GEOTECHNICAL INVESTIGATION
PROPOSED HALE AVENUE EXTENSION

MORGAN HILL, CALIFORNIA

AUGUST 31, 2020
PROJECT PA17.1024.00

SUBMITTED TO:

Mark Thomas & Company
3000 Oak Road, #650
Walnut Creek, CA 94597

PREPARED BY:

Geo-Logic Associates
dba Pacific Geotechnical Engineering
16055 Caputo Drive, Suite D
Morgan Hill, California 95037
(408) 778-2818

GEOTECHNICAL INVESTIGATION
PROPOSED HALE AVENUE EXTENSION
MORGAN HILL, CALIFORNIA

TABLE OF CONTENTS

FIGURE 1 – SITE PLAN

APPENDIX A - KEYS TO SOIL CLASSIFICATION AND DRILL HOLE LOGS

Keys to Soil Classification (Fine and Coarse Grained Soils)
Rock Quality Descriptions
Log of Exploratory Drill Holes (DH-1 through DH-12)

APPENDIX B – LABORATORY TEST RESTULS

Figure B-1	Atterberg Limits Test Results
Figures B-2 and B-3	Grain Size Test Results
Figures B-4, B-5 & B-6	Unconfined Compression Test Results
Figures B-7 thru B-11	R-value Test Reports

Corrosion Test Results (CERCO Analytical)

1. INTRODUCTION

This report presents the results of our geotechnical study for the proposed Hale Avenue Extension project in Morgan Hill, California. The approximate location of the project is shown on the Vicinity Map included with the Site Plan, Figure 1, of this report. The Site Plan shows the proposed alignment of Hale Avenue extension.

This report is based on the 100% design plans provided to us (see Section 1.4 below). If changes are made to the project design, we should be notified to review the changes and evaluate if revisions to our conclusions and recommendations are necessary.

This report presents our findings, conclusions, and geotechnical recommendations for design and construction of the proposed improvements. These findings, conclusions, and recommendations are based on information collected and reviewed during this study. The conclusions and recommendations in this report should not be extrapolated to other areas or used for other projects without our review.

1.1 Project Description

<u>Hale Avenue Extension</u>: The proposed Hale Avenue will extend from its present terminus at West Main Avenue in the north to the junction of Spring Avenue and DeWitt Avenue in the south, a distance of approximately 4,800 feet. The new road, with a 70- to 130-foot wide right-of-way, will have one traffic lane in each direction and a 4- to 16½-foot wide median. A 6-foot wide bike lane is planned on both sides of the roadway. City standard sidewalks, shoulders, landscape areas, bioswales, retaining walls, and sound walls are also proposed along sections of the roadways. Other proposed improvements include new storm drainage facilities, water mains, and light standards.

<u>Retaining Structures:</u> Three retaining walls (RW#1, RW#2, and RW#3) consisting of Caltrans Type 5 walls are proposed for the project. The ground surface behind RW#1 and RW#2 will be sloping. The ground surface in front of RW#3 will also be sloping.

Wall RW#1 will be constructed northwest of proposed Hale Avenue and West Dunne Avenue, between Station “H” 33+20.45 Lt 153.14 and Station “H” 34+06.05 Lt 74.87. The wall will be about 156 feet in length and range between approximately 6 and 8 feet in design height. The wall will follow the existing ground contours and the slope behind the wall has a gradient of about 3¾:1 (horizontal:vertical).

Wall RW#2 will be constructed northeast of proposed Hale Avenue and West Dunne Avenue, between Station “H” 33+67.50 Rt 72.42 and Station “H” 33+90.63 Rt 38.50. The wall will be about 54 feet in length and range between approximately 4 and 6 feet in design height. The wall will follow the existing ground contours and the slope behind the wall has a gradient of about 5½:1 (horizontal:vertical).

Wall RW#3 will be constructed northwest of Retaining Wall #1, between Station “H” 35+29.26 Lt 157.88 and Station “H” 35+84.23 Lt 151.81, on the south, east and north sides of an existing electrical tower. The wall will be about 90 feet in length and range between 8 and 12 feet in design height. The ground in front of the wall slopes down at a gradient of 4:1 to 6:1 (horizontal:vertical).

<u>Sound Walls</u>: Three sound walls (SW#1, SW#2, and SW#3) consisting of Caltrans standard masonry block sound walls on cast-in-drilled-hole piles are proposed along the new road alignment.

Wall SW#1 will be constructed on the east side of proposed Hale Avenue, between Station “H” 7+96.02 Rt 71.75 and Station “H” 29+78.07 Rt 65.74. The wall will be about 2,176 feet in length and the wall height will be about 8 feet.

Wall SW#2 will be constructed on the west side of proposed Hale Avenue, between Station “H” 10+49.07 Lt 38.89 and Station “H” 14+63.47 Lt 44.56. The wall will be about 432 feet in length and the wall height will be about 8 feet.

Wall SW#3 will be constructed on the west side of proposed Hale Avenue, south of West Main Avenue, between Station “H” 44+31.72 Rt 35.5 and Station “H” 52+11.28 Rt 62. The wall will be about 816 feet in length and the wall height will vary from 8 feet to 12 feet 8 inches.

<u>Detention Basins</u>: Two new detention basins are proposed on the west side of proposed Hale Avenue. Detention Basin #1 will be constructed southwest of proposed Hale Avenue and existing West Dunne Avenue, between roughly Stations “H” 29+30 and “H” 32+50. Basin #1 will be up to 7 feet in depth below ground surface (bgs) and its side slopes will have a gradient of 4:1 (horizontal:vertical).

Detention Basin #2 will be constructed between roughly Stations “H” 47+30 and “H” 48+60. Basin #2 will be about 4 to 5 feet in depth and its side slopes will have a gradient of 4:1 (horizontal:vertical).

<u>Storm Drain</u>: A new 18-inch diameter RCP storm drain pipeline with associated manholes will be constructed along proposed Hale Avenue. The invert of the pipeline will be up to about 7 to 8 feet below finished grade.

<u>Water Main</u>: A new 12-inch diameter DIP water main pipeline will be constructed along proposed Hale Avenue. The invert of the pipeline will be up to about 4 to 5 feet below finished grade.

<u>Cuts and Fills</u>: Proposed cuts and fills for construction of the new roadway generally range between 1 and 4 feet in depth. Between approximately Station 33+50 (north of West Dunne Avenue) and Station 43+50, significant cuts up to 35 feet in depth are proposed for construction of the new Hale Avenue.

Fills of roughly 8 to 10 feet deep are proposed to backfill the existing pond between roughly Stations “H” 20+25 and “H” 21+50.

1.2 Purpose and Scope of Study

The purpose of the geotechnical study was to evaluate subsurface soil conditions along the proposed Hale Avenue extension alignment and to provide geotechnical recommendations for design and construction of the proposed improvements. The following work was performed for our investigation.

1. Reviewed available geologic and geotechnical information pertinent to the site.

2. Performed a site reconnaissance to observe surface conditions and mark locations of our subsurface exploration.

3. Obtained an encroachment permit from the City of Morgan Hill (City) for our subsurface exploration work. Rights-of-entry for private properties were provided by the City.

4. Obtained a drilling permit from Pacific Gas and Electric (PG&E) for subsurface exploration on PG&E property.

5. Notified Underground Service Alert for utility clearance.

6. Coordinated our drilling schedule with the City and PG&E.

7. Performed a subsurface exploration program which included twelve conventional drill holes.

8. Collected seven bulk samples of near-surface soil along the proposed roadway alignment.

9. Performed laboratory tests on selected soil samples to measure their physical and engineering properties.

10. Performed engineering analysis on information collected from our literature research, field investigation and laboratory testing.

11. Prepared a 65% submittal draft report summarizing our findings, conclusions and recommendations. Specifically, the report included the following:

    - Project description

- Scope of work
- Field investigation and laboratory testing
- Geotechnical conditions, including site geology, surface conditions, subsurface conditions, and depth to groundwater
- Seismic data, including faulting, seismicity, site acceleration, and liquefaction potential
- Geologic hazards including tsunami, seiche, earthquake induced flooding, landslides, ground rupture, seismic shaking and scour
- Geotechnical analysis and discussion
- Soil corrosion potential
- Earthwork recommendations, including site preparation, excavations, subgrade preparation, material for engineered fill, final cut and fill slopes, fill placement and compaction, and utility trench backfill
- Foundation recommendations for the proposed retaining walls and sound walls
- Parameters for retaining wall design, including lateral earth pressures, passive resistance and soil bearing capacity
- Structural pavement sections
- A Vicinity Map showing the project area and a Site Plan showing the approximate locations of our drill holes and bulk samples
- Logs of our drill holes
- Laboratory test results

12. Reviewed the 100% submittal plans from Mark Thomas & Company.

13. Prepared this final geotechnical report for the project.

1.3 Information Provided

For our geotechnical study, Mark Thomas & Company provided the following to us.

- Sheets L-1 through L-5, PE-2 through PE-8, RW-1 through RW-5, and SW-1 through SW-5, prepared by Mark Thomas & Company, 60% Submittal, plan set dated July 9, 2012 except for sheets RW-4 and RW-5 which are dated July 20, 2012.

- City of Morgan Hill Improvement Plans for Hale Avenue Extension Project, Sheets 1, 2 and 3, prepared by Mark Thomas & Company, plan set dated September 2011.

- Sheets T-1, TS-1 through TS-3, DM-1 through DM-5, HC-1 through HC-5, L-1 through L-6, PE-1 through PE-8, D-1 through D-5, DB-1 and DB-2, U-1 through U-5, SW-1 through SW-12, and RW-1 through RW-9, 95% submittal, dated April 19, 2017.

- Sheets T-1, TS-1 through TS-3A, DM-1 through DM-5, HC-1 through HC-6, L-1 through L-6, X-1 through X-5, PE-1 through PE-8, DA-1 through DS-5, D-1 through D-5, DB-1 and DB-2, U-1 through U-5, EC-1 through EC-6, WM-1, SW-1 through SW-14, RW-1 through RW-9, S-1 through S-5, PDQ-1, SQ-1, CS-1, SC-1 through SC-3, TH-1, and THQ-1, dated August 14, 2020.

2. FIELD INVESTIGATION AND LABORATORY TESTING

Our field investigation consisted of a surface reconnaissance and a subsurface exploration program. The site reconnaissance was to observe existing surface conditions along the proposed roadway alignment. The subsurface exploration program was to explore soil conditions at selected locations along the proposed roadway alignment.

2.1 Subsurface Exploration

Our subsurface exploration program consisted of twelve exploratory drill holes (DH-1 through DH-12). The drill holes were located in the field by referencing to existing site features and pacing; therefore, their locations are approximate. The approximate locations of the drill holes are shown on Figure 1.

The drill holes were backfilled with cement grout, as required by the Santa Clara Valley Water District, and topped off with cold patch asphalt in paved areas.

Drill hole DH-1 was advanced on January 23, 2012 with a Mobile B53 truck-mounted drilling rig. Holes DH-2 through DH-6 were advanced on April 26, 2012, with a CME 55 tract-mounted drilling rig. Holes DH-7 through DH-12 were advanced on April 27, 2012 with a Mobile B53 truck-mounted drilling rig. Maximum depth of exploration ranged between 15 and 25 feet below ground surface (bgs). In the field, our personnel visually classified the materials encountered and maintained a log of each drill hole.

Soil samples were obtained from the drill holes using a 2-inch outside diameter (1.4-inch inside diameter) split-barrel sampler (also called a Standard Penetration Test sampler) and a 3-inch outside diameter (2½-inch inside diameter) split barrel sampler. Drive samples were obtained by driving a soil sampler up to 18 inches into the earth material using a 140-pound hammer falling 30 inches. The hammer on the Mobile B53 rig was operated using a wire winch and pulley system. The hammer on the CME 55 rig is an automatic trip hammer. The number of blows required to drive the samplers was recorded for each 6-inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the penetration interval indicated on the log where harder material was encountered, is shown as blows per foot (blow count) on the drill hole logs.

Visual classification of soils encountered in our drill holes was made in general accordance with the Unified Soil Classification System (ASTM D 2487 and D 2488). The results of our laboratory tests were used to refine our field classifications. Two Keys to Soil Classification, one for fine grained soils and one for coarse grained soils, and a Rock Quality Descriptions are included in Appendix A together with the logs of the drill holes.

2.2 Bulk Sampling

A total of seven bulk samples of the near-surface soils (Bulks B-1 through B-7) were collected along the proposed roadway alignment. The approximate locations of the samples are shown on Figure 1 of this report.

2.3 Laboratory Testing

Laboratory tests were performed on selected soil samples. The geotechnical tests included water content, dry density, Atterberg limits, grain size distribution, percent passing No. 200 sieve, and unconfined compressive strength. Three selected soil samples were also tested for corrosivity. Five selected bulk samples were tested for R-values.

Most of the laboratory test results are presented on the drill hole logs at the corresponding sample depths. The results of the Atterberg Limits, unconfined compressive strength, grain size distribution, and R-value tests are presented on Figures B-1 through B-11 in Appendix B. The results of the corrosion potential tests and a brief report from CERCO Analytical are also included in Appendix B.

3. FINDINGS

3.1 Site Geology

The Hale Avenue Extension project site is located in foothill terrain along the northeastern range front of the Santa Cruz Mountains close to the southwestern edge of the Santa Clara Valley. The Santa Cruz Mountains are part of a large tectonic block currently undergoing slow uplift. The core of this block is composed of metamorphic rocks of Mesozoic age (older than about 65 million years). The metamorphic rocks (known as the Franciscan Complex) in the Santa Cruz Mountains were folded and faulted together millions of years ago. Large blocks and belts of metamorphosed greenstone, sandstone and other rock types are typically mapped as separated by fault contacts, with serpentinite occurring typically as a sheared interval between the blocks of metamorphic rock (McLaughlin and others, 2001). These rocks have been uplifted relative to the geologically young alluvial fans forming the Santa Clara Valley. The San Andreas fault is a major strike-slip fault zone located southwest of the site within the Santa Cruz Mountains. A series of southwest-dipping thrust and reverse faults associated with oblique compression along the San Andreas fault zone are responsible for uplift and deformation of the sedimentary and metamorphic rock units along the northeastern range front of the Santa Cruz Mountains relative to the southwestern margin of the Santa Clara Valley.

McLaughlin and others (2001) compiled and updated geologic mapping of bedrock and valley-floor areas in the site vicinity. The City of Morgan Hill completed geologic mapping of bedrock and valley floor areas within the City limits and adjacent land in 1994. Mapping by both authors shows hillside portions of the project alignment as part of an outlier of Franciscan greenstone bedrock (fpv and KJfg, respectively) associated with a larger package of greenstone along the rangefront of the Santa Cruz Mountains. Older alluvium underlies the remainder of the project alignment.

A Ground Movement Potential Map (GMPM) was prepared by our predecessor firm Pacific Geotechnical Engineering (1994) for the City of Morgan Hill. The GMPM suggests most of the proposed Hale Avenue extension alignment is within “relatively stable ground” composed of either unconsolidated colluvium or alluvium on flat ground, or stable bedrock. A relatively short section of the alignment is in a “Ps” zone which is described as having a potential for shallow ground movements.

3.2 Surface Conditions

Between the south end of the project and West Dunne Avenue, the proposed road alignment will be constructed on undeveloped, essentially flat-lying ground covered with weed and isolated trees. Existing houses and buildings are present along the east and southwest portions of this alignment segment. In the vicinity of Stations “H” 20+00 and “H” 21+30, the new road will cross an existing pond which is about 10 feet deep. The pond is surrounded by an existing fence.

North of West Dunne Avenue and south of roughly Station “H” 44+000, the proposed road alignment will cross hilly terrain on the west side of Nob Hill. The areas consist of undeveloped land covered with surface vegetation and isolated trees. There is a Pacific, Gas and Electric (PG&E) power line supported on towers west of the proposed road alignment.

The north end of the project is on gently sloping to flat-lying ground. This portion of the alignment will cross a couple of existing buildings, the west end of Warren Avenue, and a PG&E property.

3.3 Subsurface Conditions

A brief description of the earth materials encountered in our drill holes along the proposed Hale Avenue extension alignment, from south to north, is presented below. The reference station numbers are only approximate, not from an actual survey. We did not establish right or left offset from the “H” line. For more details of the materials encountered, refer to the drill hole logs in Appendix A.

<u>DH-12</u>: Drill hole DH-12 was advanced near the intersection of proposed Hale Avenue, existing Dewitt Avenue and existing Spring Avenue, in the vicinity of Station “H” 7+60. The subsurface conditions generally consist of colluvium over alluvium over bedrock. The colluvial and alluvial soils consist of very stiff sandy clays of intermediate plasticity to a depth of about 5 feet below ground surface (bgs). The underlying greenstone bedrock, extending to the maximum explored depth of about 14½ feet bgs, is soft in rock mass hardness, severely weathered and mostly crushed.

