PRELIMINARY GEOLOGIC AND GEOTECHNICAL REPORT

Review of Site Conditions
Old Town Newhall Library
Area Adjacent to Spruce Street, Between Lyons Avenue and 11th Street
City of Santa Clarita, California

Prepared for:
City of Santa Clarita
Department of Public Works
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Dear Mr. Corder:

This report presents our opinions regarding the existing geologic and geotechnical conditions at the above-referenced site, their potential effects on the proposed library facility, and our geotechnical recommendations for design and construction.

1.0 SCOPE OF REVIEW

Our investigation included the following tasks:

1. Coordination with the City of Santa Clarita and the Project Architect, LPA.

2. Review of site plans prepared by LPA, dated 1/6/09 and 5/15/09.

3. Review of the published geologic reports and maps referenced at the end of this report.

4. Review of the referenced consultant reports provided by the City of Santa Clarita.
5. Review of Alquist-Priolo earthquake fault zone and seismic hazard maps for the Newhall Quadrangle.

6. Site reconnaissance and coordination with Underground Service Alert prior to our subsurface investigation.

7. Drilling, sampling, and logging of six hollow-stem-auger borings to a maximum depth of 50 ft.

8. Excavation of two shallow backhoe pits at Spruce Street in order to conduct in-situ tests documenting the infiltration rate of the shallow soils at depths of 1.5 ft and 3.0 ft below existing road grade.

9. Laboratory testing of selected samples obtained from the borings to help define the engineering characteristics of the alluvial soils underlying the site and future compacted fill.

10. Review of the following aerial photographs:

<table>
<thead>
<tr>
<th>YEAR</th>
<th>PHOTO</th>
<th>SCALE</th>
<th>AGENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1928</td>
<td>C300 E191 &amp; E192</td>
<td>1”=~2,000’</td>
<td>Fairchild</td>
</tr>
<tr>
<td>3/29/68</td>
<td>4-122 and 4-123</td>
<td>1”=~2,000’</td>
<td>U.S.D.A.</td>
</tr>
<tr>
<td>3/8/81</td>
<td>PW 11484-4</td>
<td>1”=~1,400’</td>
<td>Pacific Western</td>
</tr>
</tbody>
</table>

11. Review of Division of Oil and Gas records and the Munger Map Book to assess if any oil wells have been drilled at the site.

12. Review of Los Angeles County Flood Control District (LACFCD) records to assess if any water wells have been drilled at the site.

13. Evaluation of historic high ground water elevations at the project site.

14. Evaluation of potential for hydro-collapse settlement of alluvial site soils located above the ground water surface.

15. Evaluation of potential ground rupture hazard at the site.
16. Evaluation of potential ground accelerations that could be generated at the site during future earthquakes on nearby faults.

17. Evaluation of potential for liquefaction of in-situ site soils and for liquefaction-induced phenomena, including lateral spreading and liquefaction-induced settlement.

18. Evaluation of potential static and seismic settlements at the site and development of appropriate mitigation options.


20. Evaluation of infiltration rates for native soils at the site based on data obtained during our site testing.

21. Preparation of a preliminary Geologic/Geotechnical Map utilizing the 5/15/09 Site Plan by LPA as a base.

22. Preparation of this report, which summarizes the results of our investigation and analyses, and our preliminary geotechnical recommendations to assist the project architect and civil engineer in the design of the site.

2.0 SITE DESCRIPTION

The site is located in the downtown Newhall area of the City of Santa Clarita (see Location Map). The subject site occupies 2.28 acres, including a segment of Spruce Street, adjacent areas between 11th Street and Lyons Avenue, and the abandoned portion of San Fernando Road. The footprint of the proposed library building is centered on the Spruce Street cul-de-sac and overlaps the existing building on the west side of Spruce Street and a portion of a dirt lot east of Spruce Street that was the previous site of a gas station. The site also encompasses the American Legion Building, an old jail, an adjacent Car Quest building, and associated asphalt covered parking lots. Review of historic aerial photos indicates that several small structures and dirt roadways were already in existence at the site as of 1928.
Source: U.S. Geological Survey Newhall, and Oat Mountain Quadrangles, Dated 1952
(Photorevised 1969), Dated 1952 (Photorevised 1969), Respectively

Approximate Scale: 1" = 2,000'

NOTE: THIS IS NOT A SURVEY OF THE PROPERTY
3.0 PROPOSED DEVELOPMENT

The site is proposed for redevelopment with a two-story, steel-frame library facility with roughly 26,900 sq. ft of floor space. Anticipated typical column loads in the library provided by the project structural engineer (LPA) are as follows:

<table>
<thead>
<tr>
<th>Location</th>
<th>Dead Load (kips)</th>
<th>Live Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior of building - stack area</td>
<td>85</td>
<td>150</td>
</tr>
<tr>
<td>Interior of building - other areas</td>
<td>96</td>
<td>90</td>
</tr>
<tr>
<td>Perimeter of building - stack area</td>
<td>65</td>
<td>80</td>
</tr>
<tr>
<td>Perimeter of building - other areas</td>
<td>70</td>
<td>50</td>
</tr>
</tbody>
</table>

Additional proposed improvements include a water feature, “Gateway Signage”, and parking areas.

It is our understanding that on-site infiltration of rainfall is proposed in response to Low Impact Development (LID) requirements. Parameters describing the infiltration rates of the in-situ soils are provided herein to assist in the design of LID measures. It is our understanding that the existing building west of Spruce Street (currently housing a pet grooming/care shop and a tattoo shop) and the Car Quest building will be removed. The old jail and American Legion building are proposed to remain.

4.0 FIELD EXPLORATION, SAMPLING, AND LABORATORY TESTING

4.1 Subsurface Exploration

Our subsurface exploration consisted of drilling, sampling, and logging of six (6) hollow-stem-auger borings (HS-1 through HS-6). In addition, two backhoe test pits (TP-1 and TP-2) were excavated to assess the infiltration characteristics of the native alluvial soils at the site. The boring and test pit locations are shown on the attached Geologic/Geotechnical Map (see Plate I).

The borings, which were drilled by All Ways Drilling on May 14 and 15, 2009, extended to depths varying from about 30 to 50 ft. All of the borings were logged and sampled by a representative of Allan E. Seward Engineering Geology, Inc. (AESEGI). The boring logs, which are provided in Appendix A, represent AESEGI’s interpretation of subsurface
conditions at the time of exploration. This interpretation is based on observations made by AESEGI’s personnel at the time of drilling and on subsequent laboratory testing. Contacts between geologic units shown on the boring logs are approximate and may represent gradual or interbedded transitions.

Following completion of logging, the borings were backfilled with cuttings from the drilling operations to within 10 ft of the ground surface. Grout was placed in the upper 10 ft of each hole to minimize infiltration, and the surface was capped with asphalt patch in the areas of existing asphalt. The backhoe pits were backfilled using the native excavated material. Backfill material was moisture-conditioned and placed in the test pit, then compacted using the backhoe with sheepsfoot attachment. The fill was capped with moisture-conditioned base and asphalt patch, and then wheel-rolled with the backhoe.

4.2 Sampling Procedures

California Drive (CD) ring and Standard Penetration Test (SPT) samplers were used to obtain soil samples from the borings. Samples generally were obtained at 3-ft depth intervals to a depth of 15 ft and at 5-ft depth intervals below 15 ft. Recovered soil samples were sealed to preserve the in-situ moisture content and brought to our geotechnical laboratory for further classification and testing.

Bulk samples of the near-surface soils were obtained from cuttings generated by excavation of the borings. Materials in these bulk samples represent a mixture of soils within the noted depth range. In addition, bulk samples of materials were obtained from pits excavated for field infiltration testing at the site.

4.3 Laboratory Testing

Soil samples were visually classified at the site in accordance with the Unified Soil Classification System (ASTM D2487). Thereafter, the samples were brought to our geotechnical laboratory, the visual soil classifications were checked, and the boring logs were reviewed in order to select soil samples for testing.

The laboratory testing program performed on samples of on-site soils included the following tests: moisture content, dry density, percent minus no. 200 sieve (i.e., percent fines), particle-size analysis, Atterberg limits, expansion index, corrosivity (sulfate content, chloride content, pH, and resistivity), direct shear, consolidation and hydro-
consolidation, Modified Proctor (compaction), and R-value.

Laboratory test methods and results of the testing are provided in Appendix B of this report.

5.0 GEOLOGIC AND GEOTECHNICAL CONDITIONS

5.1 Geologic Setting

The site is located within the central part of the Transverse Ranges geomorphic province of southern California, in the eastern portion of the Ventura Basin. The Ventura Basin has been tectonically down-warped in the geologic past to produce a large-scale synclinal structure in which a thick sequence of Cenozoic sediments have accumulated. At shallow depths, the subject site is underlain entirely by sub-horizontal alluvial deposits. The alluvium is underlain by the Plio-Pleistocene, nonmarine Saugus Formation. No faults or folds have been identified at the site on any published geologic map of the area.

5.2 Geologic Units

The project site is underlain to the depths explored by Quaternary Alluvium (Qal). Base was observed below the asphalt in the abandoned portion of San Fernando Road, and artificial fill and/or shading have been placed in the past to backfill existing utility trenches. A cap of artificial fill may underlie the existing buildings and portions of the abandoned gas station site. These shallow units are currently undefined. Details of the geologic units observed at the site are presented in our boring logs (Appendix A).

5.2.1 Quaternary Alluvium (Qal)

Quaternary Alluvium (Qal) underlies the site to a depth of at least 50 ft (the maximum depth of our exploration). The alluvium generally consists of interbedded layers of loose to dense, poorly graded sand, silty sand, and gravely sand. Interbedded layers of sandy silts and clays are also present at some locations. Dry density of soils in the alluvium generally ranges from about 100 to 130 pcf. Additional information regarding the alluvium is provided in the boring logs attached in Appendix A.
5.2.2 Bedrock

Bedrock was not encountered within the maximum depth of our borings (i.e., about 50 ft). It is anticipated that Saugus Formation bedrock is located between 50 and 100 ft beneath the site surface.

5.3 Geologic Structure

The alluvial deposits are nearly horizontal. The geologic structure of the underlying Saugus Formation bedrock strikes roughly east to west and dips gently to the north.

5.4 Ground Water

No ground water was encountered in our hollow-stem-auger borings (which ranged from about 30 to 50 ft in depth).

Three methods were used to assess the depth to historic high ground water at the site. First, we reviewed historic high ground water depth contours prepared for the Seismic Hazard Map Report (SHMR) for the Newhall Quadrangle. Second, we reviewed historic ground water depth contours prepared for the Santa Clarita Valley area by Robson (1972). Third, we obtained historic, Los Angeles County Flood Control District (LACFCD) ground water elevation data for water wells constructed in the vicinity of the site (see Table 1 for data).

The historic high ground water contour map published in the SHMR for the Newhall Quadrangle indicates that no ground water contours have been delineated at the site (see Ground Water Map). The closest ground water contour up-canyon of the site indicates a depth of 80 ft and contours projected along trend from the southwest indicate ground water depths of 75 ft to 100 ft at the site. The closest ground water contour down-canyon of the site indicates a depth of 55 ft. These contours indicate that the depth to historic high ground water at the site is greater than 50 ft and is more likely between 75 and 100 ft. Review of Robson (1972) indicates that no contours of historic high ground water were defined at the site. Review of LACFCD water well data indicates that there are historic records for 7 water wells located within 0.5 miles of the site. Historic high and low ground water levels for each of these wells are provided in Table 1. Historic high depths to ground water in these wells have ranged from 47.5 to 103.2 ft. The closest well to the site (5871D) has been monitored for the longest period of time (1948 to 2006) and shows a
historic high ground water depth of 100.2 ft. Well 5861E shows ground water depths ranging from 33.0 to 106 ft between 2000 and 2008. However, the shallower ground water depths recorded in this well are considered a local anomaly because the nearby Wells 5861K and 5871D do not show ground water shallower than 100 ft during the same monitoring period. Based on the available data, a historic high ground water level of 70 ft is estimated for the site.

5.5 Infiltration Characteristics of Native Soils

A preliminary assessment of on-site infiltration of water through shallow native soils was performed to assist in the design of a storm water run off infiltration system at the site, in compliance with LID requirements. The rate of infiltration was measured at depths of 1.5 and 3 ft below the surface of Spruce Street. Infiltration rates ranging from about 17 to 116 cm/hour were measured. Results of the infiltration testing and description of the test procedures used are provided in Appendix C.

5.6 Oil Wells and Water Wells

Review of the Munger Map book and California Division of Oil and Gas records indicates that no oil wells have been drilled on or immediately adjacent to the site. If any oil wells are encountered during future grading operations at the site, their location(s) should be surveyed and the current well conditions evaluated immediately.

Review of LACFCD records indicates that no water wells have been drilled at the site. If a water well is encountered during future grading operations at the site, the location should be surveyed and the well conditions evaluated immediately.

5.7 Hydro-Consolidation Potential and Soil Compressibility

In-situ dry density and moisture content of unsaturated granular soils were used to select the most hydro-consolidation prone soil samples from the hollow-stem-auger borings. The maximum compression following inundation measured in our hydro-consolidation testing was 0.5% (on sample HS-5 @ 40 ft). Therefore, potential for hydro-consolidation settlement of alluvial soils at the site is expected to be low.

