a designated point on the slope. The results of this evaluation can be used to design a mitigation alternative that reduces the rockfall hazard from the current condition to a more acceptable condition.

The basic model for this evaluation is an 80-foot high rock cut sloped at 1.33 V to 1H with a catchment ditch at the toe. The rock slope consists of relatively hard material with moderate surface roughness. The catchment ditch has a 1V to 4H slope away from the pavement and a relatively soft soil surface. Rock sizes and shapes were selected for the model based on the observations made during the geotechnical investigation. The analysis evaluated containment of the expected maximum (12-inch) and average size spherical shaped rocks.

Mitigation alternatives were evaluated by modifying the model to vary the width of the catchment ditch and to reduce the effective height of the slope by the addition of rockfall mesh draped from the crest of the slope. Increasing the width of the catchment ditch allows the falling rock to lose momentum before it reaches the pavement. The use of draped mesh has the effect of reducing the amount of rock generated from the slope and accelerating the revegetation process of the freshly excavated material. The application of mesh also reduces the effective height from which the rocks fall from the bottom of the mesh to the toe of the slope.

The catchment ditch width alternatives evaluated include the existing condition (1-foot width), 5-feet, 8 feet, 10 feet and 16 feet. Rockfall mesh alternatives evaluated include no mesh, and mesh draped from the crest of the rock slope down to a height that would provide containment of approximately 70 percent of 12-inch diameter rocks for each ditch width. The CRSP was used to predict the percentage of rocks that would be contained in the catchment ditch for each alternative. The evaluation considered a range of rock sizes and rockfall events that have a source at any height along the face of the slope. Slope and rock input parameters were modified in an iterative process as the analysis provided additional data. The results of the evaluation are presented in Table 7.3.1 below.

The first column of the table presents results of the evaluation for a 12-inch diameter rock falling from a specific height. The results are for the "No Mesh" condition with a rock fall height of 80 feet and for the "With Mesh" condition using various fall heights for each ditch width until the resulting containment was approximately 70 percent. The 70 percent containment fall height is the reported bottom of mesh height for each ditch width.

The results presented in the second column of the table are for 12-inch diameter rocks falling from random heights along the slope face. The results show how the percent containment increases when the rockfall events being simulated have sources that are distributed over the exposed (un-meshed) height of the slope.

The third column presents the results of an analysis that an average size rock falling from random heights along the slope face. The average rock size was selected based on observations of rocks in the existing ditch made during the geotechnical investigation. The results show the effects of simulating a smaller rock size on the percent containment.

The results indicate that 70 % rockfall containment can be achieved with catchment ditch and draped mesh combinations that range from a 5-foot wide ditch and mesh extending to 16 feet above the toe of the slope to a 16-foot wide catchment ditch and no draped mesh. The maximum predicted bounce height within the catchment ditch is less than 1 foot for all ditch widths of 5 feet or more. The predicted maximum bounce heights for rocks that leave the catchment ditch are about 3 feet for the 5-foot wide ditch and less than 1 foot for wider catchment ditches. The bounce height predictions indicate that temporary concrete barrier can be used to mitigate the rockfall hazard during and after construction.

Percentage of Rocks Retained

	Ditch Width (ft)	Maximum Size Design Rock, Rock Originating from Top of Slope		Maximum Size Design Rock, Rock Originating from Any Slope Height		Average Size Rock, Rock Originating from Any Slope Height	
No Mesh	1	0%		1%		12%	
	5	9%		22%		50%	
	8	18%		43%		71%	
	10	24%		54%		72%	
	16	73%		83%		93%	
With Mesh		**Mesh Height**		**Mesh Height**		**Mesh Height**	
	1	8 ft.	14%	8 ft.	15%	8 ft.	41%
	5	16 ft.	36%	16 ft.	72%	16 ft.	90%
	8	20 ft.	72%	20 ft.	93%	20 ft.	97%
	10	30 ft.	65%	30 ft.	93%	30 ft.	97%

Table 7.3.1: Results of Evaluation of Selected Rockfall Mitigation Alternatives

Rockfall mesh should consist of 8 x 10 double twist hexagonal netting type, zinc and PVC coated in accordance with ASTM 975-97 (2003). Rockfall mesh should be securely anchored above the brow of the slope. The mesh should cover the slope face with no gaps.

Additional information regarding the frequency, volume and dispersion and rock dimensions from actual rockfall events at the site should be obtained so that the CRSP model can be calibrated to this slope. The data can be used to refine the design recommendations for alternatives that reduce the risk from rockfall hazard.

7.4 Mitchell Road Drainage Structures

The subsoils encountered near the proposed culvert locations on Mitchell Road are suitable for support of the light foundation loads typical of culvert pipes and inlet/outlet structures. Because the load imposed by these types of drainage structures is generally less than the weight of the soil they replace, the potential for settlement is low.

The soft sandy clay soils were found to be compressible under light to moderate foundation loads. Spread footing or mat foundations for support of a pedestrian bridge or concrete box culvert should be designed for an allowable soil bearing pressure of 1500 psf. Foundation settlement in the range of 1 to 2 inches could occur over the long term for structures supported by spread footing foundations bearing on the compressible soils. Additional foundation design recommendations can be provided if either the bridge or box culvert option is selected for this site.

7.5 Alexander Avenue/US Highway 101 Underpass

Widening of the Alexander Avenue Underpass at US Highway 101 is being evaluated through the Alexander Avenue Planning Study process. Design of the structure will require additional geotechnical investigation. Reconstruction of the underpass will impact traffic on US 101 and Alexander Avenue. Traffic impacts can be reduced by constructing abutment foundations in a phased manner that results in only a partial closure of the road during construction. The existing structure is supported on spread footings and bedrock is exposed on the east side of the underpass. New spread footing foundations can probably be designed with sufficient capacity to support the underpass. Temporary excavation support will be required to maintain stability of excavations during construction. Micropile walls with shotcrete facing may be an appropriate means of temporary excavation support. The geotechnical investigation for foundation design should include at least six exploratory borings that penetrate a minimum of 10 feet into the bedrock. Samples of the overburden soils and core samples of the bedrock should be obtained from the borings. Realignment or widening of the ramp from US 101 to Alexander Avenue should avoid the toe of the unstable slope west of the overpass. Additional geotechnical investigation and analysis will be required if the new alignment encroaches on the areas where unstable slopes have been identified.

7.6 Soil Corrosivity

The results of laboratory tests for pH, water soluble sulfates and resistivity indicate the subsoils encountered near the Alexander Avenue and Danes Drive intersection will not be aggressive toward concrete or buried metal. No special corrosion protection is recommended at this location.

Soils encountered in the borings along Mitchell Road had near neutral pH and low sulfate content but laboratory tests indicate resistivity is less than 2000 ohm-cm. These soils may be aggressive toward buried metal pipes and special corrosion protection such as heavy gauge metal or HDPE pipe materials should be considered.

7.7 Pavement Recommendations

The pavement design recommendations for Bunker and Mitchell roads were provided by CFLHD. The recommendations were presented in a report titled: "Golden Gate National Recreation Area, CA PRA GOGA 104(1) 105(2), Bunker and Mitchell Roads", Report # 10-01, May 2010. Yeh and Associates reviewed the existing pavement conditions for Bunker and Mitchell Roads as described in the report. A representative of Yeh and Associates observed pavement conditions on the roads during the site visit for the geotechnical investigation. The existing pavement conditions documented in the report are, in general, consistent with our field observations.

Three pavement designs were evaluated to address the project at Danes Drive and Alexander Avenue. Two design options are presented for the areas to be widened; a full depth Hot Asphalt Concrete Pavement (HACP) and a composite design using HACP and Aggregate Base Course (ABC). An overlay design was also evaluated to identify the additional pavement thickness that would be required to address the future traffic loading. The resulting recommended pavement thicknesses are summarized in Table 7.7.1. The outputs from the DARWin pavement design computer program for each design are attached in Appendix F.

7.7.1 Subgrade Strength

Laboratory tests indicate R-values of 70, 80, and 84 for the A-2-6 (0) soils encountered in the pavement borings within 1.5 meters (5 feet) of the existing pavement surface. For design purposes, an R-value of 70 was used to determine the resilient modulus for this pavement design.

The R-value was converted to a resilient modulus using procedures adapted from the AASHTO 1993 Pavement Design Guide. Using the equations shown below, an R-value of 70 was assigned resilient modulus of 25,317 psi.

$S1 = [(R-5)/11.29] + 3$

$MR = 10[S1 + 18.72)/6.24]$

Where: MR = resilient modulus (psi)

S1 = the soil support value

R = the R-value obtained from the Hveem Stabilometer (AASHTO T190)

A calculated resilient modulus of 25,317 psi was used as one of the inputs for the DARWin Pavement Design computer program to determine recommended pavement thickness for the various pavement options. The DARWin pavement design computer program follows the AASHTO 1993 Pavement Design Manual but the version used does not accept SI units for input.

A structural layer coefficient of 0.25 was assigned to the existing pavement. Other Structural Layer coefficients were assigned based on the "Guidelines for Completing the Pavement Investigation and Report (V1 and V2 Activities)", CFLHD January 2005.

7.7.2 Traffic Loading

Traffic loading information was supplied by Atkins (formerly PBS&J) and the 20-Year design Equivalent Single Axle Loads (ESAL) for this section was 3,388,077. The Traffic Loading study is attached in Appendix F.

Pavement Location	HACP Thickness (inches)	ABC Thickness (inches)
Widening Full Depth HACP	6.5	0.0
Widening Composite	5	5
Overlay Existing Pavements*	3	0.0

*The design thickness assumes milling to remove 2 inches of existing pavement prior to the overlay.

Table 7.7.1: Summary of Design Pavement Thickness

7.7.3 Asphalt Mix and Binder Recommendations

A nominal ½-inch mix HACP is recommended for this project. The binder should be a performance graded binder meeting the requirements for PG 64-16.

The LTPPBind program was used to determine the binder grade for this area. Based on the historic weather data for San Francisco, a grade of PG 58-10 would meet the project requirements. However, CalTrans has adopted the Superpave Binder Specifications and in accordance with those recommendations, this project is in the Northern Coastal Area. The binder recommended by CalTrans for the Northern Coastal area is PG 64-16. Because this binder will be readily available and the PG 64-16 meets or exceeds the LTPP binder, the PG 64-16 binder is recommended for use on this project.

7.7.4 Site Grading and Drainage Recommendations

The native materials encountered in the pavement borings at Alexander Avenue and Danes Drive are suitable for use as fill beneath pavements. The pavement subgrade should be proof rolled with a heavily loaded pneumatic-tire vehicle. Areas which deform more than 0.5 inch under heavy wheel loads should be removed, replaced if necessary and reworked to achieve a stable subgrade prior to paving. Earthwork should conform to the Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects.

The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Proper design of drainage should include prevention of ponding of water on or immediately adjacent to pavement areas.

8.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2002, *Standard Specifications for Highway Bridges*, 17th edition.

AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007 with 2008 Interim Revisions

Federal Highway Administration (FHWA), 2003, *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects*, FP-03.

United States Geological Survey (USGS), 2002, Earthquake Hazards Program, http://earthquake.usgs.gov/hazards/.

United States Geological Survey (USGS) Map, *Geology of the San Francisco North Quadrangle, California*, J. Schlocker, M.G. Bonilla and D.H. Radbruch, 1958

United States Geological Survey (USGS), Google Overlay Map, Geology of Marin County, California

Tsunami Inundation Map for Emergency Planning
State of California ~ County of Marin, Point Bonita Quadrangle, July 1, 2009

CFLHD, *Golden Gate National Recreation Area*, *CA PRA GOGA 104(1) 105(2), Bunker and Mitchell Roads*, Report # 10-01, May 2010

Elder, W. P., *Geology of the Golden Gate Headlands,* National Park Service, Golden Gate National Recreation Area.

Yeh and Associates, Inc., *Preliminary Geotechnical Investigation Report, Danes Drive, CA PRA/NPS GOGA 268(1),* YA Project No. 28-296B, November 21, 2009.

As-Built Plans, *Sausalito Road Undercrossing Modification*, State of California Department of Public Works Division of Highways, October 19, 1962.

As-Built Plans, *Sausalito Lateral – Underpass at Station 40+93*, The Golden Gate bridge and Highway District, September 1, 1938.

As-Built Plans, *Sausalito Lateral Resurfacing*, Golden Gate Bridge Highway and Transportation District, September 22, 1977.

Oregon Department of Transportation and Federal Highway Administration, *Rockfall Catchment Area Design Guide*, FHWA-OR-RD-02-04, 2001

9.0 LIMITATIONS

This study was conducted in accordance with generally accepted geotechnical engineering practices in this area for use by the client for design and construction purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from exploratory borings and field review and the proposed type of construction. Subsurface variations across the site are likely and may not become evident until excavation is performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, this office should be advised at once so reevaluation of the recommendations may be made. We recommend on-site observation of excavations and foundation bearing strata by a representative of the geotechnical engineer.

YEH AND ASSOCIATES, INC.

Thomas L. Allen, P.E.

