SOILS REPORT

PLAN REVIEW - PHASE 1A
(BUILDING "B")

SECOND PLAN REVIEW
(COVERED WALKWAYS & BLDG "C")

GEOTECHNICAL INVESTIGATION
Proposed Two New Buildings and
Structural Addition to Existing Building
1232 Arrowhead Avenue
Livermore, California

Prepared for:

Hindu Community Cultural Center

1232 Arrowhead Avenue

Livermore, CA 94551

Prepared by:

HENRY JUSTINIANO & ASSOCIATES

August, 2009

HENRY JUSTINIANO & ASSOCIATES
GEOTECHNICAL ENGINEERING

Project No. H-140-01
August 10, 2009

Hindu Community Cultural Center
1232 Arrowhead Avenue
Livermore, CA 94551

SUBJECT: GEOTECHNICAL INVESTIGATION
Proposed Two New Buildings and
Structural Addition to Existing Building
1232 Arrowhead Avenue
Livermore, California

Gentlemen:

Our geotechnical report for the site of the proposed two new buildings and structural addition to the existing Assembly Hall, is herewith submitted. The report presents the results of our explorations and review of published geologic maps and reports, along with our evaluations and recommendations for foundation design, and other earthwork related elements of the project.

In our opinion, the site is suitable for the proposed improvements provided the recommendations presented in this report, are incorporated into the design and adhered to during construction.

If you should have any questions or need further assistance, please do not hesitate to contact this office.

Respectfully Submitted,

HENRY JUSTINIANO & ASSOCIATES

Henry Justiniano, P.E.
Calif. No. C-42347
Exp. 3/31/2010

Enclosures
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1.0 INTRODUCTION

1.1 PURPOSE

This report presents the results of our investigation of the subject property, and the review of the published geological data pertaining to the general area.

General engineering design and geotechnical recommendations are provided, based upon the physical and strength characteristics of the subsurface materials, and take into consideration the proposed project's requisites.

1.2 SITE LOCATION

The subject site is located in the Spring Town District of the City of Livermore. Specifically, the site lies on the western side of the Arrowhead Avenue, between Treeflower and Goldenrod Drives. The precise location is illustrated on the Site Location Map, Figure 1.

1.3 SITE CHARACTERIZATION

The subject property can be characterized as two, rectangularly shaped parcels that encompass roughly 8.0 acres (Figure 2). The setting conforms to the densely populated residential area along the northeastern fringes of the relatively flat lowlands of the Livermore Valley. At present, the northern parcel hosts a Temple; an Assembly Hall; and ancillary parking that were constructed in the mid-1980's. The southern parcel is essentially vacant with a gravel surface that presently serves as a parking lot extension.

1.4 SCOPE

The scope of our work included a literature research of available and applicable geological and geotechnical data, and exploratory borings and logging of the foundation soils encountered during the field investigation. The soil data compiled was analyzed in support of the recommendations presented herein.

1.5 PROPOSED IMPROVEMENTS

In accordance with the information furnished to this office, it is proposed to expand the existing Assembly Hall and construct two new buildings on the southern parcel. In addition, the proposed improvements will include two new parking lots (Figure 2).

1.6 SUMMARY OF RESULTS

Based upon the results of our evaluations, we conclude that there are no geotechnical nor geologic considerations that would preclude the proposed site improvements. Information from our review of geological maps, the existing topography, and our exploration program, indicates that the proposed building locations are within acceptably stable terrain, and that the site would be feasible for the proposed structural addition and buildings, provided that the recommendations presented herein are incorporated into the design and adhered to, during the construction phases of the project.

2.0 GEOLOGY

2.1 REGIONAL GEOLOGY

The project site is situated within the central portion of the Coast Range Province of Northwestern California. The Coast Range Province is characterized by a structural domain that is locally controlled by north to northwest trending, subparallel mountain ridges, and narrow valleys. The internal structures are often complex folds that are associated with structural deformations that have been created by a compressional regime during the Middle Mesozoic through Early Cenozoic Eras.

Tectonic features of the region reflect a deep crustal, northwestward movement of the Pacific Plate, relative to the North American Plate. Surface displacement is largely recognized along the San Andreas Fault Zone. However, the plate boundary movement is distributed among several faults between the Pacific Ocean and Western Nevada. These major faults are often characterized by a series of parallel anastomosing fault splays that develop at the surface, in response to the differential subsurface movement.

Historically, the active faults in the San Francisco Bay Region are, from west to east, the San Gregorio, the San Andreas, the Hayward, and the Calaveras Faults. These faults remain locked and quiet over periods of tens to hundreds of years. During quiet periods, strain builds up by gradual deformation of the crust adjacent to the fault. This strain is relieved periodically in sudden fault displacements that produce earthquakes. The displacement for Bay Area faults is dominantly right-lateral strike slip, with minor oblique slip component movements.

2.2 SITE GEOLOGY

Recent compilation in the geologic map published by Graymer et al. (1997, Figure 3) indicates that the site is underlain by Pleistocene age, alluvial deposits. The deposits are described as brown, dense gravelly and clayey sand or clayey gravel that fines grades upward to sandy clay.

In general, the near surface sediments at the subject site were identified as sandy clays and clayey sands, which are in general agreement with the above cited descriptions.

2.3 FAULTING/SEISMICITY

The property is not within a current Alquist Priolo Earthquake Hazard Zone (formerly a Special Studies Zone), and previous mapping does not depict active fault traces through the site. During our reconnaissance we did not observe any geomorphic conditions within the property that would suggest the presence of an active fault trace.

Table I below presents an assessment of the faults that contribute the most significant ground-motion hazard to the site. Included in the Table is the shortest distance between the site and each fault (as measured in kilometers from the surface trace projection of the fault); the maximum moment magnitude (Mw) for the Upper Bound Earthquake (UBE) estimated for each fault.

TABLE 1
FAULT DISTANCE - MAGNITUDE - ACCELERATION

Active Fault System	Distance		Upper Bounds Magnitude (Mw)
	Miles	Kilometers	
Hayward	17.0	27.4	7.1
Calaveras	10.8	17.4	6.8
Concord-Green Valley	17.1	27.5	6.9
Greenville-Marsh Creek	1.8	2.9	6.9
San Andreas (Northern)	35.1	56.5	7.9

(Mw): Estimated Moment Magnitude from CDMG (1996) Open File Report 96-08.

The Design Basis Earthquake (DBE) ground motion is defined to have a 10% chance of exceedance in 50 years (475 year return period). Development of the DBE ground motion value requires a site specific Probabilistic Seismic Hazard Analysis (PSHA). A peak ground acceleration (PGA) estimate of 0.505 for the Design Basis Earthquake (10% probability of exceedance in 50 years) is presented in the California Geological Survey's web site for a Probabilistic Seismic Hazards Assessment for an alluvium site (Figure 4). The subject area is assigned a high hazard rating, due to its proximity to several faults . . . in particular, the Greenville or Calaveras Faults.

In summary, it will be necessary to design and construct the project in strict adherence with current standards for earthquake-resistant construction. Because of the proximity of the Greenville Fault, we would recommend that the structural design consider the potential for strong to very strong ground shaking, and that the design conform to the requirements of the California Building Code (2007).

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 FIELD INVESTIGATION

A site investigation was conducted by our firm on August 6, 2009, at the site of the proposed improvements. The subsurface exploration consisted of drilling three augered boring, with a truck mounted rotary drill rig. The boring were advanced with 4-inch O.D. augers, to a maximum depth of 50-feet. Based on our careful monitoring during the drilling, the absence of borehole collapse could be assured. As such, it was determined that the hollow stem augers that were on standby, were not necessary. The approximate locations of the borings are presented on the Site Plan, Figure 2.

The samples collected during our investigation, consisted of relatively undisturbed samples obtained by advancing into undisturbed soil, a Standard Penetration split barrel sampler, through the action of a 140-pound hammer falling a distance of 30-inches. A conventional rope and cat head arrangement was used to lift the hammer so that while energy measurements were not made, the energy ratio might be expected to be in the order of 60 percent. The in-situ strength characteristics of the underlying soil are indicated by correlating the blow counts required to drive the sampler the lower 12-inches of a 18-inch sample attempt. The soils encountered were examined and logged in the field by an Engineer from this office. The soil profiles are presented as Figures 7 thru 10.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected samples, in order to identify some of their engineering properties. Testing was conducted to establish grain size distributions and Atterberg limits for soil classification. In addition, in-situ moisture contents were measured.

The determination of Atterberg limits is used to correlate consistency changes with moisture variation, which is indicative of the expansion potential of the soil (ASTM D-4943) and to evaluate the liquefaction susceptibility of the underlying soils. Atterberg limits testing was performed on nine samples from variable depths. The results yielded Liquid Limits between 29 and 51, along with a Plasticity Index ranging from 13 to 37, which corresponds to clays of moderate to high plasticity.

