JOSHUA SPRINGS CALVARY CHAPEL
C/O WARNER ENGINEERING
7245 JOSHUA TREE LANE, SUITE B
YUCCA VALLEY, CA 92284

GEOTECHNICAL ENGINEERING REPORT
FOR THE PROPOSED IMPROVEMENTS
JOSHUA SPRINGS CHRISTIAN SCHOOL
YUCCA VALLEY, CALIFORNIA

#4
Soils

File No. 07278-01
99-08-750

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File No. 07278-01
99-08-750

Joshua Springs Calvary Chapel
c/o Warner Engineering
7245 Joshua Tree Lane, Suite B
Yucca Valley, California 92284

Attention: William Warner, PE

Subject: **Geotechnical Engineering Report**

Project: **Proposed Improvements**
Joshua Springs Christian School

It is our pleasure to present this Geotechnical Engineering Report prepared for the proposed gymnasium, pavilion, parking lot(s), and trash enclosure at the Joshua Springs Christian School.

This report presents our findings and recommendations for general site development and foundation design, incorporating the tentative information supplied to our office. This report should stand as a whole, and no part of the report should be excerpted or used to exclusion of any other part.

This report completes our scope of services in accordance with our agreement, dated July 12, 1999. Other services that may be required, such as plan review and grading observation are additional services and will be billed according to the Fee Schedule in effect at the time services are provided.

We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.

Respectfully submitted,
EARTH SYSTEMS CONSULTANTS
Southwest

Shelton L. Stringer

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TABLE OF CONTENTS

Section 1
INTRODUCTION

1.1 Project Description

This Geotechnical Engineering Report has been prepared for the proposed improvements at the Joshua Springs Christian School in the Town of Yucca Valley, California. The project will include the construction of a gymnasium, parking lot, multi-purpose tent pavilion, trash enclosure, storm water retention basin, and will use an on-sewage disposal system. Specifically, the proposed gymnasium, parking lot, and septic system will be located off Joshua Lane, immediately east of the existing Sanctuary/Administration Offices. The remaining improvements will be located at various locations on the campus. The onsite sewage disposal system will use seepage pits.

We understand that the proposed 90 by 120 foot gymnasium will be two stories and will be supported by conventional shallow continuous or pad footings. Wood-frame and stucco and manufactured steel frame construction will be used. Site development will include site grading, building pad preparation, underground utility installation, street and parking lot construction, and concrete driveway and sidewalk placement.

We used structural building column loads of up to 100 kips and a maximum wall loading of 4 kips per linear foot as a basis for the foundation recommendations. All loading is assumed to be dead plus actual live load. If actual loading is to exceed these assumed values, it may be necessary to reevaluate the given recommendations.

1.2 Site Description

The Joshua Springs Christian School Campus is bounded by Joshua Lane to the north, Nagles Street to the south, Kingston Road to the east, and Hardesty Drive to the west, in the Town of Yucca Valley, California. The site location is shown on Figure 1, located in Appendix A.

The campus consists of school buildings, playground, parking lot, and chapel. An existing convalescent hospital lies to the west of the campus. The Proposed additions are located within the school campus. The topography consists of a gently sloping alluvial plain that has been terraced by grading. The gymnasium site is generally vacant, barren sandy ground with a drainage swale along the eastern side of the building site.

1.3 Purpose and Scope of Work

The purpose for our services was to evaluate the site soil conditions and to provide professional opinions and recommendations regarding the proposed development of the site. The scope of work included the following:

- A general reconnaissance of the site.
- Shallow subsurface exploration by drilling 7 exploratory borings to depths ranging from 16.5 to 51.5 feet.
- Two percolation tests for seepage pit design.

- Laboratory testing of selected soil samples obtained from the exploratory borings.
- Review of selected published technical literature pertaining to the site.
- Evaluation of field and laboratory data.
- Engineering analysis and evaluation of the acquired data from the exploration and testing programs.
- A summary of our findings and recommendations in this written report.

This report contains the following:

- Discussions on subsurface soil and groundwater conditions.
- Discussions on regional and local geologic conditions.
- Discussions on geologic and seismic hazards.
- Graphic and tabulated results of laboratory tests and field studies.
- Recommendations regarding:
  - site development and grading criteria,
  - excavation conditions and buried utility installations,
  - structure foundation type and design,
  - allowable foundation bearing capacity and expected total and differential settlements,
  - concrete slabs-on-grade,
  - lateral earth pressures and coefficients,
  - mitigation of the potential corrosivity of site soils to concrete and steel reinforcement,
  - seismic design parameters,
  - pavement structural sections,
  - seepage pit percolation rate.

Not Contained In This Report: Although available through Earth Systems Consultants Southwest, the current scope of our services does not include:

- A corrosive study to determine cathodic protection of concrete or buried pipes.
- An environmental assessment.
- Investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Seven exploratory borings were drilled to depths ranging from 16.5 to 51.5 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory testing. The borings were drilled on July 28, 1999, using 8-inch outside diameter hollow-stem augers, and powered by a CME 45 truck-mounted drilling rig. The location of the borings is approximate, established by pacing and sighting from existing topographic features. The approximate boring locations are shown on Figure 2, located in Appendix A.

Samples were obtained within the test borings using a Standard Penetration (SPT) sampler (ASTM D 1586) and with a Modified California (MC) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586). The SPT sampler has a 2-inch outside diameter and 1.38-inch inside diameter. The MC sampler has a 3-inch outside diameter and a 2.37-inch inside diameter. The samples were obtained by driving the sampler with a 140-pound downhole hammer dropping 30 inches in general accordance with ASTM D 1586. Recovered soil samples were sealed in containers and returned to the laboratory. Bulk samples were also obtained from auger cuttings, representing a mixture of soils encountered for the depths noted.

The final logs of the borings represent our interpretation of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface investigation. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types although the transitions, however, may be gradational.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing were considered representative of soils that would be exposed and used during grading, and those deemed to be within the influence of the proposed structure. Test results are presented in graphic and tabular form in Appendix B of this report. The tests were conducted in general accordance with the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below.

Our laboratory-testing program consisted of the following tests:

- In-situ Moisture Content and Unit Dry Weight for the ring samples (ASTM D 2937).
- Maximum density tests were performed to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557-91).
- Particle Size Analysis (ASTM D422) to classify and evaluate soil composition. The gradation characteristics of selected samples were made by hydrometer and sieve analysis procedures.

- Consolidation (Collapse Potential) (ASTM D2435) to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil.

- Chemical Analyses (Soluble Sulfates & Chlorides, pH, and Electrical Resistivity) to evaluate the corrosivity of the soil on concrete and steel.

2.3 Percolation Testing

Two percolation tests were made on July 30, 1999, in the vicinity of the proposed seepage pits as shown in Figure 2. The percolation tests were made in general conformance to San Bernardino County percolation report standards.

The tests were performed within 8-inch diameter, dry-augered boreholes made to depths of 30 feet below existing ground surface and backfilled to a 20 foot depth. Sections of 3-inch perforated PVC pipe were set inside the augers prior to extracting the augers. The annulus space between the auger hole and PVC pipe was backfilled with 3/8-inch (pea) gravel to prevent caving of the boreholes. The porosity (specific yield) of the gravel was determined and a gravel pack correction factor was determined using the specific yield of the gravel. This correction factor was applied to the reported percolation rates.

The boreholes were filled with water and presaturated a minimum of overnight. Successive readings of drop in water level were made over several, 10-minute periods until a stabilized drop was recorded.

The field percolation test results and calculations, including the gravel pack correction factor are included in Appendix C. The individual absorption rates at each percolation test location are tabulated in the table below. The percolation rates indicate a uniform soil condition.

Test No	Percolation Rate (gal/sf/day)
P-1 (Boring B-3)	2
P-2 (Boring B-4)	3

Therefore, the leach line system for sanitary waste may be designed for an application rate of 2 gallons per square foot of effective absorption surface per day, per San Bernardino County Soil Percolation Report Standards.

Based on the data presented, it is our opinion the project site has the ability to percolate liquid waste without creating a nuisance or contaminating groundwater, provided the seepage pit locations are selected in areas of proven, acceptable percolation.

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration indicates that site soils consist primarily of medium dense to very dense silty sand and sand. The boring logs provided in Appendix A include detailed descriptions of the soils encountered. Soils should be readily cut by normal grading equipment.

In arid climatic regions, granular soils may have a potential to collapse upon wetting. Collapse (hydroconsolidation) may occur from the lubrication of soluble cements (carbonates) in the soil matrix causing the soil to densify from its loose configuration during deposition. A Consolidation test in the upper medium dense, silty sand indicates 8% collapse upon inundation and is considered a moderately severe site risk.

3.2 Groundwater

Free groundwater was not encountered in the borings during exploration. The depth to groundwater in the area is believed to be in excess of 100 feet. Groundwater should not be a factor in design or construction.

3.3 Geologic Setting

The project site lies at an elevation approximately 3,600 feet above mean sea level in the Morongo Basin region of the California Mojave Desert. The site lies at the base of an alluvial valley north of the Little San Bernardino Mountains that consists of Mesozoic plutonic rocks. The alluvial soils are older Quaternary deposits (undifferentiated) believed to be greater than 100 feet deep at this site.

3.4 Geologic Hazards

Geologic hazards that may affect the region include seismic hazards (surface fault rupture, ground shaking, soil liquefaction, and other secondary earthquake-related hazards), slope instability, flooding, ground subsidence, and erosion. A discussion follows on the specific hazards to this site.

3.4.1 Seismic Hazards

<u>Seismic Sources:</u> Our research of regional faulting indicates that 29 known active faults or seismic zones lie within 62 miles of the project site as shown on Table 1 in Appendix A. The Maximum Magnitude Earthquake (M_{max}) listed was taken from published geologic information available for each fault (CDMG, 1996). The M_{max} corresponds to the maximum earthquake believed to be tectonically possible.

The primary seismic hazard to the project site is strong ground shaking from earthquakes along the San Andreas and San Jacinto Faults. A further discussion of site acceleration from ground shaking follows in Section 3.4.3.

Surface Fault Rupture: **The project site lies within a currently delineated State of California, *Alquist- Priolo* Earthquake Fault Zone.** (Hart, 1994). The known trace of the recently (1992) activated Burnt Mountain Fault lies about 300 feet east of the gymnasium site (CDMG FER-230). Therefore, active fault rupture may occur at the project site. While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations. It is beyond the scope of this geotechnical report at this time to provide a detailed geologic fault hazard report.