<u>DH-11</u>: Drill hole DH-11 was advanced in the vicinity of Station “H” 12+50. The surficial soil was described as fill or disturbed colluvium consisting of very stiff sandy clay of intermediate plasticity to a depth of about 3 feet bgs. This clay layer is underlain by very stiff, colluvial sandy clay of intermediate plasticity to a depth of about 5 feet bgs, and alluvial sandy clay to clayey sand to the maximum explored depth of about 15 feet bgs. The sandy clay is hard in consistency and the clayey sand is dense to very dense in relative density.

<u>DH-10</u>: Drill hole DH-10 was advanced in the vicinity of Station “H” 18+00. The surficial soil consists of very stiff, colluvial sandy clay of intermediate plasticity to a depth of about 6 feet bgs. This soil is underlain by alluvial soils consisting of very stiff clay with sand to a depth of about 8½ feet bgs and hard sandy clay to very dense clayey sand to the maximum explored depth of about 15 feet bgs.

<u>DH-9</u>: Drill hole DH-9 was advanced in the vicinity of Station “H” 21+90. The subsurface conditions generally consist of colluvium over alluvium over bedrock. The colluvial soil consists of very stiff fat clay and sandy fat clay of high plasticity to a depth of about 5 feet bgs. The underlying alluvial soil consists generally of medium dense to very dense clayey sand to a depth of about 10 feet bgs. The underlying greenstone bedrock, extending to the maximum explored

depth of about 15 feet bgs, is soft in rock mass hardness, severely to very severely weathered and mostly crushed.

DH-8: Drill hole DH-8 was advanced in the vicinity of Station "H" 26+00. The surficial soil generally consists of very stiff, colluvial fat clay with sand of high plasticity to a depth of about 3 feet bgs. This soil is underlain by alluvial soils consisting of hard sandy fat clay of high plasticity to very dense clayey sand to a depth of about 9 feet bgs, hard clay with sand to a depth of about 12 feet bgs, and very dense clayey sand to the maximum explored depth of about 14½ feet bgs.

DH-7: Drill hole DH-7 was advanced in the vicinity of Station "H" 30+80. The surficial soil is a roughly 1-foot thick layer of fill consisting of fat clay with sand. The fill is underlain by very stiff to hard, colluvial fat clay with sand of high plasticity to a depth of about 3½ feet bgs. The colluvium is underlain by alluvial soils consisting of very dense clayey sand and hard clay with sand of intermediate plasticity to the maximum explored depth of about 15 feet bgs.

DH-2: Drill hole DH-2 was advanced in the vicinity of Station "H" 35+00. The subsurface conditions can be described as colluvial soil underlain by greenstone bedrock. The colluvial soil is stiff sandy fat clay of high plasticity to a depth of about 7½ feet bgs. The greenstone is soft in rock mass hardness, moderately weathered and mostly crushed, and extends to the maximum explored depth of about 23.7 feet bgs.

DH-3: Drill hole DH-3 was advanced in the vicinity of Station "H" 37+10. The subsurface conditions can be described as colluvial soil underlain by greenstone bedrock. The colluvial soil is stiff sandy fat clay of high plasticity to a depth of about 1½ feet bgs. The greenstone is soft in rock mass hardness, moderately severely weathered, and crushed, and extends to the maximum explored depth of about 25 feet bgs. Hard drilling was reported below a depth of roughly 13 feet.

DH-4: Drill hole DH-4 was advanced in the vicinity of Station "H" 40+00. The subsurface conditions can be described as colluvial soil underlain by greenstone bedrock. The colluvial soils consist of stiff to hard sandy fat clay of high plasticity to a depth of about 4 feet bgs, and medium dense clayey sand to a depth of about 8 feet bgs. The greenstone is soft in rock mass hardness, moderately severely weathered and crushed, and extends to the maximum explored depth of about 20 feet bgs. Very hard drilling was reported below a depth of roughly 13 feet.

DH-1: Drill hole DH-1was advanced in the vicinity of Station "H" 45+30. The subsurface conditions can be described as colluvium over alluvium over greenstone bedrock. The colluvial soil consists of very stiff fat clay with sand of high plasticity to a depth of about 5½ feet bgs. The alluvial soil consists of dense to very dense clayey sand to a depth of about 10 feet bgs. The greenstone is soft in rock mass hardness, very severely weathered, and intensely fractured to crushed, and extends to the maximum explored depth of about 14 feet bgs.

DH-5: Drill hole DH-5 was advanced in the vicinity of Station "H" 47+30. The subsurface conditions can be described as colluvium over greenstone bedrock. The colluvial soil consists of stiff to hard sandy fat clay of high plasticity to a depth of about 7½ feet bgs. The greenstone is soft in rock mass hardness, moderately to severely weathered, and crushed, and extends to the maximum explored depth of about 14 feet bgs.

DH-6: Drill hole DH-6 was advanced in the vicinity of Station "H" 52+00, near the intersection of proposed Hale Avenue and existing West Main Street. In this hole, a pavement section consisting of 3 to 4 inches of asphalt concrete with no apparent base rock was encountered. The pavement section is underlain by fill consisting of medium dense clayey sand to a depth of about 2 feet bgs. The fill is underlain by alluvial soils consisting of very stiff sandy fat clay of high plasticity to a depth of about 12 feet bgs, and medium dense clayey sand to the maximum explored depth of roughly 15 feet bgs.

3.4 GROUNDWATER

Groundwater was not encountered in any of our drill holes at the time of our field investigation.

Our review of groundwater information in the vicinity of the project suggests groundwater was reported at elevation 356.7 feet at the west end of Nob Hill Terrace in December 2011, at about elevation 352 feet near the intersection of Hale Avenue and Warren Avenue in May 2012. Two of the borings from our geotechnical investigation in October 2008 for the West Dunne Avenue Improvement project were located near the intersection of West Dunne Avenue and proposed Hale Avenue. These borings, extended to depths of 10 and 20 feet below the pavement surface, did not encountered groundwater at the time of drilling.

Fluctuations in the groundwater level may occur due to seasonal variations in rainfall and temperature, pumping from wells, groundwater recharge programs, irrigation or other factors that were not evident at the time of our investigation. Where greenstone bedrock is present at relatively shallow depth, perched groundwater condition should be anticipated, especially during and after rainy months.

3.5 Variations in Subsurface Conditions

Our interpretations of soil and groundwater conditions, as described in this report, are based on data obtained from our subsurface exploration and laboratory testing for this study. Our conclusions and geotechnical recommendations are based on these interpretations. Please realize portions of the project area have undergone previous development and grading. Therefore, it is likely that undisclosed variations in subsurface conditions exist at the site, such as old foundations, abandoned utilities and localized areas of deep and loose fill.

Careful observations should be made during construction to verify our interpretations. Should variations from our interpretations be found, we should be notified to evaluate whether any revisions should be made to our recommendations.

4. SEISMIC DATA AND EVALUATION

4.1 Seismic Data

The Greater San Francisco Bay Area is seismically dominated by the active San Andreas Fault system, the tectonic boundary between the northward moving Pacific Plate (west of the fault) and the North American Plate (east of the fault). This movement is distributed across a complex system of generally strike-slip, right-lateral, subparallel faults.

Potential sources of significant earthquake ground shaking at the site include several active and potentially active faults in the Greater San Francisco Bay area, as well as faults farther afield. The faults were first compiled on the State's Fault Activity Map (Jennings, 1974; Jennings and Bryant, 2010). This map has now been integrated into the US Geological Survey's Quaternary Fault and Fold Database (http://earthquake.usgs.gov/regional/qfaults/), and made available as a .kmz "drape" over Google Earth terrain files.

The distance to a seismic source (fault) is defined by the Next Generation Attenuation (NGA) relationships as the closest distance to the seismogenic zone, be it in the subsurface or at the surface; distances may therefore differ from distances measured on the ground surface. The distances shown on the table below are for reference only, as they are horizontal distances from the site to the surface trace of the seismic source, and not necessarily the closest distance to a (dipping) seismogenic zone. These distances were measured using the US Geological Survey's Quaternary Fault and Fold Database, with major faults listed in approximate order of distance from the site; not all sources are listed in the summary table below.

Table 4.1-1
Fault Distances and Orientation

Fault	Approximate Distance		Orientation from Project Site to Fault
	Project North	Project South	
Calaveras	6½ km	7½ km	Northeast
Sargent	11 km	10 km	Southwest
San Andreas	15 km	14 km	Southwest
Zayante-Vergeles	21 km	19 km	Southwest
Monte Vista-Shannon	26 km	26 km	Northwest
Hayward (southeast extension)	44 km	45 km	North-Northwest

No known faults have been mapped crossing the proposed roadway alignment. The project area is not located in an Alquist-Priolo Earthquake Fault Zone as defined by the California Geological Survey.

4.2 Caltrans Acceleration Response Spectrum

We have developed a Caltrans Acceleration Response Spectrum (ARS) using the Caltrans ARS Online Tool version 3.0.2 which complies with Caltrans Seismic Design Criteria (SDC) Version 2.0. The recommended Caltrans ARS curve is presented in Figure 2.

4.3 Seismicity

The Working Group on California Earthquake Probabilities (WGCEP) estimates of the probabilities of major earthquakes are now in their sixth iteration, with the greatest changes in approach being the inclusion of multifault rupture scenarios, in the progressive consideration of more potential seismic sources, the possibility of earthquakes on unrecognized faults, and the inclusion of the notion of fault “readiness”. Current estimates (WGCEP, 2014) for the San Francisco region indicate a 72% probability of a large (magnitude 6.7 or greater) earthquake in the San Francisco Bay area as a whole over the 30-year period beginning in 2014; this overall probability is greater than the previous (WGCEP, 2007) probability of 63%, due mainly to the inclusion of multifault rupture scenarios. The estimate for the Calaveras fault alone is 14.4% (revised up from the 7% presented by WGCEP, 2007); for the (northern) San Andreas fault alone, 27.4% (revised upward from the WGCEP (2007) value of 21%); and for the Hayward fault, 45.3% (revised upward from the WGCEP (2007) value of 31%).

4.4 Liquefaction and Seismic Settlement

Soil liquefaction is a phenomenon in which saturated granular soils, and certain fine-grained soils, lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and non-plastic silts. Certain gravels, plastic silts, and clays are also susceptible to liquefaction. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type; 3) relative density of granular soils; 4) moisture content and plasticity of fine-grained soils; 5) overburden pressure; and 6) depth to ground water.

The project area is not located within a Santa Clara County liquefaction hazard zone.

The potential for liquefaction of the granular soils encountered in our drill holes is judged to be low because the soils have dense to very dense relative density and no groundwater was encountered in our drill holes. The clay soils and greenstone bedrock are not considered susceptible to liquefaction.

Because the potential for liquefaction is low along the proposed road alignment, the potential for liquefaction-induced settlement is also low.

4.5 Additional Geologic Hazards Considerations

Tsunami: The project area is located in an interior valley, and is not subject to tsunami hazards (CGS Note 55).

Seiche: The project area is not located near the margin of a body of water subject to seiche (flood hazard associated with seismically-induced oscillation).

Flooding: A review of on-line FEMA FIRM flood hazard maps (https://msc.fema.gov) suggests the majority of the proposed alignment is in Zone X which includes “areas of 0.2% annual chance flood; areas of 1% chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood. The northern portion of the proposed alignment in the vicinity of West Main Avenue is in Zone AE which is “special flood hazard areas subject to inundation by the 1% annual chance flood with base flood elevations determined.”

The northern portion of the project in the vicinity of West Main Avenue is located within a potential inundation area in the event of a sudden failure of Anderson Dam (Anderson Reservoir), located east of Morgan Hill, based on Sheet 9 of the Anderson Dam Flood Inundation Maps, Santa Clara Valley Water District.

Landslides: The risk of landsliding within the flat-lying portions of the project site is low. In our opinion, the potential for deep-seated landsliding in the hilly portions of the project is low because of the presence of greenstone bedrock at shallow depth.

Ground Rupture: The risk of seismically induced ground rupture is low because the site is not crossed by any mapped active or potentially active faults.

Seismic Shaking: The project site is in an area of high seismicity. Based on general knowledge of the site seismicity, it should be anticipated that, during its useful life, the proposed project will be subject to at least one severe earthquake (magnitude 7 to 8+) that could cause considerable ground shaking at the site. It is also anticipated that the subject site will periodically experience small to moderate magnitude earthquakes.

Scour: Scour is not a consideration at the proposed roadway extension because the alignment does not cross a body of water. However, scour may occur at the detention ponds.

5. GEOTECHNICAL ANALYSIS AND DISCUSSION

In our opinion, the project area is geotechnical feasible for the proposed roadway as discussed in this report provided our recommendations are incorporated into the geotechnical aspects of the project design and construction. Our opinions, conclusions and recommendations are based on our understanding of the proposed development, literature and data review, properties of soils encountered in our subsurface exploration, laboratory test results, and engineering analyses. Geotechnical considerations for this project are discussed below.

5.1 Variable Subsurface Materials

The proposed Hale Avenue extension alignment is underlain by variable earth materials, including colluvial soils, alluvial soils and greenstone bedrock. The road subgrade along the majority of the alignment is expected to consist of fat clay of high plasticity. In the deeper cut areas, roughly between West Dunne Avenue and Nob Hill Terrace, the road subgrade is expected to consist of greenstone bedrock. In general, the pavement-support capability of the fat clay is low.

Construction of the proposed sound walls, retaining walls, underground utilities is anticipated to be within colluvial and alluvial soils along much of the alignment. In the deeper cut areas, construction of underground utilities will be in greenstone bedrock. Greenstone bedrock should also be anticipated at Retaining Wall RW#3 and portions of Retaining Walls RW#1 and RW#2. Refer to the section below for a discussion on the general characteristics of the greenstone.

5.2 Greenstone Bedrock

Greenstone bedrock was encountered in drill holes DH-1, DH-2, DH-3, DH-4, DH-5, DH-9 and DH-12, ranging between depths of roughly 1½ and 10 feet below ground surface. In the vicinity of DH-5 and DH-12, construction of the proposed pipelines may extend into the greenstone. Between approximately Stations “H” 34+00 and “H” 42+00, where deeper cuts are proposed, construction of the roadway and pipelines is anticipated to extend into greenstone.

The greenstone was described as soft in rock mass hardness (but the rock fragments are hard), moderately to very severely weathered, and intensely fractured to crushed. The weathered greenstone generally consists of brown clayey sand with variable amounts of sandy lean clay. It typically appears as dense angular coarse-grained sand with a clayey matrix and clay seams. Rock structure is variably preserved, with relict joints and fractures locally visible in drive samples. The weathered rock is closely fractured and crumbles easily in hand sample. Less weathered corestones of hard greenstone with better preserved fractures are locally present in the softer decomposed bedrock.

We expect the highly weathered greenstone will be rippable using heavy duty grading equipment such as Caterpillar D-8 size or larger dozers, depending on the amount of wear and tear the contractor is willing to accept. This is based on an assumed shear wave velocity of 1,500 m/sec for Franciscan greenstone, and rippability charts available online. It is likely that oversize rock fragments will need to be processed with specialized equipment.

5.3 Existing Fills

Existing fills, about 1 to 3 feet thick, were encountered in drill holes DH-6, DH-7 and DH-11. Prior to construction, existing fills should be removed to expose native soil. The removed fills may be reused as engineered fill if approved by the geotechnical engineer.

5.4 Expansion Potential of Site Soil

The results of our laboratory testing indicate the surface and near-surface soils encountered in our drill holes have intermediate to high plasticity, which corresponds to high to very high expansion potential.

5.5 Corrosion Evaluation

Three selected soil samples from our drill holes were tested for corrosion potential. The laboratory test results are included in Appendix B and summarized in Table 5.6-1 below.

Table 5.5-1
Corrosion Test Results

Sample Boring Number and Depth	pH	Sulfate (ppm)	Minimum Resistivity (ohm-cm)	Chloride (ppm)
DH-2, 5½ feet	7.6	N.D.	5,000	N.D.
DH-6, 1½+5½ feet	7.8	57	3,600	N.D.
DH-11, 3½+5½ feet	7.2	N.D.	5,100	N.D.
N.D. = None detected based on a detection limit of 15 ppm				

According to Caltrans Corrosion Guidelines Version (2015), a site is considered to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- chloride concentration is 500 ppm or greater
- sulfate concentration is 2,000 ppm or greater, or
- the pH is 5.5 or less.

Based on the above guidelines and laboratory test results, the samples tested do not meet Caltrans guidelines to be considered as “corrosive.”

6. EARTHWORK RECOMMENDATIONS

Earthwork construction should conform to the City of Morgan Hill Standards. General recommendations are presented below.