Sieve analyses conducted to obtain grain size distributions of the encountered materials. In general the underlying stratigraphy was classified as sandy clay.

4.0 SUBSURFACE CONDITIONS

4.1 GENERAL

The results of our geological research, and confirmed by our exploration, indicate that the subject site is underlain by sandy clays and clayey sands. As mapped, these materials correspond to Pleistocene Period, alluvial fan and fluvial deposits. An approximate 4-foot thick, highly expansive silty clay topsoil blankets the site. Beneath the surface mantle, predominantly stiff, clayey soils with intermittent loose to medium dense, clayey sand layers, were revealed to the explored depth of 50-feet.

Groundwater was encountered at depths between 9 and 11-feet.

4.2 POTENTIAL FOR LIQUEFACTION AND SEISMICALLY-INDUCED SETTLEMENT

Liquefaction occurs when a loose, saturated granular deposit changes from a solid to a liquid state, due to particle densification and increased pore pressures during seismic shaking. Recent mapping by William Lettis & Associates, California Division of Mines and Geology and the USGS (2006, Figure 5), assigns the site to alluvial fan deposits that are described as having a moderate liquefaction susceptibility. Official Mapping by the State of California, delineating Seismic Hazard Zones (2009, Figure 6), assigns the subject site to the eastern fringes of an area with a potential for liquefaction. While the maps shown in Figures 5 and 6 indicate the possibility of seismically induced settlement and liquefaction at this site, generalized maps of this kind are notoriously conservative. As indicated by Figure 3, and confirmed by our borings, the site is underlain by alluvial fan deposits. There is no historic precedent in the Bay Area for liquefaction in such deposits, although clean sands from basal channel or overbank deposits which could be susceptible to liquefaction, may be present. In the absence of such clean sand deposits, we conclude that the liquefaction potential at the site, under extreme earthquake loading, is low, and negligible with regard to any effect being realized at the surface.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based upon the results of our exploration, it is our opinion that the property may be rendered suitable for the proposed improvements. The building site consists of deep alluvial deposits that offer complex bearing characteristics for structural support of the proposed two new buildings. The existing building that is designated to receive a structural addition and the adjacent Temple building, have a satisfactory 20-year-plus, records of performance. Nevertheless, from a geotechnical engineering perspective, it will be necessary to design the foundation support with careful consideration to the potential for vertical displacements as a result of highly expansive near surface soils, and settlement due to relatively soft characteristics of the underlying soils.

Standard professional practice, demands that the proposed structural addition foundation, match the existing foundation system that supports the Assembly Hall building. Based on the information provided to this office from the subject building's Structural Engineer of record, the existing building derives support from a pier and grade beam foundation with floating slab-on-grade interior floor space. As such, design recommendations are provided herein for the design of a pier and grade beam foundation to support the proposed structural addition. However, due to a potential for pier hole wall collapse during drilling and the obstruction posed by the existing improvements to adjust the exterior grades to promote drainage, detailed recommendations are provided in the following section of this report, for contingencies to mitigate the effects of these adverse conditions.

The proposed two, new buildings, will require building pad preparations and relatively heavy mat type foundations, to mitigate the effects of the highly expansive soils and settlement related issues.

In order to avoid saturation of foundation bearing soils, resulting from surface flows, the site drainage must be planned so that the foundations are not allowed to saturate, and no ponding of water takes place near the foundation. The detailed recommendations for foundation design criteria, and other pertinent considerations, are presented in the following sections of this report.

The recommendations presented in this report, are for the soil conditions encountered during our exploration. Should other soil or rock conditions be uncovered during construction, due to non-uniformity of the geological formations, we should be contacted to evaluate the need for revision of the recommendations presented herein.

5.2 SEISMIC DESIGN

Based on the results of our investigation, we recommend that the following seismic design criteria be implemented in accordance with the California Building Code (2007):

Site Class	D
F_a	1.0
F_v	1.5
S_{ds}	1.20
S_{d1}	0.67

5.3 SITE PREPARATIONS

While no mass grading is anticipated for the future improvements, it may be desirable to raise the grades in the areas designated to receive structural improvements, to promote drainage and achieve uniformly compacted pad grades.

The pad locations for the future two new buildings are presently covered with gravels as they have served as parking areas. The existing gravels, constitutes an acceptable surface to receive fill materials. Nevertheless, it is recommended that the surface soils within areas designated to receive foundation improvements, be scarified, and moisture conditioned as necessary, prior to being compacted, as directed in the field by our Engineer. The resulting grade should produce a surface that slopes away from the future building perimeter, to act as a barrier to infiltrating surface waters. Compaction testing of the scarified and compacted surface will not be practical, due inaccuracies that can be anticipated as a result of a non-uniformity of the mixture of the native and imported materials. Subsequently, it is recommended that a minimum of 18-inches of Class II Baserock be provided to raise the pad grades relative to the surrounding future parking subgrade, to promote drainage and a firm pad subgrade. The Baserock should be compacted to 90% of the maximum dry density, based on ASTM Test Procedure D1557.

The areas designated to receive the structural additions to the existing Assembly Hall building, should be scarified and moisture conditioned to attain a moisture content between 2 and 4 percent above the optimum moisture content, in preparation for the administration of compaction efforts intended to accomplish between 85 and 90% of the maximum dry density, based on ASTM Test Procedure D1557.

All grading operations must be under the supervision of an Engineer from this office, in addition to the compaction testing procedures conducted by a Field Technician.

5.4 FOUNDATION RECOMMENDATIONS

5.4.1 Additions to Assembly Hall Building

It will be required to match the existing building foundation with the construction of drilled, cast-in-place reinforced concrete piers.

It will be necessary for the piers to be structurally integrated by grade beams, and to the existing building foundation, so that they act as a unit. Structural loads should determine pier spacing. The grade beam design should attempt to minimize the beam width to reduce the area that may be subjected to upward soil expansion pressures. The following table summarizes our recommended criteria for foundation design:

FOUNDATION DESIGN CRITERIA

Pier Diameter	Minimum 12-inches.
Pier Spacing	Minimum 3 pier diameters. Maximum 8-feet, center to center.
Pier Depth	Minimum of 25-feet, or as determined in the field by a representative from this office, during drilling.
Bearing Capacity	Maximum friction value of 200 psf commencing three feet below the lowest adjacent grade. These values may be increased by 1/3 for wind and seismic loads. Pier depths of 30-feet, should be anticipated.
Grade Beams	Minimum reinforcement of two No. 5 bars, both top and bottom. Maximum width 10-inches.

The piers should contain steel reinforcement over their entire length, with reinforcement as directed by the project Structural Engineer. Resistance to lateral forces transmitted to elements of the foundation can be computed assuming a passive resistance that commences at the top of the piers, equivalent to that caused by a fluid weighing 250 pcf. The passive force may be assumed to have a tributary horizontal width equal to 1-1/2 pier diameters. In no case, however, should these piers contain less than four No. 5 reinforcing bars, with two bars tied to the top steel bars in the grade beam.

The settlement of piers is estimated to not exceed 0.5 inch when properly constructed, both as to depth of bearing and proper clean-out prior to placement of concrete. When ground water is encountered, a tremie should be used to place concrete in the pier holes. The procedure should begin by placing concrete with the tremie hose at the bottom of the excavation, and "floating" the water above the concrete until uncontaminated concrete flows out of the top of each hole. Due to the potential for caving or sloughing, contingencies should be made to place the reinforcing steel and concrete as soon as practical, following the pier excavations.

The interior floor space should implement slabs-on-grade that have the ability to float. Complete isolation of the floor from bearing walls, columns, nonbearing partitions, stairs, and utilities, should be provided to allow the slab to move with minimum damage to the structural integrity of the building. A flexible felt joint should be provided between the grade beam and the slab, to fill the void and prevent moisture infiltration.

All slabs should be a minimum thickness as set forth by the Structural Engineer, but should not be less than 5-inches thick, and reinforced by a minimum of No. 4 bars, spaced at 18-inches each way, and centered within the entire slab.

Concrete slabs should include crack control joints for normal lineal shrinkage of the concrete materials. Where large areas of concrete slab are placed, with irregular projections or inserts within the slab area, stress concentrations will result, causing uncontrolled crack patterns. Where possible, crack control joints should be placed at stress locations where projections from a main slab, or where inserts occur, in order to control the resultant crack pattern.

All concrete slabs-on-grade should be underlain by a 4-inch thick capillary break of "pea gravel" or clean crushed rock (no fines). It is recommended that Class 2 baserock not be employed as the capillary break material. To mitigate vapor transmission, it is recommended that an impermeable membrane of 10-mil minimum thickness be placed upon the capillary break material, and overlain by 2-inches of clean sand, to assist in proper curing of the slab.