Appendix A – Site Geologic Maps

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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: PBS&J

Project Number: 210-189

Drawn by: TLA

Checked by:

Date: 04/19/11

Site Geologic Map

Project: Bunker & Mitchell Roads

Sheet Number: A-1

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Sheet Revision

Date	Revision/Issue	Checked by

Designed For: PBS&J

Project Number: 28-296C

Drawn by: TLA

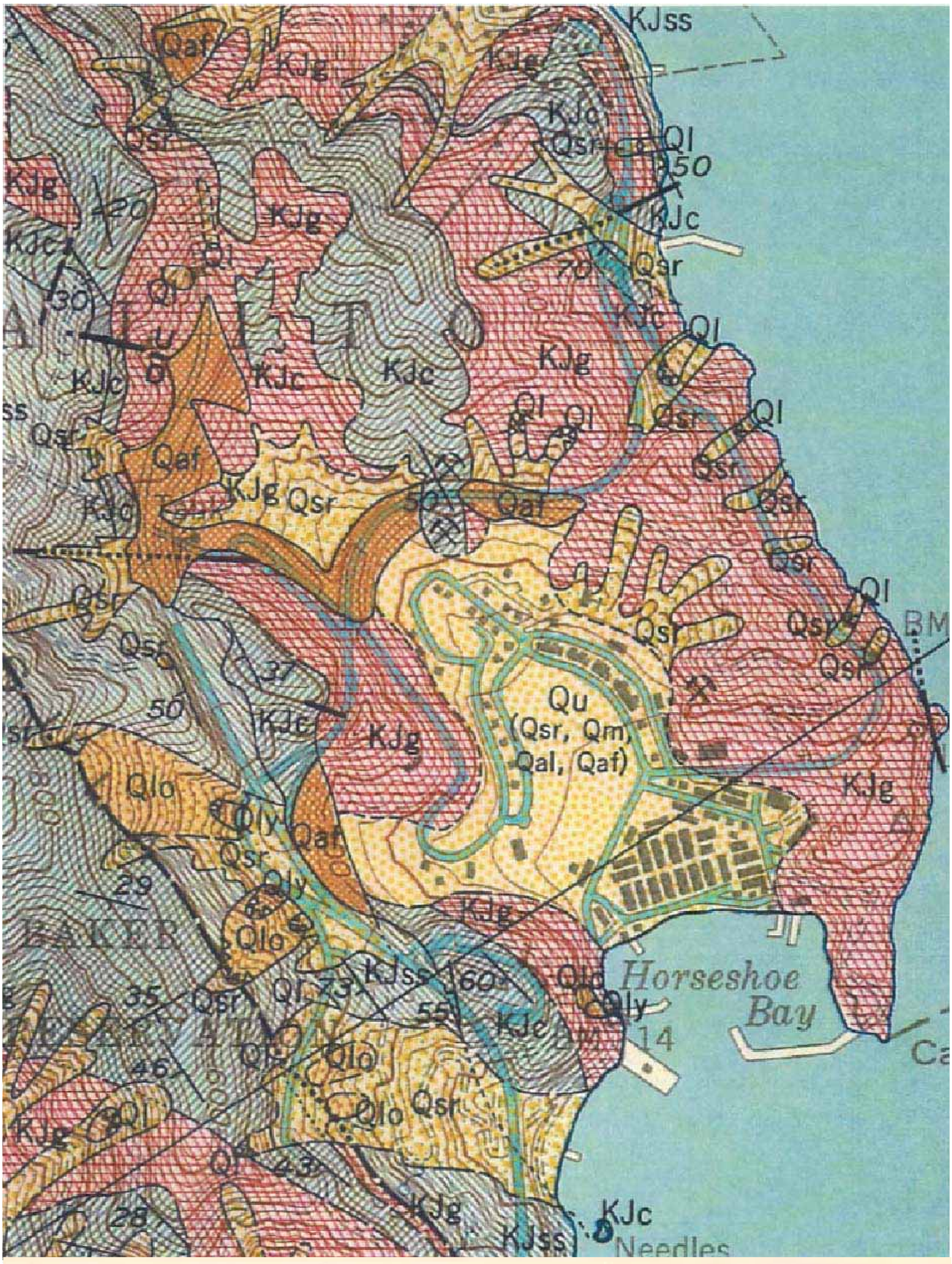
Date: 5/25/2010

Checked by: TLB

SITE GEOLOGIC MAP

Project: Alexander Avenue Planning Study

Sheet Number: A-2



Qaf — Artificial fill
Predominantly dune sand but includes silt, clay, rock waste, man-made debris, and organic waste. Probable fill material indicated in parentheses

Ql / Qly / Qlo — Landslide deposits
Generally unstratified mixtures of bedrock, sand, silt, and clay in varying proportions. Younger landslide deposits, Qly; *older landslide deposits,* Qlo

Qal — Alluvium
Predominantly clayey silty sand and clayey silt, some pebbles; generally gray to brown

Qsr — Slope debris and ravine fill
Angular rock fragments in sand, silt, and clay matrix; generally light yellow to reddish brown

Qu (Qc) — Quaternary deposits, undifferentiated
Probable Quaternary unit indicated in parentheses

KJss — Sandstone
Thick-bedded and massive graywacke sandstone interbedded with thin layers of fissile shale and fine-grained sandstone; some thick conglomerate lenses

KJc — Radiolarian chert and shale
Alternate beds of hard brittle chert, 1 to 5 inches thick, and brittle, crumbly shale, ⅛ to ½ inch thick. Locally includes bodies of massive chert

KJg — Greenstone
Aphanitic to medium-grained altered volcanic rocks; predominantly basalt. Includes flows, dikes, sills, plugs, agglomerate, and tuff; pillow lavas common locally; interbedded with radiolarian chert

Adapted From:
USGS Map

GEOLOGY OF THE SAN FRANCISCO NORTH QUADRANGLE, CALIFORNIA
By
J. Schlocker, M. G. Bonilla, and D. H. Radbruch
1958

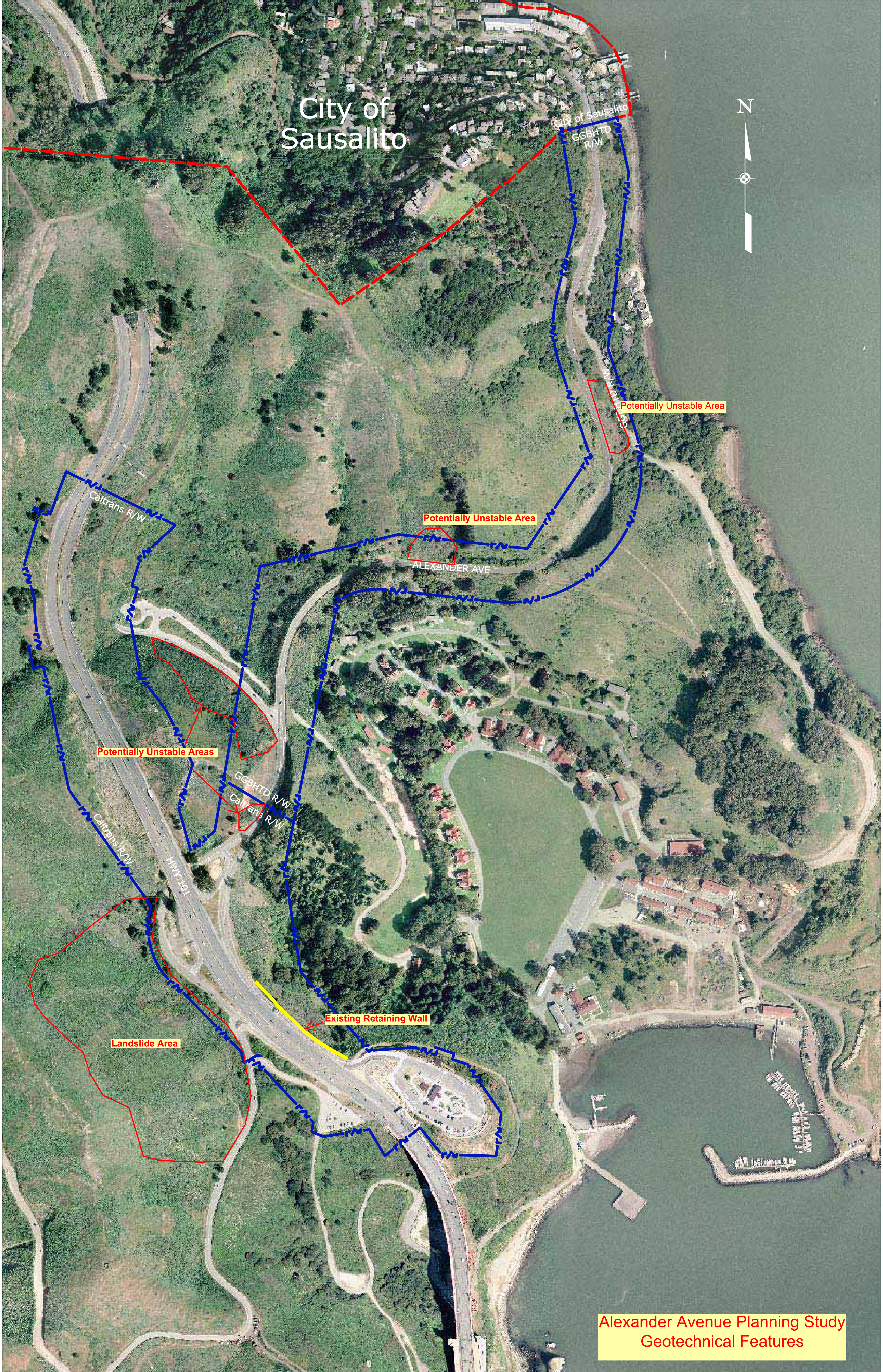
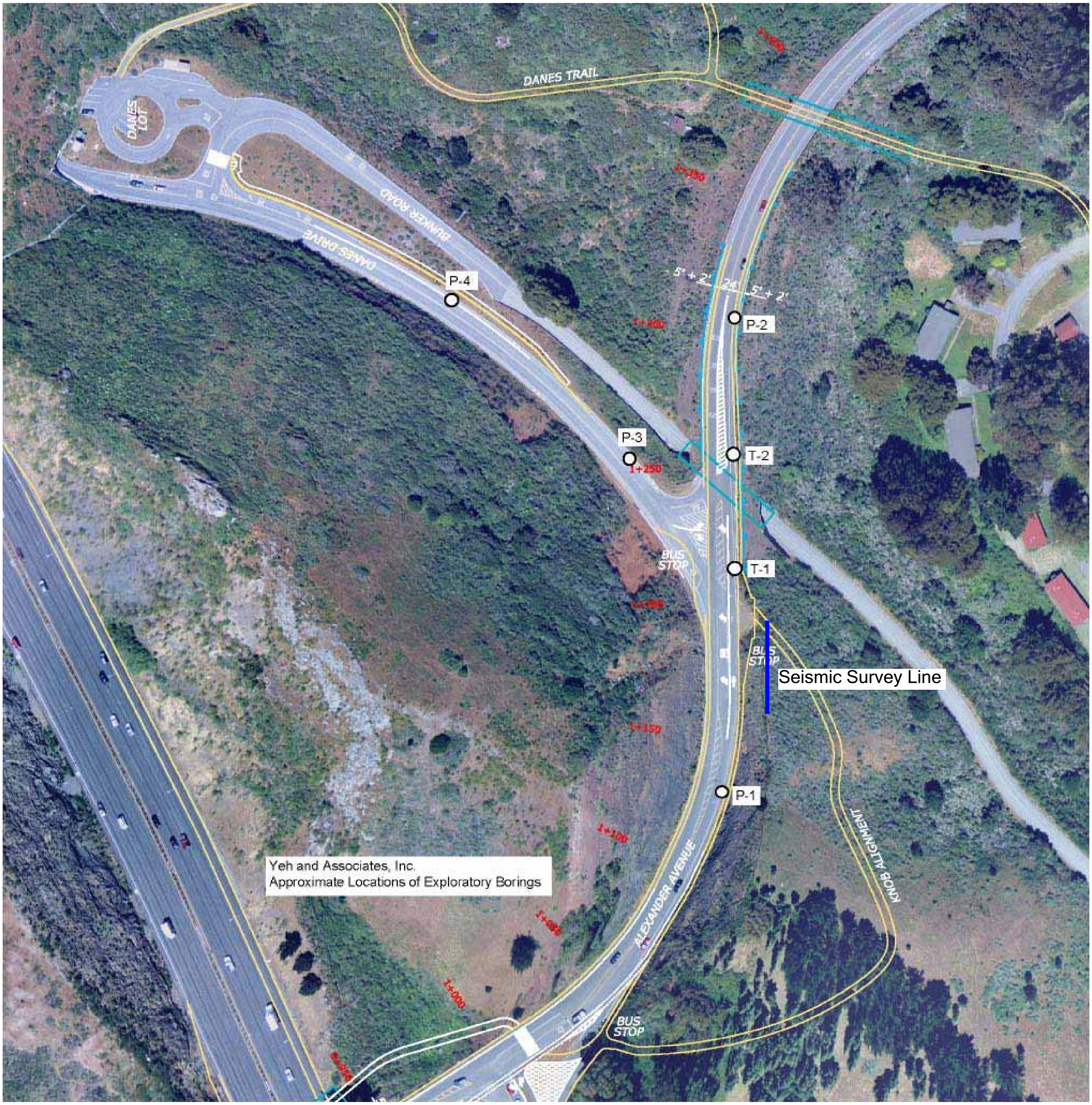

City of Sausalito
City of Sausalito
GGBHTD R/W
N
Potentially Unstable Area
Potentially Unstable Area
ALEXANDER AVE
Caltrans R/W
Potentially Unstable Areas
GGBHTD R/W
Caltrans R/W
Caltrans R/W
HWY 101
Landslide Area
Existing Retaining Wall
Alexander Avenue Planning Study
Geotechnical Features

Figure A-3

Appendix B – Exploratory Boring Locations



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Sheet Revision		
Date	Revision/Issue	Checked by

Designed For: PBS&J

Project Number: 210-189

Drawn by: TLA

Checked by:

Date: 04/19/11

Locations of Alexander Ave/Danes Dr Borings

Project: Bunker & Mitchell Rds

Figure Number: B-1

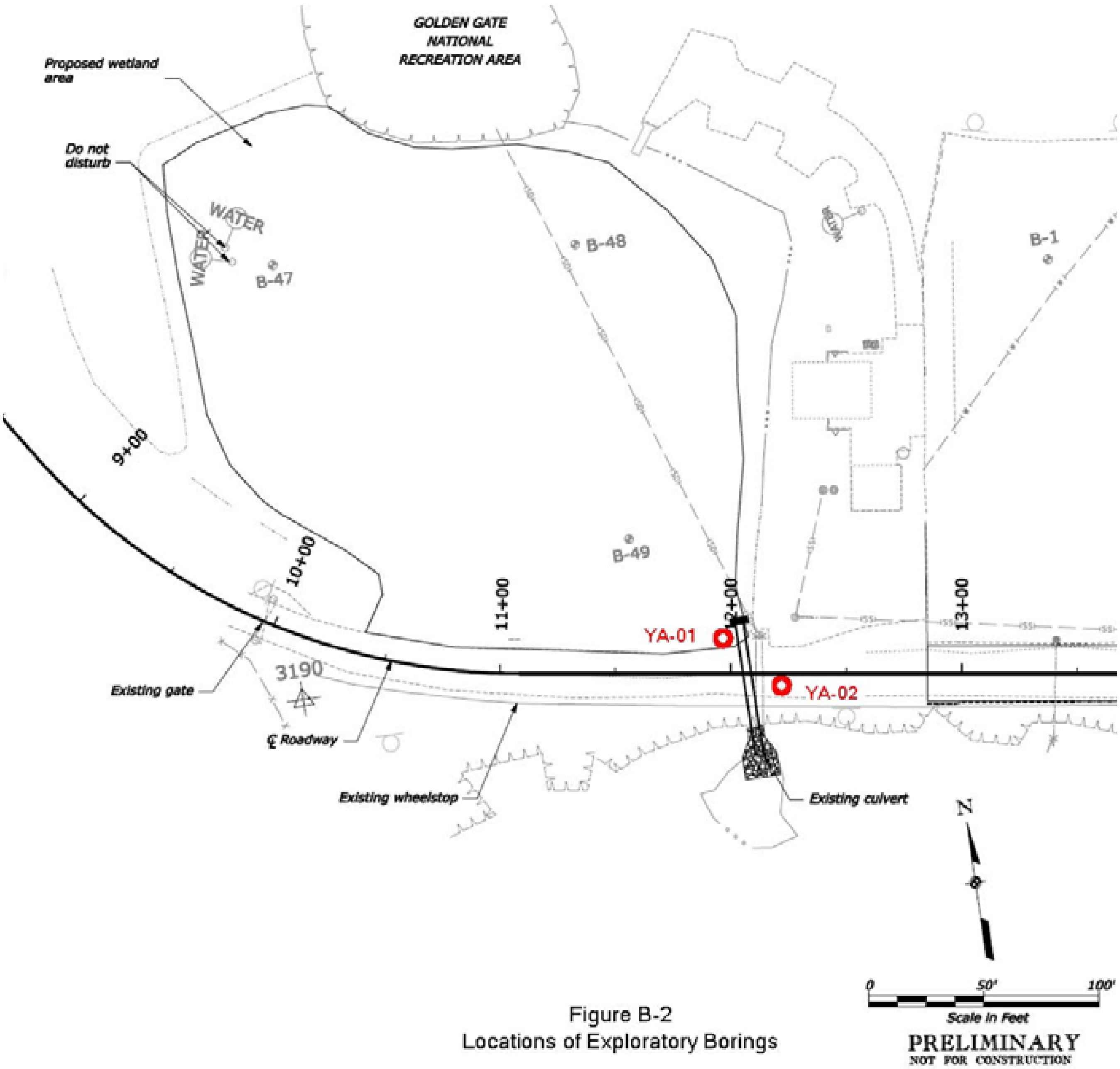

Figure B-2
Locations of Exploratory Borings

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Sheet Revision		
Date	Revision/Issue	Checked by