6.1 Demolition, Clearing and Grubbing

Prior to construction, areas to receive improvements should be cleared of designated existing structures and improvements, deleterious materials, debris, obstructions, and stumps and primary roots of trees and brush (roots over 1 inch in diameter or longer than about 3 feet in length). Holes, depressions and voids that extend below the proposed finish grade should be cleaned and backfilled with engineered fill compacted to the recommendations in this report.

After clearing, surface vegetation and organic laden soils should be stripped. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. The required stripping depth should be determined in the field by the geotechnical engineer at the time of construction. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect, or otherwise removed from the site.

6.2 Excavations

Excavations for this project will be required to achieve design grades, remove unsuitable materials, and to construct underground utilities, foundations, retaining walls and detention ponds. Excavations will extend into variable soils and greenstone bedrock.

Greenstone bedrock was encountered in drill holes DH-1, DH-2, DH-3, DH-4, DH-5, DH-9 and DH-12, ranging between depths of roughly 1½ and 10 feet below ground surface. In the vicinity of DH-5 and DH-12, construction of the proposed pipelines may extend into the greenstone. Between approximately Stations “H” 34+00 and “H” 42+00, where deeper cuts are proposed, construction of the roadway and pipelines is anticipated to extend into greenstone. The greenstone encountered in our drill holes was described as fractured and moderately to very severely weathered. More competent greenstone, however, should be anticipated.

We expect the highly weathered greenstone will be rippable using heavy duty grading equipment such as Caterpillar D-8 size or larger dozers, depending on the amount of wear and tear the contractor is willing to accept. This is based on an assumed shear wave velocity of 1,500 m/sec for Franciscan greenstone, and rippability charts available online. It is likely that oversize rock fragments will need to be processed with specialized equipment.

Construction, shoring and bracing of excavations should comply with the current CAL-OSHA safety standards and City of Morgan Hill requirements. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor.

Trench excavations adjacent to existing or proposed foundations should be above an imaginary plane having an inclination of 1½:1 (horizontal to vertical) extending down from the bottom edge of the foundations.

6.3 Removal of Existing Fills

Existing fills, about 1 to 3 feet thick, were encountered in drill holes DH-6, DH-7 and DH-11. Prior to construction, existing fills should be removed to expose native soil. The removed fills may be reused as engineered fill if approved by the geotechnical engineer.

6.4 Backfilling of existing pond

The existing pond between roughly Stations “H” 20+00 and “H” 21+20 should be backfilled prior to further construction activities. Any wet and/or soft soil at the bottom of the pond should be removed to firm soil. The exposed soil surface should then be scarified, moisture conditioned and compacted as recommended under “Subgrade Preparation” below. After proper preparation of the subgrade, the area may be raised to design grade with placement of engineered fill. The new fill should be benched into the side slopes as recommended under the “Fill Placement and Compaction” section below.

6.5 Subgrade Preparation

Subgrade soil in areas to receive engineered fills and pavements should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted to the recommendations given under the “Engineered Fill Placement and Compaction” below. Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or equipment of similar weight.

Subgrade preparation should extend a minimum of 5 feet beyond the limits of proposed improvements unless it is restrained by existing improvements. After the subgrades have been properly prepared, the areas may be raised to design grades by placement of engineered fill.

Soil with moisture content above optimum value should be anticipated during and shortly after rainy seasons. Unstable, wet or soft soil will require processing before compaction can be achieved. If construction schedule does not allow for air drying, other means such as lime treatment, excavation and replacement, geogrids or geotextile fabrics may be considered. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.6 Material for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special material is required.

Engineered fill materials should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain a minimum of 12 percent passing the No. 200 sieve. In addition to these requirements, import fill, if necessary, should have a low expansion potential as indicated by Plasticity Index of 15 or less, or Expansion Index of less than 20.

All import fills should be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

6.7 Cut and fill slopes

Temporary cut slopes will be necessary during construction. Temporary cut slopes should comply with the current OSHA requirements.

Final cut and fill slopes in the colluvial and alluvial soils should be constructed at inclinations no steeper than 3:1 (horizontal:vertical). Final cut slopes in greenstone bedrock may be constructed at inclinations no steeper than 2:1 (horizontal:vertical). Fill slopes should be over-built and cut back to their final configurations.

The detention basins will be construction with side slopes of 4:1 (horizontal:vertical). These side slopes are considered appropriate for the subsurface conditions at the basin sites.

Proper drainage gradients should be provided to prevent surface runoff from flowing over the crest of slopes which can cause erosion on the slopes. Slopes in soils should be vegetated to reduce the potential for erosion (note: it is not likely to plant in bedrock slopes).

6.8 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness, moisture conditioned to the recommended moisture content, and mechanically compacted to the recommendations below. Relative compaction or compaction is defined as the percentage ratio of field dry density of soil to the laboratory maximum dry density determined by ASTM Test Method D1557, latest edition. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills should be compacted to a minimum of 90 percent relative compaction at moisture content between 1 and 3 percent above the laboratory optimum value unless noted otherwise. Compaction within about 5 feet behind retaining walls should be performed with small equipment in thin lifts to void surcharging the walls.

In roadway areas, the upper 8 inches of the subgrade soil should be compacted to between 90 and 95 percent relative compaction. Aggregate base in pavement areas should be compacted to a minimum of 95 percent relative compaction at slightly above the optimum moisture content.

Fill placed on slopes with existing inclination steeper than 5:1 (horizontal to vertical) should be keyed and benched into the slopes as the fill is being placed. A keyway should be constructed at the base of the fill slope except at the existing pond (to be backfilled) where a toe key is not required. For planning purposes, the toe key should extend at least 24 inches below lowest adjacent grade and have a width of at least 1.5 times the width of the compaction equipment or 8 feet, whichever is wider. Benches should be constructed 4 to 6 feet horizontally into the slope as the new fill is placed and compacted in layers. The actual dimensions of the keying and benching should be determined in the field by our representative.

6.9 Utility Trench Backfill

Trench construction, including bedding and backfill, should conform to the City of Morgan Hill specifications. The City specifications call for 95 percent minimum relative compaction for trench backfill. Refer to the "Excavations" section of this report for a discussion on utility trench excavations.

6.10 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

7. FOUNDATION RECOMMENDATIONS

7.1 Retaining Walls

The project will include three proposed retaining walls (RW #1 through RW #3). Information on the walls is tabulated below. We understand Caltrans Type 5 retaining walls are proposed.

Table 7.1-1
Proposed Retaining Walls Information

Wall	Location (Stations)	Approximate Wall Height (ft)	Approximate Wall Length (ft)
RW #1	"H" 33+20.45 Lt 153.14 to "H" 34+06.05 Lt 74.87	6 to 8	156
RW #2	"H" 33+67.50 Rt 72.42 to "H" 33+97.02 Rt 38.50	4 to 6	54
RW #3	"H" 35+29.26 Lt 157.88 to "H" 35+84.23 Lt 151.81	8 to 12	90

Note: Approximate wall height is vertical distance between top of wall footing and top of wall stem as shown on the provided project plans.

7.1.1 RW#1 and RW#2

Our review of the design plans indicates RW#1 and RW#2 will each be retaining an existing slope with an inclination of about 3¾:1 (h:v) to 5¾:1 (h:v). These walls are anticipated to have essentially flat-lying ground in front of the walls.

According to Caltrans Revised Standard Plan RSP B3-4B, design of Type 5 (Case 2) retaining walls is based the following parameters.

- Soil friction angle (Ø) of 34 degrees
- Soil unit weight of 120 pcf
- Backfill slope of 2:1 (h:v) with a maximum vertical slope height of 35 feet as measured between the top of the wall foundation and ground surface above
- Horizontal seismic coefficient (k_h) of 0.2 and vertical seismic coefficient (k_v) of 0.0

Based on information from our drill hole DH-2, subsurface conditions at RW#1 and RW#2 are anticipated to consist of sandy fat clay over greenstone bedrock.

The fat clay encountered in our borings in the vicinity of these walls would not meet the minimum friction angle requirement. However, if the backfill slopes behind these walls are constructed at an inclination no steeper than the existing slope (4H:1V), the standard Type 5 wall design, from a geotechnical viewpoint, can be considered because:

- the lateral soil pressure of the fat clay (plasticity index of 36) with a 4:1 (h:v) backfill slope is about the same as that of a Ø=34 degree soil with a 2:1 (h:v) backfill slope

- the foundation stresses noted in the table included with RSP B3-4B are lower than the bearing capacity of the fat clay

7.1.2 RW#3

Retaining wall RW#3 will be constructed on the south, east and north sides of an existing PG&E tower. The top of wall elevation is anticipated to be near that of the ground surface behind the wall. The ground surface in front of the wall is sloping down at an inclination of about 4:1 to 6:1 (h:v). Subsurface conditions are anticipated to consist of 1 to 3 feet of fat clay over greenstone bedrock.

A Caltrans Type 5 (Case 2) retaining wall is proposed for RW#3. The wall design should be verified to account for the foundation loads from the PG&E tower.

Caltrans standard Type 5 retaining wall design assumes a level ground surface in front of the wall for passive resistance. The passive resistance will be reduced for a downward sloping ground surface in front of the wall. The footing foundation for RW#3 is anticipated to be in the greenstone. We recommend the footing foundation for RW#3 be embedded in the greenstone at such elevation to provide a minimum of 8 feet of horizontal offset between the toe of footing and the face of slope.

7.1.3 Wall Drainage

A subsurface drain should be installed behind each retaining wall extending from the wall bottom to about 1 to 2 feet below finished grade. The drain should consist of a 12-inch minimum wide blanket of drainage material consisting of either Class 2 Permeable material (Caltrans Standard Specifications, Section 68) or clean, 1/2 to 3/4-inch maximum size crushed rock or gravel. If crushed rock or gravel is used, it should be encapsulated in a geotextile filter fabric, such as Mirafi 140N or equivalent. Filter fabric is optional if Class 2 Permeable material is used. The top 2 feet below finish grade should be backfilled with compacted clayey soil to reduce infiltration of surface water.

A 4-inch minimum diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of each wall on a 2-inch thick bed of drain rock. The pipes should be sloped to drain by gravity to a proper collection system and be discharged at a proper outlet as designed by the project Civil Engineer.

7.2 Sound Walls

Sound Wall SW#1 will be constructed between Stations “H” 7+96.02 Rt 71.75 and “H” 29+78.07 Rt 65.74, a distance of about 2,176 feet. Sound Wall SW#2 will be constructed between

Stations “H” 10+49.07 Lt 38.89 and “H” 14+63.47 Lt 44.56, a distance of about 432 feet. Sound Wall SW#3 will be constructed between Stations “H” 44+31.72 Rt 35.5 and “H” 52+11.28 Rt 62. SW#1 and SW#2 will be 8 feet in height and SW#3 will vary from 8 feet to 12 feet 8 inches in height. We understand Caltrans standard sound walls per Caltrans Standard Plan Sheet B15-3 will be used.

The proposed sound walls may be supported on 16-inch diameter cast-in-drilled-hole (CIDH) pile foundations. The depth of the CIDH piles should be as specified in Caltrans Standard Plan Sheet B15-5 for a minimum soil friction angle of 30 degrees (Ø=30 Min) for Case 1 and Case 2 conditions.

The CIDH piles are anticipated to be constructed mostly through colluvial and alluvial soils. Locally, the CIDH piles may extend into greenstone bedrock. The foundation contractor should review the drill hole logs included in this report and provide equipment capable to complete the foundation excavations in the foundation materials.

Groundwater was not encountered in our drill holes at the time of our field investigation. However, groundwater may be present in the CIDH pile excavations during or shortly after rains. If encountered in the CIDH pile excavations, groundwater should be removed from the holes and concrete should be placed by the tremie method to displace water out of the CIDH pile excavations.

Settlement of the CIDH foundations will be due to elastic shortening of the piles and compression of the soils supporting the piles. Post-construction total settlement of a CIDH pile is anticipated to be less than 1 inch, with estimated differential settlement of ½ inch between adjacent supports.

7.3 Pavements

New flexible pavements will be constructed for the Hale Avenue Extension project. Laboratory R-value tests were performed on five selected bulk samples collected for this investigation. The measured R-values range from less than 5 to 20. An R-value of 5 was used to calculate the recommended minimum pavement sections presented in Table 7.4-1 below, for traffic indices of 6.5, 7.0, 7.5, 8.0, 8.5 and 9.0.

Table 7.3-1
Recommended Minimum Structural Pavement Sections

TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)	CLASS 3 AGGREGATE SUBBASE (inches)	TOTAL (inches)
6.5	10.0	---	---	10.0
	4.0	13.5	---	17.5
7.0	11.0	---	---	11.0
	4.0	15.5	---	19.5

TRAFFIC INDEX	HOT MIX ASPHALT (inches)	CLASS 2 AGGREGATE BASE (inches)	CLASS 3 AGGREGATE SUBBASE (inches)	TOTAL (inches)
7.5	11.5	---	---	11.5
	4.5	16.5	---	21.0
	4.5	8.0	9.5	22.0
8.0	12.5	---	---	12.5
	4.5	18.5	---	23.0
	4.5	8.0	11.5	24.0
8.5	13.5	---	---	13.5
	5.0	19.5	---	24.5
	5.0	8.0	12.5	25.5
9.0	14.0	---	---	14.0
	5.5	20.5	---	26.0
	5.5	8.0	14.0	27.5

* The City of Morgan Hill minimum structural pavement section is 4 inches of asphalt concrete over 8 inches of Class 2 Aggregate Base.

Pavement sections should be constructed on soil subgrades that have been prepared as outlined in the “Earthwork Recommendations” section of this report.

Asphalt Concrete should comply with the City of Morgan Hill specifications and Section 39 of the Caltrans Standard Specifications, latest edition. In the case of two asphalt concrete lifts, the bottom lift should be the base course, Type B, ¾-inch maximum aggregate size. The top lift should be the surface course, Type B, ½-inch maximum aggregate size. The Class 2 Aggregate Base material should conform to Section 26 of the Caltrans Standard Specifications. The Class 3 Aggregate Subbase material should conform to Section 25 of the Caltrans Standard Specifications.

8. POST-REPORT GEOTECHNICAL SERVICES

Post-report geotechnical services by GLA, typically consisting of pre-construction design consultations and reviews and construction observation and testing services, are necessary for GLA to confirm the recommendations contained in this report. This report is based on limited sampling and investigation, and, because of those constraints, may not have discovered local anomalies or other varying conditions that may exist on the project site. Therefore, this report is only preliminary until GLA can confirm that actual conditions in the ground encountered during construction conform to those anticipated in the report. Accordingly, as an integral part of this report, GLA recommends post-report, construction-related geotechnical services to assist the project team during design and construction of the project. GLA requires that it perform these services if it is to remain as the project Geotechnical Engineer-of-Record.

During design, GLA can provide consultation and supplemental recommendations to assist the project team in design and value engineering, especially if the project design has been modified after completion of our report. It is impossible for us to anticipate every design scenario and use of construction materials during preparation of our report. Therefore, retaining GLA to provide post-report consultation will help address design changes, answer questions and evaluate alternatives proposed by the project designers and contractors.

Prior to issuing project plans and specifications for construction bidding purposes, GLA should review the grading, drainage, and foundation plans and the project specifications to determine if the intent of our recommendations has been incorporated in these documents. We have found that such a review process will help reduce the likelihood of misinterpretation of our recommendations which may cause construction delay and additional cost.

Construction phase services can include, among other things, geotechnical observation and testing during site clearing, stripping, excavation, mass grading, subgrade preparation, fill placement and compaction, backfill compaction, foundation construction, and pavement construction activities.

GLA would be pleased to provide cost proposals for follow-up geotechnical services. Post-report geotechnical services may include additional field and laboratory services.

9. LIMITATIONS

In preparing the findings and professional opinions presented in this report, we have endeavored to follow generally accepted principles and practices of the engineering geologic and geotechnical engineering professions in the area and at the time our services were performed. No warranty, express or implied, is provided.

The conclusions and preliminary design recommendations contained in this report are based, in part, on information that has been provided to us. In the event that the general development concept or general location and type of structures are modified, our conclusions and recommendations shall not be considered valid unless we are retained to review such changes and to make any necessary additions or changes to our recommendations. To remain as the project geotechnical engineer-of-record, Geo-Logic Associates (GLA) must be retained to provide geotechnical observation and testing services during construction.

Subsurface exploration is necessarily confined to selected locations and conditions may, and often do, vary between these locations. Should conditions different from those described in this report be encountered during project development, GLA should be consulted to review the conditions and determine whether our recommendations are still valid. Additional exploration, testing, and analysis may be required for such evaluation. Should persons concerned with this project observe geotechnical features or conditions at the site or surrounding areas which are different from those described in this report, those observations should be reported immediately to GLA for evaluation.