5.4.2 New Building Foundations

Geotechnical conditions demand a foundation system designed to resist the effects of highly expansive soils. We recommend that the design procedures outlined in the Uniform Building Code (2001), Section 1815, be implemented, to design a structural mat foundation system.

Based upon an assumed climatic rating of 15 and effective plasticity index of 30, a soil/climatic rating factor of .17 can be assigned. Based on the soil/climatic rating factor of .17, a cantilever length can be obtained from Figure 18-III-6, as L_c = 6-feet. As such, it is recommended that the slab be designed with an ability to cantilever a distance of 6-feet.

The exterior edges should be at least 12 inches in width and have their bases located no less than 24 inches below the lowest adjacent finished subgrade. Isolated interior footings may have their depth reduced to 18 inches below the lowest adjacent subgrade.

Exterior edges should contain steel reinforcement over their entire length, with reinforcement as directed by the project Structural Engineer. In no case, however, should the exterior edge contain less than two No. 5 reinforcing bars, both top and bottom. Exterior edges constructed to the given criteria, may be designed for an allowable bearing capacity of 1,500 psf for dead load, and 2,000 psf dead load plus live load condition. These values may be increased by one-third to accommodate short duration seismic or wind loading conditions.

All concrete slabs-on-grade should be underlain by a 4-inch thick capillary break of "pea gravel" or clean crushed rock (no fines). It is recommended that Class 2 baserock not be employed as the capillary break material. To mitigate vapor transmission, it is recommended that an impermeable membrane of 10-mil minimum thickness be placed upon the capillary break material, and overlain by 2-inches of clean sand, to assist in proper curing of the slab.

5.5 DRAINAGE

It is important to divert surface run-off away from the foundation perimeter. A slope gradient of 3 percent down and away from the building perimeter, for a minimum of 5 feet, should be provided to the finish grade. Downspouts should be connected to conduits that will transport their effluent to a discharge point away from structural element-bearing soils. Adjacent areas should be slopped toward area drains that are designated to low points.

Due to physical and grade restrictions affecting the existing Assembly Hall Building, modifications to the exterior grades to promote drainage would be burdensome. As such, we recommend that a perimeter subdrain be provided along the exterior of the entire building. The subdrain trench should extend into the underlying dark-brown clay topsoil and slope at a minimum of 1% toward a discharge into the storm system. A 3-inch diameter, rigid, perforated pipe with perforations facing down, should then be placed at the base of the trench and the pipe slope verified. The trench should then be backfilled with Class II Permeable Rock material.

5.6 UTILITY TRENCHES

Utility trenches that are parallel to the sides of the slab edges, should be avoided. All trenches should be backfilled with native materials compacted uniformly to a 90% relative compaction.

5.7 PAVEMENTS

The pavement section for the driveway and parking areas should be no less than 2.5-inches of Asphaltic Concrete over 9-inches of Class II Aggregate Baserock, in accordance with the previous Geotechnical Study by Consolidated Engineering.

The performance of the final pavement will depend upon the quality of workmanship and materials. The following summarizes the recommended construction procedure to be followed:

1. Scarify the subgrade surface to a minimum of 6-inches, to properly moisture condition the soil to 2 to 5 percent above the optimum moisture content, and compact it to between 88 and 92 percent of maximum dry density.

2. Provide the necessary gradient to prevent the ponding of water.

3. Place the baserock in lifts that are within the compaction capabilities of the compaction equipment, and compact to 95 percent of maximum density.

4. Place the Asphaltic Concrete during fair weather only, and at a temperature within its' prescribed limits.

6.0 GENERAL CONDITIONS

6.1 PLAN REVIEW

Prior to the submission of design drawings and construction documents for approval by the appropriate local agency, copies of these documents should be reviewed by our firm, to evaluate whether or not the recommendations contained in this report, have been effectively incorporated into the design of the project.

6.2 CONSTRUCTION OBSERVATIONS

A representative of this firm must be present during grading of the site. This item is necessary to properly evaluate the quality of the materials and their relative compaction. Foundation excavations must be inspected by a representative of this firm, in order to make the necessary adjustments as a result of localized irregularities.

At the completion of the earthwork related construction, a report will be submitted summarizing our observations, including the results of the compaction testing program.

To allow for proper scheduling, we request a minimum of 48 hours notice prior to the commencement of earthwork operations requiring our presence.

6.3 LIMITATIONS

This report has been prepared by HENRY JUSTINIANO & ASSOCIATES for the exclusive use of the Hindu Community Cultural Center Administrators, and their representatives, for consideration of the proposed improvements to the property described in this report.

The interpretations and recommendations presented in this report, are professional judgements and are based on our evaluations of the technical information obtained during this investigation, on our understanding of the characteristics of the planned improvements, and on our general experience with similar subsurface conditions in other areas. We do not guarantee the performance of this project in any respect, only that our engineering work and judgements meet the standards of care normally exercised by our profession.

It is assumed that the borings are representative of the subsurface conditions throughout the areas designated to receive improvements. Unanticipated soil conditions are commonly encountered and cannot be fully determined by performing exploratory borings. If, during construction, subsurface conditions different from those indicated in this report, are encountered or appear to be present beneath excavations, HENRY JUSTINIANO & ASSOCIATES should be advised at once, so we can review these conditions and reconsider our recommendations, when necessary.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations, considering the time lapse or changed conditions.

The scope of our services did not include an environmental assessment, or an investigation of the presence or absence of hazardous, toxic, or corrosive materials, in the soil, surface water, groundwater, or air, on, below, or around this site.

REFERENCES

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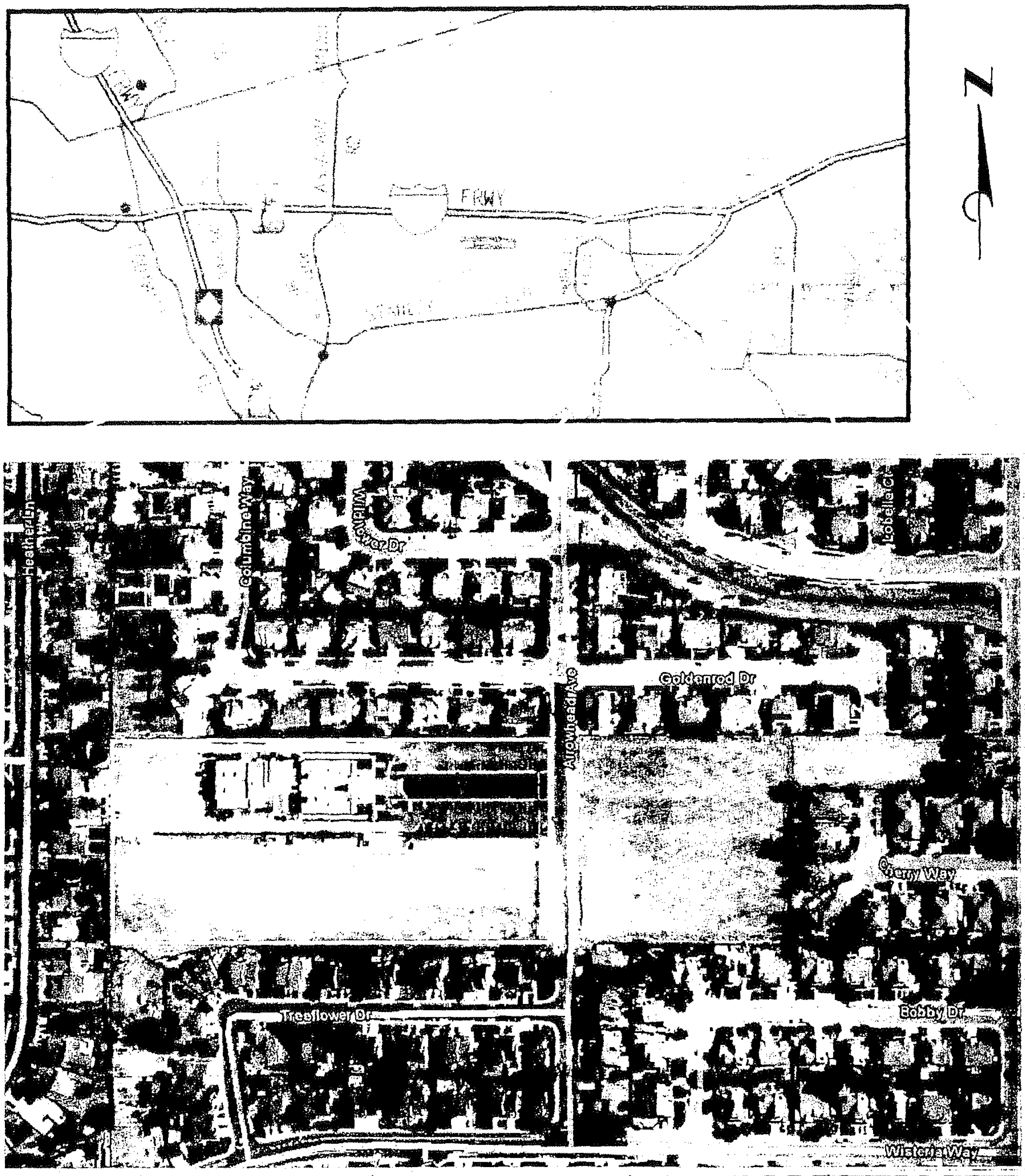