Designed For: PBS&J

Project Number: 210-189

Drawn by: TLA

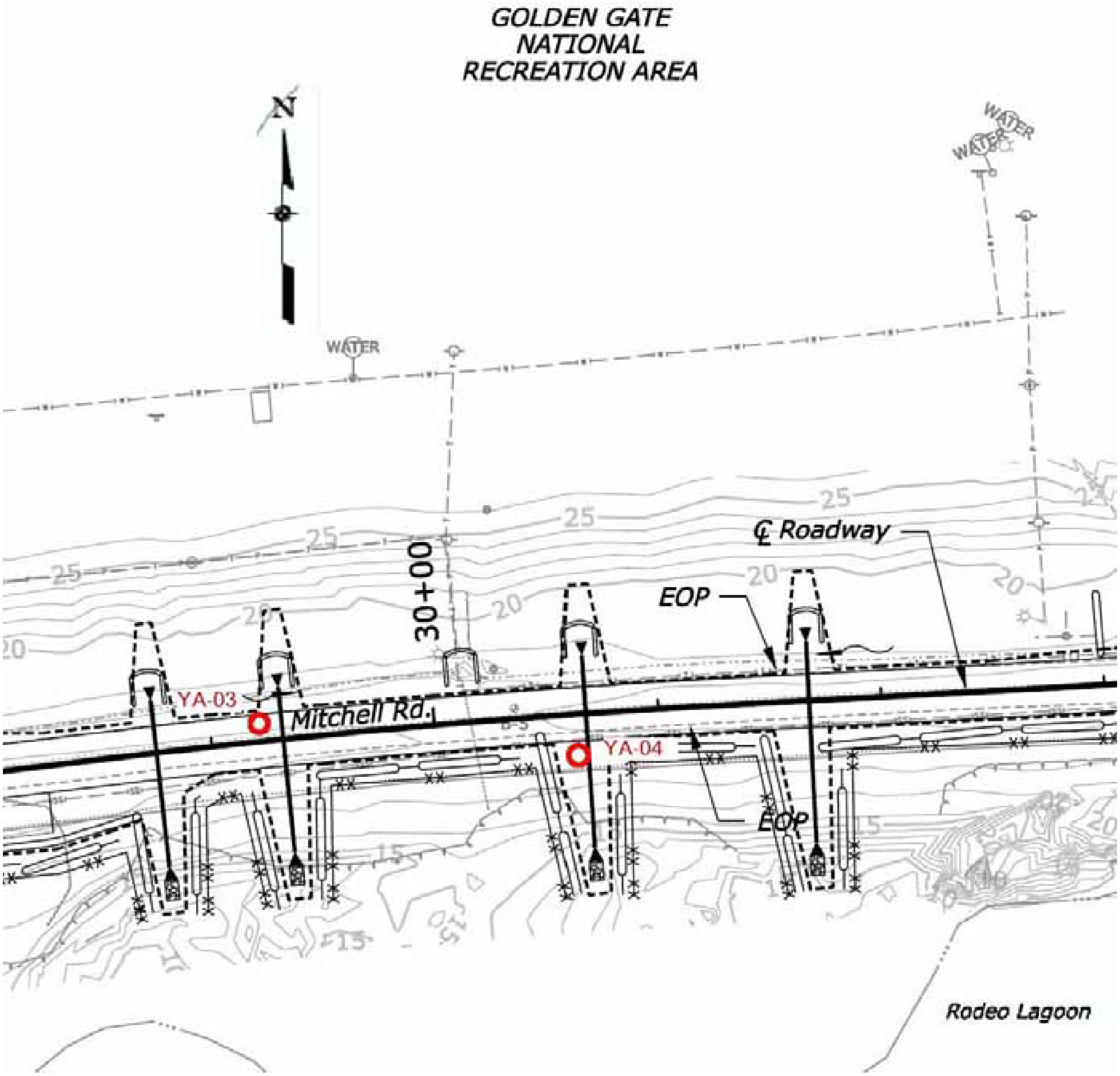
Checked by:

Date: 04/19/11

Locations of Borings YA-1 and YA-2

Project: Bunker & Mitchell Rds

Figure Number: B-2



Yeh and Associates, Inc. Consulting Geotechnical Engineers	Sheet Revision			Designed For: PBS&J		Locations of Borings YA-3 and YA-4	
	Date	Revision/Issue	Checked by	Project Number: 210-189			
5700 E. Evans Avenue Denver, CO 80222				Drawn by: TLA	Date: 04/19/11	Project: Bunker & Mitchell Rds	Figure Number: B-3
Phone: (303) 781-9590 Fax: (303) 781-9583				Checked by:			

Appendix C – Boring Logs

Legend for Symbols Used on Borehole Logs

Sample Types

Shelby Tube

Auger Cuttings

Split Spoon

Modified California

Soil Lithology

USCS Low Plasticity Clay

USCS Clayey Sand

USCS Sandy Gravel

Asphalt

Chert Bedrock

Weathered Chert

High Plasticity Clay

Fill

Clayey Gravel

Silty Sand

Sandy, Clayey Gravel

Lab Test Abbreviations

MC-Moisture Content
DD-Dry Density
#200-Percent Passing #200 Sieve
LL-Liquid Limit
PL-Plastic Limit
PI-Plastic Index
S-Sulphate Content
S/C-Swell/Consolidation
UCCS-Unconfined Compressive Strength
Re-Resistivity
PtL-Point Load Test
AASHTO-AASHTO Classification
USCS-USCS Classification

* Indicates that gradation analysis was performed, Atterberg limits were not performed, but the USCS classification was applied assuming non plastic characteristics

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Danes Drive
Project Number: 28-296B Date: 3/17/09

Boring: **T-1**
Sheet 1 of 1

Boring Began: 2/18/2009
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: T. Allen
Final By: T. Allen
Inclination: Vertical

Completed: 2/18/2009
Drill Bit:
Casing: None
Weather:

Total Depth: 36.5 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.8 ft. 10 inches ASPHALT.	
								0.8 - 8.0 ft. sandy GRAVEL slightly silty, FILL, GP-GM, green-gray, no plasticity, angular.	MC= 3.7 % + #4= 58 % #200= 7 % LL= NV PL= NP PI= NP AASHTO: A-1-a
					8/8/6	14			
								8.0 - 18.0 ft. sandy CLAY , gravelly, CH, reddish brown, medium to high plasticity, moist, soft.	
	10				4/4/2	6			
					2/2/2	4			
								18.0 - 23.0 ft. Weathered CHERT, red-brown, medium to high plasticity, moist, very stiff, angular, clay layers.	
	20				4/9/17	26			MC= 17.7 % + #4= 48 % #200= 17 % LL= 48 PL= 31 PI= 17 AASHTO: A-2-7
								23.0 - 36.5 ft. CHERT, red-brown, hard to very hard, claystone layers.	
					6/10/12	22			MC= 22.1 % + #4= 11 % #200= 10 % LL= 49 PL= 31 PI= 18 AASHTO: A-2-7 USCS: .
	30								Very hard below 32 feet
					18/50				
								Bottom of Hole at 36.5 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Danes Drive
Project Number: 28-296B Date: 3/17/09

Boring: **T-2**
Sheet 1 of 1

Boring Began: 2/18/2009
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: T. Allen
Final By: T. Allen
Inclination: Vertical

Completed: 2/18/2009
Drill Bit:
Casing: None
Weather:

Total Depth: 56.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.9 ft. 10.5 inches ASPHALT.	
								0.9 - 30.0 ft. clayey GRAVEL sandy, GC, gray, low plasticity, moist, dense, angular.	R-Value= 84
					28/28/20	48			
	10				23/31/19	50			
					9/10/7	17			MC= 4.5 % + #4= 55 % #200= 20 % LL= 28 PL= 21 PI= 7 AASHTO: A-2-4
	20				7/15/11	26			
					19/50				pH= 7.9 S= 0 % Re= 2000 ohms-cm
	30				6/20/16	36		**30.0 - 39.5 ft. silty SAND** gravelly, SM, tan, medium plasticity, moist, dense.	
					8/13/8	21			MC= 10.5 % + #4= 43 % #200= 11 % LL= 37 PL= 29 PI= 8 AASHTO: A-2-4
	40				2/3/9	12		**39.5 - 47.5 ft.** Weathered CHERT, moist, very stiff.	
					7/10/11	21			MC= 19.5 % + #4= 36 % #200= 16 % LL= 46 PL= 19 PI= 7 pH= 6.5 S= 0 % Re= 4500 ohms-cm Wet below 47.5 feet
								47.5 - 56.0 ft. CHERT, hard to very hard.	
	50				7/8/12	20			
					50				
								Bottom of Hole at 56.0 ft.	

Project: Danes Drive
Project Number: 28-296B   Date: 3/17/09
Boring: **P-1**
Sheet 1 of 1

Boring Began: 2/17/2009
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: T. Allen
Final By: T. Allen
Inclination: Vertical

Completed: 2/17/2009
Drill Bit:
Casing: None
Weather:

Total Depth: 5.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	▽ 3.9 ft	-	-	-	-
Date	-	-	-	-	-
Time	-	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.9 ft. 11 inches ASPHALT.	
								0.9 - 1.2 ft. 4 inches ABC.	
								1.2 - 5.0 ft. Slightly weathered CHERT, reddish brown.	MC= 1.1 % + #4= 95 % #200= 1 % LL= NV PL= NP PI= NP AASHTO: A-2-7
					9/10/36	46			
	5							Bottom of Hole at 5.0 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Danes Drive
Project Number: 28-296B Date: 3/17/09

Boring: **P-2**
Sheet 1 of 1

Boring Began: 2/17/2009
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: T. Allen
Final By: T. Allen
Inclination: Vertical

Completed: 2/17/2009
Drill Bit:
Casing: None
Weather:

Total Depth: 5.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.7 ft. 8 inches ASPHALT.	
					13/28/23	51		**0.7 - 5.0 ft. clayey GRAVEL**, GP-GC, green-gray, low plasticity, moist, dense, subangular to angular.	MC= 4.2 % + #4= 46 % #200= 11 % LL= 32 PL= 16 PI= 16 R-Value= 70 AASHTO: A-2-6
	5							Bottom of Hole at 5.0 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Danes Drive
Project Number: 28-296B Date: 3/17/09

Boring: **P-3**
Sheet 1 of 1

Boring Began: 2/17/2009
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: T. Allen
Final By: T. Allen
Inclination: Vertical

Completed: 2/17/2009
Drill Bit:
Casing: None
Weather:

Total Depth: 5.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.7 ft. 8 inches ASPHALT.	
								0.7 - 5.0 ft. clayey GRAVEL sandy, GC, brown, medium plasticity, moist, medium stiff, angular.	MC= 4.2 % + #4= 39 % #200= 25 % LL= 35 PL= 18 PI= 17 R-Value= 80 AASHTO: A-2-6 (0)
					5/8/9	17			
	5							Bottom of Hole at 5.0 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Danes Drive		Boring: **P-4**
Project Number: 28-296B	Date: 3/17/09	Sheet 1 of 1

Boring Began: 2/17/2009
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: T. Allen
Final By: T. Allen
Inclination: Vertical

Completed: 2/17/2009
Drill Bit:
Casing: None
Weather:

Total Depth: 5.0 ft
Ground Elevation:
Location:
Coordinates: N: E:

Ground Water Notes:

Depth	-	-	-	-
Date	-	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
								0.0 - 0.8 ft. 10 inches ASPHALT.	
								0.8 - 1.3 ft. 6 inches ABC.	MC= 3.6 % + #4= 51 % #200= 12 % LL= 24 PL= 17 PI= 7 AASHTO: A-2-6
					5/50			**1.3 - 5.0 ft.** Slightly weathered CHERT, reddish brown, hard.	
	5							Bottom of Hole at 5.0 ft.	

BORING LOG 28-296B ENG. DANES DRIVE.GPJ YEH ASSOCIATES.GDT 2/15/12

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Bunker & Mitchell Roads
Project Number: 210-189 Date: 2/9/11

Boring: **YA-01**
Sheet 1 of 1

Boring Began: 2/9/2011
Drilling Method: Hollow-Stem Auger
Drill: CME 75
Driller: Precision Sampling
Logged By: W. Hoon
Final By: T. Allen
Inclination: 90

Completed: 2/9/2011
Drill Bit: N/A
Casing: None
Weather: Sunny

Total Depth: 31.5 ft
Ground Elevation:
Location: Sta 12+00 in Parking Lot
Coordinates: N: E:

Ground Water Notes:

Depth	∇ 25.0 ft	-	-	-	-
Date	2/9/11	-	-	-	-
Time	-	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
	0–5							**0.0 - 31.5 ft. clayey SAND** trace gravel, SC, brown, medium plasticity, moist to wet, very loose to medium dense, sandy clay and sandy gravel lenses.	
	5				1/2/1	3			#200= 38% LL= 31 PL= 16 PI= 15 pH= 6.5 S= 0.01% Re= 1737ohms-cm AASHTO: A-6 (2) USCS: SC Increased clay at 8 ft. to 12 ft.
	10				3/2/3	5			#200= 11% LL= 47 PL= 42 PI= 20 AASHTO: A-7-6 (4) USCS: SC
	15				Pushed				
					Pushed				
	20				2/4/9	13			
	∇ 25				5/9/11	20			
	30				1/9/12	21			
								Bottom of Hole at 31.5 ft.	

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: Bunker & Mitchell Roads Project Number: 210-189 Date: 2/9/11	Boring: **YA-02** Sheet 1 of 1

Boring Began: 2/9/2011
Drilling Method: HQ Wireline
Drill: CME 75
Driller: Precision Sampling
Logged By: W. Hoon
Final By: T. Allen
Inclination: 90

Completed: 2/9/2011
Drill Bit: N/A
Casing: None
Weather: Sunny

Total Depth: 31.5 ft
Ground Elevation:
Location: Sta 12+25 in EB Lane
Coordinates: N: E:

Ground Water Notes:

Depth	∇ 22.0 ft	-	-	-
Date	2/9/11	-	-	-
Time	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
	0.0							**0.0 - 8.0 ft. clayey SAND**, SC, dark brown, medium plasticity, moist, very loose, sandy clay and gravelly lenses.	
	5				1/1/1	2			MC= 18.7% #200= 39% LL= 33 PL= 21 PI= 12 AASHTO: A-6 (1) USCS: SC
	8.0							**8.0 - 19.0 ft. sandy CLAY** trace gravel, CL, black to dark gray, medium plasticity, moist, soft.	
	10				1/1/1	2			
					1/1/2	3			
	15				Pushed				MC= 20.9% DD= 105.1pcf USCS: CL
	19.0							**19.0 - 22.5 ft. clayey SAND**, SC, dark brown, medium plasticity, moist, very loose, sandy clay and gravelly lenses.	
	20				0/0/1	1			
	∇				Pushed			**22.5 - 29.0 ft. sandy GRAVEL** clayey, GP-GC, gray, no to low plasticity, wet, dense.	
	25				4/19/17	36			
	29.0							**29.0 - 31.5 ft. clayey SAND** gravelly, SC, black to dark gray, medium plasticity, wet, dense.	
	30				5/12/19	31			MC= 15.5% #200= 16% LL= 31 PL= 20 PI= 11 AASHTO: A-2-6 (0) USCS: SC
								Bottom of Hole at 31.5 ft.	