Historic Seismicity: Five historic seismic events (5.9 M or greater) have significantly affected the Morongo Basin this century. They are as follows:

- *Desert Hot Springs Earthquake* - On December 4, 1948, a magnitude 6.5 M_L (6.0M_W) earthquake occurred east of Desert Hot Springs.
- *Palm Springs Earthquake* - A magnitude 5.9 M_L (6.2M_W) earthquake occurred on July 8, 1986 in the Painted Hills causing minor surface creep of the Banning segment of the San Andreas Fault.
- *Desert Hot Springs Earthquake* - On April 22, 1992, a magnitude 6.1 M_L (6.1M_W) earthquake occurred in the mountains 9 miles east of Desert Hot Springs. Structural damage and minor injuries occurred in the Palm Springs area as a result of this earthquake.
- *Landers & Big Bear Earthquakes* - Early on June 28, 1992, a magnitude 7.5 M_S (7.3M_W) earthquake occurred near Landers, the largest seismic event in Southern California for 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.6 M_S (6.4M_W) earthquake occurred near Big Bear Lake. Significant structural damage from these earthquakes occurred in Yucca Valley.

3.4.2 Secondary Hazards

Secondary seismic hazards related to ground shaking include soil liquefaction, ground deformation, areal subsidence, tsunamis, and seiches. The site is far inland so the hazard from tsunamis is non-existent. At the present time, no water storage reservoirs are located in the immediate vicinity of the site. Therefore, hazards from seiches are considered negligible at this time.

Soil Liquefaction: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. The potential for liquefaction to occur at this site is considered negligible because the depth of groundwater beneath the site exceeds 50 feet. No free groundwater was encountered in our exploratory borings.

Ground Deformation and Subsidence: Non-tectonic ground deformation consists of cracking of the ground with little to no displacement. This type of deformation is not caused by fault rupture. Rather it is generally associated with differential shaking of two or more geologic units with differing engineering characteristics. Liquefaction may also cause ground deformation. As the site is flat with consistent geologic material, and has a low potential for liquefaction, the potential for ground deformation is also considered to be low.

The potential for seismically induced ground subsidence is considered to be low at the site. Dry sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, ground shaking (cyclic shear strain), and earthquake duration (number of strain cycles). Fill areas may be susceptible to seismically induced settlement.

Slope Instability: The site area is gently sloping. Therefore, potential hazards from slope instability, landslides, or debris flows are considered low.

3.4.3 Site Acceleration and UBC Seismic Coefficients

Site Acceleration: To assess the potential intensity of ground motion, we have estimated the horizontal peak ground acceleration (PGA). Included in Table 1 are deterministic estimates of site acceleration from possible earthquakes at nearby faults. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Accelerations also are dependent upon attenuation by rock and soil deposits, direction of rupture, and type of fault. For these reasons, ground motions may vary considerably in the same general area. This variability can be expressed statistically by a standard deviation about a mean relationship.

The PGA is an inconsistent scaling factor to compare to the UBC Z factor and is generally a poor indicator of potential structural damage during an earthquake. Important factors influencing the structural performance are the duration and frequency of strong ground motion, local subsurface conditions, soil-structure interaction, and structural details. Because of these factors, an effective peak acceleration (EPA) is used in structural design.

The following table provides the probabilistic estimate of the PGA and EPA taken from the 1996 CDMG/USGS seismic hazard maps.

Estimate of PGA and EPA from 1996 CDMG/USGS Probabilistic Seismic Hazard Maps

Risk	Equivalent Return Period (years)	PGA (g) (1)	Approximate EPA (g) (2)
10% exceedance in 50 years	475	0.54	0.48

Notes:

1. Based on soft rock site, Site Class $S_{B/C}$
2. Spectral acceleration (S_A) at period of 0.3 seconds divided by 2.5 factor for 5% damping as defined by the Structural Engineers Association of California (SEAOC, 1996).

UBC Seismic Coefficients: The Uniform Building Code (UBC) seismic coefficients are based on a Design Basis Earthquake (DBE) that has an earthquake ground motion with a 10% probability of occurrence in 50 years. The UBC seismic force provisions should be regarded as a *minimum* design in that it allows for inelastic yielding of structures. The UBC design criteria permit structural damage and possible loss of use after an earthquake. The PGA and EPA estimates given above are provided for information on the seismic risk inherent in the UBC design.

The following table lists the relevant seismic and site coefficients given in Chapter 16 of the 1994 and 1997 Uniform Building Code (UBC). The 1997 UBC seismic provisions are more stringent for areas less than 10 km (6.2 miles) from major seismic sources.

UBC Seismic Coefficients for Chapter 16 Seismic Provisions

UBC Code Edition	Soil Profile Type	Seismic Source Type	Distance to Critical Source	Near Source Factors		Seismic Coefficients	
				Na	Nv	Ca	Cv
1994	S_3 S factor =1.5	---	---	---	---	Z = 0.4	Z =0.4
Ref. Table	16-J	---	---	---	---	16-I	16-I
1997	S_C (dense soil)	B	0.1 km	1.3	1.6	0.44Na = 0.52	0.64Nv = 0.90
Ref. Table	16-J	16-U	---	16-S	16-T	16-Q	16-R

Section 4
CONCLUSIONS

The following is a summary of our conclusions and professional opinions based on the data obtained from a review of selected technical literature and the site evaluation.

- The primary geologic hazard relative to site development is severe ground shaking from earthquakes originating on nearby faults. In our opinion, a major seismic event from reactivation of the Burnt Mountain Fault or rupture of the Pinto Mountain or San Andreas Faults would be the most likely cause of significant earthquake activity at the site within the estimated design life of the proposed development.

- The project site lies within an A-P Earthquake Fault Hazard Zone. Further study of the fault rupture hazard may be warranted.

- The project site is in seismic Zone 4 as defined in the Uniform Building Code. A qualified professional who is aware of the site seismic setting should design any permanent structure constructed on the site.

- Ground subsidence from seismic events or hydroconsolidation is a potential hazard in the Morongo Basin area. Adherence to the following grading and structural recommendations should reduce potential settlement problems from seismic forces, heavy rainfall or irrigation, flooding, and the weight of the intended structures.

- The soils are susceptible to wind and water erosion. Preventative measures to minimize seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction.

- Other geologic hazards including liquefaction, seismically induced flooding, and landslides are considered low or negligible on this site.

- The upper soils were found to be relatively variable medium dense to very dense. In our opinion, the soils within the building area will require over excavation and recompaction to improve bearing capacity and reduce settlement from static loading.

- We recommend that Earth Systems Consultants Southwest (ESCSW) be retained to provide Geotechnical Engineering services during project design, site development, excavation, grading, and foundation construction phases of the work. This is to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

- Plans and specifications should be provided to ESCSW prior to grading. Plans should include the grading plans, foundation plans, and foundation details. Preferably, structural loads should be shown on the foundation plans.

Section 5
RECOMMENDATIONS

SITE DEVELOPMENT AND GRADING

5.1 Site Development - Grading

A representative of ESCSW should observe site grading and the bottom of excavations prior to placing fill. Local variations in soil conditions may warrant increasing the depth of recompaction and over-excavation.

Clearing and Grubbing: Prior to site grading any existing vegetation, large roots, construction debris, trash, and any abandoned underground utilities should be removed from the proposed building and pavement areas. The surface should be stripped of organic growth along with other debris and removed from the construction area. Areas disturbed during clearing should be properly backfilled and compacted as described below.

Building Pad Preparation: Because of the severe potential for hydroconsolidation and under-compacted nature of the majority of the upper site soils, we recommend recompaction of soils in the building area. The existing surface soils within the building pad areas should be over-excavated to 48 inches below existing grade or a minimum of 36 inches below the footing level **(whichever is lower)**. The over-excavation should extend for 5 feet beyond the outer edge of exterior footings. The bottom of the sub-excavation should be scarified; moisture conditioned, and recompacted to at least 90% relative compaction (ASTM D1557) for a depth of an additional 12 inches.

Subgrade Preparation: In areas to receive fill, pavements or hardscape, the ground surface should be scarified; moisture conditioned, and compacted to at least 90% relative compaction (ASTM D1557) for a depth of 12 inches below finished subgrades. **Compaction should be verified by testing.**

Engineered Fill Soils: The native granular soil is suitable for use as engineered fill and utility trench backfill. The native soil should be placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D1557) near its optimum moisture content. **Compaction should be verified by testing.** All rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material.

Imported fill soils (if required) should be non-expansive, granular soils meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35% passing the No. 200 sieve. The geotechnical engineer should evaluate the import fill soils before hauling to the site. However, because of potential variations within the borrow site, import soil will not prequalified by ESCSW. The imported fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to at least 90% relative compaction (ASTM D1557) near optimum moisture content.

Shrinkage: The shrinkage factor for earthwork is expected to range from 5 to 25 percent for the upper excavated or scarified ***site*** soils. This estimate is based on compactive effort to achieve an average relative compaction of about 92% and may vary with contractor methods.

Subsidence is estimated to range from 0.1 to 0.2 feet. Losses from site clearing and removal of existing site improvements may affect earthwork quantity calculations and should be considered.

Site Drainage: Positive drainage should be maintained away from the structures (5% for 5 feet minimum) to prevent ponding and subsequent saturation of the foundation soils. Gutters and downspouts should be considered as a means to convey water away from foundations if adequate drainage is not provided. Drainage should be maintained for paved areas. Water should not pond on or near paved areas.

5.2 Excavations and Utility Trenches

Excavations should be made in accordance with CalOSHA requirements. From our site exploration and knowledge of the general area, we believe there is a potential for caving of site excavations (utilities, footings, etc.). Excavations within sandy soil should be kept moist, but not saturated, to reduce the potential of caving or sloughing. Where deep excavations over 4 feet deep are planned, lateral bracing or appropriate cut slopes of 1.5:1 (horizontal: vertical) should be provided.

No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope, equal to the depth of the excavation.

Utility Trenches: Backfill of utilities within road or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, road department, etc.) Utility trench backfill within private property should be placed in conformance with the provisions of this report. In general, service lines extending inside of property may be backfilled with native soils compacted to a minimum of 90% relative compaction.