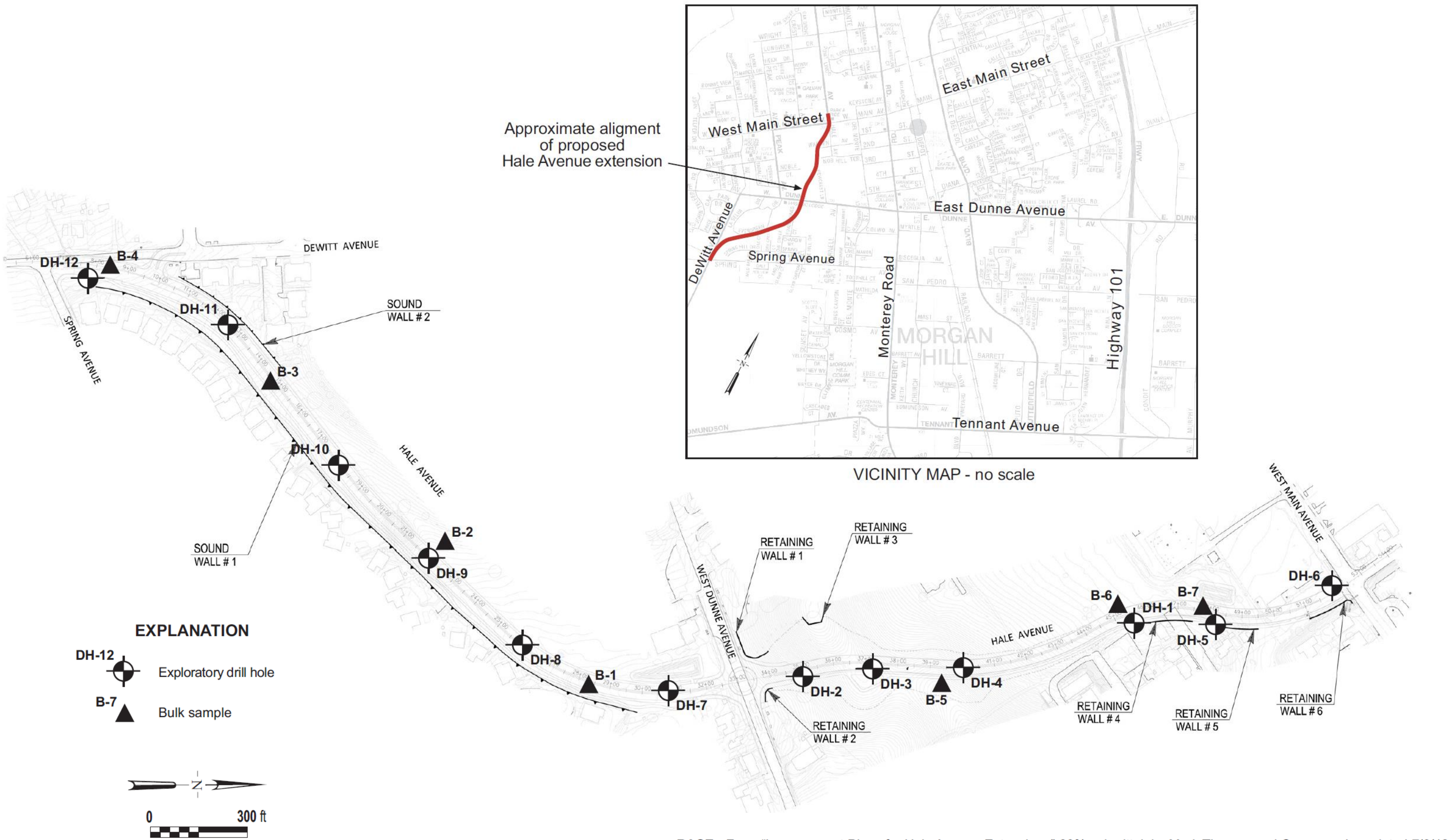
It is important that the information in this report be made known to the design professionals involved with the project, that our recommendations be incorporated into project drawings and documents, and that the recommendations be carried out during construction by the contractor and subcontractors. It is not the responsibility of GLA to notify the design professionals and the project contractors and subcontractors.

The findings, conclusions and recommendations presented in this report are applicable only to the specific project development on this specific site. These data should not be used for other projects, sites or purposes unless they are reviewed by GLA or a qualified geotechnical professional.

Report prepared by,

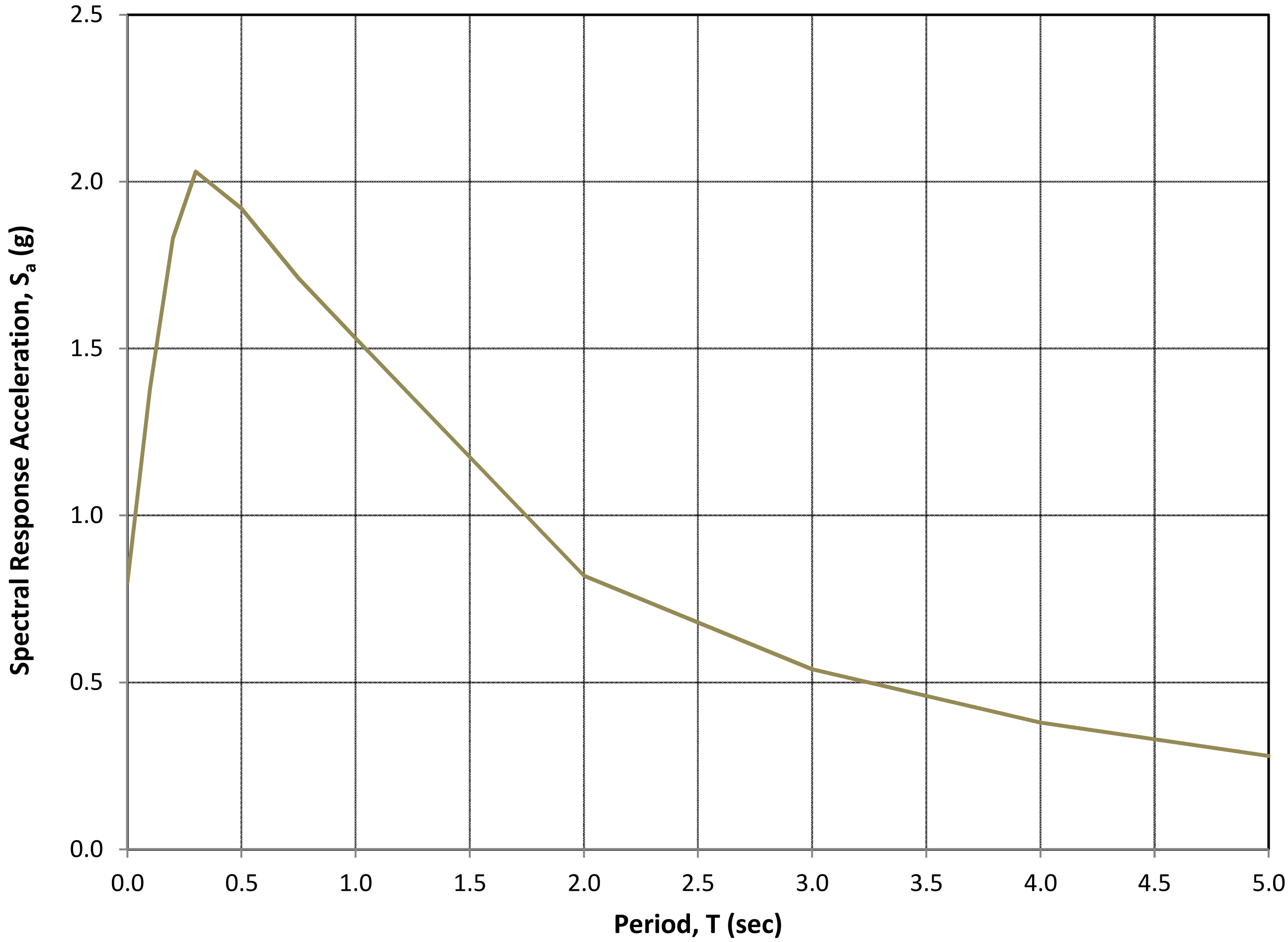
Geo-Logic Associates dba Pacific Geotechnical Engineering

Chalerm (Beeson) Liang
GE 2031



BASE: From "Improvement Plans for Hale Avenue Extension, " 60% submittal, by Mark Thomas and Company, Inc., dated 7/9/12.

Geo-Logic ASSOCIATES	16055 Caputo Drive, Suite D Morgan Hill, California 95037 Phone (408) 778-2818 Fax (408) 779-6879	Drafted By: Date: Checked By: Revision:	**SITE PLAN** HALE AVENUE EXTENSION MORGAN HILL, CALIFORNIA	**FIGURE** 1 **PROJECT** PA17.1024



Recommended Response Spectrum	
Period (sec)	**Spectral Acceleration (g)**
0	0.8
0.1	1.38
0.2	1.83
0.3	2.03
0.5	1.92
0.75	1.71
1	1.53
2	0.82
3	0.54
4	0.380
5	0.28

Note: based on V_{s30} of 270 m/s

Geo-Logic ASSOCIATES

16055 Caputo Drive
Morgan Hill, CA 95037
Phone 408-778-2818
Fax 408-779-6879

CALTRANS ARS CURVE
PROPOSED HALE AVE. EXTENSION
MORGAN HILL, CALIFORNIA

FIGURE

2

PROJECT
PA17.1024

Drafted by:	Date: August 2020
Review by:	Revision:

APPENDIX A

KEYS TO SOIL CLASSIFICATION

AND

DRILL HOLE LOGS

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS

(50% OR MORE IS SMALLER THAN NO. 200 SIEVE SIZE)

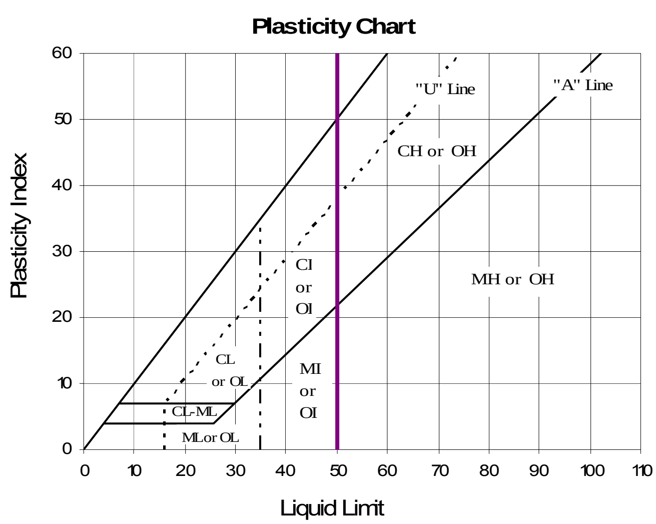
(modified from ASTM D2487 to include fine grained soils with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES
SILTS AND CLAYS (Liquid Limit less than 35) Low Plasticity	Inorganic	PI < 4 or plots below “A” line	ML	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above “A” line	CL	Lean Clay, Lean Clay with Sand or Gravel, Sandy or Gravelly Lean Clay, Sandy or Gravelly Lean Clay with Sand or Gravel
	Inorganic	PI between 4 and 7	CL-ML	Silty Clay, Silty Clay with Sand or Gravel, Sandy or Gravelly Silty Clay, Sandy or Gravelly Silty Clay with Sand or Gravel
	Organic	See footnote 3	OL	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)
SILTS AND CLAYS (35 ≤ Liquid Limit < 50) Intermediate Plasticity	Inorganic	PI < 4 or plots below “A” line	MI	Silt, Silt with Sand or Gravel, Sandy or Gravelly Silt, Sandy or Gravelly Silt with Sand or Gravel
	Inorganic	PI > 7 or plots on or above “A” line	CI	Clay, Clay with Sand or Gravel, Sandy or Gravelly Clay, Sandy or Gravelly Clay with Sand or Gravel
	Organic	See footnote 3	OI	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)
SILTS AND CLAYS (Liquid Limit 50 or greater) High Plasticity	Inorganic	PI plots below “A” line	MH	Elastic Silt, Elastic Silt with Sand or Gravel, Sandy or Gravelly Elastic Silt, Sandy or Gravelly Elastic Silt with Sand or Gravel
	Inorganic	PI plots on or above “A” line	CH	Fat Clay, Fat Clay with Sand or Gravel, Sandy or Gravelly Fat Clay, Sandy or Gravelly Fat Clay with Sand or Gravel
	Organic	See note 3 below	OH	Organic Silt (below “A” Line) or Organic Clay (on or above “A” Line) (1,2)

1. If soil contains 15% to 29% plus No. 200 material, include “with sand” or “with gravel” to group name, whichever is predominant.
2. If soil contains ≥30% plus No. 200 material, include “sandy” or “gravelly” to group name, whichever is predominant. If soil contains ≥15% of sand or gravel sized material, add “with sand” or “with gravel” to group name.
3. Ratio of liquid limit of oven dried sample to liquid limit of not dried sample is less than 0.75.

CONSISTENCY	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 – 0.5	2 – 4
FIRM	0.5 – 1.0	5 – 8
STIFF	1.0 – 2.0	9 – 15
VERY STIFF	2.0 – 4.0	16 – 30
HARD	> 4.0	> 30

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



GEO-LOGIC ASSOCIATES

KEY TO SOIL CLASSIFICATION – COARSE GRAINED SOILS

(MORE THAN 50% IS LARGER THAN NO. 200 SIEVE SIZE)

(modified from ASTM D2487 to include fines with intermediate plasticity)

MAJOR DIVISIONS			GROUP SYMBOLS	GROUP NAMES[1]
GRAVELS (more than 50% of coarse fraction is larger than No. 4 sieve size)	Gravels with less than 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3	GW	Well Graded Gravel, Well Graded Gravel with Sand
		Cu < 4 and/or 1 > Cc > 3	GP	Poorly Graded Gravel, Poorly Graded Gravel with Sand
	Gravels with 5% to 12% fines	ML, MI or MH fines	GW-GM	Well Graded Gravel with Silt, Well Graded Gravel with Silt and Sand
			GP-GM	Poorly Graded Gravel with Silt, Poorly Graded Gravel with Silt and Sand
		CL, CI or CH fines	GW-GC	Well Graded Gravel with Clay, Well Graded Gravel with Clay and Sand
			GP-GC	Poorly Graded Gravel with Clay, Poorly Graded Gravel with Clay and Sand
	Gravels with more than 12% fines	ML, MI or MH fines	GM	Silty Gravel, Silty Gravel with Sand
		CL, CI or CH fines	GC	Clayey Gravel, Clayey Gravel with Sand
		CL-ML fines	GC-GM	Silty Clayey Gravel; Silty, Clayey Gravel with Sand
SANDS (50% or more of coarse fraction is smaller than No. 4 sieve size)	Sands with less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	SW	Well Graded Sand, Well Graded Sand with Gravel
		Cu < 6 and/or 1 > Cc > 3	SP	Poorly Graded Sand, Poorly Graded Sand with Gravel
	Sands with 5% to 12% fines	ML, MI or MH fines	SW-SM	Well Graded Sand with Silt, Well Graded Sand with Silt and Gravel
			SP-SM	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel
		CL, CI or CH fines	SW-SC	Well Graded Sand with Clay, Well Graded Sand with Clay and Gravel
			SP-SC	Poorly Graded Sand with Clay, Poorly Graded Sand with Clay and Gravel
	Sands with more than 12% fines	ML, MI or MH fines	SM	Silty Sand, Silty Sand with Gravel
		CL, CI or CH fines	SC	Clayey Sand, Clayey Sand with Gravel
		CL-ML fines	SC-SM	Silty, Clayey Sand; Silty, Clayey Sand with Gravel

US STANDARD SIEVES 3 Inch ¾ Inch No. 4 No. 10 No. 40 No. 200

	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES & BOULDERS	GRAVELS		SANDS			SILTS AND CLAYS

RELATIVE DENSITY (SANDS AND GRAVELS)	STANDARD PENETRATION (BLOWS/FOOT)
Very Loose	0 - 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 - 50
Very Dense	50+

1. Add "with sand" to group name if material contains 15% or greater of sand-sized particle. Add "with gravel" to group name if material contains 15% or greater of gravel-sized particle.

MOISTURE	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soi is below the water table

GEO-LOGIC ASSOCIATES

ROCK QUALITY DESCRIPTIONS

HARDNESS**

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of the geologist's pick
Hard	Can be scratched with knife or pick only with difficulty. Hard blow with hammer required to break sample.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to ½ inch can be excavated by hard blow of point of a geologist's pick. Hand specimens broken with moderate blow.
Medium	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips about 1 inch maximum in dimension by hard blows of the point of a geologist's pick.
Soft	Can be grooved or gouged readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small pieces can be broken by finger pressure,
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces one inch or more thickness can be broken with finger pressure. Can be scratched readily by finger nail.