YEH AND ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING CONSULTANTS

Project: Bunker & Mitchell Roads
Project Number: 210-189 Date: 2/9/11

Boring: **YA-03**
Sheet 1 of 1

Boring Began: 2/9/2011
Drilling Method: HQ Wireline
Drill: CME 75
Driller: Precision Sampling
Logged By: W. Hoon
Final By: T. Allen
Inclination: 90

Completed: 2/9/2011
Drill Bit: N/A
Casing: None
Weather: Sunny

Total Depth: 21.5 ft
Ground Elevation:
Location: Sta 29+47 in WB Lane
Coordinates: N: E:

Ground Water Notes:

Depth	∇ 15.0 ft	-	-	-	-
Date	2/9/11	-	-	-	-
Time	-	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
	0							**0.0 - 13.0 ft. sandy CLAY** trace gravel, CL, gray mottled with tan-brown, medium plasticity, moist, very soft to stiff.	
	5				3/1/1	2			MC= 17.7% -#200= 51% LL= 32 PL= 17 PI= 15 pH= 7.5 S= 0% Re= 1431ohms-cm AASHTO: A-6 (4) USCS: CL
					Pushed				MC= 19.6% DD= 111.6pcf USCS: CL
	10				0/5/8	13			
	13							**13.0 - 18.0 ft. clayey SAND**, SC, yellowish brown mottled with brown, medium plasticity, moist to wet, medium dense, trace organics.	
∇	15				1/5/16	21			MC= 14.5% -#200= 41% LL= 28 PL= 17 PI= 11 AASHTO: A-6 (1) USCS: SC
	18							**18.0 - 21.5 ft. sandy CLAY** trace gravel, CL, yellowish brown, medium plasticity, moist, stiff.	
	20				1/4/5	9			
	21.5							Bottom of Hole at 21.5 ft.	

YEH AND ASSOCIATES, INC. GEOTECHNICAL ENGINEERING CONSULTANTS	Project: Bunker & Mitchell Roads Project Number: 210-189 Date: 2/9/11	Boring: **YA-04** Sheet 1 of 1

Boring Began: 2/9/2011
Drilling Method: HQ Wireline
Drill: CME 75
Driller: Precision Sampling
Logged By: W. Hoon
Final By: T. Allen
Inclination: 90

Completed: 2/9/2011
Drill Bit: N/A
Casing: None
Weather: Sunny

Total Depth: 21.5 ft
Ground Elevation:
Location: Sta 30+57 in EB Shoulder, 23'S of Center
Coordinates: N: E:

Ground Water Notes:

Depth	⊻ 14.5 ft	-	-	-	-
Date	2/9/11	-	-	-	-
Time	-	-	-	-	-

Elevation (feet)	Depth (feet)	Run / Sample Type	Recovery (%)	Rock RQD	Soil Samples Blows per 6 in	Soil Samples N	Lithology	Material Description	Field Notes and Lab Tests
	0							**0.0 - 8.0 ft. clayey SAND**, SC, dark brown, no to low plasticity, moist, loose.	
	5	X			1/3/2	5			
	8							**8.0 - 21.5 ft. sandy CLAY** trace gravel, CL, black to yellowish brown, medium to high plasticity, moist, soft to stiff.	
	10	X			3/1/1	2			MC= 22.1% -#200= 57% LL= 46 PL= 20 PI= 26 AASHTO: A-7-6 (12) USCS: CL
	12.5	X			0/1/2	3			
	⊻ 14.5								
	15	X			1/1/3	4			
	20	X			3/3/6	9			
	21.5							Bottom of Hole at 21.5 ft.	

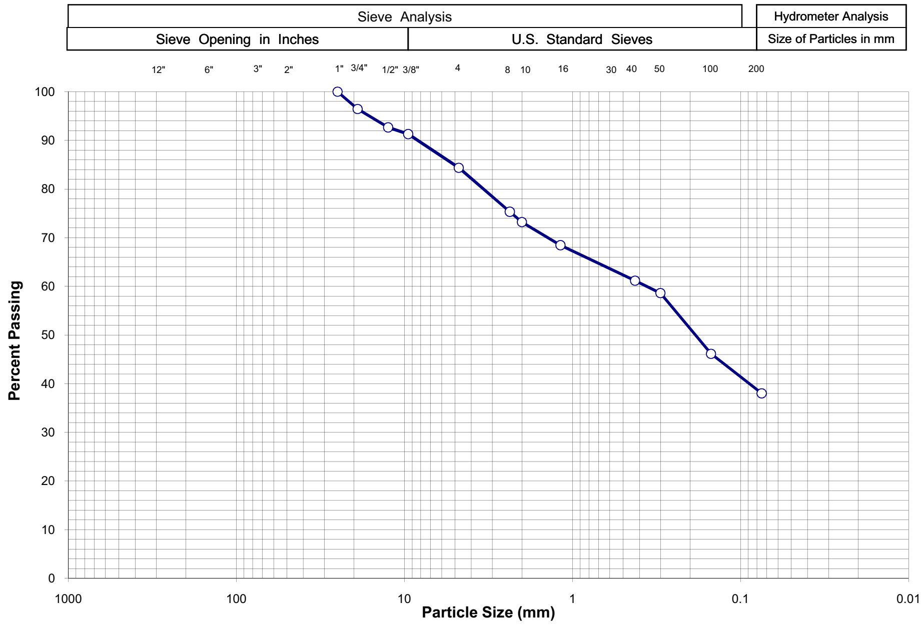
Appendix D – Laboratory Test Results

YEH & ASSOCIATES, INC

Summary of Laboratory Test Results

Project No: 210-189 Project Name: Bunker & Mitchell Roads Date: 3/5/2011

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			pH	Water Soluble Sulfate %	% Swell (+) / Consoli-dation (-)	Unconf. Comp. Strength (psf)	Resistivity	CLASSIFICATION	
Boring No.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI						AASHTO	USCS
YA-01	0-5	BULK			16	46	38	31	16	15	6.5	0.01			1737	A-6 (2)	SC
YA-01	10-11.5	CAL			11	47	42	42	22	20						A-7-6 (4)	SC
YA-01	20-21.5	SS			12	51	37	33	20	13						A-6 (1)	SC
YA-02	5-6.5	SS			13	48	39	33	21	12						A-6 (1)	SC
YA-02	15-17	SH	20.9	105.1													CL
YA-02	30-31.5	SS			29	55	16	31	20	11						A-2-6 (0)	SC
YA-03	5-6.5	SS			6	43	51	32	17	15	7.5	0.00			1431	A-6 (4)	CL
YA-03	6.5-8.5	SH	19.6	111.6													CL
YA-03	15-16.5	SS			14	45	41	28	17	11						A-6 (1)	SC
YA-04	10-11.5	SS			8	35	57	46	20	26						A-7-6 (12)	CL



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	96
½"	93
⅜"	91
#4	84
#10	73
#40	61
#200	38.0

Gravel (%)	16	LL	31	Project Name:	Bunker & Mitchell Rds
Sand (%)	46	PL	16	Boring:	YA-01
Fines (%)	38	PI	15	Sample Depth (ft):	0-5
Sample Description:	clayey SAND	SC		A-6	(2)

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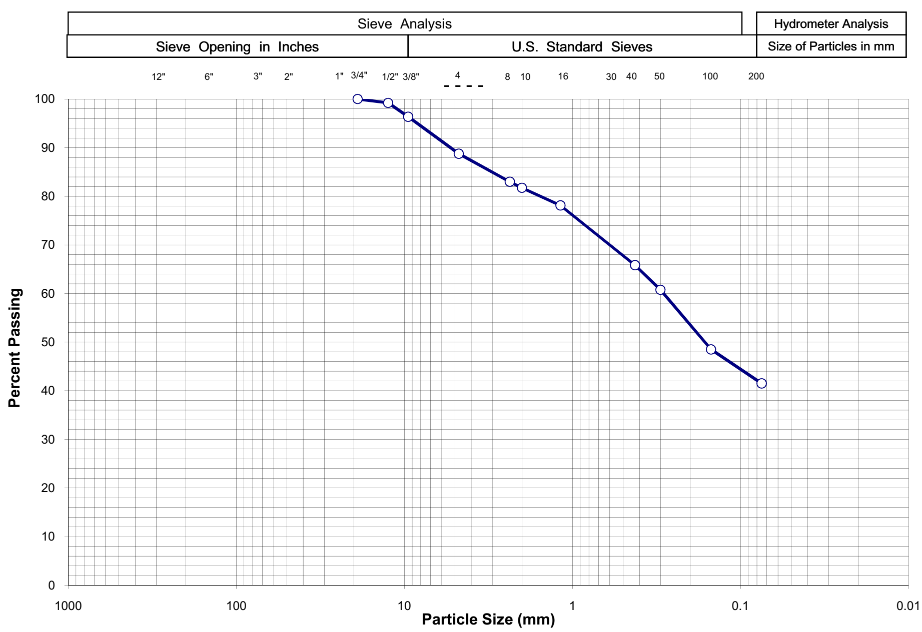
SIEVE ANALYSIS

Drawn By:		Project No.:	210-189
Checked By:	TLA	Figure No.:	-
Date:	03/03/11		

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	99
⅜"	96
#4	89
#10	82
#40	66
#200	41.5

Gravel (%)	11	LL	42	Project Name:	Bunker & Mitchell Rds
Sand (%)	47	PL	22	Boring:	YA-01
Fines (%)	42	PI	20	Sample Depth (ft):	10
Sample Description:	clayey SAND	SC		A-7-6	(4)

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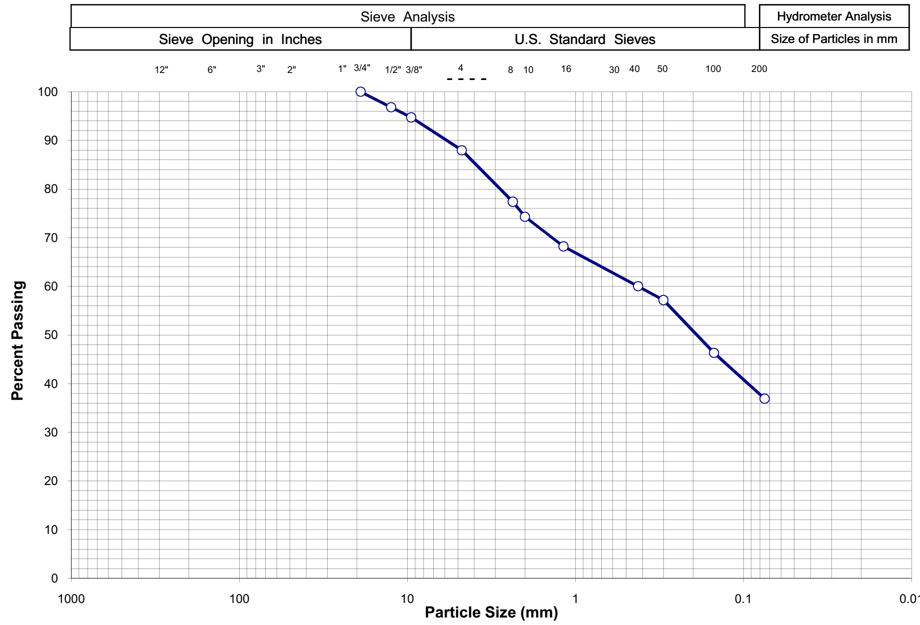
SIEVE ANALYSIS

Drawn By:		Project No.:	210-189
Checked By:	TLA	Figure No.:	-
Date:	03/03/11		

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	97
⅜"	95
#4	88
#10	74
#40	60
#200	36.9

Gravel (%)	12	LL	33	Project Name:	Bunker & Mitchell Rds
Sand (%)	51	PL	20	Boring:	YA-01
Fines (%)	37	PI	13	Sample Depth (ft):	20-21.5
Sample Description:	clayey SAND	SC		A-6	(1)

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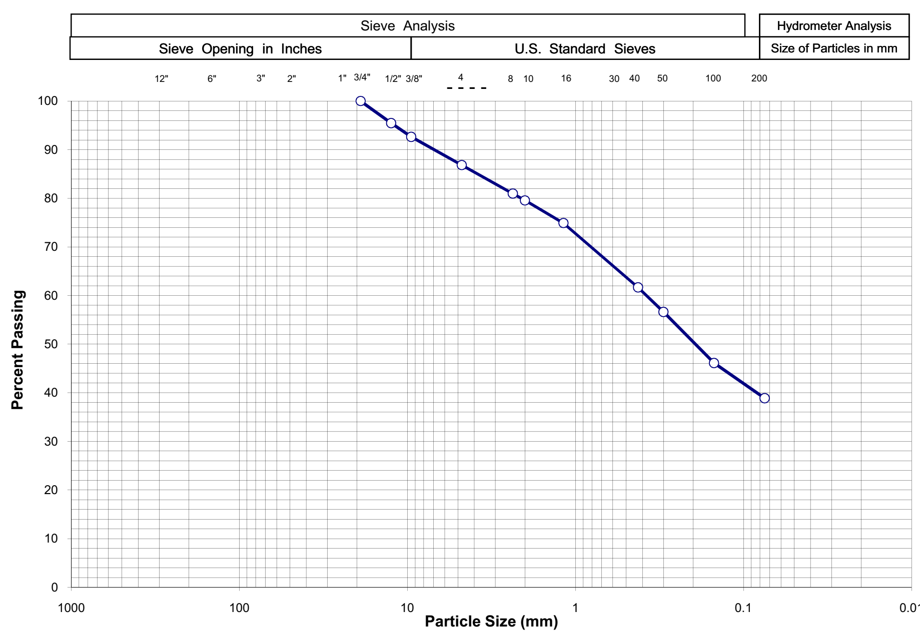
SIEVE ANALYSIS

Drawn By:		Project No.:	210-189
Checked By:	TLA	Figure No.:	-
Date:	03/03/11		

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	95
⅜"	93
#4	87
#10	80
#40	62
#200	38.9

Gravel (%)	13	LL	33	Project Name:	Bunker & Mitchell Rds
Sand (%)	48	PL	21	Boring:	YA-02
Fines (%)	39	PI	12	Sample Depth (ft):	5-6.5
Sample Description:	clayey SAND	SC		A-6	(1)