Backfill operations should be observed and tested by ESCSW to monitor compliance with these recommendations.

5.3 Slope Stability of Graded Slopes

No unprotected permanent graded slopes should be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

Slope stability calculations were not performed because of the expected minimal slope height (less than 5 feet). If slopes heights exceed 5 feet, engineering calculations should be performed to evaluate the stability of 2 to 1, horizontal to vertical, slopes. Fill slopes should be overfilled and trimmed back to competent material.

STRUCTURES

In our professional opinion, the structure foundation can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on very low expansion category soils.

5.4 Foundations

Footing design widths, depths, and reinforcing are the responsibility of the Structural Engineer. Footings should be designed for structural considerations and the geotechnical conditions described in this report. A minimum footing depth of 18 inches below lowest adjacent grade should be maintained.

Conventional Spread Foundations: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous wall foundations, 12-inch minimum width and 18 inches below grade:
  2000 psf for dead plus design live loads.

- Isolated pad foundations, 2 x 2 foot minimum in plan and 18 inches below grade:
  2400 psf for dead plus design live loads.

Allowable increases of 400 psf per each foot of additional footing width and 400 psf for each additional 0.5 foot of footing depth may be used. (A one-third increase in bearing pressure may be used for wind and seismic loading). The maximum allowable bearing pressure should be limited to 4000 psf. The allowable bearing values indicated have been determined based upon the anticipated maximum loads indicated in Section 1.1 of this report. If the indicated loading is exceeded then the geotechnical engineer must reevaluate the allowable bearing values and the grading requirements.

Minimum reinforcement for continuous wall footings should be two, No. 4 steel reinforcing bars, split between the top and the bottom of the footing. This reinforcing is not intended to supersede any structural requirements provided by the structural engineer.

Foundation excavations should be observed by a representative of ESCSW during excavation and prior to placement of reinforcing steel or concrete. Local variations in conditions may require deepening of footings.

Expected Settlement: Estimated total static settlement, based on footings founded on firm soils as recommended, should be less than 1 inch. Differential settlement between exterior and interior bearing members should be less than 1/2-inch.

Frictional and Lateral Coefficients: Lateral loads may be resisted by soil friction on the base of foundations and by passive resistance of the soils acting on foundation stem walls. Lateral capacity is based partially on the assumption that any required backfill adjacent to foundations and grade beams is properly compacted.

An allowable coefficient of friction of 0.35 may be used for dead load forces. An allowable equivalent fluid pressure of 250 pcf may be included for resistance to lateral loading. These values include a factor of safety of 1.5. Passive resistance and frictional resistance may be combined in determining the total lateral resistance. However, the friction factor should be reduced to 0.23 of dead load forces. A one-third (1/3) increase in the passive pressure may be used when calculating resistance to wind or seismic loads.

5.5 Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted soil placed in accordance with Section 5.1 of this report.

Vapor Barrier: In areas of moisture sensitive floor coverings, an appropriate vapor barrier should be installed in order to minimize moisture transmission from the subgrade soil to the slab. We recommend that an impermeable membrane (6-mil visqueen) underlie the floor slabs. The membrane should be covered with 2 inches of sand to help protect it during construction and to aide in concrete curing. The sand should be lightly moistened just prior to placing the concrete. Low-slump concrete should be used to help minimize shrinkage.

Slab thickness and reinforcement: Slab thickness and reinforcement of slab-on-grade are contingent upon the structural engineer or architect's recommendations and the expansion index of the supporting soil. Based upon our findings, a modulus of subgrade reaction of approximately 200 pounds per cubic inch can be used in concrete slab design.

Concrete slabs and flatwork should be a minimum of 4 inches thick. We suggest minimum reinforcement for concrete slabs consist of a minimum of No. 3 rebars at 18-inch centers, both horizontal directions, placed at slab mid-height to resist swell forces and cracking. Concrete floor slabs may either be monolithically placed with the foundations or doweled after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or geotechnical engineer should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab.

Control Joints: Control joints should be provided in all concrete slabs-on-grade at a maximum spacing of 36 times the slab thickness (12 feet maximum on-center, each way) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce the potential for randomly oriented, contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or saw cut (1/4 of slab depth) within 8 hours of concrete placement. Construction (cold) joints should either be thickened butt joints with one-half inch dowels at 24-inches on center or a thickened keyed-joint to resist vertical deflection at the joint. All construction joints in exterior flatwork should be sealed to prevent moisture or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region. These procedures will reduce the potential for randomly oriented cracks, however, may not eliminate them from occurring.

5.6 Retaining Walls

The following table presents lateral earth pressures for use in retaining wall design. The values are given as equivalent fluid pressures without surcharge loads or hydrostatic pressure.

Lateral Pressures and Sliding Resistance (1)	Granular Backfill
Passive Pressure	375 pcf
Active Pressure (cantilever walls) able to rotate 0.1% of structure height	35 pcf
At-Rest Pressure (restrained walls)	55 pcf
Dynamic Lateral Earth Pressure (2) acting at mid height of structure, where H is height of backfill in feet	30H psf
Base Lateral Sliding Resistance Dead load X Coefficient of Friction:	0.55

Notes:

1. These values are ultimate values. A factor of safety of 1.5 should be used in stability analysis except for dynamic earth pressure where a factor of safety of 1.2 is acceptable.
2. Dynamic pressures are based on the Mononobe-Okabe 1929 method, additive to active earth pressure. Walls retaining less than 6 feet of soil need not consider this increased pressure.

Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 35% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to 2 feet of native soil.

Drainage: A backdrain or an equivalent system of backfill drainage should be incorporated into the retaining wall design. Backfill immediately behind the retaining structure should be a free-draining granular material. In this case, the native soils are considered free draining. Waterproofing should be per the Architect's specifications. Water should not be allowed to pond near the top of the wall. To accomplish this, the final backfill grade should be such that all water is diverted away from the retaining wall.

Backfill Compaction: Compaction on the retained side of the wall within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment.

5.7 Mitigation of Soil Corrosivity on Concrete

Selected chemical analyses for corrosivity were conducted on samples at the project site. The native soils were found to have negligible sulfate ion concentration and low chloride ion concentrations. Sulfate ions can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. Chloride ions can cause corrosion of reinforcing steel. The Uniform Building Code does not require any special provisions for concrete for these low concentrations as tested. Normal concrete mixes may be used.

A minimum concrete cover of 3 inches should be provided around steel reinforcing or embedded components exposed to native soil or landscape water (to 18 inches above grade). Additionally, the concrete should be thoroughly vibrated during placement.

Laboratory testing of the soil suggests that the site soils may present a moderate potential for metal loss from electrochemical corrosion processes. Corrosion protection of steel pipes can be achieved by using epoxy corrosion inhibitors; asphalt coatings, cathodic protection, or encapsulating with densely consolidated concrete. A qualified corrosion engineer should be consulted regarding mitigation of the corrosive effects of site soils on metals.

5.8 Seismic Design Criteria

This site is subject to strong ground shaking due to potential fault movements along the San Andreas and San Jacinto Faults. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. The ***minimum*** seismic design should comply with the latest edition of the Uniform Building Code for Seismic Zone 4 using the seismic coefficients given in Section 3.4.3 of this report. The 1997 UBC seismic provisions are more stringent for sites lying close to major faults.

The UBC seismic coefficients are based on scientific knowledge, engineering judgment, and compromise. Factors that play an important role in dynamic structural performance are:

(1) effective peak acceleration (EPA),
(2) duration and predominant frequency of strong ground motion,
(3) the period of the structure,
(4) soil-structure interaction,
(5) total resistance capacity of the system,
(6) redundancies,
(7) inelastic load-deformation behavior, and
(8) the modification of damping and effective period as structures behave inelastically.

Factors 5 to 8 are accounted by the structural ductility factor (R) used in deriving a reduced value for design base shear. If further information on seismic design is needed, a site-specific probabilistic seismic analysis should be conducted.

The intent of the UBC lateral force requirements is to provide a structural design that will resist collapse to provide reasonable life safety from a major earthquake but may experience some structural and nonstructural damage. A fundamental tenet of seismic design is that inelastic yielding is allowed to adapt to the seismic demand on the structure. In other words, *damage is* *allowed.* The UBC lateral force requirements should be considered as a *minimum* design criterion. The owner and the designer should evaluate the level of risk and performance that is acceptable. Performance based criteria could be set in the design. The design engineer has the responsibility to interpret and adapt the principles of seismic behavior and design to each structure using experience and sound judgment. The design engineer should exercise special care so that all components of the design are all fully met with attention to providing a continuous load path. An adequate quality assurance and control program is urged during project construction to verify that the design plans and good construction practices are followed. This is especially important for sites lying close to the major seismic sources.

5.9 Pavements

Since no traffic, loadings were provided by the design engineer or owner, we have assumed traffic loading for comparative evaluation. The design engineer or owner should decide the appropriate traffic conditions for the pavements. Maintenance of proper drainage is necessary to prolong the service life of the pavements. The following table provides our recommendations for pavement sections.

RECOMMENDED PAVEMENTS SECTIONS

R-Value Subgrade Soils - 50 (assumed) Design Method - CALTRANS 1995

		Flexible Pavements		Rigid Pavements	
Traffic Index (assumed)	**Pavement Use**	**Asphaltic Concrete Thickness (in.)**	**Aggregate Base Thickness (in.)**	**Portland Cement Concrete (in.)**	**Aggregate Base Thickness (in.)**
4.0	Auto Parking Areas	2.5	4.0	4.0	4.0

Notes:

1. Asphaltic concrete should be Caltrans, Type B, ½-in. or ¾-in. maximum-medium grading, compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D1559) or equivalent.
2. Aggregate base should be Caltrans Class 2 (3/4 in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3. All pavements should be placed on 8 inches of moisture-conditioned subgrade, compacted to a minimum of 90% of ASTM D1557 maximum dry density.
4. Portland cement concrete should have a minimum of 3250 psi compressive strength @ 28 days.
5. Equivalent Standard Specifications for Public Works Construction (Greenbook) may be used instead of Caltrans specifications for asphaltic concrete and aggregate base.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until construction. Variations in soil or groundwater may require additional studies, consultation, and possible revisions to our recommendations.

Findings of this report are valid as of the issued date of the report. However, changes in conditions of a property can occur with passage of time whether they are from natural processes or works of man on this or adjoining properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

In the event that any changes in the nature, design, or location of the building are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or verified in writing.