FRACTURE DIMENSIONS*

Fracture	Block Size (or Spacing[1])
Crushed	~5 microns to 0.1 ft
Intensely	0.05 to 0.1 ft
Closely	0.1 to 0.5 ft
Moderately	0.5 to 1.0 ft
Slightly	1.0 to 3.0 ft
Massive	3.0 ft and larger

WEATHERING**

Fresh or Unweathered	Rock fresh, crystals bright, few joints and fractures may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, fractures and joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints and fractures stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitic rock, some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitic rock, most feldspars are dull and discolored; some show clay. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately Severe	All rock except quartz discolored or stained. In granitic rock, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitic rock, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

1 Average distance between adjacent fractures
* Source of data unknown
** Source of data: "Subsurface Investigaiton for Design and Constructio of Foundation Buildings," (1976) American Society of Civil Engineers, Manuals and Reports on Engineering Practice – No. 5

GEO-LOGIC ASSOCIATES

DATE: 1/23/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 1

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: Mobile B53, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, FAT CLAY with SAND: Dark brown (7.5YR 3/2), moist, very stiff; with fine to coarse angular sand (Greenstone derived)	CH											
		1	S D D	21	3			22		98		
		2										
brown (7.5YR 4/2), hard		3	S D D	81	4.5+ 4.5+			20		109		
		4										
		5	S									
ALLUVIUM, CLAYEY SAND: Yellowish brown (10YR 5/4), moist, dense to very dense; mostly fine to medium sand	SC	6	D	50/4.5"								
		7										
		8										
		9	S I I	86								
BEDROCK, GREENSTONE: Yellowish brown, moist, rock mass is soft due to weathering, rock fragments are hard, very severely weathered, intensely fractured to crushed, angular fractures		10										
		11										
		12										
		13	I	50/5"								
BOTTOM OF HOLE @ 14 FEET NO GROUNDWATER ENCOUNTERED		14										
		15										
		16										
		17										
		18										
		19										
		20										

DATE: 4/26/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 6" solid stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, SANDY FAT CLAY: Very dark brown (10YR 2/2), moist, stiff; with fine to medium angular sand	CH	1										
		2	S D D	14		63	57	21	36	100	6	3734
		3										
		4										
dark brown (7.5YR 5/3), hard, transitioning to weathered bedrock		5										
		6	S D D	40	4.5+ 4.5+							
		7										
BEDROCK, GREENSTONE: Brown, moist, rock mass is soft due to weathering, rock fragments are hard, moderately weathered, intensely fractured to crushed, blocky fractures		8										
		9	S D D	83	4.5+ 4.5+	22		10		127		
		10										
		11										
		12										
		13										
harder rock, less weathered with depth		14	S I	50/5"								
		15										
hard drilling		16										
		17										
		18										
		19	I	50/5"								
		20										

DATE: 4/26/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 2

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 6" solid stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
BEDROCK, GREENSTONE (continued)												
		21										
		22										
		23										
			I	50/2.5"								
BOTTOM OF HOLE @ 23.7 FEET NO GROUNDWATER ENCOUNTERED		24										
		25										
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

GEO-LOGIC ASSOCIATES

DATE: 4/26/2012

LOG OF EXPLORATORY DRILL HOLE

DH- 3

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 6" solid stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, SANDY FAT CLAY: Very dark brown (10YR 2/2), moist, stiff; with fine to medium, angular sand	CH	1	S									
BEDROCK, GREENSTONE: Yellowish brown, dry to moist, rock mass is soft due to weathering and fracturing, moderately severely weathered, crushed		2	D D	33				14		115		
		3										
		4										
		5	S									
		6	D	50/6"								
		7										
		8										
		9	D	50/6"								
		10										
		11										
		12										
		13										
hard drilling		14	S I	50/3.5"								
		15										
		16										
		17										
		18										
		19	S I	50/5"		18		11				
		20										

DATE: 4/26/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 3

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 6" solid stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
BEDROCK, GREENSTONE (continued)												
		21										
		22										
		23										
		24	S I I	86		11		9				
BOTTOM OF HOLE @ 25 FEET NO GROUNDWATER ENCOUNTERED		25										
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

DATE: 4/26/2012

LOG OF EXPLORATORY DRILL HOLE

DH- 4

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 6" solid stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, SANDY FAT CLAY: Very dark brown (7.5YR 2.5/2), moist, stiff to hard; with fine to medium, angular sand	CH	1										
		2	SDD	16	4.5			20		103		
		3										
CLAYEY SAND: Dark brown (7.5YR 3/4), moist, medium dense; mostly fine to medium, angular sand; transition to bedrock	SC	4										
		5										
		6	SDD	41		32	69	18	42	108	3	4760
		7										
BEDROCK, GREENSTONE: Dark yellowish brown, moist, rock mass is soft due to weathering and fracturing, rock fragments are hard, moderately severely weathered, intensely fractured to crushed		8										
		9	D	50/6"								
		10										
		11										
		12										
very hard drilling		13										
		14	D	50/2.5"								
weathering decreases and fracture spacing increases with depth		15										
		16										
		17										
		18										
		19										
NO GROUNDWATER ENCOUNTERED		20	SII	50/4"		11		9				
BOTTOM OF HOLE @ 20 FEET												

DATE: 4/26/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 5

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, SANDY FAT CLAY: Dark brown (7.5YR 3/3), moist, stiff to hard; with fine to coarse, angular sand	CH											
		1	S									
		2	D D	12	4.5+			18		106	4	2328
more sand, coarser with depth		3										
		4										
		5	S									
		6	D D	71	4.5+ 4.5+			17		117		
		7										
BEDROCK, GREENSTONE: Dark brown, dry to moist, rock mass is soft due to weathering and fracturing, rock fragments are hard, moderately to severely weathered, mostly crushed, blocky fractures		8										
		9	S I	50/4"								
		10										
		11										
		12										
		13										
		14	I	50/4"								
BOTTOM OF HOLE @ 13.8 FEET **NO GROUNDWATER ENCOUNTERED**		15										
		16										
		17										
		18										
		19										
		20										

DATE: 4/26/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 6

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: CME 55, 140# auto hammer **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** 12 feet

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
PAVEMENT (3-4 inches AC, no base)												
FILL, CLAYEY SAND: Black (10YR 2/1), moist, medium dense; fine to medium sand	SC	1	S									
		2	D D	17								
ALLUVIUM, SANDY FAT CLAY: Very dark brown (10YR 3/1), moist, very stiff; with fine to medium sand	CH					73		24		96		
		3										
		4										
		5	S									
		6	D D	20	2.5			24		104		
hard drilling		7										
		8										
grading to Fat Clay with Sand		9	S D	38	4							
		10	D		4	85		24		105		
		11										
CLAYEY SAND: Brown (7.5YR 4/4), moist to wet, medium dense; mostly fine sand	SC	12										
		13										
		14	S D D	24								
BOTTOM OF HOLE @ 15 FEET **NO GROUNDWATER ENCOUNTERED**		15										
		16										
		17										
		18										
		19										
		20										

DATE: 4/27/2012

LOG OF EXPLORATORY DRILL HOLE

DH- 7

PROJECT NAME: Hale Avenue Extension

PROJECT NUMBER: 2012.0005

DRILL RIG: Mobile B53, 140# downhole hammer and wire winch

LOGGED BY: CSS

HOLE DIAMETER: 8" hollow stem auger

HOLE ELEVATION: -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
FILL, FAT CLAY with SAND												
COLLUVIUM, FAT CLAY with SAND: Black (10YR 2/1), moist, very stiff to hard; with fine to medium, angular to subrounded sand	CH	1	S									
		2	D D	31	4.5 2.5			21		99		
		3	S									
ALLUVIUM, CLAYEY SAND: Brown (10YR 4/3), moist, very dense; mostly fine to medium, angular to subrounded sand; grades coarser with depth	SC	4	D	50/6"								
		5	S									
		6	D	50/4.5"		45		19		109		
		7										
		8										
		9	S									
		10	D D	73	4.5 4.3							
		11										
CLAY with SAND: Brown (10YR 5/3), moist, hard; with mostly fine sand	CI	12										
		13										
		14	S									
		15	D D	76	4.5 4.5+			22		107		
BOTTOM OF HOLE @ 15 FEET NO GROUNDWATER ENCOUNTERED		16										
		17										
		18										
		19										
		20										

DATE: 4/27/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 8

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: Mobile B53, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, FAT CLAY with SAND: Black (10YR 2/1), moist, very stiff; with fine sand	CH	1 2	S D D	30	2.5		63	27	39	100		
ALLUVIUM, CLAYEY SAND to SANDY FAT CLAY: Brown (10YR 4/3), moist, very dense sand to hard clay; mostly fine to medium, angular to subrounded sand; grades coarser with depth	SC/ CH	3 4	S D D	28/6" 50/5"				20		107		
		5 6 7 8	S D	50/4.5"		48		19				
CLAY with SAND: Brown (10YR 3/3), moist, hard; with mostly fine sand	CI	9 10 11	S D D	51	4.5			23		103		
CLAYEY SAND: Brown (10YR 4/3), moist, very dense; mostly fine to medium, angular to subrounded sand	SC	12 13 14	S D	50/6"								
BOTTOM OF HOLE @ 14.5 FEET **NO GROUNDWATER ENCOUNTERED**		15 16 17 18 19 20										

DATE: 4/27/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 9

PROJECT NAME: Hale Avenue Extension	**PROJECT NUMBER:** 2012.0005
DRILL RIG: Mobile B53, 140# downhole hammer and wire winch	**LOGGED BY:** CSS
HOLE DIAMETER: 8" hollow stem auger	**HOLE ELEVATION:** -----
SAMPLER: D = 3" OD, 2½" ID Split-spoon X = 2½" OD, 2" ID Split-spoon I = Standard Penetrometer (2" OD SPT) S = Slough in sample	**GROUND WATER DEPTH:** Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, FAT CLAY: Black (10YR 2/1), moist, very stiff	CH											
COLLUVIUM, SANDY FAT CLAY: Black (10YR 2/1), moist, very stiff; with fine to coarse sand	CH	1	S									
		2	D D	21	4.5	56		20		105		
		3	S									
		4	D D	43				17		112		
ALLUVIUM, CLAYEY SAND: Dark brown (7.5YR 3/3 to 3/4), moist, medium dense; fine to coarse, angular to subrounded sand	SC	5	S									
		6	D D	35		46		20		111		
		7										
		8										
very dense		9	S D	50/6"								
BEDROCK, GREENSTONE: Dark reddish brown, dry to moist, rock mass is soft, rock fragments are hard, severely to very severely weathered; mostly crushed, blocky fractures		10										
		11										
		12										
		13										
		14	S D D	90+								
BOTTOM OF HOLE @ 15 FEET **NO GROUNDWATER ENCOUNTERED**		15										
		16										
		17										
		18										
		19										
		20										

GEO-LOGIC ASSOCIATES

DATE: 4/27/2012	**LOG OF EXPLORATORY DRILL HOLE**	**DH- 10**

PROJECT NAME: Hale Avenue Extension | **PROJECT NUMBER:** 2012.0005

DRILL RIG: Mobile B53, 140# downhole hammer and wire winch | **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger | **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, SANDY CLAY: Black (10YR 2/1), moist, very stiff; with fine to medium, angular to subrounded sand	CI	1										
		2	S D D	24	3.5			17		111		
		3										
dark brown (7.5YR 3/3)		4	S D D	25		58	45	19	26	109		
		5										
		6	S D D	39								
ALLUVIUM, CLAY with SAND: Yellowish brown (10YR 5/4), moist, very stiff; with fine to medium, angular to subrounded sand	CI	7										
		8										
SANDY CLAY to CLAYEY SAND: Yellowish brown (10YR 5/4), moist, hard clay to very dense sand; fine to coarse, angular to subrounded sand	CI/ SC	9	S D	50/6"		52		22		106		
		10										
		11										
		12										
very hard drilling		13										
		14	S I I	24								
BOTTOM OF HOLE @ 15 FEET **NO GROUNDWATER ENCOUNTERED**		15										
		16										
		17										
		18										
		19										
		20										

DATE: 4/27/2012

LOG OF EXPLORATORY DRILL HOLE

DH- 11

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: Mobile B53, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: **Initial:** --- **Final:** ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
FILL/DISTURBED COLLUVIUM: SANDY CLAY: Very dark brown (10YR 2/2), moist, very stiff; with fine to coarse, angular to subrounded sand and fine gravel	CI	1										
		2	SDD	26				13		112		
COLLUVIUM, SANDY CLAY: Very dark brown (10YR 2/2), moist, very stiff; with fine to coarse, angular to subrounded sand	CI	3										
		4	SDD	31	4 4			19		106		
ALLUVIUM, SANDY CLAY to CLAYEY SAND: Very dark brown (10YR 2/2), moist, hard clay /dense to very dense sand; fine to coarse, angular to subrounded sand	CI/ SC	5										
		6	SDD	50								
		7										
		8										
brown (7.5YR 4/4)		9	SDD	78				19		110		
		10										
		11										
		12										
		13										
		14	SDD	29								
BOTTOM OF HOLE @ 15 FEET NO GROUNDWATER ENCOUNTERED		15										
		16										
		17										
		18										
		19										
		20										

DATE: 4/27/2012	LOG OF EXPLORATORY DRILL HOLE	DH- 12

PROJECT NAME: Hale Avenue Extension **PROJECT NUMBER:** 2012.0005

DRILL RIG: Mobile B53, 140# downhole hammer and wire winch **LOGGED BY:** CSS

HOLE DIAMETER: 8" hollow stem auger **HOLE ELEVATION:** -----

SAMPLER:
D = 3" OD, 2½" ID Split-spoon
X = 2½" OD, 2" ID Split-spoon
I = Standard Penetrometer (2" OD SPT)
S = Slough in sample

GROUND WATER DEPTH: Initial: --- Final: ---

DESCRIPTION OF EARTH MATERIALS	SOIL TYPE	DEPTH (ft)	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	% PASSING #200 SIEVE	LIQUID LIMIT	WATER CONTENT	PLASTICITY INDEX	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
COLLUVIUM, SANDY CLAY: Dark brown (10YR 3/2), moist, very stiff; with fine to medium, angular to subrounded sand	CI											
		1	S									
			D		3		46	14	24			
		2	D	27	3			19		108		
		3	S									
ALLUVIUM, SANDY CLAY: Dark brown (7.5YR 3/3), moist, very stiff; with fine to coarse sand	CI		D		4							
		4	D	24	4			20		106		
		5	S									
BEDROCK, GREENSTONE: Brown, dry to moist, soft rock mass, rock fragments are hard; severely weathered with fragments of moderately weathered rock; intensely fractured to crushed, blocky fractures			D									
		6	D	39				16		109		
		7										
		8										
		9	D	50/6"								
		10										
		11										
		12										
		13										
		14	S									
			D	50/5.5"								
BOTTOM OF HOLE @ 14.5 FEET **NO GROUNDWATER ENCOUNTERED**		15										
		16										
		17										
		18										
		19										
		20										