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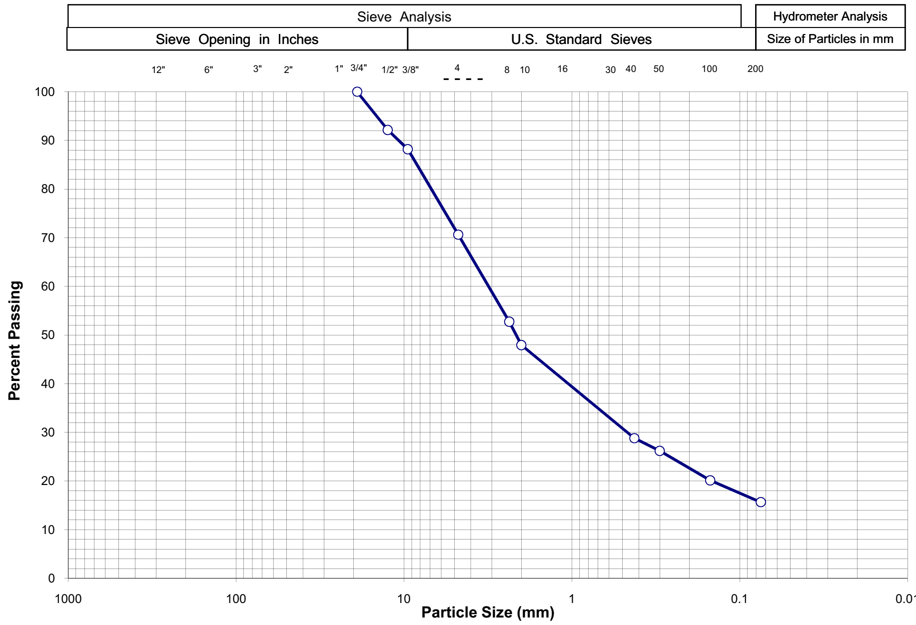
SIEVE ANALYSIS

Drawn By:		Project No.:	210-189
Checked By:	TLA	Figure No.:	-
Date:	03/03/11		

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	92
⅜"	88
#4	71
#10	48
#40	29
#200	15.7

Gravel (%)	29	LL	31	Project Name:	Bunker & Mitchell Rds
Sand (%)	55	PL	20	Boring:	YA-02
Fines (%)	16	PI	11	Sample Depth (ft):	30-31.5
Sample Description:	clayey SAND	SC		A-2-6	(0)

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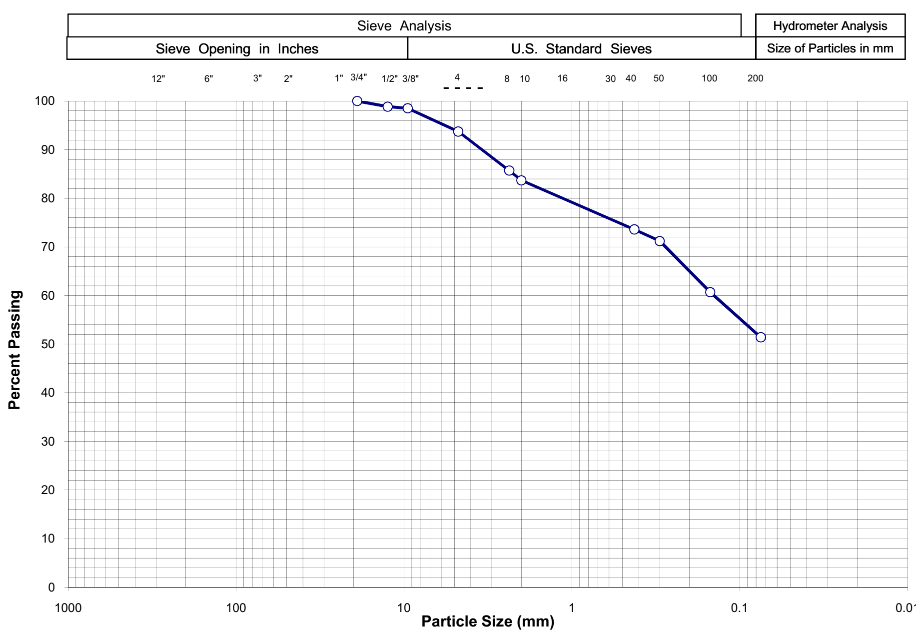
SIEVE ANALYSIS

Drawn By:		Project No.:	210-189
Checked By:	TLA	Figure No.:	-
Date:	03/03/11		

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	99
⅜"	99
#4	94
#10	84
#40	74
#200	51.4

Gravel (%)	6	LL	32	Project Name:	Bunker & Mitchell Rds
Sand (%)	43	PL	17	Boring:	YA-03
Fines (%)	51	PI	15	Sample Depth (ft):	5-6.5
Sample Description:	sandy CLAY	CL		A-6	(4)

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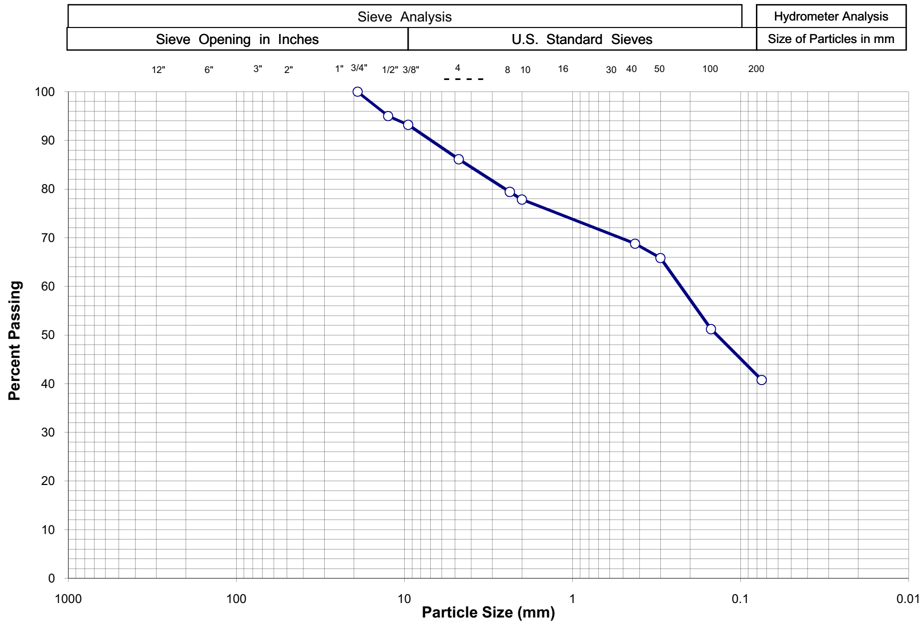
SIEVE ANALYSIS

Drawn By:		Project No.:	210-189
Checked By:	TLA	Figure No.:	-
Date:	03/03/11		

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	100
½"	95
⅜"	93
#4	86
#10	78
#40	69
#200	40.8

Gravel (%)	14	LL	28	Project Name:	Bunker & Mitchell Rds
Sand (%)	45	PL	17	Boring:	YA-03
Fines (%)	41	PI	11	Sample Depth (ft):	15-16.5
Sample Description:	clayey SAND	SC		A-6	(1)

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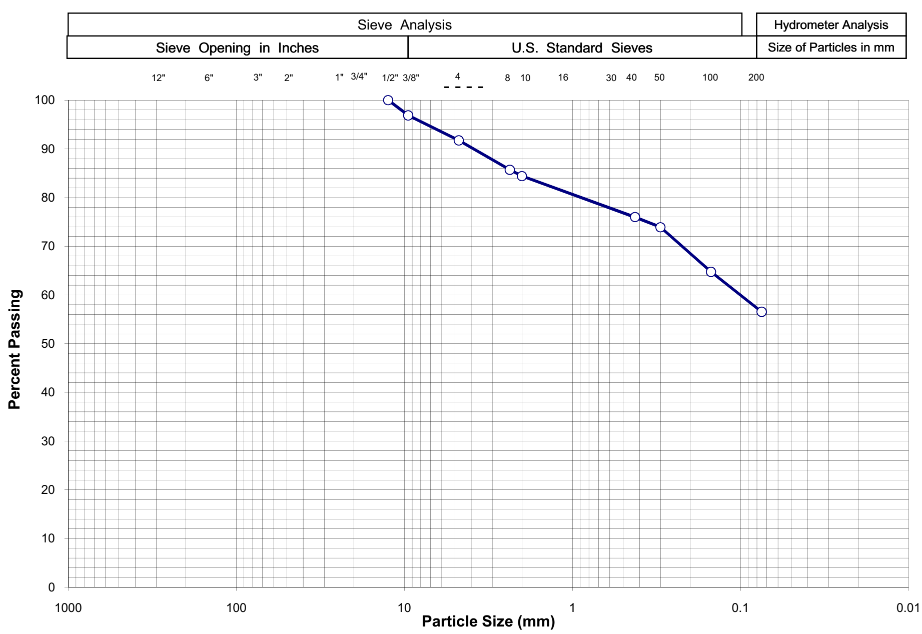
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By: Checked By: Date:	TLA 03/03/11	Project No.:	210-189
		Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	-
¾ "	-
½"	100
⅜"	97
#4	92
#10	84
#40	76
#200	56.5

Gravel (%)	8	LL	46	Project Name:	Bunker & Mitchell Rds
Sand (%)	35	PL	20	Boring:	YA-04
Fines (%)	57	PI	26	Sample Depth (ft):	10-11.5
Sample Description:	sandy CLAY	CL		A-7-6	(12)

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Geotechnical Engineering Consultants

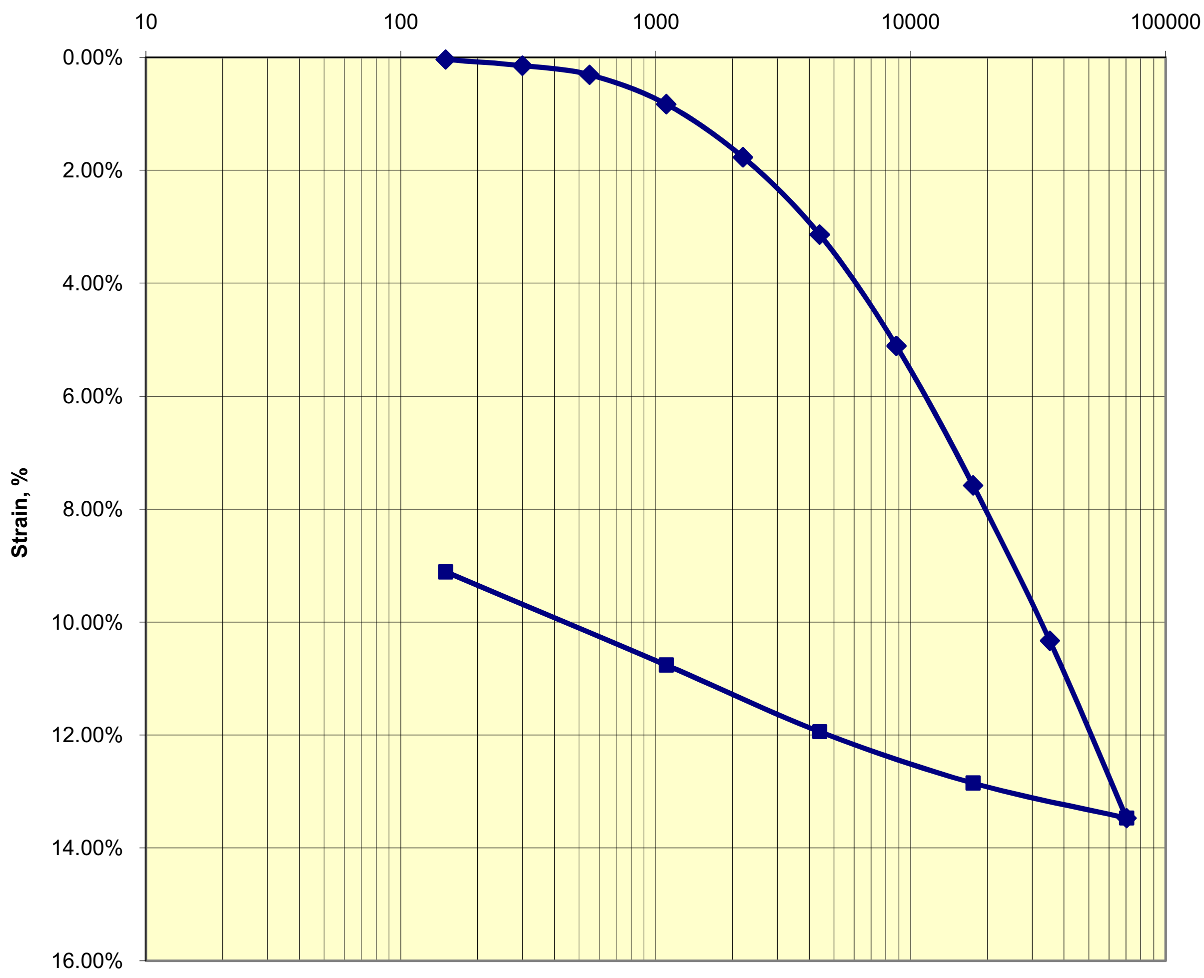
SIEVE ANALYSIS

Drawn By: Checked By: Date:	TLA 03/03/11	Project No.:	210-189
		Figure No.:	-

Revised 04/27/2004

Consolidation Test
ASTM D2435

Job No.:	687-002	**Boring:**	YA-02	**Run By:**	MD
Client:	Yeh & Associates, Inc.	**Sample:**		**Reduced:**	PJ
Project:	Mitchell Road - 210-189	**Depth, ft.:**	15-17(Tip)	**Checked:**	PJ/DC
Soil Type:	Mottled Gray Clayey SAND w/ Gravel			**Date:**	2/24/2011



Ass. Gs = 2.75	**Initial**	**Final**
Moisture %:	20.9	17.6
Dry Density, pcf:	105.1	115.8
Void Ratio:	0.633	0.482
% Saturation:	90.7	100

Remarks: Tube slightly out of round and dented. Moderate patching required due to gravel.