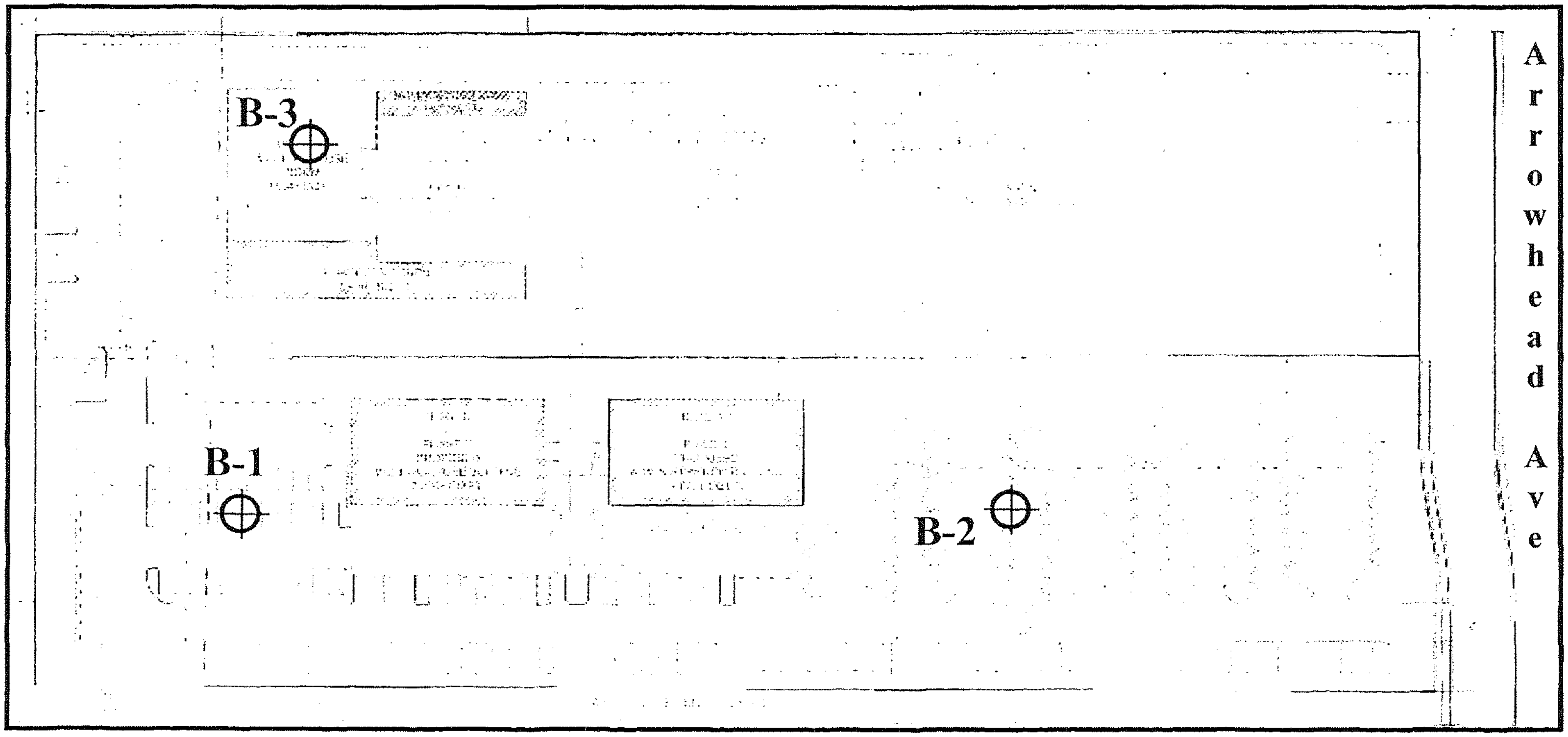
SITE LOCATION

Source: Thomas Guide, 2003
Google Earth

Project No.: H-140-01	Date: 08-10-09	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 1



SITE PLAN

Explanation

Approximate Borehole Location

Source: DeBolt Civil Engineering

Project No. : H-140-01	Date: 08-10-09	Scale: NTS

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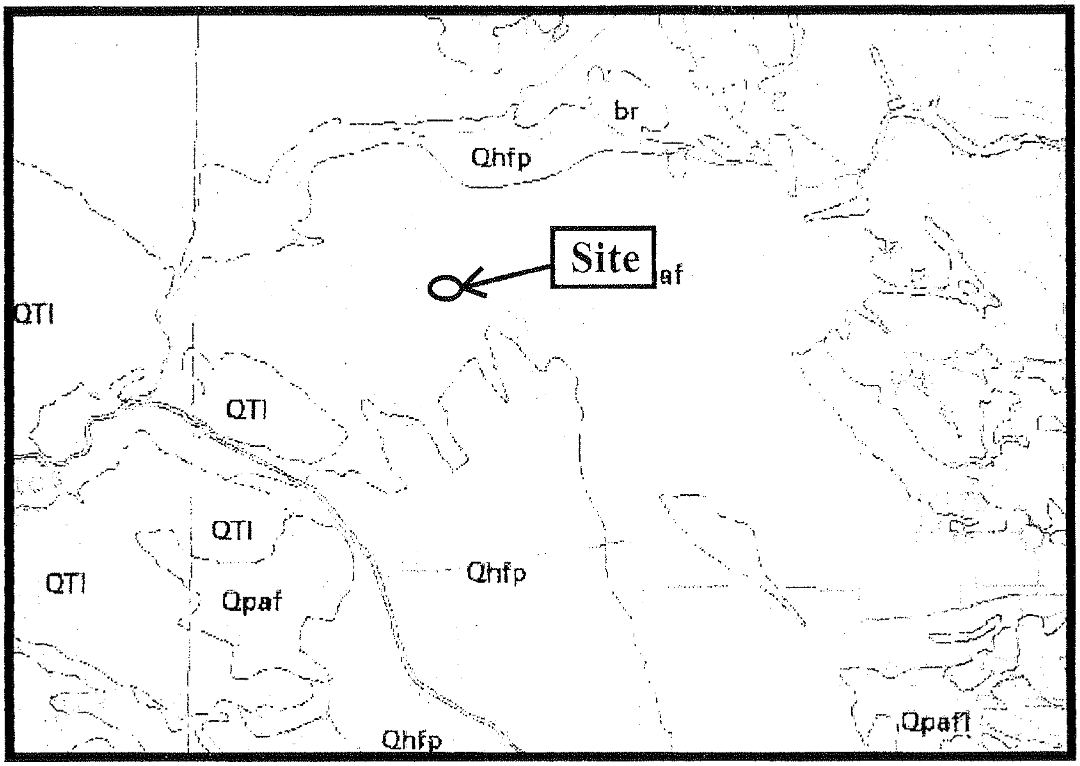
Figure No. 2



EXPLANATION

Qhfp –Floodplain deposits (Holocene)

Qpafl –Alluvial Terrace deposits (Pleistocene)

Qpaf –Alluvial Fan deposits (Pleistocene)

QTl –Livermore gravels (Pleistocene and/or Pliocene)

GEOLOGY MAP

E. J. Helley and R. W. Graymer, 1997

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Project No.: H-140-01	Date: 08-10-09	Scale: NTS