Compressibility of coarse-grained alluvial site soils was estimated using Standard Penetration Test (SPT) blow count data. Compressibility of localized clayey layers in the
alluvium was estimated using data from the consolidation test performed on sample HS-6 @ 12 ft. Recommendations to limit total settlement of footing foundations located in these deposits (resulting from static plus live loading) to less than about 1.0 inch in building pad areas are provided in the Removals and Conventional Shallow Footing Foundations sections of this report.

5.8 Expansion Potential of Soils

Based on results of Expansion Index testing and on our visual observation of samples, in-situ soils at the site are generally granular with very low to low plasticity. An Expansion Index value of less than 20 was measured on a mixture of alluvial soils in HS-1 from 0-10 ft. It is anticipated that site soils when removed, mixed, and replaced as compacted fill, will have a very low expansion potential (per UBC expansion potential classification system).

The Expansion Index of the building pad soils should be measured at the completion of grading in order to select minimum embedment of footing foundations and design parameters for concrete slab-on-grade floors.

5.9 Soil Corrosivity

Resistivity, soluble sulfate content, chloride content, and pH were measured on a mixture of alluvial soils from depths of 0-10 ft in HS-1. Results of this testing are presented in Table B1 (Appendix B) and are discussed below.

- The measured resistivity value of the soil sample was 5015 ohm-cm (moderately corrosive to ferrous metals, per Los Angeles County Department of Public Works classification).
- Sulfate content of the soil sample was beneath the detection limit. Soils in this sulfate content range have a negligible effect on concrete (California Building Code).
- Chloride concentration of the soil sample was beneath the detection limit.
- Based on the pH value measured in the soil sample (8.0), acidity of site soils is low.

Additional corrosivity testing of soils from the subgrades of footings and floor slabs should be performed after completion of grading at the project site.
5.10 Soil Shear Strength

Foundations and floor slabs of the proposed library building are anticipated to be founded in compacted fill derived from in-situ site soils. A direct shear test was therefore performed on a test specimen of sandy site soils fabricated in our laboratory to about 90% of Maximum Dry Density. The results of direct shear testing are presented in Appendix B.

5.11 Rippability

The site is underlain primarily by granular alluvium. This material can be ripped with standard grading equipment.

6.0 SEISMIC CONSIDERATIONS

6.1 Introduction

The site is located within the Transverse Ranges Geomorphic Province of southern California. The Transverse Ranges consist of a series of west-trending mountains and intervening valleys, which is contrary to the northwest geomorphic trend that is typical of most of California and reflects the underlying structural (geologic) trend. These ranges are largely the result of north-south compression, which has resulted in west-trending folds and thrust faults. Associated faults in the vicinity of the site include the San Gabriel, Santa Susana, Oak Ridge, Del Valle, and Holser reverse/thrust faults. The January 17, 1994 Northridge (M6.7) earthquake occurred on a south-dipping thrust fault which uplifted the Santa Susana Mountains at least 40 cm.

The southern California region is traversed by the San Andreas fault, which is a transform boundary between the Pacific Plate and the North American Plate. The San Andreas fault is part of a system of northwest-striking, right-lateral faults that are generally historically active, as evidenced by the June 28, 1992 Landers (M7.3) earthquake (see Fault and Earthquake Epicenter Location Map, Figure 3 at end of report). Potentially seismogenic faults in proximity to the site along with associated site-to-fault distances, maximum potential magnitudes, and slip rates are listed in Table 2 at the end of this report.

The southern California region is seismically active and commonly experiences strong ground shaking resulting from earthquakes along active faults. Earthquakes along these
faults are part of a continuous, naturally occurring process, which has contributed to the characteristic landscape of the region.

Common geologic hazards associated with earthquakes include:

1. Ground Rupture
2. Ground Motion
3. Ground Failure

6.2 Ground Rupture

Review of the Alquist-Priolo Earthquake Fault Zone Map for the Newhall Quadrangle, the Seismic Safety Element of the L.A. County General Plan, and the published Geologic Maps referenced at the end of this report indicates that no active or potentially active faults traverse the subject site. Review of the site topography and the aerial photographs listed at the beginning of this report did not reveal any lineaments or other indicators suggestive of faulting at the site. Therefore, the probability of fault-related ground rupture at the site is considered to be very low.

6.3 Ground Motion

6.3.1 Probabilistic Evaluation of Peak Ground Acceleration

Review of the Seismic Hazard Evaluation Report for the Newhall Quadrangle indicates that the Peak Horizontal Ground Acceleration (PHGA) with a 10% chance of exceedance in 50 years for alluvial conditions at the site is about 0.8g. The potential PHGA at this site equal to 2/3 of the peak acceleration with a 2% chance of exceedance in 50 years (as defined in the 2007 CBC) is 0.54g, as calculated using the U.S. Geological Survey program for "Seismic Hazard Curves and Uniform Hazard Response Spectra".

Parameters for the design site response spectrum are provided in the California Building Code Response Spectrum section of this report (Section 8.5).
6.4 Ground Failure

6.4.1 Introduction

Ground failure is a general term for seismically induced, secondary, permanent ground deformation caused by strong ground motion. This includes liquefaction, lateral spreading, ground lurching, seismic settlement of poorly consolidated materials (dynamic densification), differential materials response, sympathetic movement on weak bedding planes or non-causative faults, slope failures, and shattered ridge effects. Potential for earthquake-induced ground failure at the project site is summarized in the following sections of this report.

6.4.2 Liquefaction Potential

According to the Seismic Hazard Map for the Newhall Quadrangle, the project site is not located in a zone in which investigation of liquefaction potential is required. The depth to historic high ground water at the site is greater than 50 ft. The potential for liquefaction and associated seismic settlements and lateral spreading is therefore considered very low.

6.4.3 Potential for Other Modes of Ground Failure

Based on review of the seismic hazard map for the Newhall Quadrangle, the subject site is not located in a zone in which investigation of potential for earthquake-induced landslides is required. Potential at the site for slope failures and shattered ridge effects is considered nonexistent due to the flat nature of the site. Potential at the site for differential materials response and slippage along weak, inclined bedding planes is considered to be negligible because the site is underlain by consistent, flat-lying alluvial deposits.
7.0 GENERAL CONCLUSIONS AND GRADING RECOMMENDATIONS

7.1 Earthworks

All earthworks shall be observed and tested by the Project Geotechnical Engineer, Engineering Geologist, and/or their authorized representatives, in accordance with the recommendations contained herein and in accordance with requirements of the building codes adopted by the City of Santa Clarita.

7.1.1 Site Preparation

The purpose of site preparation is to clear the site of organics (vegetation), topsoil, and unsuitable materials, and to grade the site to provide a firm base for compacted fill, as applicable. All vegetation, topsoil debris, existing disturbed compacted fills, and undocumented artificial fill should be removed from ground surfaces on which compacted fill will be placed.

7.1.2 Removals

The alluvial soil beneath the proposed library building should be removed and replaced with compacted fill as follows. Remove existing soils to a depth of at least 11 ft beneath the perimeter foundation of the library structure, and remove existing soils to a depth of at least 14 ft beneath all other foundations of the library structure. The slope of the transition between the 11-ft deep over-excavation zone and the 14-ft deep removal zone should not exceed 3:1 (h:v). The recommended removal depths should extend outside the building footprint perimeter and interior footings at a 1:1 or flatter projection from the bottom of the footings down to the recommended removal depth. The recommended removals are intended to limit total settlement of footing foundations subjected to (dead plus live) column loads to 1.0 inch, or less. If design (dead plus live) column loads higher than 235 kips will be used for interior locations and/or if design (dead plus live) column loads higher than 145 kips will be used for perimeter locations, the recommended removal depths must be re-evaluated.
If a conventional shallow foundation is used for the proposed signage, the underlying soil should be removed to a depth of at least 3 ft below the bottom of the proposed footing. This removal should extend outside the perimeter of the signage footing at a 1:1 or flatter projection from the bottom of the footing down to the recommended removal depth.

In parking areas, existing soils should be removed and replaced with compacted fill to a depth of at least 3 ft below pavement subgrade, to provide a uniform base for the pavement. This recommendation may be re-evaluated at locations where pervious pavement is proposed to enhance infiltration of surface water runoff.

Removals areas shall be observed by the Geotechnical Engineer, or his authorized representative, prior to placement of compacted fill, to verify the removal of all unsuitable materials. Soft soils identified during field observations should be removed until firm ground is reached and then replaced with compacted fill. The exact depth and extent of necessary removals will be decided in the field during the grading operations, when observation and more location-specific evaluations can be performed.

Any existing utility trench backfill or undocumented artificial fill encountered below the removal depths recommended above should also be removed prior to placement of compacted fill.

7.1.3 Preparation of Removal Bottom Areas

After the ground surface to receive fill has been exposed, it shall be ripped to a minimum depth of six inches, brought to Optimum Moisture Content or above, and thoroughly mixed to obtain a nearly uniform moisture condition and uniform blend of materials, and then compacted to at least 90% of Maximum Dry Density, per ASTM D1557.

7.1.4 Fill Materials

Onsite soils, except debris and organic matter, may be used for compacted fill. Rocks or hard fragments larger than four (4) inches in dimension should not compose more than 25 percent of a fill and/or fill lift. Irreducible rock or similar material larger than eight (8) inches in dimension should not be placed in the fill without approval of the Geotechnical Engineer.
7.1.5 Compaction

All fill material should be placed in lifts not exceeding 8 inches prior to compaction, then water-conditioned to Optimum Moisture Content or higher, then thoroughly mixed to produce a nearly uniform moisture content and uniform blend of materials, and then compacted to at least 90 percent of Maximum Dry Density, per ASTM D1557. Additional recommendations for compaction of fill are provided in the “Recommended Earthwork Specifications” (Appendix D).

7.1.6 Shrinkage

Shrinkage (decrease in volume) of site soils when excavated and replaced as compacted fill is estimated to be on the order of about 12 to 15 percent. This estimate is based on the typical density range of in-situ soils in the upper 15 ft of the soil profile and on an average density following compaction on the order of 92 percent of Maximum Dry Density.

The preceding shrinkage estimate is only an approximation. Actual volume changes from removal and recompaction of site soils will depend on the location and depth from which the recompacted materials are obtained and on the degree of compaction achieved during grading operations. The Supervising Civil Engineer should assume that a fill shortage as large as 20 percent of the excavated volume could occur.

7.2 Drainage

Roof drainage should be collected in gutters and downspouts, and discharged at approved locations away from the proposed library structure.

Water should not be allowed to stand or pond on building pads, parking areas, level graded areas, or constructed slopes. Water that flows onto these areas should be conducted to appropriate discharge locations via non-erodible drainage devices. Drainage devices should be inspected periodically and should be kept clear of debris. Drainage and erosion control should be designed in accordance with the standards set forth in the California Building Code, as adopted by the City of Santa Clarita.

Any modification of the grade of building pad, parking areas, etc. after certification by the project Civil Engineer could adversely affect drainage at the site. Future landscaping,
construction of walkways, planters and walls, etc. must never modify site drainage unless additional measures to enhance drainage (such as area drains, additional grading, etc.) are designed and constructed in compliance with applicable City of Santa Clarita regulations.

7.3 Landscaping

Final grades should slope away from building foundations in order to enable rapid discharge of surface water runoff from the vicinity of the building foundation. An effective watertight barrier should be constructed at locations where landscaping abuts the building in order to prevent water from affecting building foundations. Plants and other landscape vegetation that require large volumes of water should not be planted adjacent to the building foundations.

7.4 Sewage Disposal

It is our understanding that sewage that will be generated at the project site will be discharged into the public sewer system.

7.5 Planters

Planters located adjacent to proposed building should either be sealed or provided with drains that discharge irrigation water well away from footing foundations of the proposed buildings.
8.0 DESIGN CONSIDERATIONS AND FOUNDATION RECOMMENDATIONS

The foundation design recommendations presented below are for a two-story structure with anticipated dead load plus live column loads up to 235 kips. It may be assumed for purposes of preliminary planning that the expansion potential classification of the footing and floor slab subgrades will be very low (see the Expansion Potential of Soils section of this report). Additional testing should be performed at the completion of grading to evaluate the expansion potential of the library building pad and to confirm the foundation design recommendations presented in this report.

8.1 Conventional Shallow Footing Foundations

8.1.1 Assumptions

The recommendations presented herein for design of footing foundations are based on the following assumptions:

- Removals beneath the proposed grade for the library structure: At least 11 ft beneath the perimeter foundations and at least 14 ft at all other locations in the building pad (see the Removals section of this report).
- Bearing foundation material: Certified compacted fill (Cef).
- Expansion potential of subgrades beneath foundations: very low to low.
- Structure type: Two-story steel-frame library building.
- Allowable width of continuous footings: Not less than 15 inches, not more than 5 ft.
- Allowable width of isolated square column footings: Not less than 24 inches. Not more than 11 ft.
- Minimum embedment of footings: 24 inches.

Excavations for footing foundations should be observed by a representative of this firm prior to placement of forms, reinforcement, or concrete, in order to verify that the excavations are embedded into suitable bearing material. The excavations should be moisture-conditioned and free of all loose or sloughed material prior to casting of concrete.
8.1.2 Vertical Bearing Capacity Parameters

- Allowable static plus sustained live load bearing pressure for footing foundations (with minimum required embedment and width): 2,000 psf.
- No increase in allowable bearing pressure for footing embedment deeper than minimum required embedment or for footing width greater than minimum required width.
- Increase to allowable (static plus sustained live load) bearing pressure when considering short-term seismic loads or wind loads: One-third.

8.1.3 Settlement

Total settlement of footing foundations caused by the allowable (static plus sustained live load) bearing pressure recommended herein is estimated to be about 1.0 inch or less at the proposed library location. The associated differential settlement over a horizontal distance of 30 ft or between adjacent footings can be assumed to equal one-half of the total settlement.

8.1.4 Lateral Resistance

- Lateral bearing pressure: 250 psf per ft of footing embedment below lowest adjacent pad grade, to a maximum of 2,500 psf.
- Friction coefficient between bottom of footing foundations and compacted subgrade: 0.25.
- Increase to lateral bearing pressure and friction coefficient for short-term wind and earthquake loadings: One-third.
- Resistance from lateral bearing pressure and from friction between footing foundations and the soil subgrade may be assumed to act concurrently.
- The lateral bearing pressure and friction coefficient values recommended herein are allowable values and include a safety factor of 1.5.
8.2 Foundation Recommendations for Proposed Signage

The sign structure proposed near the northeast corner of the site may be supported either on conventional shallow footing foundations or on cast-in-drill-hole (CIDH) pile foundations.

8.2.1 Conventional Shallow Footing Foundations

8.2.1.1 Assumptions

The recommendations presented herein for design of footing foundations for the proposed signage are based on the following assumptions:

- Embedment of footings: at least 24 inches.
- Removals (and replacement with compacted fill): At least 3 ft beneath bottom of footing (see the Removals section of this report).
- Bearing foundation material: Certified compacted fill (Cef).
- Allowable width of footings: Not less than 24 inches and not more than 8 ft.

Excavations for footing foundations should be observed by a representative of this firm prior to placement of forms, reinforcement, or concrete, in order to verify that the excavations are embedded into suitable bearing material. The excavations should be moisture-conditioned and free of all loose or sloughed material prior to casting of concrete.

8.2.1.2 Vertical Bearing Capacity Parameters

- Allowable static plus wind load bearing pressure for footing foundations (with minimum required embedment and width): 1,500 psf.
- No increase in allowable bearing pressure for footing embedment deeper than minimum required embedment or for footing width greater than minimum required width.
8.2.1.3 Settlement

Total settlement of footing foundations for the proposed signage location caused by the allowable (static plus wind load) bearing pressure recommended herein is estimated to be about 1.0 inch or less.

8.2.1.4 Lateral Resistance

- Lateral bearing pressure: 250 psf per ft of footing embedment below lowest adjacent pad grade, to a maximum of 2,500 psf.
- Friction coefficient between bottom of footing foundations and compacted subgrade: 0.25.
- Increase to lateral bearing pressure and friction coefficient for short-term wind and earthquake loadings: One-third.
- Resistance from lateral bearing pressure and from friction between footing foundations and the soil subgrade may be assumed to act concurrently.
- The lateral bearing pressure and friction coefficient values recommended herein are allowable values and include a safety factor of 1.5.

8.2.2 CIDH Pile Foundations

The diameter of the CIDH piles should be at least 12 inches. The length of the CIDH piles should be at least 15 times the diameter. Spacing of CIDH piles should be at least 3 pile diameters, center to center.

Allowable shaft friction, $\tau_a$, provided by soils on the sides of the CIDH piles may be calculated as follows:

$$\tau_a = 15 \cdot z \ [\text{psf}], \text{ where } z = \text{depth beneath ground surface.}$$

End bearing resistance of the CIDM piles should be neglected.

Lateral displacement of the CIDH piles (and the associated shear and bending moment) may be calculated assuming that the stiffness of the soil on the sides of the piles can be modeled using a coefficient of lateral subgrade reaction, $K_h$, that varies as follows:
$K_h = \frac{f \cdot z}{D}$

where:

\begin{align*}
f &= 40 \text{ kips/ft}^3 \\
z &= \text{depth beneath the ground surface} \\
D &= \text{pile diameter}
\end{align*}

The $f$ value provided above may be used for pile head lateral deflections up to about \( \frac{1}{2} \) inch. If the structural designer provides us with the lateral force and the moment that will be applied to the heads of the proposed CIDH piles by the sign structure, we can calculate lateral displacement, shear, and bending moment in the piles.

Recommendations for construction of the pile foundations will be provided in a subsequent report if needed.

### 8.3 Concrete Slabs-On-Grade

The following recommendations for design of conventional slab-on-grade floors assume that expansion potential of the fill subgrade beneath the floor slabs will be very low.

- Moisture conditioning of slab subgrade: at least 120 percent of Optimum Moisture Content (ASTM D1557) to at least 18 inches depth below pad grade.
- Slab-on-grade thickness: at least 4 inches
- Slab reinforcement: #3 Rebar at 18” each way.
- Concrete used in floor slabs should satisfy the requirements presented in Chapter 19 of the Uniform Building Code.

Floor slabs in which buildup of water vapor is not a concern may be cast directly on the compacted granular material. Where water vapor is a concern, such as in floors that will be covered with rugs, tile, linoleum, etc., floor slabs should be cast over a moisture vapor retarder section. The moisture vapor retarder section should consist of 2-inches of clean sand compacted on the compacted fill surface, overlain by a 10-mil thick Visqueen membrane (or an approved equivalent material), overlain by 2-inches of compacted clean sand. Suitability of sand for the vapor retarder section should be evaluated in our
The membrane must be properly lapped and/or sealed, and sealed around all plumbing structures and other openings. Care should be taken to prevent sharp objects in the subgrade and/or structures from puncturing the membrane.

In order to reduce moisture intrusion from utility trenches beneath slabs on grade, utility trenches should be plugged with lean concrete or concrete slurry at foundation perimeters. The plug should extend under the full width of the footing and should be extended along the utility trench at least 24 inches in the direction of the slab.

Interior floor slabs should be structurally tied to continuous footing foundations, as directed by the Structural Engineer.

Deflections, shears, and moments in slabs-on-grade caused by applied vertical pressures up to 1000 psf may be estimated using a \( k_v \) value of 40 pounds per cubic inch. The coefficient of vertical subgrade reaction, \( k_v \), is defined as the pressure applied by a slab to a subgrade divided by the settlement of the subgrade.

### 8.4 Conventional Retaining Walls

Retaining walls are not currently proposed at the site. However, retaining walls may be required where grades adjacent to perimeter footings are lower than the interior subgrade elevation. Preliminary recommendations for design and construction of these walls are provided below.

#### 8.4.1 Design Parameters

Retaining walls that are restrained against rotation and that will retain granular compacted fill with very low expansion potential should be designed to resist at-rest lateral earth pressures equal to those exerted by an equivalent fluid with a density not less than **60 pcf**. This equivalent fluid density (EFD) value assumes that drainage will be provided behind walls in order to prevent buildup of water pressure.
8.4.2 Construction Considerations

A backdrain system should be provided to prevent development of hydrostatic pressures behind retaining walls. We recommend that drainage behind retaining walls be provided using 3/4-inch crushed aggregate or a granular material that complies with the gradation requirements of Caltrans Class 2 Permeable Material.

At locations where moisture migration through walls is undesirable, the side of the wall in contact with the backfill soils should be waterproofed.

8.4.3 Retaining Wall Backfill

Retaining wall backfill to be certified by this office must be placed in accordance with the recommendations presented in this report and observed and tested by our personnel during placement. Under no circumstances will retaining wall backfill be certified by this office if our recommendations concerning backfill placement are not followed or if our personnel do not observe installation of backdrains and test the backfill during placement.

To prevent the build up of lateral soil pressures in excess of the recommended design pressures, over-compaction of fill behind walls should be avoided by placement of wall backfill in lifts not exceeding six inches in thickness and by compacting each lift with hand-operated or self-propelled compaction equipment that weighs less than 1000 pounds.

8.5 California Building Code Response Spectrum

The following parameters should be used for calculation of the California Building Code (CBC) response spectrum (2007 edition):

- Site Class = D [CBC, Table 1613A.5.2]
- $F_a = 1.0$ [CBC, Table 1613.5.3 (1)]
- $F_v = 1.5$ [CBC, Table 1613.5.3 (2)]
- $S_s = 2.02g$
- $S_1 = 0.69g$

The values for $S_s$ and $S_1$ listed above were obtained using the Seismic Hazard Curves and
Uniform Hazard Response Spectra computer program developed by the United States Geologic Survey (USGS).

8.6 Expansive Soils

Recommendations are provided in this report for mitigation of the effects of potential expansion of fill soils, including recommendations for footing embedment and reinforcement, slab thickness and reinforcement, and moisture conditioning of subgrades beneath footing foundations and slab-on-grade floors. These recommendations depend on the expansion potential that characterizes the subgrade of the building pads. The expansion potential of in-situ soils when removed, mixed, and replaced as compacted fill may be assumed for preliminary design purposes to be very low (per UBC expansion potential classification). The Expansion Index of the building pad soils should be measured at the completion of grading in order to evaluate the appropriate expansion potential classification (and to select minimum parameters for design and construction of footing foundations and concrete slab-on-grade floors).

8.7 Soil Corrosivity

Based on the results of sulfate content testing, corrosivity of soils at the site to concrete is expected to be negligible. Therefore, Type I or II Portland Cement Concrete may be used for concrete structures that will be in contact with site soils.

Based on the results of resistivity testing site soils classify as moderately corrosive to metals. Therefore, buried utilities made of ferrous metals should be protected against soil corrosivity with polyethylene extruded coating, or with tape over primer per AWWA Standard C209 or C203, or with hot-applied coal tar enamel, or as recommended by manufacturers of the utility conduits. Also, metallic pipes that penetrate concrete structures should be surrounded by plastic sleeves, rubber seals, boots, or other dielectric material in order to prevent contact between the pipe and the concrete structure. A corrosion specialist should provide final recommendations for mitigation of potential corrosion of metals by site soils.
8.8 Excavations, Shoring, and Backfilling of Excavations

Excavations deeper than 3.5 ft should conform to safety requirements for excavations set forth in the State Construction Safety Orders, enforced by the State Division of Industrial Safety, CAL-OSHA. Temporary excavations in alluvial materials at the site that are shallower than 14 ft shall be no steeper than 1.5:1 (h:v). If temporary excavations adjacent to Lyons Avenue need to be steeper than 1.5:1 (h:v), shoring or slot-cutting will be necessary at this location. Detailed recommendations for shoring and/or slot-cutting can be provided upon request. Excavations deeper than 14 ft must be approved by the Project Civil Engineer. Excavations that do not comply with these requirements should be shored. Excavation sidewalls in dry soils should be kept moist, but not saturated, at all times.

Bases of excavations or trenches should be firm and unyielding prior to construction of foundations or utilities. On-site materials, other than topsoil or soils with roots or deleterious materials, may be used for backfilling of excavations. Densification (compaction) of bedding and shading backfill by jetting may be used for clean sand provided it has a Sand Equivalent (per ASTM Test Method D2419) of 30 or greater. Recommended specifications for placement of trench backfill are presented in Appendix D.

8.9 Utility Trench Backfill

Utility trench backfill should be compacted to at least 90 percent of Maximum Dry Density (per ASTM D1557). Compaction may be accomplished with a mechanical compaction device and/or by jetting in accordance with specifications for trench backfill presented in Appendix D. If the excavated soils have dried, they should be moisture-conditioned to near Optimum Moisture Content prior to placement and compaction in trenches. No jetting should be performed in utility trenches shallower than 3 ft beneath the subgrade beneath concrete slabs-on-grade, vehicle pavements, or other structures.

8.10 Concrete Flatwork/Hardscape

The soil subgrade beneath hardscape elements, which is anticipated to have very low expansion potential, may be placed directly over soil subgrades. The subgrade beneath hardscape elements should be compacted to at least 90 percent of Maximum Dry Density (per ASTM D1557). Our recommendations assume that potential expansion of materials that will form the subgrade will be less than 3%.
Type II, Portland Cement Concrete may be used for casting of concrete hardscape elements. Crack control joints in hardscape elements should be provided at horizontal intervals not exceeding 8 ft.

Additional recommendations for curbs, gutters, and sidewalks are provided in the Pavement Design Recommendations section of this report.

9.0 TENTATIVE PAVEMENT DESIGN RECOMMENDATIONS

9.1 Asphalt Concrete Pavement Sections

Design of asphalt concrete pavement sections depends primarily on support characteristics (strength) of soil beneath the pavement section and on cumulative traffic loads within the service life of the pavement. Strength of the pavement subgrade is represented by R-Value test data. Traffic loads within service life of a pavement are represented by a Traffic Index (TI) which is calculated based on anticipated traffic loads and on the projected number of load repetitions during the design life of the pavement. The design TI value should be verified by the Project Civil Engineer prior to construction.

Based on results of R-Value testing (see Appendix B) and our judgment regarding variability of site soils, a design R-Value of 40 was selected. Pavement sections (based on the City of Santa Clarita minimum thicknesses for finish asphalt concrete, base asphalt concrete, and aggregate base courses) are provided in the following table. These sections satisfy the minimum requirements of the CALTRANS Flexible pavement design procedure for the design R-Value and a design service life of 20 years.

<table>
<thead>
<tr>
<th>ASSUMED TRAFFIC INDEX</th>
<th>PAVEMENT SECTION (THICKNESS IN INCHES)</th>
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<tbody>
<tr>
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<td>ASPHALT CONCRETE (AC)</td>
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<tr>
<td>4</td>
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<tr>
<td>5</td>
<td>3.5</td>
</tr>
<tr>
<td>6</td>
<td>4.0</td>
</tr>
<tr>
<td>7</td>
<td>4.0</td>
</tr>
</tbody>
</table>

The base course should have an R-Value of at least 78 and should comply with specifications for untreated crushed aggregate base (CAB), crushed miscellaneous base (CMB), or processed miscellaneous base (PMB), as defined Section 200-2 of the current Green Book (Standard Specifications for Public Works Construction), or aggregate base
(AB-Class 2) as defined in Section 605.3 of the current Caltrans Highway Design Manual. The preceding pavement sections provide the minimum thickness of asphalt concrete permitted by the Caltrans design procedure. Alternate designs with greater asphalt thickness and smaller base course thickness are also possible.

The pavement design should be revised, as needed, based on the results of R-Value testing on subgrade soils following grading operations.

### 9.2 Rigid Pavement Sections

Design of rigid pavement sections that will support vehicular traffic may require the use of a value for the modulus of subgrade reaction, $k$. The value $k$ is defined herein as the pressure applied by the rigid pavement divided by the resulting settlement of the compacted soil subgrade and is a function of the subgrade soil type, base thickness, and base type. The following $k$ values may be used for preliminary design purposes to represent the soil subgrade at the project site.

<table>
<thead>
<tr>
<th>Base Type</th>
<th>Base Thickness</th>
<th>Modulus of Subgrade Reaction, $k$ (lb/in$^2$)</th>
<th>Relative Compaction of Soil Subgrade (Upper 12 inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Base</td>
<td>6 inches</td>
<td>165</td>
<td>90 percent</td>
</tr>
<tr>
<td>None</td>
<td>---</td>
<td>130</td>
<td>95 percent</td>
</tr>
</tbody>
</table>

These $k$ values assume that the soil subgrade will be granular, that base materials that will support vehicle pavements will be compacted to at least 95% of Maximum Dry Density (per ASTM D1557) to a depth of at least 12 inches, and that the pavement slab settlement calculated using $k$ will be $\frac{1}{4}$-inch, or less. The modulus of subgrade reaction, $k$ may be revised, as needed, following grading operations.
9.3 Subgrade Support for PCC Curb and Gutter

It is anticipated that on-site subgrade materials at the completion of fine grading will be granular and generally will have negligible to very low expansion potential. Therefore, it may be assumed for purposes of preliminary design that PCC curbs and gutters may be cast directly over a soil subgrade that has been moisture conditioned at least to the percentage of Optimum Moisture Content and compacted at least to 90 percent of Maximum Dry Density (per ASTM D1557).

9.4 Subgrade Support for PCC Sidewalk

Portland Cement Concrete sidewalks that will not support vehicular traffic should be at least 4 inches thick, and as a minimum, should be reinforced at mid-depth with 6x6-W1.4xW1.4 welded wire-fabric reinforcement. Since on-site subgrade materials at the completion of fine grading are anticipated to be granular and generally will have negligible to very low expansion potential, PCC sidewalk slabs may be cast directly over a soil subgrade that has been moisture conditioned at least to the percentage of Optimum Moisture Content and compacted at least to 90 percent of Maximum Dry Density (per ASTM D1557).

9.5 Grading Recommendations for Pavement Construction

9.5.1 General

All grading shall be observed and tested by the Project Geotechnical Engineer, Project Engineering Geologist, and/or their authorized representatives, in accordance with the recommendations contained herein and in accordance with the current Building Code requirements of the City of Santa Clarita.

Immediately before placing base and/or asphalt concrete, any disturbed compacted fill soils (e.g., due to desiccation or over-saturation by rainfall, broken water lines, etc.) must be removed and replaced with compacted fill with the specified density and moisture content.

9.5.2 Subgrade Preparation

The top 6 inches of the sub-grade materials shall be scarified and moisture conditioned
to Optimum Moisture Content, or above. The moisture content shall be brought to the specified percentage by the addition of water, by the addition and blending of dry suitable material, or by the drying of existing material. The subgrade material shall then be compacted to a relative compaction of at least 90 percent of Maximum Dry Density, per ASTM 1557.

During processing of the top 6 inches of backfill in the pavement subgrade, all rocks larger than 3 inches in dimension shall be removed. If unsuitable material is found below the processing depth, it shall be removed and replaced as compacted fill. After compaction and trimming, the subgrade shall be firm, hard, and unyielding.

9.5.3 Placement of Aggregate Base Materials

Aggregate base material, if required, shall be watered as required to facilitate compaction, and spread and compacted in horizontal lifts of approximately equal thickness. The maximum compacted thickness of any aggregate base lift shall not exceed 6 inches. Each lift of aggregate base material shall be compacted to at least 95 percent of Maximum Dry Density, per ASTM D1557.

10.0 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

Allan E. Seward Engineering Geology, Inc should be retained to review all grading and foundation plans and specifications of this project for conformance with the recommendations provided in this report. Prior to approval of Grading and/or Building Plans, review and approval is required by manual signatures and date.

The firm should also retained to perform on-site construction observation and testing to ascertain that conditions observed during grading and construction operations correspond to the findings and conclusions presented herein and that construction generally conforms to the recommendations presented herein. If variations in subsurface soil conditions become evident during construction, the recommendations presented herein may warrant revision. The geotechnical and geological consultants should be commissioned to perform the testing and observation recommended in this report, including the following:

1. Observation of subgrades on which fill will be placed before scarifying and recomping them.
2. Observation and testing of fill placement and compaction.

3. Observation of footing foundation excavations (prior to placement of forms and reinforcement in the excavations and immediately prior to casting of concrete in the excavations) in order to verify required embedment into bearing material that is free of all loose and slough material, neatly trimmed, and moisture-conditioned.

4. Observation of installation of any temporary shoring.

5. Observation and testing of utility trenches beneath and adjacent to structures and paved areas.

6. Observation and testing of slab subgrades (prior to placing of vapor retarder sections).

7. R-Value testing of materials that will form the subgrade of proposed vehicle pavements.

Please notify this office at least 48 hours in advance of any required sampling or observations, so that appropriate personnel can be made available.

11.0 CITY OF SANTA CLARITA STATEMENT

In compliance with Section 18.12.210 of the City of Santa Clarita Building Code (effective January 1, 2008), it is the finding of this firm that the proposed grading, Building Pad, and signage at the subject site, designated on the attached Geologic/Geotechnical Map, will be safe against hazard from landslide, settlement, or slippage and will not affect offsite property, provided that all our recommendations are incorporated in the Plans and implemented during construction.

12.0 GEOLOGIST/GEOTECHNICAL ENGINEER OF RECORD

This report has been prepared assuming that all required geologic and geotechnical field inspections and observations will be performed by Allan E. Seward Engineering Geology, Inc. If these tasks are performed by another party, that party must review this report, assume full responsibility for recommendations contained herein, and assume the title and responsibility of “Geologist/Geotechnical Engineer of Record” for the specific work.

A representative of the Geologist/Geotechnical Engineer of Record shall be present to observe all grading operations. All footing excavations shall be observed by a representative
of the Geologist/Geotechnical Engineer of Record prior to placing steel or casting concrete in the excavations. A report that presents results of these observations and related testing shall be issued at the end of grading operations.

13.0 LIMITATIONS

This report has been prepared by Allan E. Seward Engineering Geology, Inc. for the exclusive use of the City of Santa Clarita and its design consultants for the specific site discussed herein. This report should not be considered transferable. Prior to use by others, this firm must be notified, as additional work may be required to update this report.

In the event that any modification in the location or design of the proposed development is planned, the conclusions and recommendations contained in this report will require a written review by this firm with respect to the planned modifications.

The proposed development is located in southern California, a geologically and tectonically active region, where large magnitude, potentially destructive earthquakes are common. Therefore, ground motions from moderate or large magnitude earthquakes could affect the project site during the design life of the proposed structures.

Typically, faulting is confined to the area adjacent to a known fault. However, absolute assurance against future fault displacement is not possible in tectonically active regions because new faults can form over time as the orientation and magnitude of deformational forces change in the earth's crust. Therefore, the location and magnitude of new ground surface ruptures during a seismic event cannot be anticipated.

The plans and specifications are not intended to depict each and every condition or detail of construction. As the knowledgeable party in the field, the Contractor is in the best position to verify that all construction is completed in a manner, which will provide watertight structures. The Contractor has the sole responsibility for ensuring the watertight integrity of proposed structures.

In performing these professional services, this firm has used the degree of care and skill ordinarily exercised under similar circumstances by reputable geologists and geotechnical engineers practicing in this or similar localities. The data presented in this report are based on results of pertinent field and laboratory testing. It should be recognized that subsurface conditions can vary in time, and laterally, and with depth at a given site. Since the
conclusions and recommendations presented in this report are based on our observations and testing, our conclusions and recommendations are professional opinions and are not meant to be a control of nature. Therefore, we make no other warranty either expressed or implied.

This report may not be duplicated without the written consent of this firm.
This opportunity to be of service is appreciated. If you have any questions regarding this report, please contact us.

Respectfully submitted,

Brian J. Swanson, CEG 2055
Associate Geologist

Kevin P. Callahan, PE 72202
Senior Staff Engineer

Reviewed by:

Eric J. Seward, CEG 2110
Principal Engineering Geologist
Vice President

Martin J. Goodman, GE 2146
Principal Geotechnical Engineer
The following attachments and appendices complete this report.

References
Location Map following page 3
Geologic Overview Map Figure 1
Fault and Earthquake Epicenter Location Map Figure 3
Summary of LAFCD Water Well Data Table 1
Summary of Nearby Faults Table 2

APPENDIX A – Boring Logs
- Boring Logs (HS-1 through HS-6) and Key to Boring Log Symbols

APPENDIX B – Laboratory Testing

APPENDIX C – Infiltration Test Data
- Infiltration Test Reports Figures C1 and C2
- Backhoe Test Pits (TP-1 and TP-2)

APPENDIX D – Earthwork Specifications and Map
Recommended Earthwork Specifications
Recommended Specifications For Placement of Trench Backfill
Drainage and Erosion Control Recommendations
Geologic/Geotechnical Map (In pocket) Plate I

Distribution: (3) City of Santa Clarita (Hardcopies and 1 Electronic copy)
Attn: Mr. Harry Corder
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Dibblee, T.W., Jr., 1996, Geologic map of the Newhall Quadrangle, Los Angeles County, California: Dibblee Geological Foundation Map #DF-56, scale 1:24,000.

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Robson, S.G., 1972, Water-Resources Investigation Using Analog Model Techniques in the Saugus-Newhall Area, Los Angeles County, California.

Treiman, J., 1987, Landslide hazards in the east half of the Newhall Quadrangle, Los Angeles County, California: Landslide Hazard Identification Map #7: California Department of Conservation Division of Mines and Geology Open-File Report 86-16LA; scale 1:24,000.


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   Former 76 Station No. 2008
   24513 San Fernando Road
   Newhall, California
   Dated April 22, 2005

2. Septic Tank Removal Report
   Former 76 Service Station No. 252008
   24513 San Fernando Road
   Newhall, California
   Dated May 31, 2005

3. Addendum to the August 19, 2005 Work Plan for Site Assessment and
   Reply to the November 3, 2005 LACDPW Review Letter
   Former 76 Service Station No. 252008
   24513 San Fernando Road
   Newhall, California
   Dated November 21, 2005

4. Health Risk Assessment and Closure Report
   76 Service Station No. 252008
   24513 San Fernando Road
   Newhall, California
   Dated June 5, 2006

Reports by R.T. Frankian & Associates

1. Phase I Environmental Site Assessment
   CarQuest of Newhall
   Los Angeles County Assessor’s
   24533 and 24535 San Fernando Road
   Santa Clarita, California

2. Phase I Environmental Site Assessment
   24509, 24515, and 24519 Spruce Street
   Los Angeles County Assessor’s
   Santa Clarita, California
   Dated March 5, 2008-JN: 2007-301-51
Source: Dibblee Geological Foundation Map #DF-56, (Dibblee, 1996)

Approximate Scale: 1" = 2,000'

Qa - Quaternary Alluvium
Qog - Quaternary Older Gravels
QTs - Saugus Formation
## SUMMARY OF LACFCD WATER WELL DATA

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<th>Surface Elevation</th>
<th>Historic High</th>
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Note: All depths and elevations in feet

* The historic high depth to ground water recorded between 1959 and 2000 is 102.0 ft

** The historic high depth to ground water recorded between 2000 and 2008 is 33.0 ft
## SUMMARY OF NEARBY FAULTS

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Closest Distance to Site (km)</th>
<th>Maximum Magnitude</th>
<th>Slip Rate (mm/yr)</th>
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<td>Verdugo</td>
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<td>Sample Type</td>
<td>GRAPHIC LOG</td>
<td>USCS SYMBOL</td>
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LABORATORY TESTS

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<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>% fines</th>
<th>Other Tests</th>
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<tr>
<td>10</td>
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<td>Curve= A Shear Test El, Cor</td>
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<td>7</td>
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<tr>
<td>DEPTH (ft)</td>
<td>SAMPLE TYPE</td>
<td>GRAPHIC LOG</td>
<td>USCS SYMBOL</td>
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<tr>
<td>50</td>
<td>25</td>
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TOTAL DEPTH 50' (Elev. 1213')
No Ground Water
Backfill tamped up to 10' below surface; bentonite grout placed up to 1' below surface and capped with bentonite chips and soil
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Blows / 12&quot;</th>
<th>Sample Type</th>
<th>Graph Log</th>
<th>USCS Symbol</th>
<th>Description</th>
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<tr>
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<td>ASPHALT CONCRETE/AGGREGATE BASE: (0 - 20&quot;)</td>
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<td>17</td>
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<td></td>
<td>QUATERNARY ALLUVIUM: QaL (20&quot; - 30&quot;)</td>
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<tr>
<td>10</td>
<td>21</td>
<td>SM</td>
<td></td>
<td></td>
<td>@ 3' Silty SAND; medium dense; damp; yellowish brown</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 6' Poorly graded SAND with silt and gravel; medium dense; damp; light brown</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 9' - loose</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>SM</td>
<td></td>
<td></td>
<td>@ 10' Silty SAND; loose; damp; brown</td>
</tr>
<tr>
<td>15</td>
<td>9</td>
<td>CL-ML</td>
<td></td>
<td></td>
<td>@ 13' Sandy, silty CLAY; firm; damp; brown</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>SM</td>
<td></td>
<td></td>
<td>@ 15' - no recovery; cobbly</td>
</tr>
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<td>@ 17' Silty SAND; medium dense; damp; dark yellowish brown</td>
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<tr>
<td>20</td>
<td>34</td>
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<td>@ 20' Poorly graded SAND with silt; medium dense; damp; yellowish brown</td>
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<tr>
<td>20</td>
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<td></td>
<td></td>
<td>@ 25' sandy, silty CLAY; stiff; damp; dark brown</td>
</tr>
<tr>
<td>30</td>
<td>22</td>
<td>SC-SM</td>
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<td></td>
<td>@ 30' Silty, clayey SAND; medium dense; damp; brown</td>
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<tr>
<td>35</td>
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<td>TOTAL DEPTH 30' (Elev. 1230')</td>
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No Ground Water
Backfill tamped up to 10' below surface; bentonite grout placed up to 1' below surface and capped with bentonite chips and asphalt patch

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Other Tests</th>
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<tr>
<td>8.2</td>
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<tr>
<td>3.6</td>
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<tr>
<td>10.7</td>
<td>121</td>
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## Drill Hole Log

**Client:** The City of Santa Clarita  
**Project:** Old Town Newhall Library  
**City of Santa Clarita, CA**  
**Job No.:** 09-2280 (1)  
**Date:** 6/4/09  
**Logged By:** KPC  
**Drilling Company:** All Ways Drilling (CME 75)  
**Drilled:** 5/14/09  
**Hammer Type:** Hollow-stem auger  
**Hammer Weight:** Automatic  
**Elevation:** 1262'  
**Boring No.:** HS-3

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<tr>
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<th>Sample Type</th>
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<th>Graphic Log</th>
<th>USGS Symbol</th>
<th>Description</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>% Fines</th>
<th>Other Tests</th>
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<td></td>
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<td>Quaternary Alluvium: Qa1 (20' - 35')</td>
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<td>@ 3' Poorly graded SAND with silt; medium dense; damp; dark grayish brown</td>
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</tr>
<tr>
<td></td>
<td>SM</td>
<td>6</td>
<td></td>
<td></td>
<td>@ 6' - loose; brown</td>
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<td></td>
<td></td>
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</tr>
<tr>
<td>2</td>
<td></td>
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<td></td>
<td>SP</td>
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<td>33</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>9</td>
<td></td>
<td></td>
<td>@ 9' Poorly graded SAND with gravel; loose; damp; brown</td>
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<tr>
<td>2.5</td>
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<td>@ 10' Sandy, silty CLAY; firm; damp; yellowish brown</td>
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<tr>
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</tr>
<tr>
<td></td>
<td>SM</td>
<td>15</td>
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<td>@ 15' Silty SAND; medium dense; damp; brown</td>
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<td>25</td>
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<td></td>
<td>SP</td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td>@ 25' Poorly graded SAND; medium dense; damp; light yellowish brown</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td>35</td>
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<td>SM</td>
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<td>2.4 108 4 Consol</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>30</td>
<td></td>
<td></td>
<td>@ 30' Silty SAND; medium dense; damp; brown</td>
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<td>36</td>
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<td>35</td>
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<tr>
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<td>TOTAL DEPTH 35' (Elev. 1227') No Ground Water</td>
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Backfill tamped up to 10' below surface; bentonite grout placed up to 1' below surface and capped with bentonite chips and asphalt patch

---

**Allan E. Seward Engineering Geology, Inc.**

**Sheet 1 of 1**
### DRILL HOLE LOG

**CLIENT:** The City of Santa Clarita  
**PROJECT:** Old Town Newhall Library  
City of Santa Clarita, CA  
**DRILLING COMPANY:** All Ways Drilling (CME 75)  
**DRILLING METHOD:** Hollow-stem auger  
**HORNSMEN TYPE:** Automatic  
**DRIVING WEIGHTS:** 140 lbs  
**DATE:** 6/4/09  
**LOGGED BY:** KPC  
**DRILLED:** 5/15/09  
**HOLE DIA.:** 4.25"/6.0"  
**AVERAGE DROP:** 30"  
**ELEVATION:** 1261'  

**BORING NO.:** HS-4  

<table>
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<tr>
<th>DEPTH (ft)</th>
<th>SAMPLE TYPE</th>
<th>BLOWS / 12&quot;</th>
<th>GRAPHIC LOG</th>
<th>USCS SYMBOL</th>
<th>DESCRIPTION</th>
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<tr>
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<td>ASPHALT; (0 - 3&quot;)</td>
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<td>QUATERNARY ALLUVIUM; Qal (3&quot; - 35&quot;)</td>
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<td>@ 3' Poorly graded SAND with silt; loose; damp; light yellowish brown</td>
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<tr>
<td>5</td>
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<td></td>
<td></td>
<td></td>
<td>@ 9' Poorly graded SAND; medium dense; damp; light yellowish brown</td>
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<tr>
<td>10</td>
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<td>@ 12' Poorly graded SAND with silt; loose; damp; light yellowish brown</td>
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<td>@ 13' Sandy lean CLAY; firm; damp; brown</td>
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<td>SM @ 15' Silty SAND with gravel; medium dense; damp; light brown</td>
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<td>@ 25' Poorly graded SAND with silt; medium dense; damp; light yellowish brown</td>
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<td>@ 30' Silty SAND; dense; damp; brown to yellowish brown</td>
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<td></td>
<td></td>
<td></td>
<td>@ 35' - medium dense; light brown</td>
</tr>
</tbody>
</table>

**TOTAL DEPTH 35' (Elev. 1226')**  
No Ground Water  
Backfill tamped up to 10' below surface; bentonite grout placed up to 1' below surface and capped with bentonite chips and asphalt patch  

**LABORATORY TESTS**  

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>% fines</th>
<th>Other Tests</th>
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<tr>
<td>3.9</td>
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<td>Consol</td>
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<td>5.3</td>
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<td>8.0</td>
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<td>DEPTH (feet)</td>
<td>DESCRIPTION</td>
<td>LABORATORY TESTS</td>
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</tr>
<tr>
<td>3</td>
<td>@ 3' Silty SAND to poorly graded SAND with silt; medium dense; damp; light brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>@ 4' - light yellowish brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>@ 6' - loose</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>@ 9' Silty SAND to poorly graded SAND with silt and gravel; loose; damp; light yellowish brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>SM @ 12' Silty SAND; loose; damp; brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>@ 15' - medium dense; dark brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>SP @ 20' Poorly graded SAND with gravel; medium dense; damp; yellowish brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>@ 25' Poorly graded SAND with silt; medium dense to dense; damp; yellowish brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>SP @ 30' Poorly graded SAND with silt and gravel; medium dense; damp; light yellowish brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>SM @ 35' Silty SAND; medium dense; damp; yellowish brown</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>ML @ 36' Sandy SILT; stiff; damp; yellowish brown</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**LABORATORY TESTS**

- Moisture Content (%)
- Dry Density (pcf)
- % fines
- Other Tests
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample Type</th>
<th>Bows / 12&quot;</th>
<th>Graphic Log</th>
<th>USCS Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>SP</td>
<td>42</td>
<td></td>
<td>SP</td>
<td>@ 40' Poorly graded SAND; medium dense; damp; light yellowish brown to brown</td>
</tr>
</tbody>
</table>

TOTAL DEPTH 40' (Elev. 1221')
No Ground Water
Backfill tamped up to 10' below surface; bentonite grout placed up to 1' below surface and capped with bentonite chips and asphalt patch
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLE TYPE</th>
<th>GRAPHIC LOG</th>
<th>USCS SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td></td>
<td></td>
<td>ASPHALT; (0 - 3&quot;) QUATERNARY ALLUVIUM; Quartzite (3&quot; - 40&quot;) @ 0'-10' Bulk sample</td>
</tr>
<tr>
<td>5</td>
<td>SC</td>
<td></td>
<td>SM</td>
<td>@ 3' Silty, clayey SAND to silty SAND; loose; damp; brown to light brown</td>
</tr>
<tr>
<td>11</td>
<td>SP</td>
<td></td>
<td></td>
<td>@ 6' Poorly graded SAND; loose; damp; light yellowish brown</td>
</tr>
<tr>
<td>22</td>
<td>SP</td>
<td></td>
<td>SM</td>
<td>@ 9' Poorly graded SAND with silt; medium dense; damp; light yellowish brown</td>
</tr>
<tr>
<td>7</td>
<td>CL</td>
<td></td>
<td>ML</td>
<td>@ 12' Sandy, silty CLAY; soft; damp; brown</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td>@ 15' - firm</td>
</tr>
<tr>
<td>12</td>
<td>SC</td>
<td></td>
<td>SM</td>
<td>@ 25' Silty, clayey SAND; medium dense; damp; brown</td>
</tr>
<tr>
<td>30</td>
<td>SP</td>
<td></td>
<td>SM</td>
<td>@ 30' Poorly graded SAND with silt; medium dense; damp; light yellowish brown</td>
</tr>
<tr>
<td>35</td>
<td>SM</td>
<td></td>
<td></td>
<td>@ 35' Silty SAND to poorly graded SAND with silt and gravel; dense; damp; brown to yellowish brown</td>
</tr>
</tbody>
</table>

LABORATORY TESTS

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>% fines</th>
<th>Other Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td></td>
<td></td>
<td>R-Value</td>
</tr>
<tr>
<td>5.0</td>
<td>110</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>16.7</td>
<td>103</td>
<td>52</td>
<td>LL = 24; PI = 4 Consol</td>
</tr>
<tr>
<td>18.5</td>
<td>106</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DEPTH (feet)</td>
<td>SAMPLE TYPE</td>
<td>GRAPHIC LOG</td>
<td>USCS SYMBOL</td>
</tr>
<tr>
<td>-------------</td>
<td>-------------</td>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>40</td>
<td>42</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TOTAL DEPTH 40' (Elev. 1222')
No Ground Water
Backfill tamped up to 10' below surface; bentonite grout placed up to 1'
below surface and capped with bentonite chips and asphalt patch
SOIL CLASSIFICATION CHART

MAJOR DIVISIONS

<table>
<thead>
<tr>
<th>SYMBOLS</th>
<th>TYPICAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAJOR DIVISIONS</td>
<td>SYMBOLS</td>
</tr>
<tr>
<td>GRAVEL AND GRAVELY SOILS</td>
<td>CLEAN GRAVELS (LITTLE OR NO FINES)</td>
</tr>
<tr>
<td></td>
<td>CLEAN SANDS (LITTLE OR NO FINES)</td>
</tr>
<tr>
<td></td>
<td>GRAVELS WITH FINES (APPRICICABLE AMOUNT OF FINES)</td>
</tr>
<tr>
<td></td>
<td>CLAYEY GRAVELS, GRAVEL-CLAY MIXTURES</td>
</tr>
<tr>
<td></td>
<td>SAND AND SANDY SOILS</td>
</tr>
<tr>
<td></td>
<td>SANDS WITH FINES (APPRICICABLE AMOUNT OF FINES)</td>
</tr>
<tr>
<td></td>
<td>SILT AND CLAYS</td>
</tr>
<tr>
<td></td>
<td>INORGANIC SiltS AND CLAYS (LITTLE OR NO NAIL)</td>
</tr>
<tr>
<td></td>
<td>INORGANIC SiltS AND CLAYS (LITTLE OR NO NAIL)</td>
</tr>
<tr>
<td></td>
<td>ORGANIC SiltS AND CLAYS (LITTLE OR NO NAIL)</td>
</tr>
<tr>
<td></td>
<td>ORGANIC SiltS AND CLAYS (LITTLE OR NO NAIL)</td>
</tr>
<tr>
<td></td>
<td>SILT AND CLAYS</td>
</tr>
<tr>
<td></td>
<td>INORGANIC CLAYS (HIGH PLASTICITY, FAT CLAYS)</td>
</tr>
<tr>
<td></td>
<td>ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS</td>
</tr>
<tr>
<td></td>
<td>HIGHLY ORGANIC SOILS</td>
</tr>
</tbody>
</table>

Notes:
(1) Dual USCS symbols, such as SP-SM, denote 5 to 12% of minor constituent.
(2) Dual USCS symbols, such as SM/ML, denote borderline soil classifications.
(3) Subsurface information from boring and test pit logs depict conditions only at the specific locations and dates indicated. Soil conditions and water levels at other locations may differ from conditions at these locations. Also, the conditions at these locations may change with time.
(4) Blow counts on logs are the number of blows to drive the sampler with the weight and drop height indicated on each log.
(5) Split-barrel sampler driving record applies only to hollow-stem auger and rotary-wash borings.
(6) These logs are subject to the limitations, conclusions, and recommendations in this report.

DENSITY OF GRANULAR SOILS

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>SPT BLOWS PER FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>&lt; 4</td>
</tr>
<tr>
<td>Loose</td>
<td>4 - 10</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>11 - 30</td>
</tr>
<tr>
<td>Dense</td>
<td>31 - 50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
</tr>
</tbody>
</table>

STRENGTH OF COHESIVE SOILS

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>SPT BLOWS PER FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt; 2</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
</tr>
<tr>
<td>Firm</td>
<td>5 - 8</td>
</tr>
<tr>
<td>Stiff</td>
<td>9 - 15</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 30</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
</tr>
</tbody>
</table>

SPLIT-BARREL SAMPLER DRIVING RECORD

<table>
<thead>
<tr>
<th>BLOW COUNT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>25 blows drove the sampler 12 inches, after initial 6 inches of seating</td>
</tr>
<tr>
<td>50/7&quot;</td>
<td>50 blows drove the sampler 7 inches, after initial 6 inches of seating</td>
</tr>
<tr>
<td>Ref/3&quot;</td>
<td>50 blows drove the sampler 3 inches during initial 6-inch seating interval</td>
</tr>
</tbody>
</table>

GROUND WATER DATA

| GROUND WATER WHILE DRILLING |
| GROUND WATER AFTER DRILLING |

LABORATORY TESTING ABBREVIATIONS

Plasticity Index PI Compaction Curve Curve=A
Liquid Limit LL Consolidation / Collapse Consol
Hydrometer %-% micron Direct Shear / Shear
Expansion Index EI Reshear / Remold Test
Corrosivity Analysis Cor

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

USCS Soil Classification and Key to Boring Log Symbols
Appendix B
GEOTECHNICAL LABORATORY INVESTIGATION

1. General
   a. The laboratory investigation used current, accepted test procedures of the American Society of Testing and Materials (ASTM) and/or California Test Standards, wherever practical.

   b. Bulk samples, Standard Penetration Test samples, and Modified California ring samples were obtained during the field investigation. Laboratory sample identification is by project name and number, boring number, and depth.

2. Geotechnical Index Parameter Tests

The following Geotechnical Index Parameters tests were performed on samples of in-situ soil collected during the field exploration.

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>NUMBER OF TESTS PERFORMED</th>
<th>TESTING STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>In-Situ Moisture Content</td>
<td>35</td>
<td>ASTM D2216</td>
</tr>
<tr>
<td>In-Situ Dry Density</td>
<td>23</td>
<td>ASTM D2937</td>
</tr>
<tr>
<td>Percent-Finer Than #200 Sieve</td>
<td>26</td>
<td>ASTM D1140</td>
</tr>
<tr>
<td>Particle-Size Analysis of Soils</td>
<td>2</td>
<td>ASTM D422</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td>1</td>
<td>ASTM D4318</td>
</tr>
<tr>
<td>Expansion Index</td>
<td>1</td>
<td>ASTM D4829</td>
</tr>
</tbody>
</table>

The purpose of each test type is briefly described below.

a. In-Situ Moisture Content (ASTM D2216) and Dry Density (ASTM D2937) testing of soils provide an indication of the strength and compressibility of in-situ soils. These data, in conjunction with sampler blow count data, aid in evaluation of soil consistency and in selection of samples for additional laboratory testing. Results of Moisture Content and Dry Density testing are recorded on the Boring Logs in Appendix A.

b. Percent Finer than #200 Sieve (ASTM D1140) testing aids in classification of soils
APPENDIX B

in accordance with the Unified Soil Classification System (USCS). Results of Percent Finer than #200 Sieve testing are recorded on the Boring Logs and Test Pit Logs in Appendix A.

c. Mechanical particle-size analyses of soil fractions larger than 75 microns (No. 200 sieve) were conducted to aid in classification of fill materials in accordance with the Unified Soil Classification System (USCS). Results of the mechanical particle size analyses are presented on Figure B1 in this Appendix.

d. Liquid and Plastic Limits (Atterberg Limits) were evaluated to aid in classification of fine-grained soils in accordance with USCS (by evaluating soil plasticity). Results of the Atterberg Limits testing are presented on Figure B2 in this Appendix.

e. Expansion Index (ASTM D4829) testing provides an index of expansion potential of soils when inundated with water. Expansion Index testing was performed on a bulk sample representative of potential future compacted fill soils. An expansion index of 1 was measured on HS-1 @ 0-10 ft depth, which indicates a very low potential for expansion.

3. Geotechnical Engineering Parameters Tests

The following Geotechnical Engineering Parameters Tests were performed on bulk samples and Modified California ring samples of soil collected at the project site.

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>NUMBER OF TESTS PERFORMED</th>
<th>TESTING STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modified Proctor</td>
<td>1</td>
<td>ASTM D1557</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>1</td>
<td>ASTM D3080</td>
</tr>
<tr>
<td>Consolidation (including Hydro-Consolidation)</td>
<td>4</td>
<td>ASTM D2435</td>
</tr>
<tr>
<td>R-Value</td>
<td>1</td>
<td>ASTM D2844</td>
</tr>
</tbody>
</table>

The purpose of each test type is briefly described below.

a. Modified Proctor (ASTM D1557) testing was performed on a selected bulk sample of site alluvial soils to assess the compacted moisture-density relationship for use during future grading operations. Results of the Modified Proctor test is presented...
b. Direct Shear testing was performed on a remolded test specimen using a displacement-controlled Direct Shear machine. The sample was prepared using material passing the No. 4 sieve compacted to about 90% of Maximum Dry Density. Prior to testing, the sample was inundated and consolidated under a normal pressure ranging from about 1,500 psf to 6,000 psf. Thereafter, the sample was sheared horizontally at a controlled displacement rate until the horizontal shear force reduced to a stable value. Results of the Direct Shear testing, including interpreted peak strength and residual shear strength parameters, are recorded on Figure B4 in this Appendix.

c. One-dimensional Consolidation (ASTM D 2435) testing was performed on selected ring samples to assess in-situ soil compressibility at applied vertical pressures ranging from 200 psf to 19,200 psf and to assess hydro-consolidation potential associated with inundation of site soils by water. The soil samples were allowed to consolidate under each applied pressure until the rate of settlement was less than 0.0002 inches per hour. Hydro-consolidation potential of granular site soils was assessed by inundating the samples with water after consolidation was completed under a pressure which approximately equals the overburden pressure at the sample depth. The percent compression due to the addition of water is referred to as “Hydro-consolidation”. Results of the consolidation testing are presented on Figures B5.1 to B5.4 in this Appendix.

d. R-Value\(^1\) testing was performed to evaluate potential support characteristics of soil subgrades that will support roadway and parking lot pavements. The R-Value test method consists of two separate measurements:

1. The R-Value (or resistance value) determines the thickness of cover or structural section required to prevent plastic deformation of the soil subgrade under imposed wheel loads.

2. The expansion pressure test determines the thickness or weight of cover required to maintain the compaction of the soil.

\(^1\) R-Value testing was performed by LaBelle-Marvin in Santa Ana, California.
The design R-Value is determined from the moisture content and density at which these two thicknesses are equal. With granular non-expansive soils, the design R-Value is determined for a density considered to be equivalent to the density that will be obtained by normal construction compaction. This density value is obtained from the exudation pressure data. Results of the R-Value testing are presented on Figures B6.1 and B6.2 in this Appendix.

3. Corrosion Tests

The following corrosivity tests were performed on a selected bulk sample of alluvial soil considered to be representative of future compacted fill at the project site.

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>NUMBER OF TESTS PERFORMED</th>
<th>TESTING STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sulfate-Content</td>
<td>1</td>
<td>California Test Method 417</td>
</tr>
<tr>
<td>Chloride-Content</td>
<td>1</td>
<td>California Test Method 422</td>
</tr>
<tr>
<td>Resistivity</td>
<td>1</td>
<td>California Test Method 532</td>
</tr>
<tr>
<td>pH</td>
<td>1</td>
<td>--</td>
</tr>
</tbody>
</table>

Soluble Sulfates Content, Chloride Content, Resistivity, and pH tests were performed to evaluate corrosivity of soil from the project site to concrete, ferrous metals, and non-ferrous metals. Results of the testing are presented in Table B1 in this Appendix.

The following attachments complete this Appendix.

LABORATORY TEST RESULTS

- Corrosivity Testing Summary Table B1
- Particle Size Distribution Test Report Figure B1
- Atterberg Limits Test Report Figure B2
- Compaction Test Report Figure B3
- Direct Shear Test Report Figure B4
- Consolidation Test Reports Figures B5.1 to B5.4
- R-Value Test Data Sheets Figures B6.1 and B6.2
## APPENDIX B
### CORROSIVITY TESTING SUMMARY

<table>
<thead>
<tr>
<th>Source</th>
<th>Depth (FT)</th>
<th>USCS Classification</th>
<th>Resistivity</th>
<th>USCS Classification</th>
<th>Corrosion Characteristics(^2)</th>
<th>PH</th>
<th>Chloride Cl (PPM)</th>
<th>Sulfate SO(_4) (%)</th>
<th>Concrete exposure to Sulfate(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-1</td>
<td>0-10</td>
<td>SP-SM</td>
<td>5015</td>
<td>Moderately Corrosive</td>
<td>8.0</td>
<td>ND</td>
<td>ND</td>
<td>ND</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

ND = Not Detected

\(^2\) Per County of Los Angeles classification
\(^3\) Per 1997 UBC – Table 10-A-4
COMPACTATION TEST REPORT

Curve No. A

Test Specification:
ASTM D 1557-02 Method A Modified
ASTM D 4718-87 Oversize Corr. Applied to

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: 0.03333 cu. ft.
Test Performed on Material Passing #4 Sieve

Soil Data
NM Sp.G. 2.65
LL PI
%>#4 15.1 %<#200 9.8
USCS SP-SM AASHTO

TESTING DATA

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>WM + WS</td>
<td>9.2</td>
<td>9.4</td>
<td>9.5</td>
<td>9.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WM</td>
<td>4.8</td>
<td>4.8</td>
<td>4.8</td>
<td>4.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WW + T #1</td>
<td>447.51</td>
<td>547.70</td>
<td>698.06</td>
<td>496.13</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WD + T #1</td>
<td>429.53</td>
<td>516.15</td>
<td>646.07</td>
<td>453.56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TARE #1</td>
<td>31.26</td>
<td>31.63</td>
<td>30.52</td>
<td>31.61</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WW + T #2</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WD + T #2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TARE #2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MOISTURE</td>
<td>3.8</td>
<td>5.5</td>
<td>7.2</td>
<td>8.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DRY DENSITY</td>
<td>131.7</td>
<td>133.4</td>
<td>133.6</td>
<td>132.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ROCK CORRECTED TEST RESULTS

Maximum dry density = 134.0 pcf
Optimum moisture = 6.5 %

Project No. 09-2280 (1)  Client: The City of Santa Clarita
Project: Old Town Newhall Library
City of Santa Clarita, CA
Source of Sample: HS-1  Depth: 0-10'

Material Description
Poorly graded sand with silt and gravel (SP-SM)

Remarks:
Mixture of on-site alluvial soils

ALLAN E. SEWARD ENGINEERING GEOLOGY, INC.
Valencia, California

Figure B3
Sample Type: Remold
Description: Poorly graded sand with silt and gravel (SP-SM)

Assumed Specific Gravity = 2.65
Remarks: Remolded to about 90% of MDD
Mixture of on-site alluvial soils
% Fines = 9.8

Sample No. | 1 | 2 | 3
--- | --- | --- | ---
Water Content, % | 7.2 | 7.2 | 7.4
Dry Density,pcf | 119.2 | 119.2 | 118.8
Saturation, % | 49.6 | 49.3 | 50.0
Void Ratio | 0.3873 | 0.3878 | 0.3920
Diameter, in. | 2.42 | 2.42 | 2.42
Height, in. | 1.00 | 1.00 | 1.00

At Test

Water Content, % | 13.7 | 12.5 | 13.0
Dry Density,pcf | 121.4 | 124.2 | 123.0
Saturation, % | 100.0 | 100.0 | 100.0
Void Ratio | 0.3625 | 0.3322 | 0.3452
Diameter, in. | 2.42 | 2.42 | 2.42
Height, in. | 0.98 | 0.96 | 0.97

Normal Stress, psf | 1500 | 3000 | 6000
Peak Stress, psf | 1296 | 2748 | 4692
Displacement, in. | 0.06 | 0.07 | 0.07
Residual Stress, psf | 900 | 1872 | 3588
Displacement, in. | 0.49 | 0.50 | 0.42
Strain rate, %/min. | 0.01 | 0.01 | 0.01

Client: The City of Santa Clarita
Project: Old Town Newhall Library
City of Santa Clarita, CA
Source of Sample: HS-1 Depth: 0-10'

Proj. No.: 09-2280 (1) Date Sampled:

DIRECT SHEAR TEST REPORT
ALLAN E. SEWARD ENGINEERING GEOLOGY, INC.
CONSOLIDATION TEST REPORT

SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th></th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT, (%)</th>
<th>SATURATION (%)</th>
<th>HEIGHT (in.)</th>
<th>VOID RATIO</th>
<th>SPECIFIC GRAVITY</th>
<th>Cc</th>
<th>P'0 (ksf)</th>
<th>Pc (ksf)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL</td>
<td>105.5</td>
<td>3.3</td>
<td>15.4</td>
<td>1.000</td>
<td>0.569</td>
<td>2.65</td>
<td>0.04</td>
<td>3.00</td>
<td>1.61</td>
<td>SP</td>
</tr>
<tr>
<td>FINAL</td>
<td>108.3</td>
<td>18.2</td>
<td>91.4</td>
<td>0.973</td>
<td>0.527</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: HS-3
Material Description: Poorly graded SAND
Remarks:

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Valencia, California

Client: The City of Santa Clarita
Project: Old Town Newhall Library
City of Santa Clarita, CA
Job No.: 09-2280 (1)

Elev./Depth: 25'
Figure B5.1
CONsolidation Test Report

Summary of Test Results

<table>
<thead>
<tr>
<th></th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SATURATION (%)</th>
<th>HEIGHT (in.)</th>
<th>VOID RATIO</th>
<th>SPECIFIC GRAVITY</th>
<th>Cc</th>
<th>P'0 (ksf)</th>
<th>Pc (ksf)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL</td>
<td>107.5</td>
<td>4.0</td>
<td>19.9</td>
<td>1.000</td>
<td>0.539</td>
<td>2.65</td>
<td>0.05</td>
<td>1.08</td>
<td>2.78</td>
<td>SP</td>
</tr>
<tr>
<td>FINAL</td>
<td>110.9</td>
<td>18.2</td>
<td>98.1</td>
<td>0.969</td>
<td>0.491</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: HS-4
Material Description: Poorly graded SAND
Remarks:

Elev./Depth: 9'

Allan E. Seward
Engineering Geology, Inc.
Valencia, California

Client: The City of Santa Clarita
Project: Old Town Newhall Library
City of Santa Clarita, CA
Job No.: 09-2280 (1)
CONSOLIDATION TEST REPORT

SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT, (%)</th>
<th>SATURATION (%)</th>
<th>HEIGHT (in.)</th>
<th>VOID RATIO</th>
<th>SPECIFIC GRAVITY</th>
<th>Cc</th>
<th>P'0 (ksf)</th>
<th>P'c (ksf)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL</td>
<td>103.9</td>
<td>4.2</td>
<td>18.6</td>
<td>1.000</td>
<td>0.592</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FINAL</td>
<td>108.1</td>
<td>17.8</td>
<td>88.9</td>
<td>0.961</td>
<td>0.530</td>
<td>2.65</td>
<td>0.05</td>
<td>4.80</td>
<td>2.45</td>
</tr>
</tbody>
</table>

Source: HS-5
Material Description: Poorly graded SAND
Remarks:

Elev./Depth: 40'

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Valencia, California

Client: The City of Santa Clarita
Project: Old Town Newhall Library
City of Santa Clarita, CA
Job No.: 09-2280 (1)

Figure B5.3
CONSOLIDATION TEST REPORT

SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th></th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>SATURATION (%)</th>
<th>HEIGHT (in.)</th>
<th>VOID RATIO</th>
<th>SPECIFIC GRAVITY</th>
<th>Cc</th>
<th>P'_o (ksf)</th>
<th>P_c (ksf)</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL</td>
<td>106.1</td>
<td>17.8</td>
<td>81.5</td>
<td>1.000</td>
<td>0.589</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FINAL</td>
<td>110.1</td>
<td>19.0</td>
<td>96.8</td>
<td>0.964</td>
<td>0.531</td>
<td>2.70</td>
<td>0.06</td>
<td>1.44</td>
<td>2.98</td>
<td>CL-ML</td>
</tr>
</tbody>
</table>

Source: HS-6
Material Description: Sandy, silty CLAY
Remarks:

Elev./Depth: 12'

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Valencia, California

Client: The City of Santa Clarita
Project: Old Town Newhall Library
City of Santa Clarita, CA
Job No.: 09-2280 (1)
Figure B5.4
**R VALUE DATA SHEET**

P.N. 09-2280(1)
Old Town Newhall Lit

PROJECT NUMBER 36264  BORING NUMBER: HS-6 @ 0'-10'

SAMPLE DESCRIPTION: Brown Slightly Gravelly Silty Sand

<table>
<thead>
<tr>
<th>Item</th>
<th>SPECIMEN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
</tr>
<tr>
<td>Mold Number</td>
<td>1</td>
</tr>
<tr>
<td>Water added, grams</td>
<td>33</td>
</tr>
<tr>
<td>Initial Test Water, %</td>
<td>9.6</td>
</tr>
<tr>
<td>Compact Gage Pressure, psi</td>
<td>325</td>
</tr>
<tr>
<td>Exudation Pressure, psi</td>
<td>117</td>
</tr>
<tr>
<td>Height Sample, Inches</td>
<td>2.65</td>
</tr>
<tr>
<td>Gross Weight Mold, grams</td>
<td>3166</td>
</tr>
<tr>
<td>Tare Weight Mold, grams</td>
<td>1965</td>
</tr>
<tr>
<td>Sample Wet Weight, grams</td>
<td>1201</td>
</tr>
<tr>
<td>Expansion, Inches x 10exp-4</td>
<td>0</td>
</tr>
<tr>
<td>Stability 2,000 lbs (180psi)</td>
<td>24 / 43</td>
</tr>
<tr>
<td>Turns Displacement</td>
<td>5.30</td>
</tr>
<tr>
<td>R-Value Uncorrected</td>
<td>56</td>
</tr>
<tr>
<td>R-Value Corrected</td>
<td>60</td>
</tr>
<tr>
<td>Dry Density,pcf</td>
<td>125.3</td>
</tr>
</tbody>
</table>

**DESIGN CALCULATION DATA**

<table>
<thead>
<tr>
<th>Traffic Index</th>
<th>Assumed:</th>
<th>4.0</th>
<th>4.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.E. by Stability</td>
<td>0.41</td>
<td>0.20</td>
<td>0.28</td>
</tr>
<tr>
<td>G. E. by Expansion</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Equilibrium R-Value**

**by EXUDATION**

| Gf = 1.25 |
| 0.0% Retained on |

REMARKS: 3/4" Sieve.

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

Figure B6.1
R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 36264
BORING NO. H.S.-680'-10'
DATE 5-22-09
TRAFFIC INDEX ASSUME 4.0
R-VALUE BY EXUDATION 77
R-VALUE BY EXPANSION

% MOISTURE AT FABRICATION

COVER THICKNESS BY EXPANSION, FT.

R-VALUE vs. EXUD. PRES.
EXUD. T vs. EXPAN. T
T by EXUDATION
T by EXPANSION

REMARKS

LaBelle - Marvin
PROFESSIONAL PAVEMENT ENGINEERING

Figure B6.2
Appendix C
APPENDIX C

INFLTRATION TEST DATA

I. INTRODUCTION

The intrinsic water infiltration rate for native alluvial soils at shallow depths below the site was measured in the field using a double ring infiltrometer, in accordance with ASTM D3385 Test Method. The infiltrometer testing was conducted on 5/13/09 at two locations (test pits TP-1 and TP-2) selected by Mr. Harry Corder. This testing was performed to provide data to assist in the design of a Low Impact Development (LID) system for infiltration of surface water run off at the site.

II. PROCEDURE

Test Pits TP-1 and TP-2 are located on Spruce Street, as shown on the Geotechnical Map (Plate I). A backhoe was used to excavate TP-1 and TP-2 through the street pavement section to depths of about 1.5 ft and 3 ft below the existing street elevation, respectively. The rate of infiltration was then measured at each location using a Gilson HM-128 double ring infiltrometer.

A level surface was carefully prepared with minimal disturbance at the test elevation in each test pit to allow consistent water infiltration within each ring. The outer and inner rings were then hydraulically pressed into the soil using the backhoe arm until the desired depth of embedment was achieved. Any disturbed soil adjacent to the rings was gently tamped to provide a uniform surface. Embedment depths of each ring are noted on the attached Infiltration Test Reports. Tap water from an on-site faucet was poured into the inner ring and annular space (i.e., the area between the inner and outer rings) using a water hose and pail. Splash guards were placed on the soil surface to protect against erosion when the initial water supply was poured into the rings. Two calibrated plastic cylinders (with 5,000 mL and 13,000 mL capacity) were used to control the flow of water into the inner ring and annular space, respectively, and constant-level float valves were used to maintain a constant water level. After a constant water level was achieved, the float valves were adjusted to maintain the same water level in both the inner ring and annular space throughout the test. During the test, the volume of water required to maintain a constant head in the inner ring and annular space was recorded at selected intervals by measuring the change in water level in the graduated cylinders. The temperature of the water at each interval, the pH of the water, and the initial soil temperature were also recorded. The test was conducted until a relatively constant rate of infiltration was achieved.
Following completion of the infiltration test, each test pit was excavated an additional 3 ft in order to document the soil profile and any potential low permeability layers below the test surface. The moisture pattern of the soil beneath the infiltration rings was also observed. The subsurface profile in each test pit was logged (see attached logs) and samples were collected for classification testing.

III. RESULTS

The volume of water used during each time interval was converted into an incremental infiltration velocity for both the inner ring and the annular space based on the calculated areas. Ground and water temperatures, incremental volume measurements, and a plot of the incremental infiltration rate versus total elapsed time are provided on the Infiltration Test Reports (Figures C1 and C2). The rates of infiltration for the inner ring and annular space for each test location are shown in the following table.

<table>
<thead>
<tr>
<th>Test Pit</th>
<th>Depth of Test Surface Below Spruce Street, FT</th>
<th>Measured Infiltration Rates, cm/hour</th>
<th>Design Rate, cm/hour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Inner Ring</td>
<td>Annular Space</td>
</tr>
<tr>
<td>TP-1</td>
<td>1.5</td>
<td>17</td>
<td>22</td>
</tr>
<tr>
<td>TP-2</td>
<td>3.0</td>
<td>116</td>
<td>139</td>
</tr>
</tbody>
</table>

Infiltration rates measured in the inner rings were 20% to 30% slower than those measured in the annular space. This condition reflects divergent flow of water away from the outer ring, resulting in more rapid infiltration from the Annular Space. The infiltration rates obtained for the inner rings are appropriate for the design of areal type LID measures (BMP’s) such as porous pavement and infiltration basins.
APPENDIX C

Excavation of the test pits following the infiltration test exposed poorly graded sand and gravelly sand with interbeds of fine silty sand to a depth of about 2.5 ft. Poorly graded sand with gravelly sand interbeds were observed below a depth of 2.5 ft. The fines content (i.e. percent passing the no. 200 sieve size) of the poorly graded sand and silty sand were 1.4 and 38.9 percent, respectively. It can be inferred from this data that the higher rate of infiltration in TP-2 (relative to TP-1) is due to the absence of fine-grained soil layers below the test elevation. The material below the test surface in each test pit showed no discernable wetting front. This is consistent with the granular, permeable nature of the alluvium. All excess water had infiltrated by the time the excavation below the test surface was completed. Additional field testing may be required to assess the rate of infiltration at specific locations and elevations, depending on the details of the final design.

The following attachments complete this Appendix.

- Infiltration Test Reports
- Backhoe Test Pit Logs (TP-1 and TP-2)

Figures C1 and C2
INfiltration Test Report

Client: City of Santa Clarita
Project: Old Town Newhall Library
Project No.: 09-2280
Tested By: KPC/BJS
Test Date: 5/13/09

<table>
<thead>
<tr>
<th>Test Location: TP-1 @ 1.5 ft</th>
<th>Depth to Water Table: &gt; 50 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather: 31°C, clear</td>
<td>Soil Temp: 26.0°C at depth 2.5 ft</td>
</tr>
<tr>
<td>Average Water Temp: 29.7°C</td>
<td>pH 7.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Inner Ring</th>
<th>Area (cm²)</th>
<th>Depth to Lq.* (cm)</th>
<th>Volume (cm³)</th>
<th>Penetration of Rings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>700</td>
<td>7.6</td>
<td>5,320</td>
<td>Inner: 4&quot;</td>
</tr>
<tr>
<td>Annular Space</td>
<td>2128</td>
<td>7.6</td>
<td>16,173</td>
<td>Outer: 4.5&quot;</td>
</tr>
</tbody>
</table>

*Liquid level maintained using a constant-level float valve.