Moisture-Density-Porosity Report

Cooper Testing Labs, Inc. (ASTM D 2937)

CTL Job No:	687-002	**Project No.**	210-189	**By:**	RU
Client:	Yeh & Associates, Inc.	**Date:**	02/24/11		
Project Name:	Mitchell Rd	**Remarks:**			

Boring: **Sample:** **Depth, ft:**	YA-03 6.5-8.5(Tip)							
Visual Description:	Brown Clayey SAND w/ Gravel							
Actual G_s	2.75							
Assumed G_s								
Moisture, %	19.6							
Wet Unit wt, pcf	133.5							
Dry Unit wt, pcf	111.6							
Dry Bulk Dens.pb, (g/cc)	1.79							
Saturation, %	99.9							
Total Porosity, %	35.0							
Volumetric Water Cont, Θw	35.0							
Volumetric Air Cont., Θa	0.0							
Void Ratio	0.54							
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

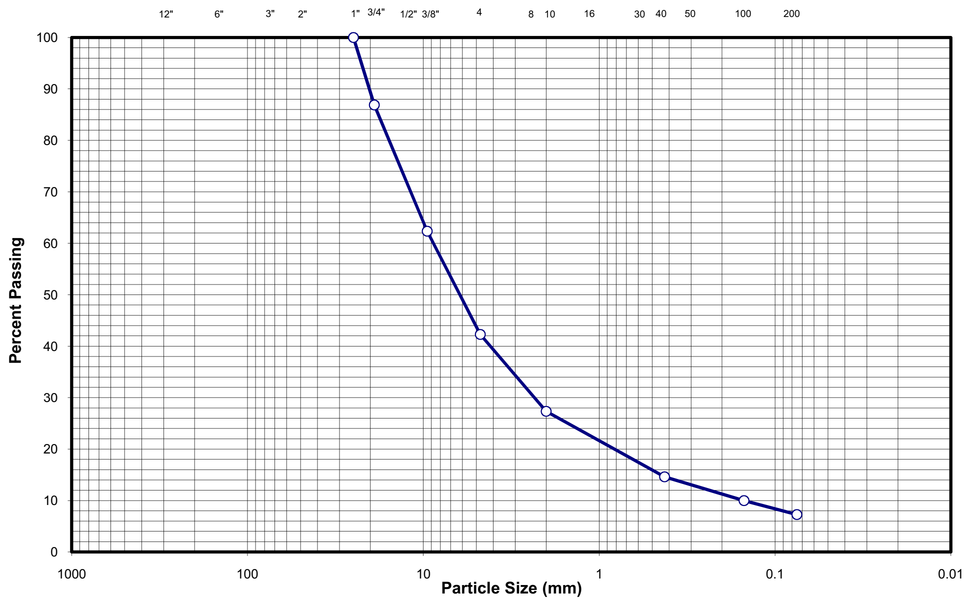
YEH & ASSOCIATES, INC

Summary of Laboratory Test Results

Project No: 28-296 Project Name: Danes Drive Date: 3/17/2009

Sample Location			Sample Type	Natural Moisture Content (%)	Gradation			Atterberg			pH	Water Soluble Sulfate %	% Swell (+) / Consoli-dation (-)	Resistivity ohm.cm	R-VALUE	CLASSIFICATION	
Boring No.	Depth (m)	Depth (ft)			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI						AASHTO	USCS
T-1	0-1.5	0-5	SS	3.7	58	35	7	NV	NP	NP						A-1-a (0)	GP-GM
T-1	6.1	20	SS	17.7	48	31	17	48	31	17						A-2-7 (0)	SP-SM
T-1	7.6	25	SS	22.1	11	78	10	49	31	18						A-2-7 (0)	SP-SM
T-2	.3-1.5	1-5	Bulk												84		
T-2	4.6	15	SS	4.5	55	25	20	28	21	7						A-2-4 (0)	GM-GC
T-2	7.6	25	SS								7.9	0.000		2000			
T-2	10.7	35	SS	10.5	43	46	11	37	29	8						A-7-4 (0)	SP-SM
T-2	12.2	40	SS								6.5	0.000		4500			
T-2	13.7	45	SS	19.5	36	48	16	46	19	17						A-2-7 (0)	SM
P-1	.5-.6	1.5-2	Bulk	1.1	95	4	1	NV	NP	NP						A-2-7 (0)	GP
P-2	.3-1.5	1-5	Bulk	4.2	46	43	11	32	16	16					70	A-2-6 (0)	GP-GC
P-3	.3-1.5	1-5	Bulk	4.2	39	36	25	35	18	17					80	A-2-6 (0)	GC
P-4	.3-.6	1-2	Bulk	3.6	51	36	12	24	17	7						A-2-6 (0)	GM-GC

Sieve Analysis		Hydrometer Analysis
Sieve Opening in Inches	U.S. Standard Sieves	Size of Particles in mm



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	87
½"	-
⅜"	62
#4	42
#10	27
#40	15
#200	7

Gravel (%)	58	LL	NV	Project Name:	Danes Drive
Sand (%)	35	PL	NP	Sample ID:	T-1
Fines (%)	7	PI	NP	Sample Depth:	1.5 m (5 ft.)
Sample Description:	GRAVEL and SAND				

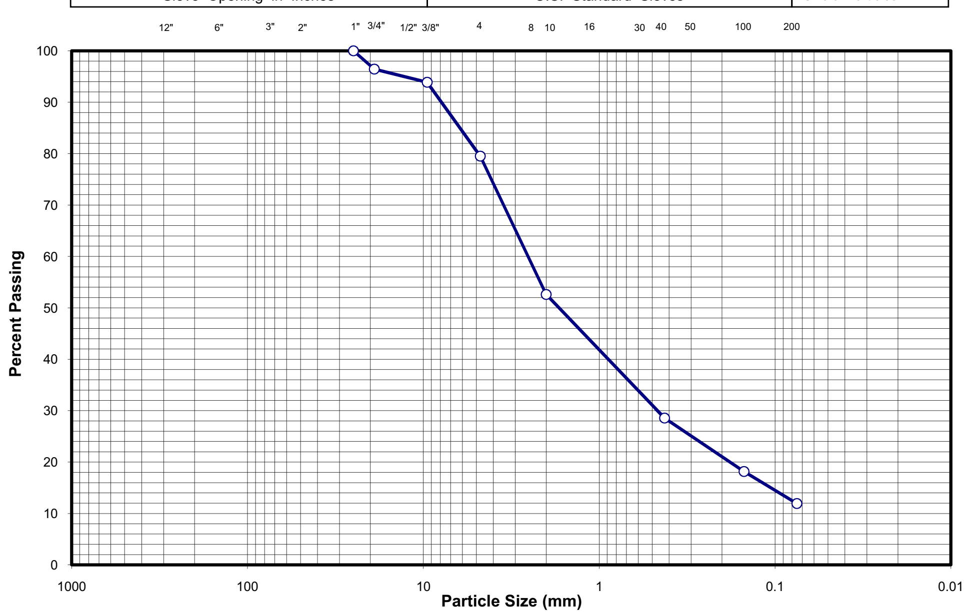
Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By: BB	Checked By: TA	Project No.:	28-296
Date: 02/25/09		Figure No.:	-

Revised 04/27/2004

Sieve Analysis		Hydrometer Analysis
Sieve Opening in Inches	U.S. Standard Sieves	Size of Particles in mm



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	96
½"	-
⅜"	94
#4	80
#10	53
#40	29
#200	12

Gravel (%)	20	LL	48	Project Name:	Danes Drive
Sand (%)	68	PL	31	Sample ID:	T-1
Fines (%)	12	PI	17	Sample Depth:	6.1 m (20 ft)
Sample Description:	Weathered Bedrock				

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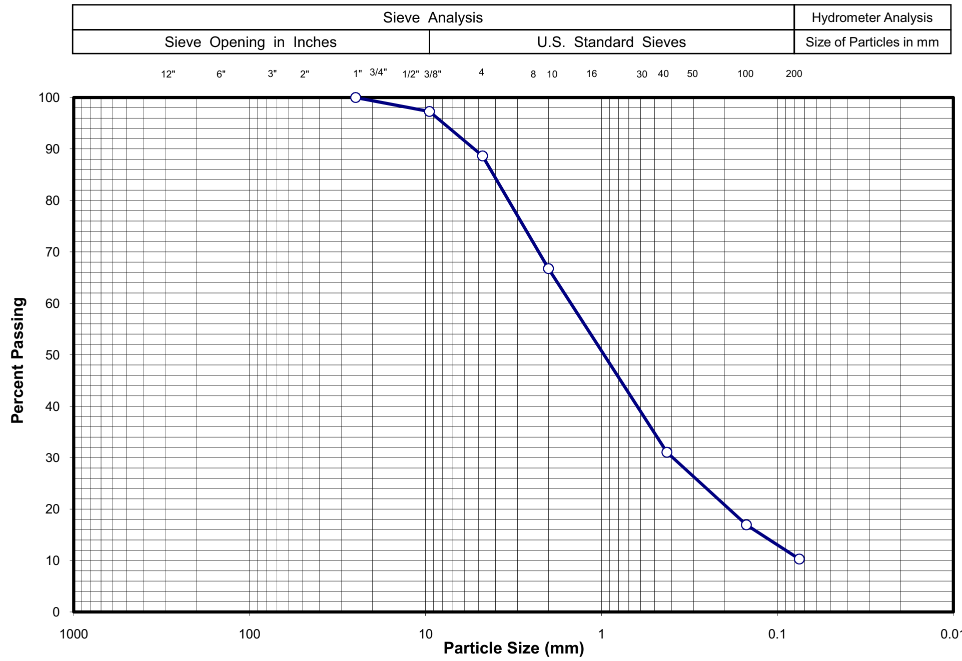
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By: BB	Checked By: TA	Project No.:	28-296
Date: 02/25/09		Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	-
½"	-
⅜"	97
#4	89
#10	67
#40	31
#200	10

Gravel (%)	11	LL	49	Project Name:	Danes Drive
Sand (%)	79	PL	31	Sample ID:	T-1
Fines (%)	10	PI	18	Sample Depth:	7.6 m (25 ft)
Sample Description:	Weathered Bedrock				

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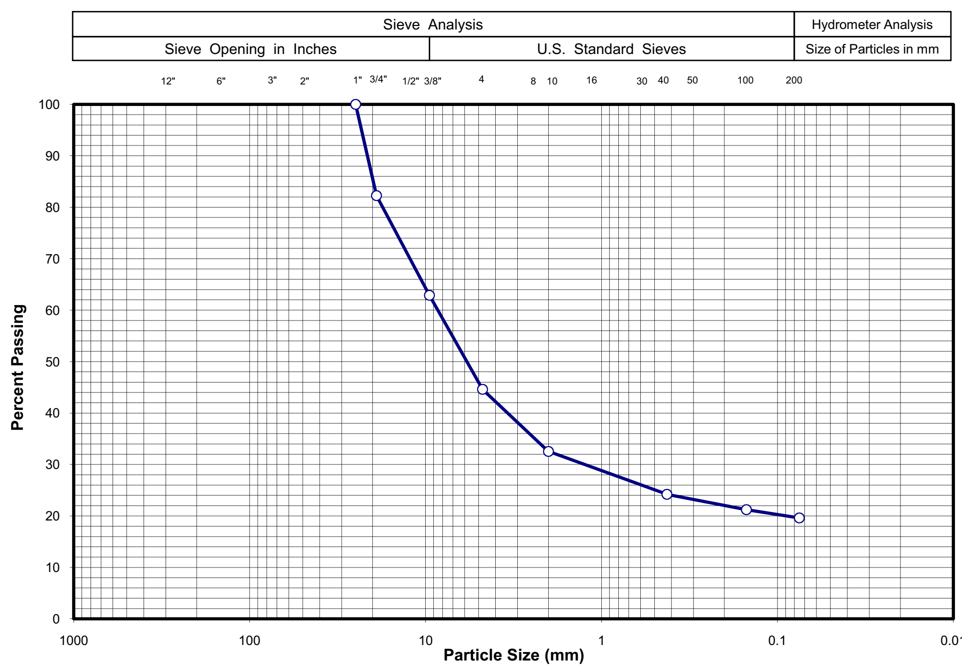
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/25/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	82
½"	-
⅜"	63
#4	45
#10	33
#40	24
#200	20

Gravel (%)	55	LL	28	Project Name:	Danes Drive
Sand (%)	25	PL	21	Sample ID:	T-2
Fines (%)	20	PI	7	Sample Depth:	4.6 m (15 ft)
Sample Description:	GRAVEL with sand and clay				

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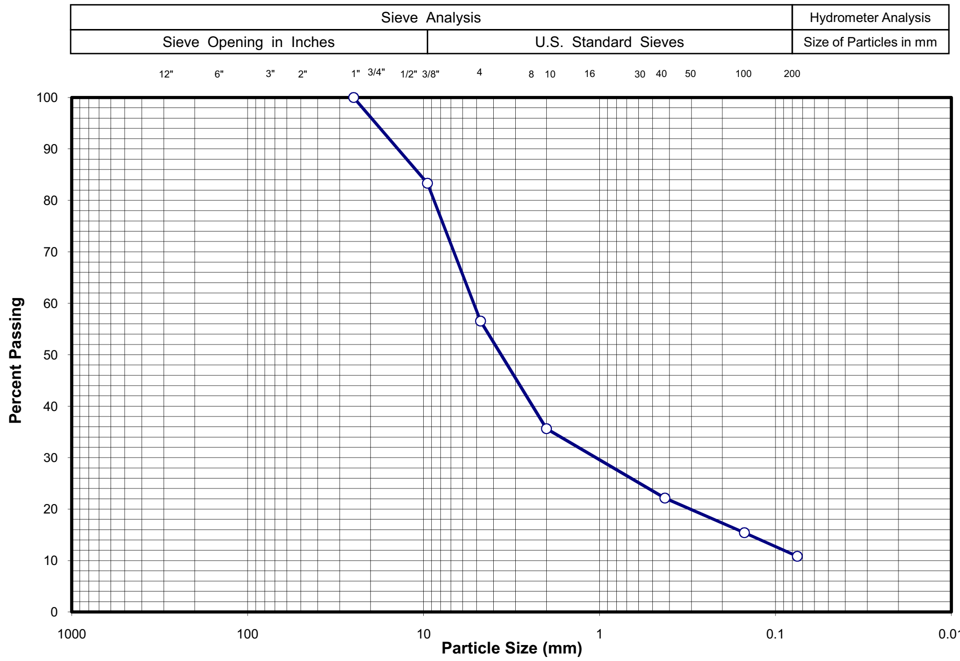
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/26/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	-
½"	-
⅜"	83
#4	57
#10	36
#40	22
#200	11

Gravel (%)	43	LL	37	Project Name:	Danes Drive
Sand (%)	46	PL	29	Sample ID:	T-2
Fines (%)	11	PI	8	Sample Depth:	10.7 m (35 ft)
Sample Description:	silty SAND with gravel				

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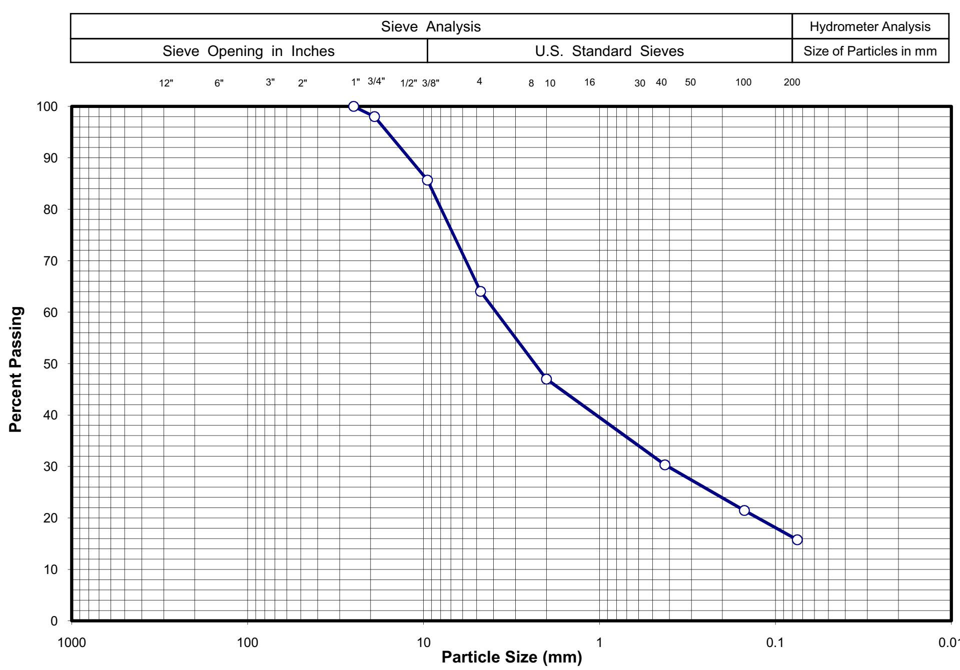
SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/26/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	98
½"	-
⅜"	86
#4	64
#10	47
#40	30
#200	16