This report is issued with the understanding that the owner, or his representative, has the responsibility that the information and recommendations contained herein are brought to the attention of the architect and engineers for the project and are incorporated into the plans and specifications for the project. The owner, or his representative, also has the responsibility to take the necessary steps to see that the general contractor and all subcontractors carry out such recommendations in the field. It is further understood that the owner or his representative is responsible for submittal of this report to the appropriate governing agencies.

As the Geotechnical Engineer of Record for this project, ESCSW has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee is express or implied. This report was prepared for the exclusive use of the Client and their authorized agents

ESCSW should be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If ESCSW is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations.

Although available through Earth Systems Consultants Southwest, the current scope of our services does not include an environmental assessment, or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air on, below, or adjacent to the subject property. Prior to purchase or development of this site, we suggest that an environmental assessment be conducted which addresses environmental concerns.

6.2 Additional Services

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining ESCSW as the geotechnical consultant from beginning to end of the project will provide continuity of services. *The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.*

Construction monitoring and testing would be additional services provided by our firm. The costs of these services are not included in our present fee arrangements, but can be obtained from our office. The recommended review, tests, and observations include, but are not necessarily limited to the following:

- Consultation during the final design stages of the project.
- Review of the building and grading plans to observe that recommendations of our report have been properly implemented into the design.
- Observation and testing during site preparation, grading and placement of engineered fill as required by UBC Sections 1701 and 3317 or local grading ordinances.
- Consultation as required during construction

-o0o-

Appendices as cited are attached and complete this report

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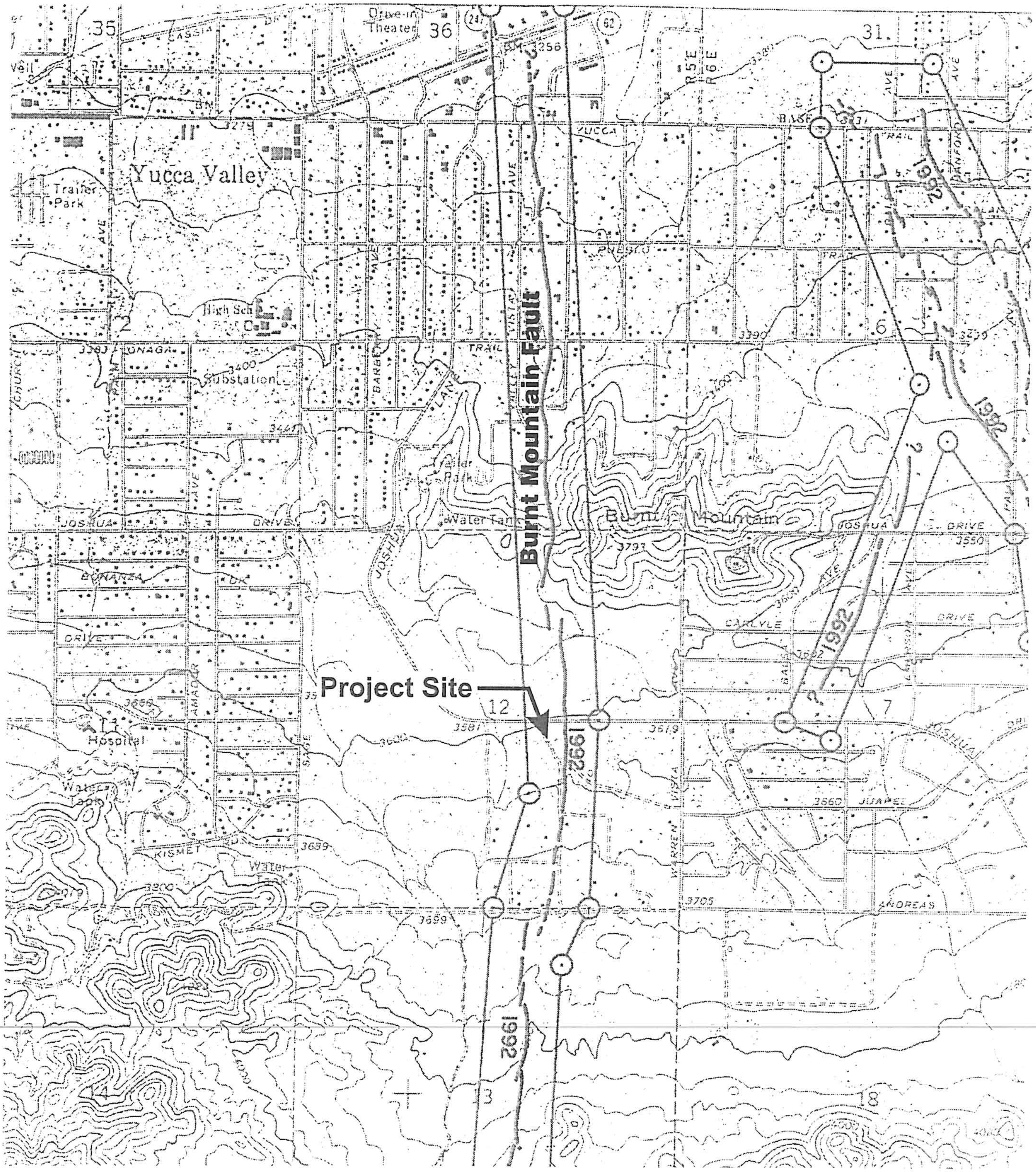
APPENDIX A

Site Location and Boring Locations
Table 1 Fault Parameters
Logs of Borings



Reference: Yucca Valley South Quadrangle 7.5 min. Earthquake Fault Map (1993)

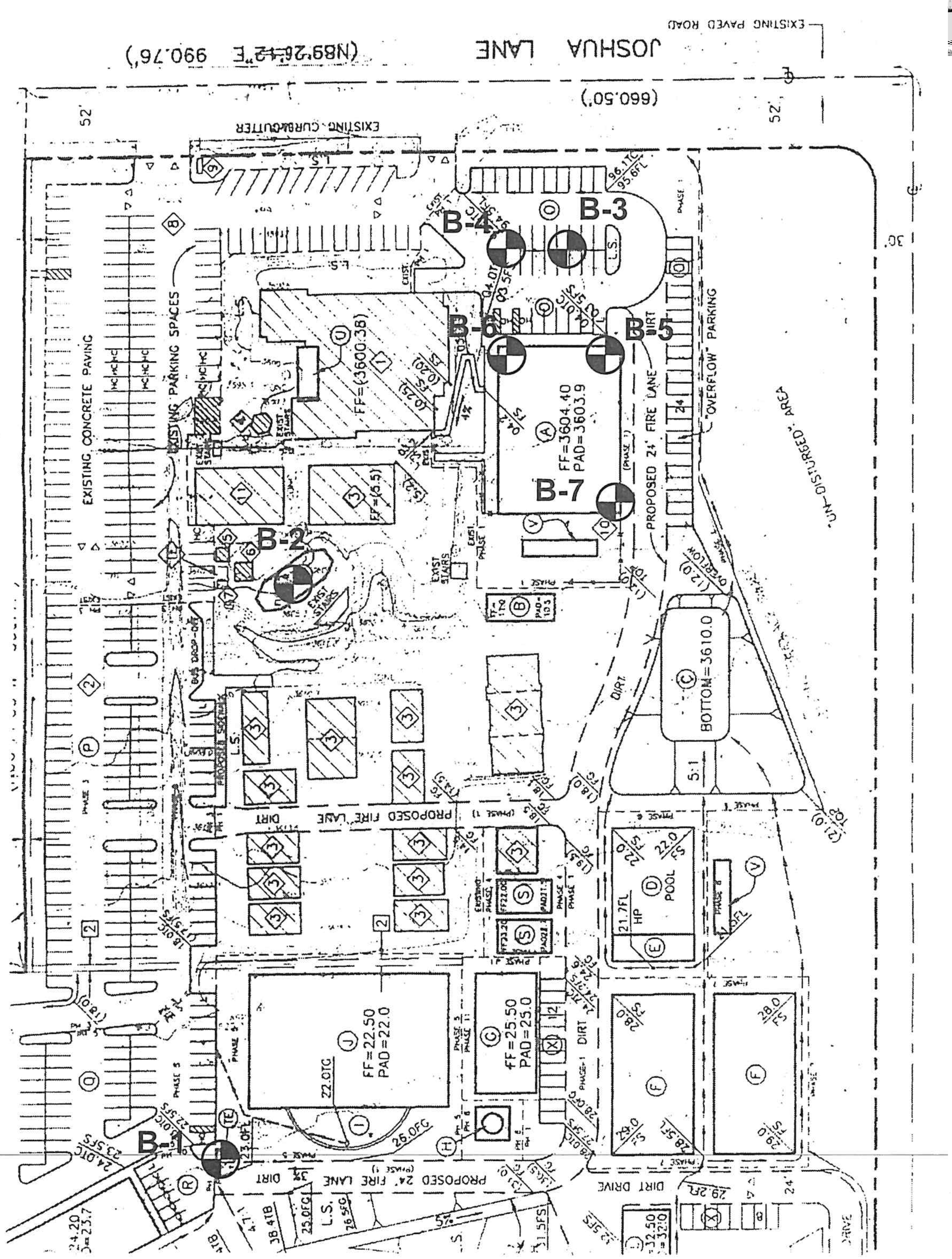
Scale: 1" = 2,000'

0 2,000 4,000

Figure 1 - Site Location

Project Name: Joshua Springs Christian School
Project No.: 07278-01

Earth Systems Consultants
Southwest



LEGEND

◒ Approximate Boring Location

Scale: 1" = 120'

0 120 240

Figure 2- Boring Locations

Project Name: Joshua Springs Christian S
Project No.: 07278-01

Earth Systems Consult
Southwest

Earth Systems Consultants
Southwest
79-811B Country Club Drive, Bermuda Dunes, CA 92201
Phone (760) 345-1588 FAX (760) 345-7315

Boring No: B-2

Project Name: Joshua Springs Christian School
Project Number: 07278-01
Boring Location: See Figure 2

Drilling Date: July 28,1999
Drilling Method: 8-inch Hollow Stem Auger
Drill Type: CME 45
Logged By: Cliff Batten

Page 1 of 1

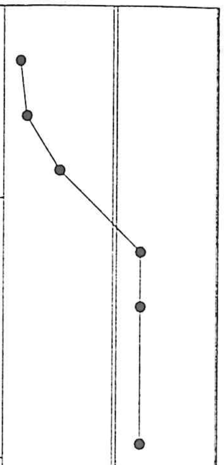
Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			■	5,6,7	SM	112.0	4.5	SILTY SAND: Red brown; medium dense; damp; fine to coarse grained.
			■	7,8,10		111.3	1.9	
5		◧		10,13,15				
		◧		32,41,48	SP			SAND: Red brown; dense to very dense; dry to damp; fine to coarse grained, micaceous.
10		◧		33, 50 for 4"				
15		◧		48, 50 for 2"				

Total Depth: 16.5 feet
No groundwater or bedrock encountered.

Graphic Trend
Blow Count Dry Density



Boring No: B-3

Project Name: Joshua Springs Christian School
Project Number: 07278-01
Boring Location: See Figure 2

Drilling Date: July 28,1999
Drilling Method: 8-inch Hollow Stem Auger
Drill Type: CME 45
Logged By: Cliff Batten

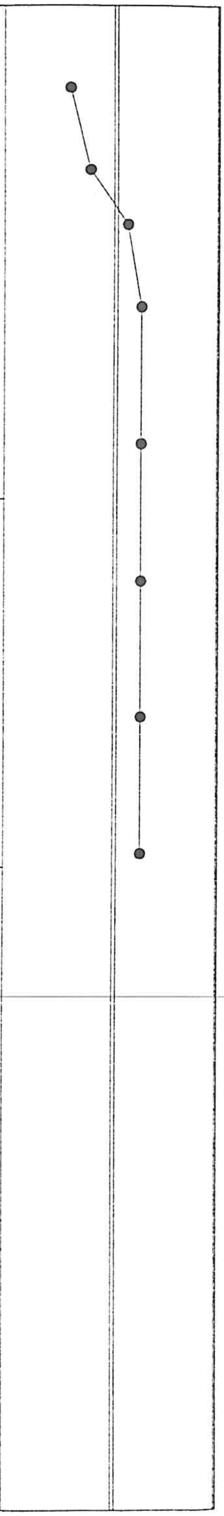
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	SPT	MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SM			SILTY SAND: Red brown; dense to very dense; dry to damp; fine to medium grained.
			■	14,22,27			104.8	1.8	
5			■	17,27,36			105.6	1.8	
		◨		21,22,33					
10		◨		41,50 for 2"					
15		◨		50 for 5"					
						SP			SAND: Red brown; very dense; dry; fine to coarse grained, micaceous.
20		◨		50 for 5"					
25		◨		50 for 5"					
30		◨		50 for 5"					
									Total Depth: 31.5 feet No groundwater or bedrock encountered.
35									
40									
45									
50									
55									

Graphic Trend
Blow Count Dry Density



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Boring No: B-4

Project Name: Joshua Springs Christian School
Project Number: 07278-01
Boring Location: See Figure 2

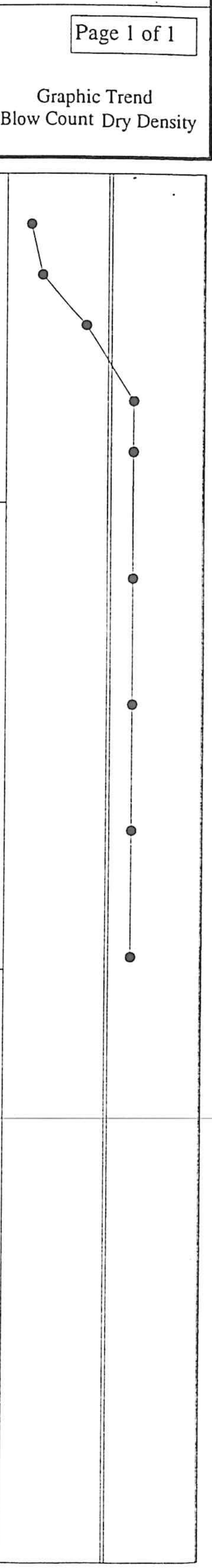
Drilling Date: July 28,1999
Drilling Method: 8-inch Hollow Stem Auger
Drill Type: CME 45
Logged By: Cliff Batten

Description of Units

Page 1 of 1

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	Sample Type SPT	Sample Type MOD Calif.	Penetration Resistance (Blows/6")	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0					SM			SILTY SAND: Red brown; medium dense to very dense; dry to damp; fine to medium grained.
			■	7,9,11		107.3	2.0	
			■	12,14,16		103.0	2.0	
5		◧		13.16,19				
		◧		29,31,53				
10		◧		40,42,50 for 5"				
					SP			SAND: Red brown; very dense; dry; fine to coarse grained, micaceous.
15		◧		31,36,42				
20		◧		50 for 6"				
25		◧		37,41,48				
30		◧		40,43,48				
								Total Depth: 31.5 feet No groundwater or bedrock encountered.
35								
40								
45								
50								
55								



Boring No: B-5
Project Name: Joshua Springs Christian School
Project Number: 07278-01
Boring Location: See Figure 2

Drilling Date: July 28,1999
Drilling Method: 8-inch Hollow Stem Auger
Drill Type: CME 45
Logged By: Cliff Batten

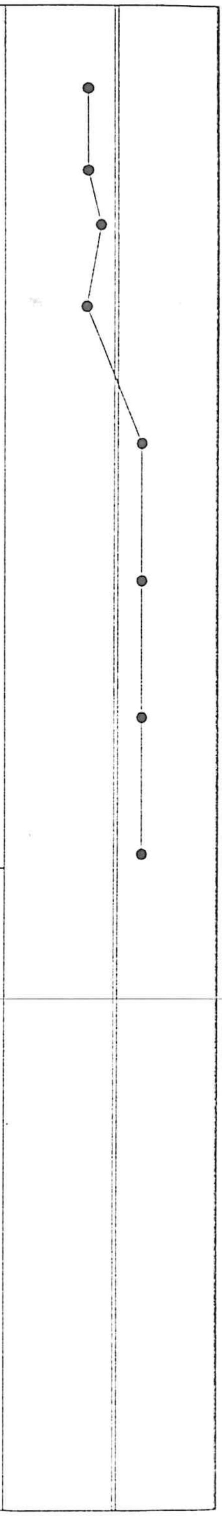
Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	SPT	MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0						SM			SILTY SAND: Red brown; dense to very dense; dry to damp; fine to coarse grained, micaceous.
			■	26,28,33			106.3	1.9	
5			■	30,30,31			106.0	1.7	
		■		17,19,22					
10		■		21,20,20					
									cobbles
15		■		40, 50 for 2"					
20		■		41,49,50					
									cobbles
25		■		33,33,35					
30		■		18,27,50					
									Total Depth: 31.5 feet No groundwater or bedrock encountered.
35									
40									
45									
50									
55									

Graphic Trend
Blow Count Dry Density



Boring No: B-6
Project Name: Joshua Springs Christian School
Project Number: 07278-01
Boring Location: See Figure 2

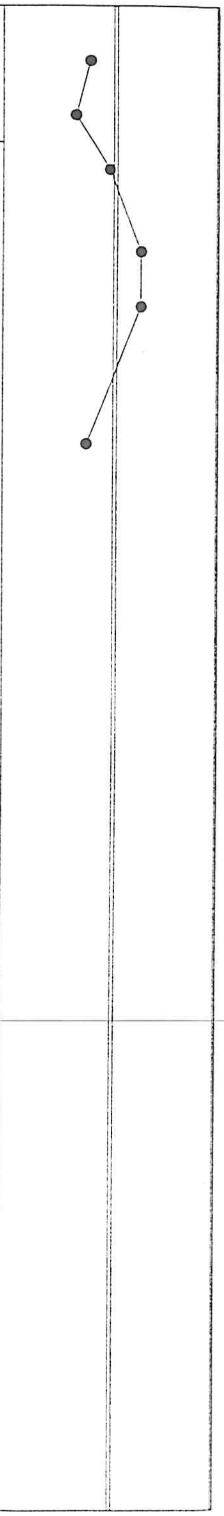
Drilling Date: July 28,1999
Drilling Method: 8-inch Hollow Stem Auger
Drill Type: CME 45
Logged By: Cliff Batten

Description of Units

Page 1 of 1

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type Bulk	SPT	MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0			■	6,27,36		SM	---	2.0	SILTY SAND: Red brown; dense; dry to damp; fine to medium grained.
			■	21,27,26			104.2	1.9	
5		◧		22,23,27		SP			SAND: Red brown; dense to very dense; damp; fine to coarse grained, micaceous.
		◧		16,27,40					
10		◧		21, 50 for 3"					
									- cobbles
15		◧		19,20,21					
20		◧		---					
25		◧		---					
30		◧		---					
35		◧		---					
40		◧		---					
45		◧		31,37,38					
50		◧		41,50 for 4"					
55									Total Depth: 51.5 feet No groundwater or bedrock encountered.



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Boring No: B-7

Project Name: Joshua Springs Christian School
Project Number: 07278-01
Boring Location: See Figure 2

Drilling Date: July 28,1999
Drilling Method: 8-inch Hollow Stem Auger
Drill Type: CME 45
Logged By: Cliff Batten

Page 1 of 1

Description of Units

Note: The stratification lines shown represent the approximate boundary between soil and/or rock types and the transition may be gradational.

Depth (Ft.)	Sample Type: Bulk	Sample Type: SPT	Sample Type: MOD Calif.	Penetration Resistance (Blows/6")	Symbol	USCS	Dry Density (pcf)	Moisture Content (%)	Description of Units
0	■				⁙	SM			SILTY SAND: Red brown; dense; dry to damp; fine to medium grained.
	■		■	19,22,28	⁙		104.4	2.0	
5			■	20,21,20	⁙		---	1.9	
		◧		24,24,30	▒	SP			SAND: Red brown; dense to very dense; damp; fine to coarse grained, micaceous.
10		◧		16,31,33	▒				
15		◧		17,18,26	▒				
20		◧		15,20,24	▒				
					▒				- cobbles
25		◧		37, 50 for 2"	▒				
30		◧		50 for 5"	▒				
35		◧		50 for 4"	▒				
40		◧		41,50 for 5"	▒				
45		◧		50 for 4"	▒				
50		◧		50 for 4"	▒				
55									Total Depth: 51.5 feet No groundwater or bedrock encountered.

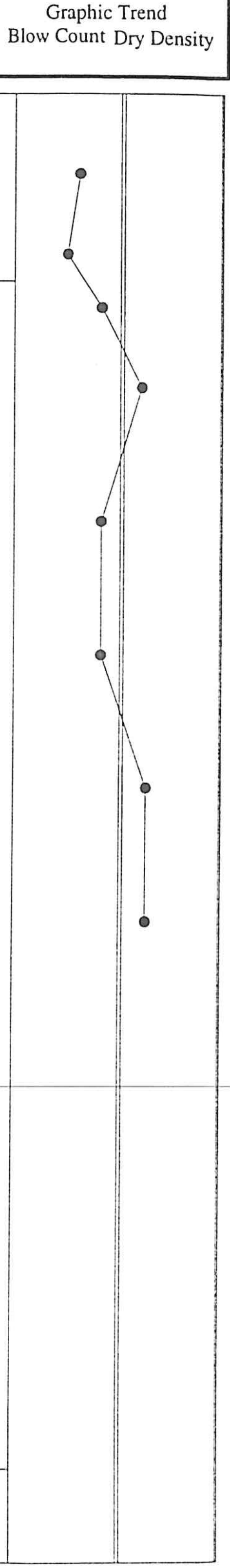


Table 1
Fault Parameters

& Deterministic Estimates of Mean Peak Ground Acceleration (PGA)

Fault Name or Seismic Zone	Distance (mi) & Direction from Site	UBC Fault Type	Fault Length (km)	Maximum Magnitude Mmax (Mw)	Geologic Slip Rate (SR) (mm/yr)	Average Return Period (yrs)	Date of Last Rupture (year)	Largest Historic Event >5.5M (year)	Est. mean Site PGA (g)
Reference Notes: (1)		(2)	(1)	(4)	(3)	(3)	(3)	(5)	(6)
Mojave Faults									
Burnt Mtn	0.1 E	B	20	**6.4**	0.6	5,000	1992	7.3 1992	**0.48**
Eureka Peak	1.7 ENE	B	19	**6.4**	0.6	5,000	1992	6.1 1992	**0.44**
Morongo	2.2 NNW	C	23	**6.5**	0.6	1,170		5.5 1947	**0.44**
Pinto Mountain	2.4 N	B	73	**7.0**	2.5	500			**0.50**
Landers	4.7 N	B	83	**7.3**	0.6	5,000	1992	7.3 1992	**0.45**
S. Emerson-Copper Mtn.	13 ENE	B	54	**6.9**	0.6	5,000			**0.21**
N. Johnson Valley	14 N	B	36	**6.7**	0.6	5,000			**0.17**
Blue Cut	16 SE	B	30	**6.8**	1	760			**0.17**
N. Frontal Fault Zone (E)	16 NNW	B	27	**6.7**	0.5	1,730			**0.18**
Bullion Mtn-Mesquite Lake	21 NE	B	88	**7.0**	0.6	5,000			**0.14**
Calico - Hidalgo	22 NNE	B	95	**7.1**	0.6	5,000			**0.15**
Lockhart-Old Wmn Spgs	22 NW	B	149	**7.3**	0.6	5,000			**0.16**
N. Frontal Fault Zone (W)	27 WNW	B	50	**7.0**	1	1,310			**0.13**
Helendale-S. Lockhart	32 WNW	B	97	**7.1**	0.6	5,000			**0.10**
Ludlow	39 ENE	B	23	**7.0**	0.6	5,000			**0.08**
Cleghorn	52 WNW	B	25	**6.5**	3	216			**0.04**
Mannix	60 NNW	B	14	**6.6**	0.6	5,000		5.9 1947	**0.04**
Gravel Hills-Harper Lake	61 NNW	B	66	**6.9**	0.6	5,000			**0.05**
San Andreas Fault System									
- San Bernardino Mtn	10 SW	A	107	**7.3**	24	433			**0.30**
- Coachella Valley	12 SSW	A	95	**7.4**	25	220	c. 1690	6.5 1948	**0.29**
- Banning	14 SSW	A	98	**7.4**	10	220	c. 1690	6.2 1986	**0.25**
- Whole S. Calif. Zone	10 SW		425	**7.9**	---	---	1857	7.8 1857	**0.39**
San Jacinto Fault System									
- Hot Spgs-Buck Ridge	33 SSW	C	70	**6.5**	2	354		6.3 1937	**0.07**
- San Jacinto Valley	37 WSW	B	42	**6.9**	12	83		6.8 1899	**0.08**
- Anza Segment	37 SSW	A	90	**7.2**	12	250	1918	6.8 1918	**0.09**
- Coyote Creek	44 S	B	40	**6.8**	4	175	1968	6.5 1968	**0.06**
- San Bernardino Seg.	48 W	B	35	**6.7**	12	100		6.0 1923	**0.05**
- Whole Zone	37 WSW		245	**7.5**	---	---			**0.12**
Elsinore Fault System									
- Temecula Segment	60 SW	B	42	**6.8**	5	240			**0.04**
- Julian Segment	60 SW	A	75	**7.1**	5	340			**0.05**
- Whole Zone	60 SW		250	**7.5**	---	---			**0.07**
Los Angeles Area Faults									
Cucamonga	59 W	B	24	**7.0**	5	650			**0.06**

Notes:

1. Jennings (1994) and CDMG (1996)
2. CDMG (1996), where Type A faults, Mmax > 7 and slip rate >5 mm/yr; Type C, Mmax<6.5, SR< 2mm/yr.
3. CDMG (1996) and WGCEP (1995)
4. CDMG (1996) based on Wells & Coppersmith (1994)
5. modified from Ellsworth Catalog (1990) in USGS Professional Paper 1515, Mw = moment magnitude,
6. The estimates of the mean Site PGA are based on the attenuation relationship of:
   Weighted average of Campbell & Bozorgnia; Boore, Joyner & Fumal; and Sadigh (1994)
   (mean plus sigma values are about 1.6 times higher)

APPENDIX B

Laboratory Test Results

07278-01 Aug 10, 1999

PARTICLE SIZE ANALYSIS

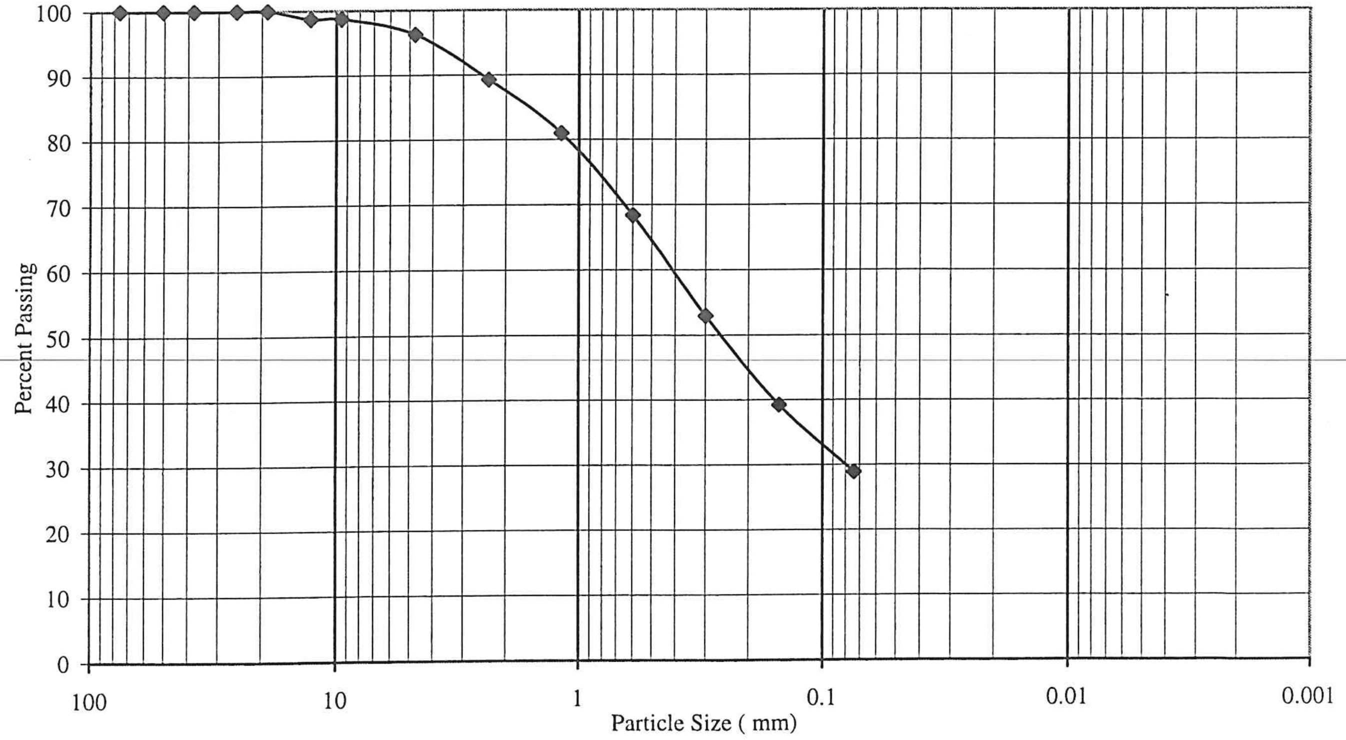
ASTM D-422

Job Name: Joshua Springs Calvary Chapel
Sample ID: **B-7 @ 0 to 5 feet**
Description: **Silty F to C Sand (SM), trace clay**

Sieve Size	Percent Passing
1-1/2"	100
1"	100
3/4"	100
1/2"	99
3/8"	99
#4	96
#8	89
#16	81
#30	68
#50	53
#100	39
#200	29

% Gravel:	**4**
% Sand:	**67**
% Silt:	**22**
% Clay (3 micron):	**7**

(Clay content by short hydrometer method)



07278-01

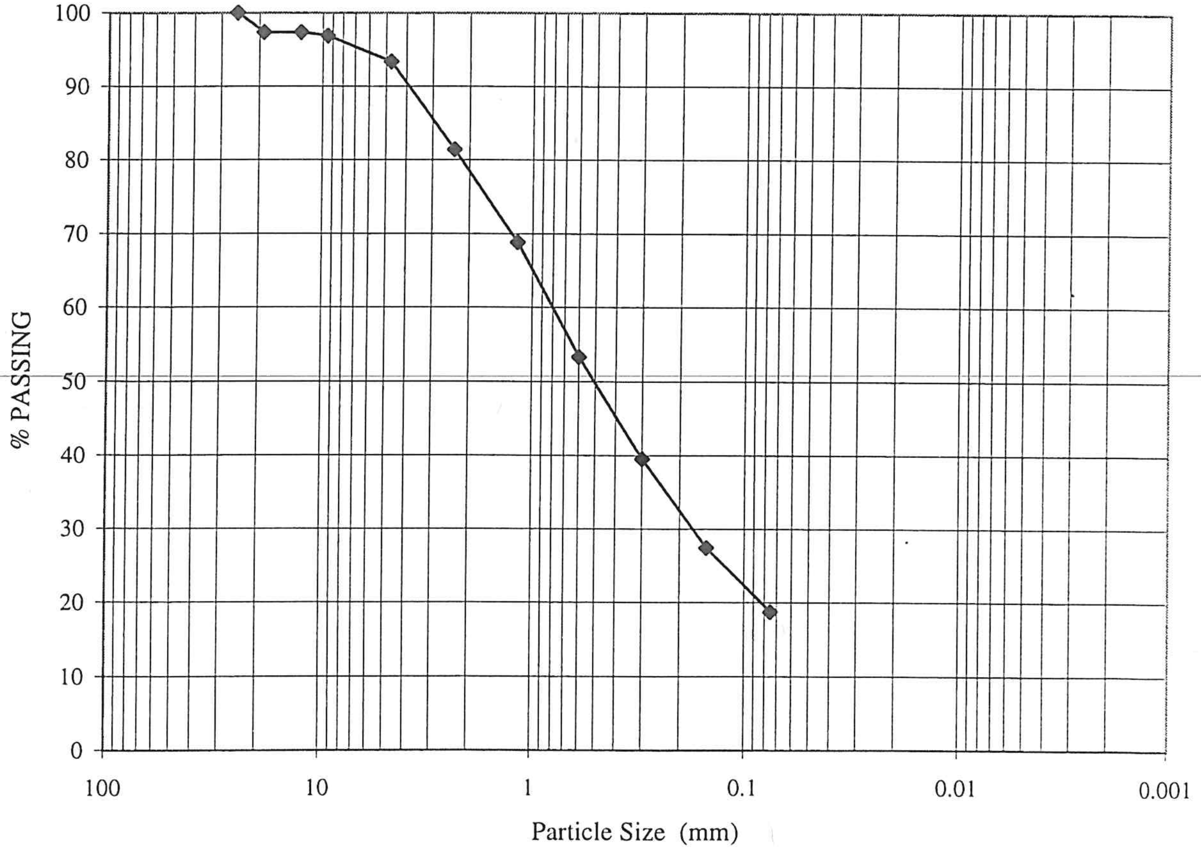
Aug 10, 1999

SIEVE ANALYSIS

ASTM C-136

JOB NAME: Joshua Springs Calvary Chapel
SAMPLE ID: B-1 @ 0-5 feet
DESCRIPTION: Silty F to C Sand (SM)

SIEVE SIZE	% PASSING
1 1/2"	100
1"	100
3/4"	97
1/2"	97
3/8"	97
#4	93
#8	81
#16	69
#30	53
#50	40
#100	27
#200	19



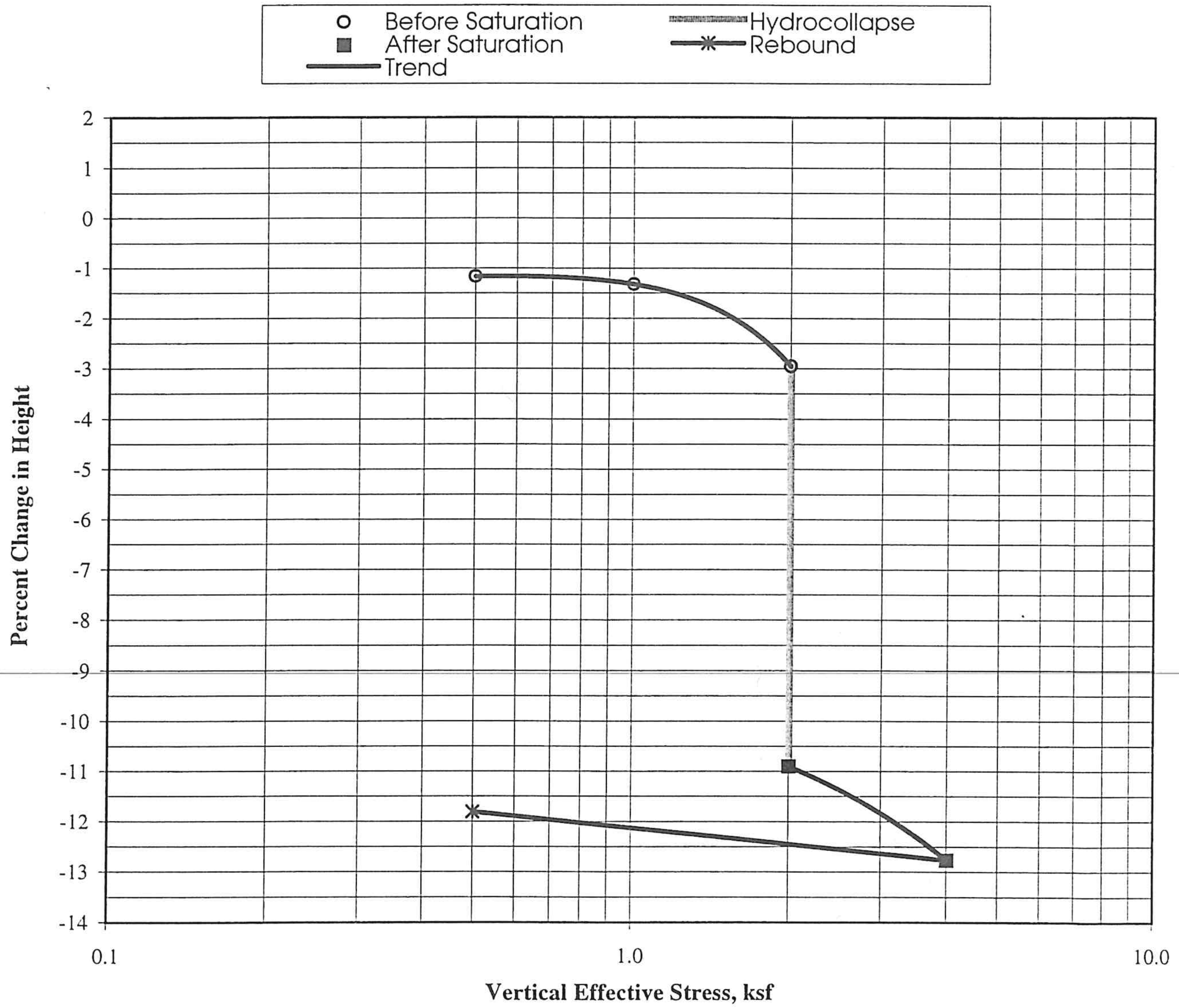
CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Joshua Springs Calvary Chapel		Initial Dry Density:	107.3 pcf
B-4 @ 1 foot		Initial Moisture, %:	2.0%
Silty F Sand (SM)		Specific Gravity:	2.67 (assumed)
Ring Sample		Initial Void Ratio:	0.553

Hydrocollapse: 8.0% @ 2.0 ksf

% Change in Height vs Normal Presssure Diagram



07278-01 Aug 10, 1999

MAXIMUM DENSITY / OPTIMUM MOISTURE

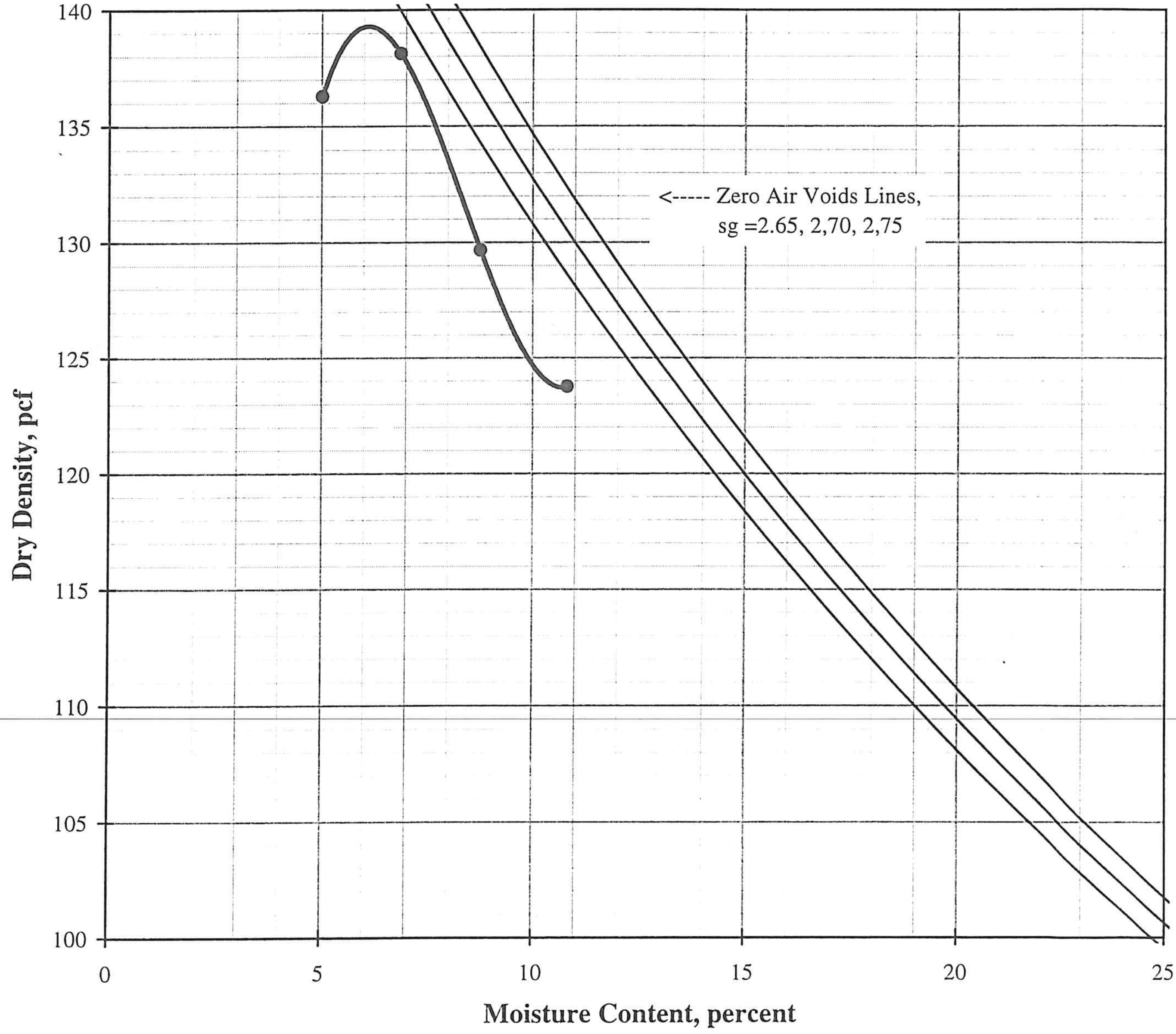
ASTM D 1557-91 (Modified)

Job Name:	Joshua Springs Calvary Chapel
Sample ID:	B-1 @ 0-5 feet
Location:	Native
Description:	Silty F to C Sand (SM), trace clay

Procedure Used: B
Prep. Method: Moist
Rammer Type: Manual

Maximum Density: **139.5 pcf**
Optimum Moisture: **6%**

Sieve Size	% Retained
3/4"	0.0
3/8"	1.7
#4	5.7



07278-01 Aug 10, 1999

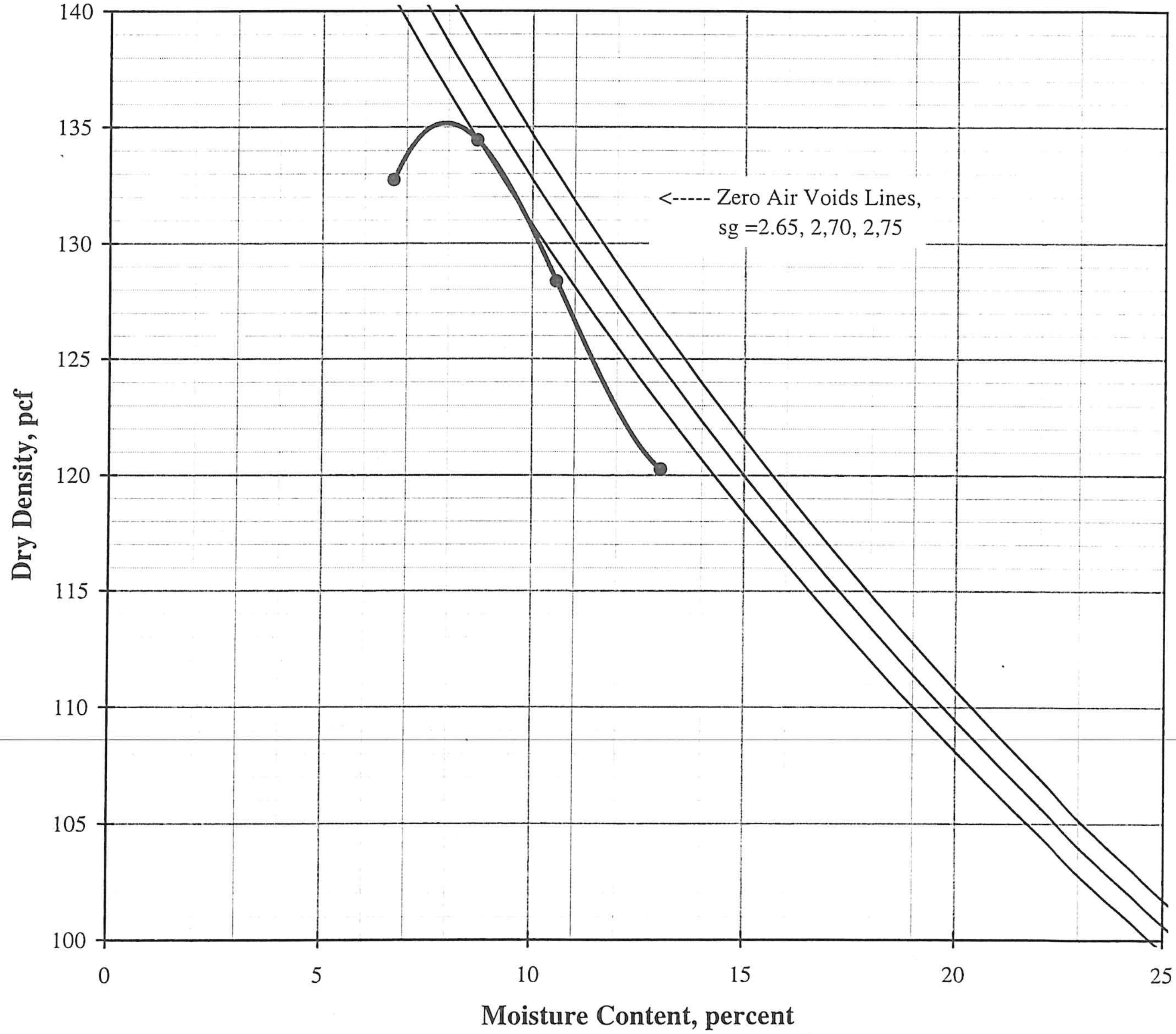
MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-91 (Modified)

Job Name:	Joshua Springs Calvary Chapel	Procedure Used: B
Sample ID:	B-7 @ 0-5 feet	Prep. Method: Moist
Location:	Native	Rammer Type: Manual
Description:	Brown Silty F to C Sand with Gravel to 1/2" (SM)	

Maximum Density: **135 pcf**
Optimum Moisture: **8%**

Sieve Size	% Retained
3/4"	0.0
3/8"	1.6
#4	3.9



SOIL & PLANT LABORATORY
and CONSULTANTS, Inc.
79-607 Country Club Drive
Suite 7
Bermuda Dunes, CA 92201
760-772-7995

SOIL ANALYSIS

for: Earth Systems Consultants Southwest

report date: 8-4-99

inv./lab#: 231

				Ohms-cm	ppm			meq/L		ppm	mg/Kg
No.	Description	Sat.%	pH	Res	NO_3^-N	PO_4^-P	K	Ca + Mg	Na	Cl	SO_4
07278-01	Joshua Springs Colony Chapel										
	B1 @ 0-5'		7.45	2750						21	N.D.

N.D. = None Detected

APPENDIX C

Percolation Test Results

PERCOLATION TEST RESULTS FOR SEEPAGE PIT OR DRY WELL

CLIENT: Joshua Springs Calvary Chapel
PROJECT: New Gymnasium
JOB NO: 07278-01

Date: 7/30/99

BORING **P-1** Depth: 20.0 ft Borehole Dia: 0.67 ft

Pipe Stickup:	Gravel to:	Tape Corr:	Gravel Factor:
0 - 0.0	4 - 0.0	0 - 0.0	0.46

Reading No.	Time	t Time Interval (min)	Total Depth of Hole (ft-in.)	Initial Water Level (ft-in.)	Final Water Level (ft-in.)	F Fall in Water Level (ft)	L(avg) Average Wetted Length (ft)	Qcorr = Q*GF F/t*D*9/L(avg) Percolation Rate (gal/sf/day)
1	10:30							
	11:00	30	20 - 0.0	8 - 2.0	15 - 0.0	6.83	8.42	4.5
2	11:30							
	12:00	30	20 - 0.0	8 - 2.0	16 - 0.0	7.83	7.92	5.5
3	12:30							
	12:40	10	20 - 0.0	18 - 2.0	18 - 8.0	0.50	1.58	5.2
4	12:45							
	12:55	10	20 - 0.0	18 - 2.0	18 - 7.0	0.42	1.63	4.2
5	13:00							
	13:10	10	20 - 0.0	18 - 2.0	18 - 5.0	0.25	1.71	2.4
6	13:15							
	13:25	10	20 - 0.0	18 - 2.0	18 - 5.5	0.29	1.69	2.9
7	13:30							
	13:40	10	20 - 0.0	18 - 2.0	18 - 5.0	0.25	1.71	2.4
8	13:45							
	13:55	10	20 - 0.0	18 - 2.0	18 - 5.0	0.25	1.71	2.4

PERCOLATION TEST RESULTS FOR SEEPAGE PIT OR DRY WELL

CLIENT: Joshua Springs Calvary Chapel
PROJECT: New Gymnasium
JOB NO: 07278-01 Date: 7/30/99

BORING P-2 Depth: 20.0 ft Borehole Dia: 0.67 ft

Pipe Stickup:	Gravel to:	Tape Corr:	Gravel Factor:
0 - 0.0	4 - 0.0	0 - 0.0	0.46

Reading No.	Time	t Time Interval (min)	Total Depth of Hole (ft-in.)	Initial Water Level (ft-in.)	Final Water Level (ft-in.)	F Fall in Water Level (ft)	L(avg) Average Wetted Length (ft)	Qcorr = Q*GF F/t*D*9/L(avg) Percolation Rate (gal/sf/day)
1	08:00 08:30	30	20 - 0.0	8 - 2.0	19 - 2.0	11.00	6.33	9.6
2	08:35 09:05	30	20 - 0.0	8 - 2.0	19 - 0.0	10.83	6.42	9.3
3	09:30 09:40	10	20 - 0.0	18 - 2.0	18 - 10	0.67	1.50	7.4
4	09:45 09:55	10	20 - 0.0	18 - 2.0	18 - 6.0	0.33	1.67	3.3
5	10:00 10:10	10	20 - 0.0	18 - 2.0	18 - 6.0	0.33	1.67	3.3
6	10:15 10:25	10	20 - 0.0	18 - 2.0	18 - 6.5	0.38	1.65	3.8
7	10:35 10:45	10	20 - 0.0	18 - 0.0	18 - 4.0	0.33	1.83	3.0
8	10:55 11:05	10	20 - 0.0	18 - 0.0	18 - 4.0	0.33	1.83	3.0