Figure C1
**INfiltration Test Report**

**Client:** City of Santa Clarita  
**Test Location:** TP-2 @ 3 ft  
**Project:** Old Town Newhall Library  
**Depth to Water Table:** > 50 ft  
**Project No.:** 09-2280  
**Weather:** 27° C, clear  
**Tested By:** KPC/BJS  
**Soil Temp:** 26.2° C at depth 4 ft  
**Test Date:** 5/13/09  
**Average Water Temp:** 25.2° C  
**pH:** 7.9

<table>
<thead>
<tr>
<th>Area (cm²)</th>
<th>Depth to Liq. * (cm)</th>
<th>Volume (cm³)</th>
<th>Penetration of Rings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inner Ring</td>
<td>700</td>
<td>6.4</td>
<td>4,480</td>
</tr>
<tr>
<td>Annular Space</td>
<td>2128</td>
<td>6.4</td>
<td>13,620</td>
</tr>
</tbody>
</table>

*Liquid level maintained using a constant-level float valve.

**Figure C2**
<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Geologic Type</th>
<th>Description</th>
</tr>
</thead>
</table>
| 0           | Asphalt       | 0-5"

**QUERNARY ALLUVIUM**: Qal (5" - 4.5")
- @ 5" Interbedded silty, fine-grained SAND, poorly graded SAND, and gravelly SAND; medium dense; moist; light yellowish brown
- @ 2' Sandy SILT; firm; damp to moist; brown
- @ 2' 4" Poorly graded SAND; medium dense; damp to moist; light yellowish brown

**Subhorizontal**

**COMMENTS:**
- Infiltration test conducted at a depth of 1.5 ft below the top of asphalt.
- Trench was subsequently deepened for purposes of logging.
- Small bag samples at 2' and 3'.
- Soil is moist due to infiltration test (wetting front not discernable).
- Test pit measures 55" wide by 58" long.

**TOTAL DEPTH:** 4.5 feet

**LABORATORY TESTS**
- Moisture Content (%)
- Dry Density (pcf)
- Other Tests

**SCALE:** 1 inch = 5 feet

**Surface Level of Infiltration Testing**

**No Ground Water**

**No Caving**
**TRENCH LOG NO. TP-2**

**CLIENT:** The City of Santa Clarita  
**PROJECT:** Old Town Newhall Library  
City of Santa Clarita, CA  
**DATE:** 6/4/09  
**LOGGED BY:** BJS  
**EXCAVATED:** 5/13/09  
**ELEVATION:** 1261±

**EXCAVATION METHOD:** Rubber-tired Backhoe

<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>SAMPLE TYPE</th>
<th>USCS SYMBOL</th>
<th>DESCRIPTION</th>
<th>ATTITUDES</th>
<th>LABORATORY TESTS</th>
</tr>
</thead>
</table>
| 0          | SP          |             | ASPHALT; (0 - 4")  
QUATERNARY ALLUVIUM; Qa1 (4" - 6")  
@ 4" Poorly graded SAND and gravelly SAND with 2- to 4-inch thick interbeds of silty, clayey SAND; loose to medium dense; damp; yellowish gray to light yellowish brown; silty sand beds vary in thickness due to erosion by overlying gravelly sand  
@ 2.5' Poorly graded SAND with gravelly SAND interbeds; medium dense; moist; yellowish brown | Subhorizontal | % fines=38.9  
% fines=1.4 |

**COMMENTS:**  
Infiltration test conducted at a depth of 3 ft below the top of asphalt.  
Trench was subsequently deepened for purposes of logging.  
Small bag samples at 2.5' and 3'.  
Soil is moist due to infiltration test (wetting front not discernable).  
Test pit measures 50" long by 43" wide.

**TOTAL DEPTH:** 6 feet  
**SCALE:** 1 inch = 5 feet

---

No Ground Water  
No Caving
APPENDIX D

RECOMMENDED EARTHWORK SPECIFICATIONS

The following specifications are recommended to provide a basis for quality control during the placement of compacted fill or backfill, as applicable.

1. Areas on which compacted fill will be placed shall be observed by Allan E. Seward Engineering Geology, Inc. (AESEGI) prior to the placement of fill.

2. All drainage devices shall be properly installed and observed by AESEGI and/or the owner’s representative(s) prior to placement of backfill.

3. Fill soils shall consist of imported soils or on-site soils which are free of organics, cobbles, and deleterious material, provided that each material is approved by AESEGI. AESEGI shall evaluate and/or test the import material for its conformance with the report recommendations prior to its delivery to the site. The contractor shall notify AESEGI at least 72 hours prior to importing material to the site.

4. The thickness of the controlled lifts in which Fill is placed shall be compatible with the type of compaction equipment used. The fill materials shall be brought to Optimum Moisture Content or above, thoroughly mixed during spreading to obtain a near uniform water content and a uniform blend of materials, and then placed in lifts with a pre-compaction thickness not exceeding 8 inches. Each lift shall be compacted to the specified percentage of Maximum Dry Density determined in accordance with ASTM Test Method D1557. Density testing shall be performed by AESEGI to verify relative compaction. The contractor shall provide proper access and level areas for testing.

5. Rocks or rock fragments less than eight (8) inches in the largest dimension may be utilized in the fill, provided they are not placed in concentrated pockets. However, rocks larger than four (4) inches in dimension shall not be placed within three (3) ft of finish grade.

6. Rocks greater than eight (8) inches in largest dimension shall be taken offsite, or placed in areas designated by the Geotechnical Engineer to be suitable for rock disposal.

7. Where space limitations do not allow for conventional fill compaction operations, special backfill materials and procedures may be required. Pea gravel or other select fill can be used in areas of limited space. A sand and Portland Cement slurry (2 sacks per cubic-yard of slurry mix) shall be used in limited space areas for shallow backfill near
APPENDIX D

final pad grade, and pea gravel shall be placed in deeper backfill near drainage systems.

8. AESEGI shall observe the placement of fill and conduct in-place field density tests on the compacted fill in order to check adequacy of in-situ water content and relative compaction. Where measured in-situ density of compacted fill soil is lower than the required relative compaction, the soil shall be water-conditioned and recompacted until adequate relative compaction is achieved.

9. The Contractor shall achieve with the specified relative compaction out to the finish slope face of fill slopes, buttresses, and stabilization fills, as set forth in the specifications for compacted fill. This may be achieved either by overbuilding the slope and cutting back as necessary, by direct compaction of the slope face with suitable equipment, or by other procedures which produce the required result.

10. Any abandoned underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or others not discovered prior to grading are to be removed or treated to the satisfaction of the Geotechnical Engineer and/or the controlling agency for the project.

11. The Contractor shall have suitable and sufficient equipment during a particular operation to handle the volume of fill being placed. When necessary, fill placement equipment shall be shut down temporarily in order to permit proper compaction of fill, correction of deficient areas, or to facilitate required field testing.

12. The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications.

13. Final reports shall be submitted after completion of earthwork and after the Geotechnical Engineer and Engineering Geologist have finished their observations of the work. No additional excavation or filling shall be performed without prior notification to the Geotechnical Engineer and/or Engineering Geologist.

14. Whenever the words “supervision”, “inspection”, or “control” are used, they shall mean observation of the work and/or testing of the compacted fill by AESEGI to assess whether substantial compliance with plans, specifications and design concepts has been achieved. However, these words do not refer to direction by AESEGI of the actual work of the Contractor or the Contractor’s workers.
APPENDIX D

RECOMMENDED SPECIFICATIONS
FOR PLACEMENT OF TRENCH BACKFILL

1. Trench excavations in which backfill will be placed shall be free of trash, debris or other deleterious materials prior to backfill placement, and shall be observed by a representative of Allan E. Seward Engineering Geology, Inc. (AESEGI).

2. Except as stipulated herein, soils obtained from the excavation may be used as backfill if they are free of organics and other deleterious materials.

3. Rocks generated by trench excavation operations that do not exceed three (3) inches in largest dimension may be used as trench backfill material. However, material larger than 3-inches in dimension may not be placed within 12 inches of the top of pipes. No more than 30 percent of the backfill volume shall contain particles larger than 1-½ inches in dimension, and particles larger than 1-½ inches in dimension shall be well mixed with finer soil.

4. Soils (other than aggregates) with a Sand Equivalent (SE) greater than or equal to 30 (as determined by ASTM Standard Test Method D2419) or other soils authorized by the Geotechnical Engineer or his representative in the field, may be used for bedding and shading material in pipe trenches.

5. Trench backfill other than bedding and shading shall be compacted by mechanical methods as tamping sheepsfoot, vibrating or pneumatic rollers, or other mechanical tampers to achieve the specified density. The backfill materials shall be brought to Optimum Moisture Content or above, thoroughly mixed during spreading to obtain a near uniform water content and uniform blend of materials, and then placed in horizontal lifts with a pre-compaction thickness not exceeding 8 inches. Trench backfills shall be compacted to the specified percentage of Maximum Dry Density determined in accordance with ASTM Test Method D1557.

6. The Contractor shall select the equipment and procedure for achieving the specified density without damage to the pipe, the adjacent ground, existing improvements, or completed work.

7. Observations and field tests shall be performed during construction by AESEGI to confirm that the required degree of compaction has been achieved. Where achieved compaction is less than that specified value, the water content shall be adjusted as
APPENDIX D

necessary and additional compactive effort shall be made until the specified compaction is achieved. Field density tests may be omitted at the discretion of the Geotechnical Engineer or his representative in the field.

8. Whenever, in the opinion of AESEGI or the Owner’s Representative(s), an unstable condition is being created either by cutting or filling, the work shall not proceed until an investigation has been made and the excavation plan has been revised, if deemed necessary.

9. Fill material shall not be placed, spread, or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by AESEGI indicate the water content and density of the fill materials and of the fill surface over which they are to be compacted satisfy the requirements of the specifications.

10. Whenever the words “supervision”, “inspection”, or “control” are used, they shall mean observation of the work and/or testing of the compacted fill by AESEGI to assess whether substantial compliance with plans, specifications and design concepts has been achieved.
APPENDIX D

DRAINAGE AND EROSION CONTROL RECOMMENDATIONS

Slopes and pads for this project shall be designed to direct surficial runoff away from structures and to reduce water-induced surficial erosion/sloughing. Permanent erosion control measures shall be initiated immediately following completion of grading. All constructed slopes will undergo some erosion when subjected to sustained water influx. To maintain appropriate long-term drainage and erosion control, the following points shall be incorporated in slope protection, landscaping, irrigation, and modifications to slopes, pads and structures:

1. All interceptor ditches, drainage terraces, down-drains and any other drainage devices shall be maintained and kept clear of debris. A qualified Engineer should review any proposed additions or revisions to these systems in order to evaluate their impact on slope erosion.

2. Retaining walls shall have adequate freeboard to provide a catchment area for minor slope erosion. Periodic inspection, and if necessary, cleanout of deposited soil and debris shall be performed, particularly during and after periods of rainfall.

3. The future developers shall be made aware of the potential problems, which may develop when drainage is altered by landscaping and/or by construction of retaining walls and paved walkways. Ponded water, water directed over slope faces, leaking irrigation systems, over-watering, or other conditions which could lead to excessive soil moisture, must be avoided.

4. Surficial slope soils may be subject to water-induced mass erosion. Therefore, a suitable proportion of slope planting shall have root systems which will extend well below three feet. We suggest consideration of drought-resistant shrubs and low trees for this purpose. Intervening areas can then be planted with lightweight surface plants with shallower root systems. All plants shall be lightweight and require low moisture. Any loose slough generated during planting of shrubs, trees, and other surface plants shall be removed from slope faces.

5. Construction delays, climate/weather conditions, and plant growth rates may necessitate additional short-term, non-plant erosion control measures such as matting, netting, plastic sheets, deep (5-ft) staking, etc.
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6. Significant erosion can be initiated by seemingly insignificant events such as rodent burrowing, human trespass (footprints, etc.), small concentrations of uncontrolled surface/subsurface water, or poor compaction of utility trench backfill on slopes.

7. High and/or fluctuating water content in slope materials is a major factor in slope erosion and/or slope failures. Therefore, all possible precautions shall be taken to maintain moderate and uniform soil moisture in soil and rock slopes. Slope irrigation systems shall be properly operated and maintained and irrigation system controls shall be placed under strict control.
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EROSION CONTROL REFERENCES


5. "Rain-Care and Protection of Hillside Homes", brochure undated, published by Building and Safety Division, Los Angeles County Engineer.


10. "Grading Guidelines (8 pages, stapled sheets)", Building and Safety Division, Department of County Engineer, County of Los Angeles (undated, but probably about 1977).