Gravel (%)	36	LL	46	Project Name:	Danes Drive
Sand (%)	48	PL	29	Sample ID:	T-2
Fines (%)	16	PI	17	Sample Depth:	13.7 m (45 ft)
Sample Description:	silty SAND				

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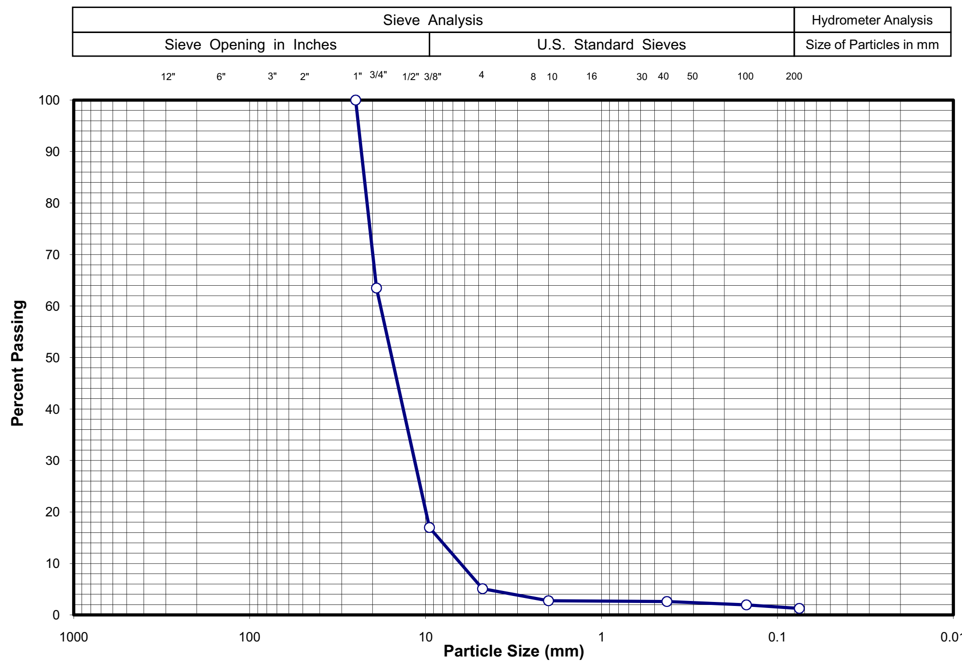
SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/26/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	63
½"	-
⅜"	17
#4	5
#10	3
#40	3
#200	1

Gravel (%)	95	LL	NV	Project Name:	Danes Drive
Sand (%)	4	PL	NP	Sample ID:	P-1
Fines (%)	1	PI	NP	Sample Depth:	0.5-0.6 m (1.5-2.0 ft)
Sample Description:	slightly sandy GRAVEL				

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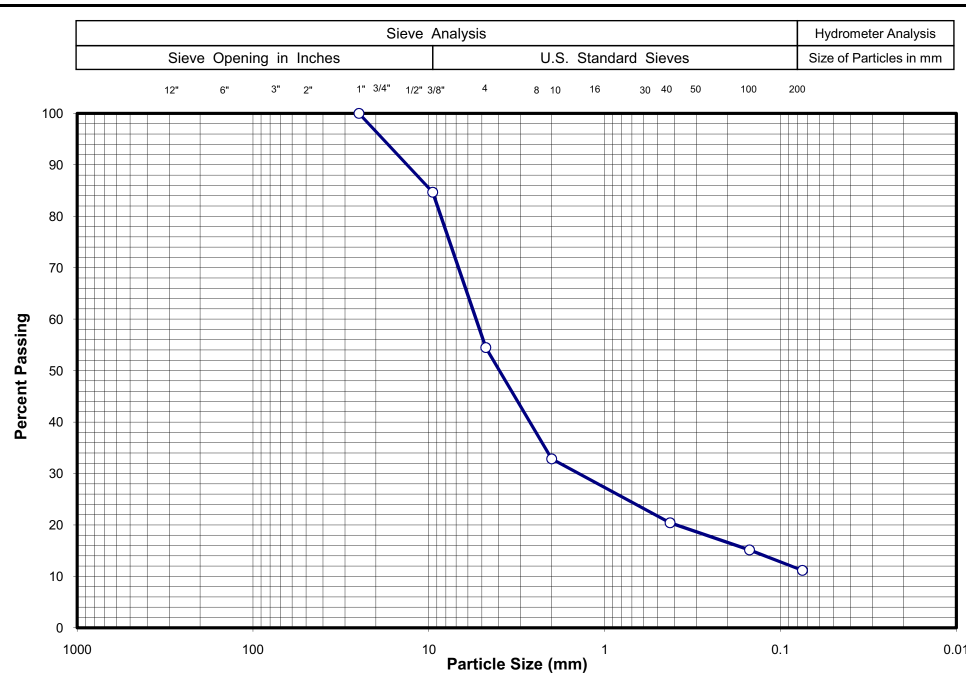
SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/25/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	-
½"	-
⅜"	85
#4	54
#10	33
#40	20
#200	11

Gravel (%)	46	LL	32	Project Name:	Danes Drive
Sand (%)	43	PL	16	Sample ID:	P-2
Fines (%)	11	PI	16	Sample Depth:	0.3-1.5 m (1-5 ft)
Sample Description:	clayey GRAVEL with sand				

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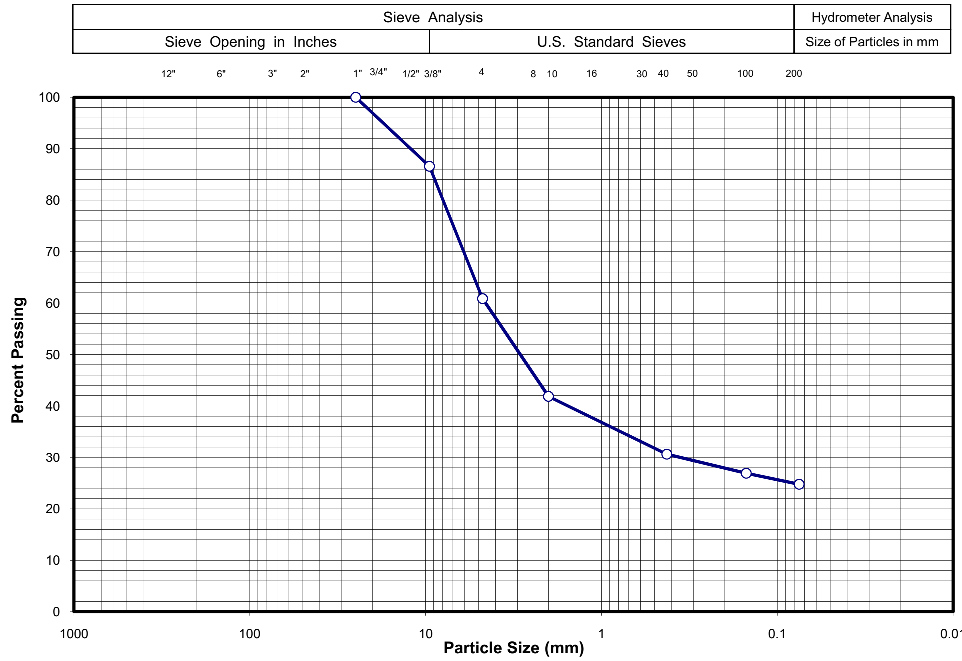
SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/25/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	-
½"	-
⅜"	87
#4	61
#10	42
#40	31
#200	25

Gravel (%)	39	LL	35	Project Name:	Danes Drive
Sand (%)	36	PL	18	Sample ID:	P-3
Fines (%)	25	PI	17	Sample Depth:	0.3-1.5 m (1-5 ft)
Sample Description:	sandy CLAY with gravel				

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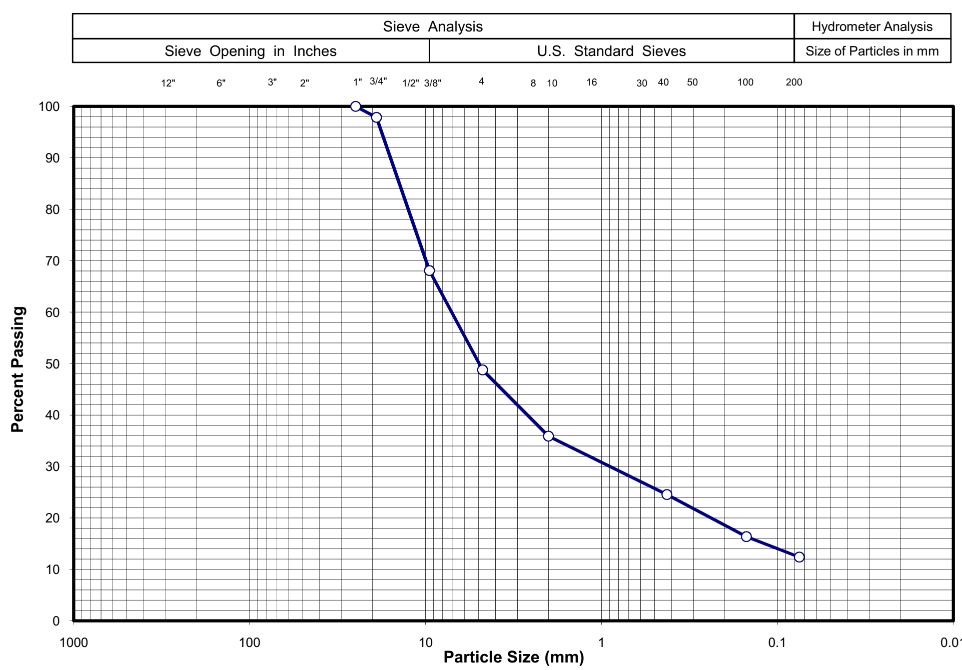
SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/25/09	Figure No.:	-

Revised 04/27/2004



Sieve Size	% Passing
3"	-
2 ½"	-
2"	-
1 ½"	-
1"	100
¾ "	98
½"	-
⅜"	68
#4	49
#10	36
#40	25
#200	12

Gravel (%)	51	LL	24	Project Name:	Danes Drive
Sand (%)	37	PL	17	Sample ID:	P-4
Fines (%)	12	PI	7	Sample Depth (m):	0.3-0.6 m (1-2 ft)
Sample Description:	clayey GRAVEL with sand				

Yeh & Associates, Inc.
Geotechnical Engineering Consultants

SIEVE ANALYSIS

Drawn By:	BB	Project No.:	28-296
Checked By:	TA		
Date:	02/25/09	Figure No.:	-

Revised 04/27/2004

R-value Test Report (Caltrans 301)

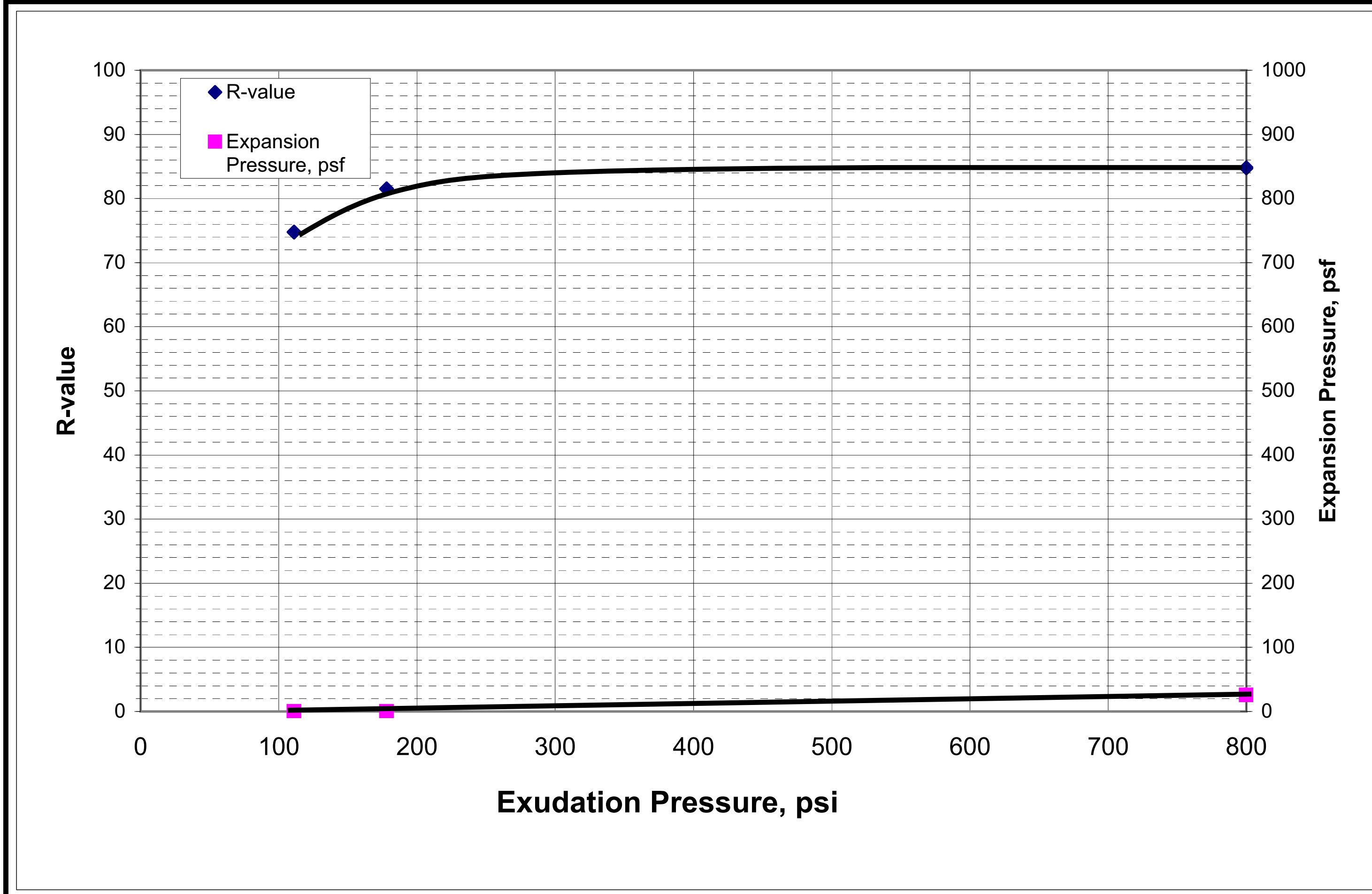
COOPER TESTING LABORATORY

Job No.:	687-001	**Date:**	02/25/09
Client:	Yeh and Associates, Inc.	**Tested**	MD
Project:	Danes Drive - 28-296	**Reduced**	RU
Sample	T-2 @ 1-5'	**Checked**	DC
Soil Type:	Brown Clayey SAND w/ Gravel		

Initial Moisture,	2.1%	
R-value by Stabilometer	84	
Expansion Pressure	15	psf

Specimen Number	A	B	C	D
Exudation Pressure, psi	178	111	800	
Prepared Weight, grams	1300	1300	1300	
Final Water Added, grams/cc	47	54	32	
Weight of Soil & Mold, grams	3339	3313	3306	
Weight of Mold, grams	2102	2079	2106	
Height After Compaction, in.	2.48	2.42	2.51	
Moisture Content, %	5.8	6.3	4.6	
Dry Density, pcf	142.8	145.2	138.4	
Expansion Pressure, psf	0.0	0.0	25.8	
Stabilometer @ 1000				
Stabilometer @ 2000	17	28	13	
Turns Displacement	4.7	3.72	5.1	
R-value	82	75	85	

Remarks:

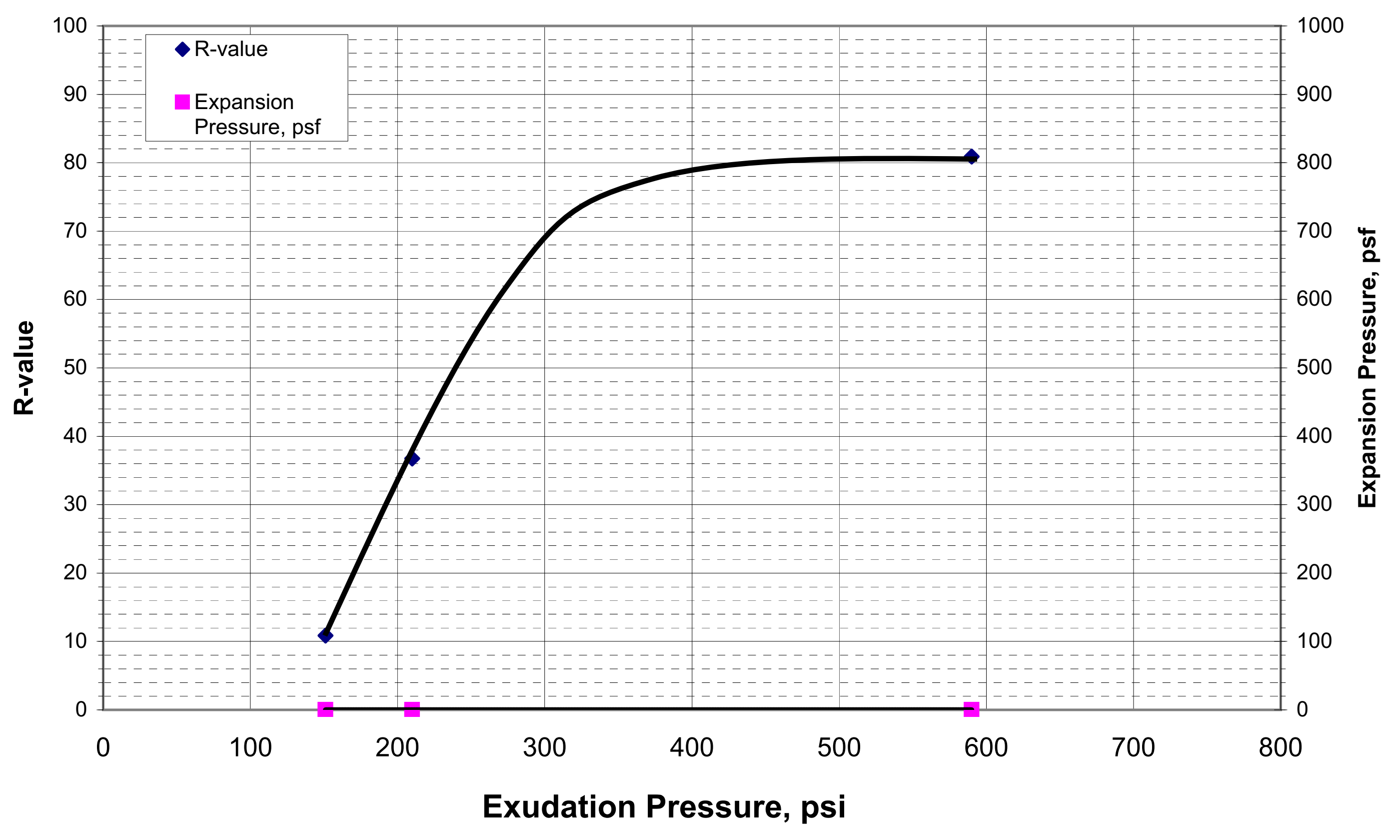


R-value Test Report (Caltrans 301)

Job No.:	687-001	**Date:**	02/25/09	**Initial Moisture,**	5.7%
Client:	Yeh and Associates, Inc.	**Tested**	MD	**R-value by Stabilometer**	**70**
Project:	Danes Drive - 28-296	**Reduced**	RU		
Sample	P-2 @ 1-5'	**Checked**	DC	**Expansion Pressure**	**0** **psf**
Soil Type:	Brown Sandy CLAY w/ Gravel				

Specimen Number	A	B	C	D
Exudation Pressure, psi	151	590	210	
Prepared Weight, grams	1300	1300	1300	
Final Water Added, grams/cc	47	19	34	
Weight of Soil & Mold, grams	3329	3308	3342	
Weight of Mold, grams	2107	2109	2086	
Height After Compaction, in.	2.37	2.29	2.42	
Moisture Content, %	9.5	7.2	8.4	
Dry Density, pcf	142.6	147.9	144.9	
Expansion Pressure, psf	0.0	0.0	0.0	
Stabilometer @ 1000				
Stabilometer @ 2000	138	24	94	
Turns Displacement	3.1	2.9	2.78	
R-value	11	81	37	

Remarks:



R-value Test Report (Caltrans 301)

Job No.:	687-001	**Date:**	02/25/09
Client:	Yeh and Associates, Inc.	**Tested**	MD
Project:	Danes Drive - 28-296	**Reduced**	RU
Sample	P-3 @ 1-5'	**Checked**	DC

Soil Type: Brown Clayey SAND w/ Gravel

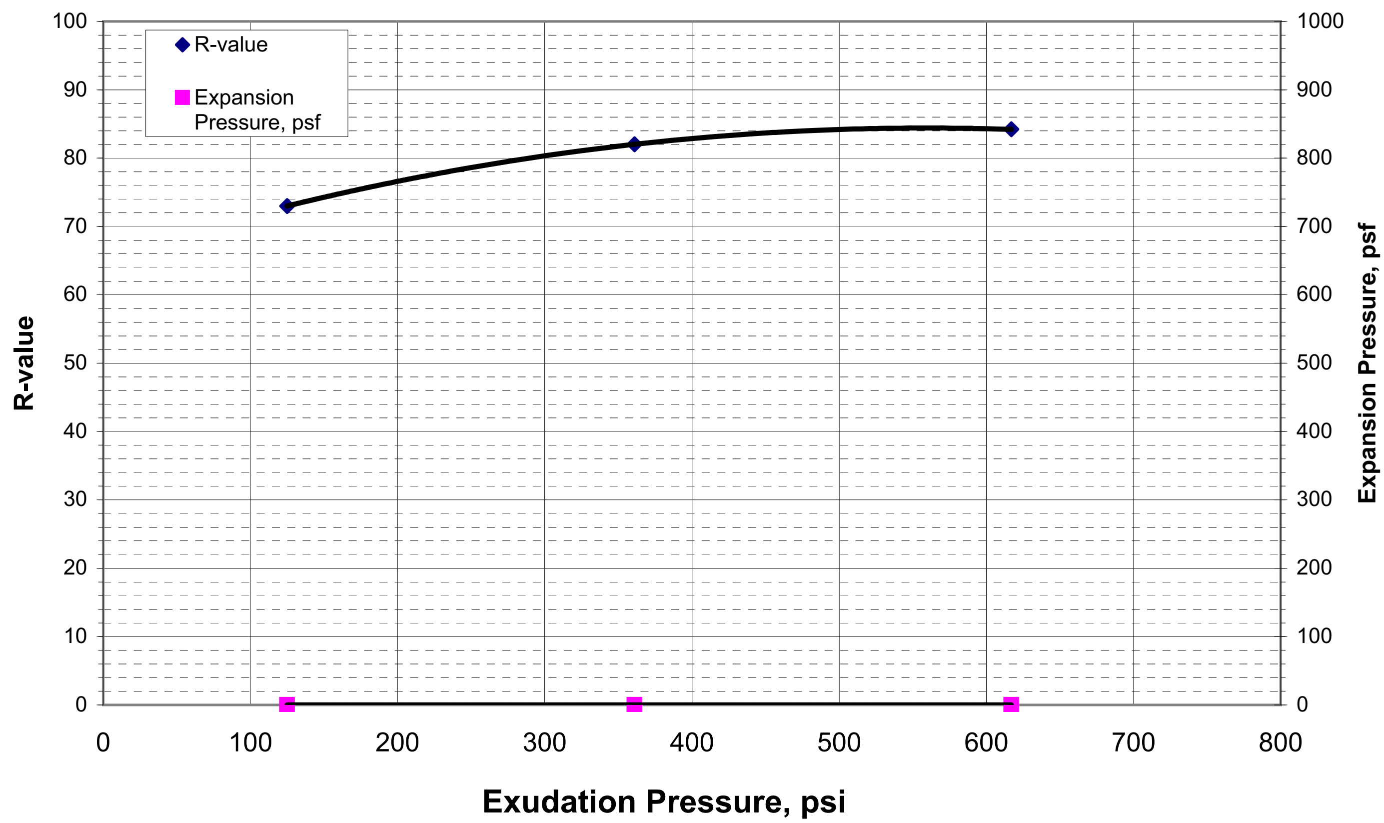
Initial Moisture, 3.4%

R-value by Stabilometer 80

Expansion Pressure 0 psf

Specimen Number	A	B	C	D
Exudation Pressure, psi	125	617	361	
Prepared Weight, grams	1300	1300	1300	
Final Water Added, grams/cc	72	54	57	
Weight of Soil & Mold, grams	3378	3308	3355	
Weight of Mold, grams	2089	2104	2091	
Height After Compaction, in.	2.47	2.24	2.48	
Moisture Content, %	9.1	7.7	7.9	
Dry Density, pcf	144.8	151.1	143.0	
Expansion Pressure, psf	0.0	0.0	0.0	
Stabilometer @ 1000				
Stabilometer @ 2000	36	14	20	
Turns Displacement	3.22	4.17	3.78	
R-value	73	84	82	

Remarks:



Appendix E – Drilling Photos

Danes Drive P-1

Danes Drive P-2

Danes Drive P-3

Danes Drive P-4

Danes Drive T-1

Danes Drive T-2

Bunker Mitchell YA-01

Bunker Mitchell YA-03

Bunker Mitchell YA-04

Appendix F – Pavement Design

TECHNICAL MEMORANDUM

TO: **Matt Wessell**

FROM: **Anna Smith**

DATE: **March 16, 2009**

SUBJECT: **ESAL Summary for Danes Drive**

Roadway improvements are being planned for the intersection of Alexander Avenue and Danes Drive, which is located in Golden Gate National Recreation Area in California. This memorandum summarizes the existing and future traffic volume assumptions that will be used to determine the pavement structure depth.

1.0 EXISTING AND FUTURE TRAFFIC VOLUMES

The most recent average daily traffic (ADT) counts available within the project area were from an intersection study performed for Danes Drive and Alexander Avenue by PBS&J in 2002. That study found the daily bi-directional volumes on Alexander Avenue south of the intersection to be approximately 9,420 vehicles per day (vpd) over an average week in September. This leg of the intersection carries the highest volumes and was therefore used for the ESAL calculation for the entire intersection. This data was factored up by 2% to estimate the 2010 volumes (construction year), which results in an approximate ADT of 11,000 vpd. The two percent growth rate is often used as an industry standard when no other traffic projections are available.

The assumptions for the ESAL calculations are based off of the Central Federal Lands "Pavement Investigation and Report Guidelines". According to the Guidelines, the project must use a 20-year design life for flexible pavements. There was no available data on future projected growth rates in the area, but it is anticipated to be relatively low growth. According to the Guidelines, when no growth projections are available, a two percent annual growth rate should be used for the analysis. Applying the two percent annual growth rate results in an ADT of approximately 16,400 vpd in 2035.

The daily truck percentages within the project limits are approximately 5%. This percentage was assumed to remain constant for the 2030 volumes. An assumption of 4% single unit trucks (including buses and motor homes) and 1% heavy trucks was made for both the existing and 2030 conditions.

2.0 ESAL CALCULATIONS

For the ESAL calculations, an assumption of 95% passenger cars, 4% single unit trucks (1% buses, 2% two axle single unit truck, 1% three axle single unit truck), and 1% heavy trucks was used for the existing and future volumes at the intersection of Danes Drive and Alexander Avenue. The truck factor classifications for ESAL calculations were divided into three categories: passenger cars and pickups, single unit trucks and buses, and combination trucks.

A straight-line growth projection was assumed over the 20-year period between 2010 and 2030 with 2020 as the midpoint year. The following assumptions were made regarding the ESAL calculations:

- intersection improvements will be constructed with flexible pavement
- one lane of traffic in each direction
- directional distribution is 60%
- annual growth will be 2% per year from 2010 to 2030
- truck percentages in 2010 and 2030 were assumed to remain at 5% (1% buses, 2% two axle single unit truck, 1% three axle single unit truck, and 1% heavy trucks)
- 100% of the traffic will travel in the design lane

The following tables show the results of the ESAL calculation for the roadway under the two percent annual growth scenario. The ESAL calculations will be used to determine the recommended pavement structure depth for the section of roadway that is being reconstructed.

Table 2.1
ESAL Calculations for 2% Annual Growth

Vehicle Type	2010 AADT	Vehicle Type	2020 AADT	2030 AADT	Truck Factor	Lane Factor	Directional Traffic	18 KiP ESAL
Passenger	10,488	95%	13,034	15,580	0.0004	1.00	0.60	22,836
Single Unit Trucks	442	4%	549	656	0.85	1.00	0.60	2,043,182
Tractor Trailer	110	1%	137	164	2.2	1.00	0.60	1,322,059
Totals	11,040	100%	13,720	16,400				**3,388,077**

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Flexible Structural Design Module

Danes Drive
Golden Gate Recreation Area, Ca.
28-296
Composite Section Widening Widening

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	3,388,077
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.49
Roadbed Soil Resilient Modulus	25,317 psi
Stage Construction	1
Calculated Design Structural Number	2.60 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	ABC R > 80	0.14	1	5	12	0.70
2	HACP	0.4	1	5	12	2.00
Total	-	-	-	10.00	-	2.70

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Flexible Structural Design Module

Danes Drive
Golden Gate Recreation Area, Ca.
28-296
Full Depth Widening

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	3,388,077
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.49
Roadbed Soil Resilient Modulus	25,317 psi
Stage Construction	1
Calculated Design Structural Number	2.60 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Full Depth HACP	0.4	1	6.5	12	2.60
Total	-	-	-	6.50	-	2.60

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Flexible Structural Design Module

Danes Drive
Golden Gate Recreation Area, Ca.
28-296
Overlay Existing with 2" Milling

Flexible Structural Design

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Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.49
Roadbed Soil Resilient Modulus	25,317 psi
Stage Construction	1
Calculated Design Structural Number	2.60 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Existing Pavement	0.25	1	6	12	1.50
2	New HACP	0.4	1	3	12	1.20
Total	-	-	-	9.00	-	2.70