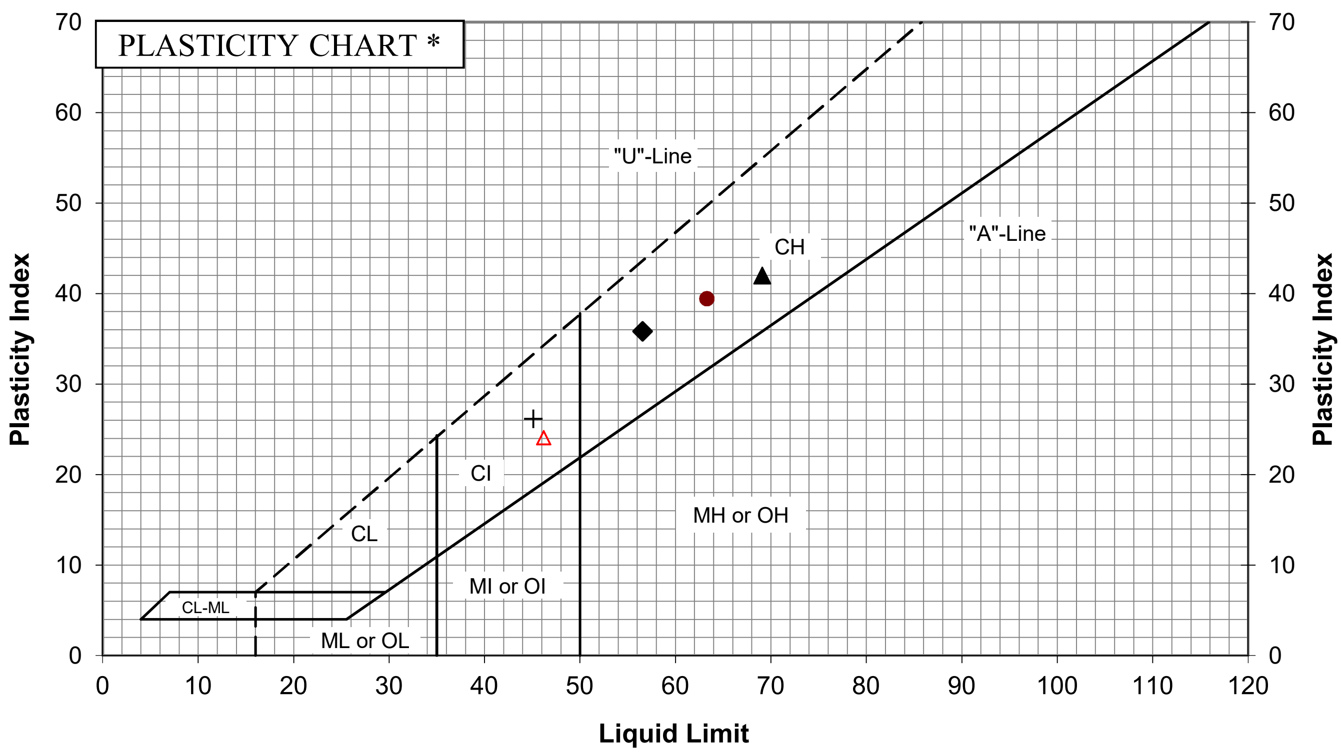
APPENDIX B

LABORATORY TEST RESULTS

ATTERBERG LIMITS TEST RESULTS

PROJECT NAME	Hale Avenue Extension	PROJECT No.	PA17.1024

DATE OF TEST	5/21/2012	5/21/2012	5/21/2012	5/21/2012	5/21/2012
KEY SYMBOL	♦	▲	●	+	Δ
DRILL HOLE No.	2	4	8	10	12
DEPTH (ft)	2	6	1.5	4	1.5
NATURAL WATER CONTENT (%)	21	18	27	19	14
% Retained No. 40 SIEVE (Est.)	30	52	18	33	44
% PASSING No. 200 SIEVE	63	32	---	58	---
LIQUID LIMIT	57	69	63	45	46
PLASTIC LIMIT	21	27	24	19	22
PLASTICITY INDEX	36	42	39	26	24
CLASSIFICATION SYMBOL	CH	CH	CH	CI	CI



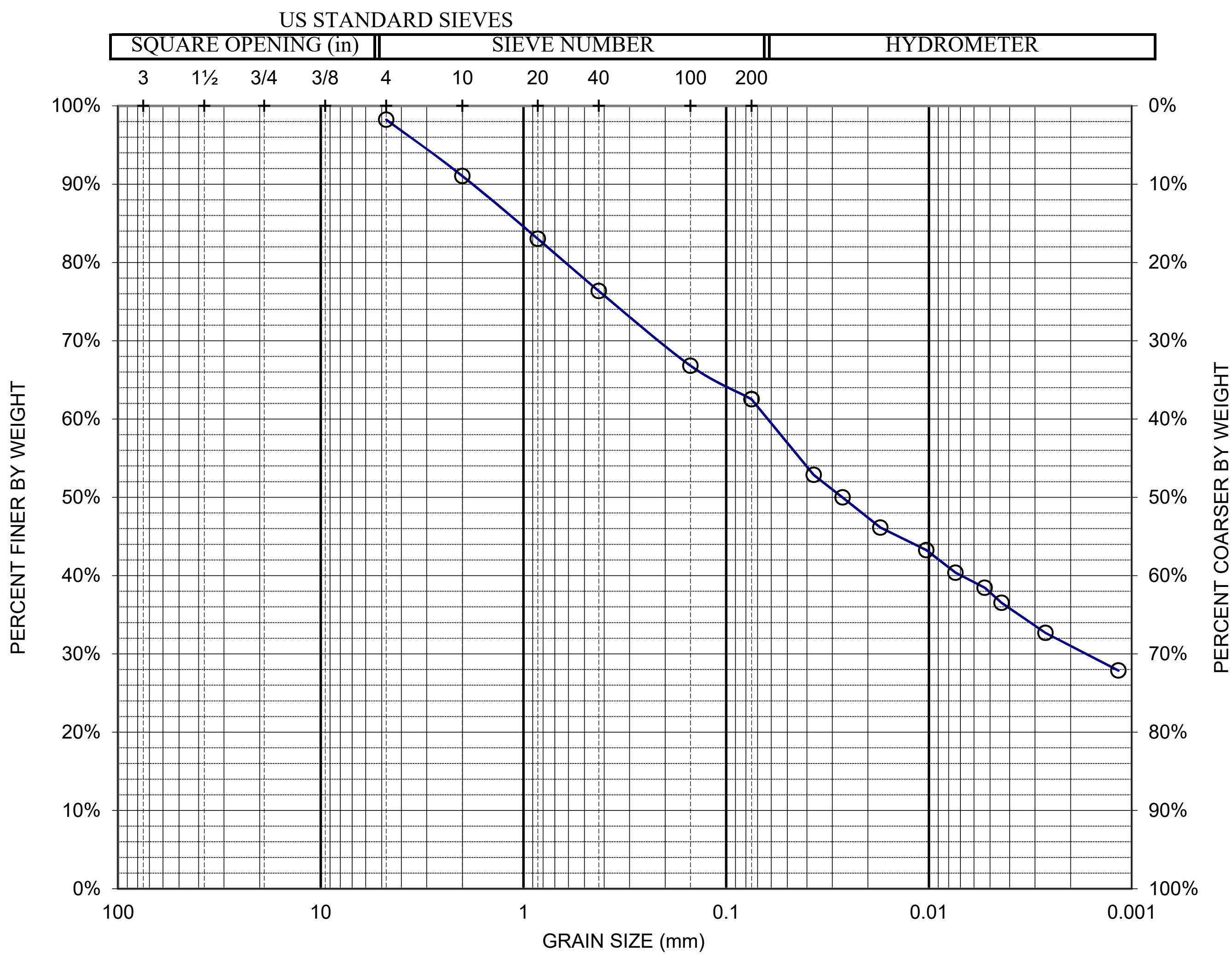
* Based on the Unified Soil Classification System modified to incorporate the "intermediate" classifications CI, MI, and OI for soils with liquid limits between 35 and 50. In the unmodified Unified Soil Classification System, such soils would be classified as CL, ML and OL, respectively.

GRAIN SIZE TEST RESULTS

PROJECT NAME	Hale Avenue Extension					**PROJECT No.**	PA17.1024
DRILL HOLE No.	2	**DEPTH (ft)**	2	**SAMPLE**	0	**DATE OF TEST**	5/21/2012

SOURCE/QUARRY: --

DESCRIPTION OF SOIL: SANDY FAT CLAY: Dark brown (10YR 2/2), moist



	GRAVEL		SAND			SILT & CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	1.8%		35.7%			62.5%

REMARKS:

GEO-LOGIC ASSOCIATES

Figure B-2

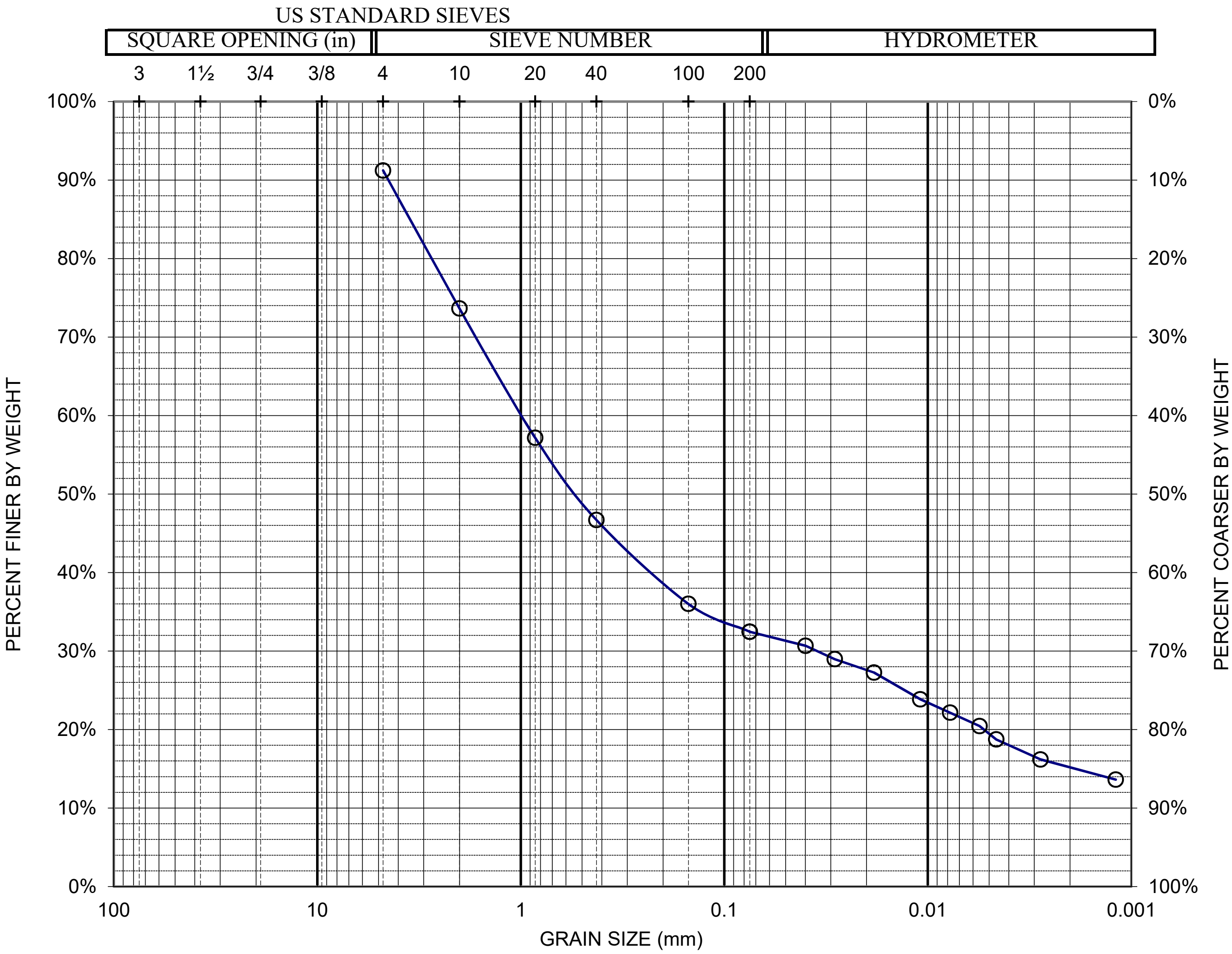
RevOct20070

GRAIN SIZE TEST RESULTS

PROJECT NAME	Hale Avenue Extension					**PROJECT No.**	PA17.1024
DRILL HOLE No.	4	**DEPTH (ft)**	6	**SAMPLE**	0	**DATE OF TEST**	5/21/2012

SOURCE/QUARRY: --

DESCRIPTION OF SOIL: CLAYEY SAND: Dark brown (7.5YR 3/4), moist



	COARSE	FINE	COARSE	MEDIUM	FINE	
COBBLES	GRAVEL		SAND			SILT & CLAY
	8.8%		58.8%			32.5%

REMARKS:

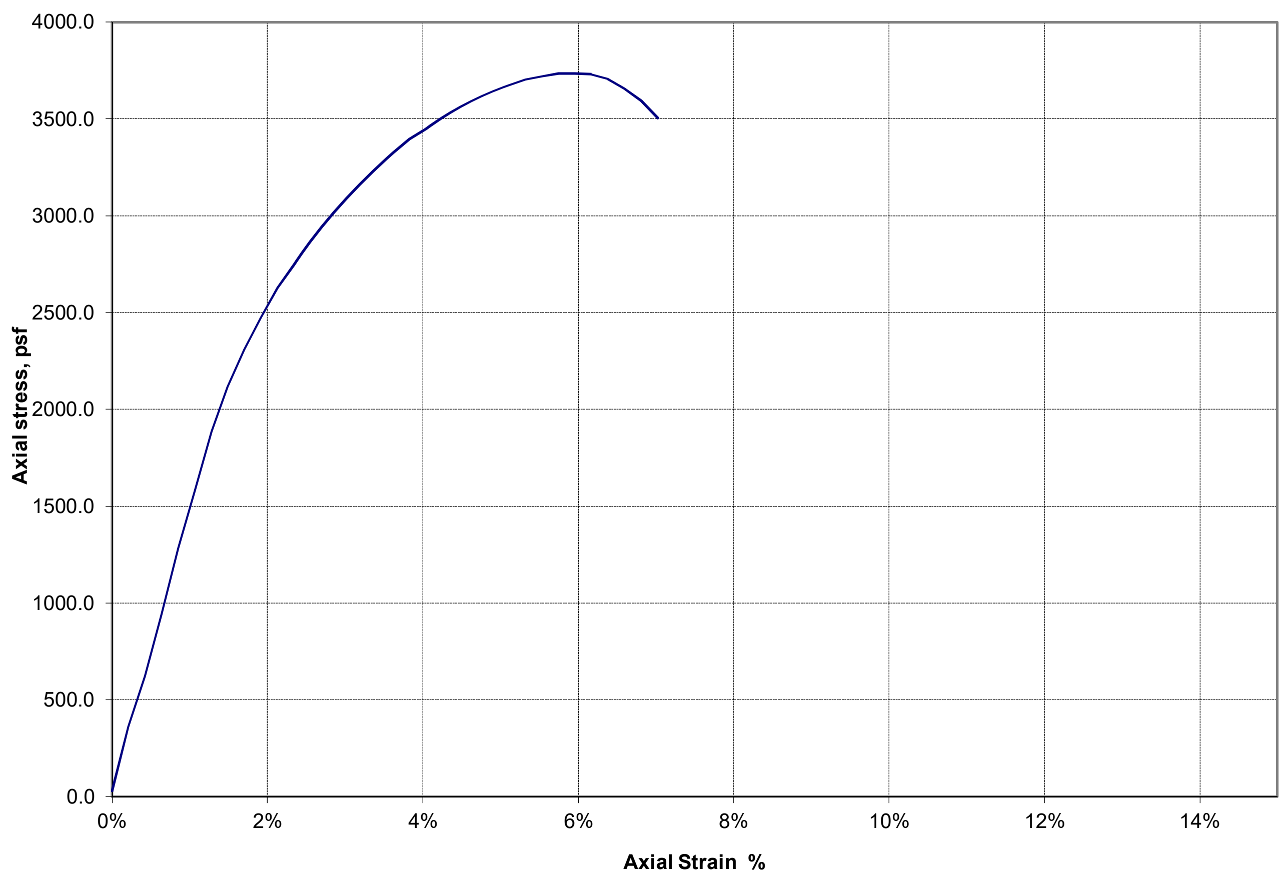
RevOct20070

GEO-LOGIC ASSOCIATES

Figure B-3

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Hale Avenue Extension			**PROJECT No.**	PA17.1024
DRILL HOLE No.	2	**DEPTH (ft)**	2.0	**SAMPLE**	**DATE OF TEST** 5/21/2012
DESCRIPTION OF SOIL	**SANDY FAT CLAY: Very dark brown (10YR 2/2), moist**				
LENGTH OF SAMPLE:	4.70 **in.**		**WATER CONTENT:**	21%	
DIAMETER OF SAMPLE:	2.39 **in.**		**DRY DENSITY:**	100	**pcf**
HEIGHT TO DIA. RATIO:	2.0		**MAX COMPRESSIVE STRENGTH:**	3734	**psf**
STRAIN RATE:	1.56% **per minute**		**STRAIN AT MAX STRENGTH:**	6.0%	



REMARKS:

Rev 032806

GEO-LOGIC ASSOCIATES

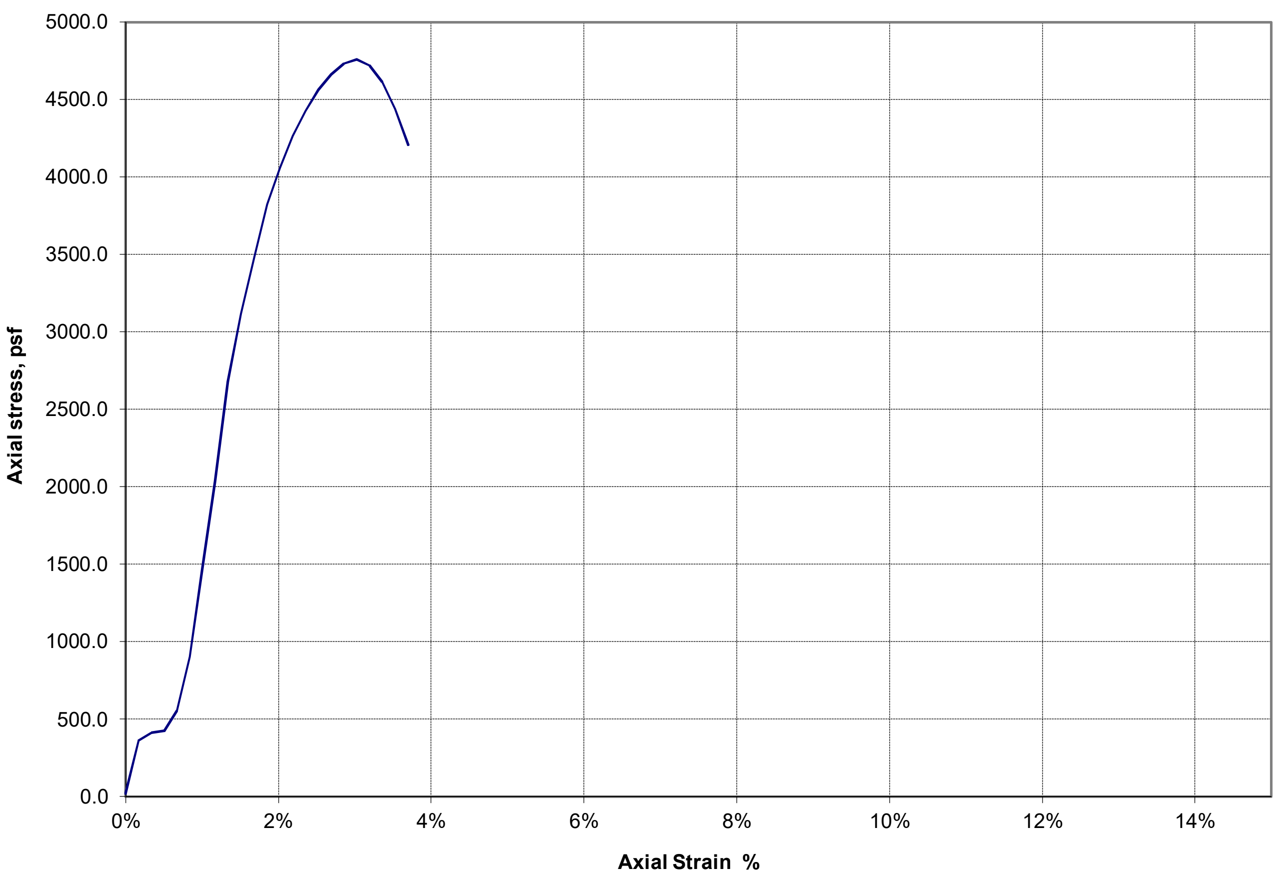
Figure B-4

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME Hale Avenue Exyension			**PROJECT No.** PA17.1024
DRILL HOLE No. 4	**DEPTH (ft)** 6.0	**SAMPLE**	**DATE OF TEST** 5/21/2012

DESCRIPTION OF SOIL **CLAYEY SAND: Dark brown (7.5YR 3/4), moist**

LENGTH OF SAMPLE:	5.95 **in.**	**WATER CONTENT:**	18%
DIAMETER OF SAMPLE:	2.40 **in.**	**DRY DENSITY:**	108 **pcf**
HEIGHT TO DIA. RATIO:	2.5	**MAX COMPRESSIVE STRENGTH:**	4760 **psf**
STRAIN RATE:	1.35% **per minute**	**STRAIN AT MAX STRENGTH:**	3.0%



REMARKS:

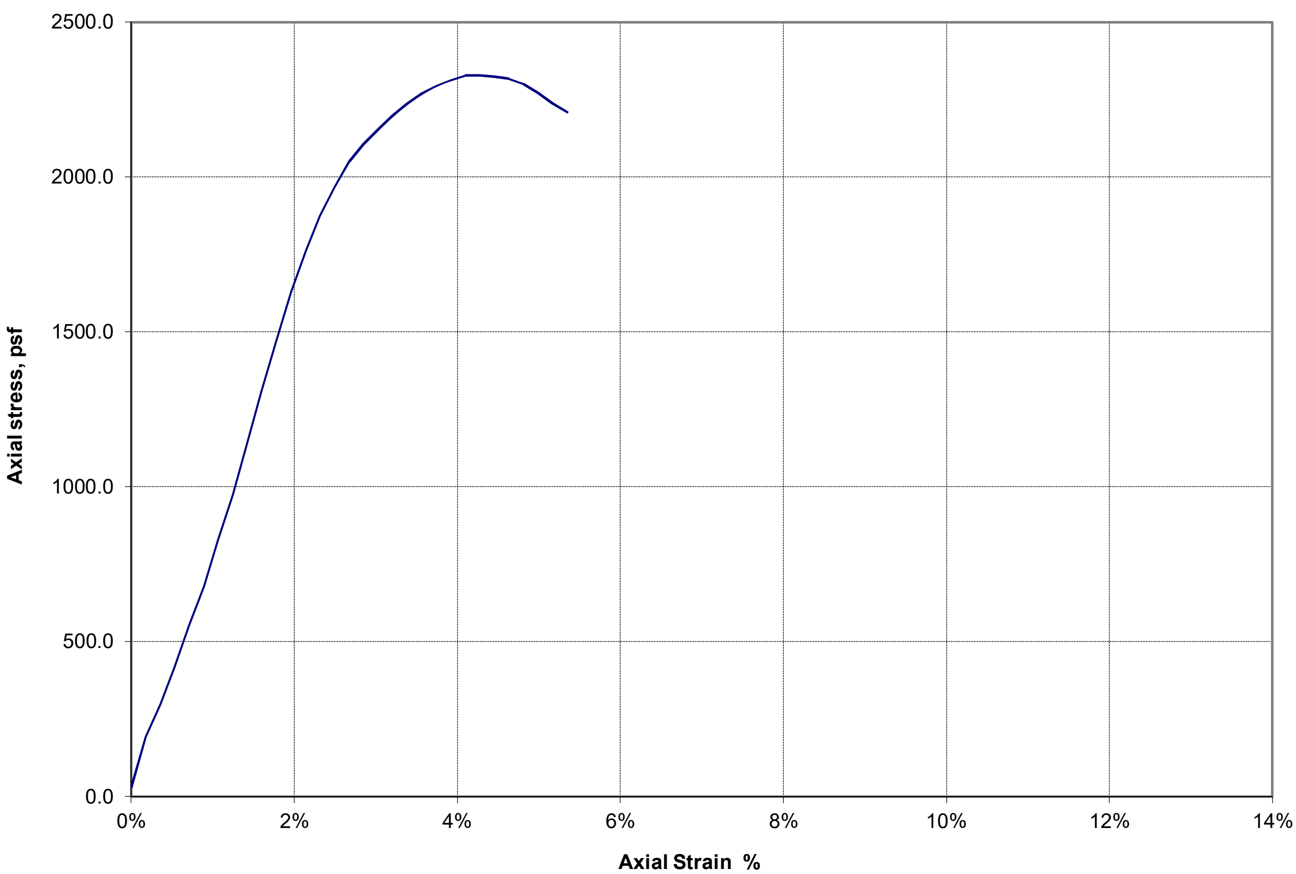
Rev 032806

GEO-LOGIC ASSOCIATES

Figure B-5

UNCONFINED COMPRESSION TEST RESULTS

PROJECT NAME	Hale Avenue Extension			**PROJECT No.**	PA17.1024
DRILL HOLE No. 5	**DEPTH (ft)** 2.0	**SAMPLE**		**DATE OF TEST**	5/21/2012
DESCRIPTION OF SOIL	**SANDY FAT CLAY: Dark brown (7.5YR 3/3), moist**				
LENGTH OF SAMPLE:	5.61 **in.**		**WATER CONTENT:**	18%	
DIAMETER OF SAMPLE:	2.37 **in.**		**DRY DENSITY:**	106 **pcf**	
HEIGHT TO DIA. RATIO:	2.4		**MAX COMPRESSIVE STRENGTH:**	2327.8 **psf**	
STRAIN RATE:	1.36% **per minute**		**STRAIN AT MAX STRENGTH:**	4.3%	



REMARKS:

Rev 032806

GEO-LOGIC ASSOCIATES

Figure B-6

R-value Test Report (Caltrans 301)

Job No.:	226-219	**Date:**	07/18/12	**Initial Moisture,**	13.6%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**<5**
Project:	Hale Avenue Extension - 2012.0005	**Reduced**	RU		
Sample	Bulk 1	**Checked**	DC	**Expansion Pressure**	**psf**
Soil Type:	Brown Sandy CLAY near Clayey SAND				

Specimen Number	A	B	C	D
Exudation Pressure, psi	317	142		
Prepared Weight, grams	1200	1200		
Final Water Added, grams/cc	86	115		
Weight of Soil & Mold, grams	3203	3261		
Weight of Mold, grams	2099	2085		
Height After Compaction, in.	2.6	2.69		
Moisture Content, %	21.8	24.5		
Dry Density, pcf	105.6	106.3		
Expansion Pressure, psf	98.9	38.7		
Stabilometer @ 1000				
Stabilometer @ 2000	142	150		
Turns Displacement	3.68	4.93		
R-value	8	3		

Remarks:

Soil extruded from the mold giving a false exudation pressure (exudation pressures for Specimens A & B are greater than the values reported). Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.

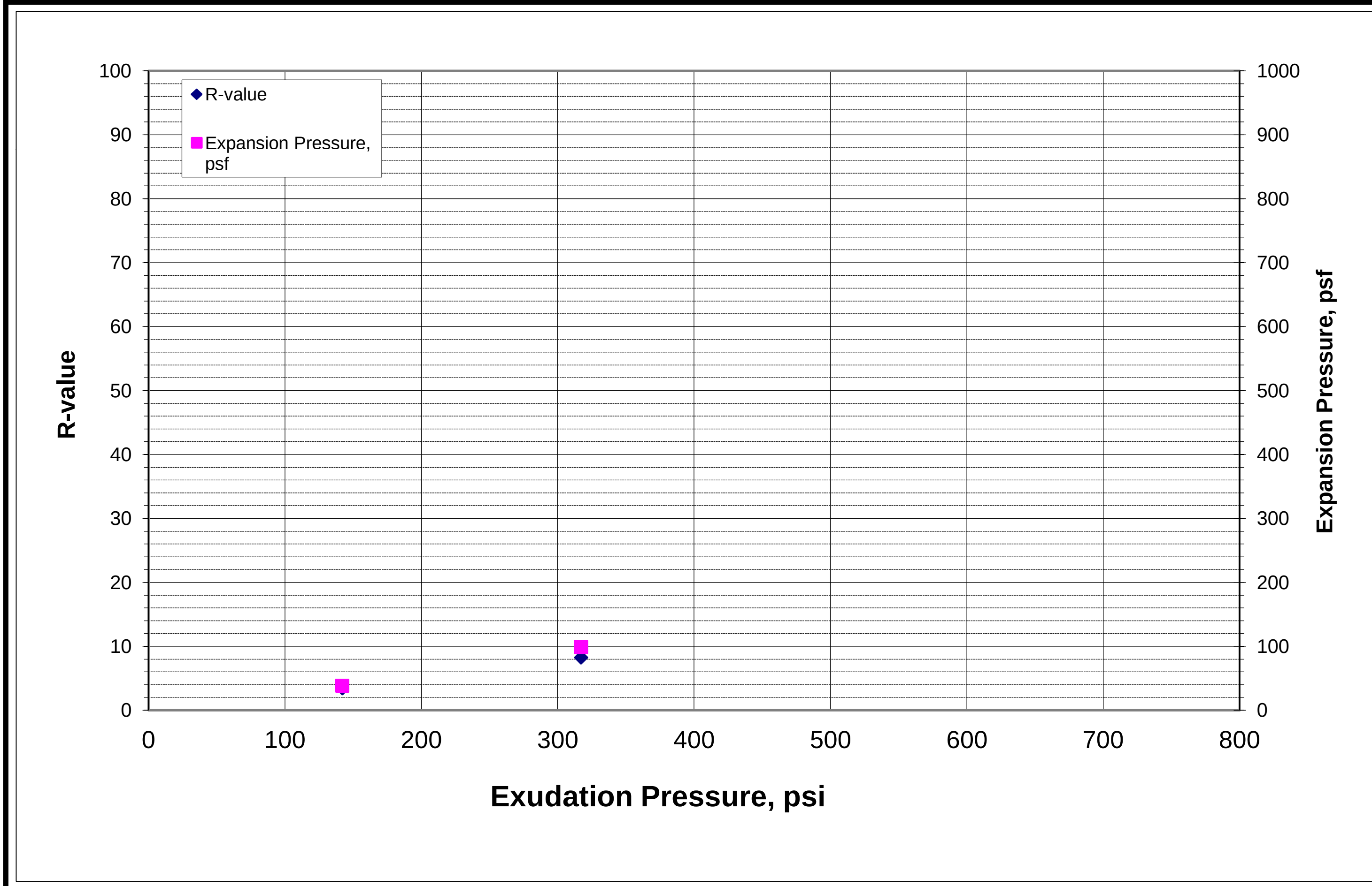


Figure B-7

R-value Test Report (Caltrans 301)

Job No.:	226-219	**Date:**	07/17/12	**Initial Moisture,**	10.8%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	**20**
Project:	Hale Avenue Extension - 2012.0005	**Reduced**	RU		
Sample	Bulk 3	**Checked**	DC	**Expansion Pressure**	**40** psf
Soil Type:	Dark Brown Clayey SAND, trace Gravel				

Specimen Number	A	B	C	D
Exudation Pressure, psi	217	481	352	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	103	55	79	
Weight of Soil & Mold, grams	3249	3213	3232	
Weight of Mold, grams	2106	2095	2084	
Height After Compaction, in.	2.68	2.65	2.46	
Moisture Content, %	20.3	15.9	18.1	
Dry Density, pcf	107.3	110.2	119.7	
Expansion Pressure, psf	17.2	81.7	51.6	
Stabilometer @ 1000				
Stabilometer @ 2000	136	90	115	
Turns Displacement	3.26	4.2	3.07	
R-value	13	35	24	

Remarks:

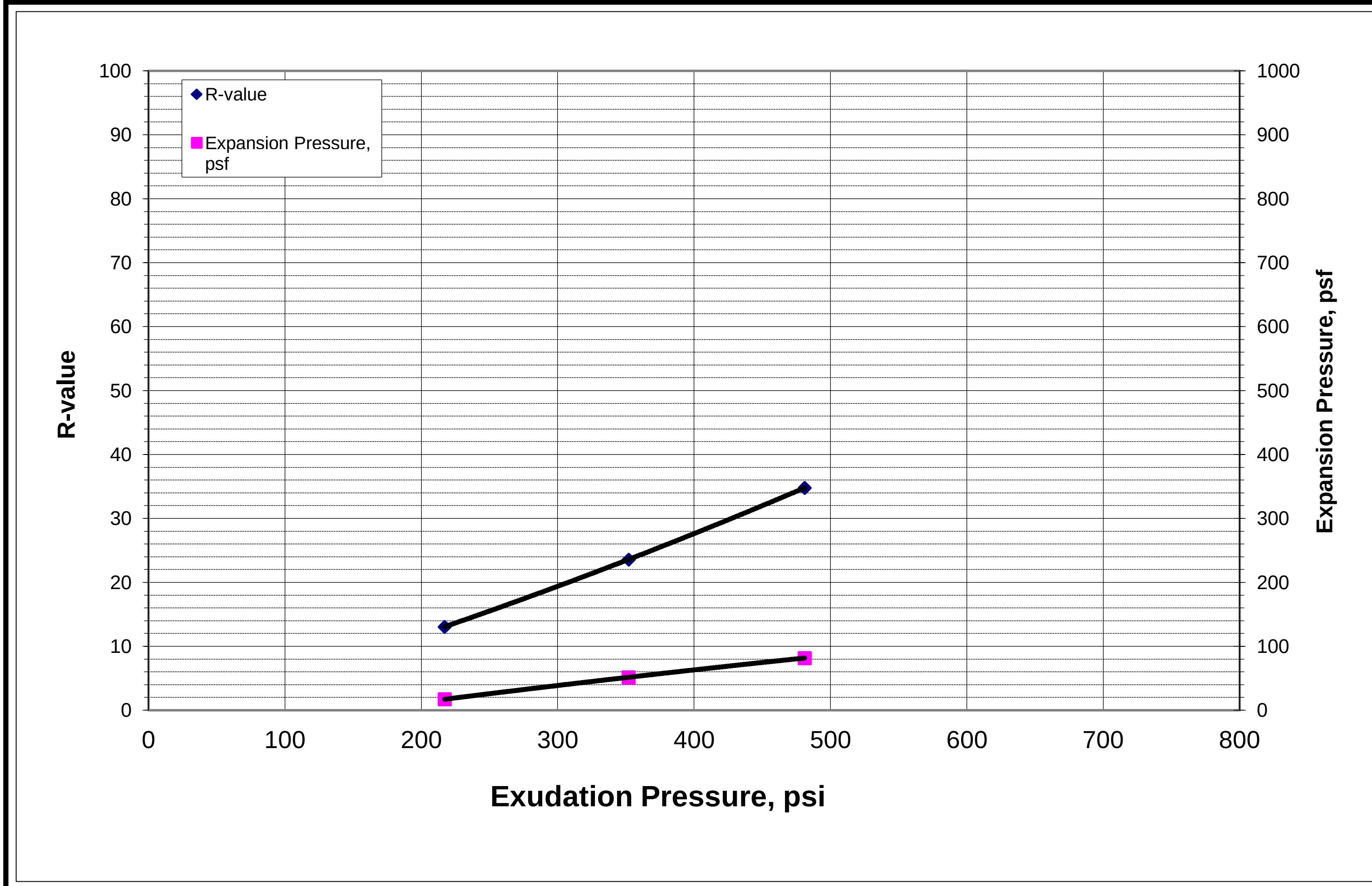


Figure B-8

COOPER TESTING LABORATORY

R-value Test Report (Caltrans 301)