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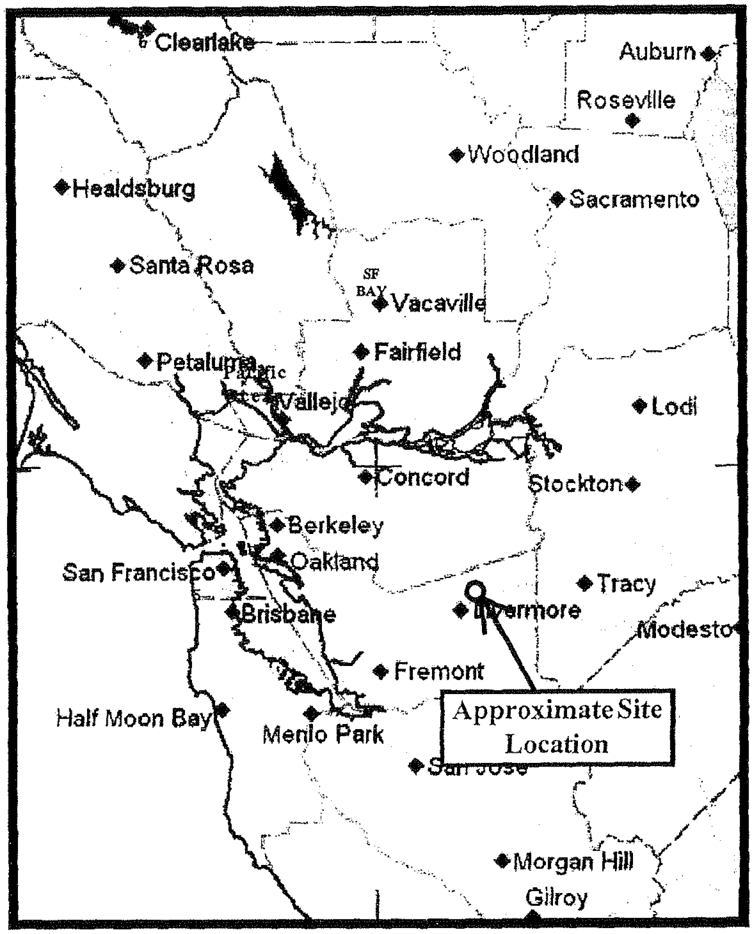
Figure No. 3

Shaking (%g)
Pga (Peak Ground Acceleration)
Firm Rock
< 10%
10 - 20%
20 - 30%
30 - 40%
40 - 50%
50 - 60%
60 - 70%
70 - 80%
> 80%
The unit "g" is acceleration of gravity.



PROBABILISTIC SEISMIC HAZARD MAP (Modified)

(10% Probability of Exceedance in 50 Years)
Peak Horizontal Ground Acceleration
Firm-Rock Site Condition

Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) (revised 2003)

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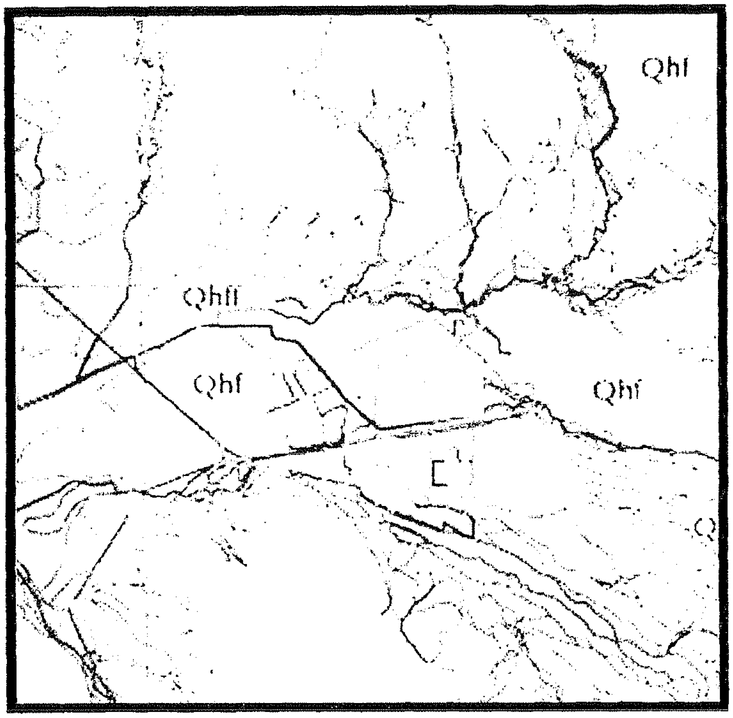
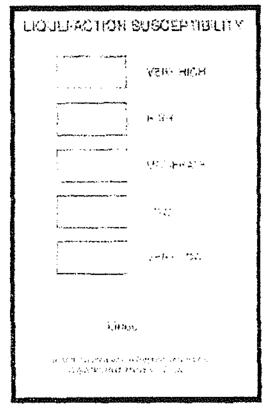
Figure No. 4



EXPLANATION



MAPS OF LIQUEFACTION SUSCEPTIBILITY IN THE CENTRAL SAN FRANCISCO BAY REGION CALIFORNIA

R. C. Witter, K. L. Knudsen, J. M. Sowers, C. M. Wentworth, R. D. Koehler, and C. E. Randolph,

California Geological Survey, U.S. Geological Survey

2006

Project No.: H-140-01	Date: 08-10-09	Scale: NTS

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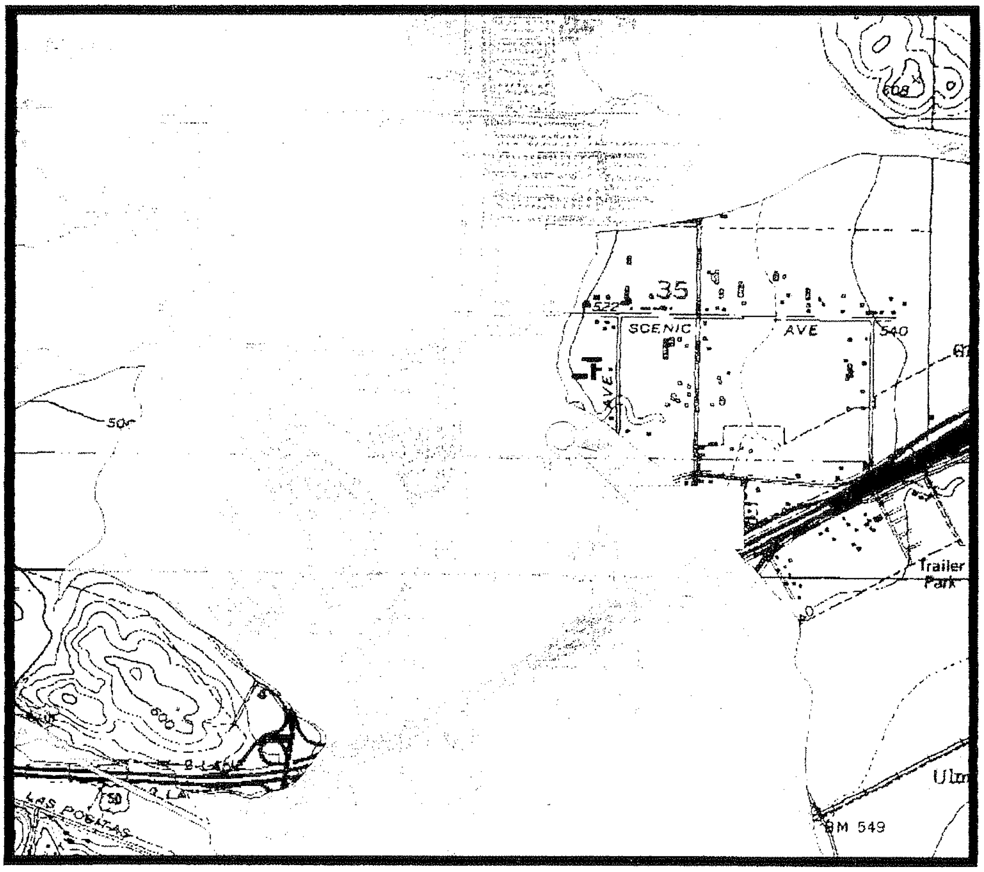
Figure No. 5



EXPLANATION

Liquefaction

Areas where historical occurence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

STATE OF CALIFORNIA SEISMIC HAZARD ZONES

ALTAMONT QUADRANGLE OFFICIAL MAP
RELEASED FEBRUARY 27, 2009 (MODIFIED)

Project No.: H-140-01	Date: 08-10-09	Scale: As Shown

Henry Justiniano & Associates

Soils and Foundation Engineering

Figure No. 6

Exploration Boring Log by:

Henry Justiniano & Associates

Boring Log No.: B-1

Project: Arrowhead

Client: Hindu Community Cultural Center

Date Drilled: 07/30/09

Equipment Used: Mobile Drill, 140 Lb., 30 inch Drive, 4" Continuous Flight, Sampler As Noted.

Location: 98' South, 132' East of Western Common Property Corner

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
2	**Atterberg Limits** Liquid Limit = 51 Plasticity Index = 37		13.7	8	B-1-A SPT		6" Gravel @ Surface Black-Grey, Silty CLAY Highly Plastic Moist, Medium Stiff to Stiff
			15.6	11	B-1-B SPT		Tan, Sandy CLAY Moist, Stiff
10	**Sieve**		17.0	16	B-1-C SPT		Groundwater level after drilling 9-Feet Tan, Sandy CLAY w/1"-2" Sand Lenses Moist, Stiff
	Atterberg Limits Liquid Limit = 29 Plasticity Index = 14 **Sieve**			10	B-1-D SPT		Tan, Clayey SAND Wet, Loose to Medium Dense
20	**Atterberg Limits** Liquid Limit = 29 Plasticity Index = 13 **Sieve**		16.8	10	B-1-E SPT		Tan, Sandy CLAY Wet, Stiff
	Atterberg Limits Liquid Limit = 34 Plasticity Index = 21 **Sieve**		15.3	13	B-1-F SPT		Tan, Sandy CLAY Wet, Stiff
30	**Sieve**		15.0	24	B-1-G SPT		Tan, Sandy CLAY Wet, Very Stiff
40	**Sieve**		15.2	20	B-1-H SPT		Tan, Sandy CLAY Wet, Very Stiff

Figure No. 7

Exploration Boring Log by:
Henry Justiniano
& Associates

Boring Log No.: B-1 Continued
Project: Arrowhead
Client: Hindu Community Cultural Center
Date Drilled: 07/30/09
Equipment Used: Mobile Drill, 140 Lb., 30 inch Drive, 4" Continuous Flight, Sampler As Noted.
Location: 98' South, 132' East of Western Common Property Corner

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
42							Tan, Sandy CLAY Wet, Very Stiff
50				37	B-1-I SPT		Tan, Clayey SAND No Recovery Wet, Dense Terminated at 50 feet.
60							
70							
80							

Figure No. 8

Exploration Boring Log by:
Henry Justiniano
& Associates

Boring Log No.: B-2
Project: Arrowhead
Client: Hindu Community Cultural Center
Date Drilled: 07/30/09
Equipment Used: Mobile Drill, 140 Lb., 30 inch Drive, 4" Continuous Flight, Sampler As Noted.
Location: 98' South, 250' West of Eastern Common Property Corner

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
2			12.6	13	B-2-A SPT		3" Gravel @ Surface Dark Brown, Silty CLAY Highly Plastic Slightly Moist, Stiff
	Sieve		17.7	11	B-2-B SPT		Tan, Sandy CLAY Moist, Stiff
10	Sieve		17.3	9	B-2-C SPT		Tan, Clayey SAND Wet, Loose to Medium Dense Groundwater level during drilling 11-Feet
	Sieve		18.7	9	B-2-D SPT		Tan, Clayey SAND Wet, Loose
20	Atterberg Limits Liquid Limit=42 Plasticity Index=25 Sieve		16.9	11	B-2-E SPT		Tan, Sandy CLAY Wet, Stiff
	Sieve		15.9	17	B-2-F SPT		Tan, Sandy CLAY Wet, Very Stiff
30	Atterberg Limits Liquid Limit=34 Plasticity Index=21 Sieve		12.1	24	B-2-G SPT		Tan, Sandy CLAY Wet, Very Stiff Terminated at 30 feet.
40							

Figure No. 9

Exploration Boring Log by:

Henry Justiniano & Associates

Boring Log No.: B-3

Project: Arrowhead

Client: Hindu Community Cultural Center

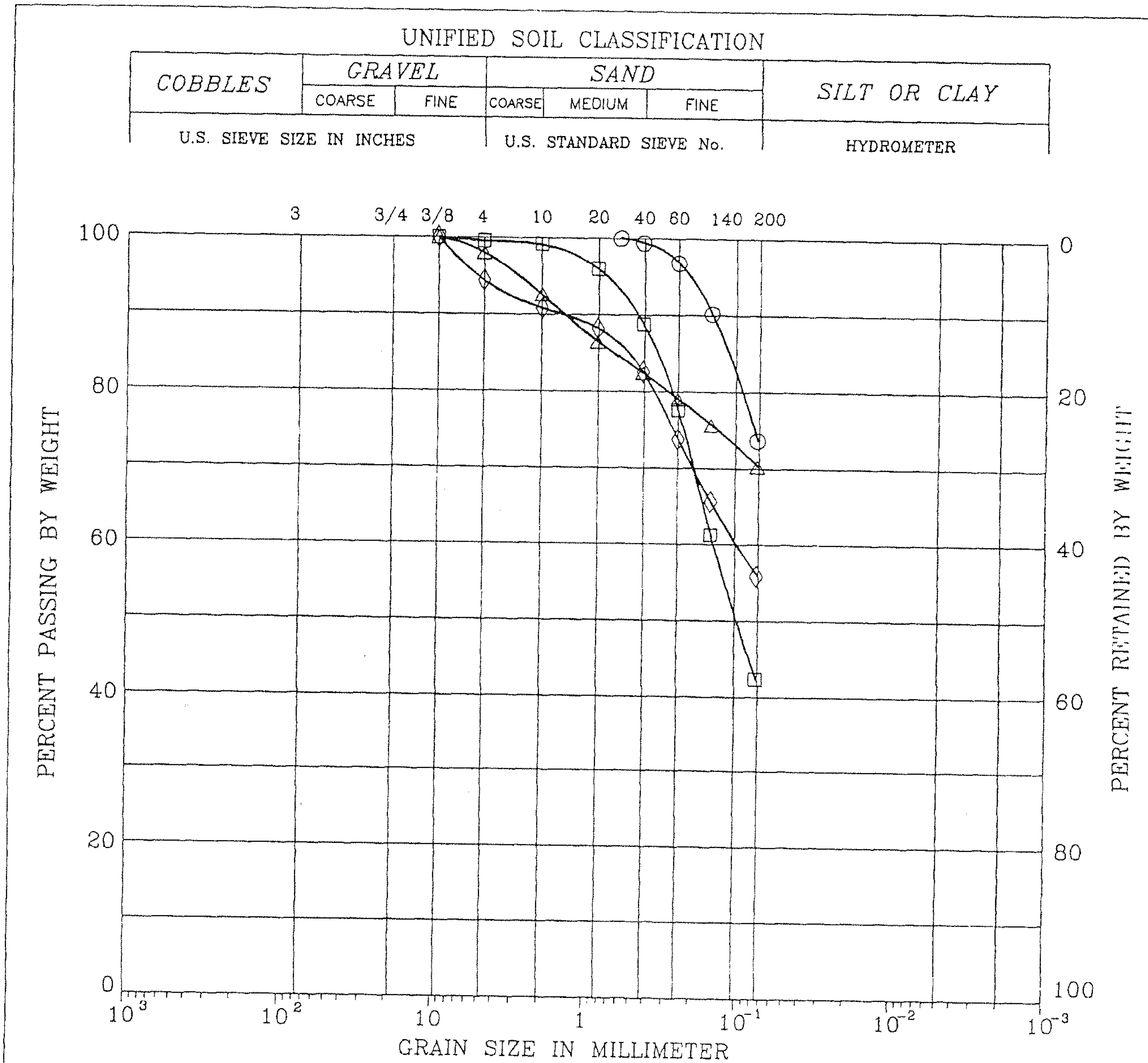
Date Drilled: 07/30/09

Equipment Used: Mobile Drill, 140 Lb., 30 inch Drive, 4" Continuous Flight, Sampler As Noted.

Location: 18' South, 50' West of Northwestern Corner of Western Existing Building

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
2	**Atterberg Limits** Liquid Limit = 33 Plasticity Index = 20		12.6	13	B-3-A SPT		Dark Brown, Silty CLAY Highly Plastic Moist, Stiff
			17.3	9	B-3-B SPT		Tan, Clayey SAND Wet, Loose to Medium Dense
10							Groundwater level during drilling 10-Feet
	Atterberg Limits Liquid Limit = 43 Plasticity Index = 27 **Sieve**		18.7	9	B-3-C SPT		Tan, Sandy CLAY w/2" Sand Lense, Mid Sample Wet, Stiff
20							
	Atterberg Limits Liquid Limit = 34 Plasticity Index = 17 **Sieve**		16.9	11	B-3-D SPT		Tan, Sandy CLAY Wet, Stiff
30	**Sieve**		17.7	11	B-3-E SPT		Tan, Clayey SAND Gravelly Sand Layer @ 28.5-29 Wet, Loose Terminated at 30 feet.
40							

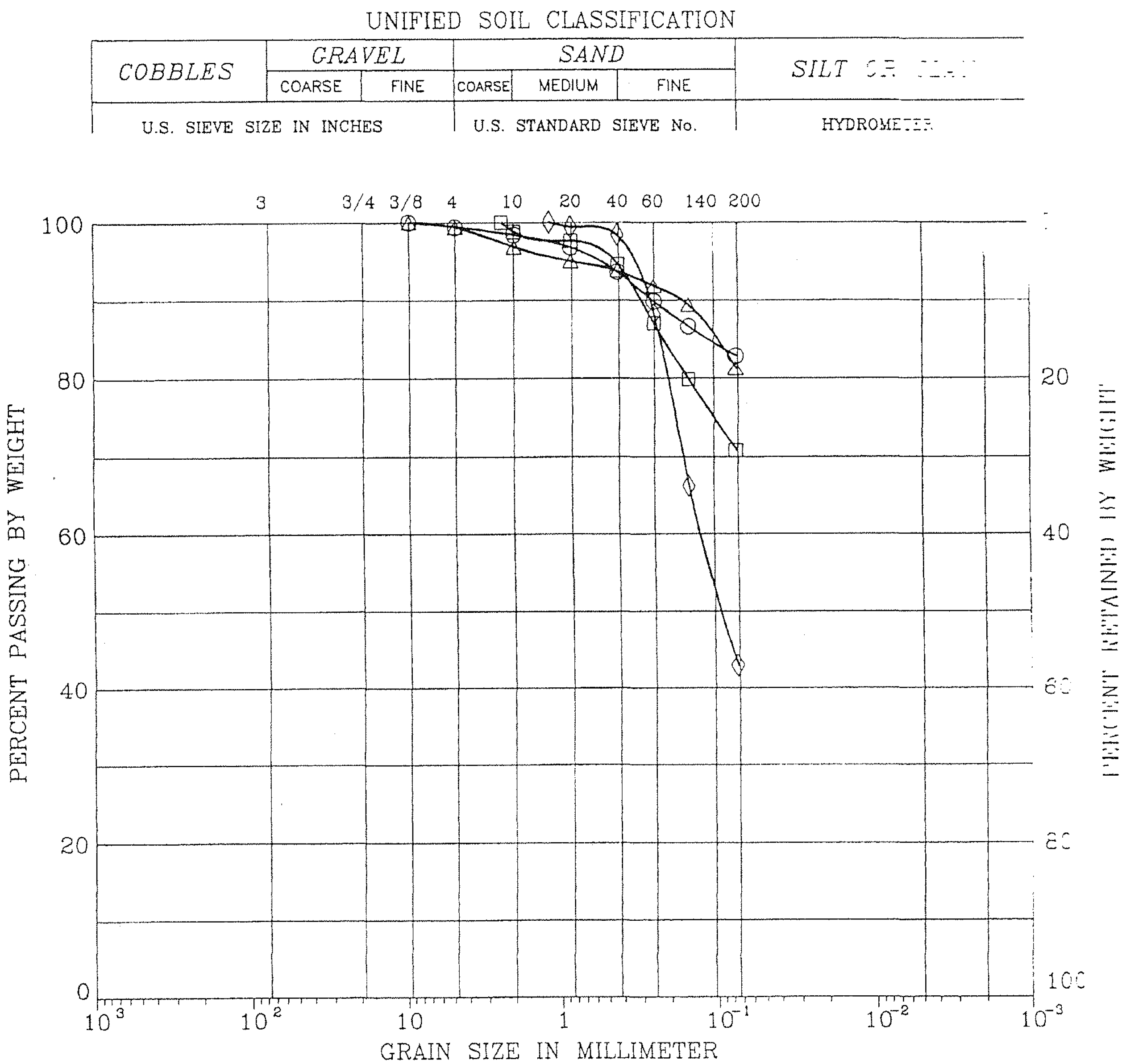
Figure No. 10



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-1-C	8.5-10			INORG. SILTS AND CLAYS (ML-CL)
□	B-1-D	13.5-15	29	14	CLAYEY SAND (SC)
△	B-1-E	18.5-20	29	13	INORGANIC CLAYS (CL)
◇	B-1-F	23.5-25	34	21	INORGANIC CLAYS (CL)

Remark :

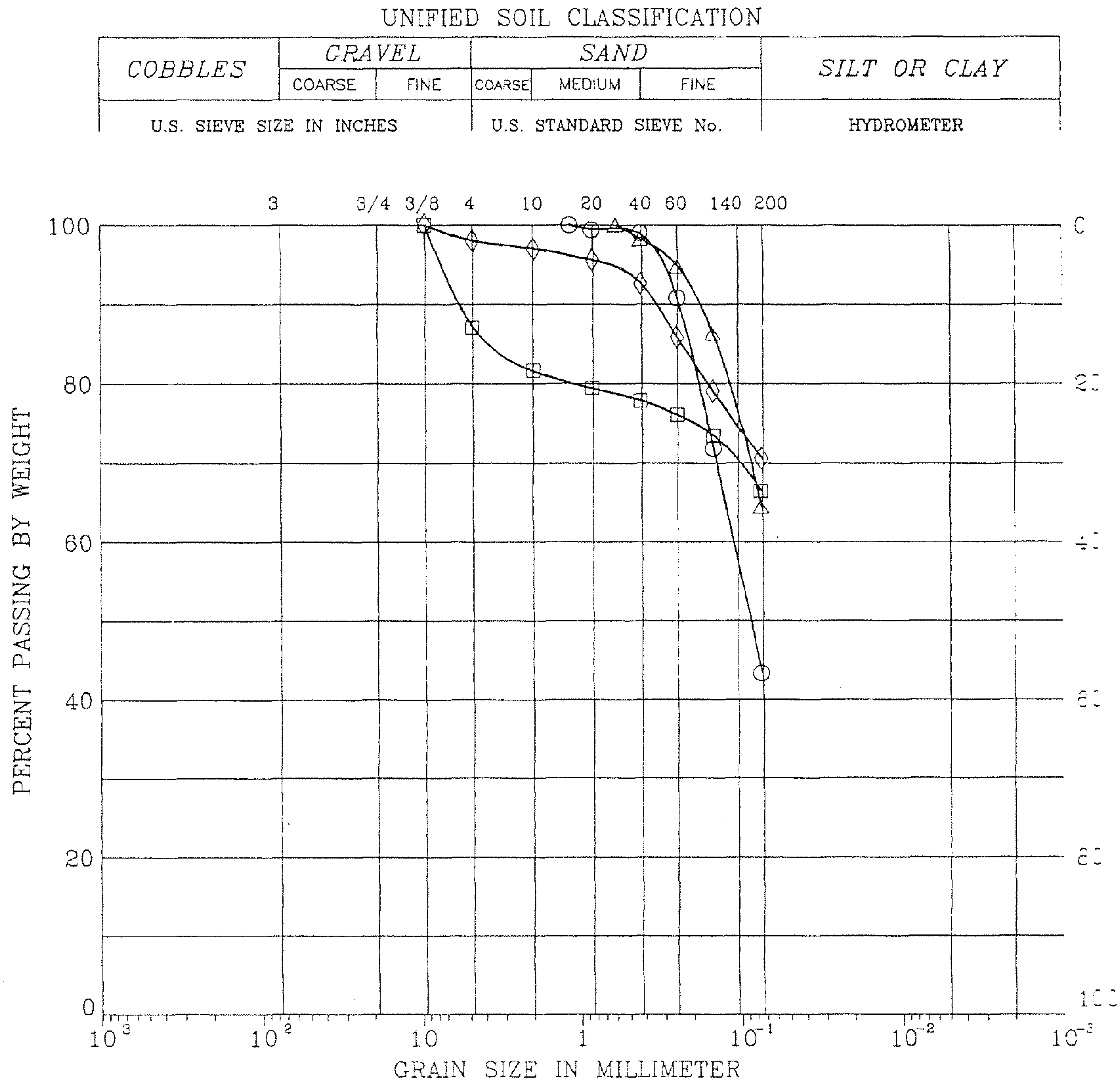
Project No.H-14001	Hindu Community Cultural Center
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 11



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-1-G	28.5-30			INORG. SILTS AND CLAYS (ML-CL)
□	B-1-H	38.5-40			INORG. SILTS AND CLAYS (ML-CL)
△	B-2-B	5-6.5			INORG. SILTS AND CLAYS (ML-CL)
◇	B-2-C	8.5-10			SILTY, CLAYEY SAND (SM-SC)

Remark :

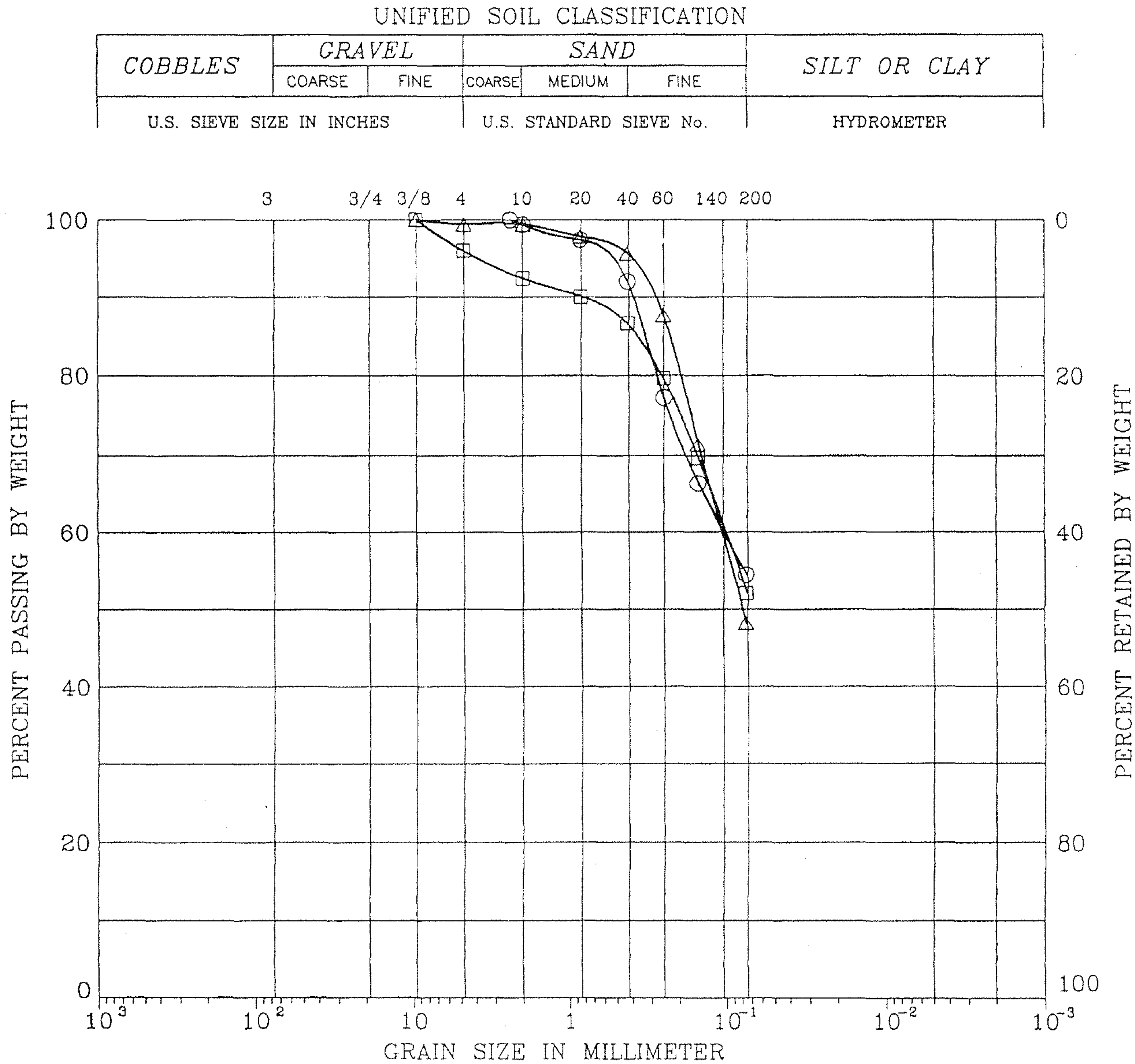
Project No.H-14001	Hindu Community Cultural Center
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 12



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-2-D	13.5-15			SILTY, CLAYEY SAND (SM-SC)
□	B-2-E	18.5-20	42	25	INORGANIC CLAYS (CL)
△	B-2-F	23.5-25			INORG. SILTS AND CLAYS (ML-CL)
◊	B-2-G	28.5-30	34	21	INORGANIC CLAYS (CL)

Remark :

Project No.H-14001	Hindu Community Cultural Center
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 13



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-3-C	13.5-15	43	27	INORGANIC CLAYS (CL)
□	B-3-D	23.5-25	34	17	INORGANIC CLAYS (CL)
△	B-3-E	28.5-30	.		SILTY, CLAYEY SAND (SM-SC)

Remark :

Project No.H-14001	Hindu Community Cultural Center
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 14

NRY JUSTINIANO & ASSOCIATES
ECHNICAL ENGINEERING

October 21, 2009
Project No. R-120-01

Hindu Community Cultural Center
1232 Arrowhead Avenue
Livermore, CA 94551

SUBJECT: PLAN REVIEW
Proposed Structural Additions
Hindu Community Cultural Center
Assembly Hall, Phase 1-A
1232 Arrowhead Avenue
Livermore, California

REFERENCES: GovindaRao & Associates, Foundation Plans and Supporting Structural Calculations for Hindu Community and Cultural Center, Project Arrowhead, Dated August 21, September 2, and October 19, 2009.

Henry Justiniano & Associates, Geotechnical Investigation, Proposed Two New Buildings and Structural Additions to Existing Building, 1232 Arrowhead Avenue, Livermore, California, Project No. H-140-01, Dated August 10, 2009.

Gentlemen:

In accordance with your request, we have reviewed the above referenced items in performance of this plan review for the proposed additions to the existing building, at the above subject Center. The purpose of our review was to determine if the foundation plans and supporting structural calculations have incorporated the geotechnical recommendations of the referenced soils report.

The plans indicate that the proposed foundation will derive support from 25-feet deep, 18-inch diameter piers. In estimating the pier capacity to carry vertical loads, the structural calculations have adhered to our recommended criteria. The plans designate pier reinforcement consisting of eight No. 5 vertical bars. Grade beam reinforcement will consist of two No. 5 bars, both top and bottom.

Slab-on-grade floors are designated 5-inch minimum thickness slab, reinforced by No. 4 bars at 12-inches on center each way. The slab will be underlain by 2-inches of sand, a moisture barrier, and 4-inches of gravel. A .5-inch felt joint is called for, at the grade beam/slab transition, as recommended.

The structural details call for the finished grade to have a 5 percent slope away from the building perimeter. Downspouts should be connected to conduits that will transport their effluent to a discharge point away from structural element-bearing soils.

In summary, it is our opinion that the foundation plans and structural calculations have incorporated the recommendations prescribed by the soils report.

All grading and foundation drilling operations should be conducted under the supervision of our Engineer.

If you should have any questions or need further assistance, please do not hesitate to contact this office.

Sincerely yours,

REGISTERED PROFESSIONAL ENGINEER
HENRY JUSTINIANO
C-042347
STATE OF CALIFORNIA

Henry Justiniano, P.E.
Calif. No. C-42347
Exp. 3/31/2010

cc: Seven Hills (2)

ᐧRY JUSTINIANO & ASSOCIATES
ECHNICAL ENGINEERING

May 28, 2010
Project No. R-120-01

Hindu Community Cultural Center
1232 Arrowhead Avenue
Livermore, CA 94551

SUBJECT: SECOND PLAN REVIEW
Proposed Trash Enclosure, Compost Area
Covered Walkways and Building C
Hindu Community Cultural Center
1232 Arrowhead Avenue
Livermore, California

REFERENCES: GovindaRao & Associates, Foundation Plans, Structural Details and Supporting Calculations for Hindu Community and Cultural Center, Phase 1-A, Trash Enclosure, Sheet S-5, Dated May 24, 2010; Phase 1-A, Compost Area, Sheet S-6, Dated May 24, 2010; Building C, Foundation and Roof Framing Plan, SheetS-1B, Dated March 12, 2010.

Henry Justiniano & Associates, Geotechnical Investigation, Proposed Two New Buildings and Structural Additions to Existing Building, 1232 Arrowhead Avenue, Livermore, California, Project No. H-140-01, Dated August 10, 2009. Plan Review, Proposed Structural Additions, Hindu Community Cultural Center Assembly Hall, Phase 1-A, Dated October 21, 2009.

Gentlemen:

In accordance with your request, we have reviewed the above referenced items in performance of this plan review for the proposed improvements to the above subject Community Cultural Center. The purpose of our review was to determine if the foundation plans and supporting structural calculations have incorporated the geotechnical recommendations of the referenced soils report.

The foundations for Building C, the trash enclosure and the compost area, are designed implementing mat foundations with thickened edges, in accordance with our recommendations. The slabs will be 9-inches in minimum thickness, with reinforcement consisting of two mats of No. 5 bars at 12-inches on center, each way, top and bottom. The structural computations arrive at the above-mentioned slab section by implementing the recommended capacity to cantilever 6-feet. All thickened edges will be a minimum of 12-inches in width, with the building having a minimum embedment of 24-inches, while the relatively lighter trash enclosure and the compost area slabs, will have their depth reduced to 22-inches, which is acceptable. The covered walkways will utilize a slab-on-grade that is designated with a 6-inch minimum thickness slab, reinforced by No. 5 bars at 12-inches on center each way. All slabs will be underlain by 2-inches of sand, a 10-mil moisture barrier, over a 4-inch layer of drain rock, except for the compost area, where the moisture barrier is rightly omitted.

We note that this office has been retained to observe all foundation excavations, in order to make any necessary adjustments, based on the conditions encountered.

In summary, it is our opinion that the foundation plans and structural calculations have incorporated the recommendations prescribed by the soils report.

If you should have any questions or need further assistance, please do not hesitate to contact this office.

Sincerely yours,

REGISTERED PROFESSIONAL ENGINEER
HENRY JUSTINIANO
C042347
STATE OF CALIFORNIA

Henry Justiniano, P.E.
Calif. No. C-42347
Exp. 3/31/2012