Job No.:	226-219	**Date:**	07/13/12	**Initial Moisture,**	9.5%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	14
Project:	Hale Avenue Extension - 2012.0005	**Reduced**	RU		
Sample	Bulk 5	**Checked**	DC	**Expansion Pressure**	20 psf
Soil Type:	Olive Brown Clayey SAND, trace Gravel				

Specimen Number	A	B	C	D
Exudation Pressure, psi	171	217	335	429
Prepared Weight, grams	1200	1200	1200	1200
Final Water Added, grams/cc	56	41	32	21
Weight of Soil & Mold, grams	3281	3260	3263	3212
Weight of Mold, grams	2102	2087	2106	2098
Height After Compaction, in.	2.5	2.43	2.46	2.3
Moisture Content, %	14.6	13.3	12.4	11.4
Dry Density, pcf	124.6	129.1	126.7	131.6
Expansion Pressure, psf	0.0	0.0	21.5	47.3
Stabilometer @ 1000				
Stabilometer @ 2000	154	140	128	120
Turns Displacement	3.78	3.52	3.33	3.31
R-value	3	9	16	18

Remarks:

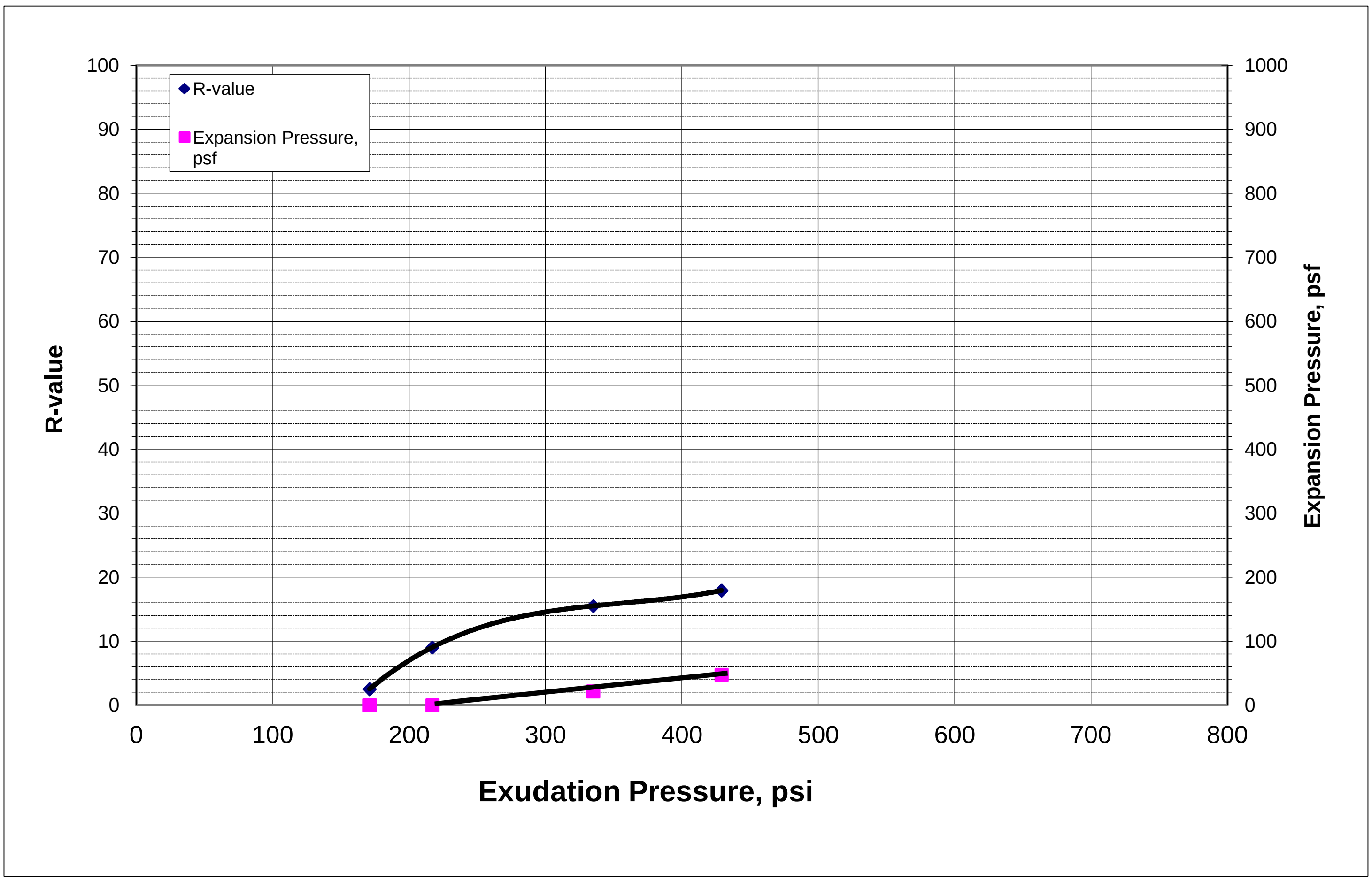


Figure B-9

R-value Test Report (Caltrans 301)

Job No.:	226-219	**Date:**	07/18/12
Client:	Pacific Geotechnical Engineering	**Tested**	MD
Project:	Hale Avenue Extension - 2012.0005	**Reduced**	RU
Sample	Bulk 6	**Checked**	DC
Soil Type:	Dark Yellowish Brown Sandy CLAY		

Initial Moisture,	19.4%
R-value by Stabilometer	**<5**
Expansion Pressure	**psf**

Specimen Number	A	B	C	D
Exudation Pressure, psi	193	386		
Prepared Weight, grams	1200	1200		
Final Water Added, grams/cc	85	45		
Weight of Soil & Mold, grams	3144	3120		
Weight of Mold, grams	2084	2078		
Height After Compaction, in.	2.65	2.45		
Moisture Content, %	27.8	23.9		
Dry Density, pcf	94.7	104.0		
Expansion Pressure, psf	25.8	47.3		
Stabilometer @ 1000				
Stabilometer @ 2000	154	146		
Turns Displacement	4.74	3.47		
R-value	2	6		

Remarks:

Soil extruded from the mold giving a false exudation pressure (exudation pressures for Specimens A & B are greater than the values reported). Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.

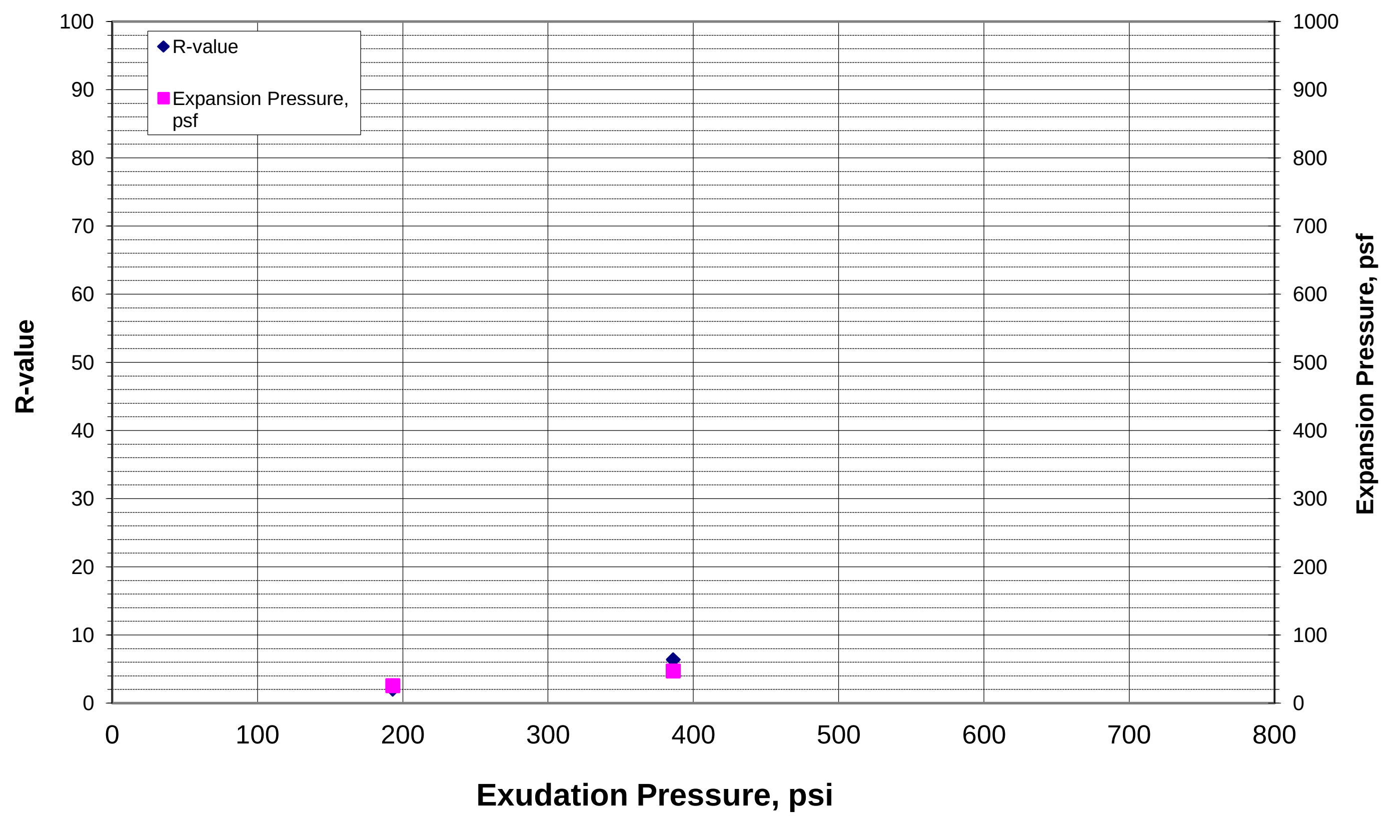


Figure B-10

R-value Test Report (Caltrans 301)

COOPER TESTING LABORATORY

Job No.:	226-219	**Date:**	07/18/12	**Initial Moisture,**	18.5%
Client:	Pacific Geotechnical Engineering	**Tested**	MD	**R-value by Stabilometer**	<5
Project:	Hale Avenue Extension - 2012.0005	**Reduced**	RU		
Sample	Bulk 7	**Checked**	DC	**Expansion Pressure**	psf
Soil Type:	Brown Sandy CLAY				

Specimen Number	A	B	C	D
Exudation Pressure, psi	205			
Prepared Weight, grams	1200			
Final Water Added, grams/cc	68			
Weight of Soil & Mold, grams	3151			
Weight of Mold, grams	2099			
Height After Compaction, in.	2.59			
Moisture Content, %	25.2			
Dry Density, pcf	98.2			
Expansion Pressure, psf	60.2			
Stabilometer @ 1000				
Stabilometer @ 2000	156			
Turns Displacement	4.61			
R-value	1			

Remarks:

Soil extruded from the mold giving a false exudation pressure. Per Caltrans, the R-Value test was terminated and an R-Value of less than 5 was reported.

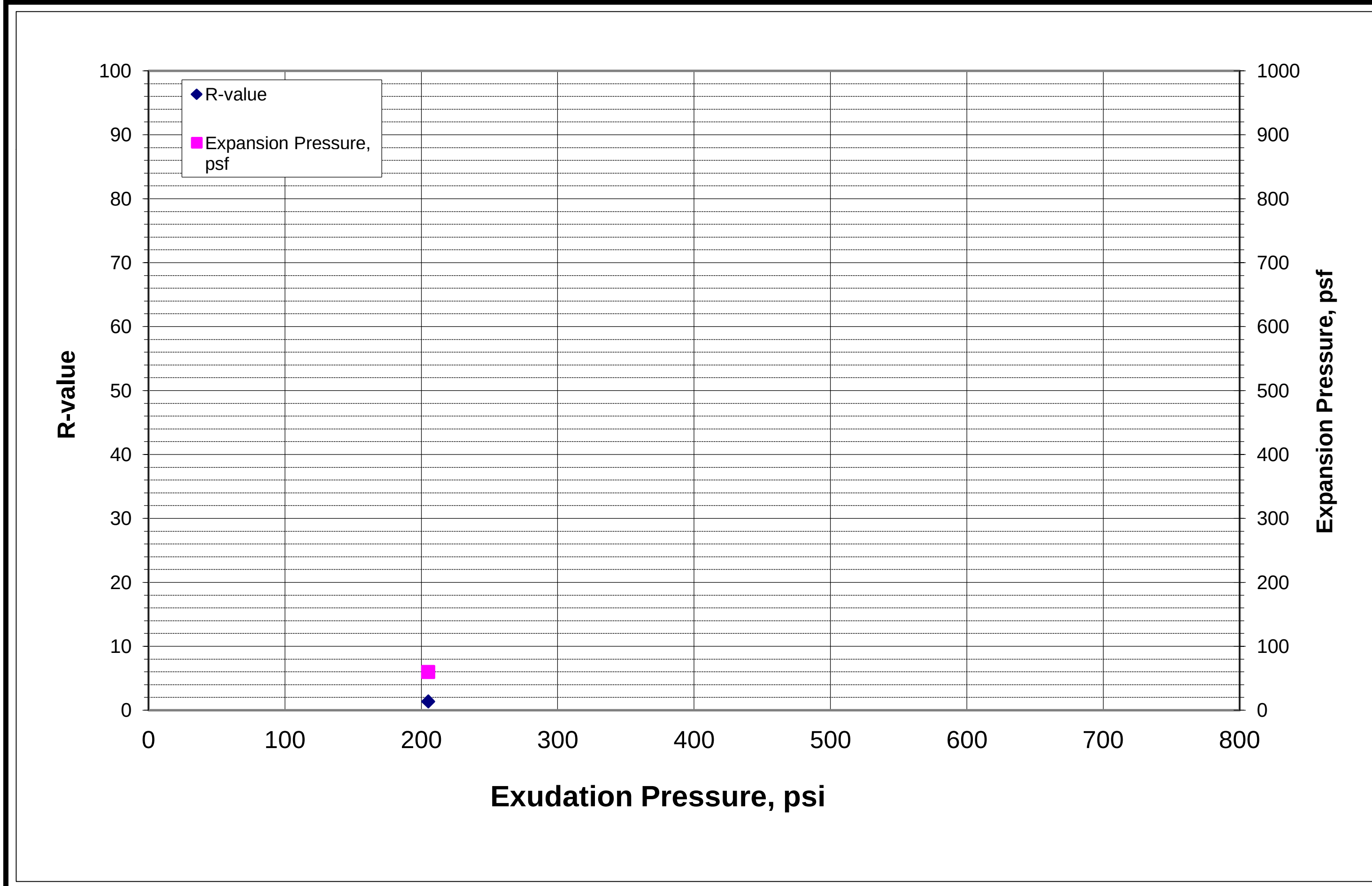


Figure B-11

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

30 May, 2012

Job No.1205107
Cust. No.10854

Mr. Beeson Liang
Pacific Geotechnical Engineering
16055-D Caputo Drive
Morgan Hill, CA 95037

Subject: Project No.: 2012.0005
Project Name: Hale
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Liang:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on May 14, 2012. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, all samples are classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a detection limit of 15 mg/kg.

The sulfate ion concentrations ranged from none detected to 57 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.2 to 7.8, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 420 to 450-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Pacific Geotechnical Engineering
Client's Project No.: 2012.0005
Client's Project Name: Hale
Date Sampled: 11-May-12
Date Received: 14-May-12
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 30-May-2012

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1205107-001	DH2 @ 5.5	450	7.6	-	5,000	-	N.D.	N.D.
1205107-002	DH6 @ 1.5+5.5	420	7.8	-	3,600	-	N.D.	57
1205107-003	DH11 @ 3.5+5.5	440	7.2	-	5,100	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	29-May-2012	25-May-2012 & 30-May-2012	-	24-May-2012	-	25-May-2012	25-May-2012

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits