SANTA PAULA GEOLOGIC HAZARD ABATEMENT DISTRICT (GHAD) PLAN OF CONTROL

SANTA PAULA, CALIFORNIA

PREPARED

BY

ENGEO INCORPORATED

PROJECT NO. 7290.200.101

MARCH 10, 2008
REVISED MAY 23, 2008

ENGEO INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
7290.200.101

March 10, 2008
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Ms. Tiffany Sukay
Comstock Homes
312 12th Street
Manhattan Beach, CA 90266

Subject: Santa Paula Geologic Hazard Abatement District (GHAD)
Santa Paula, California

**GEOLOGIC HAZARD ABATEMENT DISTRICT (GHAD)
PLAN OF CONTROL**

Dear Ms. Sukay:

ENGEO Incorporated is pleased to present this Santa Paula Geologic Hazard Abatement District (GHAD) Plan of Control.

We are pleased to be of service to you on this project. If you have any questions concerning the contents of our report, please do not hesitate to contact us.

Very truly yours,

ENGEO Incorporated

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I. AUTHORITY AND SCOPE

The Santa Paula Geologic Hazard Abatement District ("GHAD" or "District") is proposed to be formed under authority of Public Resources Code §§ 26500, *et seq.*

Section 26509 of the Public Resources Code requires a Plan of Control, prepared by a State-Certified Engineering Geologist, as a prerequisite to formation of a GHAD. Pursuant to Section 26509, this Plan of Control was prepared by an Engineering Geologist certified pursuant to Section 7822 of the Business and Professions Code and describes, in detail, the geologic hazards, their location, and the areas affected by them. It also provides a plan for the prevention, mitigation, abatement, or control thereof.

As used in this Plan of Control, and as provided in Section 26507, "geologic hazard" means an actual or threatened landslide, land subsidence, soil erosion, earthquake, fault movement, or any other natural or unnatural movement of land or earth.

Property Identification

The GHAD boundary is shown on Figure 1 and incorporated by reference. The legal description of the land to be included within the Ridgeview at Vista Glen GHAD is included in Exhibit A, which is incorporated by reference.

II. BACKGROUND

The initial GHAD jurisdiction includes the Ridgeview at Vista Glen property, consisting of approximately 14.1 acres at the northern terminus of 10th Street, immediately north of the existing Santa Paula Hospital facility in Santa Paula, California. This GHAD will provide for the prevention, mitigation, abatement, and control of geologic hazards including earth movements that could impact this development.

The majority of the proposed District is situated on an elevated hillside terrace located on the southern flank of the east-west trending ridge of Sulphur Mountain, approximately 1.5 miles northwest of the confluence of the Santa Clara River and Santa Paula Creek. The topography within the area of proposed GHAD boundary varies from a relatively flat slope of 10:1 (horizontal:vertical) or flatter on the terrace portion of the site (location of an old avocado orchard) to steeper ascending slopes along the western side of the site (approximately 3:1), and to very steep descending slopes along the eastern and northern portions of the site. Elevations range from approximately 595 feet above mean sea level (msl) at the southern boundary to approximately 720 feet above msl along the western boundary of the District (Albus-Keefe [AKA], 2006; DRC, 2006).

The descending slopes along the northern and eastern sides of the site range from 1:1 to as steep as ½:1 near the upper part of the slope, and decrease to approximately 1:1 to 1½:1 on the lower flanks. The slopes at the eastern side (referred to in the referenced reports as the easterly bluff) extend down over a horizontal distance of 260 to 360 feet to the rear of existing single-family homes with a change in elevation of 180 to 200 feet. The slopes at the northern side of the site extend into a narrow east-draining canyon with a runout that is over 600 feet from existing homes at the base of the slope.

The western site boundary lies adjacent to an ascending slope that rises at about 3:1 to the top of a north-south-southwest trending ridge with an elevation of about 800 feet above msl. Several existing homes are located along the eastern flank of this ridge; they are aligned along the southern portion of the western site boundary. The grading plan indicates cuts of up to 24 feet into this slope and the installation of retaining walls or Verdura walls.

Proposed Development

The project development plan includes 75 residential lots, three parks, an underground detention basin, and associated streets (Figure 2). Several retaining walls and Verdura walls will be constructed at the site, reaching a maximum height of 21 feet. Verdura walls are segmental wall units that are used in conjunction with geogrid to help armor an engineered fill slope. Cut and fill slopes are proposed at a maximum ratio of 2:1 to heights of 35 feet or less. Fills over cut slopes are proposed to a maximum height of approximately 43 feet. Additionally, three GEOBRUGG screen walls will be installed at the base of the debris flow-prone areas.

Open Space

It is anticipated that title for the open space will pass to the GHAD. As the open space within and immediately adjacent to the proposed development and within the tract boundary is an amenity that benefits all of the property owners within the development, the funding of the maintenance of the open space will be shared by all current and future property owners within the GHAD's boundaries. Oversight of the actual physical maintenance responsibility for parcels of open space will pass to the GHAD.

The GHAD will assume responsibilities that relate to its position as a GHAD and other duties as a responsible land owner within GHAD-owned open space areas as shown in Figure 2. The

GHAD is charged with responsibilities that relate to the prevention, mitigation, abatement, or control of geologic hazards, which includes the maintenance of facilities that enhance geologic as well as hydrogeologic stability, such as drainage facilities and associated improvements. These duties may include the monitoring and maintenance of drainage facilities which, if subject to improper care, could result in decreased slope stability, the prime concern of the GHAD. As currently planned, the drainage facilities to be maintained by the GHAD include Integrated Management Practice (IMP) water quality treatment facilities, the detention basin, concrete-lined drainage ditches, and open space storm drain facilities.

The GHAD intends to mitigate or abate landslide or erosion hazards that could directly affect improved, developed, and accepted properties (as defined in Section VI) within the project, in accordance with Section VII. The GHAD will also perform maintenance of water control and conveyance facilities in open space areas and may assume other peripherally related open space responsibilities in GHAD-owned open space areas, such as erosion control, retaining and screen wall maintenance, mowing, trail maintenance, and selected other maintenance associated with open space.

III. SITE GEOLOGY

The following section is a summary of geologic conditions at the site as described by AKA in the 2006 and 2007 reports, the latter of which was produced following rough grading of the site and preparation of building pads for Phase 1 development in the southeastern portion of the site. The relevant references are presented in their entirety in Appendices B through E to provide more detail regarding geologic conditions.

Geologic Units

The site is situated on an elevated alluvial terrace associated with the ancestral Santa Paula Creek. The terrace was covered with varying amounts of non-engineered fill associated with past agricultural activities. Beneath the fill, the material was described as "Older colluvium" – a finer grained and more weathered material than the underlying terrace deposits. Older colluvium is labeled Q_{colo} on the rough grading map of the AKA October 2007 report. Underlying the colluvium are Late Pleistocene-age non-marine terrace deposits, which, in turn unconformably overlie Saugus Formation bedrock (Dibblee, 1992). Each of these units is described more completely below.

Fill. Non-engineered fill associated with previous agricultural activities was reported throughout much of the site during the initial exploration (AKA, May 2006). Fill encountered during field investigations typically consisted of soft to dense combinations of clay, silt, sand, and rock fragments ranging from a few inches to 4 feet in thickness. These materials were reportedly completely removed during preliminary site grading in the Fall of 2007 (AKA, 2007).

Colluvium. Colluvial deposits were mapped over nearly the entire site, with thicker accumulations observed within drainage swales of the easterly descending slope. Older colluvium (labeled Q_{colo} on the rough grading map) underlies the majority of the relatively level portions of the site. These deposits typically consist of silt, clayey silt, sandy silt, silty clay, and sandy clay. During grading, the upper 5 to 7 feet of weathered material were removed to expose competent older colluvium prior to placement of engineered fill (AKA, 2007).

<u>Terrace Deposits.</u> Underlying the colluvium, Late Pleistocene-age stream terrace deposits (Qt) were mapped across the flatter portion of the site during excavation for the rough grading (AKA, 2007). Terrace deposit materials typically consist of poorly to locally well-stratified gravels, cobbles, and boulders in a matrix of clayey sand, silty sand, and sand. These materials were reportedly moist and dense to very dense. During the excavation of the keyway along the easterly bluff, the AKA Geologist observed that the base of the terrace deposits was generally sloping down slightly to the southeast out of the slope face. AKA also noted some local scour features at the base of the unit, particularly where the underlying bedrock unit was composed of granular materials.

<u>Bedrock - Saugus Formation.</u> The Plio-Pleistocene-age Saugus Formation underlies the entire project area. Even though this formation is only slightly older than the nearly flat-lying terrace deposits, it is a tilted and folded sequence of non-marine sediments comprising massive to thinly bedded clayey siltstone, sandy siltstone, silty sandstone, sandstone, and conglomerate interbeds, generally 1 foot to 6 feet in thickness with thin clay seams, typically ¼-inch thick or less. The bedrock units were reportedly observed to be light brown, reddish-brown, pale olive-gray to olive-brown in color, soft to moderately hard, damp to moist, slightly to moderately weathered, and locally contained some calcium carbonate mineralization along joints. A number of clay seams were described as appearing to be tectonically sheared, polished, and locally striated in the down dip direction. AKA attributed these features to flexural slip along bedding planes during rapid uplift and folding of the bedrock in the region.

Bedding orientation of the bedrock was explored in the backcut and exploratory trench excavated during the Phase 1 east bluff stabilization (AKA, 2007). Bedding plane surfaces between sandstone and siltstone units were the most useful for determining strike and dip. The orientation of these bedding planes was N63°E ± 20°, dipping 48° ± 11° to the southeast. Joints exposed in the large diameter boreholes, in the keyway backcut, and in the exploratory trench were observed to be high-angle, discontinuous, non-planar, tight, and lined with calcium carbonate and/or iron oxide staining. Joint attitudes measured in the exploratory trench along the bluff top were oriented generally north-south and northwest-southeast with moderate (mainly to the east) to vertical dip angles (AKA, 2006 & 2007).

Landslides and Surficial Features

East Bluff Slopes. The California Geological Survey has placed the easterly descending natural slope within a seismically induced landslide hazard zone in accordance with the Seismic Hazards Mapping Act (CGS, 2002). No deep-seated landslides were identified by AKA within or immediately adjacent the site; however, localized surficial failures have occurred within over-steepened portions of the eastern slopes and similar events are anticipated to occur over time within the steep terrain.

Historical debris flows and surficial failures have been reported within the swales of the east bluff slope by local residents and city officials. Apparently the historical debris flows originated within the natural drainage swales and locally inundated city streets as well as some private residential properties with muddy runoff. According to AKA, some surficial slope failures have also occurred near the base of the eastern slopes as a result of non-engineered over-steepened construction excavations created by homeowners.

Based on observations of the eastern bluff, AKA and Kane (2007) have identified those portions within the project boundaries that are susceptible to future surficial instabilities. They consider the entire area, except for the areas identified as natural drainage swales (chutes), as posing low risk to down-slope properties. AKA anticipates that surficial failures will generally result in isolated failures that move short distances. These mobilized earth materials would likely migrate down-slope over extended periods of time through the natural process of erosion and creep without directly impacting downhill properties. Surficial failures within and adjacent the natural drainage swales are anticipated to continue to generate debris flows during periods of prolonged rainfall that could impact downhill properties. This adverse condition can be mitigated through the implementation of debris barrier systems within the natural drainage swales (Kane, 2007).

Seismicity

The closest Type A fault is the San Andreas fault, located approximately 32 miles northeast of the site. The closest Type B fault is the onshore segment of the Oak Ridge Fault, located approximately 1.7 miles southeast of the site. The San Cayetano Fault is 4 miles northeast of site. AKA performed a deterministic analysis to estimate peak horizontal ground acceleration (PHGA) for the site. The largest estimated mean PHGA is 0.64g with a standard deviation of 0.43g, corresponding to a 7.0 moment magnitude event along the onshore segment of the Oak Ridge Fault (AKA, 2006).

Faulting. The site is not located within an Alquist-Priolo Earthquake Zone (CGS, 1998 & 1999). Alquist-Priolo Earthquake Zones are specified zones that delineate areas of known active faults, as defined by the State of California. Several active fault zones have been designated by the State north and south of the site. The closest known fault zones are located approximately 1.5 miles or more northeast of the site and include the Orcutt, Timber Canyon and San Cayetano, and a few unnamed faults. Other known fault zones located greater distances to the southeast and southwest of the site include the Oak Ridge and Ventura faults (Dibblee 1990 & 1992).

The San Cayetano and Oak Ridge faults located northeast and southeast of the site, respectively, are considered the most significant of the mentioned faults. The San Cayetano fault to the north is an active north-dipping reverse fault that trends roughly east-west. Studies indicate that this fault displaces Tertiary and Quaternary rocks with as much as 9 km of stratigraphic separation (Rockwell, 1988). The Oak Ridge fault to the south is an active, mostly south-dipping, reverse fault that trends to the northeast along the south side of the Santa Clara River Valley (Leighton, 2007). The 1994 Northridge earthquake is believed to have occurred on a continuation of the Oak Ridge fault system (Yeats & Huftile, 1995).

Previous subsurface investigations for the site, geologic mapping during rough grading, review of geologic literature and review of topographic expression for the site and nearby vicinity have not found evidence of active faulting within or immediately adjacent to the site. Furthermore, there are no known active fault zones that project directly towards or through the site.

AKA (2007) observed bedding-plane reverse faults in at least two areas of the site during rough grading. To evaluate the relative age of fault activity, AKA excavated exploratory trenches roughly parallel to the fault traces to expose several vertical feet of bedrock overlain by terrace deposits and/or older colluvial soils. Dr. Thomas Rockwell was contracted by AKA to visit the site and conduct an independent evaluation of the fault features. Dr. Rockwell concluded that the bedding plane thrusts exposed on the Santa Paula terrace are inactive and should not be considered a constraint to development based on the following lines of evidence: (1) there is no scarp or lineament that can be identified in aerial photography of the site, nor is there any known active bedding plane fault that projects towards or through the site; (2) the fault cuts a basal gravel deposit, but does not cut the soil developed through the gravel; and (3) the fault does not penetrate or offset the overlying colluvial deposits which are capped by a late Pleistocene soil. These three observations all support the conclusion that the faults are inactive in the Holocene time frame.

Groundwater

Static groundwater was not encountered during the 2006 AKA field exploration. AKA did report slight seepage in one boring at a depth of 53.5 feet below ground surface. Additionally, perched groundwater was encountered at a bedrock contact ranging in depth between 20 and 28 feet below the ground surface in four additional borings. Based on available publications and reports, the historic high groundwater level is reported to be approximately 40 feet below the ground surface in the lower Santa Paula Creek Valley; approximately 200 feet below the site (AKA, 2006).

IV. GEOLOGIC HAZARDS

The following geologic hazards were identified for the site in the previous site studies and are expected to remain to some extent after site grading has been completed.

- Slope instability
- Expansive soils
- Compressible terrace deposits
- Seismically induced ground shaking

Slope Instability

Slope stability is the GHAD's prime geotechnical concern at this site. Slope instability is an important consideration for hillside projects throughout the Southern California area, and this site has slope-related geotechnical issues typical of many other similar hillside sites. Future stability depends on numerous factors, including changes in the occurrence of natural or artificial groundwater, future grading, and earthquake ground shaking.

This section describes several types of slope instability that are within the GHAD's responsibility, subject to the provisions of Sections VI and VII.

Landslides. Landslides are a common geologic phenomenon and are part of the process of mass wasting. Weathered or fractured bedrock and soil are transported downslope over geologic time as a result of gravitational and hydrostatic forces. Landslides include a variety of morphologies and are further defined by type of materials, wetness, and mode of movement. They can consist of mass movements of earth materials that are primarily intact and occur along discrete shear surfaces, or they can consist of flowing earth materials. In the case of movement of coherent blocks, the slip or shear surfaces can be rotational (conchoidal or concave), such as for earth slumps, or planar, as for translational earth slide or bedrock block glides. Most landslides are actually "complex landslides", sliding, falling and flowing with more than one type of movement and/or material.

Landslides and earth movement in the Saugus Formation are typically rotational slumps and earthflows but can also include translational landslides with a basal contact along weak bedding planes or discontinuities, along a component of the true dip, or as wedge-type failures formed by intersecting planes of weakness (FUGRO, 2007). Depth of movement may exceed 25 feet below the ground surface (FUGRO, 2007). The orientation of Saugus Formation beds at the site is such that the down-dip direction does not intersect the slope face and, therefore, is less likely to fail due to adverse dip conditions. Mapped joints were nearly vertical or dipped more steeply than the slope face. As with the bedding, this orientation is favorable for stability of the bedrock along the easterly bluff.

Mass movements involving soil and colluvium are generally in the form of an earthflow or a debris flow. The sources of these features are confined to the upper 3- to 5-foot-thick clayey soil mantle. In the winter rainy season, earthflows can typically move at a rate of several feet per day. Debris flows are much faster due to their higher water content. Kane (2007) estimated velocities of 2 to 6 meters per second for the design of debris flow mitigative measures.

AKA (2006) identified the upper portion of the easterly bluff of the site as having a factor of safety of less than 1.5 against gross failure by landslide. They recommended excavating this slope, installing drainage, and rebuilding the slope with geogrid to reinforce the slope. In the Fall of 2007, this work was done on a portion of the bluff at the rear of Lots 21 through 25 and 28 through 30.

The keyway back cut was formed at a 1:1 (horizontal:vertical) slope with a 25- to 35-foot-wide keyway cut at least 2 feet into competent rock. A backdrain was installed at the heel of the keyway (AKA, 2007) and the slope was reconstructed with select granular material and geogrid reinforcement. As described in the Rough Grading Report by AKA (2007), Mirafi 10XT grids were installed from the slope face into the slope extending a length equal to the height of the slope. Intermediate geogrid layers (Mirafi 2XT) were placed between layers of the Mirafi 10XT for slope facing stability. Cohesive on-site soils were placed on the outer 2 to 4 feet of the slope face to mitigate piping of granular soils through the geogrid. Verdura wall blocks were also installed every 2.5 feet to anchor the geogrid and to help armor the slope (AKA, 2007).

<u>Soil Creep.</u> Soil creep is the slow, often imperceptible, deformation of slope materials under low stress levels, which normally affects the shallow portion of the slopes, but can be deep seated where a weak zone of soil or bedrock exists. It results from gravitational and seepage forces, and may be indicative of conditions favorable for landsliding. Creep can be caused by wetting and drying of clays, by solution and crystallization of salts, by the growth of roots, by burrowing animals and by down-slope movement of saturated ground. Colluvium refers to the mantle of loose soil and weathered bedrock debris that moves down hillsides by creep-related processes. Areas susceptible to soil creep are shown on Figure 2 in yellow.

<u>Erosion.</u> The GHAD is also concerned with erosion and sedimentation in open space or affecting developed lots or improvements, subject to the provisions of Sections VI and VII. Erosion is defined as the process by which earth materials are loosened and removed by running water on the ground surface or in the subsurface. Sedimentation is the consequent depositing or settling of soil or rock particles from a state of suspension in a liquid.

Undeveloped hilly terrain either in a natural condition or particularly on cut or fill slopes can be subject to erosion. Landslide deposits, which are sometimes in a loosened condition, are particularly prone to erosion. Earth flow-, debris flow- and mud flow-type landslides typically have an area of deposition or accumulation (sedimentation area) at their base. Graded slopes in the GHAD, particularly those in excess of 20 feet in vertical height and those not sufficiently vegetated, can be subject to erosion and, therefore, can become a source of transported sediment. Slopes with higher susceptibility to erosion are shown in blue on Figure 2.

Debris Flows. Three areas have been identified as being prone to debris flows (AKA, 2007; Kane, 2007). The debris flow areas are located down-slope of the easterly bluff, below Lots 23 through 25, 28, 29, and 30 (orange areas on Figure 2). The Kane report (2007) lists the characteristics of the debris flow chutes as follows:

Debris Flow Chute	Soil	Area (ft^2)	Total Volume (ft^3)
A	Silt w/Cobbles	5,032	15,096
B	Silt w/Cobbles	10,793	32,379
C	Silt w/Cobbles	5,705	17,115

To mitigate the risk posed by the debris flows to adjoining properties, GEOBRUGG debris flow screen walls designed by Kane and Associates (2007) will be installed at the locations indicated as green dashed lines on Figure 2.

Expansive Soils

Near-surface soil and clayey bedrock at the site could exhibit a moderate potential for expansion. These potentially expansive soils could impact the planned site development. Expansive soils shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. The potential for expansive soils has been identified in previous reports for the site. Shrink and swell of expansive soils on slopes contributes to the mechanism of creep, which can result in shallow slope instability.

Compressible Terrace Deposits

AKA (2007) indicates that some terrace deposits are expected to undergo settlement due to the weight of proposed overlying fills. Total settlements and differential settlements of 2½ inches and ½ inch over 30 feet, respectively, can be expected in localized areas. The majority of the anticipated settlement will occur during loading by the fill (AKA, 2007).

Seismically Induced Ground Shaking

As identified in the geotechnical report (AKA, (2006), an earthquake of moderate to high magnitude generated within the Southern California region could cause considerable ground shaking at the site, similar to that which has occurred in the past. It appears that seismic slope stability has been considered in the remedial grading plans; however, seismically generated slope failures could occur within the oversteepened portions of the natural easterly slope. Measures have been taken to mitigate this risk, such as the rebuilt upper portion of the easterly bluff and the installation of the GEOBRUGG debris flow control systems.

Ground Rupture. No known active faults are known to project through the site nor does the site lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The potential for ground rupture due to an earthquake beneath the site is considered very small.

Ground Shaking. The site is in a seismically active area that has historically been affected by generally moderate to occasionally high levels of ground motion. The site lies in relative close proximity to several active faults; therefore, during the life of the proposed development, the property will probably experience similar moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the southern California region.

V. CRITERIA FOR GHAD RESPONSIBILITY

Prevention, Mitigation, Abatement and/or Control of Geologic Hazards

Subject to the following exceptions, the primary mission of the GHAD is the prevention, mitigation, abatement and control of geologic hazards within its jurisdictional boundaries that have damaged, or that pose a significant threat of damage to site improvements within developed areas. As used herein, the term "site improvements" means buildings and outbuildings, roads, sidewalks, improved paths, utilities, improved trails, swimming pools, tennis courts, gazebos, cabanas, geologic stabilization features, or similar improvements.

Exceptions

The intent of this Plan of Control is not to extend the GHAD's responsibilities to every potential situation of slope instability. Specifically, the following are excluded from the GHAD's responsibilities.

Isolated or Remote Slope Instability

The GHAD does not have responsibility to monitor, abate, mitigate or control slope instability that does not involve damage to or pose a significant threat to damage site improvements.

Single Property

The GHAD will not prevent, mitigate, abate or control geologic hazards which are limited in area to a single parcel of property unless the geologic hazard has damaged, or poses a significant threat of damage to site improvements located on other property within the GHAD jurisdictional boundaries. As used herein, the term "site improvements" means buildings, roads, sidewalks, utilities, improved trails, swimming pools, tennis courts, gazebos, cabanas, geologic stabilization features, or similar improvements. This exclusion does not apply to geologic hazards existing on recreational property, and open space property within the GHAD-owned property.

Geologic Hazards Resulting From Negligence of Property Owner

The GHAD may decline to prevent, mitigate, abate or control geologic hazards which occurred or resulted from any negligence of the homeowner and/or the homeowner's contractors, agents or employees in developing, investigating, grading, constructing, maintaining or performing or not performing any post-development work on the subject property.

Property Not Accepted

The GHAD does not have responsibility to repair damaged site improvements which are situated on a parcel of real property that the GHAD has not accepted in accordance with Section VI, below. The GHAD, however, may monitor, abate, mitigate or control slope instability on a parcel of real property which (1) the GHAD has not accepted in accordance with Section VI, below, and (2) that is not excluded from GHAD responsibility by Paragraphs 1, 2 and 3; provided that GHAD responsibility on such parcel is limited to the extent necessary to address damage or a significant threat to damage site improvements which are within a parcel of real property which the GHAD has accepted in accordance with Section VI, below.

Geologic Hazard Which Requires Expenditure in Amount Exceeding the Value of the Threatened or Damaged Improvement

The GHAD may elect not to prevent, mitigate, abate or control a geologic hazard where the anticipated expenditure required to be funded by the GHAD to prevent, mitigate, abate or control the geologic hazard will exceed the value of the structure(s) and site improvement(s) threatened with damage or loss.

VI. ACCEPTANCE

Activation of Assessment

Subject to applicable law, an annual assessment must be promptly implemented on all residential parcels in the GHAD. The assessment will be levied by the GHAD on each individual residential parcel beginning the first fiscal year after the City of Santa Paula issues a building permit for that parcel.

Responsibility for GHAD Activities

The party that, on the date that the Final Map within the boundaries of the GHAD is recorded by the City of Santa Paula, owns the developable parcels shown on that Final Map has the responsibility to perform all the activities of the GHAD on property within that Final Map. Following a period of at least 3 years after the first residential building permit is issued by the City of Santa Paula, or 1 year after the completion of the initial mass grading (as defined by the City-approved grading plans), whichever is later, the suitability of transfer of GHAD-related shall be determined by the GHAD Manager. If determined to be appropriate, maintenance responsibility shall be transferred to the GHAD. This transfer date may be extended at the sole discretion of the project developer provided that the assessments continue to be levied during the extension period and that notice of such extension is delivered to the GHAD General Manager at least 30 days before the transfer date. The petitioners for formation of the GHAD intend that the approximately 4-year period between the levying of the GHAD assessment and the GHAD becoming responsible to perform activities on property within each Final Map will allow the District to accumulate reserve funds without incurring significant expenses. Such reserve funds are needed to address periodic major events which cannot be funded from annual revenues.

VII. GHAD MAINTENANCE AND MONITORING RESPONSIBILITIES

The GHAD is responsible for maintaining geologic stabilization features (e.g. ditches, benches, walls) in the common open space and the unimproved areas including hillside slopes extending uphill from debris benches and outside of the private lot boundaries. The GHAD's maintenance responsibilities include prevention, abatement, vegetation control, and control of landslide and erosion hazards within the subdivision open space and hillsides exclusively within the GHAD Boundary, as provided in Section VI. Adjacent slopes, open space, and improvements outside of the GHAD boundary are not maintained by or the responsibility of the GHAD.

General maintenance of the surface drainage improvements in the open space and on the hillsides, such as the concrete V-ditches, will be the GHAD's responsibility. The GHAD is also responsible for general maintenance of the detention basin, storm drain inlets and outlets in open space and subdrain outlets. Potential geologic hazards such as landslides and slope erosion within the open space are the GHAD's responsibility. The GHAD has the following maintenance responsibilities:

- Trail maintenance.
- Inspection and maintenance of CDS water quality treatment unit and underground detention system.
- Inspection and maintenance of concrete-lined drainage ditches in open space area.
- Subdrains.
- Storm drain inlets, outfalls and pipelines within the open space area.
- Slopes.
- Vegetation control within the open space.

- Maintenance of retaining walls and GEOBRUGG screen walls.
- Splash walls.

Geotechnical Techniques for Mitigation of Landslide and Erosion Hazards

The techniques which may be employed by the GHAD to prevent, mitigate, abate, or control geologic hazards include, but are not limited to, the following.

A. Removal of the unstable earth mass.

B. Stabilization (either partial or total) of the landslide by removal and replacement with compacted, drained fill.

C. Construction of structures to retain or divert landslide material or sediment.

D. Construction of erosion control devices such as gabions, riprap, geotextiles, or lined ditches.

E. Placement of drained engineered buttress fill.

F. Placement of subsurface drainage devices (e.g. underdrains, or horizontal drilled drains).

G. Slope correction (e.g. gradient change, biotechnical stabilization, slope trimming or contouring).

H. Construction of additional surface ditches and/or detention basins, silt fences, sediment traps, or backfill or erosion channels.

Potential landslide and erosion hazards can often be mitigated best by controlling soil saturation and water runoff and by maintaining the surface and subsurface drainage system. Maintenance shall be provided for lined surface drainage ditches and drainage terraces including debris benches or drop inlets. Maintenance of the open space, including the clearing of fire trails, will be the responsibility of the GHAD. The GHAD also shall monitor the use of the open space by other entities.

VIII. PRIORITY OF GHAD EXPENDITURES

Emergency response and scheduled repair expenditures by the GHAD may be prioritized by the General Manager, based upon available funds and the approved operating budget. When available funds are insufficient to undertake all of the identified remedial and preventive stabilization measures, the expenditures must be prioritized in accordance with the GHAD Board of Director's directions or as follows in descending order of priority:

A. Prevention, mitigation, abatement or control of geologic hazards that have either damaged or pose a significant threat of damage to residences, critical underground utilities or paved streets.

B. Prevention, mitigation, abatement or control of geologic hazards which have either damaged or pose a significant threat of damage to ancillary structures, including but not limited to detention basins.

C. Prevention, mitigation, abatement or control of geologic hazards which have either damaged or pose a significant threat of damage to open space amenities.

D. Prevention, mitigation, abatement or control of geologic hazards which have either damaged or pose a significant threat of damage limited to loss of landscaping or other similar non-essential amenities.

E. Prevention, mitigation, abatement or control of geologic hazards existing entirely on open space property and which have neither damaged nor pose a significant threat of damage to any site improvements.

IX. MAINTENANCE AND MONITORING SCHEDULE

Geologic features and GHAD maintained facilities should be inspected by the GHAD General Manager's designees as presented below. The site inspections should be undertaken at appropriate intervals as determined by the GHAD General Manager using supporting documents prepared for the site and its improvements. The GHAD budget should provide for four or more inspections in years of heavy rainfall. Generally, the inspections should take place in October, before the first significant rainfall; mid-winter, as necessary during heavy rainfall years (cumulative rainfall exceeding the historic average); and in early April at the end of the rainy season. The frequency of the inspections should increase, depending upon the intensity and recurrence of rainfall. Site inspections should increase sufficiently to provide for mitigation of potential hazards. Additionally, site inspections should be performed following moderate to significant seismic events to evaluate potential damage or displacement to geologic facilities or GHAD-maintained facilities related to ground shaking.

The GHAD should obtain copies of geologic or geotechnical exploration reports related to site development and keep these reports on file in the records of the GHAD. In addition, copies of any earthwork-related testing and observation reports that will be finalized at the completion of grading, when as-built drawings are available, must be provided to the GHAD and maintained as part of the GHAD records.

Following are guidelines for a monitoring plan. The actual timing, scope, frequency and other details regarding such maintenance, inspection and similar activities shall be at the discretion of the GHAD General Manager.

- The Engineer and/or Geologist retained by the District should carry out an inspection of lined surface ditches at least twice a year. One inspection should be in the fall prior to the onset of winter rains. The inspection shall check for sedimentation and cracking or shifting of the concrete-lined ditches. Repairs and maintenance, as needed, should be undertaken including removal of excess silt or sediment in ditches and patching or replacement of cracked or broken ditches, prior to the beginning of the next rainy season.

- Subsurface drain outlets and horizontal drilled drain outlets, if any, should be checked. Water flowing from these outlets should be measured and recorded during each inspection. The inspections should take place at least twice annually, preferably in the fall and spring. Any suspicious interruption in flow should signal a need to unplug or clean the affected drain. If included, animal/rodent screens and covers should be checked and replaced as necessary.

- Retaining walls, splash walls and GEOBRUGG screen walls should be inspected as part of the site monitoring program. At a minimum, repairs and maintenance should be performed according to the manufacturer's recommendations. Recommendations for the proposed GEOBRUGG products have been included in Appendix A. During the twice-yearly scheduled site monitoring events any barriers should be viewed for debris or larger rocks against the barrier and for sagging ropes. In addition, the braking elements and wire rope clips should be checked. During repairs and maintenance, the GEOBRUGG walls shall be accessed from the top of the descending slope; i.e., from adjacent properties within the GHAD boundary. In case of emergency situations in which access from the top of the slope is not efficient or practical, the GEOBRUGG walls may be accessed from adjacent properties outside of the boundary, provided express permission to enter these properties is granted by the landowners. Two properties in particular that would likely provide suitable emergency access include Lot 6 within the Harvey Tract (at the end of Harvey Drive) and Parcel A, accessible at the end of Cadway Street. Procedures to secure Right-of-Entry for properties within the GHAD are presented in Section X.

- Inlets, outfalls, or trash racks, if used, must be kept free of debris and spillways maintained. It is anticipated that initially at least once every 2 years, cleanup of vegetation and removal of silt would be in order. Attention should be given to plantings or other obstructions which may interfere with access by power equipment.

- The underground detention basin and CDS water quality treatment unit should be checked and/or serviced as part of the site monitoring program. At a minimum, repairs and maintenance should be performed according to the manufacturer's recommendations. A maintenance guide is presented in Appendix F.

- The trails should be inspected on an annual basis. Positive drainage should be maintained. Depressions, rutting, and other surface expressions that could collect and allow water to infiltrate into the subsurface should be repaired. Additionally, ground cover along the trails should be maintained to allow for intended trail performance.

- An annual inspection of slopes and vegetative cover shall be made by the Engineer and/or Engineering Geologist to assess the effectiveness of the preventive maintenance program and to make recommendations as to which landslide or erosion measures should be undertaken in the next fiscal year. Any appropriate site-specific study of landslide or erosion conditions shall be determined at that time. Consultants, if necessary, will be retained to undertake the needed studies. An annual inspection report to the GHAD shall be prepared by the District Engineer and/or Engineering Geologist.

X. RIGHT OF ENTRY

GHAD officers, employees, consultants, contractors, agents, and representatives shall have the right to enter upon all lands within the GHAD's jurisdiction, as shown on Figure 1, which is incorporated by reference, for the purpose of performing the activities described in this Plan of Control. Such activities include, without limitation: (1) the inspection, maintenance and monitoring of site improvements including detention, water quality and sedimentation basins, maintenance roads, deflection walls, drainage ditches, storm drains, outfalls and pipelines; (2) the monitoring, maintenance and repair of slopes, including repaired or partially repaired landslides; and (3) the management of erosion and geologic hazards within the open space areas shown on Figure 1. Should the GHAD need to access private residential lots to fulfill its duties under the Plan of Control, the GHAD endeavors to provide the affected landowner and/or resident with 72 hours advanced notice unless, in the reasonable judgment of the GHAD, an emergency situation exists which makes immediate access necessary to protect the public health and safety, in which case no advance notice is required, but the GHAD must inform the landowner and/or resident as soon as reasonably possible.

The foregoing right-of-entry and indemnity provision must be recorded in the chain of title for all residential parcels and common area lots, and it shall be included in all Covenants, Conditions and Restrictions (CC&Rs) and homebuyer disclosure statements prepared for parcels within the District boundary.

XI. MISCELLANEOUS PROVISIONS

The GHAD may, in accordance with applicable law, expand its jurisdictional boundaries as depicted in attached Exhibit A, which is incorporated by reference, to include other real property that requires geologic hazard abatement. Nothing in this Plan is intended to, nor will it, limit the GHAD's ability to take such action either upon request by a developer or upon the motion of the Board of Directors.

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Albus-Keefe and Associates, Inc. (AKA); Foundation Design Recommendations for Retaining and Screen Walls, Santa Paula, California; April 17, 2006; Project No. 1489.00.

AKA; Supplemental Geotechnical Investigation and Rough Grading Plan Review, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, Santa Paula, California; May 3, 2006; Project No. 1489.00.

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DRC; Mass Grading Plans, Tract 5606–Ridgeview at Vista Glen, Santa Paula, California; April 10, 2006; Sheets 1-16.

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Leighton & Associates, 2007, Preliminary Geotechnical Investigation Report for the East Area 1 Specific Plan, Santa Paula Area of Unincorporated Ventura County, California; April 2007.

Rockwell, Thomas, 1988, Neotectonics of the San Cayetano Fault, Transverse Ranges, California, GSA Bulletin; April 1988; v. 100; no. 4; p. 500-513.

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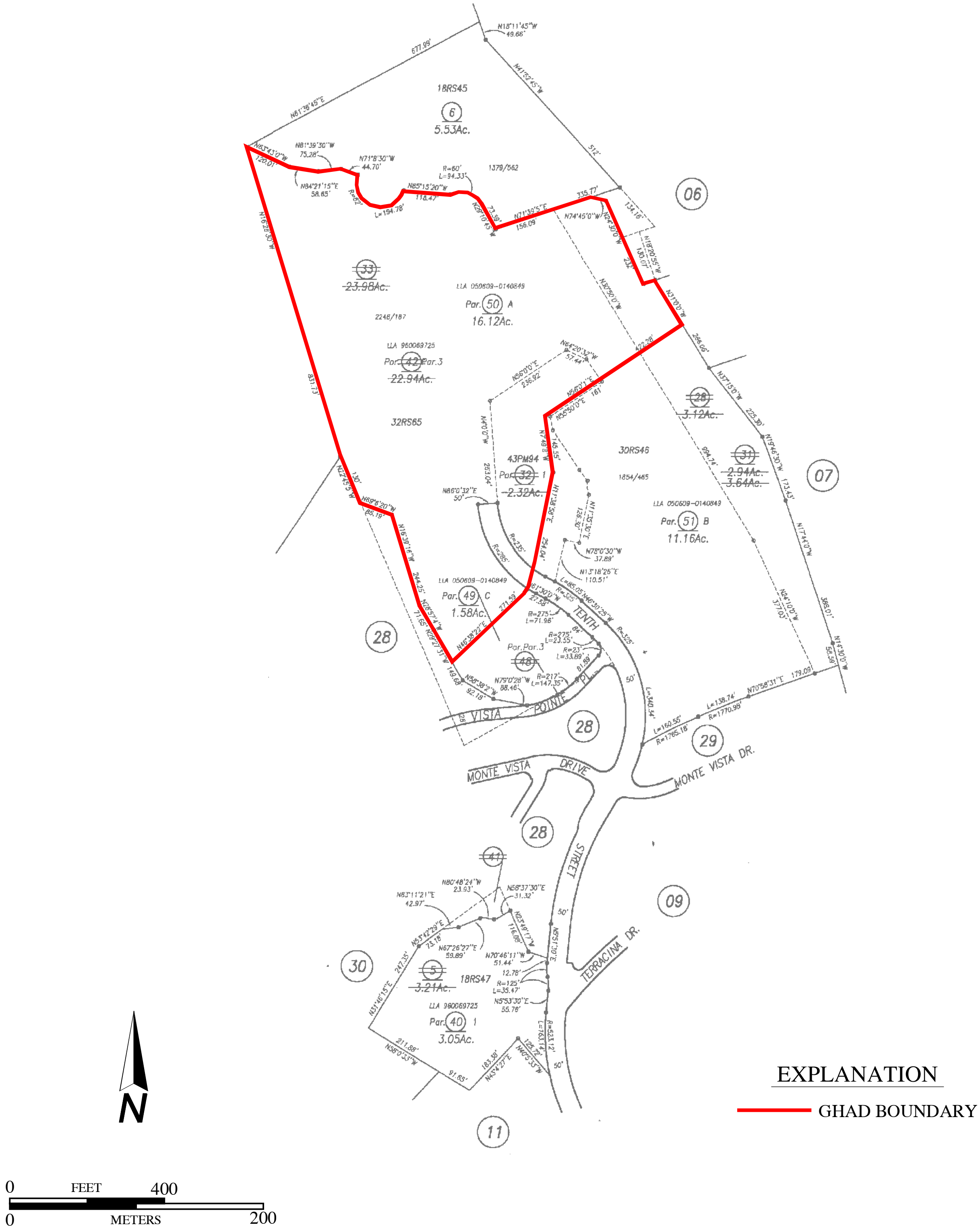
LIST OF FIGURES



BASE MAP SOURCE: LAND AMERICA LAWYERS TITLE

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	GHAD BOUNDARY RIDGEVIEW AT VISTA GLEN SANTA PAULA, CALIFORNIA	PROJECT NO.: 7290.200.101		FIGURE NO.
		DATE: MAY 2008		1
		DRAWN BY: DLB	CHECKED BY: JA	

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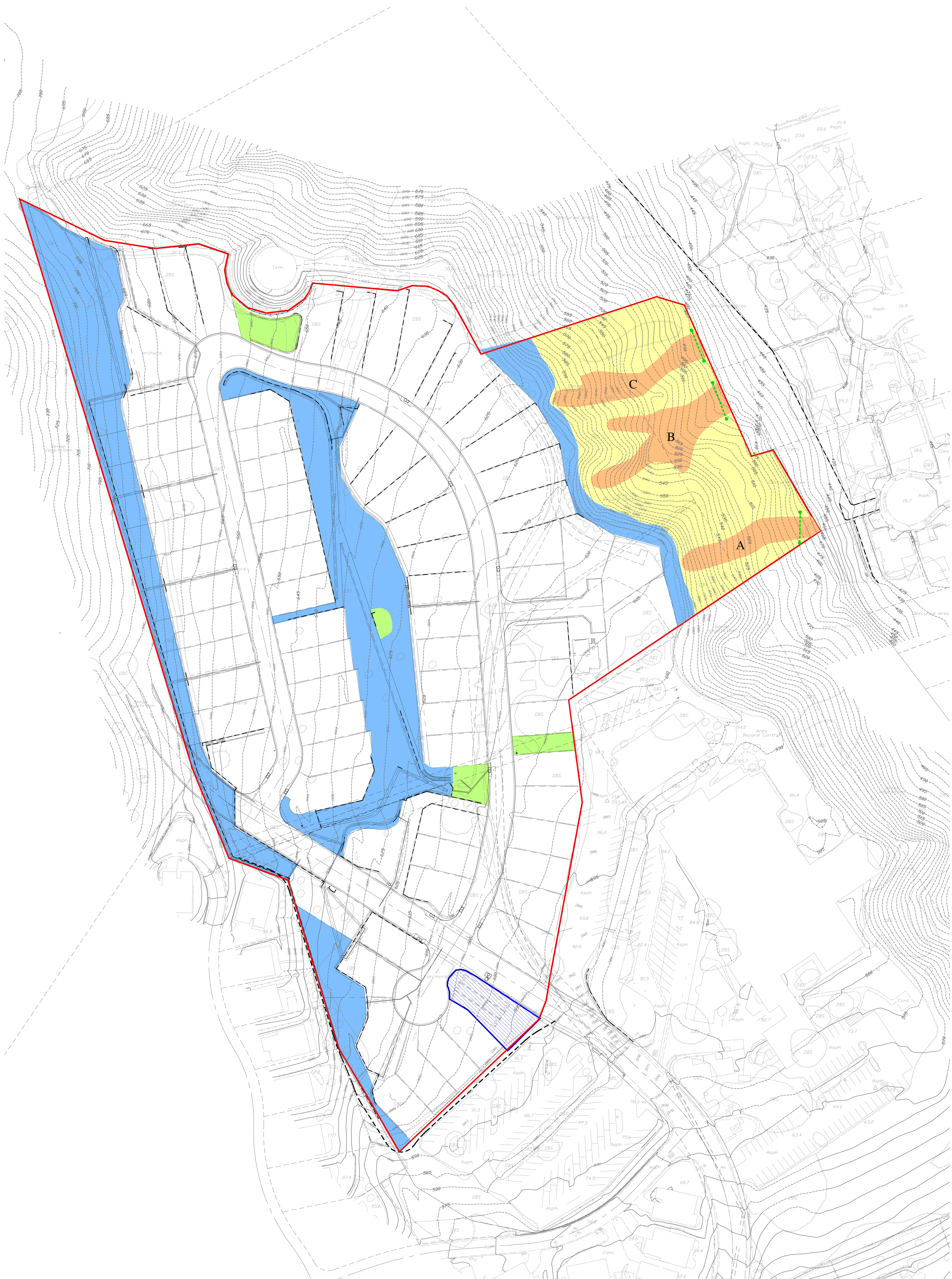


EXPLANATION

APPROXIMATE LOCATION OF NATURAL SLOPE AREAS PRONE TO SURFICIAL INSTABILITY

APPROXIMATE LOCATION OF NATURAL DRAINAGE SWALE (DEBRIS FLOW CHUTE)

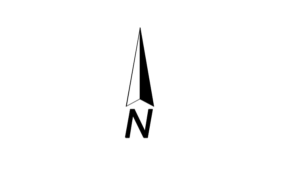
APPROXIMATE LOCATION OF NET BARRIERS

GHAD BOUNDARY

APPROXIMATE LOCATION OF GRADED AND ENGINEERED HILL SLOPES WITH LOW SUSCEPTIBILITY TO EROSION

APPROXIMATE LOCATION OF PARK

DETENTION BASIN



NOTE: ALL COLORED AREAS ARE OPEN SPACE SUBJECT TO GHAD CONTROL

BASE MAP SOURCE: UNKNOWN

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

SITE MAP WITH GHAD BOUNDARY AND GEOLOGIC HAZARDS
RIDGEVIEW AT VISTA GLEN
SANTA PAULA, CALIFORNIA

PROJECT NO.: 7290.200.101	
DATE: MAY 2008	
DRAWN BY: SRP	CHECKED BY: JA
FIGURE NO. 2	

ORIGINAL FIGURE PRINTED IN COLOR

LEGAL DESCRIPTION

Real property in the City of Santa Paula, County of Ventura, State of California, described as follows:

ALL THAT CERTAIN REAL PROPERTY SITUATED IN THE COUNTY OF VENTURA, STATE OF CALIFORNIA, DESCRIBED AS FOLLOWS:

PARCEL A OF LOT LINE ADJUSTMENT NO. 2005-CDP-02 RECORDED JUNE 9, 2005 AS INSTRUMENT NO. 20050609-0140849 OF OFFICIAL RECORDS BEING A PORTION OF AN UNNUMBERED LOT LYING NORTHERLY OF LOTS 23, 24, 25 AND 26 OF THE RANCHO SANTA ANA PAULA Y SATICOY IN THE CITY OF SANTA PAULA, COUNTY OF VENTURA, STATE OF CALIFORNIA, ACCORDING TO THE MAP THEREOF RECORDED IN BOOK "A", PAGE 290 OF MISCELLANEOUS RECORDS (TRANSCRIBED RECORDS FROM SANTA BARBARA COUNTY), IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

EXCEPTING THEREFROM THE OIL, MINERALS, AND OTHER RIGHTS GRANTED TO EDWARD W. HASKELL BY DEED DATED DECEMBER 24, 1864, RECORDED IN BOOK "B", PAGE 153 OF DEEDS, OTHER THAN 50/100THS INTEREST IN AND TO ALL OIL, GAS, HYDROCARBON SUBSTANCES AND OTHER MINERALS IN OR UPON OR THAT MAY BE PRODUCED FROM SAID LAND AS GRANTED TO THE MCKEVETT CORPORATION IN DEEDS RECORDED JANUARY 12, 1942 IN BOOK 652, PAGE 127 AND RECORDED APRIL 12, 1960, IN BOOK 1854, PAGE 465 BOTH OF OFFICIAL RECORDS.

APN: 100-0-010-315 and 100-0-010-325 and 100-0-010-425 and 100-0-010-485

APPENDIX A

GEOBRUGG

Rocco® Rockfall Protection System Maintenance Manual
RXI-100 Energy Class 5

7290.200.101
March 10, 2008
Revised May 23, 2008

GEOBRUGG®

ROCCO®

Rockfall Protection System

Maintenance Manual RXI-050/100/200

RXI-050 Energy Class 3 (500 kJ)

RXI-100 Energy Class 5 (1000 kJ)

RXI-200 Energy Class 7 (2000 kJ)

- **Lifespan**
- **Inspections**
- **Criteria for repair and replacement**
- **Emptying and cleaning barriers**
- **Repair and replacement of components**
- **Inspection checklist**
- **Rope assembly drawings**
- **ISO 9001 Certificate**

Proof of energy absorption capacity of the system based on one-to-one field tests

Testing agency: Swiss Federal Institute for Forest, Snow and Landscape Research (WSL)
Birmensdorf, Switzerland

Certification agency: Swiss Agency for Environment, Forest and Landscape (SAEFL)
Bern, Switzerland

Edition: 100-N-FO / 02
Date: 04/29/05

Purpose and organization of the maintenance manual

This maintenance manual serves the regular maintenance and repair of a rockfall protection barrier, in order to guarantee a long service life for the structure and its safe and unrestricted function after impacts and during its lifespan. The maintenance manual should be understood as recommendation. Only standard situations are described. In case of extraordinary situations this manual may not be relevant to or sufficient for maintaining or repairing the barrier. In particular cases it is recommended to ask the manufacturer for technical advice.

This maintenance manual consists of the following parts:

- Lifespan
- Inspections
- Criteria for repair and replacement
- Emptying and cleaning the barrier
- Repair and replacement of components
- Inspection checklists, rope assembly drawings
- ISO 9001 certificate

No claims are made that this document is complete. It describes standard applications and does not take into account any project-specific parameters. Geobrugg cannot be held liable for any extra costs that may be incurred for special cases. In case of uncertainties, please contact the manufacturer. The General Sales Conditions of Fatzer AG and all its subsidiaries are applicable.

Responsible for the content of this manual:

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Geobrugg Protection Systems
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CH-8590 Romanshorn, Switzerland

Tel. ++41-71-466 81 55
Fax. ++41-71-466 81 50
E-Mail info@geobrugg.com

Romanshorn, 04/29/05

FATZER AG Geobrugg Protection Systems
Hofstrasse 55 • CH-8590 Romanshorn

(Stamp / authorized signatures)

I Range of application

This maintenance manual is valid for the Geobrugg RXI-050, RXI-100 and RXI-200 rockfall barriers. Please refer to the corresponding system drawings and product manuals:

RXI-050	(500 kJ)	System Drawing GS-1054	Product Manual No. 112-N-FO
RXI-100	(1000 kJ)	System Drawing GS-1057	Product Manual No. 106-N-FO
RXI-200	(2000 kJ)	System Drawing GS-1060	Product Manual No. 102-N-FO

II Quality of the system components

The Geobrugg division of Fatzer AG has been certified since August 22, 1995, in accordance with the **Quality Management System Requirements (ISO 9001: 2000, Rev. 2004), registration no. 11774-03.** The certifying body is the Swiss Association for Quality and Management Systems (SQS), which belongs to **EQ-Net 9000**. The quality manual completely specifies how to test the system components (raw material, commercial and end products) comprehensively in order to exclude deficiencies in quality. The relevant certificates are attached as appendices.

III Functional efficiency of the barrier systems

The functional efficiency of the system is based on one-to-one rockfall tests, carried out and tested in accordance with the guidelines for approval of rockfall protection nets in Walenstadt, Switzerland. The one-to-one rockfall tests are carried out by dropping a block vertically into the middle field of a three-field barrier. The distance between posts is 10 m, and an impact velocity of 25 m/s is reached. These approval tests are carried out jointly by the Swiss Federal Expert Commission on Avalanches and Rockfall (FECAR) and the Swiss Federal Institute for Forest, Snow and Landscape Research (WSL).

IV Quality control for maintenance

An appraisal of the damages should be carried out using the checklist in the maintenance manual. The maintenance manual describes in detail the different steps that must be followed by local building contractors for maintenance of the barriers. Unfortunately a photograph of the damages is always subject to subjective criteria. Contact the manufacturer in case of doubt, in order to guarantee the continuing quality and functional efficiency of the barrier.

V Product liability

Rockfall, landslides, debris flows or avalanches are sporadic and unpredictable. The cause is human (buildings, etc.), for example, or forces beyond human control (weather, earthquakes, etc.). The multiplicity of factors that may trigger such events means that guaranteeing the safety of persons and property is not an exact science.

However, the risks of injury and loss of property can be substantially reduced by appropriate calculations that apply good engineering practices, and by using predictable parameters along with the corresponding implementation of flawless protective measures in identified risk areas.

Monitoring and maintenance of such systems are an absolute requirement to ensure the desired safety level. System safety can also be diminished through events, natural disasters, inadequate dimensioning or failure to use standard components, systems and original parts, but also through corrosion (caused by environmental pollution or other man-made factors as well as other external influences).

The system may be under tension from stones and boulders in the net. This must be taken into consideration when components are removed or ropes are released. For this reason the elements must be dismantled and replaced according to good professional practice. Geobrugg cannot assume liability for inappropriate disassembly.

Table of contents

Appendices

A Checklist "Inspection of barriers"

B RXI-050/100/200 rope assembly drawings

C ISO 9001 Certificate

1 Lifespan

1.1 Lifespan of one component

The lifespan of one component is influenced on one hand by **damage events**, which can impair the functional efficiency of an element, and on the other hand by corrosion, which affects the load-bearing capacity of an element. If a system element is no longer fully functionally efficient, as a rule it must be replaced. The exact point at which an element must be replaced must be determined for each individual element. If a component loses load-bearing capacity because of corrosion, its lifespan depends upon the selected **safety factor**. If the loss of load-bearing capacity (thinner metal cross section = lower tensile strength) renders the element unsafe, it must be replaced in any case. A combination of corrosion and damage event is also possible, if a damage event also damages the corrosion protection. This in turn impairs the lifespan of the component in question. An analysis of an RXI model rockfall protection construction can differentiate among 6 different elements, which are described in the following section on lifespan.

1.2 Lifespan considering corrosion

1.2.1 Post and base plate anchorages

The post anchors are left in the rough state, but they have the required corrosion addition of 2 mm on every external surface. Theoretically these anchors should fulfill their function in a neutral environment for at least 50 years.

1.2.2 Spiral rope anchors

The rope anchors consist of thick galvanized spiral ropes and an added corrosion protection tube in the loop area. The tube is also grouted in order not to expose the rope to the air. In a neutral environment, this corrosion protection should have an even longer lifespan than the post anchor.

1.2.3 Posts and base plates

These can be hot dip galvanized or untreated. The hot dip galvanizing of the posts is more for appearance, and not necessarily a longer lifespan. Hot dip galvanizing is, however, recommended for the base plates, because they lie on the ground and are soon covered with soil and boulders. This means they cannot be inspected directly.

1.2.4 Support-, RUNTOP-, anchor- and retaining ropes

In the basic construction design these ropes are made out of galvanized wires in accordance with DIN 2078. A general lifespan cannot be given since this depends greatly upon environmental influences. Corrosion and thus a thinning wire cross section is physically possible only if the zinc layer deteriorates (is used up). Tests on wire mesh covering in Germany indicated that zinc takes 15 to 20 years to deteriorate. However, the ropes can also be made out of special galvanized wire (Geobrugg Supercoating). Such aluminum-zinc alloys for protection against corrosion last at least two to three times longer than customary galvanized wires. In this way a lifespan of 40 to 60 years is possible.

1.2.5 Brake elements

The brake elements installed in the RXI systems are located outside the impact area and thus largely protected against mechanical damage.

The built-in brake rings consist on the one hand of hot dip galvanized steel tubes and on the other hand of short pressed aluminum compression sleeves. The hot dip galvanized steel tubes should fulfill their function for at least 40-50 years in a neutral environment. Since as a rule these brake elements are suspended above the soil, they dry out quickly. For this reason the danger of corrosion is relatively slight, which makes them last longer.

1.2.6 Rocco ring nets

These nets consist of robust, thick steel wires with a minimum diameter of 3.0 mm. The wires are specially galvanized, i.e., treated with an aluminum-zinc alloy (see section 1.2.4). The zinc equips the metal with an anodic corrosion protection layer. Its characteristics protect even when a couple of millimeters of the protective layer are gone, since the zinc acts directly over the gap. With Geobrugg Supercoating a lifespan of up to 60 years can be expected in a neutral environment.

2 Inspections

Rockfall protective barriers from GEOBRUGG need little or no maintenance if the kinetic energy released in rockfall events does not exceed the load limits for which the system was constructed. Lesser maintenance measures are nevertheless required after such impacts depending on the frequency.
Other repairs may be required in case of rockfall events with kinetic energy in excess of the construction-specific load limits. The most standard (if also seldom occurring) maintenance tasks are described below.
Rockfall events that occur only seldom obviously require less maintenance than those occurring frequently. Maintenance measures for events occurring often (daily) below the energy absorption limits of a system are generally limited to the routine inspections described below. Large events with rock avalanches in the range of the system limits can often require smaller repairs. Large events that exceed the system limits may require more extensive repairs under some circumstances.
As a rule the components used in a rockfall protection barrier must **only require maintenance** when rocks plunge into the barrier or when extensive corrosion is present, which impairs the expected lifespan. Regular inspections are nevertheless recommended in order to ensure the performance of the barrier, which can be impaired through rocks lying in the net. These inspections can also uncover possible damage to the system caused by impacts or corrosion.

2.1 Regular inspections

Interval for regular inspections

The appropriate interval depends above all on the following parameters:

- rockfall frequency
- vegetation

Quick check / routine check

In a standard situation two normal visual inspections per year are sufficient. If rockfall is frequent, more inspections may be called for. Inspections should be carried out before winter begins and after the end of winter.

Anchorage

Regular optical checks of the condition of the anchorage should be carried out every 10 years.

The regular quick check of the structure at least twice per year includes the following:

- Are there larger rocks lying in the net?
- Were brakes activated? To what extent were they strained?
- Was the useful height reduced by sagging ropes?
- If so, to what extent?
- Check the wire rope clips again with a torque wrench.
- Is corrosion visible?
- Clear out boulders, soil, broken rocks, dried leaves, etc. behind the barrier to avoid the formation of "ski jumps". This would impair the useful height, the flexibility and thus the energy distribution function.

2.2 Inspection after events

After reported or recorded events an immediate inspection is required. The following items must be checked (see checklist in appendix):

- Were brake rings strained? To what extent were they strained?
- By how much was the useful height of the system reduced?

- Is the net damaged? To what extent was it damaged?
- How many rings were broken?
- How many rings are bent?

- Were support-, anchor- and retaining ropes damaged (heavily warped or bent)?
- To what extent were they damaged?
- Were rope slings damaged?
- Were individual wires damaged?
- Was a section of the rope over tensioned?

- Were posts, base plates or connecting pins damaged?
- Is damage visible on the GEWI anchors of the base plates?
- Was a rod anchor bent? How badly was it bent?
- Was a rod anchor pulled out of the soil? How far was it pulled out of the soil?

- Is damage visible on the spiral rope anchors?
- Was a metal tube damaged?
- Was a loop damaged?
- Was a spiral rope anchor pulled out of the soil?
- How far was it pulled out of the soil?

If one or more of these elements was damaged, the damage must be appraised and staged in accordance with the criteria given in section 3, which follows.

2.3 Accessibility

The accessibility of the barrier should be guaranteed to the extent that all load-bearing components can be inspected. The necessary infrastructure will depend on the terrain. The figure below gives an example for a secure climbing ladder.

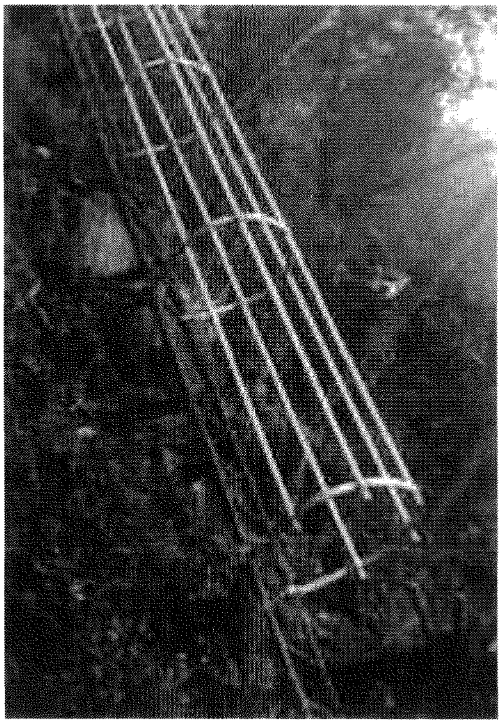


Figure 1 Example of a ladder for climbing

3 Criteria for maintenance and replacement

Section no.

3.1 General / useful height

see 5

Rubble may not be allowed to collect behind the barrier to more than 1/4 to 1/3 of the height of the net. Collected rubble under the net can lead to a dead load that overloads the entire system. For this reason collected rubble should be cleared out of these systems.

HINT: Routine cleaning of the systems is UNAVOIDABLE for orderly functioning, as a precautionary measure against unnecessary damages!

The residual useful height of the barrier after an impact is a good indicator for the damages that may be expected. A noticeable sag in the support rope or nets, as well as a considerable alteration in the angle of the posts, is an indication of strain in one or more of the brake rings, which may have to be replaced. Depending on the selected safety factor, when the barrier height is restored the system may need to be retensioned if the useful height was reduced by over 30%.

see 6.1

see 6.2

3.2 Rope sling with brake rings

see 6.3

The brake rings should be replaced if over 50% of the maximum tensile strain has been reached. This corresponds to an elongation of ca. 40 cm. Attention must also be paid to holding the additional sagging of the net caused by strain in the brake rings within limits, since this has considerable effect on the useful height of the structure. The relevant ropes can also be retensioned without replacing the brake elements simply by retensioning the support rope.

3.3 Ring net

As a rule, it is not necessary to replace entire nets even when some rings are bent.

- If individual wires have slipped out of a clip, they should be secured with a wire rope clip of the appropriate size.
- If ring wires are found that are obviously compressed or that have been badly bent, these rings must be replaced.
- If a ring exhibits fracture areas, the ring in question must be replaced.
- The entire net must be replaced if more than 10 rings exhibit fracture areas and/or at the same time several rings are compressed and badly bent.

see 6.4

see 6.5

3.4 Support ropes (with RUNTOP ropes)

see 6.6

With the exception of external influences, such as rockfall striking a rope, the life-span is exclusively determined by corrosion. The rope must be replaced if the rope cross-section shows substantial reduction. This is normally recognized by finding cracked outer wires. In this case the rope becomes brittle and loses its tensile strength within a few years. In case of doubt a piece of rope must be cut out so that the tensile strength of the rope can be checked. If the test result is positive, the entire rope must be replaced.
Whether the entire rope or only the affected rope section must be replaced depends on the extent of the damage to the rope. Ropes can be damaged by external forces. Crushed and cracked wires are an indication. Replacement of the rope or a section of the rope is always indicated if more than ca. 10% of the cross section is affected. In case only a section of the rope is affected, only the affected rope section must be replaced. If there are distinct kinks in the rope, however, it is recommended that the rope or a section of the rope be replaced. In case one or more strands are broken, the rope or this section of the rope must always be replaced.

3.5 Retaining ropes

see 6.7

The same criteria apply as for the support ropes. However, it is often more efficient and more cost effective to replace the entire retaining rope.

3.6 Lateral anchor rope, intermediate anchor rope

see 6.8

The same criteria apply for replacement of these ropes as for the support ropes. However, it is more efficient and more cost effective always to replace the entire rope.

3.7 Posts

see 6.9

The most important function of the posts is maintaining the useful height of the nets. Slightly bent posts need only be replaced if their condition leads to a considerable loss of height. It is recommended to replace a post if it is bent more than 15°.

3.8 Hinge tube between post and base plate

see 6.10

Impacts onto the post can bend or break the hinge tube between post and base plate (predetermined breaking point in order to avoid damages to the base plates and anchorages). Bent or broken hinge tubes must be replaced.

3.9 Base plate

see 6.11

Only very severe impacts, those that reach or exceed the construction limits, should impair the base plate. If plates in the area of the rope assemblies, the support block or the anchor holes are bent, a replacement is in order. Slight warping is not acceptable if the weld seams are affected. If a weld seam is defective, the base plate must be replaced.

3.10 Wire mesh

see 6.12

The wire mesh prevents smaller rocks from falling through the ring net. For this reason the entire surface must be covered with wire mesh. Especially the gaps between the bottom support rope and the soil must be covered. The wire mesh can be pulled down in an impact and must be reattached or replaced. Broken wires in the wire mesh lead to gaps. For this reason these areas must be covered with new wire mesh.

3.11 Anchorage of the posts

see 6.13

Impacts, especially those next to the posts or base plates, can damage anchors. In case a rod anchor (GEWI anchor) is badly bent at the point (>15°) or cracks are visible, the anchor must be replaced. The anchor also must be replaced if it is pulled over 3 cm out of the soil, since under some circumstances its load-bearing capacity may be lowered.

3.12 Spiral rope anchors

see 6.14

Spiral rope anchors need only be replaced if serious damage to the wires is present. If one of the steel tubes of an anchor head is damaged, this does not lower the load limits. It can, however, lead to a lowered lifespan because of the reduction in corrosion protection. The anchor must also be replaced if it has been pulled over 3 cm out of the soil, since its load-bearing capacity may have decreased.

4 Tools for maintenance of rockfall barriers

4.1 Recommended tools

The following tools should be on hand for maintenance:

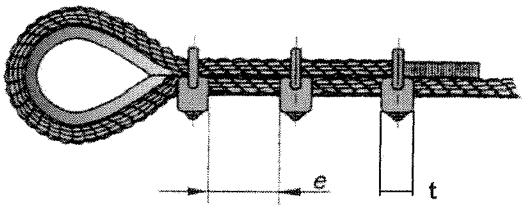
- one or two 6 m long ladders
- two hand winches with 20 kN tractive force (e.g., HABEGGER model hand winch)
- two hand winches with 7.5 kN tractive force (e.g., LUGAL model hand winch)
- various slings, each 1 m in length
- 5/8" and 7/8" shackles
- torque wrench, range 50-120 Nm
- socket or fork wrench set
- various tools, such as hammer, pliers, etc.
- hemp ropes
- tape measure
- angle water level
- cutting disk
- two eccentric clamps, small, for ropes with a diameter to 16 mm (draw tongs for wire ropes)
- at least two eccentric clamps, large, for ropes with a diameter to 22 mm (draw tongs for wire ropes)
- two complete sets of monkey wrenches

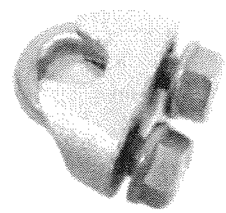
4.2 Applying the wire rope clips

The first wire rope clip is attached close to the thimble or the loop. The wire rope clips must be spaced so that the distance between them **e** is a multiple of from 1.5 to 3 times the width **t** of the wire rope clip.

The clip stirrups ("u-bolts") are always applied on the unstressed rope end, the jaws ("saddle") always on the stressed rope ("Never saddle a dead horse").

Extract from EN 13411-5 (DIN 1142)





Nominal size [mm]	Required torque (1) [N * m]	Required number of wire rope clips	Wrench width [mm]
16	55.0	4	22
19	75.0	4	22
22	120.0	5	24
22 GEOBINEX	120.0	10	24

Table 1 Torque values and number of wire rope clips

The torque values given apply to greased screw-nut connections.

During installation and before starting operation, tighten the hexagonal nuts to the required torque.

After installation of the barrier, the torque of the rope connections on the lateral and upslope anchors must be rechecked or readjusted.

(1) The torque values given are 10 % higher than those recommended in the standard. This is based on the deviation in common torque wrenches.

5 Emptying and cleaning the barrier

There are different ways to clean rocks, boulders or soil out of rockfall barriers. The method used depends on the local border conditions and the amount of material in the nets.

Warning: **The system may be under tension from the rocks and boulders found in the net. This must be taken into consideration when removing components or releasing ropes!**

5.1 Taking the ring net off the top support rope

The shackles on the top support rope and on the edge towards the neighboring ring nets are removed. The net can be taken down and laid upon the ground. Then the net can be manually or mechanically cleaned with a front-end loader or similar. Care must be taken not to damage the ring net. On very steep slopes, the rocks slide and roll downslope on their own after the net has been taken down. Make sure that the rocks rolling down the slope do not cause any damage (see figure 2).

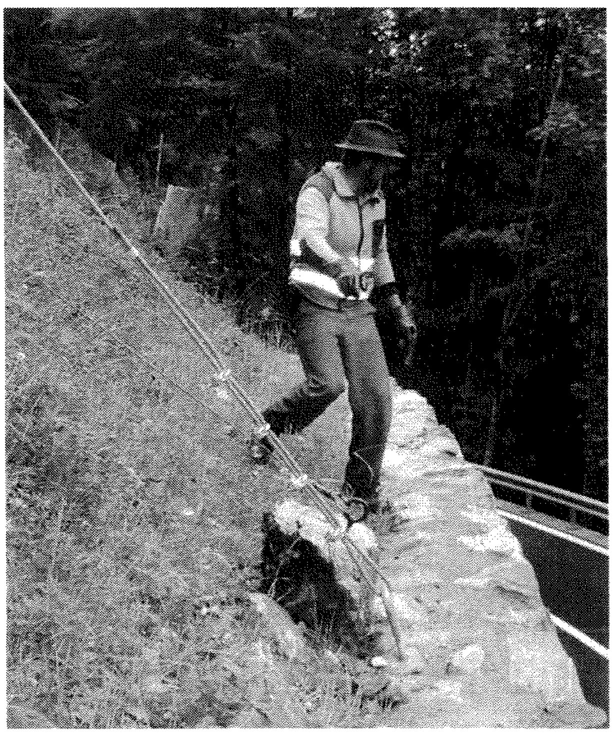


Figure 2 Emptying rocks safely (with rope secured)

Be aware that the filled ring net can be under great tension because of the weight in the net. This should be taken into consideration when removing the shackles. The net should be secured with additional ropes (three to six ropes), which are fastened above on the net (these are best applied on the second ring row), guided over the support rope and either held by workers or guided with hand winches to anchors laid uphill. On very steep slopes, the bottom edge of the net can also be secured in a similar way. Then the entire net with its contents can be laid down like a basket on safe ground.

If the tension in the net is too great for the solutions described above, an auxiliary rope with a length of ca. 30 m must be pulled through the second ring row and over the post head. The rope is fastened to the upslope anchors on the left and right next to the field that sustained an impact. To empty out the net, a rope hoist (Habegger) must be installed on one side. After that, the shackle is released with the aid of pliers and removed. The net can now be emptied out while maintaining control over it. Care must be taken in unfavorable conditions that the barrier is not allowed to tip over upslope.

5.2 Removing the nets from the bottom support ropes

This method can be employed if the lower shackles are accessible, even when there are rocks in the net. As the shackles are released, the net contents slide or roll out of the net. If the weight of the rocks makes it impossible to raise the net, the net can be hoisted up with ropes and hand winches.

In order to transport the rocks or boulders safely downslope, it is recommended to convey them safely and efficiently downwards using gutters or large pipes made of synthetic material. These can be filled manually above in the barrier.

5.3 Breaking up the rocks

Large blocks that cannot be lifted out or evacuated must be broken up into smaller sizes. The following methods come into question depending on the situation:

- manual
- explosive (see figure 3)
- expansion cement ("cold explosive agent"). For this the rocks must be drilled, filled with the "propellant" (e.g., the Betonamit company) and water added. After about one day the rock is broken and can be cleared.

Figure 3 Breaking up a block with explosive

5.4 Emptying nets manually or mechanically without taking down the net

In certain cases one can reach the barrier from the back and remove rocks manually or with a front-end loader or similar. Care must be taken not to damage the net while emptying it. The material can then be dumped downslope, over the net, ready for removal. If the area behind the barrier is accessible, the truck can be loaded directly.

If a crane is available, the rocks can also be fitted with a wire hanger. For this the rocks must be drilled. The eyelet bolt can be fastened chemically (e.g., HILTI "HIT"), or a drilled-through screw can be fastened with a nut. So-called heavy-duty dowel pins, which use friction for fastening, can also be used.

Figure 4 Providing the block with towing hooks

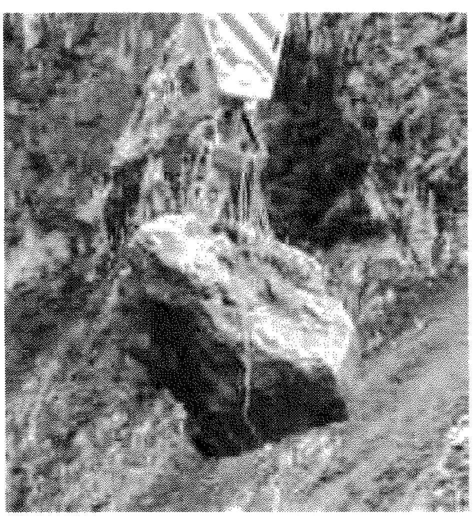


Figure 5 Lifting out the block with the aid of a crane

6 Repairing and replacing components

6.1 Tensioning sagging nets

The affected top support rope must be fastened with a hand winch on the rope end that forms a loop with the wire rope clips (1). The other end of the hand winch can be fastened on the top support rope as shown (2). The hand winch is tensioned (3). The wire rope clips are released until the rope can slide through. Then the rope is again tensioned with the hand winch. Finally the wire rope clips are tensioned with a torque wrench with the right torque value (4).

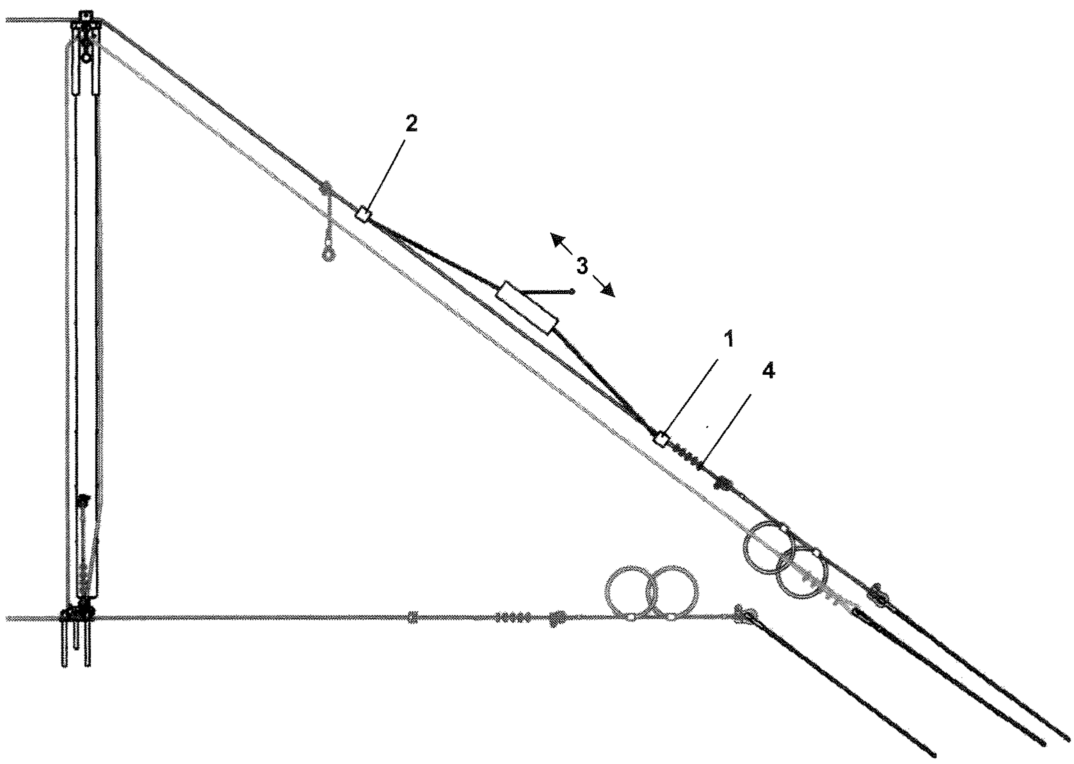


Figure 6 Tensioning sagging nets

6.2 Tensioning RUNTOP ropes and replacing RUNTOP rope rods (predetermined breaking point)

After an impact the broken RUNTOP rope rods must be replaced in the impact area and the RUNTOP ropes retensioned. To do this it is recommended to release the RUNTOP rope wire rope clips on the side of the RUNTOP rope that points in the direction of the impact. In this way it is possible that the corresponding RUNTOP rope end with the wire rope clip can be refastened in the correct position in accordance with the manual. Afterwards a small come-along winch (e.g., LUGAL type) can be attached on the other side of the RUNTOP ropes between RUNTOP and support rope. Then the clip on this side is released, the RUNTOP rope with the rings on it is retensioned, and the clip reattached as per the manual.

6.3 Replacing rope slings with brake rings

A hand winch is fastened to the support rope on one side and on the other side to the spiral rope anchor (1). The rope is tensioned until the rope sling with brake rings (2) is no longer under tension and can be replaced. After releasing the hand winch, the support rope must be retensioned as described in section 6.1.

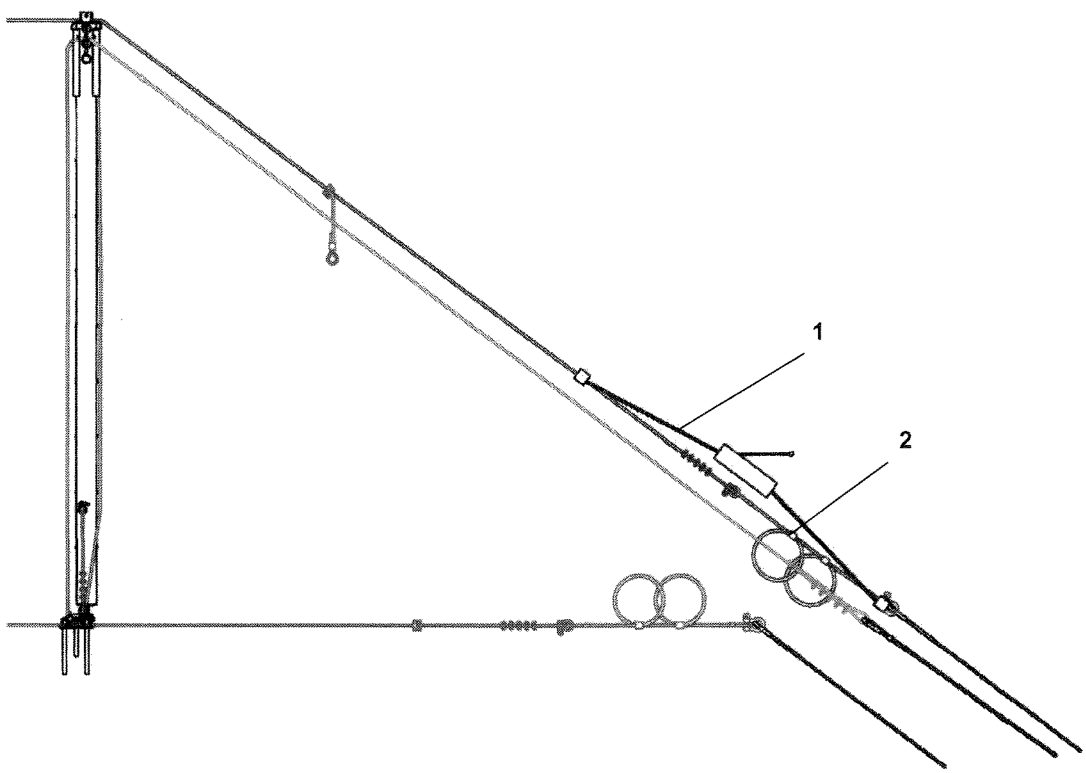


Figure 7 Replacing rope slings with brake rings

6.4 Replacing Rocco rings in a ring net

The simplest way to replace individual rings is by installing a single ring in the net with shackles. Appropriate prefabricated rings can be obtained from Geobrugg. Care should be taken to order the right type of ring. The rings can then be fastened to the neighboring rings with four shackles. Afterwards the damaged ring is cut out.

In order to replace a ring identically to the rest of the net, the following easily carried out procedure must be followed:
A portion of a replacement rope with a diameter of 4 to 8 mm is pulled through the (4) rings adjacent to the affected ring. This temporary rope is tensioned to a diameter that is smaller than the single ring, and then fastened with a wire rope clip (1). Now the ring bundle can be inserted into the four neighboring rings (2) by sticking a wire end into the neighboring rings. Then the bundle can be rotated until the entire bundle has been inserted. Then the ring bundle is fastened with three wire rope clips of the right size along the length of the ring (3). Finally the temporary rope can be released.

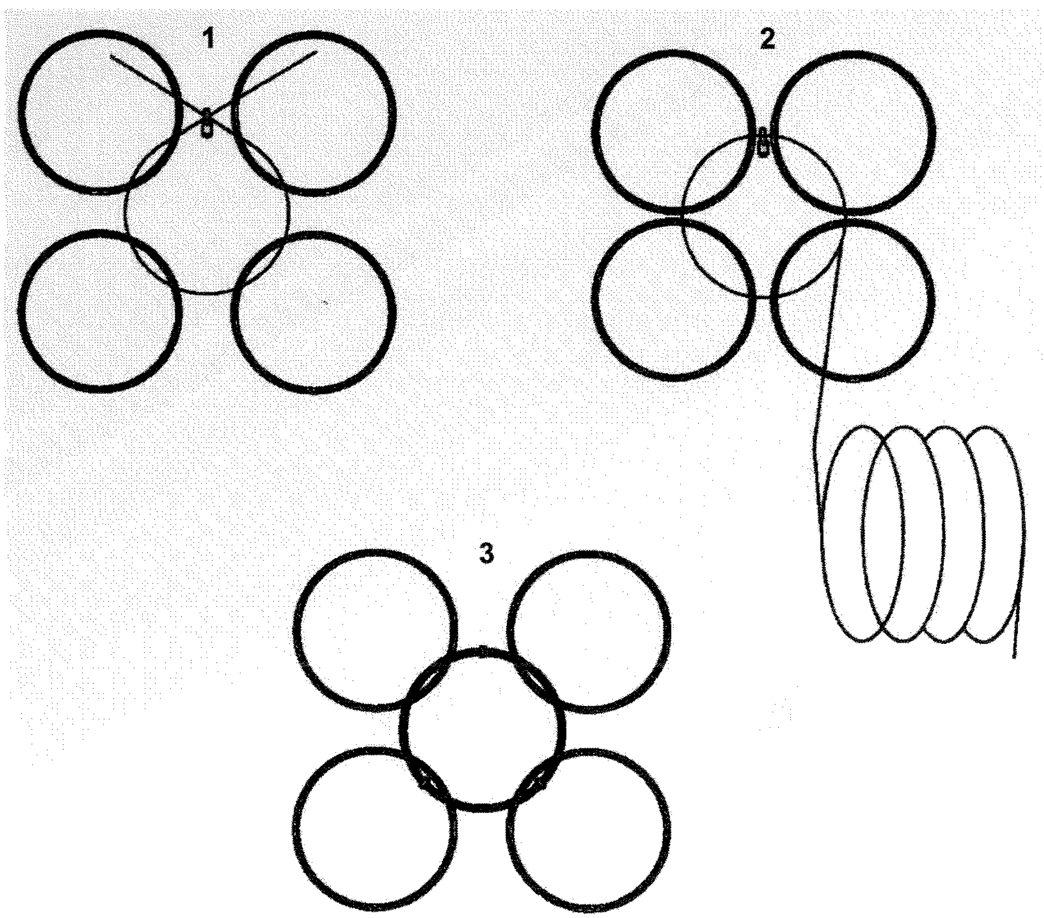


Figure 8 Replacing a Rocco ring

6.5 Replacing a ring net field

Be aware that rocks may be lying in the net and could fall out of the net when it is removed. Above all, the required safety precautions must be taken. Remember that the nets could be under high mechanical tension because of the rocks in the net.
A net field is replaced as follows:

a) The shackles to the neighboring nets are released.

b) Depending on the situation, either the shackles on the top support ropes or the shackles on the bottom support ropes are released first. If the net is full of rocks, and there is danger of the rocks rolling downwards in an uncontrolled fashion, the net can be released in a way that is the reverse of the installation of the net.

c) Now the net can be removed.

d) The nets can be laid down still packed on the appropriate positions between the two neighboring posts. It is best to begin with a field on the border. The top edge rings of the ring nets are marked with paint. For the assembly, pull a further rope through the second row of the ring net. The net should not yet be unfolded.

e) The nets are raised into position with a further rope. This rope is fastened on one of the neighboring posts (1), pulled through rings in the second row (2) and guided over the rope fastening of the other post to its base plate (3). The rope is tensioned with a hand winch (4) until the net is at the height of the top support rope. During this step the packaging of the nets can be cut open.

f) Now the net can be opened like a curtain and can be attached on the end with an additional shackle to the vertical rope if on a field lying on the border, or else to a neighboring net.

g) The net is then temporarily fastened to the top support rope with a pair of shackles.

h) If needed the net can be pulled to the other end with a hand winch.

i) After this the additional rope can be taken off and prepared for the next net. This neighboring net is fastened to the top support rope in the same manner.

j) This procedure is repeated until all nets are temporarily fastened to the top support rope.

k) If needed the nets can be shifted sideways until they are evenly distributed along the entire fence line.

l) The rings at the bottom of the net are similarly fastened temporarily to the bottom support rope with 2 or 3 shackles.

m) The nets are permanently fastened as follows: The four ROCCO rings next to the post are installed on the RUNTOP rope (5), the rest on the support rope (6). The corner rings with the end posts are fastened according to the product manual of the specific barrier.

n) This procedure is repeated until all nets are fastened on the support ropes and connected to each other.

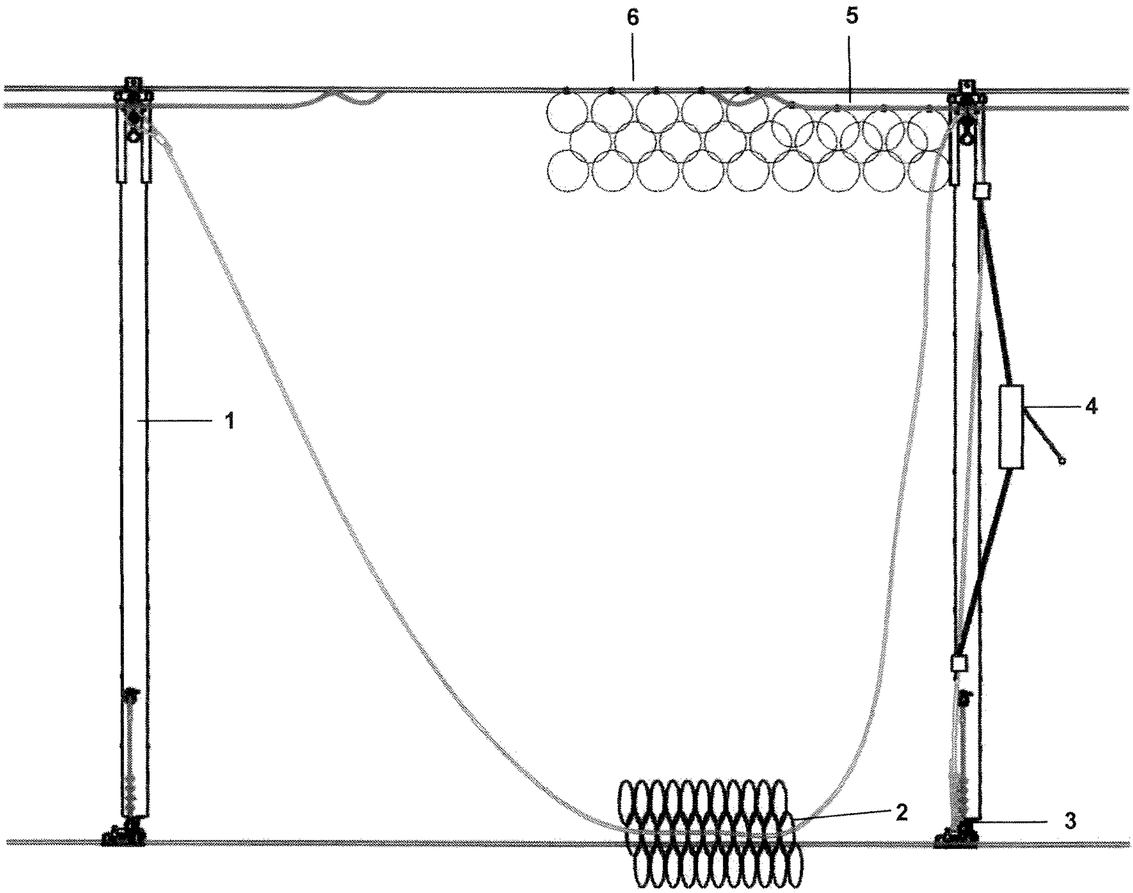


Figure 9 **Replacing a Rocco net**

6.6 Replacing a support rope (incl. RUNTOP ropes)

It is only necessary to replace a support rope if the rope itself was damaged. Normally only the rope slings with elongated brake rings are replaced.

6.6.1 Top support ropes

Depending on the situation, it is probably simplest to dismantle the affected support rope and the ring nets completely and take off all shackles on the top and edges of the ring nets. The whole is then reassembled with a new support rope. It may be more cost effective in some circumstances to fasten the ring net temporarily on the upper rings with an auxiliary rope that is led through the second ring row and suspended from one post to the next and tensioned. Then only the shackles on the upper rings of the net need to be taken off. Every situation calls for a decision as to which procedure is more cost effective.

The support ropes are replaced as follows (also see corresponding product manual):

a) Make sure that the lateral anchor ropes of the affected segment are in the right position. Make sure that the retaining ropes are in the right position (i.e., the angle of the post must be correct). These ropes may possibly need replacing and/or rearranging if they are also affected.

b) Take off the wire rope clips on the end of the support ropes. Lay the old support ropes on the ground and then take them away.

c) Roll out the new support rope on the ground, downslope and parallel to the fence line.

d) The top support ropes are guided over the post head.

e) The RUNTOP rope runs on the downslope side of the post and is fastened onto the support rope on the ends (with thimbles/loop) by shackle. It is centered across from the post, stretched and fastened on both sides before the loop with a wire rope clip EN 13411-5 (DIN 1142).

f) After the support ropes and RUNTOP ropes have been fully installed, the reassembly of the nets can be begun.

g) New wire rope clips should always be used for the end loops.

6.6.2 Bottom support ropes

Normally only the shackles on the lower rings are taken off for the preparatory tasks.

The procedure is as follows:

a) Take off the wire rope clips on the end of the support ropes. Remove the old support ropes.

b) Roll out the new support rope on the ground, downslope and parallel to the fence line.

c) The support ropes are guided under the rope guiding tube onto the base plate. Therefore, they have to be guided underneath the overturn securing rope of the post.

d) The RUNTOP rope runs downslope of the base plate and is fastened on the ends (with thimbles/loop) by shackle to the support rope. It is centered across from the post, stretched and fastened on both sides before the loop with a wire rope clip EN 13411-5 (DIN 1142).

e) After the support ropes have been fully installed, the nets can be attached to the support ropes with the shackles.

h) New wire rope clips should always be used for the end loops.

6.7 Replacing a retaining rope

Depending on the situation, the post for the affected retaining rope must first be secured with a hand winch and brought back to the correct angle before the retaining ropes can be relinquished. The angle of the post must look like this:

- Up to a slope inclination of 30° (0°< ß 30°) the post axis must be at an angle of 15° to the horizontal downhill.
- For steeper slopes (30°< ß 45°) the post axis must be at an angle of 75° to the slope.

Then the brand-new retaining rope is installed. Retaining ropes are installed as follows:
Hang the loop of the retaining rope on the head of the post. The end of the rope is then inserted into the anchor loop. The loop thus formed is fastened with wire rope clips EN 13411-5 (DIN 1142). New wire rope clips should be used for the loop.

6.8 Replacing a lateral anchor rope, intermediate anchor rope

If the lateral anchor rope is not severed, a new rope must be installed before releasing the damaged rope. Otherwise the system could collapse.

- Hang the loop of the lateral anchor rope on the head of the post.
- The end of the rope is then inserted into the anchor loop of the lateral anchor, tightened and fastened with wire rope clips in accordance with EN 13411-5 (DIN 1142).

6.9 Replacing a post

If a middle post must be replaced, in most cases the defective post can be removed without disassembling the system. To do this, the securing rod on the post head is released, then the hinge tube on the base plate is removed, and finally the post is pulled backwards out of the support block. Then the defective post is removed and a new one installed in the reverse sequence.

If this procedure is impossible, or if a border post must be replaced, the entire barrier must be turned over uphill. For this, first lateral anchor ropes, intermediate anchor ropes and downslope anchor ropes must be released. Generally the top support ropes must be released by slightly releasing the wire rope clips on the loops in order to be able to turn over the barrier. Then the entire barrier can be turned over uphill. The new post is laid next to the damaged post. Now all ropes on the head of the post are hung around the new post. The old post can be released and the new one installed on the base plate.

6.10 Replacing a hinge tube between post and base plate

Before replacing a bent or broken hinge tube, the post must first be drawn back into its position on the support block. After that a new connecting pin is inserted and fixed with spring cotters.

6.11 Replacing a base plate

The procedure for replacing a base plate is the same as for replacing a post.

6.12 Replacing wire mesh

Cut through the baling wire and remove the damaged mesh wire. Replace with new mesh wire and fasten this with double baling wire or stranded wire. Make sure that the mesh wire overlaps with the neighboring field. This also applies for the top support rope. Besides, the gap between the bottom support rope and the soil must be covered.

6.13 Replacing base plate anchors

If one or both anchors must be replaced, the base plate must be laid beside the old base plate at ca. 30 to 50 cm distance. Two new anchor holes must be drilled and new anchors cemented in. See product manual for the anchorage procedure.

6.14 Replacing spiral rope anchors

If a spiral rope anchor must be replaced, a new borehole should be drilled beside the old spiral rope anchor at ca. 50 cm distance. Care must be taken that the spiral rope anchor is long enough. See corresponding product manual for the anchorage procedure.

7 Final inspection

Above all, the following points must be checked upon completion of the maintenance:

a) Are the support ropes and the lateral anchor ropes connected to the right anchor?

b) Ring net on support rope and the RUNTOP ropes applied correctly?

c) Are the rope assemblies on the post foot and on the post head correct?

d) Were double clips for the RUNTOP rope / support rope separation correctly installed?

e) For the support rope separation, do the bottom support ropes go to the appropriate anchors and not to the base plates?

f) Right number of wire rope clips on the end connections of the ropes? Are the wire rope clips correctly placed?

g) Check the torque on the wire rope clips on the end connections.

h) Are the nets connected to each other correctly?

i) Is the border net correctly fastened to the vertical ropes?

j) Is the sag of the top support rope less than 3% of the post spacing?

Checklist "Inspection of barriers"

This checklist is intended for the inspection of a barrier. Please enter your observations, check off the relevant boxes and take pictures.

The paragraph numbers next to the box (e.g., no. 3.1) are reference numbers to the corresponding description in this maintenance manual.

The paragraph describes the criteria for repair and replacement.

Location:

General:

Impact area:

The following objects were found behind the barrier:

leaves / soil / wood	up to 20 cm	☐		
boulders / gravel	> 20 cm	☐	3.1	
rocks	up to 100 kg	☐		rocks up to ca. 35 cm size
	> 100 kg	☐	3.1	
	> 500 kg	☐	3.1 / 3.2	rocks over 60 cm size
	....	☐	3.1 / 3.2 /3.3	

All wire rope clips exhibit the right torque value.	yes	☐
	no	☐

Visible damage:

A) support ropes / rope slings with brake rings

deformed rope	yes	☐	3.4
	no	☐	
Net sagging between posts	up to 20 cm	☐	
	> 20 cm	☐	
	> 50 cm	☐	(3.1)
	> 1 m	☐	3.1
stretched out brake rings	up to 20 cm	☐	
	> 20 cm	☐	3.1
	> 50 cm	☐	3.2

B) retaining ropes

deformed rope	yes	☐	3.5
	no	☐	
angle between post and soil	ca. 70°	☐	
	ca. 80°	☐	(3.5)
	> 90°	☐	3.5

C) lateral anchor ropes, intermediate anchor ropes

deformed rope	yes	☐	3.6
	no	☐	

D) spiral rope anchor

damaged loop	yes	☐	3.12
	no	☐	
pulled out of the ground (in cm)	up to 1 cm	☐	
	> 1 cm	☐	(3.12)
	> 5 cm	☐	3.12

E) ring net

compressed rings/ broken wires	1 ring	☐	3.3
	< 5 rings	☐	
	> 5 rings	☐	

F) wire mesh

torn down / perforated	yes	☐	3.10
	no	☐	

G) posts / base plate

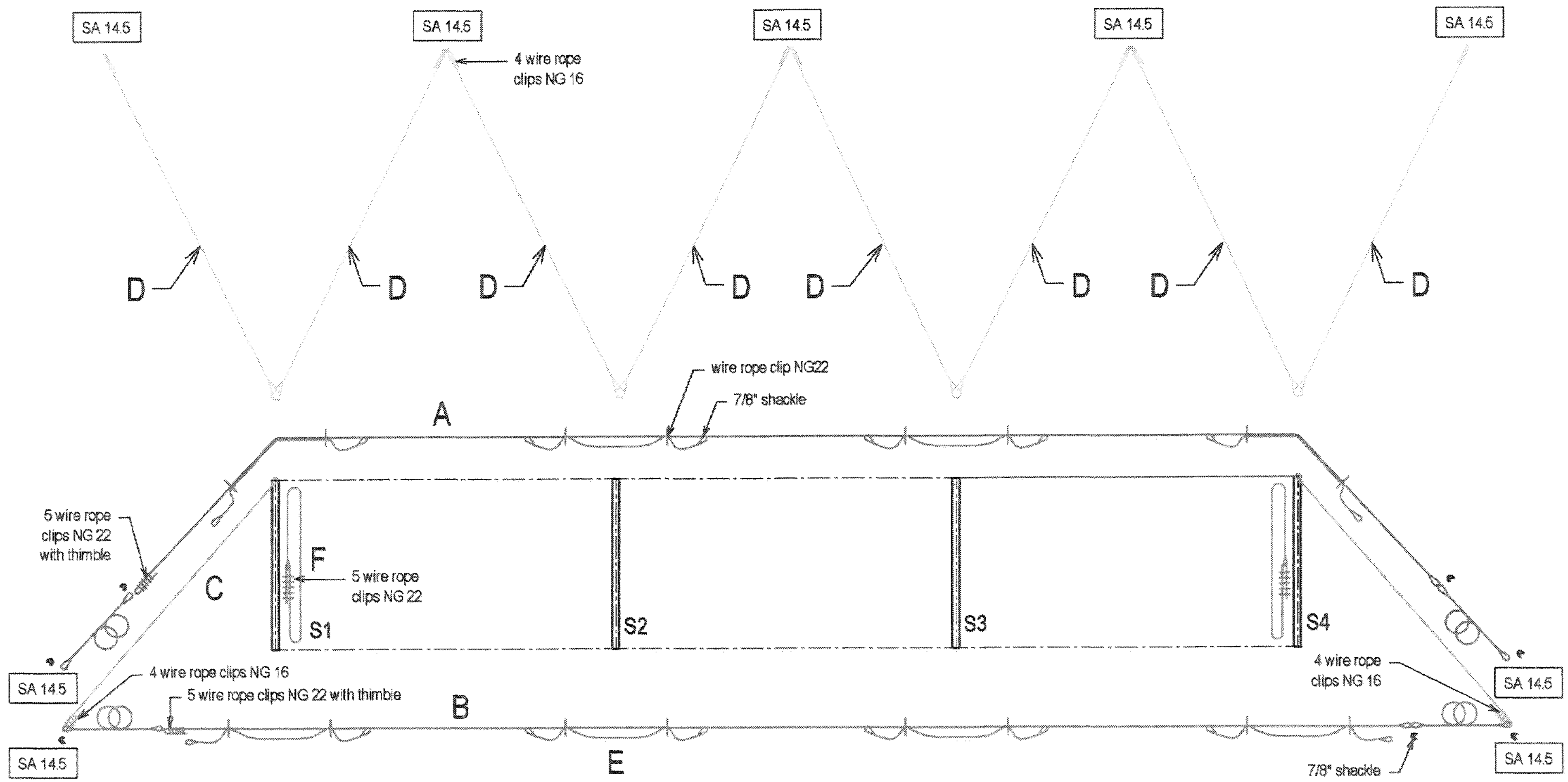
deformed post	yes	☐	3.7
	no	☐	
hinge tube bent	yes	☐	3.7 / 3.8
	no	☐	
deformed base plate	yes	☐	3.9
	no	☐	

damaged rod anchor	no	☐	
pulled out of the ground	> 3 cm	☐	3.11

Comments/ suggestions / sketches:

Name of inspector: ....

Date: .... Signature:...



A: Top support rope Ø20 mm with 4 brake rings GS-8002

B: Bottom support rope Ø20 mm with 4 brake rings GS-8002

C: Lateral anchor rope Ø16 mm

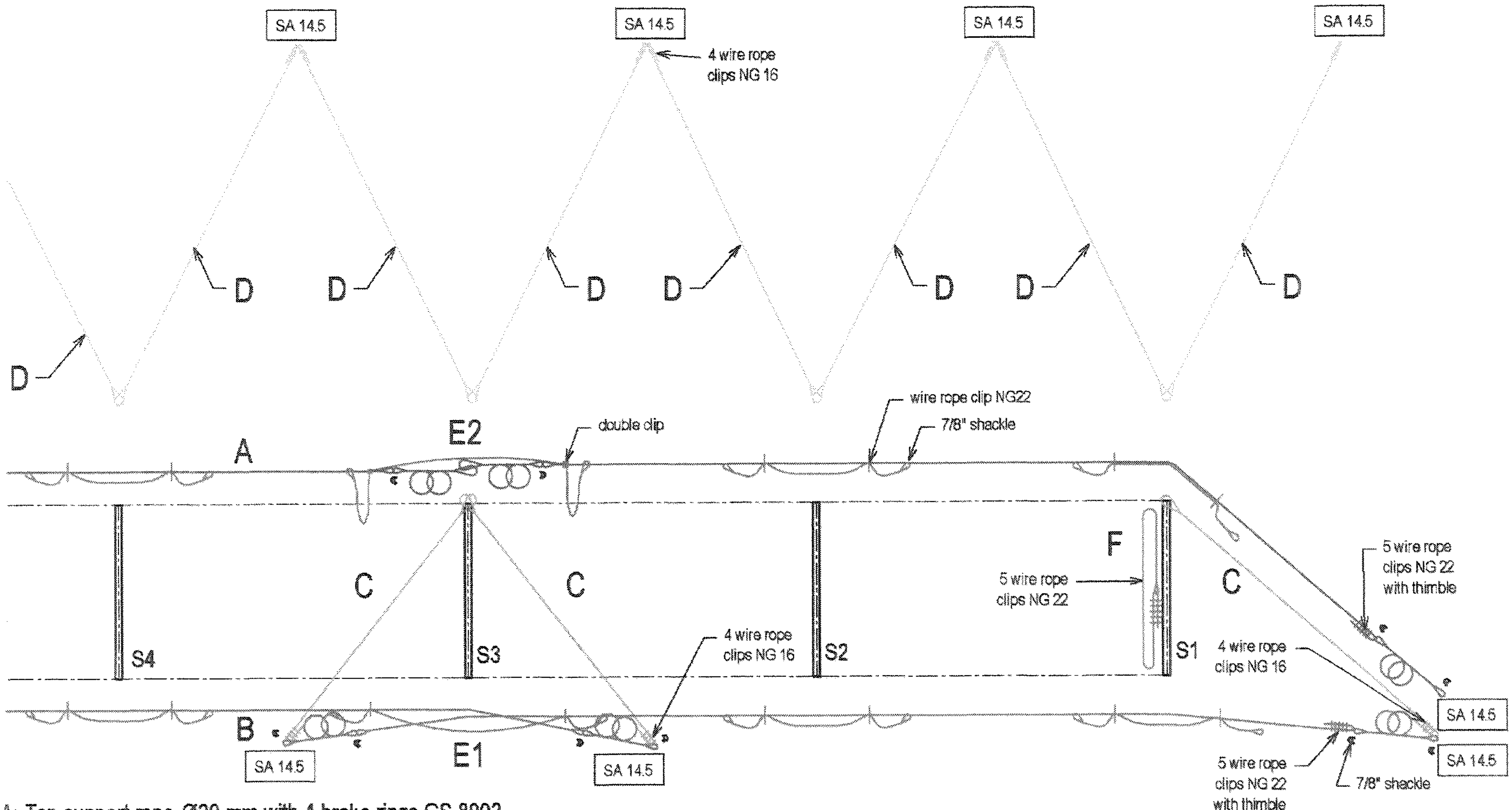
D: Upslope anchor rope Ø14 mm

E: RUNTOP rope Ø20 mm, L=5.6m

F: Vertical rope Ø20 mm

modification:	M:%	substitute for: GS-1055e ed. 24.01.05
-		replaced by:

RXI-050 Rope assembly Energy class 3 (500 kJ)	drawn	19.04.05	ol
	checked	19.04.05	ze
	edition	19.04.05	DF
FATZER AG CH-8590 Romanshorn GEOBRUGG	GS-1055 e		



A: Top support rope Ø20 mm with 4 brake rings GS-8002

B: Bottom support rope Ø20 mm with 4 brake rings GS-8002

C: Lateral anchor rope Ø16 mm

D: Upslope anchor rope Ø14 mm

E1: RUNTOP rope Ø20 mm L=5.6m / E2: RUNTOP rope Ø20 mm L=9.6m

F: Vertical rope Ø20 mm

C: Intermediate anchor rope Ø16 mm

modification:	M:%	substitute for: GS-1056e ed. 24.01.05
-		replaced by:

RXI-050
Support rope separation
Energy class 3 (500 kJ)

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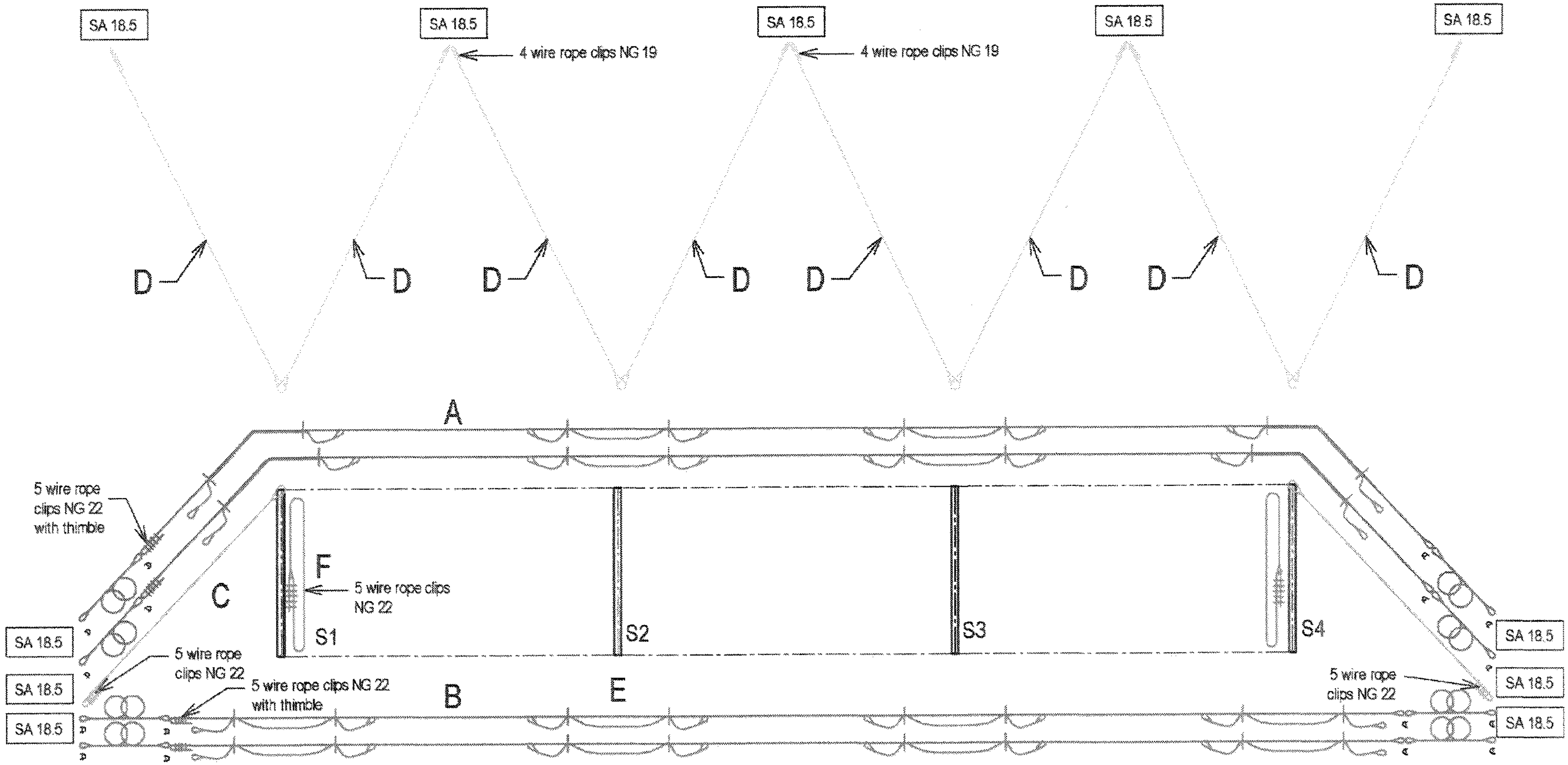
checked	19.04.05	Ze
edition	19.04.05	DF

FATZER AG
CH-8590 Romanshorn

GEOBRUGG

GS-1056 e



A: Top support rope Ø20 mm with 4 brake rings GS-8002

B: Bottom support rope Ø20 mm with 4 brake rings GS-8002

C: Lateral anchor rope Ø20 mm

D: Upslope anchor rope Ø18 mm

E: RUNTOP rope L=5.6m Ø20 mm

F: Vertical rope Ø22 mm

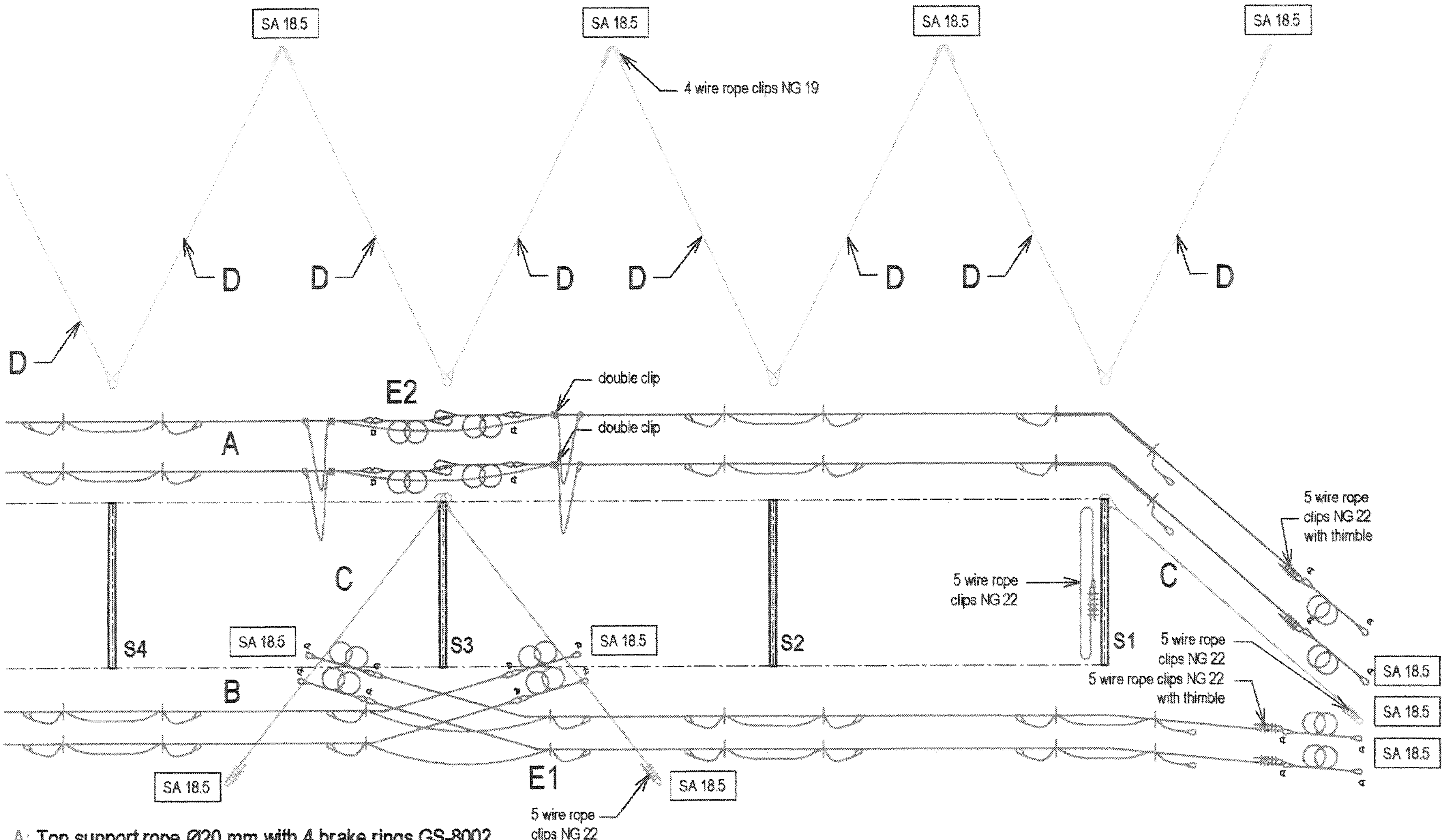
modification:	M:%	substitute for: GS-1058 e ed. 26.01.05
-		replaced by:

RXI-100 **Rope assembly** **Energy class 5 (1000 kJ)**	drawn	19.04.05	ol
	checked	19.04.05	ze
	edition	19.04.05	OF

FATZER AG
CH-8590 Romanshorn

GEOBRUGG

GS-1058 e



A: Top support rope Ø20 mm with 4 brake rings GS-8002

B: Bottom support rope Ø20 mm with 4 brake rings GS-8002

C: Lateral anchor rope Ø20 mm

D: Upslope anchor rope Ø18 mm

E: RUNTOP rope Ø20 mm E1: 5.6 m, E2: 9.6 m

F: Vertical rope Ø22 mm

G: Intermediate anchor rope Ø20 mm

modification:	M:%	substitute for: GS-1059e ed. 26.01.05
-		replaced by:

RXI-100
Support rope separation
Energy class 5 (1000 kJ)

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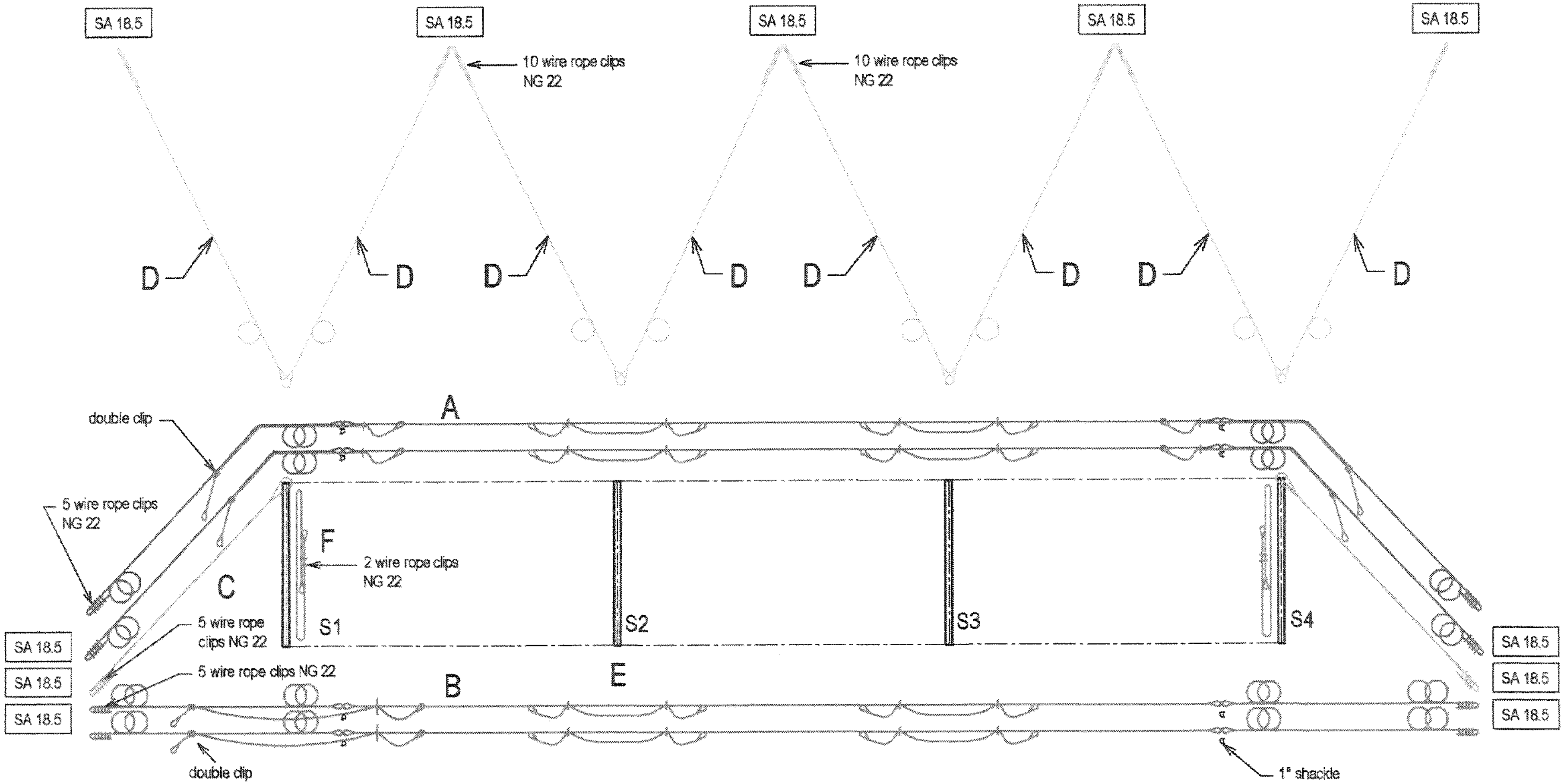
checked	19.04.05	ze
edition	19.04.05	OF

FATZER AG
CH-8590 Romanshorn

GEOBRUGG

GS-1059 e



A: Top support rope Ø22 mm with 8 brake rings GS-8002

B: Bottom support rope Ø22 mm with 8 brake rings GS-8002

C: Lateral anchor rope Ø22 mm

Upslope anchor rope Ø22 mm GEOBINEX with 1 brake ring GN-9017

E: RUNTOP rope Ø22 mm, L=7.6m

F: Vertical rope Ø22 mm

modification:	M:%	substitute for: GS-1061 e ed. 26.01.05
-		replaced by:

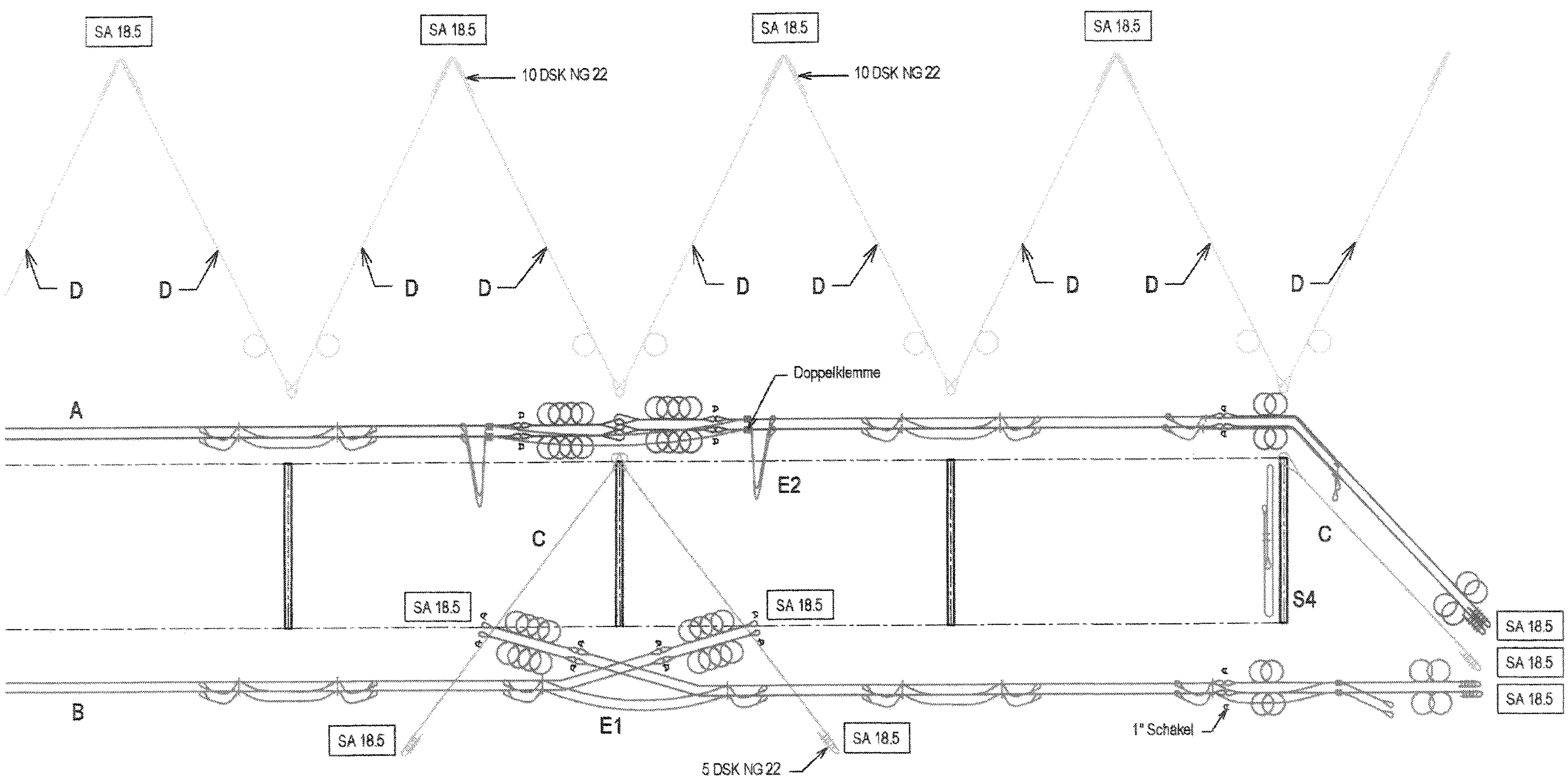
RXI-200
Rope assembly
Energy class 7 (2000 kJ)

drawn	19.04.05	ol
checked	19.04.05	ze
edition	19.04.05	or

FATZER AG
CH-8590 Romanshorn

GEOBRUGG

GS-1061 e



A: Oberes Tragseil Ø22 mm mit 8 Bremsringen GS-8002

B: Unteres Tragseil Ø22 mm mit 8 Bremsringen GS-8002

C: Seitenabspannung Ø22 mm

D: Rückhalteseil Ø22 mm GEOBINEX mit Bremsring GN-9017

E: Laufseil Ø22 mm E1: L=7.6 m, E2: L=15.6 m

F: Vertikalseil Ø22 mm

G: Zwischenabspannung Ø22 mm

Änderung	M:%	Ersatz für: GS-1062d Ausg. vom 26.01.05
-		Ersetzt durch:

RXI-200
Tragseiltrennung
Energieklasse 7 (2000 kJ)

Gezeich.	19.04.05	ol
Geprüft	19.04.05	ZE
Ausgabe	19.04.05	OF

FATZER AG
CH-8590 Romanshorn
GEOBRUGG

GS-1062 d

The Swiss Association for Quality and Management Systems

SQS herewith certifies that the company named below has a management system
which meets the requirements of the normative bases specified below
and issues the company

Fatzer Inc.

CH-8590 Romanshorn

certified areas

FATZER	GEOBRUGG	BRUGG ROPE STRUCTURES
Steel Wire Ropes	Protection Systems	Rope Structures

Field of activity

Rope and Netting Technologies

on the basis of the audit result the

SQS Certificate ISO 9001:2000

CH-3052 Zollikofen, August 4, 2004
This SQS Certificate is valid up to and including August 3, 2007
Scope number 17
Registration number 11774

President SQS	Managing Director SQS
X. Edelmann	T. Zahner

SCES 002, 023

IQNet

THE INTERNATIONAL CERTIFICATION NETWORK

CERTIFICATE

IQNet and SQS

hereby certify that the organization

Fatzer Inc.

CH-8590 Romanshorn

certified areas

Steel Wire Ropes, Protection Systems, Rope Structures

Field of activity

Rope and Netting Technologies

has implemented and maintains a

Management System

which fulfills the requirements of the following standard

ISO 9001:2000

Scope No: 17

Issued on: 2004-08-04

Validity date: 2007-08-03

Registration Number: ***CH-11774***

Dr. Fabio Roversi
President of IQNet

Theodor Zahner
Managing Director SQS

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APPENDIX B

ALBUS-KEEFE AND ASSOCIATES

Supplemental Geotechnical Investigation and Rough Grading Plan Review
Proposed 75-Lot Residential Development
Western Terminus of 10th Street, Santa Paula, California
May 3, 2006

7290.200.101
March 10, 2008
Revised May 23, 2008

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

May 3, 2006
J.N.: 1489.00

Ms Tiffany Sukay
Comstock Homes
321 12th Street, Suite 200
Manhattan Beach, California 90266

Subject: **Supplemental Geotechnical Investigation and Rough Grading Plan Review, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California.**

Dear Ms Sukay:

Albus-Keefe & Associates, Inc., pleased to present to you our Supplemental Geotechnical Investigation and Rough Grading Plan Review report for the proposed residential development at the subject site. This report presents the results of our review of geologic data, previous geotechnical reports for the site and surrounding area, review of aerial photographs, review of the rough grading plans for the site, exploratory drilling, laboratory testing, engineering and geologic analyses, and our conclusions and recommendations pertaining to the proposed site development.

We appreciate this opportunity to be of service to you. If you have any questions regarding the contents of this report, please do not hesitate to call.

Sincerely yours,

ALBUS-KEEFE & ASSOCIATES, INC.

Patrick M. Keefe
Principal Engineering Geologist
CEG 2022

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purposes of our investigation were to evaluate the nature of subsurface soil and geologic conditions, to evaluate their engineering characteristics, and to provide geotechnical recommendations with respect to site earthwork, as well as design and construction of structural foundations and associated site improvements. The scope of our work included the following:

- Review of published geologic and seismic data
- Review of aerial photographs and previous geotechnical reports
- Review of the referenced rough grading plans
- Review of previous geotechnical reports
- Drilling and surface logging of 13 exploratory borings utilizing a hollow-stem auger drill rig
- Drilling and down-hole logging of 3 exploratory borings utilizing a bucket auger drill rig
- Selective sampling of soil and bedrock materials
- Laboratory testing of samples obtained from site exploration
- Engineering analyses of data obtained from the exploratory excavations and laboratory testing
- Evaluation of site seismicity, slope stability, liquefaction, and settlement
- Preparation of this report.

1.2 PROPOSED SITE DEVELOPMENT

Based on our review of the referenced 20-scale rough grading plans, the site will be developed to create 75 residential building pads, three park sites, a detention basin, and associated streets. Several conventional and mechanically-reinforced segmental retaining walls reaching maximum heights of approximately 21 feet are proposed. Other anticipated site improvements include utility services, asphalt and concrete pavements and decorative hardscape.

Cut and fill grading will be performed to achieve the desired surface configuration for the proposed development. Maximum depths of proposed cuts and fills are approximately 24 feet each. Cut and fill slopes are proposed at a slope ratio of 2:1 (H:V) or flatter, to maximum heights of approximately 35 feet each. Fill-over-cut slopes are proposed within the site to a maximum height of approximately 43 feet.

Details of the future residential structures within the subject development are not known at this time. However, we anticipate that residential structures will consist of slab-on-grade, one- and/or two-story, wood-framed structures yielding relatively light structural loads.

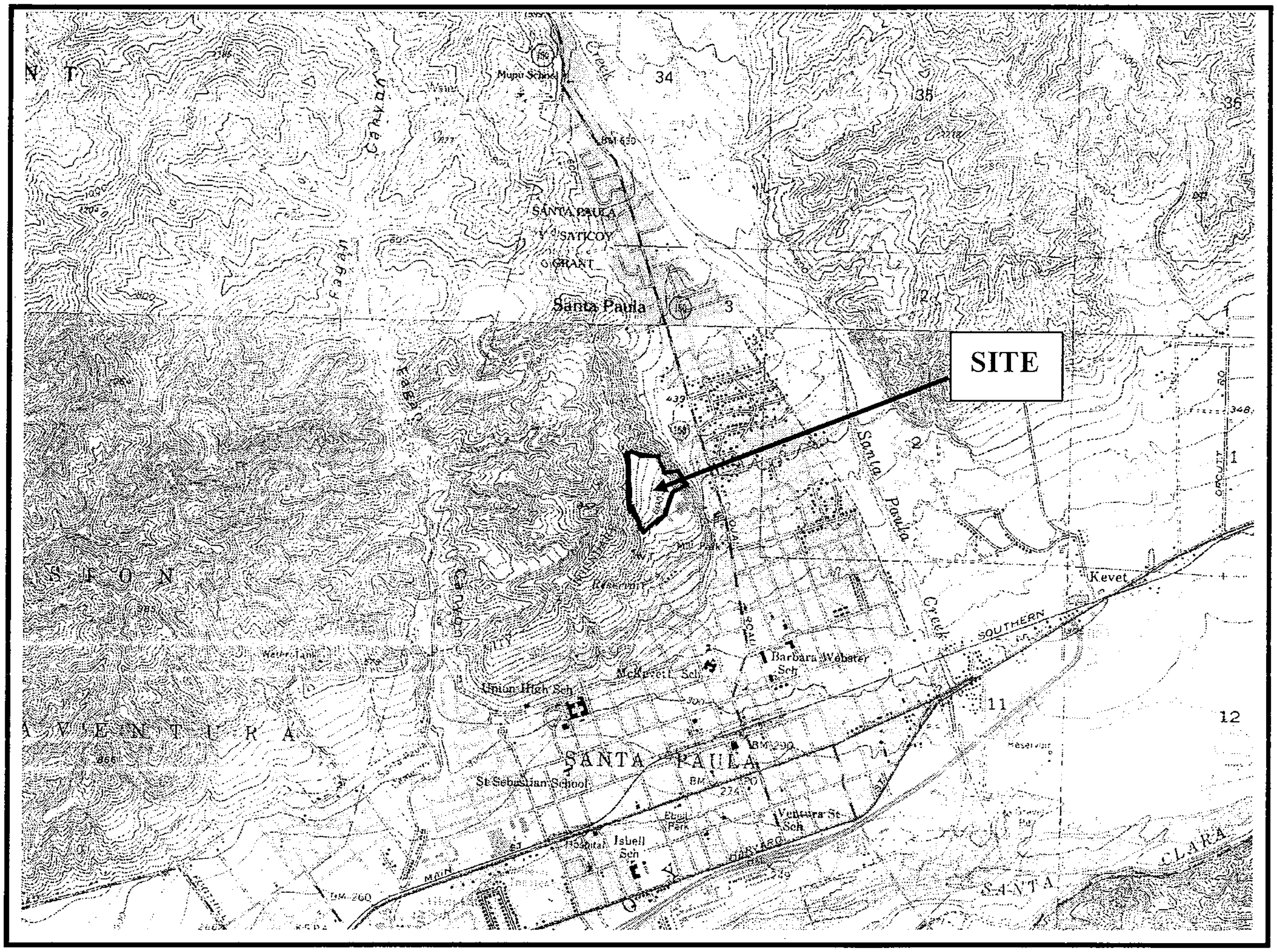
1.3 SITE LOCATION AND DESCRIPTION

The project area comprises roughly 14.1 acres of land in the City of Santa Paula, California. The site is located at the northerly terminus of 10th Street, immediately north of the existing Santa Paula Memorial Hospital facility. The property is bounded by the Santa Paul Memorial Hospital facility to the south, by existing residential developments to the southwest and east, and by vacant hillside terrain to the north and west. The site, in relationship to the surrounding area, is shown on Figure 1, Site Location Map.

The majority of the site is situated on an elevated plateau located northwest of the confluence of the Santa Clara River and Santa Paula Creek. Within the area of proposed construction, the topography varies from relatively flat within the southeastern portion of the site to gently sloping terrain that ascends towards the northwest at a maximum slope ratio of approximately 3:1. Beyond the gently sloping terrain, the northeastern and eastern portions of the site consists of a natural slope that descend as much as approximately 200 feet to the northeast and east at an average slope ratio of 1.5:1 to 2:1 (H:V). However, localized areas as steep as 0.5:1 (H:V) are present along the terrace deposit/bedrock contact near the top of the plateau and along flanks of drainage swales that descend to the bottom of the hillside. Beyond the westerly property boundary, the natural terrain ascends to the northwest at a slope ratio of approximately 2.5:1 to the top of a ridge line at an elevation of approximately 800 msl.

The site has recently been used as an avocado orchard. Existing improvements within the site are limited to some irrigation piping throughout the site, some asphalt paving, and fencing along the northern, southern and western perimeters. In addition, an east-west-trending overhead power line easement cuts across the southern portion of the site and a water tank is present immediately north of the northerly property boundary.

Vegetation within the area of proposed construction consists of a relatively large grove of mature avocado trees. The northerly and easterly descending slope is generally covered with heavy native brush and numerous trees. Evidence of a recent controlled burn was observed within some of the vegetation which has resulted in portions of the slope having relatively sparse vegetation, while others areas are covered with thick growths of vegetation.



NO SCALE

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FIGURE 1 – SITE LOCATION MAP
Comstock Homes
75-Home Residential Subdivision

From:
USGS 7.5 Minute Santa Paula Peak Quadrangle
1951 (photorevised 1988)
&
USGS 7.5 Minute Santa Paula Quadrangle
1951 (photorevised 1967)

2.0 INVESTIGATION

2.1 AERIAL PHOTOGRAPHS AND LITERATURE REVIEW

We have reviewed aerial photographs for the site and vicinity dating from April 1970 to April 2005. Based on our review, the site has been used for agricultural purposes from before 1970 through the present.

Geolabs – Westlake Village (Geolabs) prepared a due-diligence geotechnical investigation report and a grading plan review report dated March 21, 2005, and August 10, 2005, respectively, for the site. These reports involved excavation of two exploratory borings with a hollow-stem auger, two exploratory borings with a bucket auger, and excavation of 9 exploratory test pits with a rubber-tire backhoe. In addition, the Geolabs reports included logs from a previous geotechnical study performed by Earth System Southern California. The report prepared by Earth System was not available for our review; however, the logs from Earth System report were included in the Geolabs report. Pertinent exploration data and laboratory test data from the Geolabs report are included herein within Appendix C.

2.2 GEOLOGIC MAPPING

Geologic mapping of the site and surrounding areas was performed to identify existing improvements as well as the aerial distribution of the surficial earth materials within and adjacent the site. The results of our geologic mapping are plotted on the enclosed Geologic Maps, Plates 1 through 8 and Geologic Cross Sections, Plates 9 and 10.

2.3 SUBSURFACE EXPLORATION

Subsurface explorations were performed by engineering geologists from this firm in March and April, 2006. The subsurface exploration involved excavation and down-hole logging of 3 exploratory borings using a bucket auger drill rig and excavation and surface logging of 13 exploratory borings using a hollow-stem-auger drill rig. Depths of the borings varied from approximately 28.5 to 60.0 feet below the existing ground surface. Soil samples of representative earth materials were obtained at selected depths. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented in the Boring Logs, Plates A-1 through A-41. The approximate locations of the exploratory borings by this firm, and the exploratory borings and test pits by others are shown on the Geologic Maps, Plates 1 through 7.

Bulk, Standard Penetration Test (SPT) and relatively undisturbed samples were obtained at selected depths within the exploratory borings for subsequent laboratory testing. Relatively undisturbed samples were obtained using a 3-inch O.D., 2.5-inch I.D., California split-spoon soil sampler lined with brass rings. SPT samples were obtained using a standard SPT soil sampler. When utilizing the hollow-stem auger drill rig, during each sampling interval, the sampler was driven 18 inches with successive drops of a 140- pound automatic hammer. The number of blows required to advancing the split-spoon sampler and SPT sampler was recorded for each six inches of advancement. The total blow count for the lower12 inches of advancement per soil sample is recorded on the boring logs. When utilizing the bucket auger drill rig, the sampler was driven 12 inches with 12-inch drops of the drill rigs kelly bar. The total blow count was recorded on the boring logs. All samples were placed in sealed containers or plastic bags and transported to our laboratory for analyses. The borings were backfilled with auger cuttings upon completion of sampling.

2.4 LABORATORY TESTING

Selected samples of representative earth materials from the borings were returned to our laboratory for testing. Tests consisted of in-situ moisture and dry density, maximum dry density and optimum moisture content, expansion potential, soluble sulfate content, particle-size distribution, shear strength, Atterberg limits, and consolidation characteristics. Descriptions of laboratory test criteria and summaries of the test results are presented in Appendix B and on the boring logs in Appendix A.

3.0 GEOLOGIC CONDITIONS

3.1 GEOLOGIC SETTING

The project area is located northwest of the confluence of the Santa Clara River and the Santa Paula Creek, in the City of Santa Paula, California. The site is situated on the west flank an elevated alluvial terrace (Qt) associated with the ancestral Santa Paula Creek. Late Pleistocene-age non-marine terrace deposits unconformably overlie bedrock materials of the Saugus Formation. Bedrock beneath the site consists of non-marine sediments of early Pleistocene-age (Dibblee, 1992). Undocumented artificial fills are present within the site related to previous agricultural activities. The distribution of geologic units is shown on the enclosed Geologic Maps (Plates 1 through 8) and Cross Sections (Plates 9 and 10).

3.2 GEOLOGIC UNITS

3.2.1 Non-Engineered Artificial Fill

Non-engineered artificial fill associated with previous agricultural activities is present throughout much of the site. The fill material exposed in our borings were typically damp to very moist, soft to stiff or medium dense to dense, and consisted of clayey silt, silty clay, silt, silty sand with clay, and sandy silt with clay and locally contained rock fragments. Based on our observations, these fills generally range from a few inches to approximately 4 feet in thickness. Due to the relatively thin and sporadic distribution of the non-engineered fill within the site, these materials are not shown on the enclosed Geologic Maps.

3.2.2 Colluvium

Colluvial deposits were generally observed within the drainage swales of the easterly descending slope and thinly mantle the sloping bedrock. These materials are generally comprised of clayey silt and silty sand that are various shades of gray and brown, moist to very moist, soft to firm and porous. Varying amounts of gravels, cobbles and boulders associated with the terrace cap were observed locally within the colluvial materials. The colluvium is estimated to range in thickness from approximately 2 feet to as much as 10 feet within the drainage swales.

3.2.3 Terrace Deposits (Qt)

Late Pleistocene-age terrace deposits underlie the majority of the relatively level portions of the site and cap the underlying bedrock. The upper portions of the terrace deposits consist primarily of silt, clayey silt, sandy silt, silty clay, and sandy clay that are various shades of brown. These deposits are typically damp to very moist, stiff to hard, and have varying degrees of porosity. The lower portion of the terrace deposits consist of silty sand and sand that contain increasing amounts of gravel, cobbles and boulders directly over the bedrock contact. These materials were generally observed to be damp to moist and dense.

3.2.4 Saugus Formation (TQsa)

Early Pleistocene-age Saugas Formation underlies the entire project area. The Saugus Formation contains non-marine sediments that consist of clayey siltstone, siltstone, sandy siltstone, silty sandstone, and sandstone units with some thin clay seams.

The bedrock units were observed to be light brown, reddish-brown, pale olive-gray to olive-brown in color, soft to moderately hard, damp to moist, slightly to moderately weathered, and locally contains some calcium carbonate mineralization along joints.

Where observed, bedding within the Saugus Formation is massive to thinly bedded, but often indistinct with gradational contacts. Cross bedding and scour features were also observed.

3.3 GEOLOGIC STRUCTURE

3.3.1 Bedding and Jointing

Bedding plane surfaces within the sandstone units are generally gradational to moderately-well developed while bedding plane surfaces within the siltstone units are well developed and distinct where in contact with the sandstone. Bedding observed, and as indicated in the referenced geologic publications, typically strikes to the northeast and dips toward the southeast at angles varying from approximately 36 to 64 degrees.

Joint surfaces observed were generally high-angle, dipping toward the northeast, tight and locally contained calcium carbonate mineralization.

3.3.2 Faulting

Our subsurface investigation and review of geologic literature, previous geotechnical reports, and review of topographic expression for the site and near vicinity did not indicate the presence of faulting within or immediately adjacent the site. No faults were mapped within the site, nor is the site located within the boundaries of an Alquist-Priolo Earthquake Fault Zone.

3.4 LANDSLIDES AND SURFICIAL FAILURES

No deep seated landslides were observed within or adjacent the site. Some relatively shallow surficial failures were observed locally in over-steepened portions of the easterly natural slope.

Based on our review of geologic publications, the California Geological Survey has mapped the easterly-descending natural slope within a seismically-induced landslide hazard zone in accordance with the Seismic Hazards Mapping Act. A detailed discussion of our independent slope stability analyses is presented in Sections 4.2 and 5.3 of this report.

3.5 GROUNDWATER

Static groundwater was not encountered to the depths explored during this investigation (up to 60 feet). However, slight seepage was encountered in boring B-1 at a depth of 53.5 feet and locally perched groundwater was encountered in borings HS-1, 2, 4, and 5 near the contact with the bedrock at depths ranging from 20 to 28 feet. Based on our review of available publications and reports, historic high groundwater was estimated to be greater than 40 feet below the ground surface of the lower Santa Paula Creek Valley, which lies approximately 200 feet below the subject site.

4.0 ANALYSES

4.1 SEISMICITY

We have performed integrated historical and deterministic seismic hazard analyses utilizing computer programs *EQSEARCH* (Blake, 1989, updated 2004), *EQFAULT* (Blake, 1989, updated 2002), and *UBCSEIS* (Blake, 1989, updated 2000). A brief description of the programs and their functions are discussed below:

EQSEARCH performs historical seismic analyses that computes estimated ground motions at the site using a catalog of historical earthquake data within a 62-mile (100-km) radius of the site and a selected attenuation relation to model subsurface earth materials similar to the site. The results of analyses can be utilized to estimate how historical earthquakes may have shaken the site.

EQFAULT performs deterministic seismic analyses that computes estimated ground motion of the site using a selected attenuation relation to model earth materials similar to the site and a catalog of up to 222 digitized, 3-D California faults as earthquake sources within a 62-mile (100-km) radius of the site. The results of analyses can deterministically estimate how future earthquakes may shake the site.

UBCSEIS performs an analysis to compute the distance to California faults. The program indicates the type of fault as classified in the 1997 U.B.C. and develops the U.B.C. seismic design parameters based on the selected soil profile type.

Pertinent results from the historical and deterministic seismic hazard analyses are provided below:

Historical Event: Based on the computer program EQSEARCH, the earthquake that occurred on September 24, 1827 appears to have affected the site the most during the past 205 years. This earthquake was located approximately 25.6 miles (41.2 km) from the site and was estimated to be magnitude 7.0. Peak horizontal ground accelerations (PHGA) were estimated for the historical earthquake using Bozorgnia, Campbell and Niazi attenuation equation (1999) for Pleistocene soil sites. The largest estimated mean PHGA experienced at the site since 1800 is 0.11g (fraction of gravity) with a standard deviation of 0.07g.

Deterministic Event: Based on the computer program EQFAULT and using Bozorgnia, Campbell and Niazi attenuation equation (1999) for Pleistocene soil sites, the largest estimated mean PHGA is 0.64g with a standard deviation of 0.43g, associated with a moment magnitude of 7.0 earthquake along the Onshore segment of the Oak Ridge Fault.

U.B.C. Faults: Based on the computer program UBCSEIS, the closest Type A fault is the San Andreas Fault located approximately 32.3 miles (52.0 km) from the site. The program also indicates the closest Type B fault is the onshore segment of the Oak Ridge Fault located approximately 1.7 miles (2.7 km) from the site.

4.2 SLOPE STABILITY

Geologic cross sections depicting various slope conditions within the site were analyzed with respect to slope stability. Our analyses of gross slope stability included evaluation of representative temporary slopes, the highest fill slope, and representative natural slopes. A specific cross section was not prepared for the highest fill slope. Instead, a generic slope configuration representing the most critical condition anticipated was used in the analyses. Some conditions exist where natural slopes are locally over-steepened and assumed to have inadequate factors of safety and were not analyzed. Analyses for this issue are discussed in Section 4.4. The project will also entail local backcut conditions that will not provide adequate temporary stability and were not specifically analyzed. We have assumed these areas will be shored to maintain the required factors of safety.

The analyses were performed using the computer program Slope/W. Details of the program are provided in Appendix D. Selection of shear strength parameters used in the analyses was based on the results of direct shear tests of representative materials performed by this firm, by direct shear tests reported by previous consultants for the site and previous experience with similar materials. A summary of the values utilized is provided in Table D-1 in Appendix D. A summary of the calculated factors of safety is provided in Table D-2 in Appendix D, and plots of the analyses are presented on Plates D-1 through D-13 in Appendix D. All slopes were analyzed for seismic stability using a pseudo-static factor of 0.15. No increases in shear strength were used for seismic analyses. All slopes analyzed yielded static, temporary, and pseudo-static factors of safety greater than 1.5, 1.25, and 1.1, respectively.

4.3 SETTLEMENT

Engineering analyses were performed to estimate the settlement potential of shallow spread footings anticipated to support proposed residential structures and proposed fills. Our analyses were based on results of consolidation testing of terrace deposits anticipated to underlay the footings and fill. From these tests, we obtained values for primary compression index, recompression index, and preconsolidation pressure. Testing indicates the primary compression index generally ranges from 0.037 to 0.080 with an average of 0.057 and the recompression index generally ranges from 0.006 to 0.013 with an average of 0.008. Tests indicate the terrace deposits are overconsolidated although the degree of overconsolidation varied depending on the quality of the sample. The average extent of overconsolidation from samples of reasonable quality suggest a value of about 1100 psf over the in-situ stress. The higher-quality samples suggest values of 1800 to 2300 psf over the in-situ stress. Samples taken from the upper 7 feet indicate even greater overconsolidation apparently due to cyclic drying and wetting but these higher degrees of overconsolidation were ignored. From the test data, we used the average primary compression index of 0.057 and a recompression index of 0.008. The preconsolidation pressure was assumed to be at least 1875 psf which represents an overburden of about 15 feet. This thickness is consistent with a conservative geologic model of erosion from the mesa.

For analysis of a typical footing, the stress distribution was based on a Boussinesq distribution below the center of the footing. To account for the three-dimensional effects of the stress distribution, calculated settlements were multiplied by a rigidity factor of 0.7. Using a footing 1.25 feet wide, embedded 1.5 feet, and carrying a bearing pressure of 2000 psf, we estimate a total settlement of approximately 0.12 inches. A summary of this calculation is provided on Plate E-1 in Appendix E. Contribution from soils located below a depth where the incremental stress increase is less than 10% of the applied stress was ignored.

Settlement of the terrace deposit due to the weight of proposed fills was analyzed where the proposed fill is the thickest. This condition occurs at Lot 66 where the proposed fill will be approximately 24 feet at the top of the slope. The thickness of the fill decreases away from the slope and down the slope face and forms the general shape of a triangular load having a base width of about 225 feet. For the analysis, a peak thickness of 20 feet for this triangular load was selected in consideration of the three-dimensional effects of the fill along the slope. The stress distribution was based on a Boussinesq distribution below the peak of the triangular load created by the fill. Using the compression indices and preconsolidation pressure discussed above, we obtain a total settlement of 2.3 inches. A summary of this calculation is provided on Plate E-2 in Appendix E. Another similar analysis was performed at a location 30 feet from the peak of the load to evaluate differential settlement. At this location, we obtain a total settlement of 1.9 inches. A summary of this calculation is provided on Plate E-3 in Appendix E.

4.4 SHEAR PIN DESIGN

Engineering analyses were performed to develop design parameters for shear pins (soldier pile) that can be used to provide supplemental support to the over-steepened natural slope supporting Lots 23 through 25 and 28 through 30. The anticipated locations of the shear pins are indicated on the Geologic Maps, Plates 4, 5, 8 and 9. Although conditions along the bluff vary considerably, we have based our design on the worst case condition which entails a slope of about ½ to 1 (H:V) for a height of approximately 25 feet. Below this, we have assumed the slope is approximately 2 to 1 (H:V). A general cross section detail of this condition is provided on Plate E-5 in Appendix E. The load that will be applied to the piles was determined from the greater force resulting from limit equilibrium stability analyses or the active pressure exerted by the retained soil. The piles were modeled as having a column section above the potential failure surface and an embedded section below the potential failure surface. The lateral load required to achieve a static and seismic factor of safety of at least 1.5 and 1.1, respectively, were calculated from the stability analysis. The analysis of stability is provided on Plates E-7 and E-8 in Appendix E. The resulting force is assumed to act on the column section at a height equal to 1/3 of the distance above the failure plane. The active pressure acting on the portion above the potential failure plane was also calculated and found to slightly exceed the force derived by the stability analysis. The analysis of the active pressure force is provided on Plate E-6 in Appendix E. Based on the active pressure loading configuration, the allowable lateral capacity of a 30-inch-diameter concrete pile with a total length of 34 feet was developed using the elastic method proposed Poulos (1976) for piles adjacent slopes. A summary of this analysis is provided on Plate E-9 in Appendix E. The results indicate this pile configuration can provide a lateral capacity of 40 kips with an estimated lateral deflection of approximately 0.8 inches.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible provided the recommendations presented in this report are incorporated into the design and construction of the project. It is also our opinion that the proposed development will not adversely impact the stability of adjoining properties if the recommendations presented in this report are incorporated in the site development.

5.2 GEOLOGIC HAZARDS

5.2.1 Ground Rupture

No known active faults are known to project through the site nor does the site lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The potential for ground rupture due to an earthquake beneath the site is considered very small.

5.2.2 Ground Shaking

The site is in a seismically active area that has historically been affected by generally moderate to occasionally high levels of ground motion. The site lies in relative close proximity to several active faults; therefore, during the life of the proposed development, the property will probably experience similar moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the southern California region. Potential ground accelerations have been determined for the site and are presented in Section 4.1 of this report. Structural designs should consider the potential for ground accelerations as discussed herein.

5.2.3 Liquefaction

Engineering research of soil liquefaction potential (Youd, et al., 2001) indicates that generally three basic factors must exist concurrently in order for liquefaction to occur. These factors include:

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.
- A relatively loose, non-cohesive, silty and/or sandy soil.
- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

Current groundwater is estimated to be more than 200 feet below the ground surface. Future site development could result in some minor perched water forming on the bedrock contact within the sandy/gravely terrace deposits. However, blow counts of the lower terrace deposits are relatively high and are generally not considered susceptible to liquefaction. Therefore, liquefaction is unlikely at the site.

5.2.4 Landslides

The California Geological Survey has mapped the easterly-descending natural slope within a seismically-induced landslide hazard zone in accordance with the Seismic Hazards Mapping Act. A detailed slope stability analysis including pseudo-static analyses has been conducted for the site and is summarized herein. Provided the recommendations presented in this report are incorporated in the site development, geologic hazards associated with deep seated landslides are not anticipated. However, localized surficial failures have occurred within over-steepened portions of the natural slopes and similar events are anticipated to occur over time within the steep terrain as a result of inclement weather and/or seismic events. Construction of containment barriers or debris walls within significant natural collection areas such as drainage swales or near the base of the natural slope can be implemented to mitigate the potential for adverse impacts associated with the downward migration of earth materials on sloping terrain.

5.2.5 Seiche and Tsunami

The site is elevated more than 1000 feet above sea level and is located a substantial distance from any significant body of water. As such, the potential for any hazards related to seiche and tsunami are considered remote.

5.3 SLOPE STABILITY

Analyses were performed to evaluate various temporary, natural, and permanent slope configurations as previously discussed in Section 4.2. Nearly all these slopes will provide the appropriate calculated factors of safety. The exceptions to this consist of locally over-steepened slopes at the top of the natural bluff and one temporary backcut anticipated for construction of a retaining wall in Lots 70 through 72. With regard to the bluff stability, the current grading plans do not accurately depict the over-steepened conditions that exist near the top of the bluff in some locations. This condition appears to be due to effects in the photogramic from thick vegetation in the areas. Based on our field observations, the worst condition appears to be oversteepened slopes up to 25 feet in height at a maximum gradient of ½ to 1 (H:V). Analysis for this geometry indicate the installation of a shear pin at the top of slope can adequately mitigate the condition. The shear pins can consist of a 30-inch-diameter concrete pile having a total length of 34 feet below current grades and spaced 10 feet center to center. The anticipated locations of these pins are indicated on the Geologic Map, Plates 4, 5, 8 and 9. Where the temporary backcut at Lots 70 though 72 are present, the backcut will generally require the installation of temporary shoring between the walls and the property line. Specific design parameters can be provided to a consultant who specializes in the design of shoring systems.

5.4 SETTLEMENT

Analyses indicate anticipate shallow spread footings are not anticipated to undergo a total settlement greater than ¼ inch when supported by either compacted fill or competent terrace deposits. Differential settlement of the footings is not anticipated to exceed ½ of the total settlement. Terrace deposits are anticipated to undergo some settlement due to the weight of proposed fills. Our analyses suggest a total settlement of up to approximately 2 ½ inches. The analyses also indicate the related differential is not expected to be more than ½ inch over 30 feet. Since the terrace deposits are generally partially saturated, the primary consolidation process is anticipated to occur very rapidly during loading. Assuming the secondary component of settlement would comprise about 1/3 of the total calculated settlement over a period of 50 years, post-construction total and differential settlement due to consolidation of the terrace deposits should not exceed approximately 1 inch and ¼ inch, respectively. Combining settlement due to the foundation load with settlement of terrace deposits due to fill loads, we obtain an overall maximum total and differential settlement of 1 ¼ inches and 3/8 inches over 30 feet, respectively. These values are considered within the tolerable limits of proposed site development.

5.5 EXCAVATION AND MATERIAL CHARACTERISTICS

Based on the results of our subsurface exploration, the onsite near surface soil materials should be readily excavated with conventional earth moving equipment. Typical terrace deposits have moisture contents at or slightly over optimum and should therefore require only minor drying or addition of water during grading. Portions of the terrace deposits located near the contact with bedrock (typically 20 feet or more below current ground surface) will contain significant amounts of cobbles.

5.6 SHRINKAGE, BULKING AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate existing non-engineered artificial fill and terrace deposit materials will shrink approximately 1% to 6%. Significant excavations within bedrock materials are not anticipated within the site. Subsidence from scarification and recompaction of exposed surfaces is expected to be negligible.

The above estimates of shrinkage and subsidence are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during the grading.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with all applicable requirements of the grading codes of the City of Santa Paula, California and CALOSHA, in addition to recommendations presented herein.

6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of rough grading, we recommend a meeting be held between the owner, City Inspector, grading contractor, civil engineer, and geotechnical consultant to discuss proposed work and logistics.

We also recommend that a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site grading. This is to observe compliance with the design specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. If conditions are encountered during construction that appears to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

6.1.3 Site Clearing

All vegetation and other deleterious materials should be removed from the site. The project geotechnical consultant should be notified at the appropriate times to provide observation services during clearing operations to verify compliance with the above recommendations. Voids created by clearing should be left open for observation by the geotechnical consultant. Should any unusual soil conditions or subsurface structures be encountered during site clearing and/or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

6.1.4 Ground Preparation

All existing artificial fill (Qaf) and the upper 5 to 7 feet of the terrace deposits (Qt) are considered unsuitable for support of proposed fills and site development. These materials should be removed to expose competent terrace deposits. Estimated depths of unsuitable earth materials, based on subsurface exploration conducted by Geolabs, Inc., and during this investigation, are indicated on the Geologic Maps, Plates 1 through 8. The actual depths of removal should be evaluated in the field by a representative of this office based on actual conditions exposed during grading.

Exploratory trenches excavated by Geolabs were backfilled without compaction. As such, backfill in these trenches should be removed and replaced with compacted fill.

Where removals are limited by existing structures or property lines, special grading techniques, such as slot cutting, or other acceptable design criteria may be required. Under such conditions, specific recommendations should be provided by this office.

6.1.5 Lot Capping

Transition Lots: Proposed rough grading will create cut/fill transitions or shallow fill conditions within building pads. The cut and/or shallow fill portions of these lots should be overexcavated at least 3 feet below finish pad grades and replaced with a compacted fill blanket. The overexcavation should extend across the entire lot. A transition lot capping detail is provided on Plate E-1, Appendix E.

Cut Lots: Due to the potential variability of materials beneath the site, some cut lots may expose materials with differing expansion characteristics. Cut lots that will support structures should be brought to near rough grade then observed by the geotechnical consultant. If significant differential materials are exposed, the lot should be overexcavated and replaced with fill. Generally, the overexcavation will be 3 feet below finish pad grades. The overexcavation should extend across the entire lot. A cut lot capping detail is provided on Plate E-2, Appendix E. The geotechnical consultant should provide final recommendations for cut lot overexcavation during grading.

6.1.6 Fill Placement

Onsite earth materials may be re-used as compacted fill provided they are clear of debris, vegetation, and particles exceeding 6 inches in diameter. Asphalt debris generated during site demolition can likely be reduced to no more than 6 inches in maximum dimension and incorporated within fill soils during earthwork operations. Following removals, the exposed grade should first be scarified to a depth of 6 inches to 8 inches, watered or air-dried as necessary to achieve a uniform moisture content slightly over optimum, and then recompacted to at least 90% of the laboratory standard. All fill should be placed in lifts no greater than 6 to 8 inches in loose thickness, watered as necessary to achieve a uniform moisture content to 110 and 125 percent of optimum moisture content, then compacted in place to at least 90 percent of the laboratory standard. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultant has approved the preceding lift.

If imported soils are required to bring the site to proposed grades, imported soils should have a maximum particle size of 4 inches and have an expansion index (EI) less then 50. Potential import soils should be sampled by the geotechnical consultant at the source, if possible, tested for expansion and maximum density, and approved by the geotechnical consultant prior to being used.

The laboratory standard for maximum dry density and optimum moisture content for each soil type used should be determined in accordance with ASTM D 1557.

Fills should be maintained relatively level and should not slope more than 20 to 1 (H:V). Where fills will be placed on ground that slopes at 5 to 1 (H:V) or greater, the ground surface should be excavated to create a series of level benches prior to placement of fill.

6.1.7 Temporary Excavations

Temporary construction slopes in site materials may be cut vertically up to a height of 4 feet. Temporary slopes over 4 feet in site materials should be layed back at a maximum gradient of 1 to 1 or properly shored. Limited portions of site materials may be relatively dry and cohesionless and as such, may be prone to sloughing and possible caving. Excavations should not be left open for prolonged periods of time. Where practical, hydraulic shoring with appropriate lagging may be utilized for vertical utility trench excavations up to 10 feet in depth. The project geotechnical consultant should observe all temporary cuts to confirm anticipated conditions and to provide alternate recommendations if conditions dictate. All trench excavations should conform to the requirements of CALOSHA.

Temporary excavations for the retaining wall construction on Lots 70 through 72 will require temporary shoring to maintain proper stability. A specialist in shoring should be retained to prepare appropriate shoring plans. This office should coordinate with the selected specialist to provide appropriate shoring design parameters based on the anticipated shoring system and configuration.

6.1.8 Fill Slopes

Fill slopes (fill over natural slopes, fill over cut slopes) should be constructed with a keyway having a minimum width of 15 feet and a minimum embedment of 2 feet into competent terrace deposits. A minimum fill thickness of approximately 10 to 15 feet should be maintained throughout fill slope construction to mitigate against sliver fills and cut/fill transitions within finished slopes. Details for fill slope construction are presented on Plate E-4.

Where practical, fill slopes should be constructed by over filling and trimming to a compacted core. The face of slopes that are not over-built should be backrolled with a sheepsfoot roller at least every 4 vertical feet of slope construction. The process should provide compacted fill to within 12 inches of the slope face. Finished slopes should be track-walked with a small dozer or rolled with a vibratory compactor and grid roller in order to compact the slope face. The slope face materials will tend to dry out prior to final face compaction. As such, the addition of water to the slope face will likely be required prior to compaction to achieve the required degree of compaction at the time of slope face compaction.

6.1.9 Cut Slopes

All cut slopes should be inspected by an engineering geologist at intervals not exceeding 10 vertical feet during rough grading to evaluate the competency of the slope.

6.1.10 Sliver Cut/Sliver Fill Slopes

For sliver cut or thin fill conditions it is recommended that a backcut and keyway be established such that a minimum fill thickness of 10 feet is maintained for all sliver fill conditions. Where the design cut is insufficient to remove all unsuitable materials, overexcavation and replacement with a stabilization fill will be required.

6.1.11 Stabilization Fills

Cut slopes exposing unsuitable surficial soils may require replacement with stabilization fill slopes. General details for stabilization fill slope construction are presented in Appendix E, Plate E-5. However, specific recommendations should be provided by the geotechnical consultant during grading depending on the actual conditions exposed.

6.1.12 Slope Backdrains

Slope backdrains are generally recommended in fill key excavations and stabilization fill slopes. The locations and necessity of slope backdrains will be determined by the project geotechnical consultant in the field during rough grading. General details for slope backdrains are presented in Appendix E, Plate E-6.

6.2 SHEAR PINS FOR NATURAL SLOPES

As discussed in Section 5.3, portions of the natural bluff slope are unstable. Although differing systems may be employed, we have assumed the condition will be mitigated by the installation of shear pins. Unless another system is employed, the shear pins should consist of piles having a diameter of 30 inches and a length of 34 feet as measured from the existing grade. The piles should be spaced no more than 10 feet center to center at the locations indicated on the Geologic Maps, Plates 4, 5, 8, and 9. A specific structural plan should be prepared by a structural engineer familiar with such systems. Detailed loading diagrams for shear and moment should be provided by this office for the structural engineer.

6.3 CONTAINMENT BARRIERS OR DEBRIS WALLS

Construction of containment barriers and/or debris walls should be implemented to mitigate the potential for adverse impacts associated with the downward migration of earth materials (debris flows) within the easterly natural slope. The topographic expression shown on the base map for the referenced rough grading plans does not provide adequate resolution within the easterly natural slope areas due to thick brush cover. As such, we are unable to determine appropriate locations and design parameters for the recommended containment barriers and/or debris walls. Additional survey control and photogramic resolution is currently being acquired for the site to provide more accurate topographic information. A separate report summarizing our review of the site topography, geotechnical conditions associated with debris flow potential and recommended mitigation measures will be submitted under separate cover.

6.4 SEGMENTAL RETAINING WALLS

The rough grading plans indicate that mechanically reinforced segmental retaining walls up to 21 feet high will be constructed within the site during rough grading operations. At this time, the wall profiles are not available for our review. As such, design criteria for these walls cannot be prepared at this time. The geotechnical consultant should evaluate future segmental wall profiles and determine the geogrid type, spacing and embedment lengths, as well as backcut criteria and drainage requirements prior to wall construction. Segmental wall designs should include analyses of internal, external, and global wall stability with consideration of proposed site improvements. Segmental wall designs should follow the guidelines outlined in the most current edition of the National Concrete Masonry Association Manual.

Backfill materials considered suitable for use behind the proposed segmental retaining walls is limited within the site. Therefore, it may be necessary to mine these materials from the lower portions of the terrace deposits. Based on our subsurface exploration, these materials will be most readily available within the eastern portion of the site. Temporary excavations created during any mining operations within the site should be evaluated by this firm. Additional overexcavations of building pads may be required to mitigate the potential for differential settlement.

6.5 EROSION PROTECTION FOR SLOPES

Surface drainage should be directed in a non-erosive manner away from the tops of all slopes and retaining walls (masonry or segmental). Until permanent vegetation is established, slopes should be provided with short-term erosion protection such as jute matting, polymer applicants or other approved erosion control devices. The project landscape architect should provide specific recommendations for erosion protection.

6.6 SEISMIC DESIGN PARAMETERS

For design of the project in accordance with Chapter 16 of the 2001 C.B.C., the following table presents the seismic design factors:

TABLE 6.1
C.B.C. Seismic Design Parameters

Parameter	Value
Seismic Zone Factor, Z	0.4
Soil Profile Type, S	S_D
Near Source Factor , Na	1.2
Near Source Factor , Nv	1.5
Seismic Coefficient, Ca	0.54
Seismic Coefficient, Cv	0.96

6.7 POST GRADING CONSIDERATIONS

6.7.1 Site Drainage

Drainage should be designed to carry surface water away from all structures in accordance with local code requirements. No rain or excess water should be allowed to pond against building walls or foundations. We recommend that landscape areas drain to proper facilities at a minimum gradient of 2%. Sufficient area drains should be provided to ensure water does not pond.

6.7.2 Utility Trenches

All utility trench backfill should be compacted to at least 90 percent of the laboratory standard. Trench backfill should be brought to a uniform moisture slightly over optimum, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. For deep trenches with sloped walls, backfill material should be placed in lifts no greater than 8 inches in thickness, and then compacted by rolling with a sheepsfoot tamper or similar equipment. The project geotechnical consultant should perform density testing, along with probing, to verify adequate compaction.

Within shallow trenches (less than 18 inches deep) where pipes may be damaged by heavy compaction equipment, such as under building floor slabs, imported clean sand having a Sand Equivalent (SE) of 30 or greater may be utilized. The sand should be placed in the trench, thoroughly moistened, and then compacted with a vibratory compactor.

Where utility trenches are proposed parallel to any building footing (interior and/or exterior trenches), the bottom of the trench should not be located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base. For utility trenches located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base or crossing footing trenches, concrete or slurry should be used as trench backfill.

Site conditions are generally not considered suitable for flooding or jetting of backfill materials, unless the bottoms of the trenches extend into the granular soils. The project geotechnical consultant should verify suitable conditions in each case if jetting and flooding is to be considered.

6.8 PRELIMINARY FOUNDATION DESIGN

6.8.1 Soil Expansion

The recommendations presented herein for post-tension foundation systems are based on soils with a Medium expansion potential. Following site grading, additional testing of site soils should be performed by the project geotechnical consultant to confirm the existing expansion potential for the site. If site soils with significantly different expansion potentials are encountered, the recommendations contained herein may require modification.

6.8.2 Settlement

Foundations should be designed for total and differential settlement up to 1 1/4 inch and ½-inch over 30 feet, respectively.

6.8.3 Allowable Bearing Value

A bearing value of 1,500 pounds per square foot may be used for continuous beams founded at a minimum depth of 12 inches below the lowest adjacent grade. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.8.4 Lateral Resistance

A passive earth pressure of 250 pounds per square foot per foot of depth up to a maximum value of 1000 pounds per square foot may be used to determine lateral bearing for beams. A coefficient of friction of 0.30 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for wind and seismic forces.

The above values are based on beams placed directly against competent native soils or compacted fill. In the case where beam sides are formed, all backfill against the beams should be compacted to at least 90 percent of the laboratory standard.

6.8.5 Foundation Setbacks

The bottom outer edge of foundations located adjacent a top of slope should be setback from the slope face a horizontal distance of at least 1/3 the height of the slope. The horizontal distance should not be less than 7 feet but need not exceed 40 feet.

6.8.6 Beam Dimensions

Perimeter edge beams for both one-story and two-story structures should be founded at a minimum depth of 15 inches below the lowest adjacent final ground surface. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs.

6.8.7 Slab on Grade

The thickness of the floor slabs should be determined by the project structural engineer with consideration of the requirements of UBC 1816; however, we recommend a minimum slab thickness of 4.5 inches.

All dwelling area floor slabs constructed on-grade should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 10-mil Visqueen or equivalent. A minimum of two (2) inches of clean sand having an SE of at least 30 should be placed over the membrane to promote uniform curing of the concrete. This vapor barrier system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Pre-saturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage floor slab areas should be thoroughly moistened to achieve a moisture content that is at least 110 percent over the optimum moisture content. This moisture content should penetrate to a minimum depth of 12 inches below the bottoms of the slabs.

Design in accordance with 1997 UBC Section 1816, may be based on the following parameters:

TABLE 6.2
Post-Tension Design Parameters

Parameter	Value
% Clay (portion passing No. 200 sieve)	50
Plastic Index	25
Plastic Limit	20
Clay Type	Montmorillonite
Depth to Constant Soil Suction (feet)	5
Constant Soil Suction (pF)	3.6
Velocity of Moisture Flow (in./mo.)	0.5
Subgrade Modulus (pci)	150

Values for e_m may be estimated from Figure 18-III-14 of the UBC based on the selected Thornthwaite moisture index. Although the UBC indicates a Thornthwaite index of –20, consideration should be given to non-climatic factors such as irrigation practices that could affect the assumed value. Values for y_m may utilize Table 18-III based on the parameters provided in the table above and the estimated e_m. Using a Thornthwaite index of –20, the e_m and y_m values are summarized below:

Edge Lift Moisture Variation Distance, e_m	2.6 feet
Edge Lift, y_m	0.316 inches
Center Lift Moisture Variation Distance, e_m	5.3 feet
Center Lift, y_m	1.360 inches

6.9 RETAINING WALLS

6.9.1 Allowable Bearing Value

For footings located at least 7 feet horizontally from the face of a slope as measured from the bottom of footing, a bearing value of 2000 pounds per square foot (psf) may be used for continuous and isolated footings founded at a minimum depth of 12 inches below the lowest adjacent grade. Continuous and isolated footings should also have a minimum width of 12 and 24 inches, respectively. The above bearing value may not be increased for additional width or depth. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.9.2 Lateral Resistance

For footings located at least 7 feet horizontally from the top of a slope as measured from the top of slope to the footing face, a passive earth pressure of 250 pounds per square foot per foot of depth up to a maximum value of 1500 pounds per square foot may be used to determine lateral bearing for footings. A coefficient of friction of 0.33 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for wind and seismic forces.

Where the footing is within 7 feet horizontally of a descending slope as measured from the top of slope to the footing face, a passive earth pressure of 80 pounds per square foot per foot of depth up to a maximum value of 1500 pounds per square foot may be used to determine lateral bearing for footings. A coefficient of friction of 0.33 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for wind and seismic forces.

The above values are based on footings placed directly against competent native soils or compacted fill. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of the laboratory standard.

6.9.3 Footing Reinforcement

All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations herein. Where recommended removals are limited due to space restrictions, greater reinforcement may be recommended. Specific recommendations should be provided by the geotechnical consultant during grading based on as-built conditions exposed in the field.

6.9.4 Footing Observations

All footing trenches should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. All loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.9.5 Earth Pressures

Conventional retaining walls should be designed for the pressures as indicated in the table below. The values are based on typical onsite materials as well as on drained backfill conditions and do not consider hydrostatic pressures. Relatively clayey materials should not be used for wall backfill. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the earth pressures provided in the table below.

TABLE 6.3
Earth Pressures

Backfill Condition	Active Pressure Wall Height up to 6 feet (pcf)	Active Pressure Wall height over 6 feet and under 15 feet (pcf)	Restrained Walls all Heights (pcf)
Level	35	45	70
2 to 1 slope	63	81	105

6.9.6 Drainage and Moisture-Proofing

All retaining walls should be constructed with a perforated pipe and gravel subdrain to prevent entrapment of water in the backfill. The perforated pipe should consist of 4-inch diameter, ABS SDR-35 or P.V.C. Schedule 40 with the perforations laid down. The pipe should be embedded in ¾- to 1½-inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend at least one foot up the wall above the footing. Where walls will be located at the grade splits inside the homes, the gravel should extend up to within 12 inches of the finish grade. Filter fabric should consist of Mirafi 140N, or equal. Non-perforated drain outlets should be provided at a minimum of every 100 linear feet. Outlet pipes should be directed to positive drainage devices, such as graded swales, and/or area drains.

The use of weepholes may be considered in locations where aesthetic issues from potential nuisance water are not a concern. Weepholes should be 2 inches in diameter and provided at least every 6 feet on center. Where weepholes are used, perforated pipe may be omitted from the gravel subdrain.

Retaining walls supporting backfill should also be coated with a waterproofing compound or covered with such material to inhibit infiltration of moisture through the walls. Waterproofing material should cover any portion of the back of wall that will be in contact with soil and should lap over and cover the top of footing. The top of footing should be finished smooth with a trowel to inhibit the infiltration of water through the wall. The project structural engineer should provide specific recommendations for waterproofing, water stops, and joint details.

6.9.7 Retaining Wall Backfill

Most onsite soils may be used for backfill of retaining walls. However, clayey materials with an EI over 50 should not be used for wall backfill. The project geotechnical consultant should approve all backfill used for retaining walls. All wall backfill should be brought to relatively uniform moisture content slightly over optimum, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. Flooding or jetting of backfill material is not recommended.

6.10 CEMENT TYPE

Laboratory testing of site soils indicates **negligible** soluble sulfate content. We recommend following the procedures provided in 2001 C.B.C. Section 1904.3.1 and Table 19-A-4 for negligible sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

6.11 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 7 feet in each direction. Cold joints should be keyed or provided with dowels spaced 18 inches on center. Special jointing detail should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to a moisture content of at least 120 percent of optimum to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete.

The geotechnical consultant should observe and verify the density and moisture content of subgrade soils prior to pouring concrete to ensure that the required compaction and pre-moistening recommendations have been met.

Drainage from flatwork areas should be directed to local area drains and/or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. The concrete flatwork should also be sloped at a minimum gradient of 2% away from building foundations and masonry walls.

6.12 PRELIMINARY PAVEMENT RECOMMENDATIONS

6.12.1 Subgrade Preparation

Prior to placement of pavement elements, the upper 12 inches of subgrade soils should be moisture-conditioned to at least 110 percent of the optimum moisture content and compacted to at least 90 percent of the laboratory standard. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding compacted soil or aggregate base materials.

6.12.2 Preliminary Pavement Design

Based on the soil conditions present at the site and estimated traffic indexes, preliminary pavement sections are recommended in the table below. For preliminary design purposes, an "R"-value of 5 was used to determine the pavement design criteria presented below. The sections presented below are for planning purposes only and should be re-evaluated subsequent to site grading. Final pavement sections should be based on actual R-value testing of in-place soils and analysis of anticipated traffic.

TABLE 6.3
Flexible Pavement Sections

Location	Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
10th Street	6.0	4.0	13.0
All other interior streets	5.5	3.5 4	12.0 10.0

6.12.3 Pavement Materials

Aggregate base materials should be either Crushed Aggregate Base, Crushed Miscellaneous Base, or Processed Miscellaneous Base conforming to Section 200-2 of the Standard Specification for Public Works Construction (Greenbook). The materials should be brought to a uniform moisture content near optimum then compacted to at least 95 percent of ASTM D1557-91. Asphalt concrete materials and construction should conform to Section 203 of the Greenbook.

7.0 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus-Keefe & Associates, Inc.*, be engaged to review the final grading and foundation plans prior to construction. This is to verify that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. If we are not provided the opportunity to review these documents, we take no responsibility for misinterpretation of our recommendations.

We recommend that a geotechnical consultant be retained to provide soil engineering services during construction of the project. These services are to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly, the project geotechnical consultant should review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appears to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

8.0 INVESTIGATION LIMITATIONS

This report is based on the proposed development and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory testing for this investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty.

This report has been prepared for the exclusive use of **Comstock Homes** to assist the project consultants in the design of the proposed development. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

This report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

This report is subject to review by the controlling governmental agency.

Sincerely yours

ALBUS-KEEFE & ASSOCIATES, INC.

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Geolabs – Westlake Village, 2005, "Grading Plan Review, 16.17 Acre Parcel, Ridgeview at Vista Glen, Santa Paul Memorial Hospital Property, City of Santa Paula, California, date August 10, 2005.

Plans

Rough Grading Plan by Development Resource Consultants, Inc., Sheets 5 through 11 of 16, dated April 10, 2006, Scale: 1"=20'.

Aerial Photographs

Agency	Date Flown	Flight No.	Photo No.
Continental	4-3-79	2-VEN-14	3, 4
Continental	2-80	VEB-3	19, 20
Continental	1-8-88	VEN-4	10, 11
Inland Aerial Surveys	4-25-05	05-5939	2-1, 2-2

APPENDIX A

EXPLORATORY BORING LOGS

EXPLORATION LOG

Project:	Proposed Residential Development		Boring No.: LEGEND
Location:	10th Street, Santa Paula		Elevation:
J.N.:	1489.00	Client: Comstock	Date: 3/1/2006
Drill Method:	(drill rig type)	Driving Weight: (hammer wt. and drop)	Logged by: DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		EXPLANATION							
		Heavy solid lines separate geologic units.							
		Thin solid Lines separate material types within geologic unit.							
		Dashed lines indicate unknown depth of material type change.							
		Heavy double line indicates bottom of boring.							
5									
		Solid black rectangle in Core column represents California Split-Spoon sampler (2.5in. ID, 3in. OD).							
		Gray shaded rectangle in Core column represents SPT sampler.							
10		**Cross-out rectangle** in Core column represents sample not recovered.							
		Light gray Rectangle in Bulk column represents large bag sample.							
15		**Other Laboratory Tests:** MAX = Maximum Dry Density/Optimum Moisture Content SO4 = Soluble Sulfate Content DSR = Direct Shear, Remolded DS = Direct Shear, Undisturbed SA = Sieve Analysis (1" through #200 sieve) PSA = Particle Size Analysis (SA with Hydrometer) -200 = Percent Passing #200 Sieve HYD = Hydrometer Only CON = Consolidation\Collapse							

BORING LOG

Project:	Proposed Residential Development		Boring No.:	B-1
Location:	10th Street, Santa Paula		Elevation:	606
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	Bucket Auger	Driving Weight: See Notes	Logged by:	MP

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Silty Clay with Sand (CL): Dark brown; moist; stiff; fine-grained sand; some pinhole pores.							
5									
				2	■	▒	16.6	110.9	MAX EXP SO4 DSR AT
10				P/6" 2/6"	■		16.6	109.6	
		@ 12 feet: Becomes rocky, cobbles to 8-inches in diameter.							
15		Silty Sand with Gravel and Cobbles (GM-SM): Reddish- to yellowish-brown; damp to moist; dense; fine- to coarse-grained sand; fine to coarse gravel; trace clay; trace boulders to 30-inches in diameter.				▒			MAX DSR SA
20									

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BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-1**
Location:	**10th Street, Santa Paula**	**Elevation:**	**606**
J.N.:	**1489.00** — **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**Bucket Auger** — **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 23 feet: Few boulders to 18-inches.							
25	TQsa	**BEDROCK - Saugus Formation (TQsa)** Clayey Siltstone: Olive brown to pale gray; damp to moist; soft to moderately hard; moderately weathered trace fine-grained sand. Contact: N75E, 24S - contact is irregular. @ 26.5 feet: Bedding - N48E, 55S; 1/4-inch thick sand layer; layer is truncated by terrace deposits above.		8			16.3	114.6	MAX EXP SO4 DSR AT
30		@ 30 feet: Bedding - N56E, 50S; 1.5-inch thick reddish-brown layer; discontinuous.		13			16.0	114.0	
35		Sandy Siltstone: Pale gray; damp; soft to moderately hard; fine grained sand; contact is gradational, no distinct bedding. @ 35 feet: Becomes moderately hard; massive.							
		Siltstone: Pale gray; damp to moist; moderately hard; indistinctly bedded to massive; contacts are gradational.							
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-1**
Location:	**10th Street, Santa Paula**	**Elevation:**	**606**
J.N.:	**1489.00** — **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**Bucket Auger** — **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40	**TQsa**			14	■		14.4	116.5	
		Sandy Siltstone: Pale gray; damp to moist; moderately hard. Bedding - N65E, 43S; bedding is indistinct.							
		@ 44 feet: Becomes yellowish-brown.							
45									
		@ 46 feet: 6-inch thick layer of Clayey Siltstone: Olive brown to pale gray; damp to moist; moderately hard; indistinctly bedded, contact is gradational.							
50		Silty Sandstone: Yellowish-brown; damp to moist; moderately hard; fine-grained sand; contact is gradational and indistinct.		35	■		11.5	122.6	
		@ 52 feet: Bedding - N65-70E, 36S; 3-inch thick zone containing 1/2-inch diameter calcium carbonate nodules.							
		@ 53.5 feet: Slight seepage	▽						
55		Clayey Siltstone: Reddish-brown and olive gray; damp to moist; moderately hard; contact is distinct, but irregular and contains rip-up clasts; upper 8 inches is reddish-brown, olive gray below. Bedding/Contact - N60E, 54S							

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-1**
Location:	**10th Street, Santa Paula**	**Elevation:**	**606**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**Bucket Auger** **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
60		Total Depth = 60 feet; downhole logged to 57 feet Slight Seepage at 53.5 feet Moderate Caving of Cobbles and Boulders from 15-25 feet Casing was installed in borehole from surface to 24 feet. Following logging, casing was removed and borehole backfilled with cuttings.							
65		**Notes** Driving Weights 0-25 feet = 2,500 lbs. 26-45 feet = 1,500 lbs. 46-70 feet = 750 lbs.							
70									
75									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-2**
Location:	**10th Street, Santa Paula**	**Elevation:**	**629**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**Bucket Auger** **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Silty Clay (CL): Dark grayish-brown; moist; stiff; porous; trace fine-grained sand.							
5				3	■		14.1	102.0	
10		@ 10 feet: Very porous; slight decrease in density, firm to stiff.		3	■		16.3	107.5	
15		@ 15 feet: Becomes rocky, gravel and cobbles to 8-inches in diameter.							
		Silty Sand with Gravel and Cobbles (SM): Medium brown; moist; dense; fine-grained sand; fine to coarse gravel; cobbles to 10-inches in diameter.							
20									

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BORING LOG

Project:	Proposed Residential Development		Boring No.:	B-2
Location:	10th Street, Santa Paula		Elevation:	629
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	Bucket Auger	Driving Weight: See Notes	Logged by:	MP

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Gravelly Sand with Cobbles and Boulders (SW): Yellowish-brown; moist; dense; fine- to coarse-grained sand; fine to coarse gravel; few cobbles and trace boulders. @ 22 feet: Few boulders to 24 inches in diameter.							
25	TQsa	**BEDROCK - Saugus Formation (TQsa)** Clayey Siltstone: Yellowish-brown and pale gray; moist; soft; moderately weathered; trace clay; contact is irregular and roughly horizontal. @ 26.0 feet: Bedding - N50E, 50S; 1/4-inch thick clay seam; striations pointing down-dip; polished and plastic; some calcium carbonate mineralization. Bedrock 1-inch above and below the clay seam is slightly disturbed. On northeast side of borehole, clay seam pinches out. @ 28 feet: Becomed moderately hard; massive.							
30				11	■		14.4	119.9	
		@ 33 feet: Fracture - N39W, 67N; fracture is tight; calcium carbonate lined; bedrock continues massive.							
35									
		@ 38 feet: Bedding - N65E, 48S; Layer containing pea-sized cemented nodules spaced 3 to 6 inches apart.							
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-2**
Location:	**10th Street, Santa Paula**	**Elevation:**	**629**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**Bucket Auger** **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40	**TQsa**	Sandy Siltstone: Yellowish-brown; moist; moderately hard. contact is indistinct.		12	■		14.9	119.0	
		@ 42.5 feet: Bedding - N70E, 52S; Discontinuous fine- to coarse-grained sand layer; pinches out on south side of borehole, continuous around 60% of borehole.							
45									
		@ 48 feet: Bedding - N62E, 64S; 4- to 6-inch thick fine- to coarse-grained sand layer with fine gravel, continuous around 70% of borehole							
50		Clayey Siltstone: Yellowish-brown; moist; soft to moderately hard. @ 49.5 feet: Bedding - N68E, 48S; series of sand and clay layers; clay layers are polished; paper thin to 1/4-inch thick; plastic.		16	■		18.8	108.4	
55									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-2**
Location:	**10th Street, Santa Paula**	**Elevation:**	**629**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**Bucket Auger** **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
60									
		Total Depth = 58 feet; downhole logged to 52 feet No Groundwater Moderate Caving of Cobbles and Boulders from 16-23 feet Casing was installed in borehole from 4 to 20 feet. Following logging, casing was removed and borehole backfilled with cuttings.							
65		**Notes** Driving Weights 0-25 feet = 2,500 lbs. 26-45 feet = 1,500 lbs. 46-70 feet = 750 lbs.							
70									
75									

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BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-3**
Location:	**10th Street, Santa Paula**	**Elevation:**	**691**
J.N.:	**1489.00** / **Client: Comstock**	**Date:**	**3/10/2006**
Drill Method:	**Bucket Auger** / **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qaf	**ARTIFICIAL FILL (Qaf)** Silty Clay to Clayey Silt (CL-ML): Dark grayish-brown; very moist; soft.							
5	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt with Sand (ML): Dark grayish-brown; very moist; soft; fine-grained sand; few fine to coarse gravel.							
10		@ 10 feet: Few coarse gravel and small cobbles to 6 inches in diameter. @ 12 feet: Becomes reddish-brown; very moist; soft.							
15		@ 15 feet: Increase in cobbles.							
20									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-3**
Location:	**10th Street, Santa Paula**	**Elevation:**	**691**
J.N.:	**1489.00** — **Client: Comstock**	**Date:**	**3/10/2006**
Drill Method:	**Bucket Auger** — **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 21 feet: Becomes moist to very moist; stiff.							
25		@ 24 feet: Becomes Olive brown; moist; some calcium carbonate stringers; roughly horizontal erosional contact within terrace deposits. @ 26.5 feet: 2- to 4-inch thick Silty Sand with Gravel layer.							
	TQsa	**BEDROCK - Saugus Formation (TQsa)** Clayey Siltstone: Olive brown; moist; soft; moderately weathered; some calcium carbonate mineralization. Contact - N10W, 27E @ 29 feet: Fracture - N56W, 80N; fracture is tight.							
30		Sandy Siltstone: Olive gray to yellowish-brown; moist; soft to moderately hard; fine-grained sand; thinly bedded to laminated. Bedding - N75E, 42S. @ 32.5 feet: Fracture - N18W, 63N; fracture is tight and calcium carbonate lined. @ 33.5 feet: Bedding/Lamination - N66E, 45S							
35									
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**B-3**
Location:	**10th Street, Santa Paula**	**Elevation:**	**691**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/10/2006**
Drill Method:	**Bucket Auger** **Driving Weight: See Notes**	**Logged by:**	**MP**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40									
		Total Depth = 40 feet; downhole logged to 37 feet No Groundwater Following logging, borehole was backfilled with cuttings. **Notes** Driving Weights 0-25 feet = 2,500 lbs. 26-45 feet = 1,500 lbs. 46-70 feet = 750 lbs.							
45									
50									
55									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-1**
Location:	**10th Street, Santa Paula**	**Elevation:**	**598**
J.N.:	**1489.00** — **Client: Comstock**	**Date:**	**3/1/2006**
Drill Method:	**8" Hollow Stem Auger** — **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Silty Clay with Sand (CL): Dark grayish-brown; wet in upper foot, damp below; stiff; some fine-grained sand, some pinhole pores; trace sandstone fragments.		25			14.1	107.3	EXP SO4 AT
5		@ 5 feet: Becomes very stiff		39			15.3	114.7	
		@ 7 feet: Becomes dark brown, pinhole porosity continues.		30			16.3	110.6	CON
10		@ 10 feet: Becomes stiff, pinhole porosity continues.		22			17.1	108.4	CON
15		@ 15 feet: Becomes very stiff, pinhole porosity continues.		28			18.4	106.7	CON
20									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-1**
Location:	**10th Street, Santa Paula**	**Elevation:**	**598**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/1/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Sandy Silt (ML): Medium brown; moist; stiff; fine-graind sand; some pinhole porosity; trace clay.		25			18.1	101.8	CON
25		Sandy Silt with Clay (ML): Dark grayish-brown; moist; stiff; fine-grained sand; trace porosity.		22			19.2	106.8	
30		Sand (SP): Yellowish-brown; very moist; very dense; fine-grained sand; rock fragment in tip. @ 31.5 feet: Becomes cobbley		10/6" 50/5"			15.2	111.6	
35				50/5"					
	TQsa	**BEDROCK - Saugus Formation (TQsa)** Sandstone: Yellowish-brown; damp; moderately hard; fine-grained sand; slightly weathered; moderately cemented.		50/6"			3.7	136.0	
40		Total Depth = 37.5 feet Perched Groundwater at 27 feet							

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BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-2**
Location:	**10th Street, Santa Paula**	**Elevation:**	**610**
J.N.:	**1489.00** — **Client: Comstock**	**Date:**	**3/1/2006**
Drill Method:	**8" Hollow Stem Auger** — **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt to Silty Clay (ML-CL): Dark grayish-brown; moist; very stiff; trace pinhole pores; trace sandstone fragments.							
				21	■		16.9	111.9	
5				28	■		17.7	109.8	CON
10		Sandy Silt with Clay to Clayey Silt with Sand (ML): Medium brown; moist; stiff; fine-grained sand; trace pinhole pores.		22	■		18.0	105.3	CON
15		@ 15 feet: Becomes very stiff, some darker mottleing, trace pinhole pores.		41	■		18.9	108.7	
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-2
Location:	10th Street, Santa Paula		Elevation:	610
J.N.:	1489.00	Client: Comstock	Date:	3/1/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 20 feet: Becomes stiff; some calcium carbonate mineralization; trace pinhole pores.		22	■		19.9	106.3	CON
25		Sandy Silt with Clay (ML): Medium brown; very moist to wet; stiff; no visible pores.		20	■		20.7	104.3	CON
		Sandy Clay (CL): Yellowish-brown; very moist; stiff; fine-grained sand.							
30		Clayey Sand (SC): Yellowish-brown; very moist; dense; fine-grained sand.		10/6" 50/5"	■		9.4	123.3	
		Silty Sand (SM): Reddish-brown; moist; dense to very dense; fine-grained sand; some small cobbles. @ 32.0 feet: Becomes cobbley							
35		Total Depth = 34.0 feet (Refusal) Perched Groundwater at 28 feet							
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-3**
Location:	**10th Street, Santa Paula**	**Elevation:**	**614**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/1/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Sandy Silt (ML): Dark brown; moist; very stiff; fine-grained sand; trace pinhole pores.							EXP SO4 AT
5				33			15.8	113.7	
10				26			18.9	105.5	CON
15		@ 15 feet: No change, some porosity to 1/16" diameter.		28			18.5	108.5	
20									

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BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-3**
Location:	**10th Street, Santa Paula**	**Elevation:**	**614**
J.N.:	**1489.00** — **Client: Comstock**	**Date:**	**3/1/2006**
Drill Method:	**8" Hollow Stem Auger** — **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Sandy Silt to Silty Sand (ML-SM): Yellowish-brown mottled with orange iron oxide staining; moist; stiff/medium dense; fine-graind sand; trace pinhole porosity.		24	■		21.7	102.7	
		@ 22.0 feet: Becomes cobbley							
25	TQsa	**BEDROCK - Saugus Formation (TQsa)** Silty Sand (SM) Yellowish-brown; moist; very dense to hard; fine-grained sand; highly weathered; trace pinhole pores; some charcoal/carbon fragments; trace calcium carbonate stringers; rock in sampler tip at 25 feet.		24/6" 50/5"	X				
				64	■		16.3	112.2	DS
30		Sandy Siltstone: Grayish-brown with reddish-brown iron oxide staining; moderately hard; parts along bedding planes (+/- 45^{O}); slightly weathered; some calcium carbonate stringers.		84	■		17.5	111.7	DS
		Total Depth = 31.5 feet No Groundwater							
35									
40									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-4
Location:	10th Street, Santa Paula		Elevation:	604
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Sandy Silt with Clay (ML): Brown to grayish-brown; moist; stiff becoming very stiff with depth; fine-grained sand; some pinhole pores; trace fine gravel.							
5				28	■		15.3	114.0	CON
10		@ 10 feet: No change.		37	■		17.5	107.8	
15		Silty Sand (SM): Medium brown; very moist; medium dense; fine-grained sand; trace pinhole porosity.		28	■		18.0	108.0	CON
20									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-4**
Location:	**10th Street, Santa Paula**	**Elevation:**	**604**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Sandy Silt with Clay to Silty Sand with Clay (ML-SM): Medium brown; moist to very moist; stiff/medium dense; fine-graind sand; slight increase in pinhole porositytrace calcium carbonate stringers, some small root decay.		25	■		18.6	105.6	CON
25		Clayey Silt (ML): Yellowish-brown; moist to very moist; very stiff; trace fine-grained sand.		39	■		19.8	107.6	
		@ 28 feet: Becomes cobbley	▽						
30	TQsa	**BEDROCK - Saugus Formation (TQsa)** Sandy Siltstone: Gray and yellowish-brown; damp; soft to moderately hard; fine-grained sand; moderately to highly weathered; appears thinly bedded.		51	■		16.8	111.4	
35		@ 35 feet: Becomes moderately hard; decrease in weathering.		27/6" 50/5"	■		13.7	118.9	DS
		Total Depth = 36.0 feet Perched Groundwater at 28 feet							
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-5**
Location:	**10th Street, Santa Paula**	**Elevation:**	**602**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt with Sand (ML): Dark brown; moist; stiff; some porosity; trace fine-grained sand and cobbles.							
5				23	■		14.7	107.9	
10		Sandy Silt (ML): Reddish-brown; moist; very stiff; fine-grained sand; trace porosity;some clay, some calcium carbonate mineralization.		36	■		15.8	111.7	CON
15		@ 15 feet: No change.		31	■		18.9	102.8	CON
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-5
Location:	10th Street, Santa Paula		Elevation:	602
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 20 feet: No change.		32	■		19.8	107.3	
25		Silty Sand (SM): Light brown; moist; dense; trace calcium carbonate mineralization; no visible pores.		36	■		18.5	109.5	
30		Sandy Silt with Clay (ML): Dark brown; moist to very moist; stiff; trace porosity.		30	■		19.9	107.1	
35		Clayey Silt (ML): Yellowish-brown; moist; very stiff; trace fine-grained sand. @ 37.0 feet: Becomes cobbley		33	■		17.9	109.9	
		Total Depth = 39 feet (Refusal) No Groundwater							
40									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-6
Location:	10th Street, Santa Paula		Elevation:	636
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt with Sand (ML): Dark brown; moist; stiff; fine-grained sand; some pinhole pores.							
				62	■		12.0	113.7	
5				21	■		12.9	100.9	
10		@ 10 feet: Becomes very stiff.		27	■		18.6	108.6	
15		@ 15 feet: No change.		34	■		18.1	111.5	
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-6
Location:	10th Street, Santa Paula		Elevation:	636
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Clayey to Sandy Silt (ML): Yellowish-brown; very moist; very stiff; fine-graind sand; trace porosity.	▽	28	■		19.6	108.6	
		@ 23 feet: Becomes cobbley							
25									
	TQsa	**BEDROCK - Saugus Formation (TQsa)** Sandy Siltstone: Gray and light brown; damp; soft to moderately hard; fine-grained sand; moderately weathered.		80	■		13.4	119.1	DS
30		Total Depth = 28.5 feet Perched Groundwater at 20 feet							
35									
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-7**
Location:	**10th Street, Santa Paula**	**Elevation:**	**622**
J.N.:	**1489.00** / **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**8" Hollow Stem Auger** / **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qaf	**ARTIFICIAL FILL (Qaf)** Road fill.							
	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt (ML): Dark brown; moist; stiff; trace fine-grained sand.							
5				36			12.7	122.5	CON
		Sandy Silt to Silty Sand (ML-SM): Dark brown; moist; very stiff/dense; fine-grained sand; some porosity.							
10		@ 10 feet: Becomes porous to very porous; decrease in sand content, Sandy Silt (ML); stiff.		20			17.3	97.5	CON
15		@ 15 feet: Becomes very stiff.		18					
20									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-7**
Location:	**10th Street, Santa Paula**	**Elevation:**	**622**
J.N.:	**1489.00** / **Client: Comstock**	**Date:**	**3/2/2006**
Drill Method:	**8" Hollow Stem Auger** / **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**DL**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Clayey Silt with Sand (ML): Medium brown; moist to very moist; very stiff; fine-grained sand; trace porosity.		33			18.5	109.7	DS
25		Silty Sand to Sand (SM-SP): Yellowish-brown; moist; dense; fine-grained sand.		26					
30		@ 30 feet: No Change.		46			19.4	104.5	DS
		@ 32.0 feet: Becomes cobbley							
35		Total Depth = 34.0 feet (Refusal) No Groundwater							
40									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-8
Location:	10th Street, Santa Paula		Elevation:	643
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt with Sand (ML): Dark brown; moist; stiff; fine-grained sand; some porosity.							
5									
10									
15									
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-8
Location:	10th Street, Santa Paula		Elevation:	643
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20									
	Qt								
25									
30									
35									
		@ 37.0 feet: Thin gravel layer.							
40									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-8
Location:	10th Street, Santa Paula		Elevation:	643
J.N.:	1489.00	Client: Comstock	Date:	3/2/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	DL

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40		Sand (SP): Yellowish-brown; moist; dense; fine-grained sand.		38			8.9	104.1	
45									
50		@ 50 feet: Becomes medium dense.		29					
		@ 53 feet: Becomes cobbley.							
55									
		Total Depth = 56.5 feet (Refusal) No Groundwater							

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-9**
Location:	**10th Street, Santa Paula**	**Elevation:**	**622**
J.N.:	**1489.00** / **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**8" Hollow Stem Auger** / **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**CM**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Silt (ML): Dark brown; damp; hard; slightly porous; roots; rock fragments in sampler.							
				50/5"			11.6	109.1	
5		Sandy Silt (ML): Reddish-brown; dry to damp; very stiff; some rock fragments.		49			10.6	118.4	
10		Silt (ML): Reddish-brown; moist; hard; minor pinhole pores.		60			17.3	111.3	
15		Clayey Silt (ML): Reddish-brown; moist; very stiff; trace fine-grained sand.		48			19.8	106.7	
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-9
Location:	10th Street, Santa Paula		Elevation:	622
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 20 feet: No Change		29					
25		@ 25 feet: Trace pinhole pores.		40			20.6	106.5	
30		Sandy Silt (ML): Yellowish-brown; moist; very stiff; fine-grained sand.		21					
35		@ 35 feet: Increase in sand content, Silty Sand to Sandy Silt (SM-ML); very dense.		34/6" 50/4"			19.4	107.1	
		@ 37.5 feet: Becomes cobbley							
40									

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Geotechnical Consultants

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-9
Location:	10th Street, Santa Paula		Elevation:	622
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40		Silty Sand to Sandy Silt (SM-ML): Yellowish-brown; damp to moist; very dense/hard; fine-grained sand; rock fragments in sampler tip.		45/6" 50/4"					
45									
		Total Depth = 47.0 feet (Refusal) No Groundwater							
50									
55									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-10**
Location:	**10th Street, Santa Paula**	**Elevation:**	**686**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**CM**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Sandy Clay (CL): Dark brown; damp to moist; very stiff; fine-grained sand; no visible pores.							
5				42			17.9	108.4	
		Silty Clay (CL): Dark brown; moist; very stiff; no visible pores; sandstone rock fragments.		29			12.6	114.4	
10		Silty Clay to Clayey Silt (CL-ML): Mottled red and brown; moist; very stiff; trace fine-grained sand.		37			12.1	115.3	
15		@ 15 feet: No change.		22					
20									

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Geotechnical Consultants

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-10
Location:	10th Street, Santa Paula		Elevation:	686
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Silty Sand to Sandy Silt (SM-ML): Yellowish-brown; moist; very dense/hard; fine-graind sand; some coarse gravel.		55			5.8	disturbed	
		@ 22 feet: Becomes cobbley							
25				50/3"					
	TQsa	**BEDROCK - Saugus Formation (TQsa)** Siltstone: Light brown; dry to damp; moderately hard; some calcium carbonate stringers.							
30				35/6" 50/4"			11.1	110.0	
		Total Depth = 31.5 feet No Groundwater							
35									
40									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-11
Location:	10th Street, Santa Paula		Elevation:	691
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt (ML): Dark brown; dry to damp; very stiff; some pinhole pores.							
				43			7.1	116.6	
5		Sandy Silt (ML): Reddish-brown; damp; hard; fine-grained sand; pinhole pores.		61			9.0	125.7	
		@ 8 feet: No Recovery, sandstone rock fragment in tip.		50					
10		Silty Clay (CL): Yellowish-brown; damp; very stiff; fine gravel.		25					
15		Sandy Silt (ML): Yellowish-brown; damp; hard; fine-grained sand; trace clay; some coarse gravel.		64			10.0	125.6	
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-11
Location:	10th Street, Santa Paula		Elevation:	691
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 20 feet: Becomes light brown; very stiff.		28					
25	TQsa	**BEDROCK - Saugus Formation (TQsa)** Clayey Siltstone: Olive brown; damp; soft to moderately hard; some calcium carbonate stringers.		73			17.8	112.7	
30		Sandy Siltstone: Olive brown; damp; moderately hard; fine-grained sand; trace clay.		30/6" 50/5"					
35		Siltstone: Yellowish-brown; damp; moderately hard.		33/6" 50/2"			17.0	108.9	
40									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-11
Location:	10th Street, Santa Paula		Elevation:	691
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40		@ 40 feet: No Change.		50/5"					
		Total Depth = 41.5 feet No Groundwater							
45									
50									
55									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-12**
Location:	**10th Street, Santa Paula**	**Elevation:**	**653**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**CM**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt (ML): Dark brown; moist; very stiff; porous.							
5				39			16.0	114.2	
10		@ 10 feet: Pinhole pores.		36			18.3	109.1	
15		@ 15 feet: No visible pores.		21					
20									

BORING LOG

Project:	Proposed Residential Development		Boring No.:	HS-12
Location:	10th Street, Santa Paula		Elevation:	653
J.N.:	1489.00	Client: Comstock	Date:	3/9/2006
Drill Method:	8" Hollow Stem Auger	Driving Weight: 140 lbs @ 30" Autoham.	Logged by:	CM

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	Clayey Silt (ML): Yellowish-brown; moist; very stiff; trace pinhole porosity; trace fine gravel.		34			19.5	107.3	
25		Sandy Silt to Silty Sand (ML-SM): Yellowish-brown; moist; very stiff/medium dense; fine-grained sand.		21					
30		Clayey Silt (ML): Yellowish-brown; moist; very stiff; coarse gravel in tip.		38			19.6	99.0	
35		@ 35 feet: Decrease in clay content; some gravel.		32					
40									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-12**
Location:	**10th Street, Santa Paula**	**Elevation:**	**653**
J.N.:	**1489.00** / **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**8" Hollow Stem Auger** / **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**CM**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40		Silt (ML): Yellowish-brown; moist; very dstiff; coarse gravel in sampler tip.		29					
		Total Depth = 41.5 feet No Groundwater							
45									
50									
55									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-13**
Location:	**10th Street, Santa Paula**	**Elevation:**	**647**
J.N.:	**1489.00** / **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**8" Hollow Stem Auger** / **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**CM**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0	Qt	**TERRACE DEPOSITS (Qt)** Clayey Silt with Sand (ML): Dark brown; moist; very stiff; fine-grained sand; pinhole pores.							
5				43			14.9	114.5	
10		@ 10 feet: No visible pores.		20					
15		Silty Sand to Sandy Silt (SM-ML): Yellowish-brown; moist; very stiff/dense; fine-grained sand; porous.		43			17.7	111.8	
20									

BORING LOG

Project:	**Proposed Residential Development**	**Boring No.:**	**HS-13**
Location:	**10th Street, Santa Paula**	**Elevation:**	**647**
J.N.:	**1489.00** **Client: Comstock**	**Date:**	**3/9/2006**
Drill Method:	**8" Hollow Stem Auger** **Driving Weight: 140 lbs @ 30" Autoham.**	**Logged by:**	**CM**

Depth (Feet)	Litho-logy	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
20	Qt	@ 20 feet: Becomes very stiff/medium dense		25					
25		@ 25 feet: Trace fine gravel.		44			17.4	110.1	
		@ 30 feet: Refusal on Cobbles.							
30				20/1"					
		Total Depth = 30.0 feet (Refusal) No Groundwater							
35									
40									

APPENDIX B

LABORATORY TEST PROGRAM

LABORATORY TESTING PROGRAM

Soil Classification

Soils encountered within the exploratory borings and trenches were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488-93). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Boring and Trench Logs provided in Appendix A.

In Situ Moisture and Density

Moisture content and dry density of in-place soil materials were determined in representative strata. Test data are summarized in the Boring Logs provided in Appendix A.

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content of onsite soils were determined for a selected sample in general accordance with Method A of ASTM D1557-91. Pertinent test values are given on Table B-1.

Direct Shear

Direct shear tests were performed for undisturbed soil samples and a sample remolded to 90 percent of the maximum dry density. AMEC Earth and Environmental performed this test in general accordance with ASTM D3080-98. Three specimens were prepared for each test. The test specimens were artificially saturated, and then sheared under varied normal loads at a maximum constant rate of 0.0083 inches per minute. Results are graphically presented on Plates B-1 through B-9.

Consolidation

Consolidation tests were performed in general accordance with ASTM D 2435-96. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample. Loads were applied in a geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were inundated at a surcharge loading approximately equal to the existing or proposed total overburden pressures in order to evaluate the effects of a sudden increase in moisture content (hydroconsolidation potential). Results of these tests are graphically presented on Plates B-10 through B-26.

Atterberg Limits

An Atterberg limit test (Liquid Limit, Plastic Limit and Plasticity Index) was performed on a selected sample to verify visual classifications. AMEC Earth and Environmental of Anaheim, California, performed the test in general conformance with ASTM D 4318-95. Test results are presented on Table B-1.

Expansion Potential

An Expansion index test was performed on a selected sample of soil materials in accordance with California Building Code Standard 18-2. Expansion potential classifications were determined from C.B.C. Table 18-I-B on the basis of the expansion index values. Test result and expansion potential are presented on Table B-1.

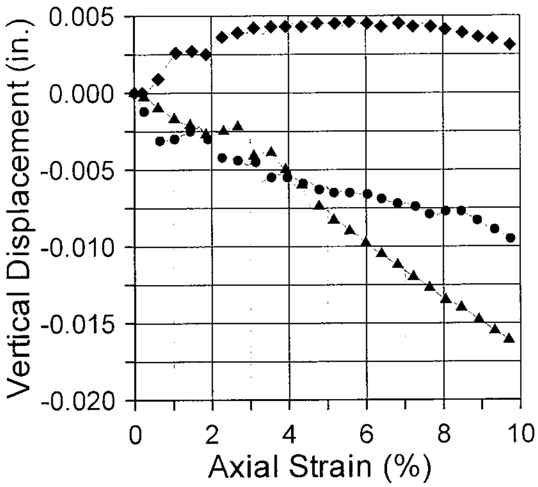
Particle Size Analyses

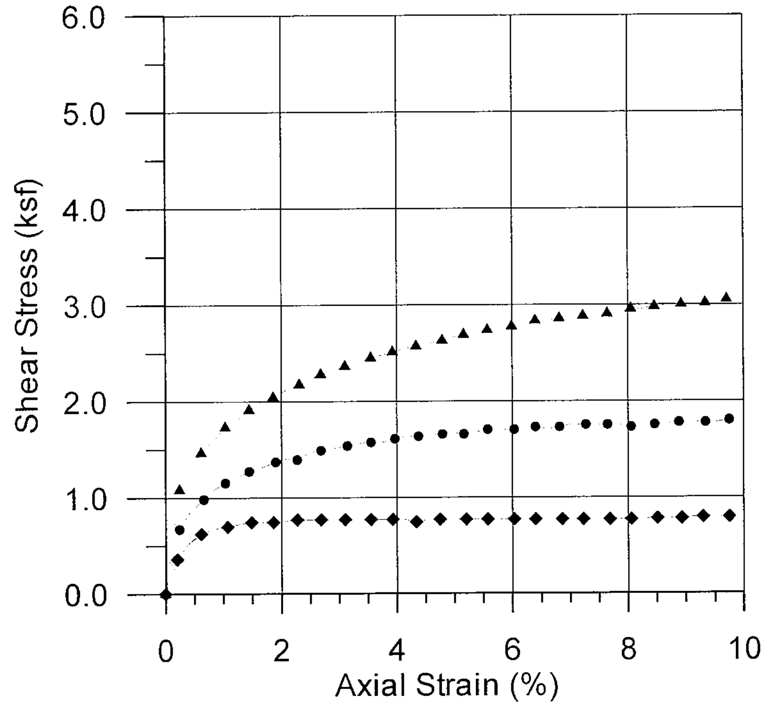
Particle size analyses were performed on representative samples of site materials in accordance with ASTM D 422-63. The results are presented graphically on the attached Plates B-26.

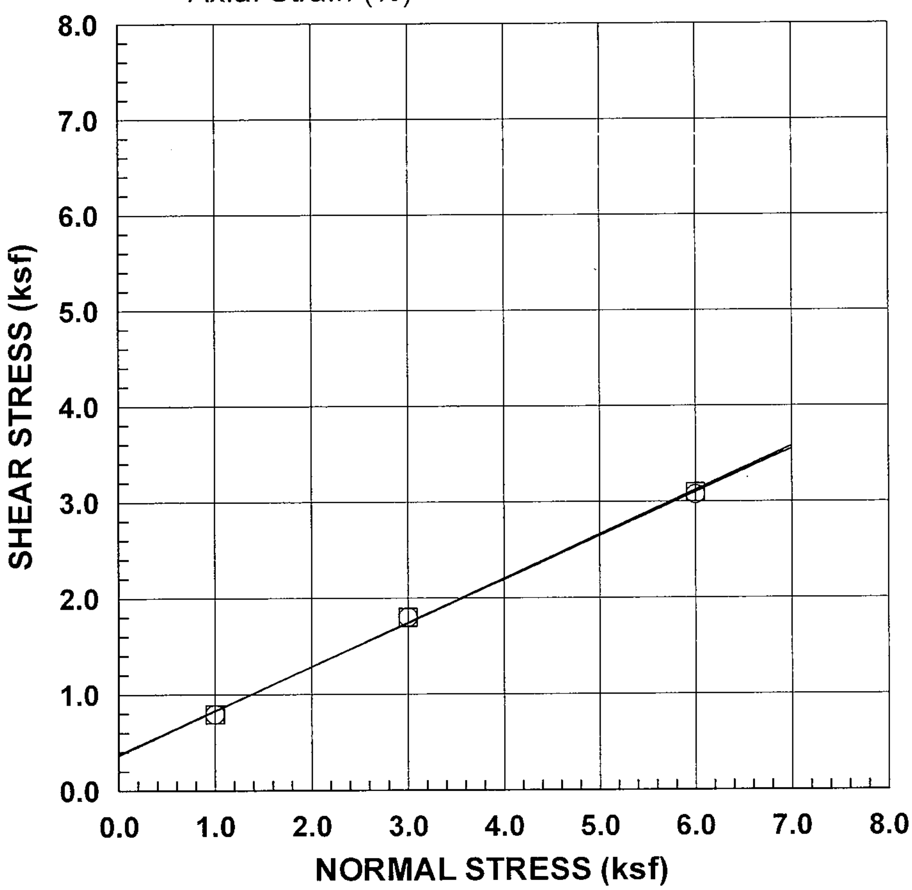
TABLE B-1
SUMMARY OF LABORATORY TEST DATA

Boring Number	Depth (feet)	Soil Type	Test Results	
B-1	6	Silty Clay with Sand (CL)	Maximum Dry Density	= 121.0 pcf
			Optimum Moisture Content	= 12.0%
			Expansion Index	= 40
			Expansion Potential	= Low
			Liquid Limit	= 35.3
			Plastic Limit	= 17.1
			Plasticity Index	= 18.2
			Soluble Sulfate Content	= 0.053%
B-1	16	Silty Sand & Gravel (GM-SM)	Maximum Dry Density	= 129.5 pcf
			Optimum Moisture Content	= 9.5%
B-1	26	Bedrock – Clayey Siltstone	Maximum Dry Density	= 118.5 pcf
			Optimum Moisture Content	= 11.5%
			Expansion Index	= 66
			Expansion Potential	= Medium
			Liquid Limit	= 37.2
			Plastic Limit	= 16.6
			Plasticity Index	= 20.6
HS-1	0-5	Silty Clay with Sand (CL)	Expansion Index	= 38
			Expansion Potential	= Low
			Soluble Sulfate Content	0.029%
			Liquid Limit	= 32.4
			Plastic Limit	= 16.6
			Plasticity Index	= 15.8
HS-3	2-6	Silty Clay with Sand (CL)	Expansion Index	= 34
			Expansion Potential	= Low
			Soluble Sulfate Content	0.049%
			Liquid Limit	= 33.5
			Plastic Limit	= 17.1
			Plasticity Index	= 16.4

Specimen No.	1	2	3
Normal Stress (ksf)	1	3	6
Peak Stress (ksf)	0.792	1.8	3.096
Peak Displacement (in)	9.091	9.731	10.33
Ultimate Stress (ksf)	0.792	1.8	3.072
Ultimate Displacement (in)	0.246	0.245	0.245
Initial Dry Density	111.1	111.2	111.2
Initial Moisture Content (%)	9.744	9.673	9.673
Strain Rate (in/min)	0.020		







Strain Legend

◆ 1 ksf

● 3 ksf

▲ 6 ksf

Strength Legend

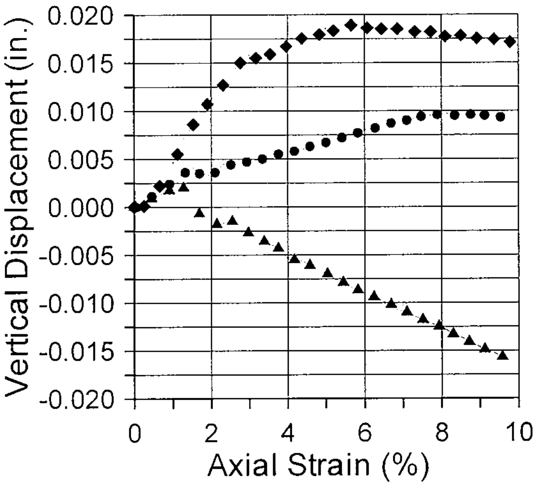
□ Peak

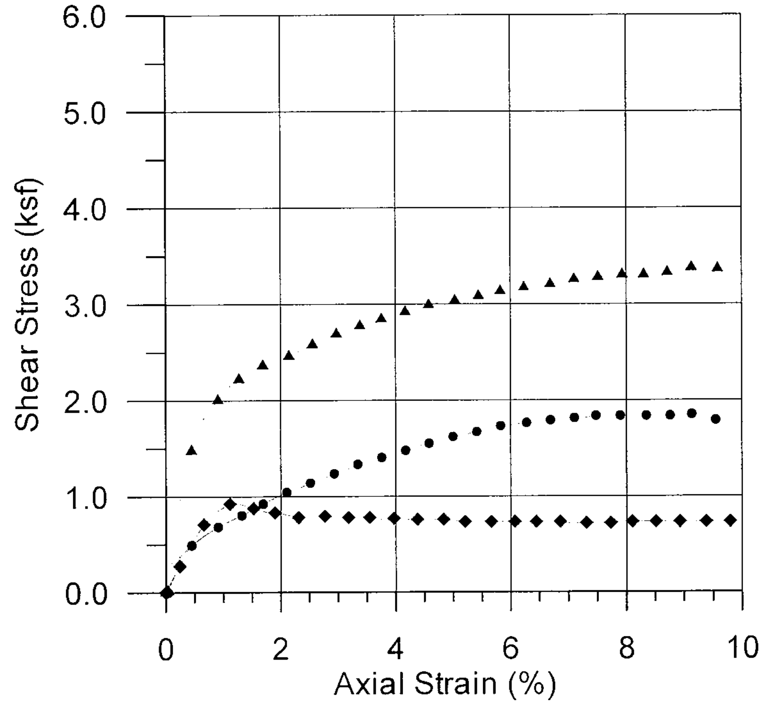
○ Ultimate

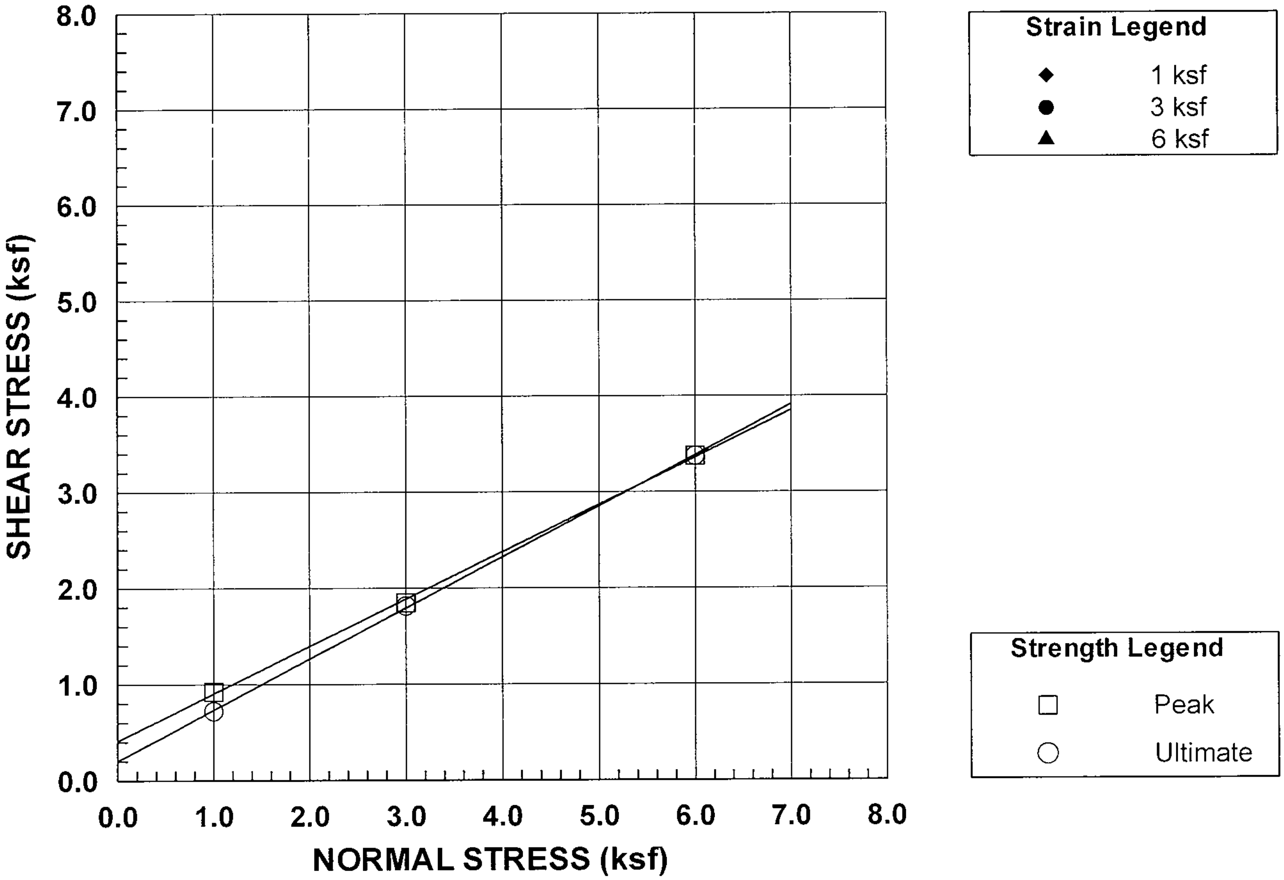
SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
B-1 @ 6'	REMOLDED-SATURATED	Sandy Silt (ML)

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**DIRECT SHEAR**	**Job No: 1489.00**
		Plate No: B-1

Specimen No.	1	2	3
Normal Stress (ksf)	1	3	6
Peak Stress (ksf)	0.924	1.848	3.372
Peak Displacement (in)	1.095	9.132	9.132
Ultimate Stress (ksf)	0.72	1.812	3.372
Ultimate Displacement (in)	0.246	0.242	0.242
Initial Dry Density	116.5	116.5	116.5
Initial Moisture Content (%)	9.487	9.487	9.487
Strain Rate (in/min)	0.050		







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
B-1 @ 16'	REMOLDED-SATURATED	Silty Sand (SM)

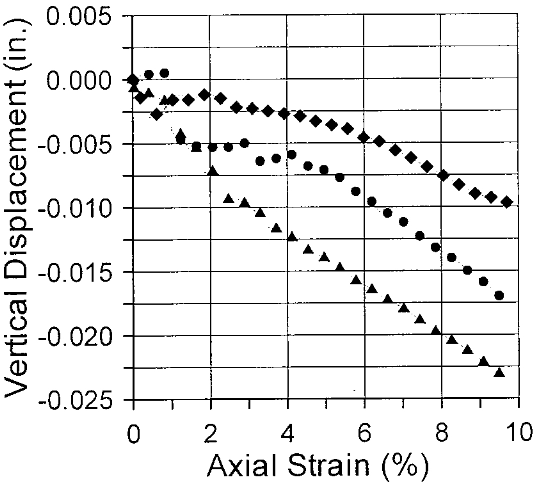
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

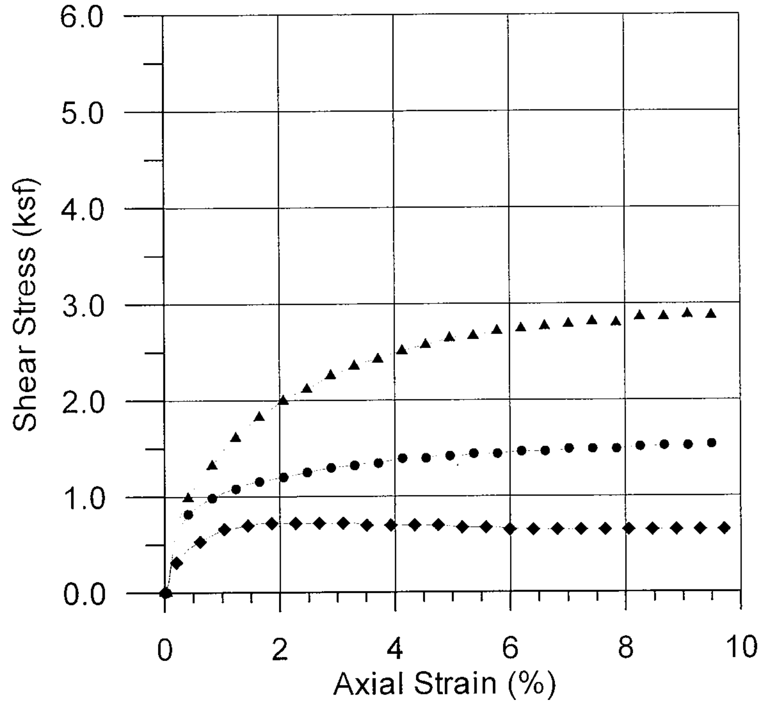
DIRECT SHEAR

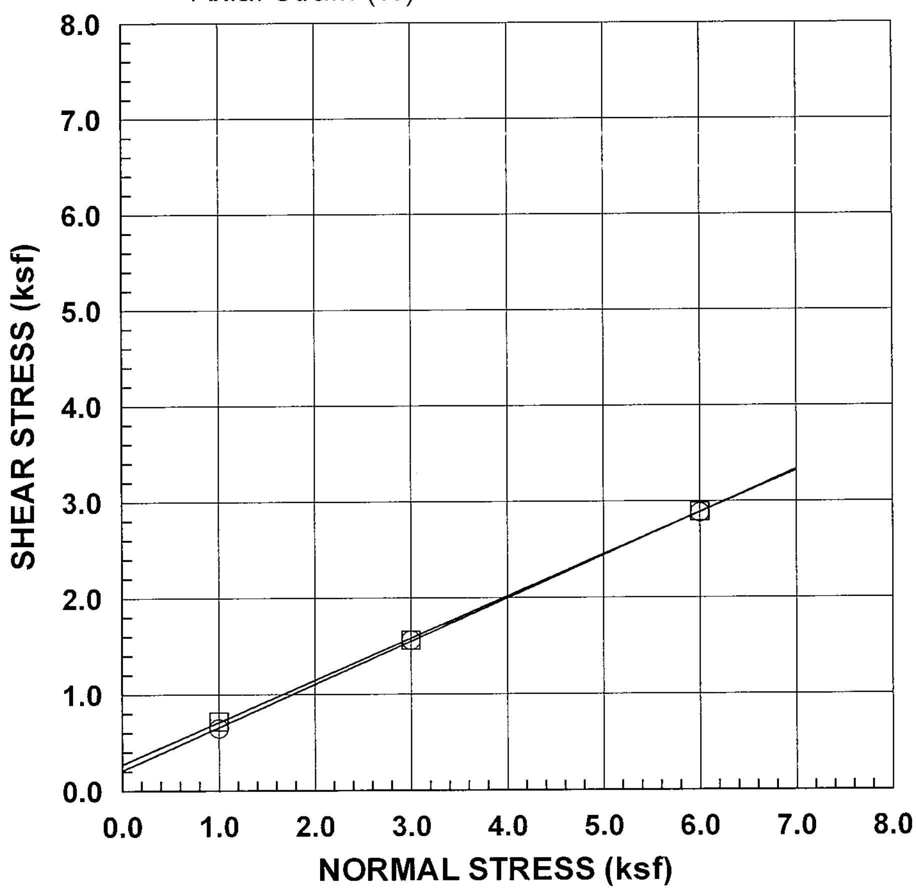
Job No: 1489.00

Plate No: B-2

Specimen No.	1	2	3
Normal Stress (ksf)	1	3	6
Peak Stress (ksf)	0.72	1.56	2.892
Peak Displacement (in)	1.653	9.917	10.12
Ultimate Stress (ksf)	0.648	1.56	2.88
Ultimate Displacement (in)	0.245	0.24	0.24
Initial Dry Density	106.7	106.7	106.7
Initial Moisture Content (%)	11.47	11.47	11.47
Strain Rate (in/min)	0.010		







Strain Legend

- ◆ 1 ksf
- ● 3 ksf
- ▲ 6 ksf

Strength Legend

- □ Peak
- ○ Ultimate

SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
B-1 @ 26'	REMOLDED-SATURATED	Ckayey Silt (ML) trace Sand

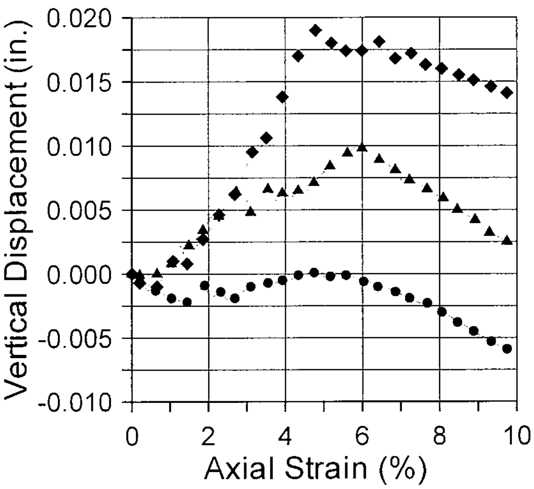
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

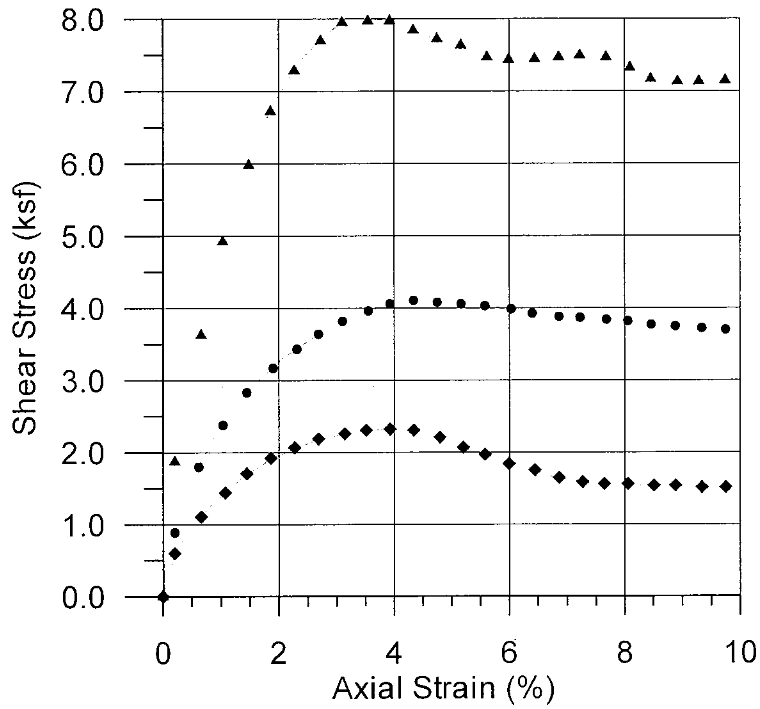
DIRECT SHEAR

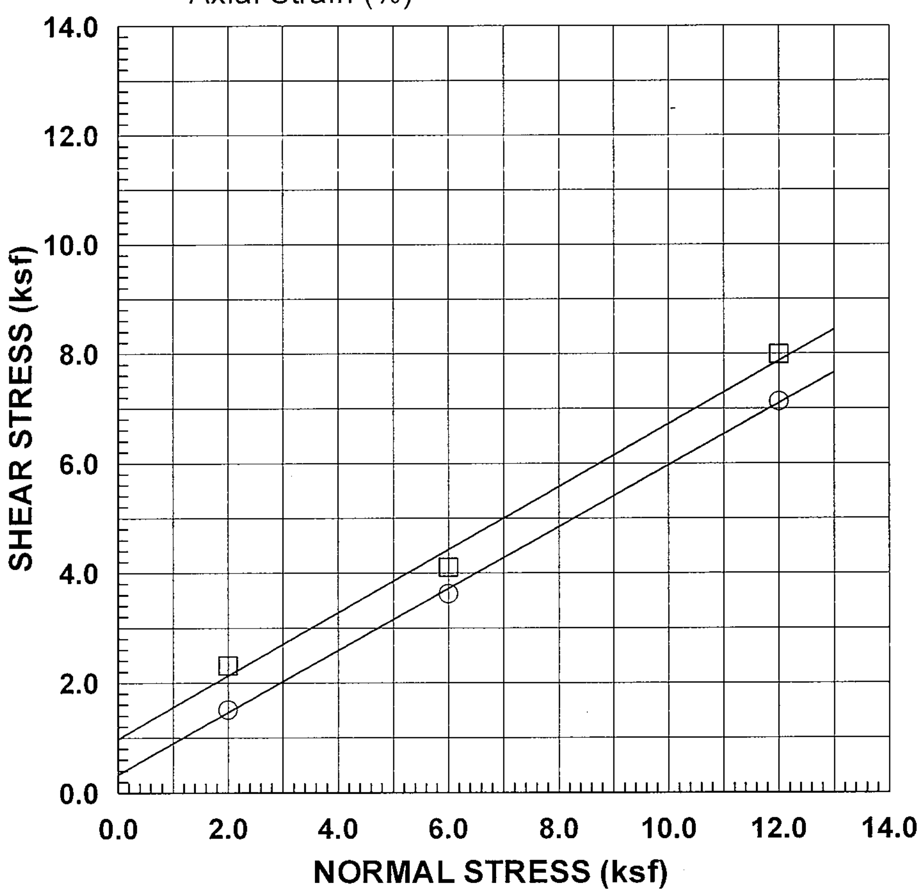
Job No: 1489.00

Plate No: B-3

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	2.316	4.104	7.992
Peak Displacement (in)	4.339	4.339	0
Ultimate Stress (ksf)	1.512	3.624	7.128
Ultimate Displacement (in)	0.245	0.246	0.246
Initial Dry Density	113.1	111.7	111.5
Initial Moisture Content (%)	16.35	16.35	16.35
Strain Rate (in/min)	0.020		







Strain Legend

◆ 2 ksf
● 6 ksf
▲ 12 ksf

Strength Legend

□ Peak
○ Ultimate

SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
HS-3 @ 27'	UNDISTURBED-SATURATED	SANDY SILT (ML)

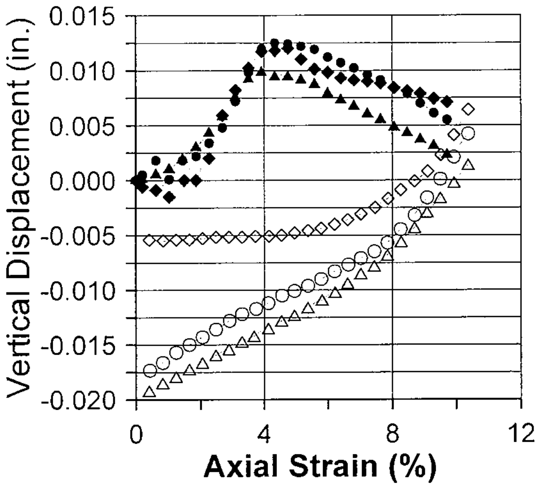
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

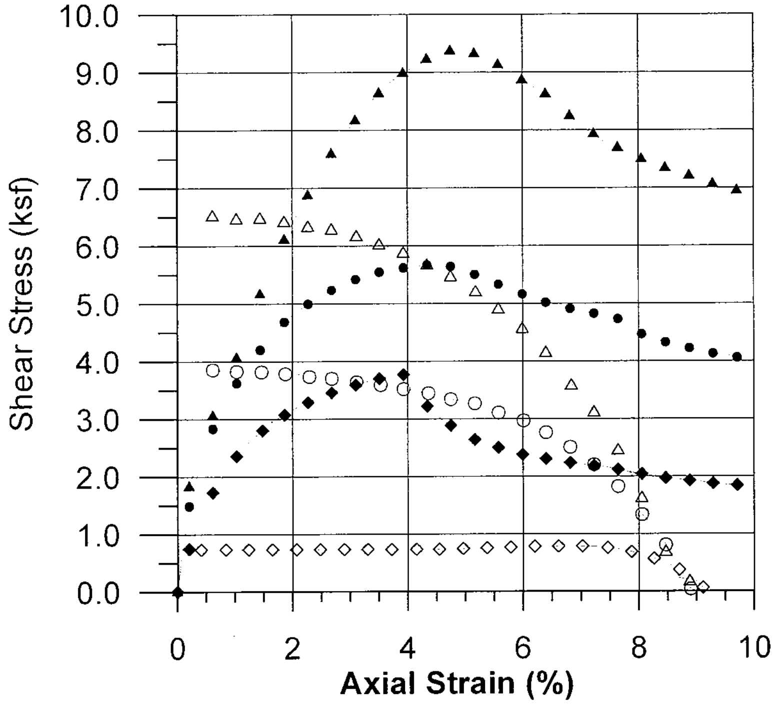
DIRECT SHEAR

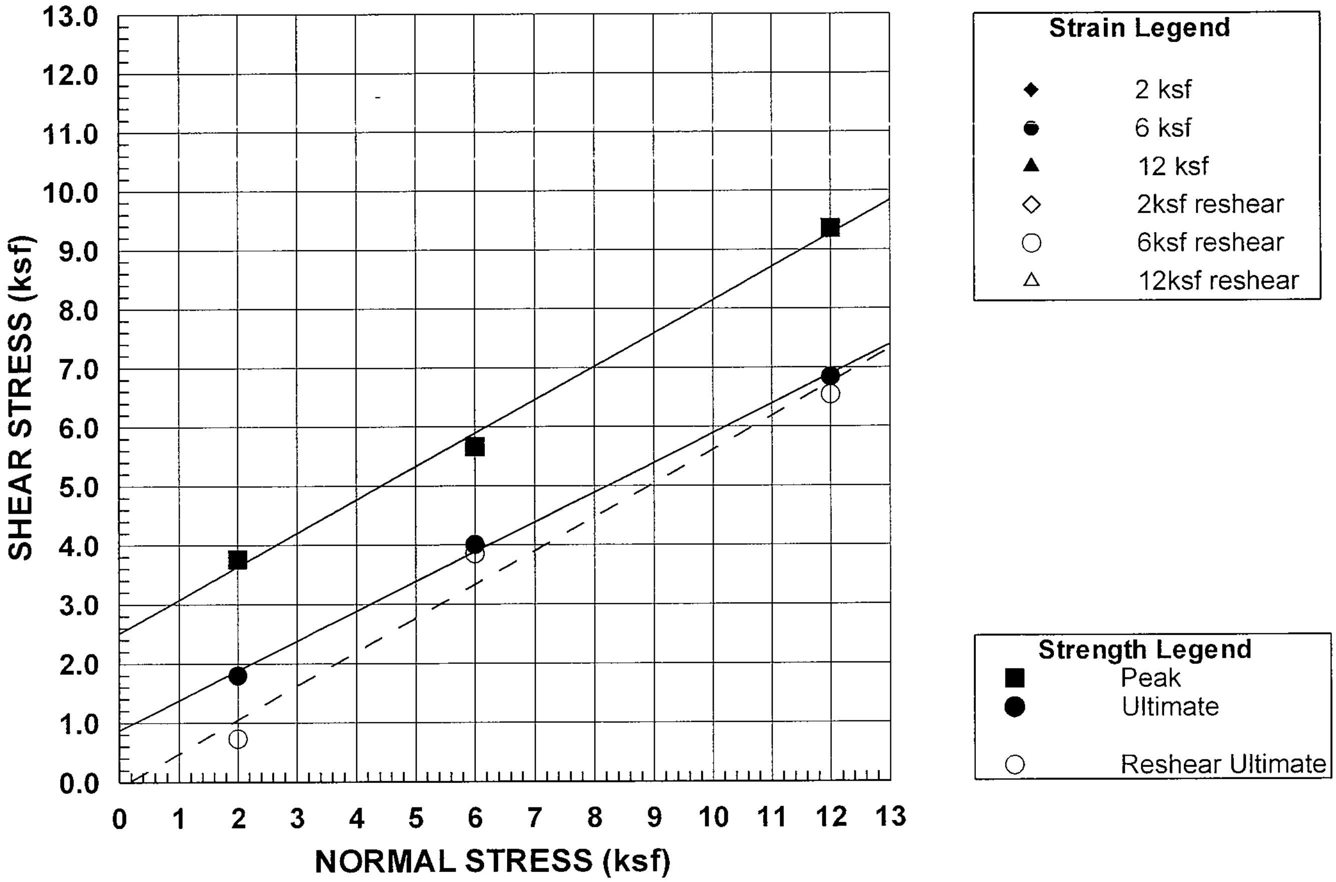
Job No: 1489.00

Plate No: B-4

Specimen No.	1	2	3
Normal Stress (ksf)	2	6	12
Peak Stress (ksf)	3.768	5.664	9.359
Peak Displacement (in)	3.926	4.339	4.752
Ultimate Stress (ksf)	1.8	4.008	6.84
Ultimate Displacement (in)	0.245	0.245	0.245
Initial Dry Density	101.2	99.58	101.4
Initial Moisture Content (%)	30.71	30.71	30.71
Strain Rate (in/min)	0.010		







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
HS-3 @ 30'	UNDIST-SAT-RESHEAR	Silty Clay (CL) with Gravel

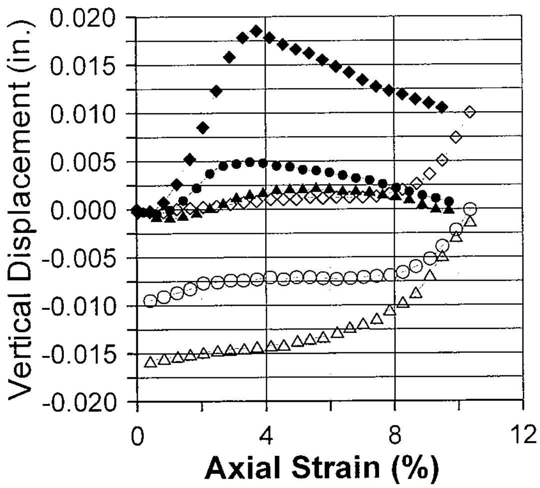
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

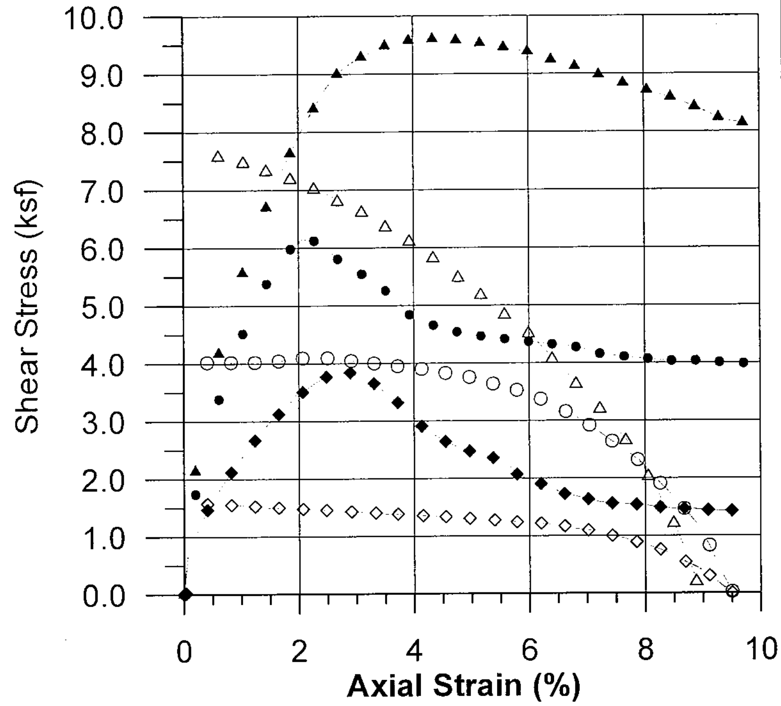
DIRECT SHEAR

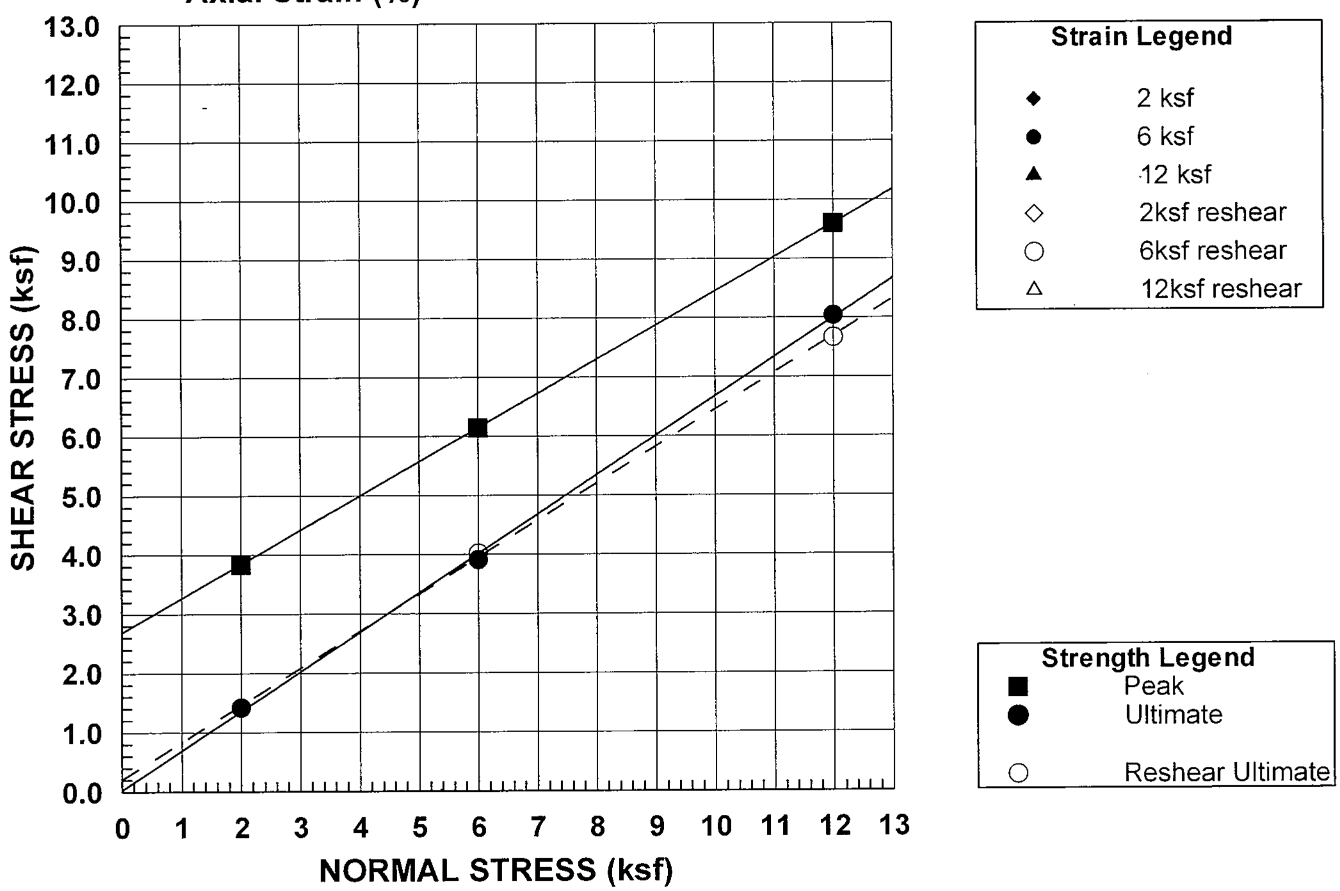
Job No: 1489.00

Plate No: B-5

Specimen No.	1	2	3
Normal Stress (ksf)	2	6	12
Peak Stress (ksf)	3.84	6.144	9.599
Peak Displacement (in)	2.686	2.066	4.339
Ultimate Stress (ksf)	1.416	3.912	8.04
Ultimate Displacement (in)	0.24	0.245	0.245
Initial Dry Density	119.7	118.8	118.6
Initial Moisture Content (%)	13.7	13.7	13.7
Strain Rate (in/min)	0.010		







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
HS-4 @ 35'	UNDIST-SAT-RESHEAR	Sandy Clay (CL)

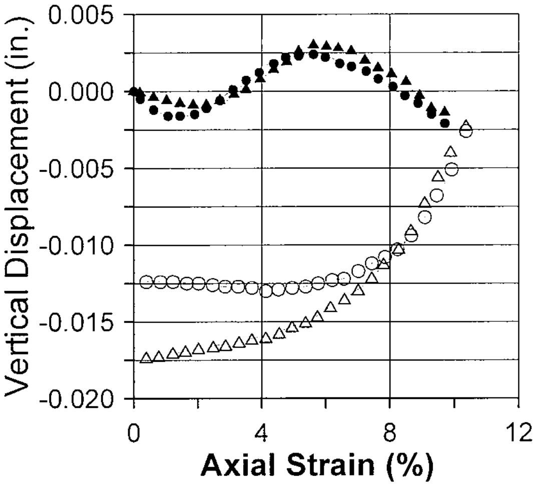
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

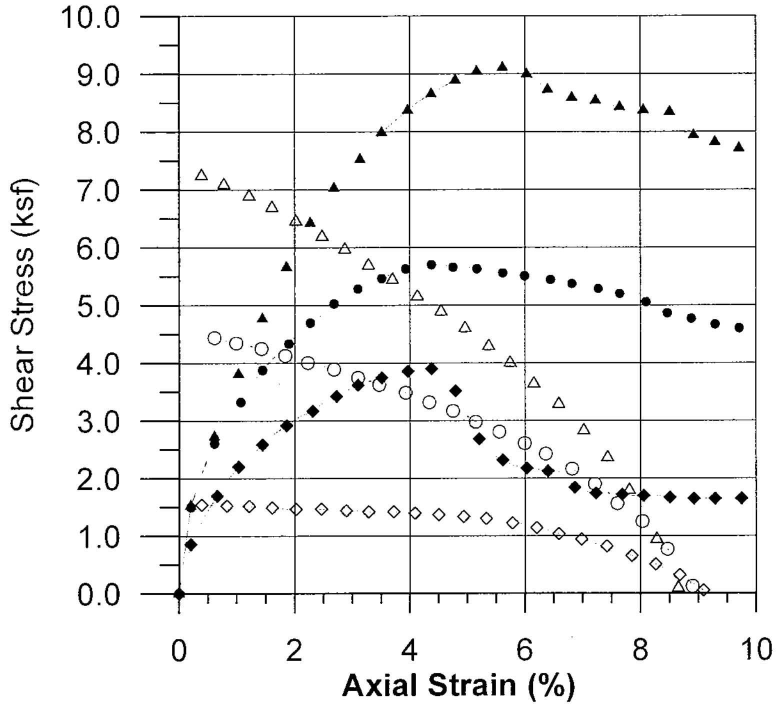
DIRECT SHEAR

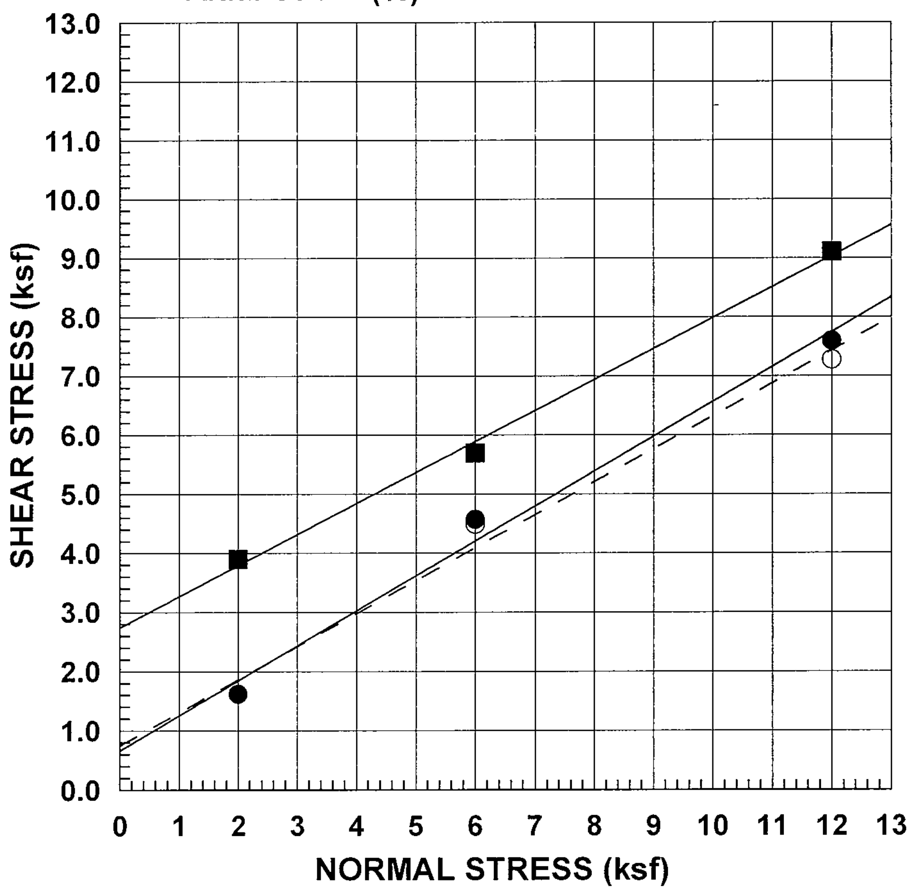
Job No: 1489.00

Plate No: B-6

Specimen No.	1	2	3
Normal Stress (ksf)	2	6	12
Peak Stress (ksf)	3.9	5.7	9.107
Peak Displacement (in)	0	4.339	4.752
Ultimate Stress (ksf)	1.62	4.572	7.596
Ultimate Displacement (in)	0.245	0.246	0.245
Initial Dry Density	118.8	119.6	119.3
Initial Moisture Content (%)	13.37	13.37	13.37
Strain Rate (in/min)	0.020		







Strain Legend

- ◆ 2 ksf
- ● 6 ksf
- ▲ 12 ksf
- ◇ 2ksf reshear
- ○ 6ksf reshear
- △ 12ksf reshear

Strength Legend

- ■ Peak
- ● Ultimate
- ○ Reshear Ultimate

SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
HS-6 @ 27'	UNDIST-SAT-RESHEAR	Sandy Silt (ML) some Gravel

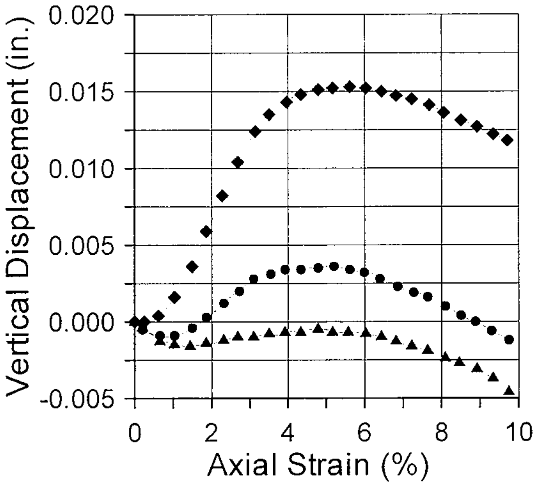
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

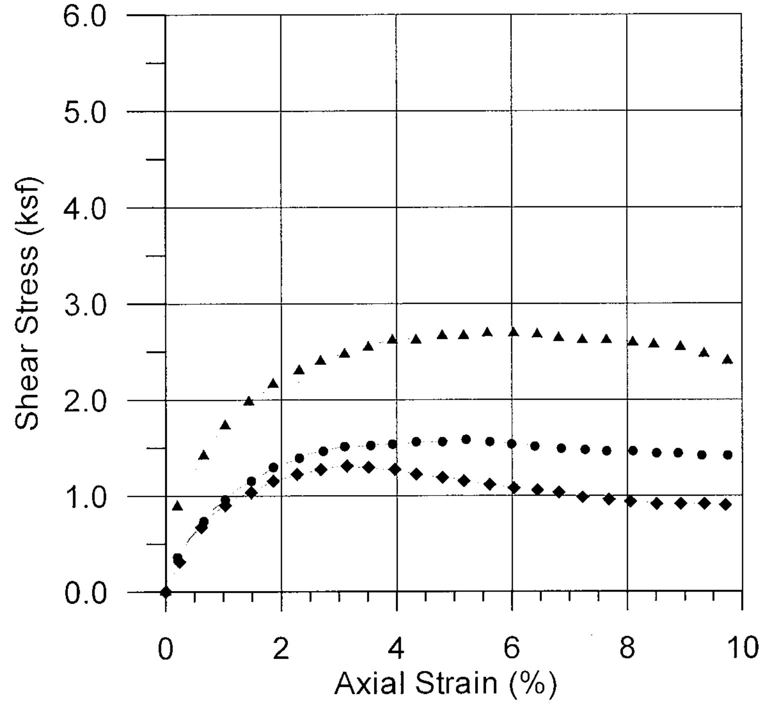
DIRECT SHEAR

Job No: 1489.00

Plate No: B-7

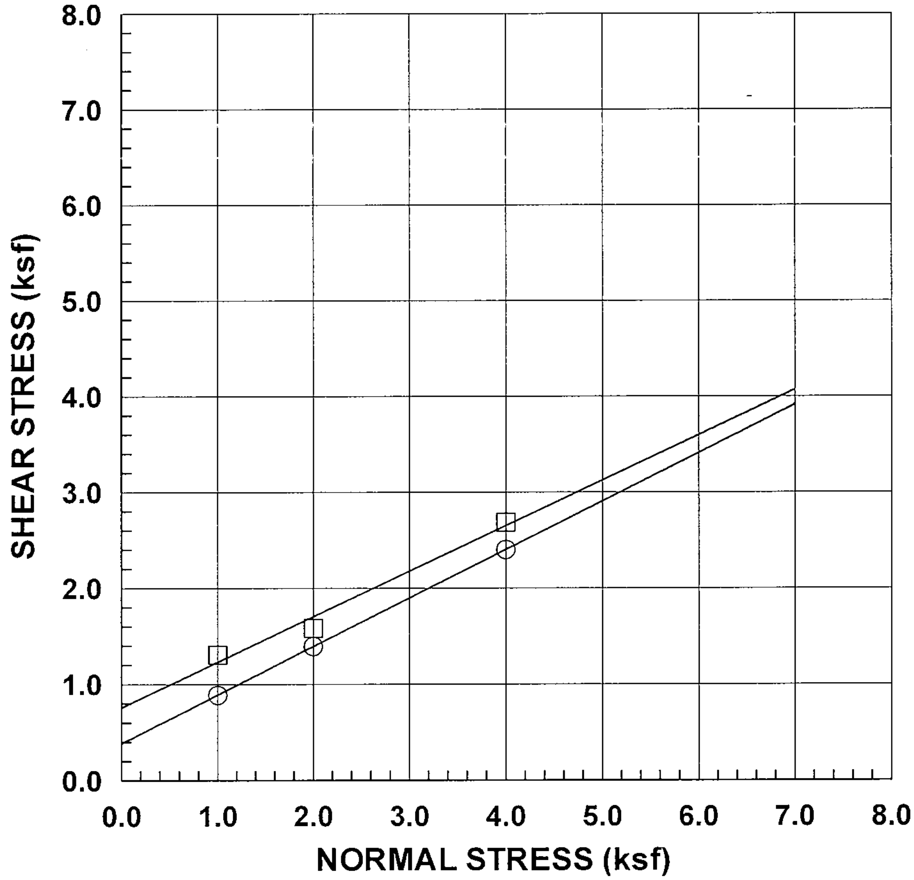
Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	1.308	1.584	2.688
Peak Displacement (in)	0	0	5.579
Ultimate Stress (ksf)	0.888	1.392	2.4
Ultimate Displacement (in)	0.246	0.246	0.245
Initial Dry Density	109.9	110.2	109.7
Initial Moisture Content (%)	18.63	18.63	18.63
Strain Rate (in/min)	0.020		





Strain Legend

◆ 1 ksf
● 2 ksf
▲ 4 ksf



Strength Legend

□ Peak
○ Ultimate

SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
HS-7 @ 20'	UNDISTURBED-SATURATED	Sandy Silt (ML)

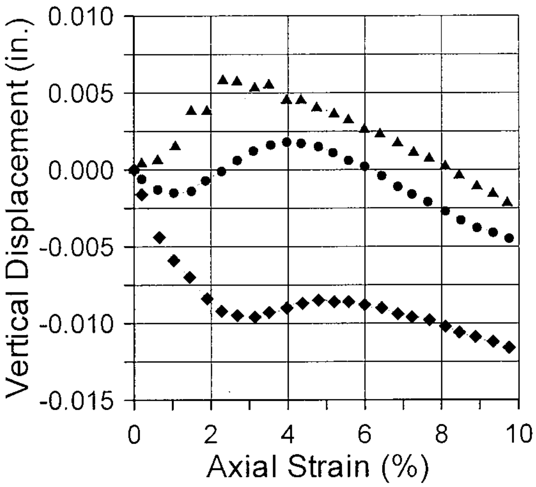
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

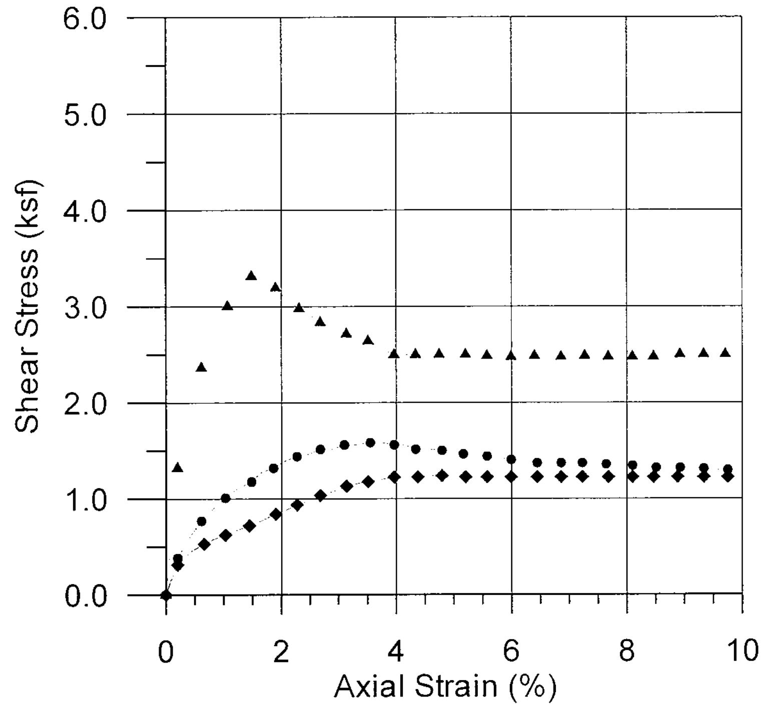
DIRECT SHEAR

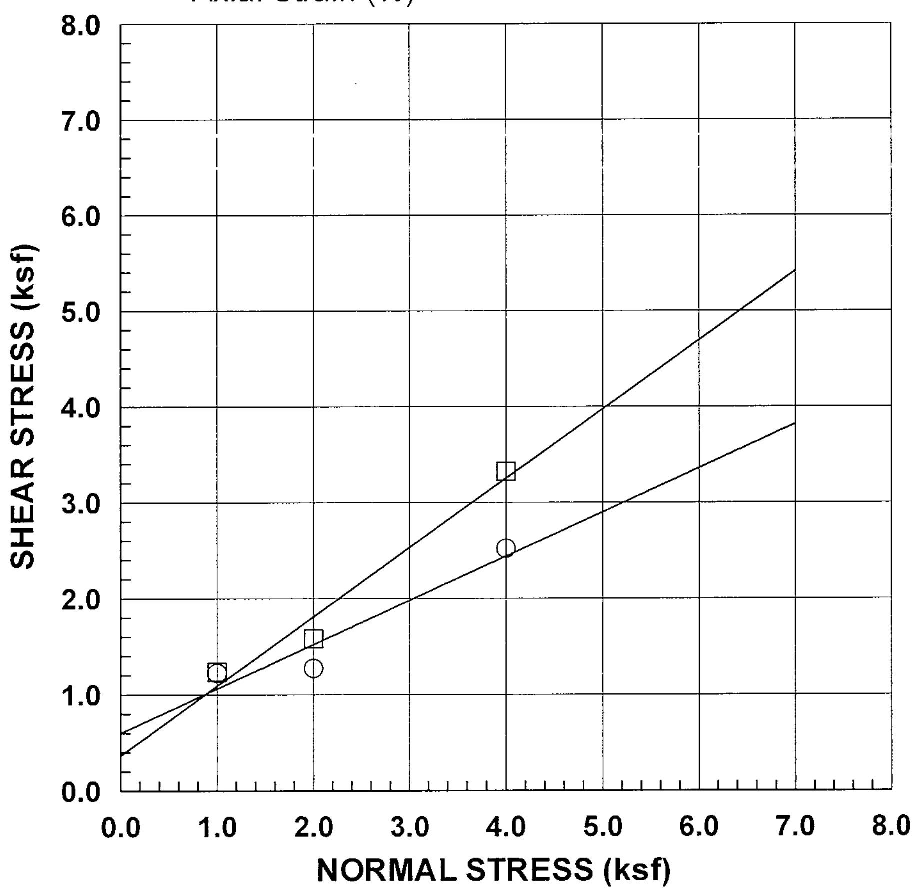
Job No: 1489.00

Plate No: B-8

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	1.236	1.584	3.324
Peak Displacement (in)	4.566	0	1.653
Ultimate Stress (ksf)	1.224	1.272	2.52
Ultimate Displacement (in)	0.245	0.245	0.245
Initial Dry Density	103.3	106.4	106.6
Initial Moisture Content (%)	19.43	19.43	19.43
Strain Rate (in/min)	0.020		







Strain Legend

- ◆ 1 ksf
- ● 2 ksf
- ▲ 4 ksf

Strength Legend

- □ Peak
- ○ Ultimate

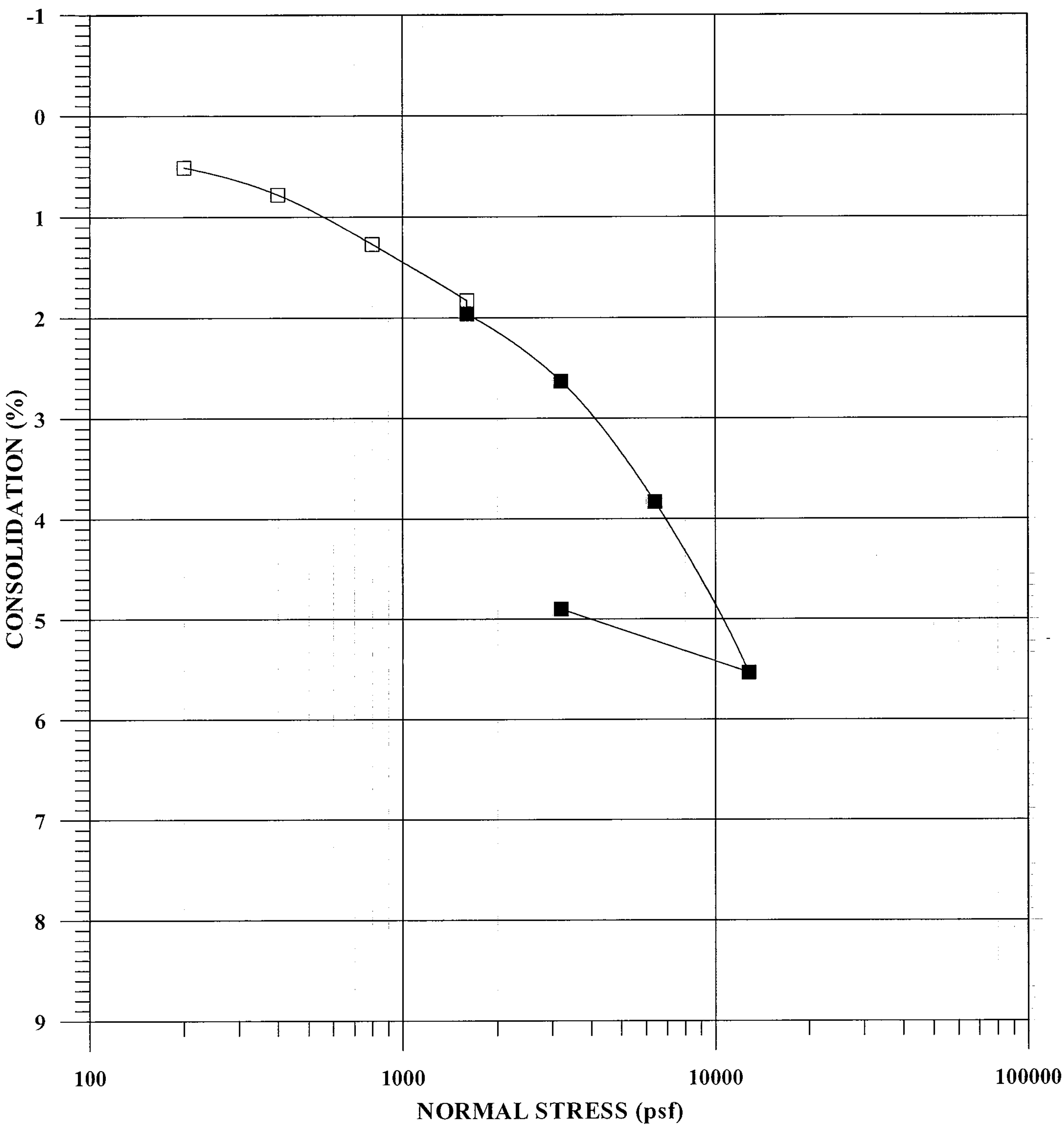
SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
HS-7 @ 30'	UNDISTURBED-SATURATED	Silty Sand to Sandy Silt (SM-ML)

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

DIRECT SHEAR

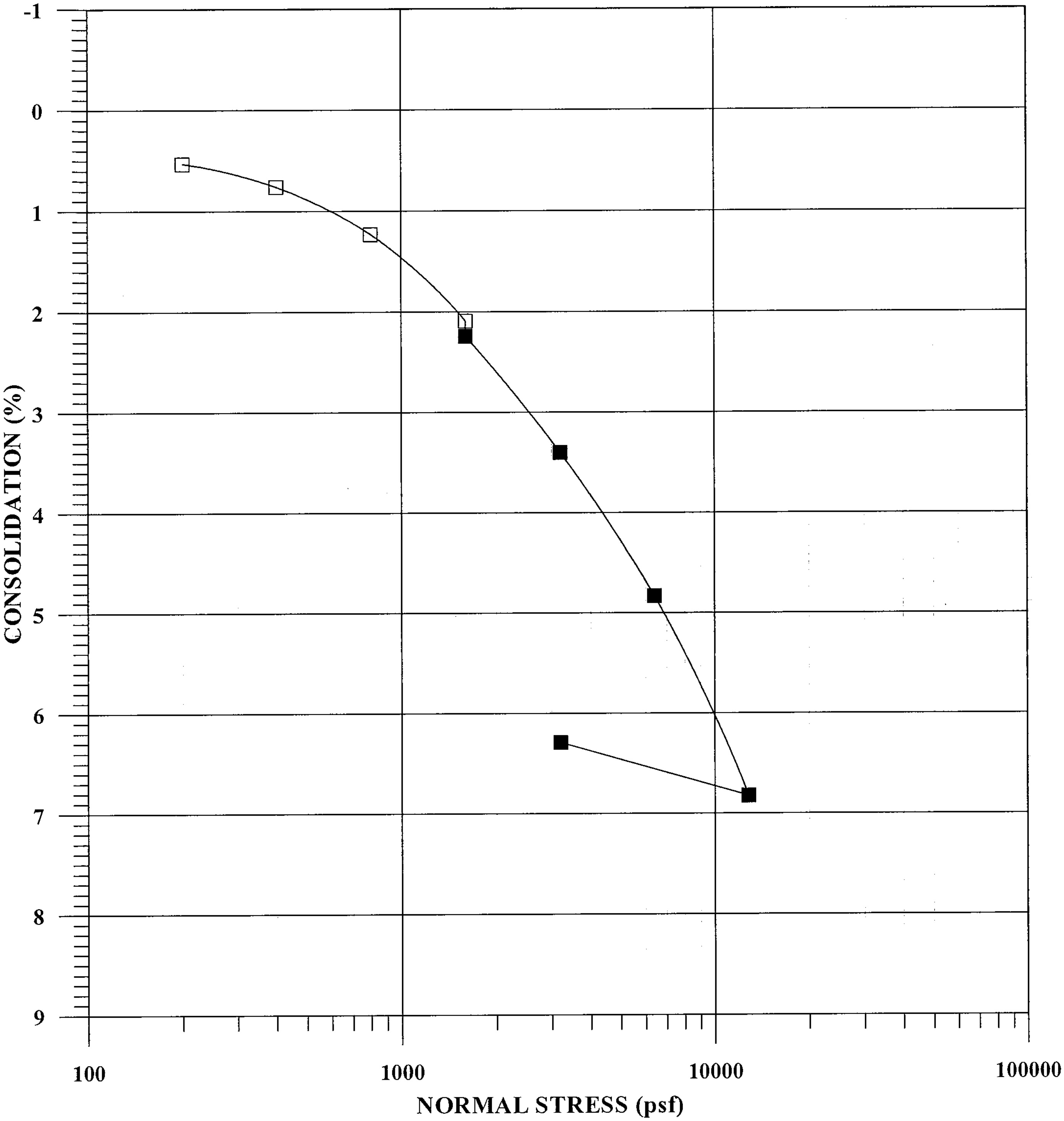
Job No: 1489.00

Plate No: B-9



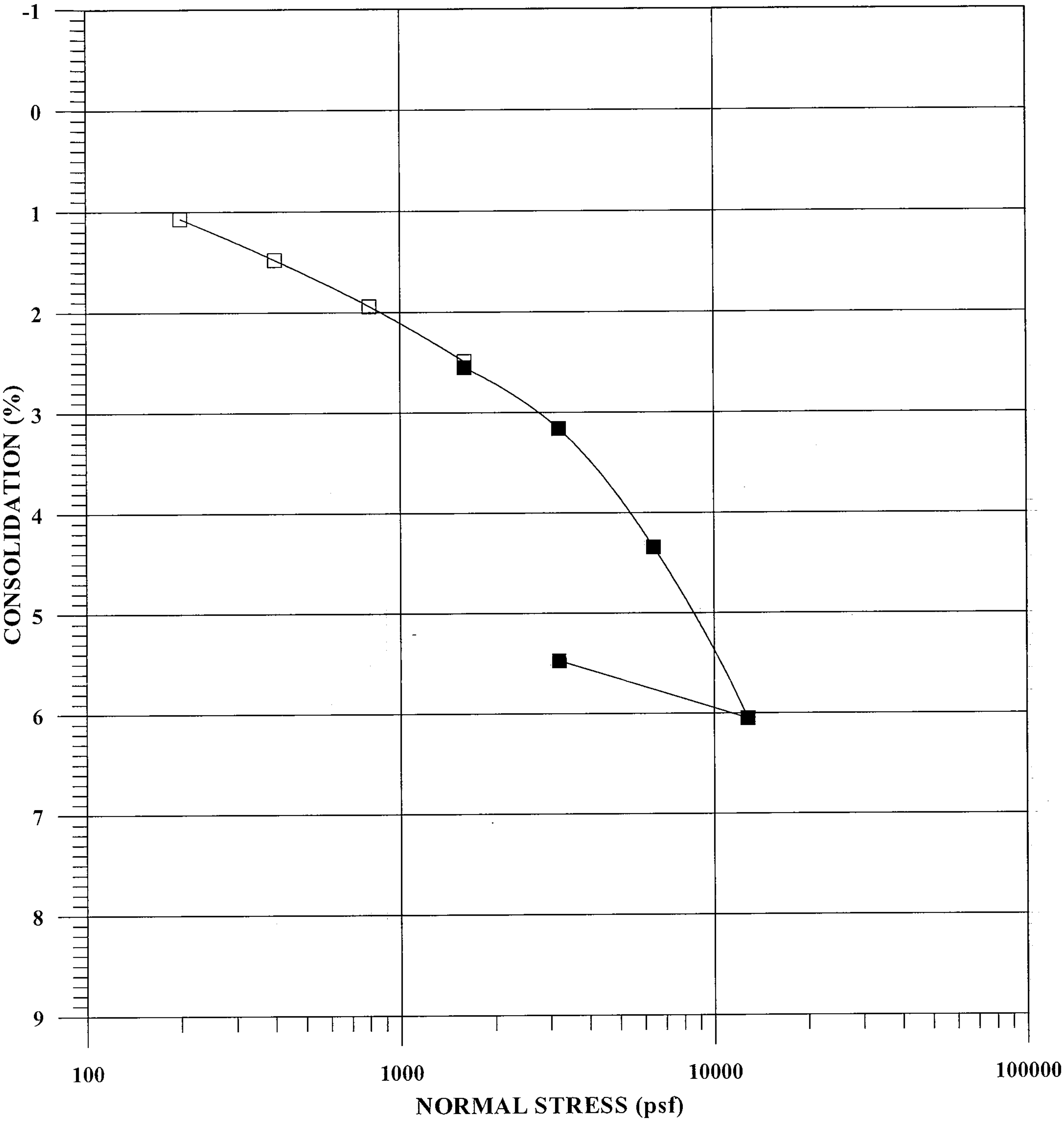
Sample Location:	HS-1	Initial Dry Density (pcf):	106.9	
Sample Depth:	7'	Initial Moisture Content (%):	16.3	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	18.5	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**CONSOLIDATION TEST RESULTS**	**Job No: 1489.00** **Figure No: B-10**



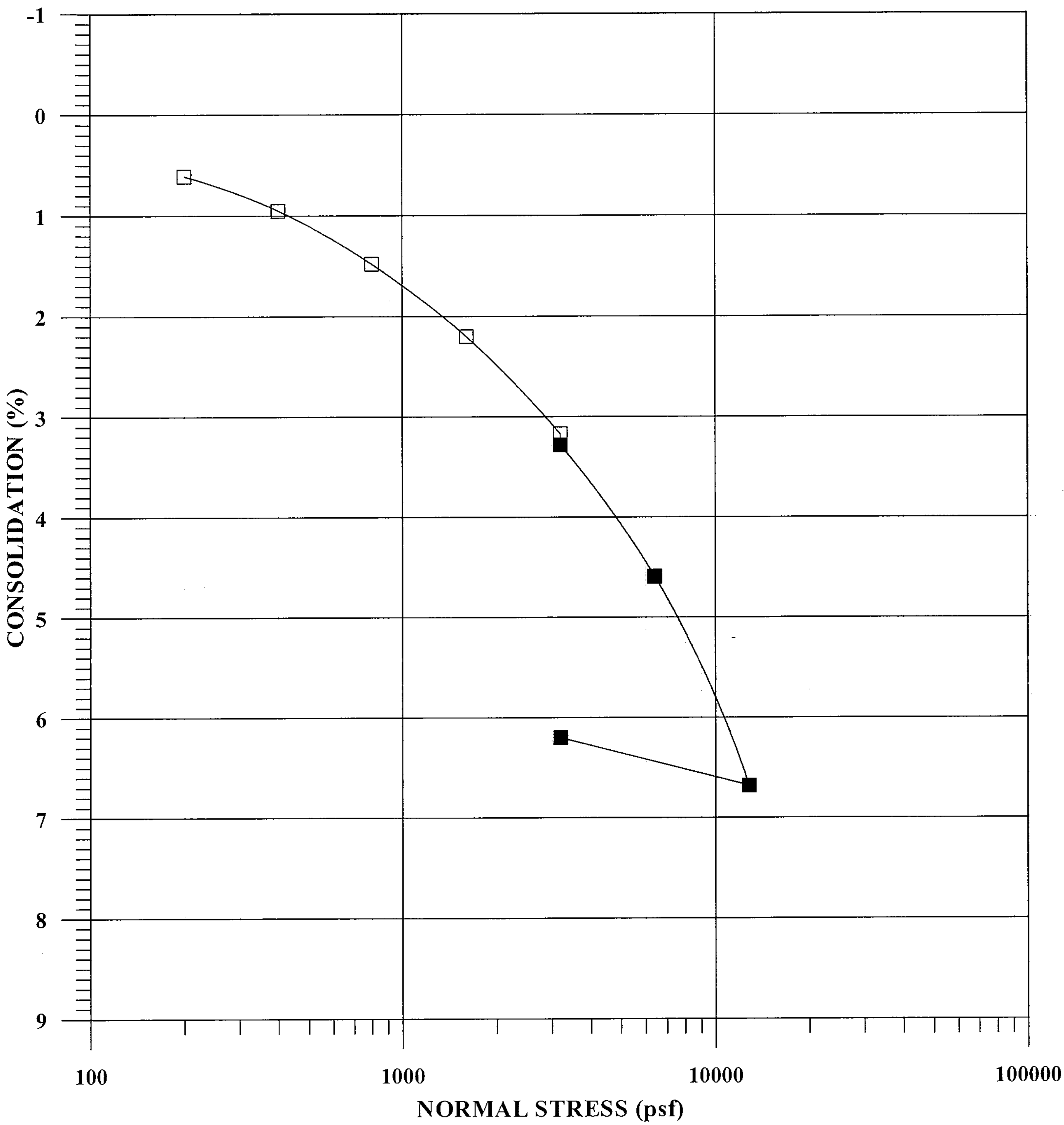
Sample Location:	HS-1	Initial Dry Density (pcf):	108.1	
Sample Depth:	10'	Initial Moisture Content (%):	16.4	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	18.9	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**CONSOLIDATION TEST RESULTS**	**Job No: 1489.00**
		Figure No: B-11



Sample Location:	HS-1	Initial Dry Density (pcf):	107.5	
Sample Depth:	15'	Initial Moisture Content (%):	18.6	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	19.1	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**CONSOLIDATION TEST RESULTS**	**Job No: 1489.00** **Figure No: B-12**



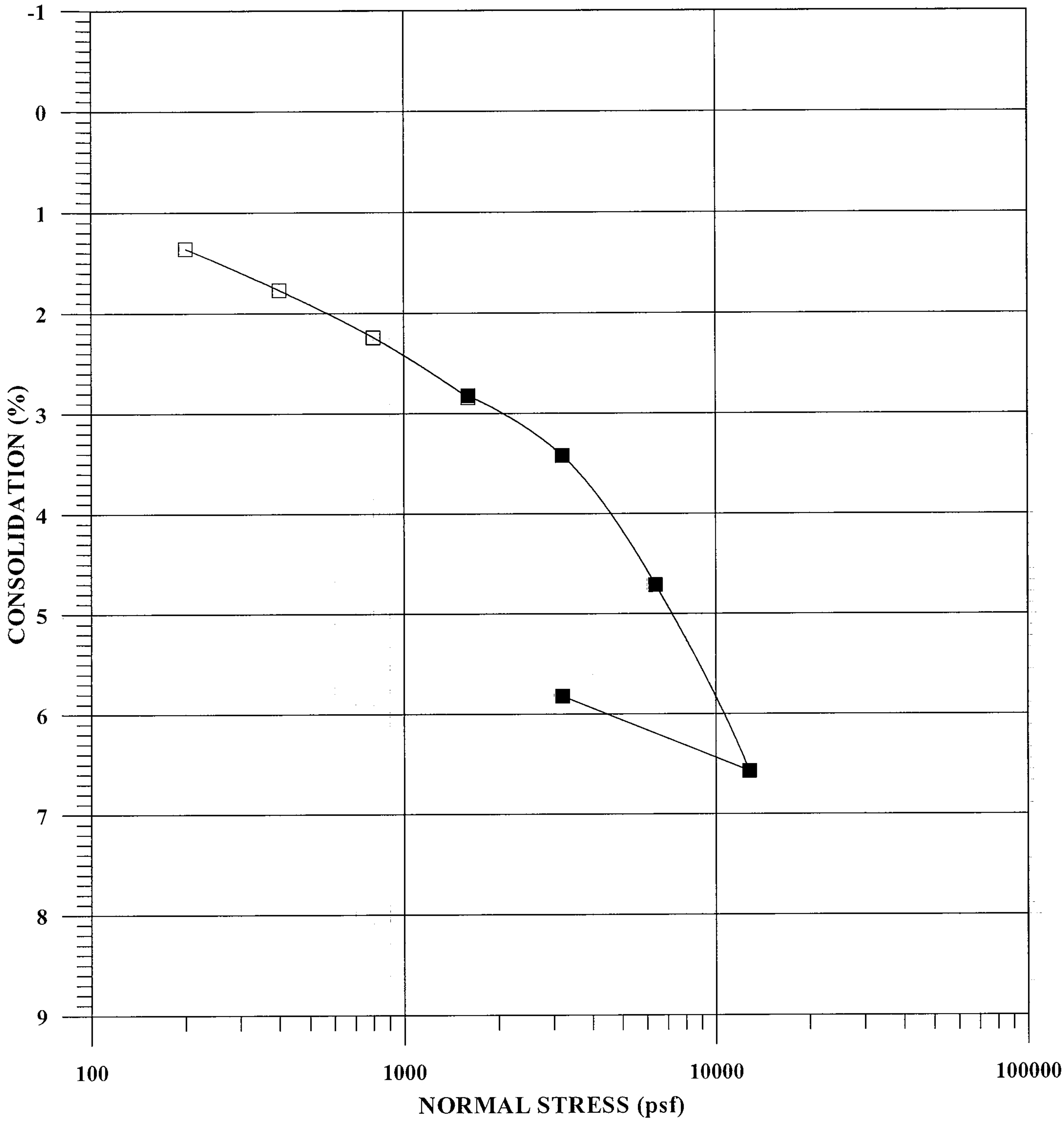
Sample Location:	HS-1	Initial Dry Density (pcf):	103.8	
Sample Depth:	20'	Initial Moisture Content (%):	18.8	Legend □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	21.0	

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

CONSOLIDATION TEST RESULTS

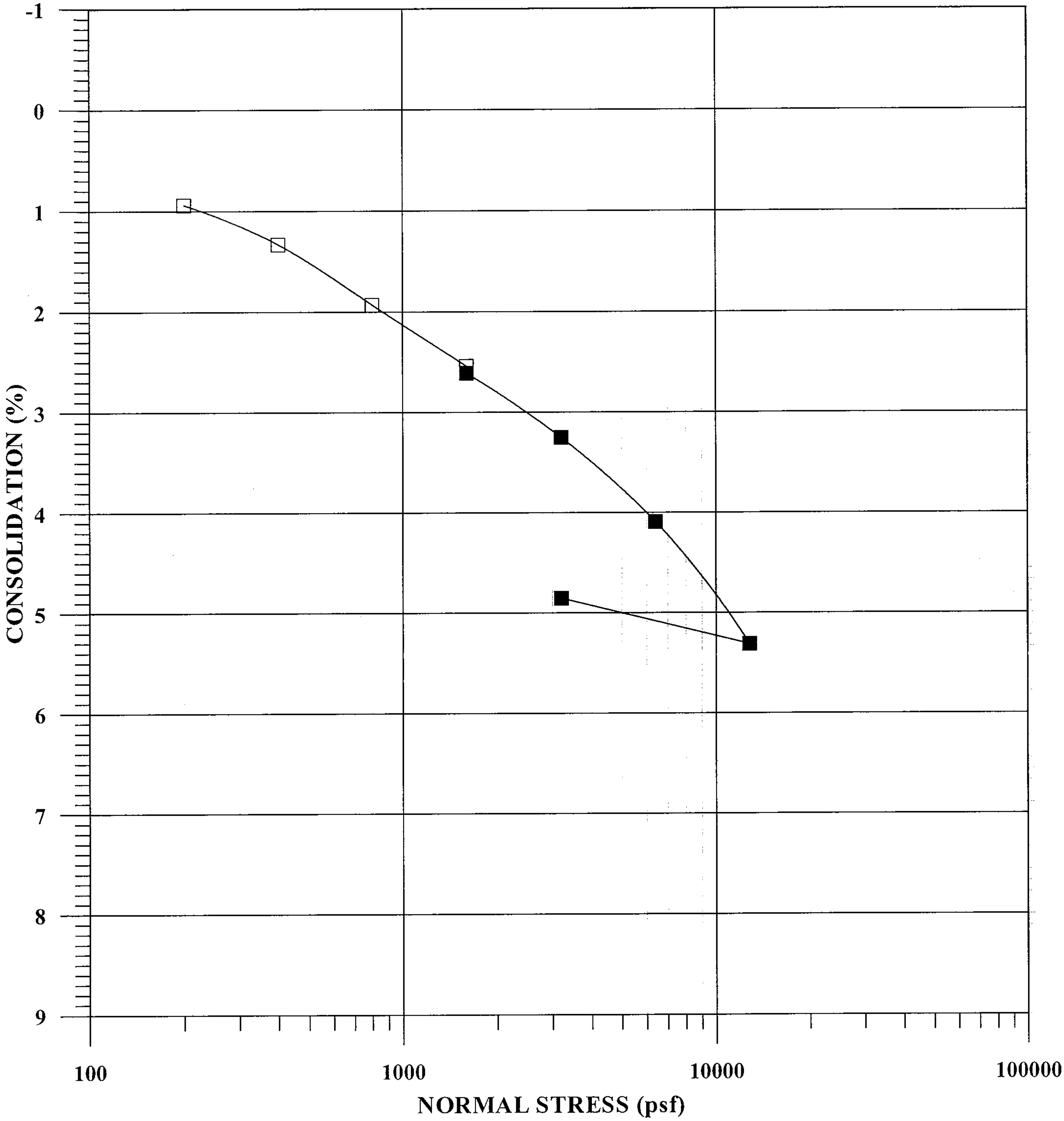
Job No: 1489.00

Figure No: B-13



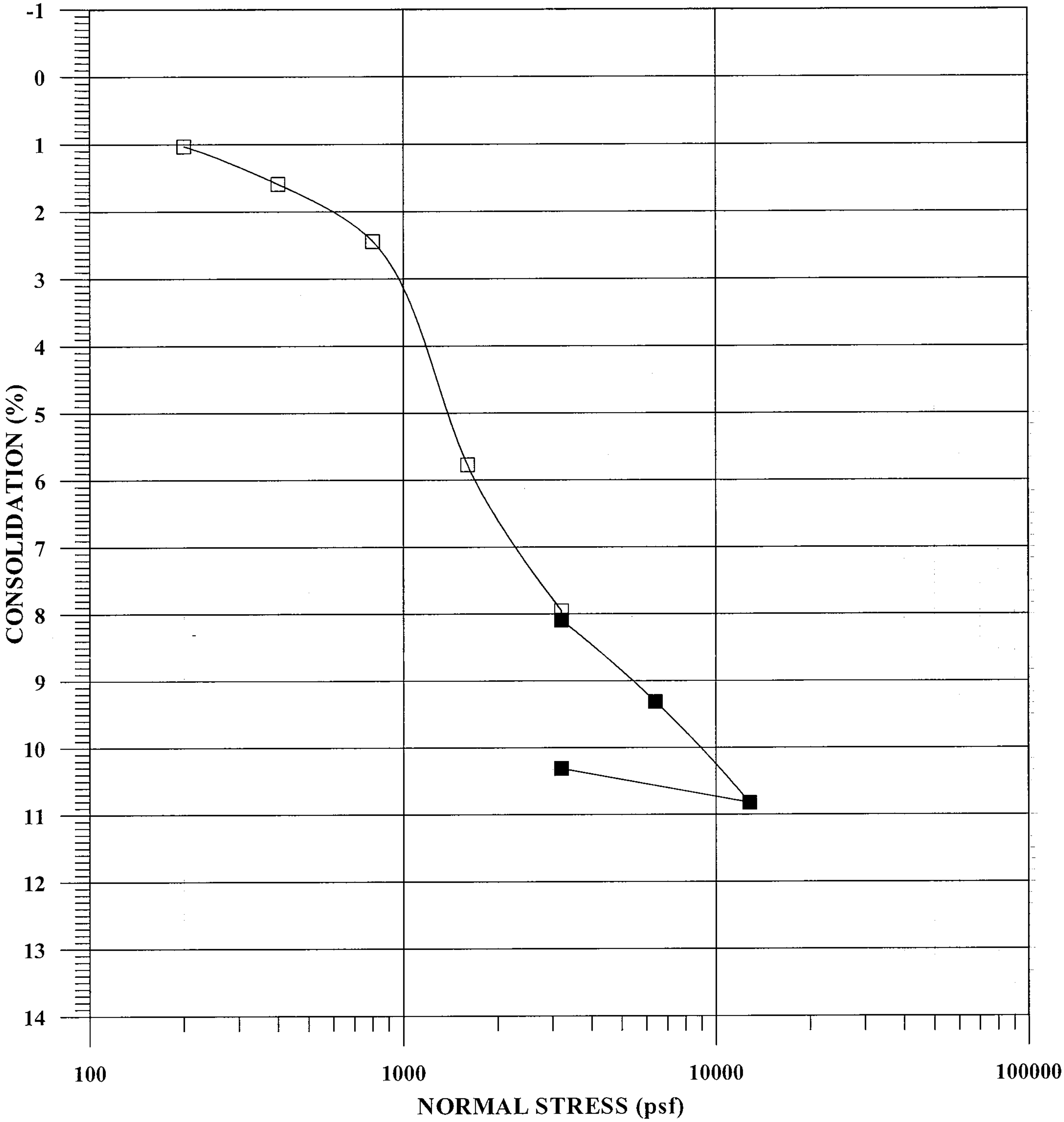
Sample Location:	HS-2	Initial Dry Density (pcf):	109.9	
Sample Depth:	5'	Initial Moisture Content (%):	18.5	**Legend** ☐ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	18.8	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00
		Figure No: B-14



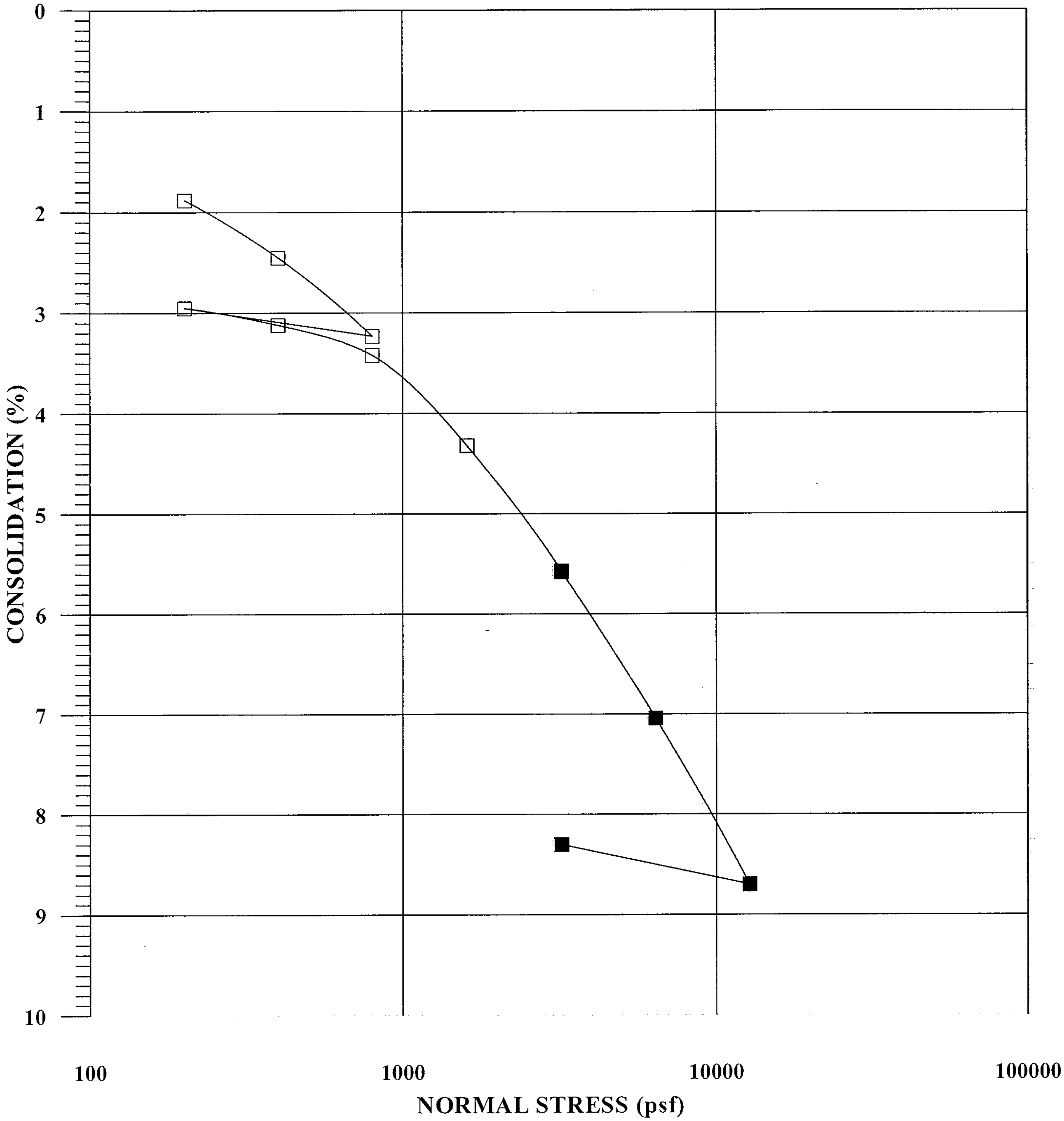
Sample Location:	HS-2	Initial Dry Density (pcf):	106.8	
Sample Depth:	10'	Initial Moisture Content (%):	18.5	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	20.0	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**CONSOLIDATION TEST RESULTS**	**Job No: 1489.00** **Figure No: B-15**



Sample Location:	HS-2	Initial Dry Density (pcf):	106.1	
Sample Depth:	20'	Initial Moisture Content (%):	20.7	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	20.1	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00
		Figure No: B-16



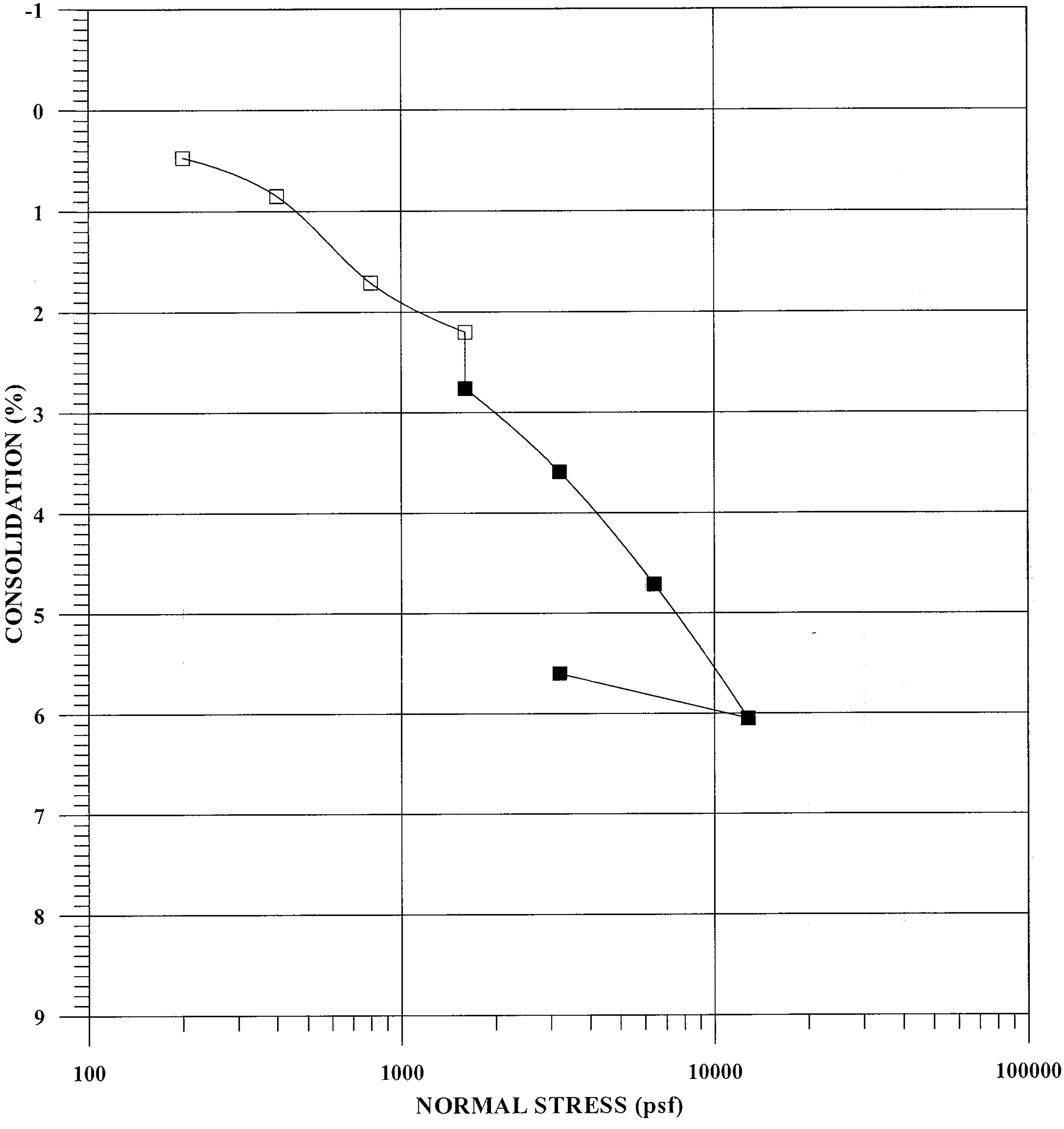
Sample Location:	HS-2	Initial Dry Density (pcf):	109.3	
Sample Depth:	25'	Initial Moisture Content:	20.4	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content:	18.2	

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

CONSOLIDATION TEST RESULTS

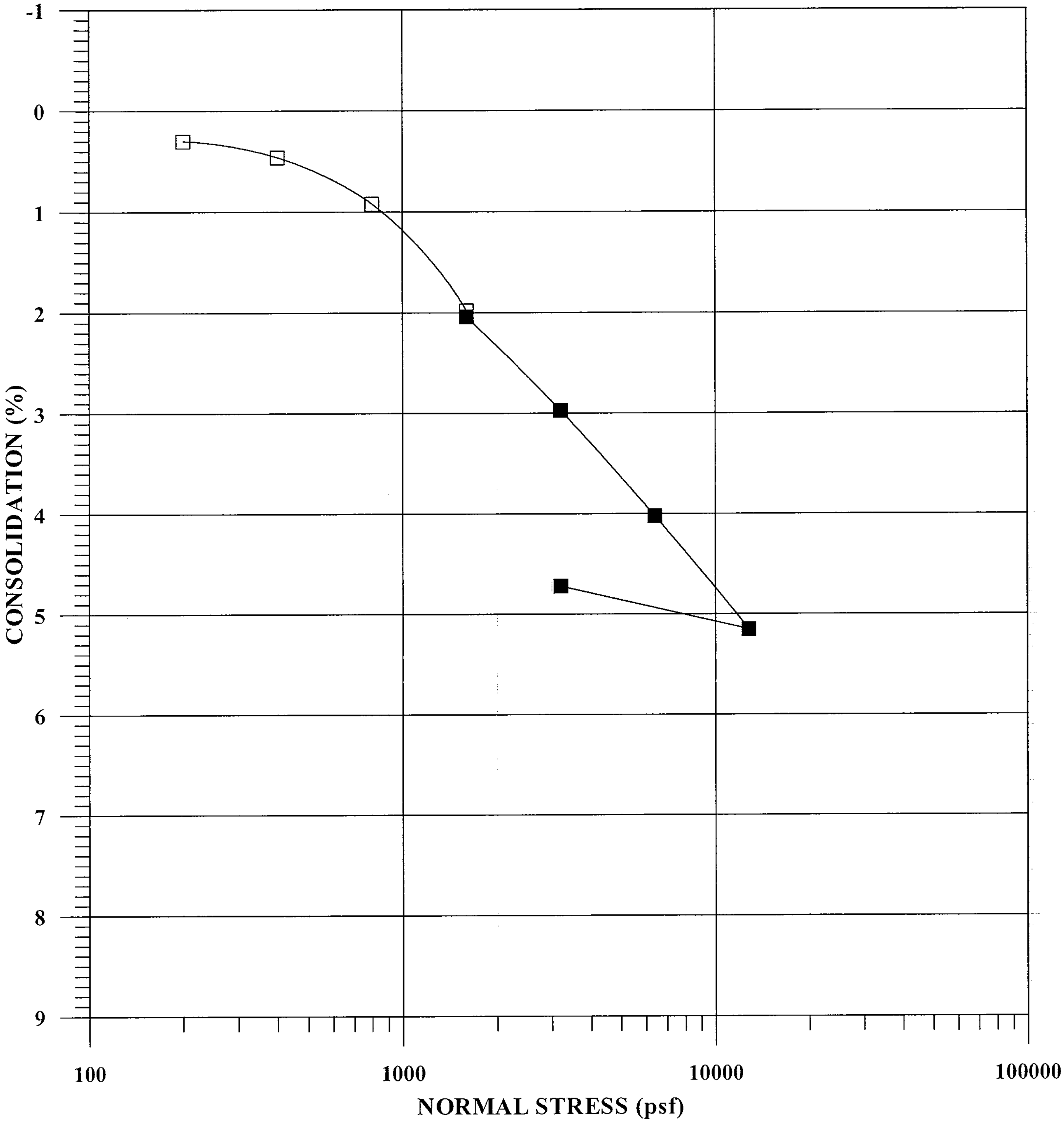
Job No: 1489.00

Plate No: B-17



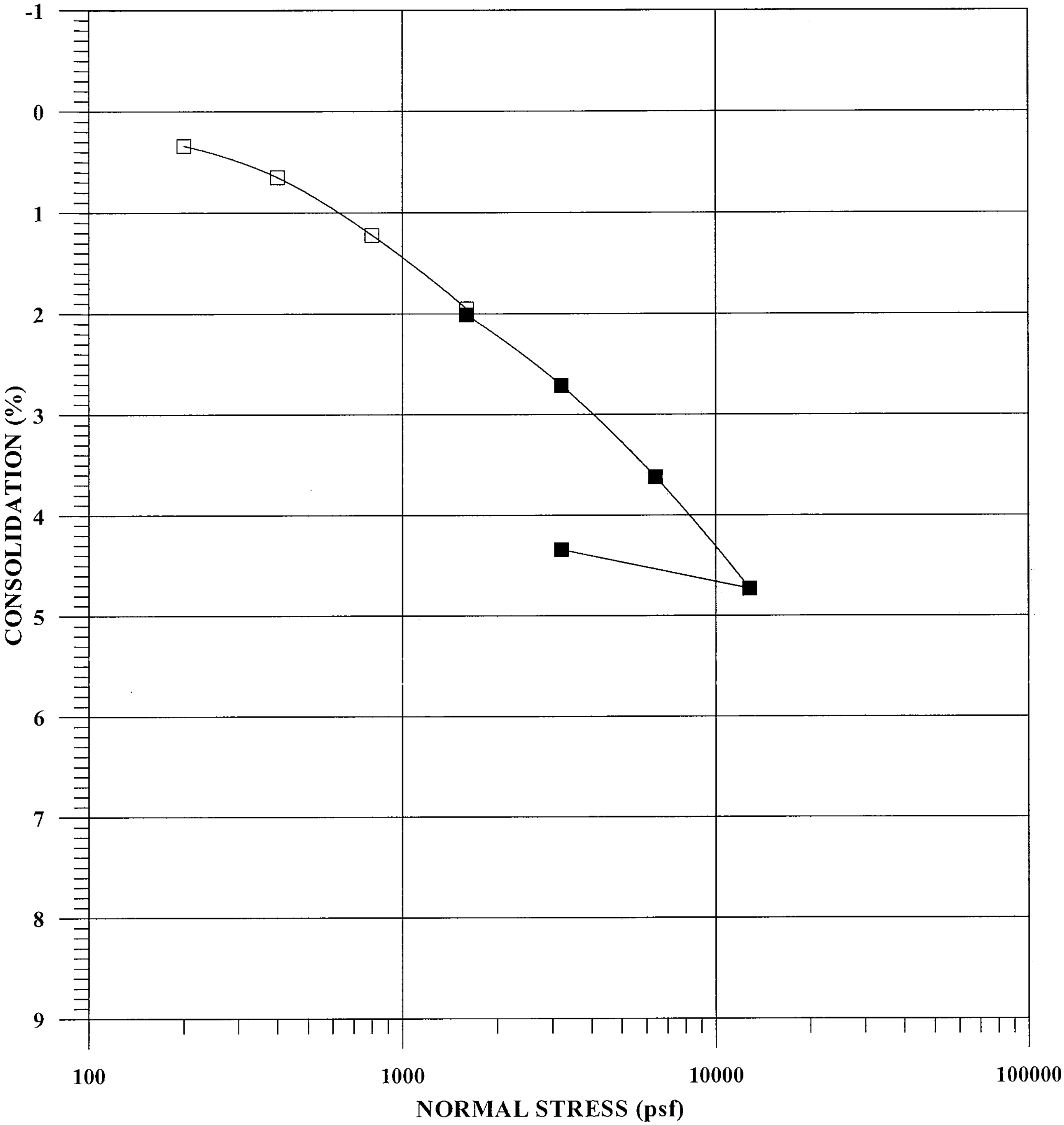
Sample Location:	HS-3	Initial Dry Density (pcf):	107.3	
Sample Depth:	10'	Initial Moisture Content (%):	17.7	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	20.0	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-18



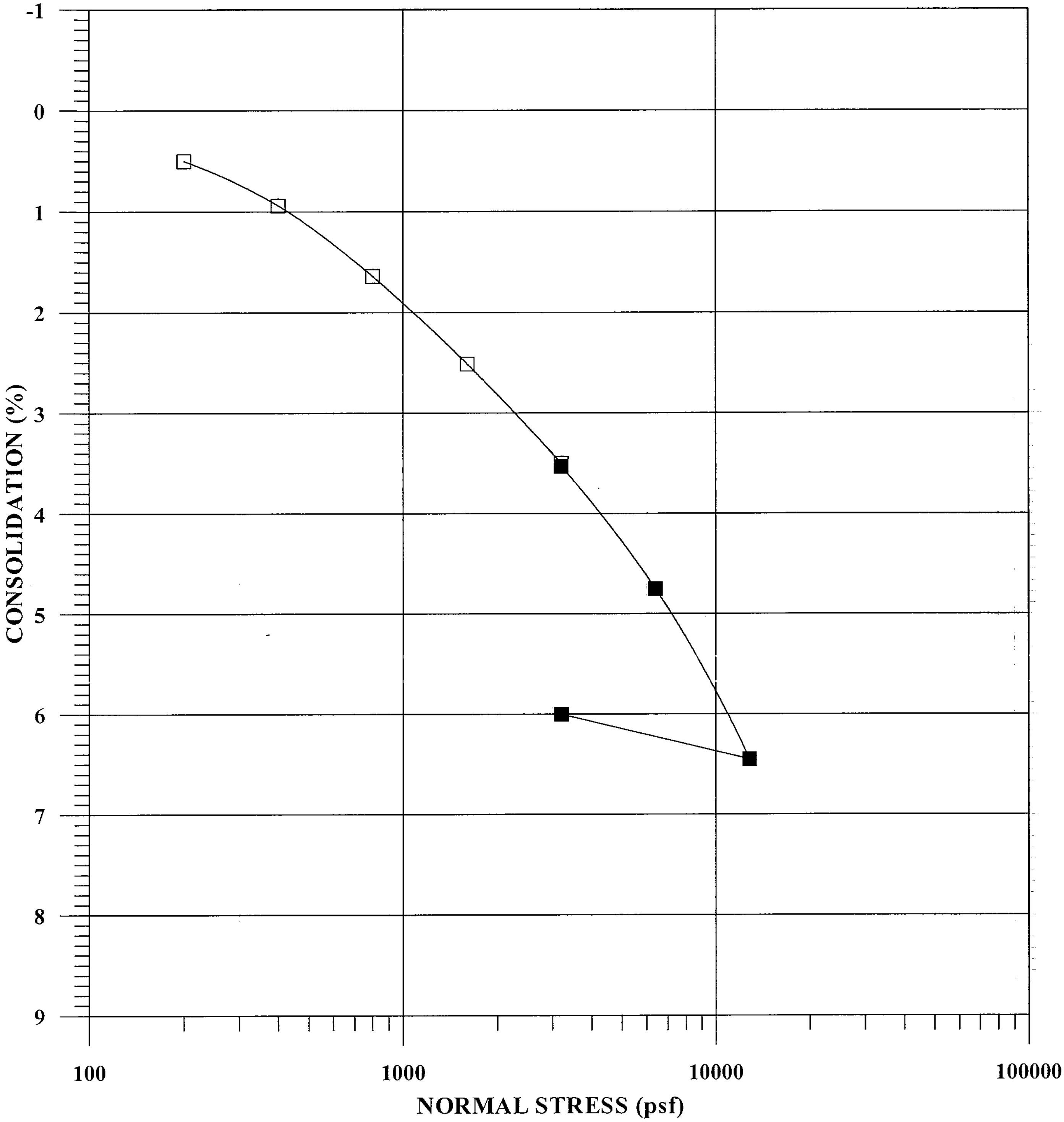
Sample Location:	HS-4	Initial Dry Density (pcf):	112.9	
Sample Depth:	5'	Initial Moisture Content (%):	15.2	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	17.7	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-19



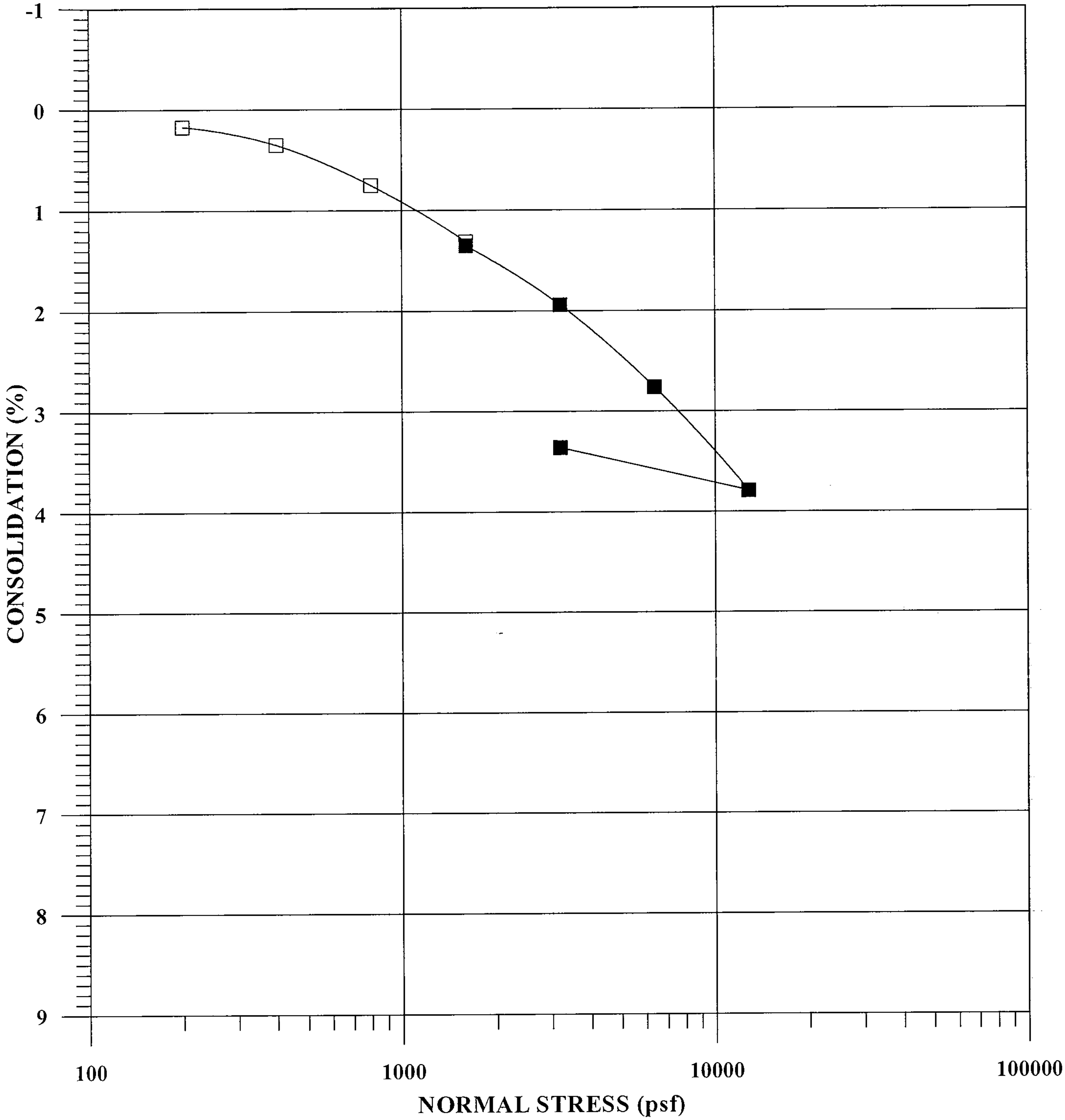
Sample Location:	HS-4	Initial Dry Density (pcf):	108.5	
Sample Depth:	15'	Initial Moisture Content (%):	18.1	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	19.7	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-20



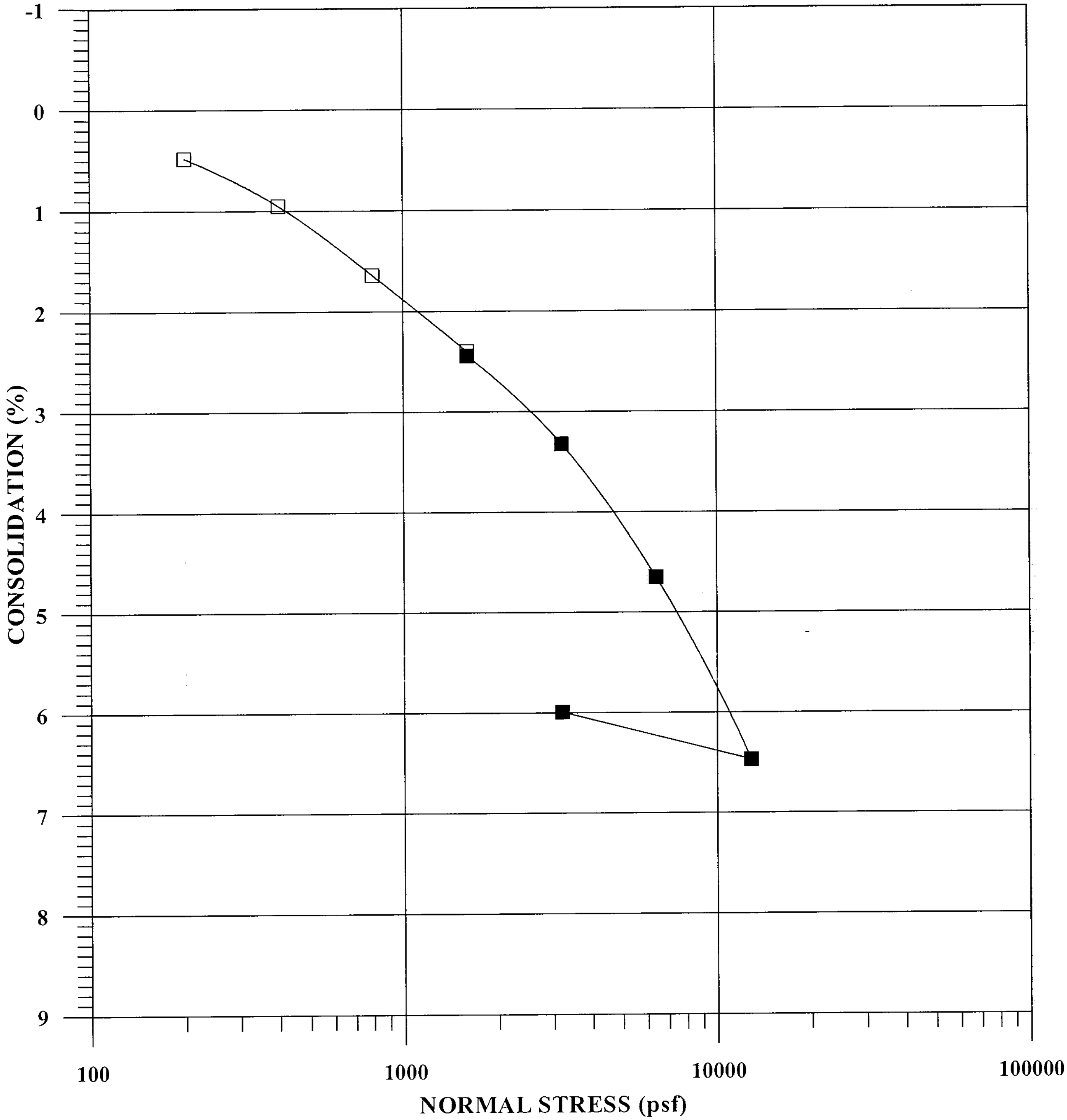
Sample Location:	HS-4	Initial Dry Density (pcf):	107.7	
Sample Depth:	20'	Initial Moisture Content (%):	17.6	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	18.6	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-21



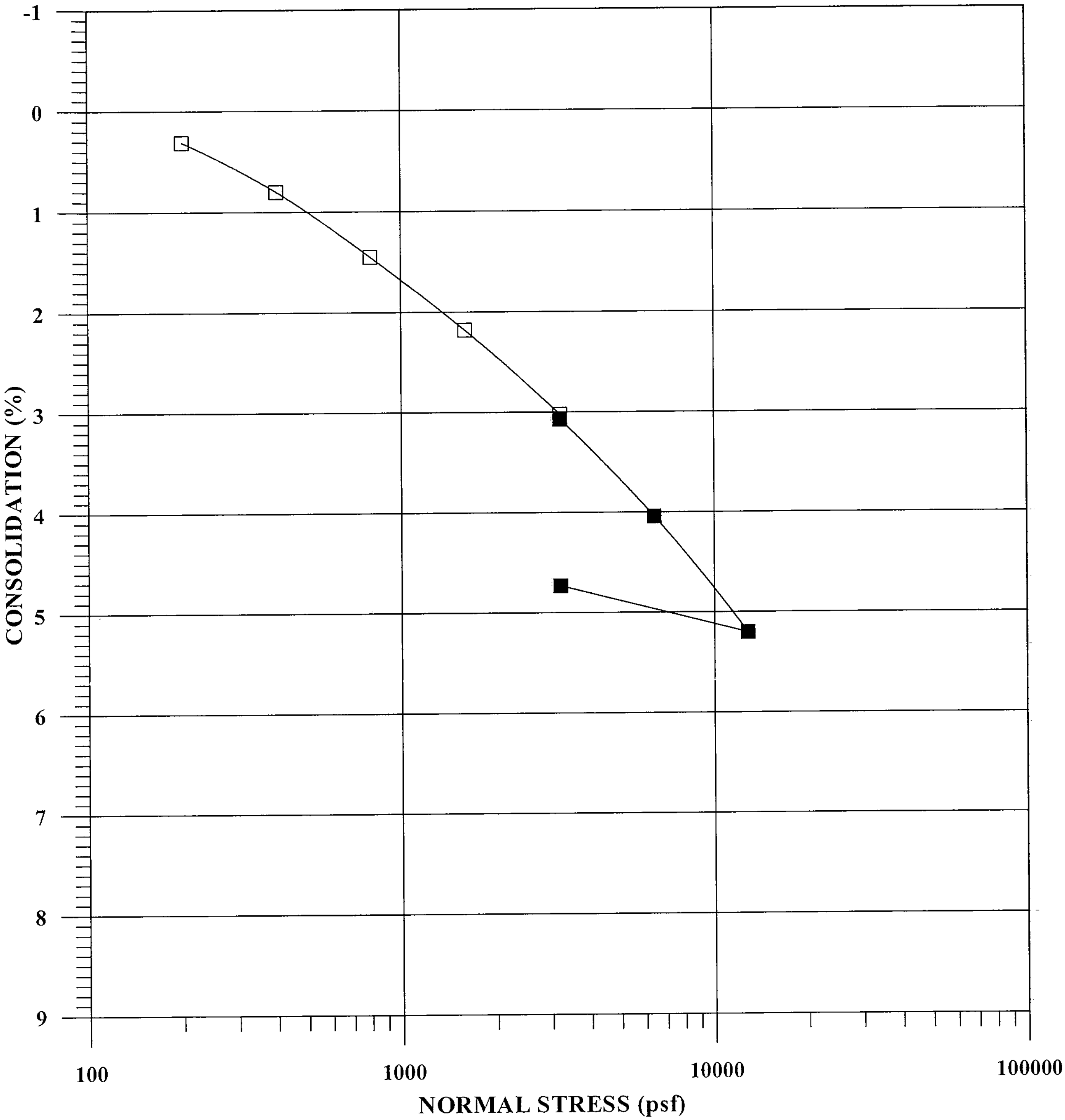
Sample Location:	HS-5	Initial Dry Density (pcf):	111.0	
Sample Depth:	10'	Initial Moisture Content (%):	16.0	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	18.7	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-22



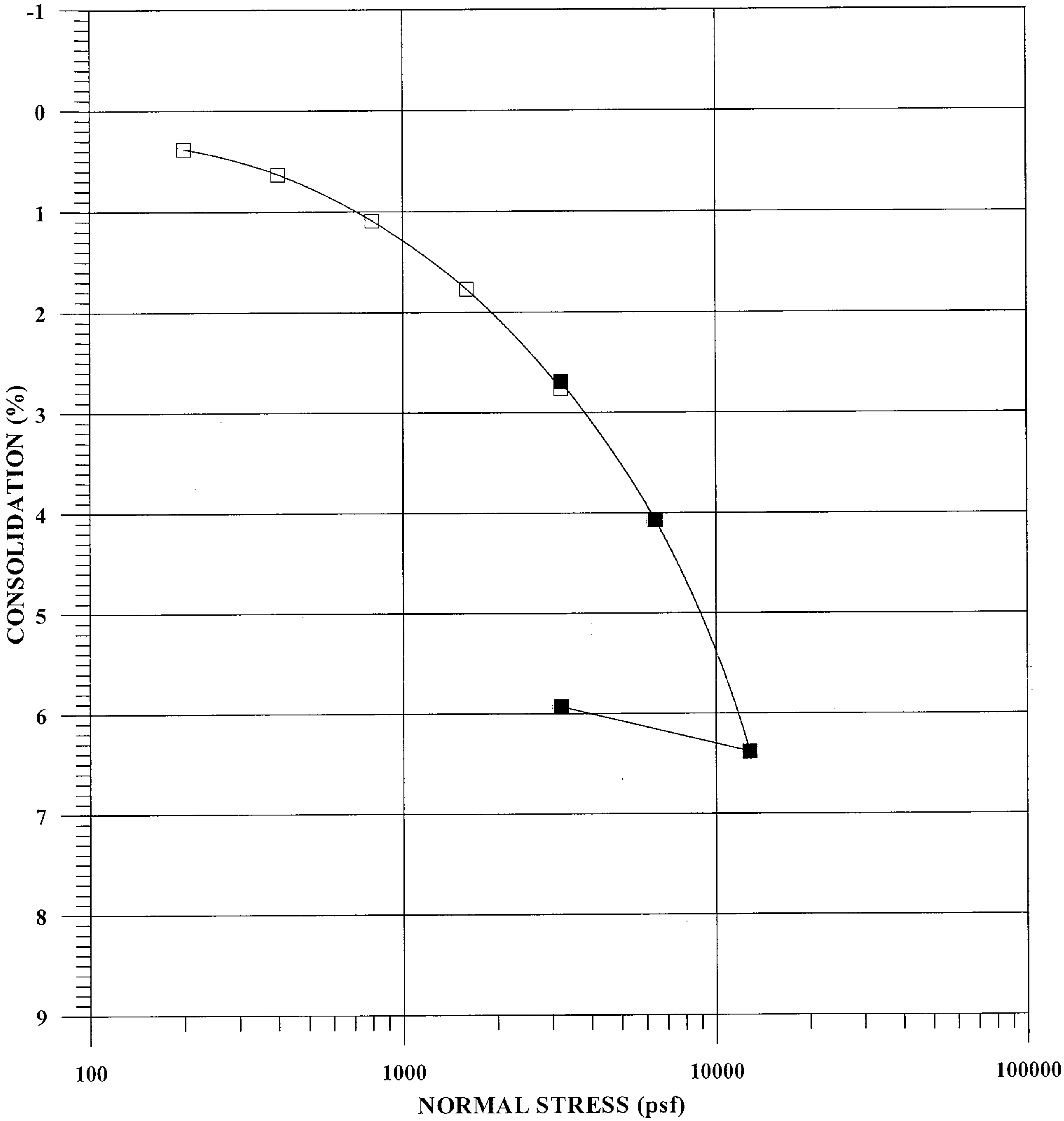
Sample Location:	HS-5	Initial Dry Density (pcf):	106.8	
Sample Depth:	15'	Initial Moisture Content (%):	18.2	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	20.9	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-23



Sample Location:	HS-7	Initial Dry Density (pcf):	118.6	
Sample Depth:	5'	Initial Moisture Content (%):	13.9	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	15.9	

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	CONSOLIDATION TEST RESULTS	Job No: 1489.00 Figure No: B-24



Sample Location:	HS-7	Initial Dry Density (pcf):	104.1	
Sample Depth:	10'	Initial Moisture Content (%):	17.1	**Legend** □ Field Moisture ■ Saturated
Classification:		Final Moisture Content (%):	21.2	

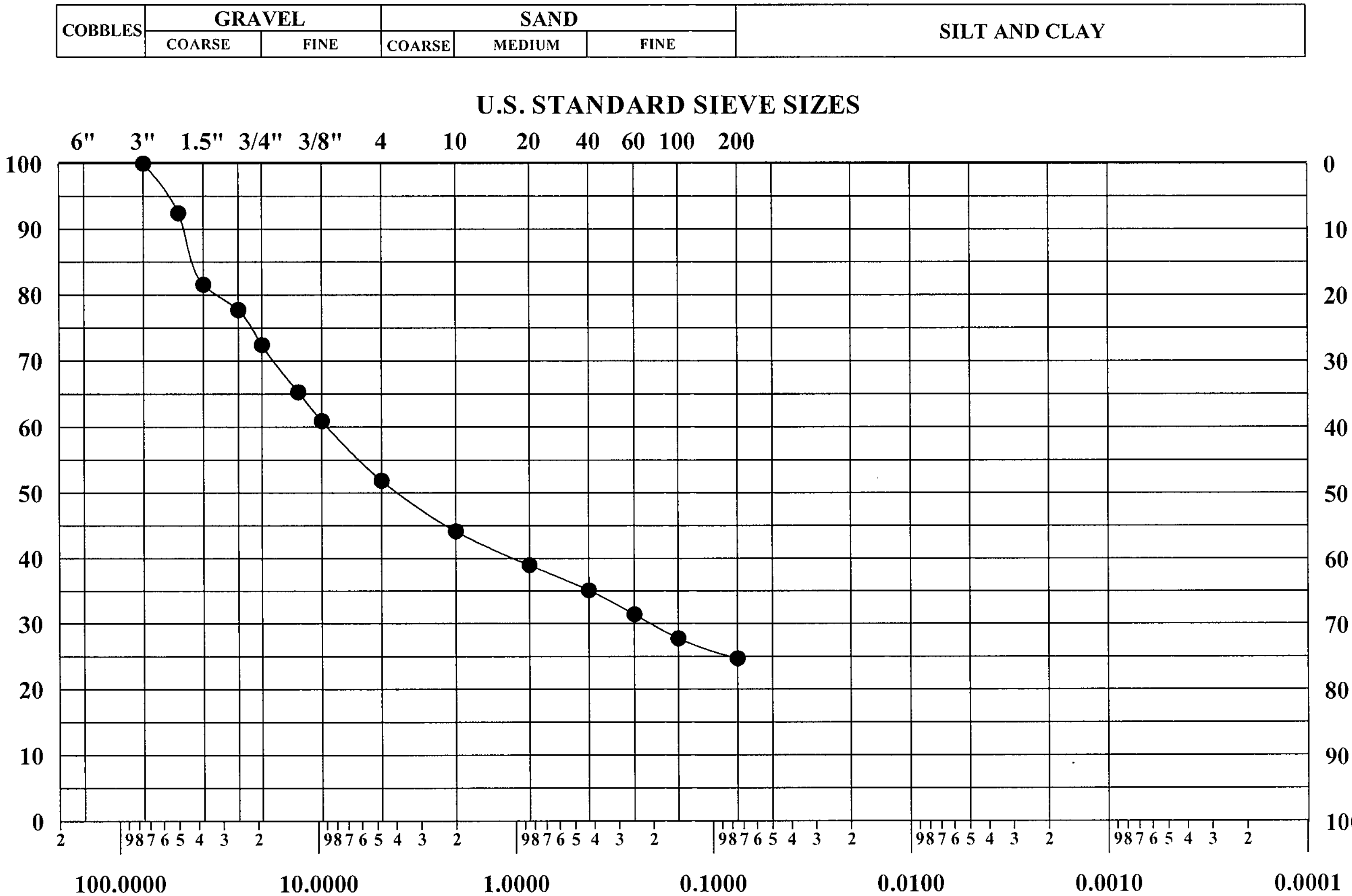
ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**CONSOLIDATION TEST RESULTS**	**Job No: 1489.00** **Figure No: B-25**

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GRAIN SIZE DISTRIBUTION

Job No: 1489.00
Plate No: B-26

UNIFIED SOIL CLASSIFICATION



LOCATION	DEPTH	SYMBOL	LL	PI	CLASSIFICATION
B-1	16'	●——●			Silty Sand & Gravel (GM-SM)
		■- - - -■			
		▲—— - - ▲			
		◆- - —— ◆			

APPENDIX C

EXPLORATION LOGS AND LABORATORY TEST RESULTS BY OTHERS

CLIENT: Comstock	PROJECT: Proposed 65 Lots	W.O.: 8988
LOCATION: Santa Paula	ELEVATION: 630'±	DATE: 1/26/05
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 18"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Terrace Deposits:	
5	P/1/2	C	X	18.5	110.7	Strong brown SILT with clay, stiff, moist.	
10	NR	C	X			Subrounded coarse gravel to boulder in yellow brown coarse grained SAND with silt and clay matrix, matrix supported, medium dense to dense, moist.	
15	2	C	X	12.0	120.8	Yellow brown subrounded coarse gravel to boulder in yellow brown coarse grained SAND with silt and clay matrix, matrix suported, medium dense to dense, wet.	
20						San Pedro Formation: Light olive brown fine grained sandy SILTSTONE, hard, moist.	
	4/5/7	C	X	16.4	114.7		
25						Total Depth - 23' Groundwater at 20' Caving from 16-20'	
30							
35							
40							
45							

ADDITIONAL COMMENTS: C = Modified California Sampler

SUBSURFACE DATA

LOG OF BORING GWV2

CLIENT: Comstock	PROJECT: Santa Paula Hospital	W.O.: 8988
LOCATION: Santa Paula	ELEVATION: 615'±	DATE: 1/27/05
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 18"

	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Terrace Deposits: Strong brown GRAVEL and SILT, poorly consolidated, friable, moist. @1' - Strong brown SILT with clay, stiff, moist.	
5	P/1/1	C		17.9	110.2	Strong brown SILT with clay, stiff, moist, porous, trace roots.	
10	P/P/1	C		20.0	109.4	Strong brown SILT with clay, stiff, moist, porous.	
15						Grades to yellow brown GRAVEL with sand and silt, 50% subrounded coarse gravel to boulder (up to 24" diameter), well graded, poorly consolidated, friable, dense, moist (boulders make up 15-20% of clasts). Boulders predominant (greater than 24" diameter), clasts imbricated to the south, local channels (gravel to cobble).	@17' Channel N80W/4SW
20	2/2/3	C		18.8	111.4	Scoured contact (20.8'-21.5'). San Pedro Formation: Olive SILTSTONE with thinly bedded friable coarse grained SANDSTONE and gravel to cobble CONGLOMERATE interbeds, hard, moist. Black (manganese?) staining along bedding.	@21' scoured contact N50E/17NW
25						Olive thinly bedded coarse grained SANDSTONE interbed within olive SILTSTONE.	@25' B N66E/44SE
30	4/12	C		15.2	119.6	Groundwater seeping along thinly bedded conglomerate bed within olive SILTSTONE, hard, moist. @30' - Olive SILTSTONE, hard, moist.	
35							
40	6/9/10	C		15.5	119.1	Groundwater seeping along bedding contact between olive SILTSTONE, hard, moist, and friable coarse grained SANDSTONE, caving at 40'.	@40' B N65E/52SE
45							

ADDITIONAL COMMENTS: C = Modified California Sampler

SURFACE DATA

LOG OF BORING GWV2

CLIENT: Comstock	PROJECT: Santa Paula Hospital	W.O.: 8988
LOCATION: Santa Paula	ELEVATION: 615'±	DATE: 1/27/05
RIG TYPE: 24" Bucket	HAMMER WEIGHTS: Kelly Bar Weights	DROP: 18"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
40							
45							
50	20-6"	C		18.2	113.8	Olive SILTSTONE, hard, moist.	
55						Total Depth - 55' Groundwater at 29' and 40' Minor caving from 13-17'; caving at and below 40' Downhole logged to 40'	
60							
65							
70							
75							
80							
85							

ADDITIONAL COMMENTS: C = Modified California Sampler

Logged by: SBS

Geolabs-Westlake Village

CLIENT: Comstock	PROJECT: Santa Paula Memorial Hopsital	W.O.: 8988
LOCATION: Santa Paula	ELEVATION: 610'±	DATE: 2/2/05
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Agricultural Fill: Dark brown silty clayey SAND, moist, loose.	
	2/7/10	C		19.2		@2.5' - Terrace Deposits: Dark brown silty CLAY, moist, stiff to medium stiff, trace coarse sands and fine gravels.	
5	7/14/23	C		18.1	112.9	Dark brown to dark reddish brown silty CLAY, moist, hard, trace light yellowish brown sand veins.	
	9/14/30	C		17.6	111.5	@7.5' - Dark reddish brown clayey SILT to silty CLAY, moist, hard.	
10	8/22/34	C		17.6	111.2	Dark reddish brown clayey SILT with trace very fine sands, moist, hard.	
15	6/18/25		S			Dark reddish brown clayey SILT with trace very fine sands, moist, hard.	
20	12/20/22	C		20.2	17.3	Dark reddish brown very fine grained sandy SILT, moist to very moist, hard.	
25	9/14/20		S			Dark reddish brown very fine grained sandy SILT with trace clay, very moist, hard.	
30	50-3"	C				No recovery, orange brown fine grained SAND, cobble in shoe.	Logged from cuttings
35	50-1"		S			No recovery, light brown to tan fine grained silty SAND, moist, very dense.	Logged from cuttings
40						Total Depth - 35' No groundwater No caving	
45							

ADDITIONAL COMMENTS:

Blows per 6"
C = Modified California Sampler
S = Standard Penetration Test

CLIENT: Comstock	PROJECT: Santa Paula Memorial Hospital	W.O.: 8988
LOCATION: Santa Paula	ELEVATION: 612'±	DATE: 2/2/05
RIG TYPE: 8" HSA	HAMMER WEIGHTS: 140 lbs.	DROP: 30"

Depth	N	U	B	M	DD	DESCRIPTION	ATTITUDES
0						Agricultural Fill: Dark brown silty clayey SAND, moist, loose.	
	13/22/40	C		16.4	112.2	@2.5' - Terrace Deposits: Dark brown silty CLAY, moist, hard, trace coarse sands and fine gravels.	
5	11/20/27	C		17.2	112.6	Dark brown silty CLAY with trace red fine sand lenses, moist, hard, trace coarse sands.	
	12/20/26	C		17.4	111.0	@7.5' - Dark brown to dark reddish brown very fine grained sandy SILT with clay, moist to very moist, hard, continued trace red sand lenses and trace fine gravels.	
10	9/12/20	C		19.3	105.7	Dark reddish brown very fine grained sandy SILT, very moist to wet, hard, trace fine gravels, some sandier lenses with increased moisture.	
15	6/11/13		S			Dark reddish brown interbedded fine grained sandy SILT and silty SAND, very moist to wet, hard/medium dense, sand layers saturated.	
20	10/13/18	C		21.4	106.2	Dark reddish brown interbedded fine grained sandy SILT and silty SAND, very moist to wet, hard/medium dense, sand layers saturated.	
25	9/12/30		S			Medium brown silty fine grained SAND, grading to light brown very fine grained silty SAND, moist to very moist, dense.	
	50-3"		S			Light brown silty SAND with gravels and cobbles, moist, very dense.	
30							
35						Total Depth - 28', refusal on rock Perched groundwater from 10-20' No caving	
40							
45							

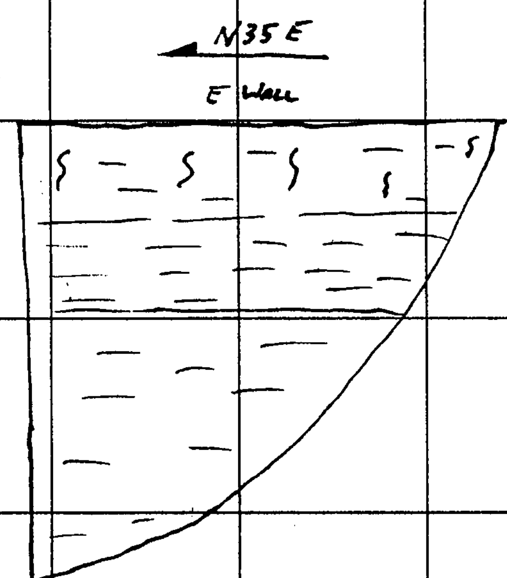
ADDITIONAL COMMENTS:

Blows per 6"
C = Modified California Sampler
S = Standard Penetration Test

LOG OF EXCAVATION Trench No. TP1	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 2	Terrace Deposits: Dark brown SILT, very stiff, damp, abundant 1-2mm pores, abundant roothairs.	
2 - 4.5	Grades to dark brown SILT with clay, hard, moist, weakly cemented, no visible pores.	
4.5 - 12	Grades to dark brown clayey SILT, very stiff, moist, abundant pinhole pores, easier to excavate.	
	Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

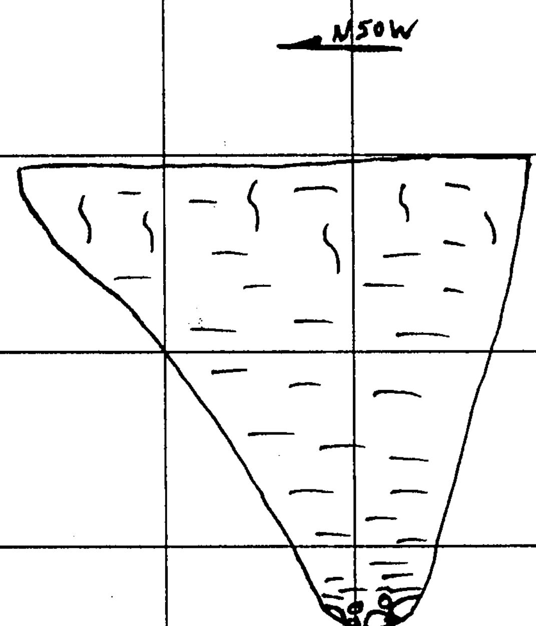


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP1

LOG OF EXCAVATION Trench No. TP2	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 2	Terrace Deposits: Dark yellowish brown silty SAND, dense, damp, abundant 1-2mm pores, abundant roots and rootlets.	
2 - 5	Grades to dark yellowish brown SILT with clay, hard, moist, weakly cemented, abundant pinhole pores.	
5 - 12	Grades to dark yellow brown CLAY with silt, very stiff, moist, abundant pinhole pores, easy to excavate.	
@12'	Subrounded coarse cobble-boulder clasts of arkosic sandstone.	
	Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

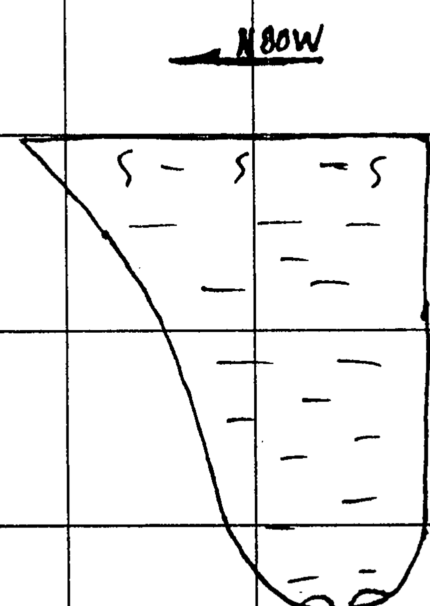


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP2

LOG OF EXCAVATION Trench No. TP3	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 2	Terrace Deposits: Dark yellowish brown clayey SILT, very stiff, damp, occasional rootlets and roothairs.	
2 - 12	Grades to dark yellowish brown clayey SILT, very stiff, moist, abundant pinhole pores, easy to excavate.	
@12'	Subrounded coarse cobble-boulder clasts of arkosic sandstone.	
	Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

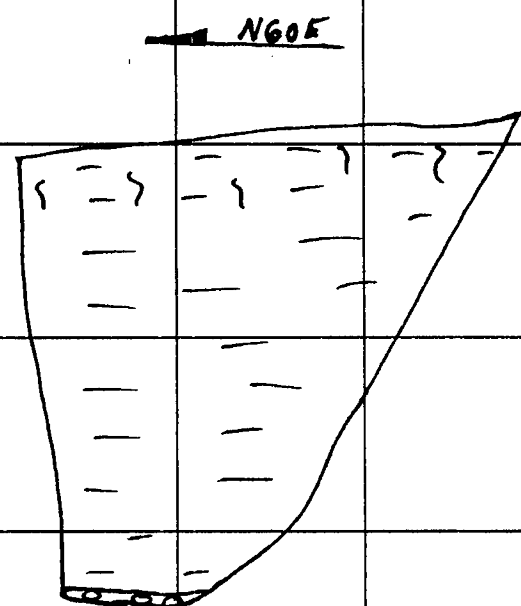


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP3

LOG OF EXCAVATION Trench No. TP4	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 2	<u>Terrace Deposits</u>: Dark yellowish brown clayey SILT, very stiff, damp, abundant 1-2mm pores, abundant roots and rootlets.	
2 - 12	Grades to dark yellowish brown clayey SILT, very stiff, moist, abundant pinhole pores, weakly cemented from 2-4'.	
@12'	Grades to light olive brown fine grained sandy SILT with subrounded cobble-boulder clasts.	
	Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

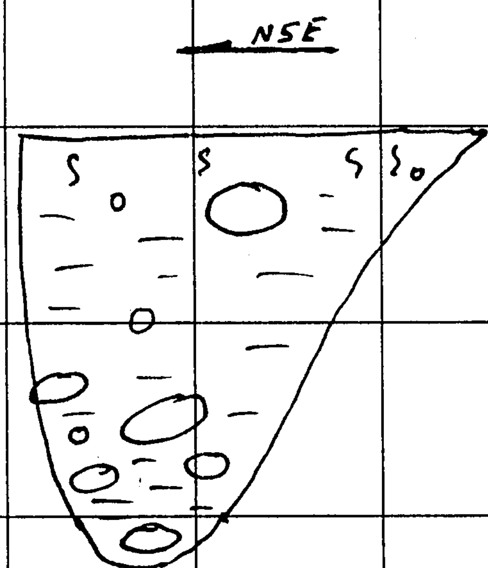


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP4

LOG OF EXCAVATION Trench No. TP5	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 6	Terrace Deposits: Dark yellowish brown very clayey SILT with subrounded coarse cobble-boulder clasts (of arkosic sandstone), very stiff, damp to 2', moist below, abundant roothairs and 1-2mm pores to 2' depth, abundant pinhole pores below.	
6 - 12	Grades to subrounded cobble-boulder clasts (of angular sandstone) in dark yellowish brown clayey SILT, very stiff, moist, abundant pinhole pores.	
	Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

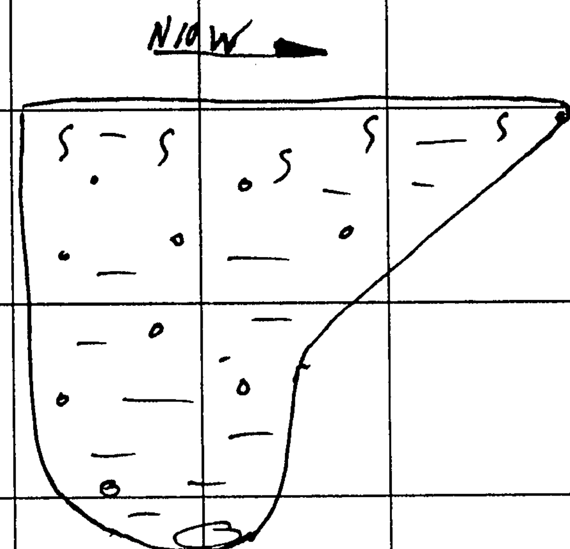


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP5

LOG OF EXCAVATION Trench No. TP6	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 12	Terrace Deposits: Dark yellowish brown clayey SILT with occasional (less than 5%) subrounded fine to coarse gravel clasts (of arkosic sandstone), very stiff, moist, abundant 1-2mm pores and rootlets to 2', abundant pinhole pores below. @10' - Occasional (5-10%) subrounded cobbles in dark yellowish brown clayey SILT. Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

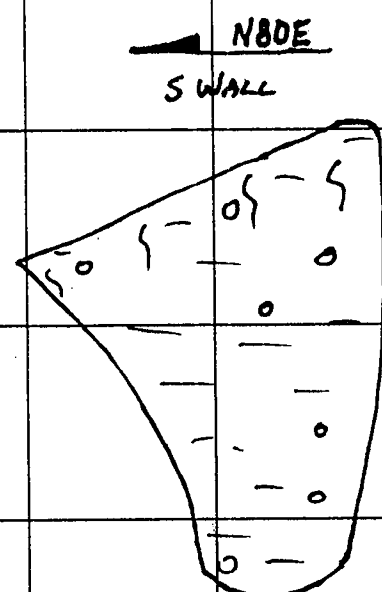


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP6

LOG OF EXCAVATION Trench No. TP7	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 12	Terrace Deposits: Dark yellowish brown clayey SILT with occasional (less than 5%) subrounded fine gravel to coarse cobble clasts (of tan arkosic sandstone and olive gray graywacke), very stiff, moist, abundant pinhole pores, abundant rootlets, roothairs, and 1-2mm pores to 2'. Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

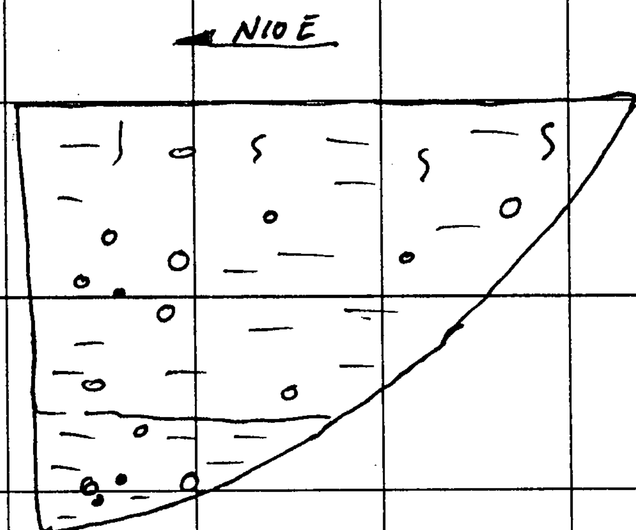


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP7

LOG OF EXCAVATION Trench No. TP8	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 8	Terrace Deposits: Dark yellowish brown clayey SILT with occasional (less than 5%) subrounded fine gravel to cobble clasts (of arkosic sandstone and graywacke), abundant pinhole pores, abundant rootlets and 1-2mm pores to 2' depth, easy to excavate.	
8 - 12	Grades to light olive brown fine grained sandy SILT with clay and occasional (less than 5%) subrounded fine gravel clasts, abundant pinhole pores, very stiff, moist.	
	Total Depth - 12' No groundwater No caving Backfilled	

Graphic Log

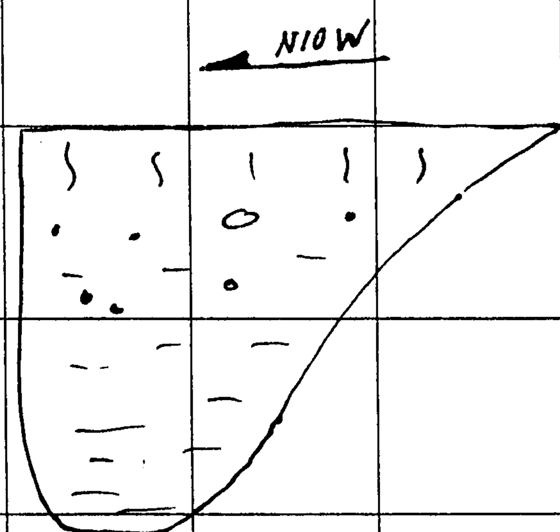


scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP8

LOG OF EXCAVATION Trench No. TP9	Logged By: SBS	Date Excavated: 7/11/05	Client: Comstock Homes

Depth (ft)	Description	Comments
0 - 6	Terrace Deposits: Dark yellowish brown clayey SILT with occasional (5-10%) subround fine to coarse gravel and infrequent (less than 5%) cobble-boulder clasts (of arkosic sandstone and graywacke), very stiff, moist, abundant pinhole pores, abundant 1-2mm pores and roots and rootlets to 2'.	
6 - 11	Grades to dark yellowish brown fine grained sandy SILT, very stiff, moist, abundant pinhole pores.	
	Total Depth - 11' No groundwater No caving Backfilled	

Graphic Log



scale 1" = 5'	GEOLABS-WESTLAKE VILLAGE	W.O. 8988	PLATE TP9

Laboratory Test Summary

W.O. 8988

Excavation	Depth	Geology	Sample Description	ST	w	DD	S	Max	Opt	EI	LL	PI	e	n	WD	SD	BD	Consol	Shear
GWV1 (TD= 23 ft, No GW)	5	Terrace Deposits	SILT with clay	(U)	18.5	111	97	121	12	23			0.51	0.34	131	133	70.1	C-GWV1.5	
	5	Terrace Deposits	SILT with clay	(U)	18.5	111	97	121	12	23			0.51	0.34	131	133	70.1	C-GWV1.5	
	16	Terrace Deposits	›arse SAND w/ gravel to boulder clas	(U)	12	121	84						0.38	0.28	135	142	79.3		
	23	San Pedro Formation	fine sandy SILTSTONE	(U)	16.4	115	96						0.46	0.31	134	135	72.8		
GWV2 (TD= 55 ft, No GW)	5	Terrace Deposits	SILT with clay	(U)	17.9	110	93						0.52	0.34	130	133	70.8	C-GWV2.5	
	10	Terrace Deposits	SILT with clay	(U)	20	109	100						0.53	0.35	131	131	68.3	C-GWV2.10	
	10	Terrace Deposits	SILT with clay	(U)	20	109	100						0.53	0.35	131	131	68.3	C-GWV2.10	
	21	San Pedro Formation	edded SLTS, SS, and CONGLOMEI	(U)	18.8	111	100						0.50	0.34	132	132	69.7		
	30	San Pedro Formation	SILTSTONE	(U)	15.2	120	100						0.4	0.28	138	137	74.5		
	40	San Pedro Formation	coarse SANDSTONE	(U)	15.5	119	100						0.40	0.29	138	137	74.1		
	50	San Pedro Formation	SILTSTONE	(U)	18.2	114	100						0.47	0.32	135	133	70.5		
GWV3 (TD= 35 ft, No GW)	2.5	Terrace Deposits	silty CLAY	(U)	19.2														
	5	Terrace Deposits	silty CLAY	(U)	18.1	113	100						0.48	0.32	133	133	70.6	C-GWV3.5	
	7.5	Terrace Deposits	clayey SILT to silty CLAY	(U)	17.6	112	94						0.50	0.33	131	134	71.2	C-GWV3.7.5	
	10	Terrace Deposits	clayey SILT with trace fine sand	(U)	17.6	111	94						0.50	0.33	131	134	71.2		
	20	Terrace Deposits	fine sandy SILT	(U)	20.2	107	97						0.56	0.36	129	130	68		
GWV4 (TD= 28 ft, No GW)	2.5	Terrace Deposits	silty CLAY	(U)	16.4	112	90						0.49	0.33	131	135	72.8		
	5	Terrace Deposits	silty CLAY with trace fine sand	(U)	17.2	113	95						0.49	0.33	132	134	71.8		S-GWV4.5
	7.5	Terrace Deposits	fine sandy SILT with clay	(U)	17.4	110	90						0.52	0.34	129	134	71.5		
	10	Terrace Deposits	fine sandy SILT	(U)	19.3	106	89						0.58	0.37	126	132	69.1	C-GWV4.10	
	20	Terrace Deposits	terbedded sandy SILT and silty SAN	(U)	21.4	106	100						0.57	0.37	129	129	66.6	C-GWV4.20	
	20	Terrace Deposits	terbedded sandy SILT and silty SAN	(U)	21.4	106	100						0.57	0.37	129	129	66.6	C-GWV4.20	

GEOLABS-WESTLAKE VILLAGE

Excavation	Depth	Geology	Sample Description	ST	w	DD	S	Max	Opt	EI	LL	PI	e	n	WD	SD	BD	Consol	Shear

LEGEND

Depth = Sample Depth (ft)
ST = Sample Type*
w = Initial Moisture Content (%)
DD = Initial Dry Unit Weight (pcf)
Max = Maximum Dry Unit Weight (pcf)
Opt = Optimum Moisture Content (%)
EI = Expansion Index
S = Degree of Saturation (%)

LL = Liquid Limit
PI = Plasticity Index
e = Void Ratio
n = Porosity
WD = Initial Wet Unit Weight (pcf)
SD = Saturated Unit Weight (pcf)
BD = Bouyant (Submerged) Unit Weight (pcf) - Assuming water unit weight of 62.4 pcf

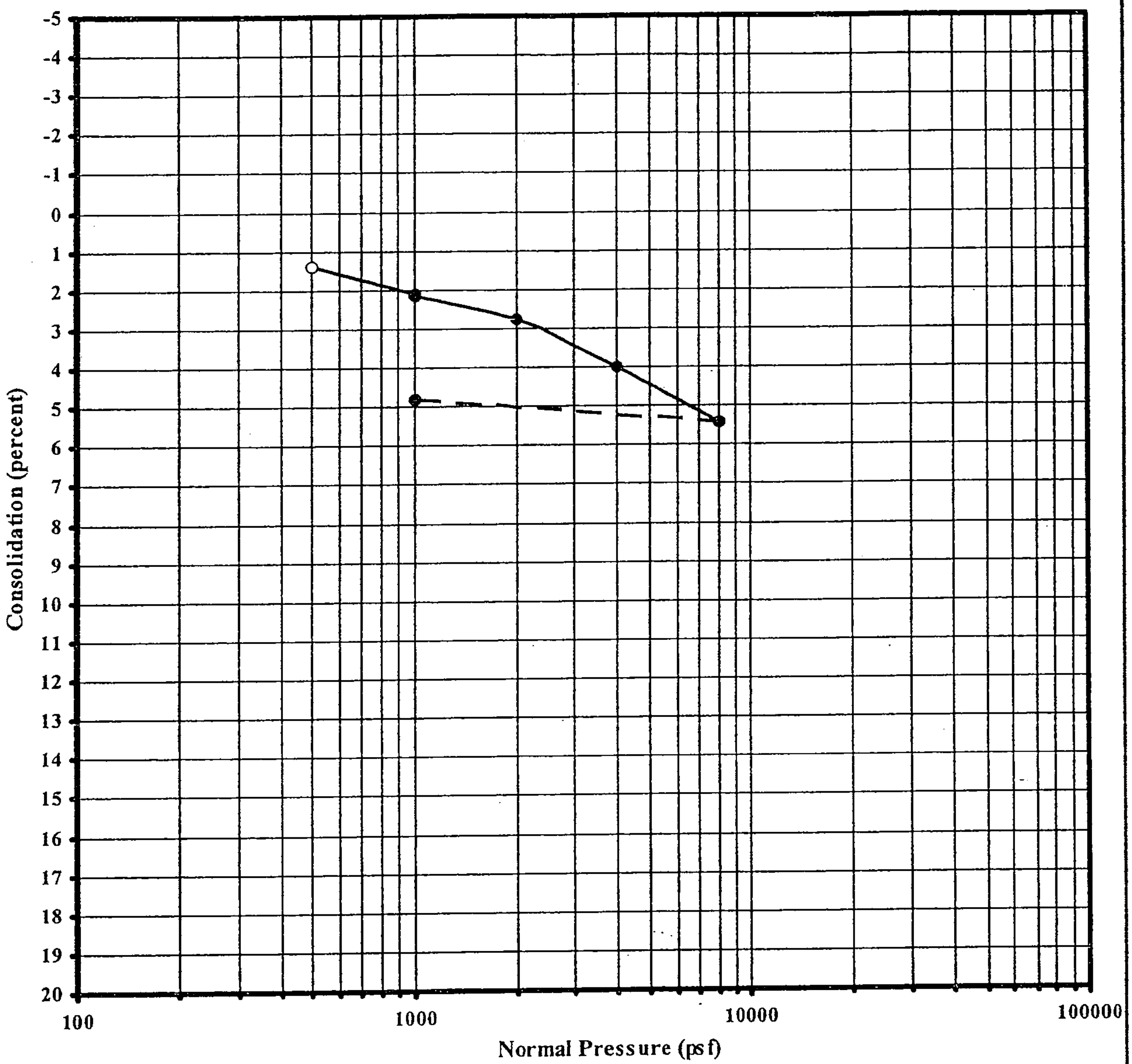
* Sample Types: (U) = relatively Undisturbed; (S) = SPT; (B) = Bulk

Consol = Consolidation Test Diagram (Plate No.)
Shear = Shear Test Diagram (Plate No.)

GEOLABS-WESTLAKE VILLAGE

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV1

Sample Depth: 5 ft.

Initial Moisture: 18.5 %

Init. Dry Density: 110.7 pcf

Geologic Unit: Terrace Deposits

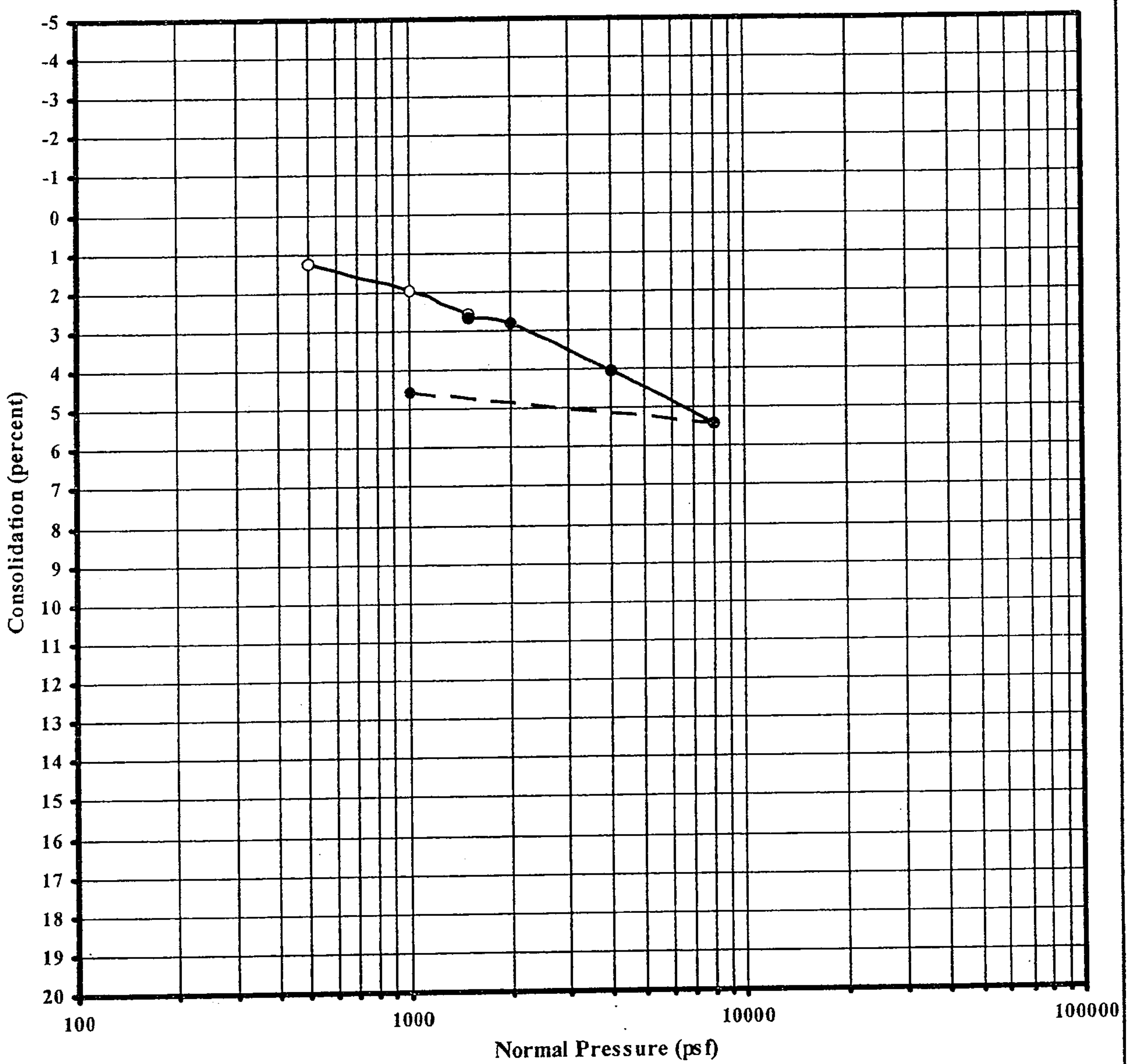
Material: SILT with clay

Santa Paula Hospital, Comstock Homes

W.O. 8988

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1500 psf

Sample Location: GWV1

Sample Depth: 5 ft.

Initial Moisture: 18.5 %

Init. Dry Density: 110.7 pcf

Geologic Unit: Terrace Deposits

Material: SILT with clay

GEOLABS-WESTLAKE VILLAGE

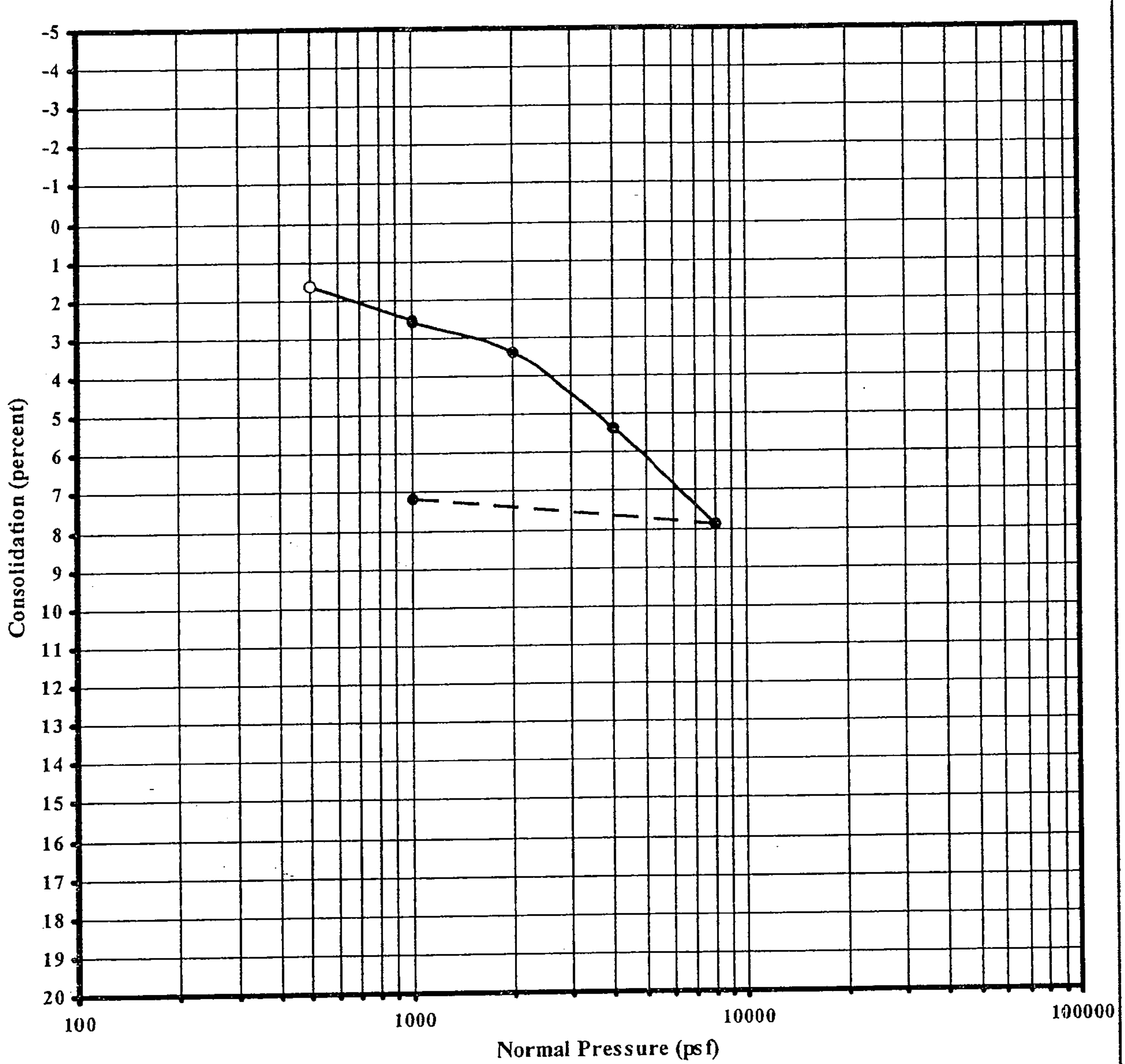
PLATE C-GWV1.5

Santa Paula Hospital, Comstock Homes

W.O. 8988

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV2

Sample Depth: 5 ft.

Initial Moisture: 17.9 %

Init. Dry Density: 110.2 pcf

Geologic Unit: Terrace Deposits

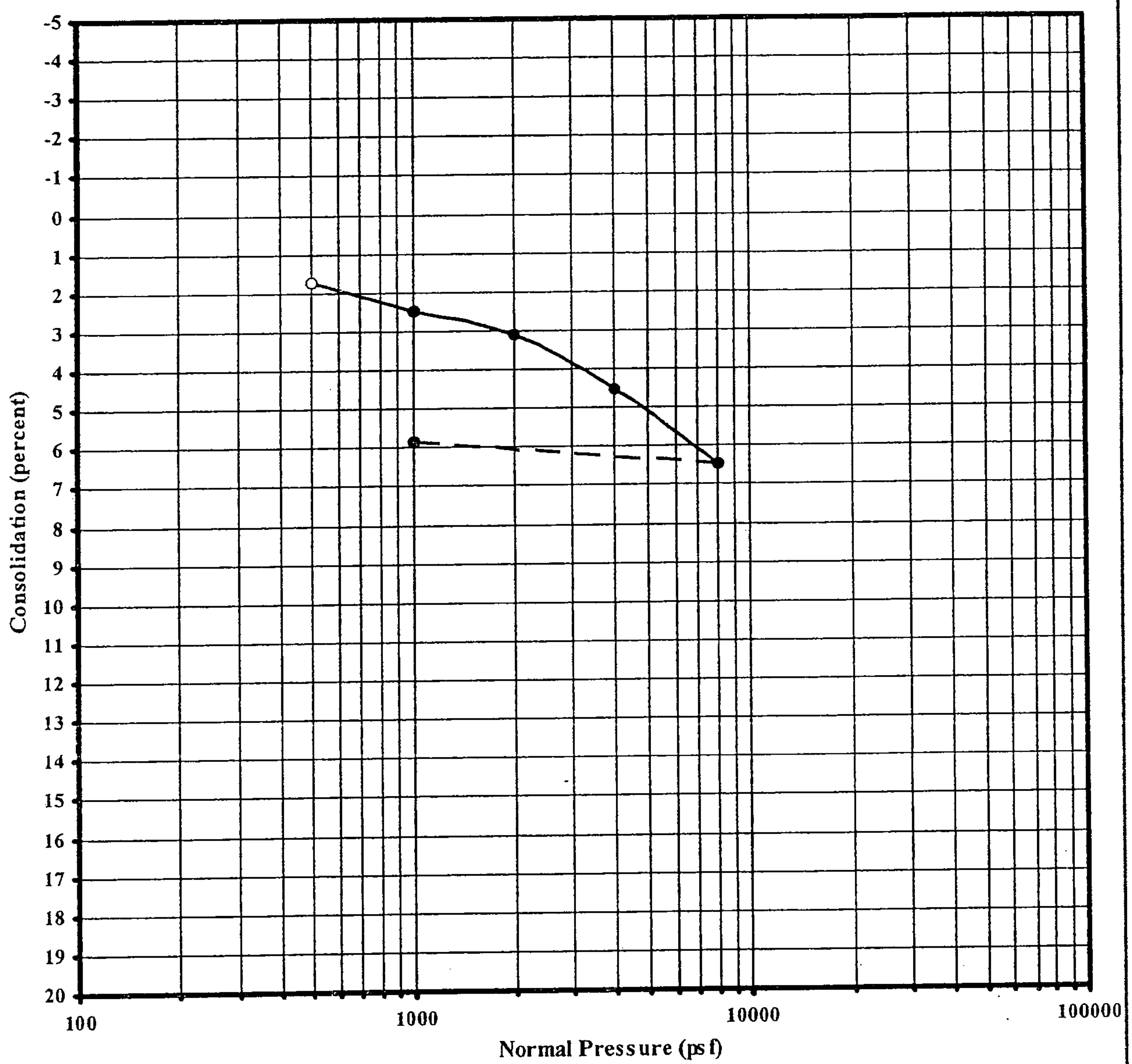
Material: SILT with clay

GEOLABS-WESTLAKE VILLAGE

PLATE C-GWV2.5

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV2

Sample Depth: 10 ft.

Initial Moisture: 20 %

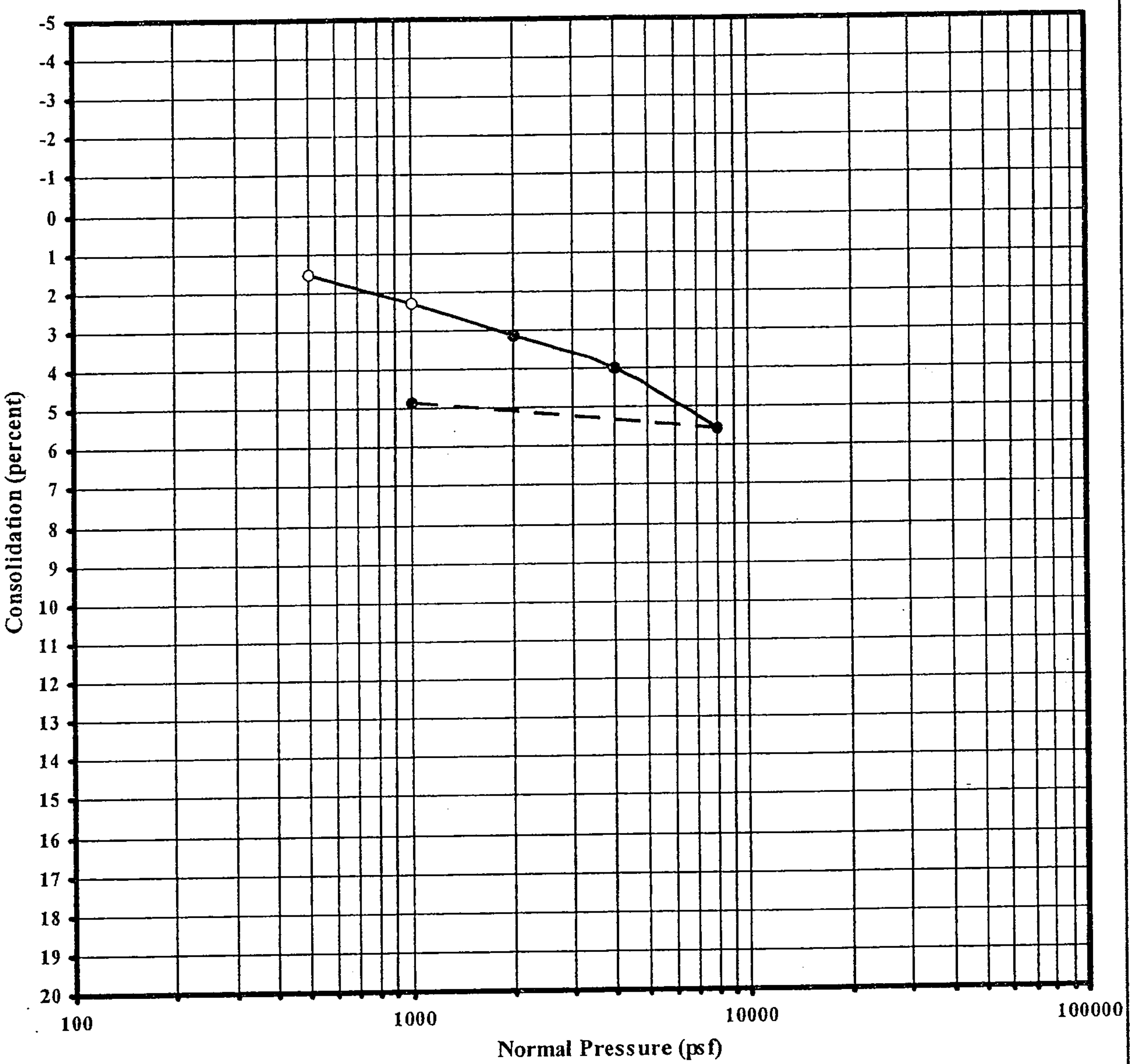
Init. Dry Density: 109.4 pcf

Geologic Unit: Terrace Deposits

Material: SILT with clay

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 2000 psf

Sample Location: GWV2

Sample Depth: 10 ft.

Initial Moisture: 20 %

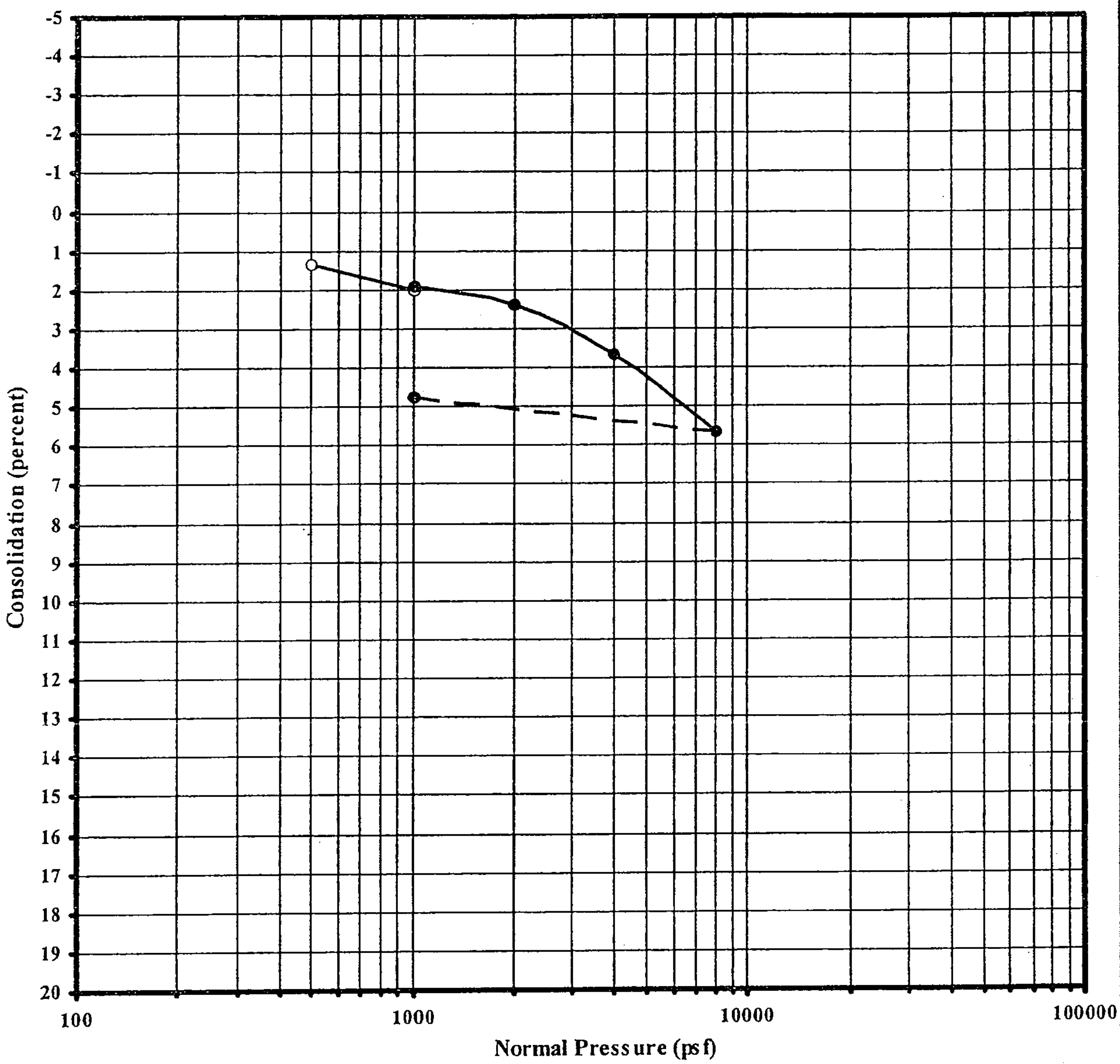
Init. Dry Density: 109.4 pcf

Geologic Unit: Terrace Deposits

Material: SILT with clay

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV3

Sample Depth: 5 ft.

Initial Moisture: 18.1 %

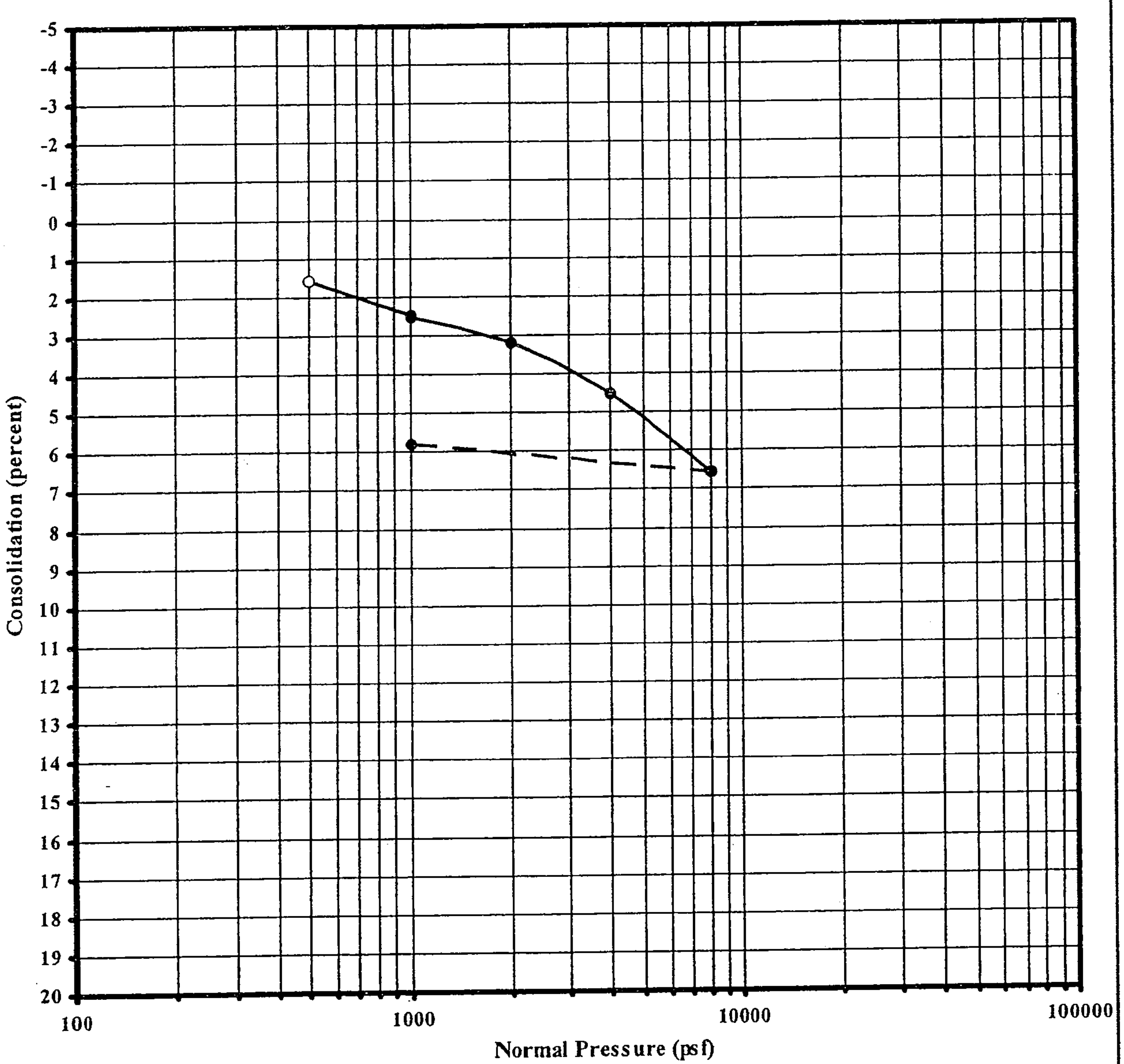
Init. Dry Density: 112.9 pcf

Geologic Unit: Terrace Deposits

Material: silty CLAY

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV3

Sample Depth: 7.5 ft.

Initial Moisture: 17.6 %

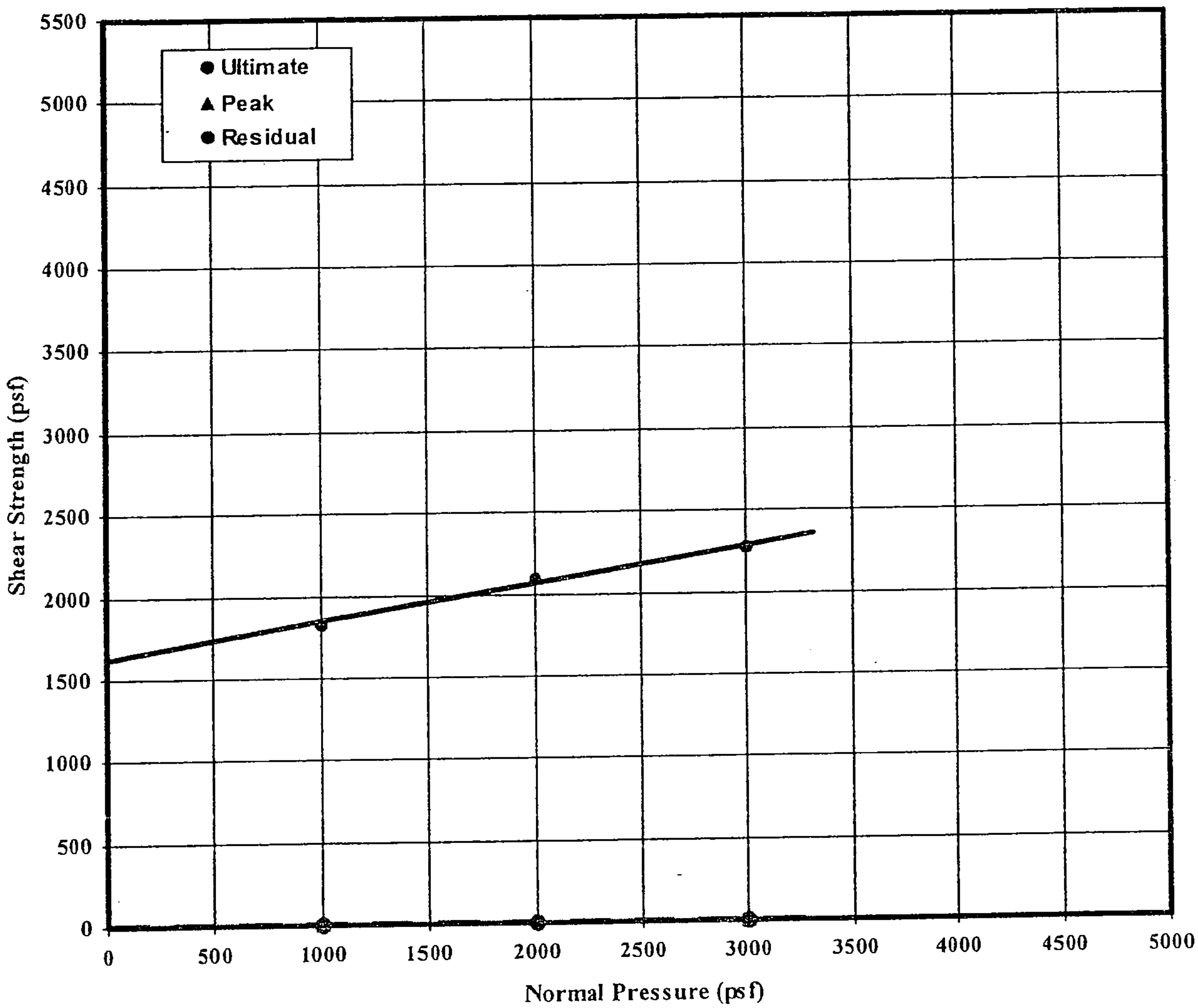
Init. Dry Density: 111.5 pcf

Geologic Unit: Terrace Deposits

Material: clayey SILT to silty CLAY

SHEAR TEST RESULTS

Undisturbed Sample

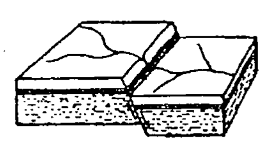


	Friction Angle	Cohesion
Ultimate Shear Strength:	12 deg	1625 psf
Peak Shear Strength:		
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

Dry Density:	112.6 pcf
Moisture:	19.7 %

Sample Location: GWV4
Sample Depth: 5 ft.
Geologic Unit: Terrace Deposits
Material: silty CLAY with trace fine sand

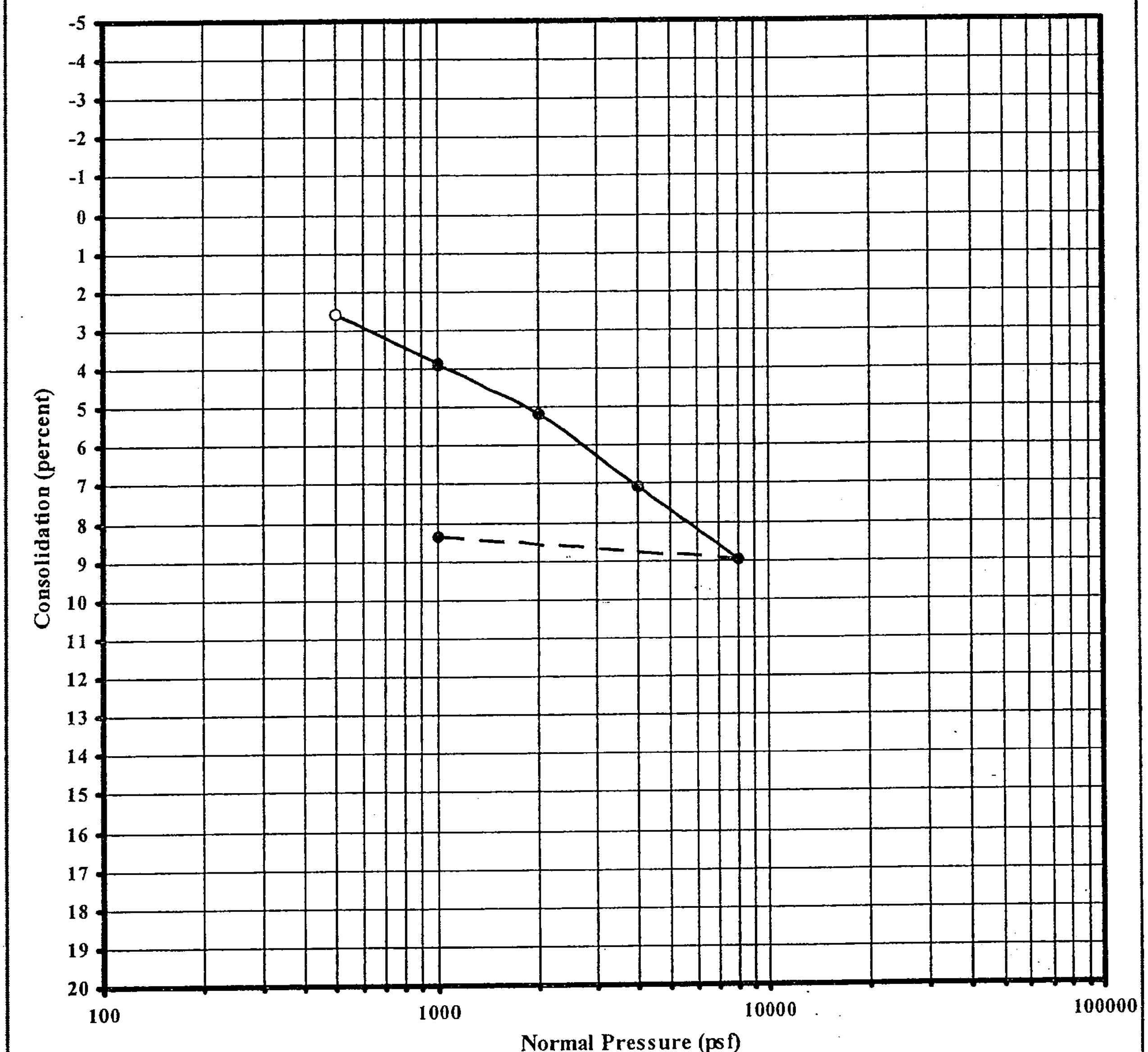


GEOLABS-WESTLAKE VILLAGE

PLATE S-GWV4.5

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV4

Sample Depth: 10 ft.

Initial Moisture: 19.3 %

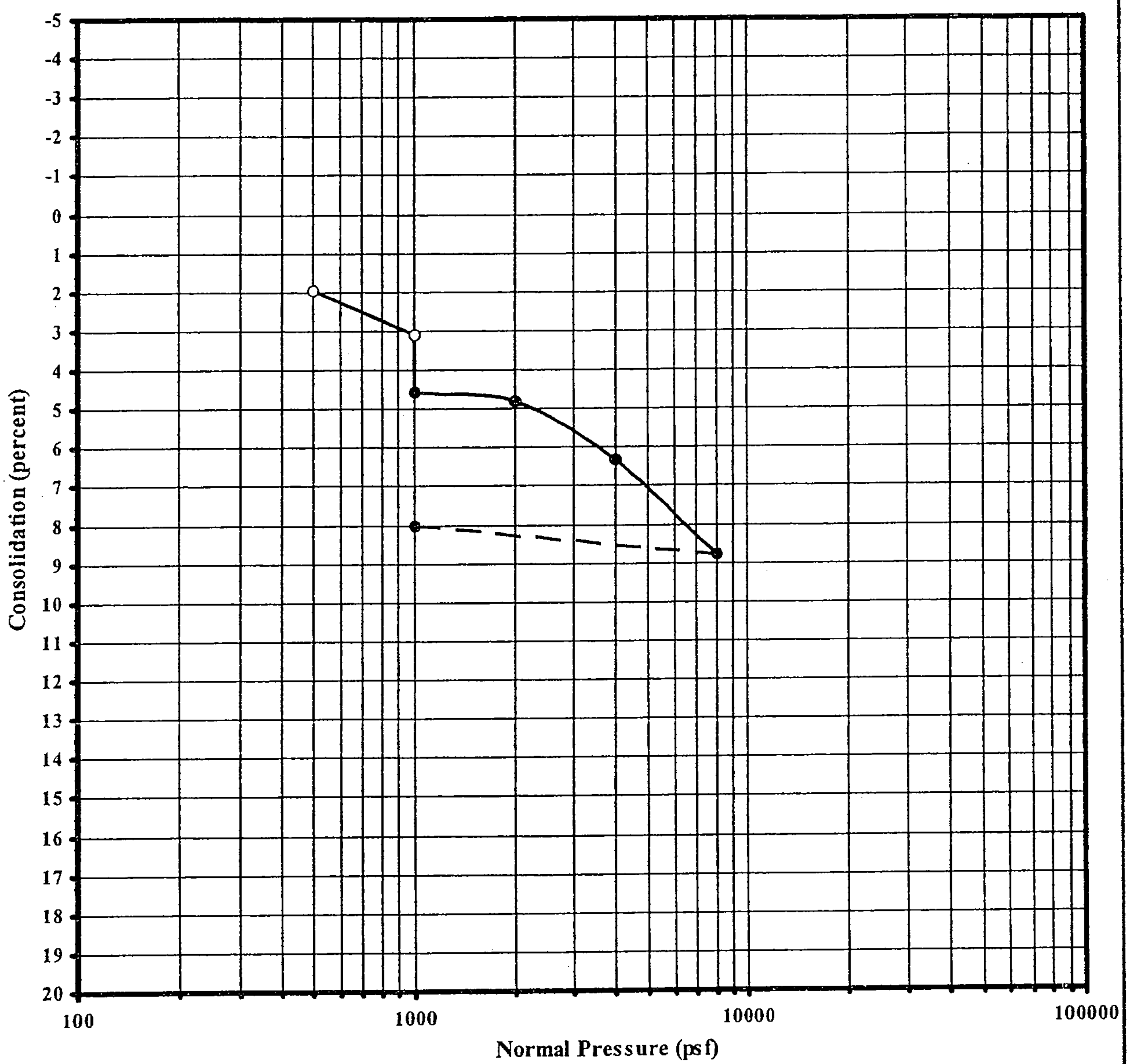
Init. Dry Density: 105.7 pcf

Geologic Unit: Terrace Deposits

Material: fine sandy SILT

CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 1000 psf

Sample Location: GWV4

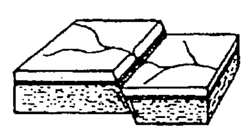
Sample Depth: 20 ft.

Initial Moisture: 21.4 %

Init. Dry Density: 106.2 pcf

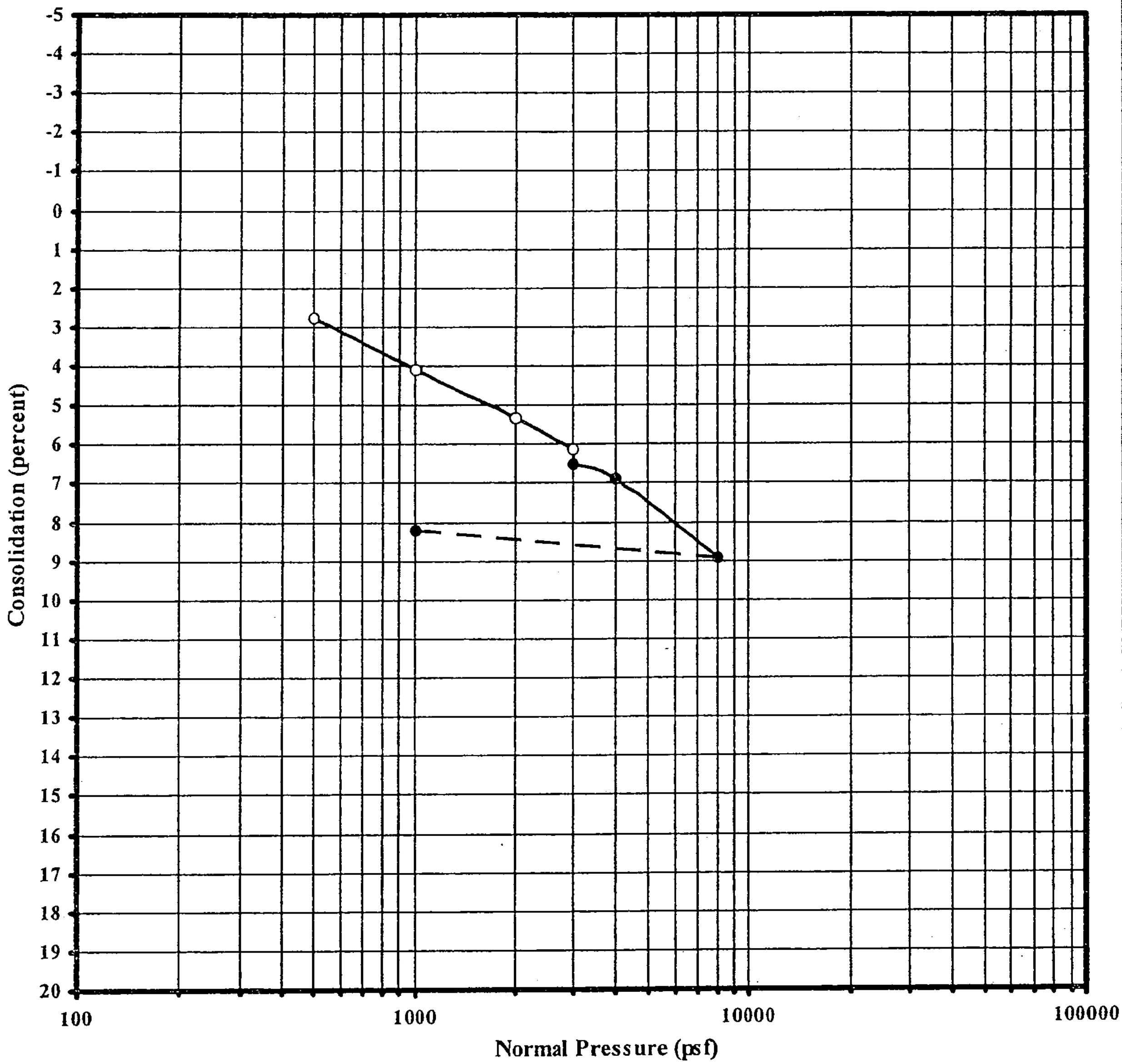
Geologic Unit: Terrace Deposits

Material: Interbedded sandy SILT and silty SA



CONSOLIDATION RESULTS

Undisturbed Sample



Sample Inundated At Normal Pressure of 3000 psf

Sample Location: GWV4

Sample Depth: 20 ft.

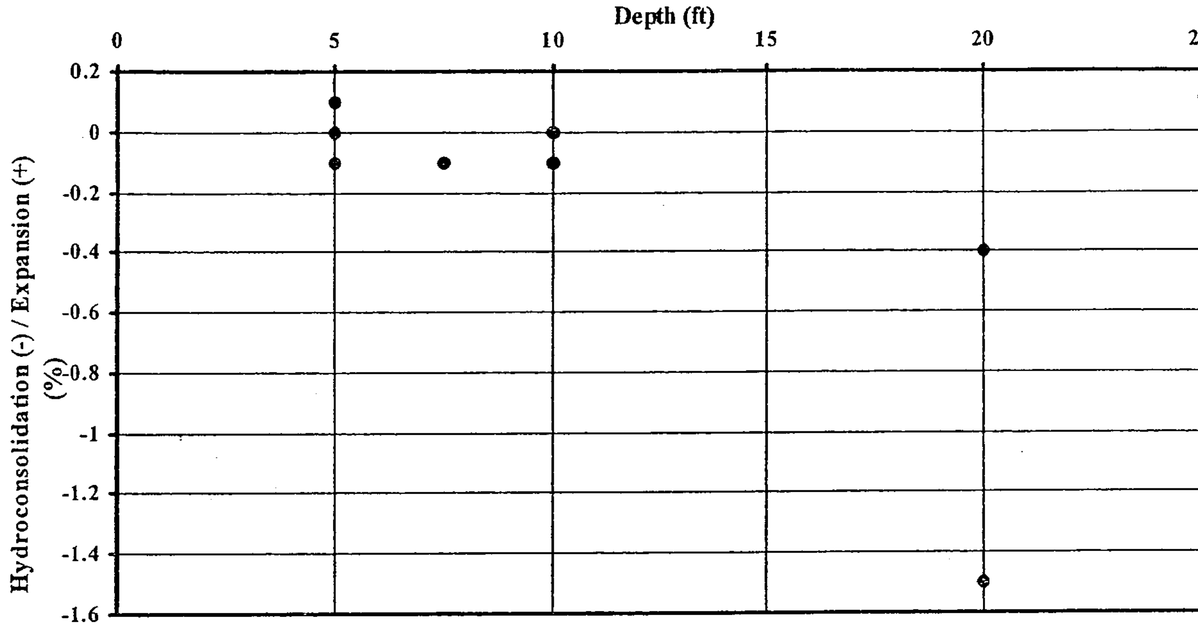
Initial Moisture: 21.4 %

Init. Dry Density: 106.2 pcf

Geologic Unit: Terrace Deposits

Material: Interbedded sandy SILT and silty SA

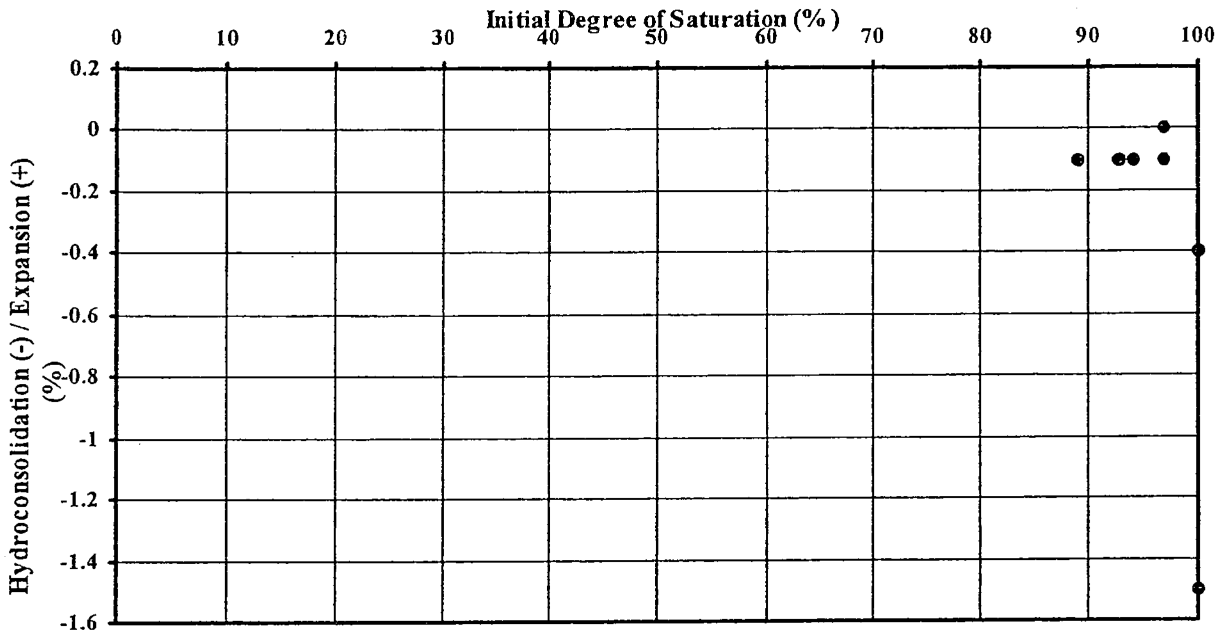
HYDROCONSOLIDATION/EXPANSION VS. DEPTH
Terrace Deposits



Excavation	Depth (ft)	Field DD (pcf)	M (%)	e	S (%)	Volume Change (%)	Terrace Deposits Material
GWV1	5	110.7	18.5	0.51	96.9	-0.1	SILT with clay
GWV1	5	110.7	18.5	0.51	96.9	-0.1	SILT with clay
GWV1	5	110.7	18.5	0.51	96.9	0.0	SILT with clay
GWV1	5	110.7	18.5	0.51	96.9	0.0	SILT with clay
GWV2	5	110.2	17.9	0.52	92.7	-0.1	SILT with clay
GWV3	5	112.9	18.1	0.48	101.1	0.1	silty CLAY
GWV3	7.5	111.5	17.6	0.50	94.1	-0.1	clayey SILT to silty CLAY
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV4	10	105.7	19.3	0.58	88.9	-0.1	fine sandy SILT
GWV4	20	106.2	21.4	0.57	100	-1.5	Interbedded sandy SILT and silty SAND
GWV4	20	106.2	21.4	0.57	100	-1.5	Interbedded sandy SILT and silty SAND
GWV4	20	106.2	21.4	0.57	100	-0.4	Interbedded sandy SILT and silty SAND
GWV4	20	106.2	21.4	0.57	100	-0.4	Interbedded sandy SILT and silty SAND

DD = Field Dry Density, M = Field Moisture, e = initial void ratio, S = initial degree of saturation, Volume Change = percent of hydroconsolidation(-) or expansion (+)

HYDROCONSOLIDATION/EXPANSION VS. SATURATION
Terrace Deposits

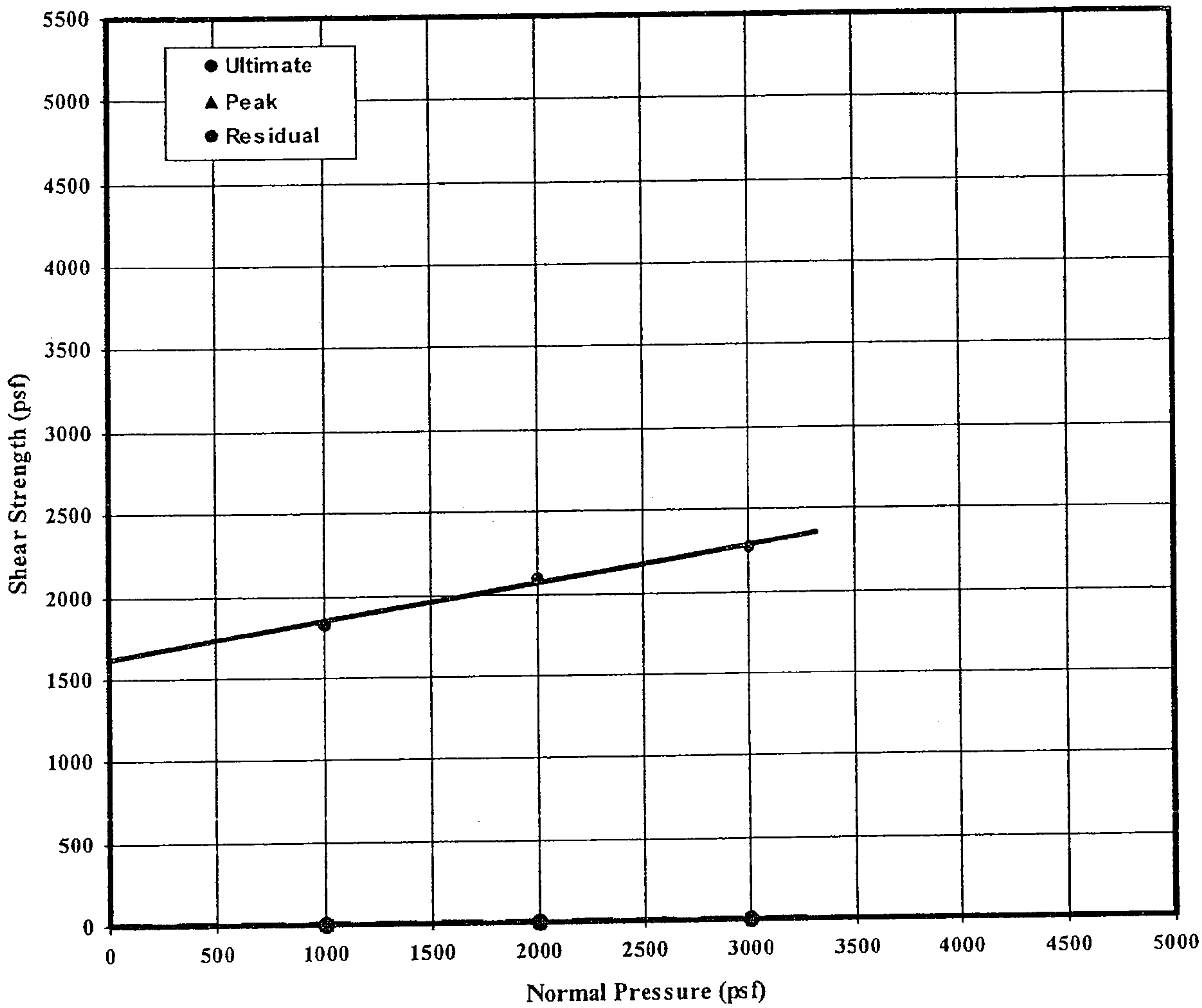


Excavation	Depth (ft)	Field DD (pcf)	M (%)	e	S (%)	Volume Change (%)	Terrace Deposits Material
GWV1	5	110.7	18.5	0.51	96.9	-0.1	SILT with clay
GWV1	5	110.7	18.5	0.51	96.9	-0.1	SILT with clay
GWV1	5	110.7	18.5	0.51	96.9	0.0	SILT with clay
GWV1	5	110.7	18.5	0.51	96.9	0.0	SILT with clay
GWV2	5	110.2	17.9	0.52	92.7	-0.1	SILT with clay
GWV3	5	112.9	18.1	0.48	101.1	0.1	silty CLAY
GWV3	7.5	111.5	17.6	0.50	94.1	-0.1	clayey SILT to silty CLAY
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV2	10	109.4	20.0	0.53	101	0.0	SILT with clay
GWV4	10	105.7	19.3	0.58	88.9	-0.1	fine sandy SILT
GWV4	20	106.2	21.4	0.57	100	-1.5	Interbedded sandy SILT and silty SAND
GWV4	20	106.2	21.4	0.57	100	-1.5	Interbedded sandy SILT and silty SAND
GWV4	20	106.2	21.4	0.57	100	-0.4	Interbedded sandy SILT and silty SAND
GWV4	20	106.2	21.4	0.57	100	-0.4	Interbedded sandy SILT and silty SAND

DD = Field Dry Density, M = Field Moisture, e = initial void ratio, S = initial degree of saturation, Volume Change = percent of hydroconsolidation(-) or expansion (+)

SHEAR TEST RESULTS

Undisturbed Sample



	Friction Angle	Cohesion
Ultimate Shear Strength:	12 deg	1625 psf
Peak Shear Strength:		
Residual Shear Strength:		

Displacement Rate: 0.01 in/min

Dry Density:	112.6 pcf
Moisture:	19.7 %

Sample Location: GWV4
Sample Depth: 5 ft.
Geologic Unit: Terrace Deposits
Material: silty CLAY with trace fine sand

GEOLABS-WESTLAKE VILLAGE

PLATE S-GWV4.5

M. J. Schiff & Associates, Inc.
Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711
Phone: (909) 626-0967 Fax: (909) 626-3316
E-mail lab@mjschiff.com
website: mjschiff.com

Table 1 - Laboratory Tests on Soil Samples

Geolabs
Santa Paula Hosp.
Your #8988, MJS&A #05-0185LAB
17-Feb-05

Sample ID			B1 @ 5'
Resistivity		**Units**	
as-received		ohm-cm	20,000
saturated		ohm-cm	2,000
pH			7.9
Electrical Conductivity		mS/cm	0.13
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	40
magnesium	Mg^{2+}	mg/kg	27
sodium	Na^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	95
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	na
nitrate	NO_3^{1-}	mg/kg	na
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

Earth Systems Southern California
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: 1
PROJECT NAME: Assisted Living Facility - S.P. Mem. Hosp.
PROJECT NUMBER: VT-22251-02
BORING LOCATION: Per Plan

DRILLING DATE: July 6, 2000
DRILLING METHOD: 6 " Hollowstem Auger
DRILL: Mobile Drill B-80
LOGGED BY: P. Boales

VERTICAL DEPTH (feet)	SAMPLE TYPE: Bulk	SAMPLE TYPE: SPT	SAMPLE TYPE: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0–5	X		■	11/16		ML/CL	108.8	17.8	Very fine sandy silt and clay with some organics in upper few feet, very stiff, moist, dark yellowish brown (Soil/Terrace Deposits)
5			■	8/12		ML/CL	104.4	16.6	
10			■	6/11		CL	106.8	18.3	Very fine sandy silty clay, stiff, very moist, dark yellowish brown (Soil/Terrace Deposits)
15			■	45/50		GM	--	--	Gravels and cobbles in clayey silt matrix, very dense, moist, moderate yellowish brown (Terrace Deposits)
20									TOTAL DEPTH: 17 Feet (Refusal in Cobbles)
									No Groundwater Encountered
25									
30									
35									
40									

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

Earth Systems Southern California

1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: 2
PROJECT NAME: Assisted Living Facility - S.P. Mem. Hosp.
PROJECT NUMBER: VT-22251-02
BORING LOCATION: Per Plan

DRILLING DATE: July 6, 2000
DRILLING METHOD: 6 " Hollowstem Auger
DRILL: Mobile Drill B-80
LOGGED BY: P. Boales

VERTICAL DEPTH (feet)	SAMPLE TYPE: Bulk	SAMPLE TYPE: SPT	SAMPLE TYPE: Mod Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	X					SM			Silty very fine to fine sand, medium dense, dry, moderate yellowish brown (Soils Disturbed by Agricultural Equipment)
	X		■	12/22		ML/CL	108.2	16.6	
5			■	9/15		ML/CL	110.4	15.6	Very fine sandy silt and clay with minor organics in upper few feet, very stiff, moist, dark yellowish brown (Soil/Terrace Deposits)
10			■	12/13		ML/CL	108.2	15.9	
						GM			Gravels and cobbles in clayey silt matrix, very dense, moist, moderate yellowish brown (Terrace Deposits)
15									**TOTAL DEPTH: 17 Feet (Refusal in Cobbles)**
									No Groundwater Encountered
20									
25									
30									
35									
40									

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

Earth Systems Southern California
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: 3
PROJECT NAME: Assisted Living Facility - S.P. Mem. Hosp.
PROJECT NUMBER: VT-22251-02
BORING LOCATION: Per Plan

DRILLING DATE: July 6, 2000
DRILLING METHOD: 6 " Hollowstem Auger
DRILL: Mobile Drill B-80
LOGGED BY: P. Boales

VERTICAL DEPTH (feet)	SAMPLE TYPE: Bulk	SAMPLE TYPE: SPT	SAMPLE TYPE: Mod Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0						SM			Silty VF-F sand, med. dense, dry, mod. yel. bn. (Disturbed)
				15/17		ML/CL	106.3	16.8	
5				10/18		ML/CL	--	--	Very fine sandy silt and clay with minor organics in upper few feet, very stiff, moist, dark yellowish brown with some pressure faces on sample from 10 feet (Soil/Terrace Deposits)
10				10/14		ML/CL	107.5	19.1	
15				12/14		ML/CL	104.1	19.6	
20				--		GM	--	--	Gravels and cobbles in clayey silt matrix, very dense, moist, moderate yellowish brown (Terrace Deposits) Note: Too rocky to sample
25				23/55		TQs	117.8	16.2	Weathered claystone, hard, slightly moist, mottled light olive gray and light olive brown with carbonate veinlets to about 28 feet, light olive brown (only) without carbonates below about 28 feet (Saugus Formation)
30				25/55		TQs	106.7	19.1	
35				30/50		TQs	108.5	19.1	Very fine sandy siltstone, hard, slightly moist, light olive brown (Saugus Formation)
40						TQs			Silty claystone, hard, slightly moist, massive, light olive gray and light olive brown (Saugus Formation) **LOG CONTINUED ON NEXT PAGE**

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

Earth Systems Southern California
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: 3 (Continued)
PROJECT NAME: Assisted Living Facility - S.P Mem. Hosp.
PROJECT NUMBER: VT22263-01
BORING LOCATION: Per Plan

DRILLING DATE: July 6, 2000
DRILLING METHOD: 6" Hollowstem Auger
DRILL: Mobile Drill B-80
LOGGED BY: P. Boales

VERTICAL DEPTH (feet)	SAMPLE TYPE: Bulk	SAMPLE TYPE: SPT	SAMPLE TYPE: Mod Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
40				25/50		TQs	115.1	16.8	
45				39/65		TQs	115.4	18.1	Silty claystone, hard, slightly moist, massive, light olive gray and light olive brown (Saugus Formation)
50				30/50		TQs	104.0	19.9	Very fine sandy clayey siltstone, hard, thinly bedded with about 35° diip, slightly moist, light olive brown and moderate yellowish brown (Saugus Formation)
55									**TOTAL DEPTH: 51 Feet** **No Groundwater Encountered**
60									
65									
70									
75									
80									

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

Earth Systems Southern California

1731-A Walter Street, Ventura, California 93003
PHONE. (805) 642-6727 FAX: (805) 642-1325

BORING NO: 4
PROJECT NAME: Assisted Living Facility - S.P. Mem. Hosp.
PROJECT NUMBER: VT-22251-02
BORING LOCATION: Per Plan

DRILLING DATE: July 6, 2000
DRILLING METHOD: 6 " Hollowstem Auger
DRILL: Mobile Drill B-80
LOGGED BY: P. Boales

VERTICAL DEPTH (feet)	SAMPLE TYPE Bulk	SAMPLE TYPE SPT	SAMPLE TYPE Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0–6			■	11/15		ML/CL	110.9	16.6	Very fine sandy silt and clay with some organics in upper few feet, very stiff, moist, dusky yellowish brown in upper few feet, dark yellowish brown below (Soil/Terrace Deposits)
			■	9/17		ML/CL	108.6	15.5	
6–19			■	14/16		CL	98.5	17.4	Very fine sandy silty clay, very stiff, moist to very moist, dark yellowish brown (Soil/Terrace Deposits)
			■	14/17		CL	99.8	16.3	
19–26			■	17/17		SM/GM	106.8	16.6	Clayey silty very fine to fine sand with some gravels, and with some cobble layers, dense to very dense, moist, dark yellowish brown (Terrace Deposits)
			■	34/100		SM/GM	102.6	20.5	

TOTAL DEPTH: 26 Feet (Refusal in Cobbles)

No Groundwater Encountered

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

TABULATED TEST RESULTS

REMOLDED SAMPLES

BORING AND DEPTH	1 @ 0-5'	2 @ 0-5'	1 & 2 @ 0-5'
USCS	ML/CL	ML/CL	ML/CL
MAXIMUM DENSITY (pcf)	115.0	115.5	--
OPTIMUM MOISTURE (%)	12.0	11.5	--
COHESION (psf)	250	220	--
ANGLE OF INTERNAL FRICTION	24°	27°	--
EXPANSION INDEX	43	43	--
pH	8.0	7.7	--
SOLUBLE CHLORIDES (mg/Kg)	0	72	--
RESISTIVITY (OHMs/cm)	1,610	530	--
SOLUBLE SULFATES (mg/Kg)	77	230	--
RESISTANCE ("R") VALUE	--	--	13
GRAIN SIZE DISTRIBUTION (%)			
GRAVEL	0.0	0.0	--
SAND	33.6	33.8	--
SILT	36.9	40.2	--
CLAY	29.5	26.0	--

RELATIVELY UNDISTURBED SAMPLES

BORING AND DEPTH	3 @ 2'	3 @ 10'	3 @ 25'	3 @ 35'
USCS	ML/CL	ML/CL	Claystone	Siltstone
IN-PLACE DENSITY (pcf)	106.3	107.5	117.8	108.5
IN-PLACE MOISTURE (%)	16.8	19.1	16.2	19.1
COHESION (psf)	890	610	2,610	360
ANGLE OF INTERNAL FRICTION	35°	25°	35°	41°

BORING AND DEPTH	3 @ 40'	3 @ 50'	4 @ 2'	4 @ 15'
USCS	Claystone	Siltstone	ML/CL	CL
IN-PLACE DENSITY (pcf)	115.1	104.0	110.9	99.8
IN-PLACE MOISTURE (%)	16.8	19.9	16.6	16.3
COHESION (psf)	2,920	1,530	770 !	330
ANGLE OF INTERNAL FRICTION	55°	43°	46°	27°

IN-PLACE DENSITIES

BORING & DEPTH	DRY DENSITY	% MOISTURE	RELATIVE COMPACTION
1 @ 2'	108.8	17.8	95
5'	104.4	16.6	91
10'	106.8	18.3	--
2 @ 2'	108.2	16.6	94
5'	110.4	15.6	96
10'	108.2	15.9	94
3 @ 2'	106.3	16.8	92
10'	107.5	19.1	93
15'	104.1	19.6	91
25'	117.8	16.2	--
30'	106.7	19.1	--
35'	108.5	19.1	--
40'	115.1	16.8	--
45'	115.4	18.1	--
50'	104.0	19.9	--
4 @ 2'	110.9	16.6	96
5'	108.6	15.5	94
10'	98.5	17.4	--
15'	99.8	16.3	--
20'	106.8	16.6	--
25'	102.6	20.5	--

IN-PLACE DENSITIES, MOISTURES, AND SATURATIONS

SG= 2.65

Boring/Test Pit Depth	Moist Unit Weight (pcf)	Dry Unit Weight (pcf)	Moisture Content (%)	Degree of Saturation (%)
B-3 @ 2'	124.2	106.3	16.8	80
B-3 @ 10'	128.0	107.5	19.1	94
B-3 @ 15'	124.5	104.1	19.6	88
B-4 @ 2'	129.3	110.9	16.6	90
B-4 @ 5'	125.4	108.6	15.5	79
B-4 @ 10'	115.6	98.5	17.4	68
B-4 @ 15'	116.1	99.8	16.3	66
B-4 @ 20'	124.5	106.8	16.6	80
B-4 @ 25'	123.6	102.6	20.5	89

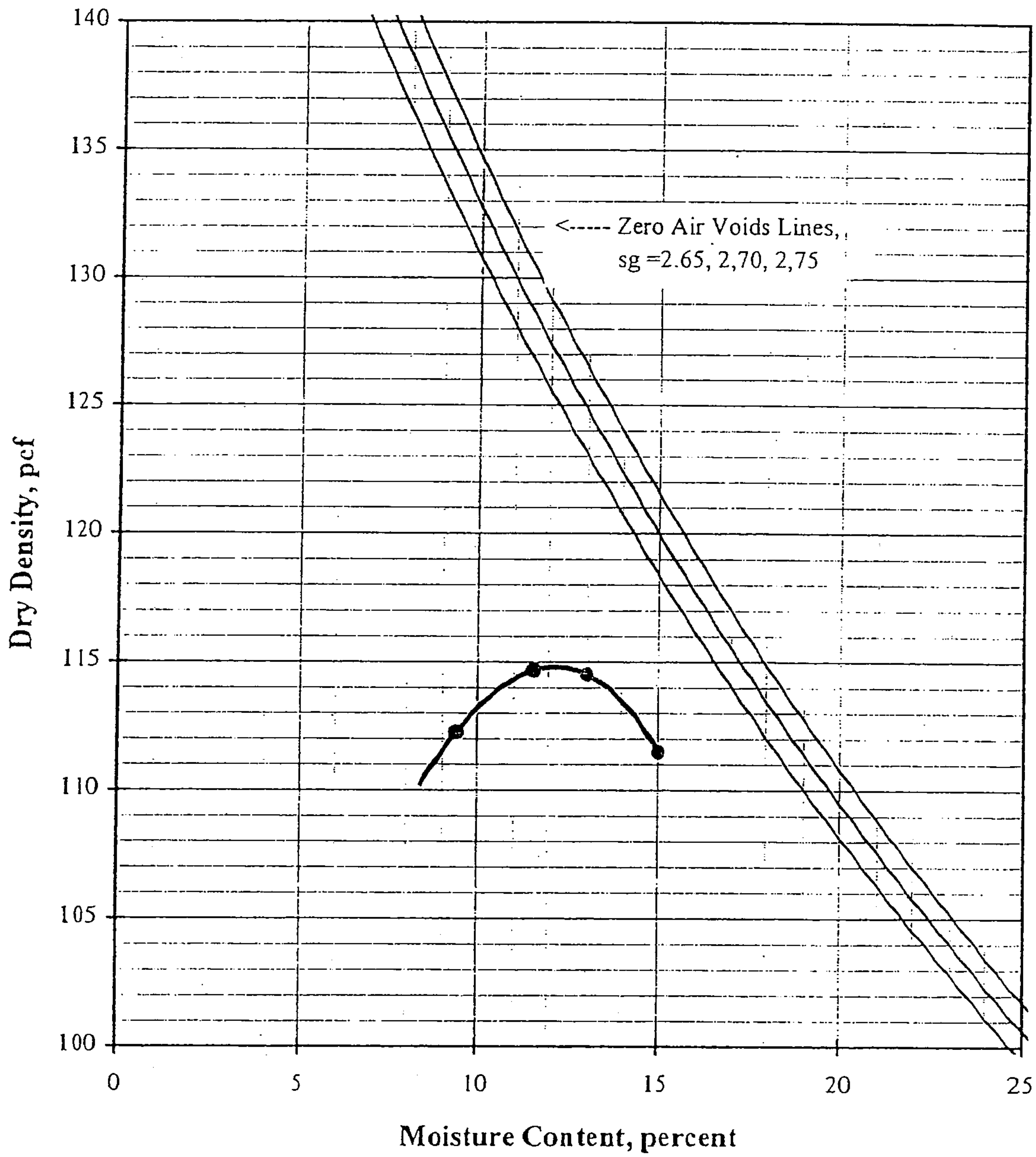
MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-91 (Modified)

Job Name: Santa Paula Memorial Hospital Assisted Living Facility
Procedure Used: A
Sample ID: 1 @ 0 - 5
Prep. Method: Moist
Location:
Rammer Type: Manual
Description: Silt

Maximum Density: 115 pcf
Optimum Moisture: 12%

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



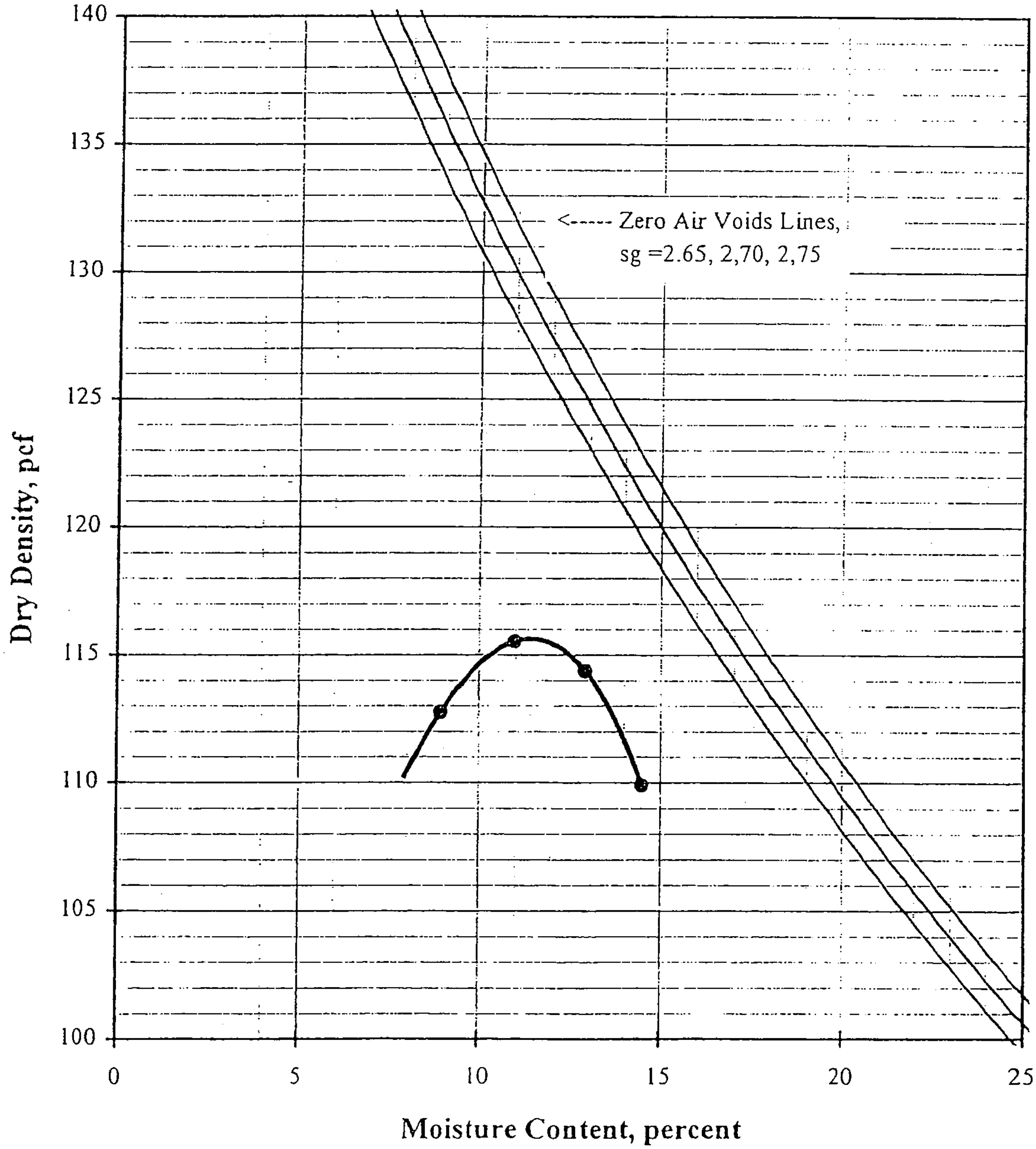
MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-91 (Modified)

Job Name: Santa Paula Memorial Hospital Assisted Living FaciliProcedure Used: A
Sample ID: 2 @ 0 - 5 Prep. Method: Moist
Location: Rammer Type: Manual
Description: Silt

Maximum Density: **115.5 pcf**
Optimum Moisture: **11.5%**

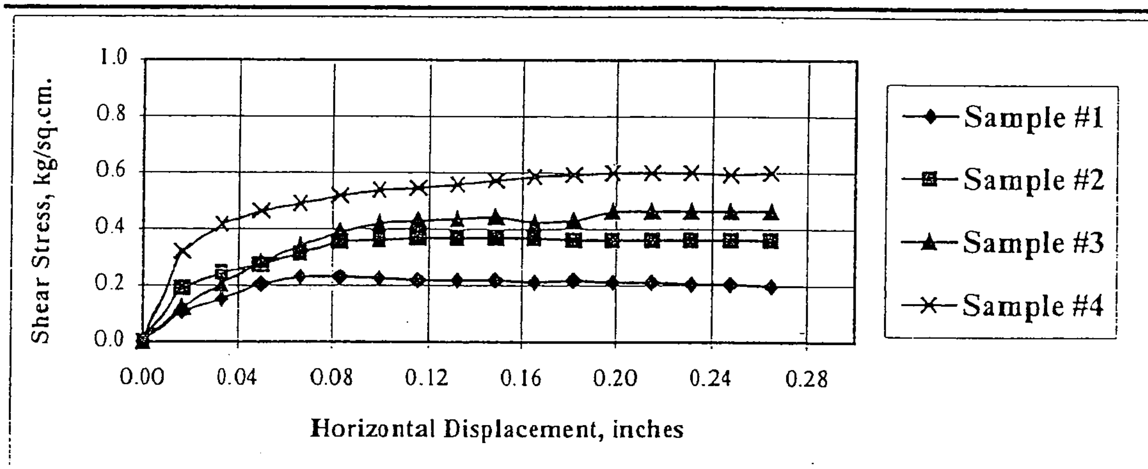
Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	0.0



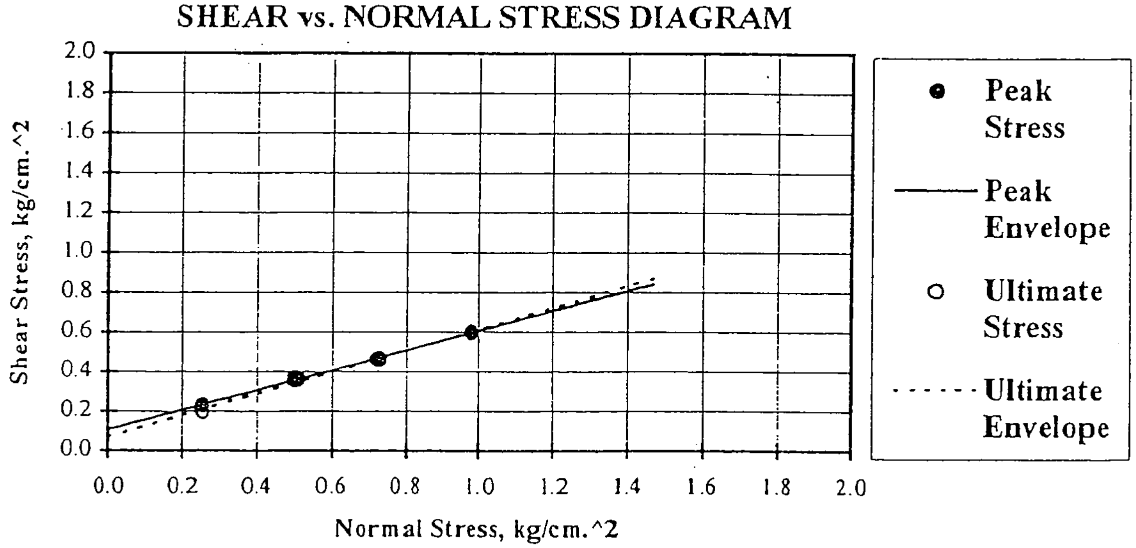
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 104.5 pcf
2 @ 0 - 5 Initial Mosture Content: 10.9 %
Silt Peak Friction Angle (Ø): 27°
Remolded Cohesion (c): 0.107 kg/cm^2 (220 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	104.5	104.5	104.5	104.5	104.5
Moisture Content, %	10.9	10.9	10.9	10.9	10.9
Saturation, %	49	49	49	49	49
At Test					
Moisture Content, %	21.7	21.9	21.7	20.9	21.5
Saturation, %	97	98	97	94	97
Normal Stress, kg/cm^2	0.25	0.51	0.73	0.98	
Peak Stress, kg/cm^2	0.23	0.37	0.46	0.60	
Ultimate Stress, kg/cm^2	0.20	0.36	0.46	0.60	



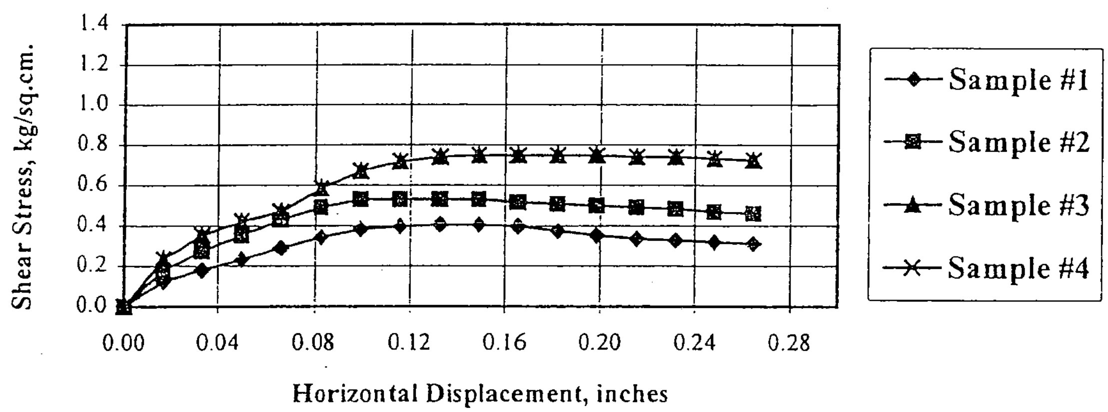
SHEAR vs. NORMAL STRESS DIAGRAM



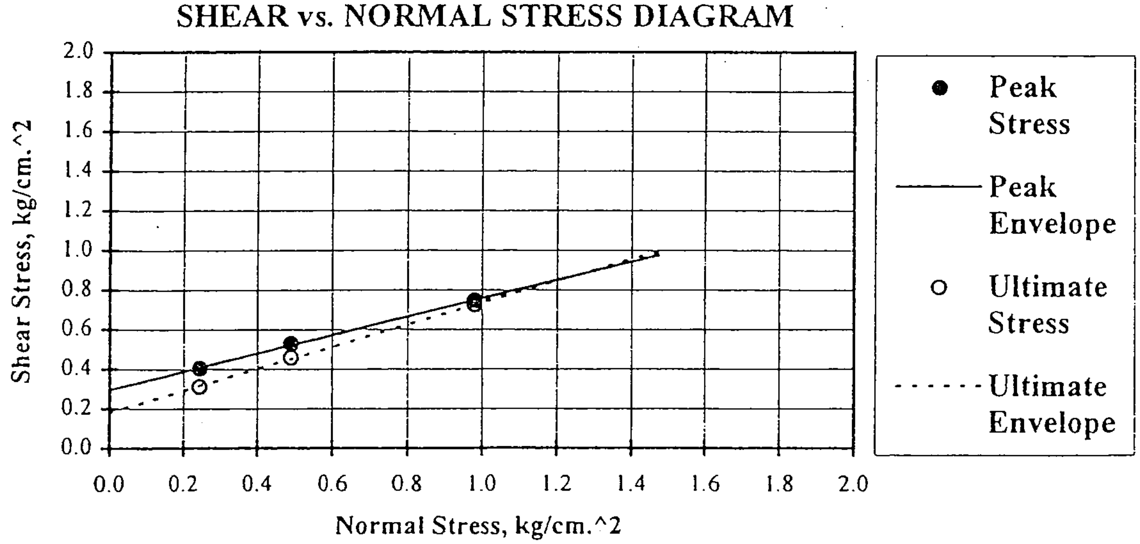
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 107.2 pcf
3 @ 10 Initial Mosture Content: 19.1 %
Clay Peak Friction Angle (Ø): 25°
Undisturbed Cohesion (c): 0.296 kg/cm^2 (610 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	106.4	107.4	107.5	107.5	107.2
Moisture Content, %	19.1	19.1	19.1	19.1	19.1
Saturation, %	90	93	93	93	92
At Test					
Moisture Content, %	21.0	20.4	20.3	20.3	20.5
Saturation, %	99	99	99	99	99
Normal Stress, kg/cm^2	0.24	0.49	0.98	0.98	
Peak Stress, kg/cm^2	0.40	0.53	0.75	0.75	
Ultimate Stress, kg/cm^2	0.31	0.46	0.72	0.72	



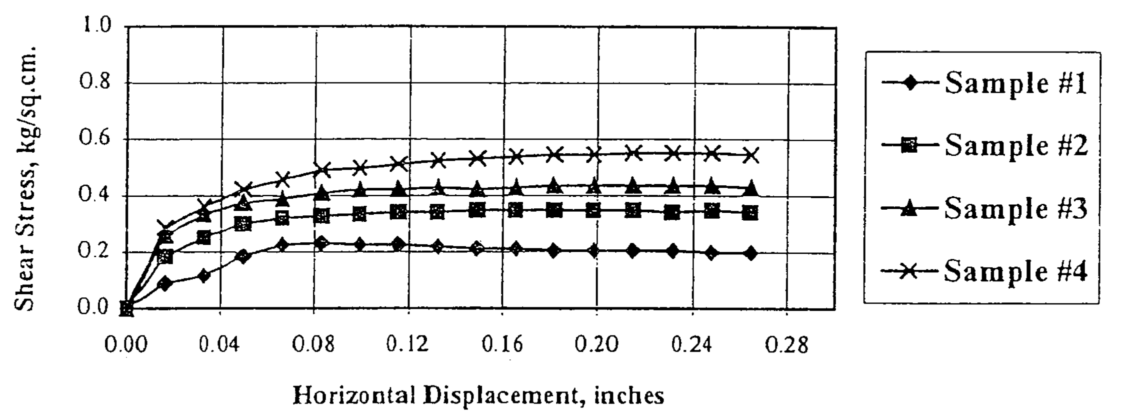
SHEAR vs. NORMAL STRESS DIAGRAM



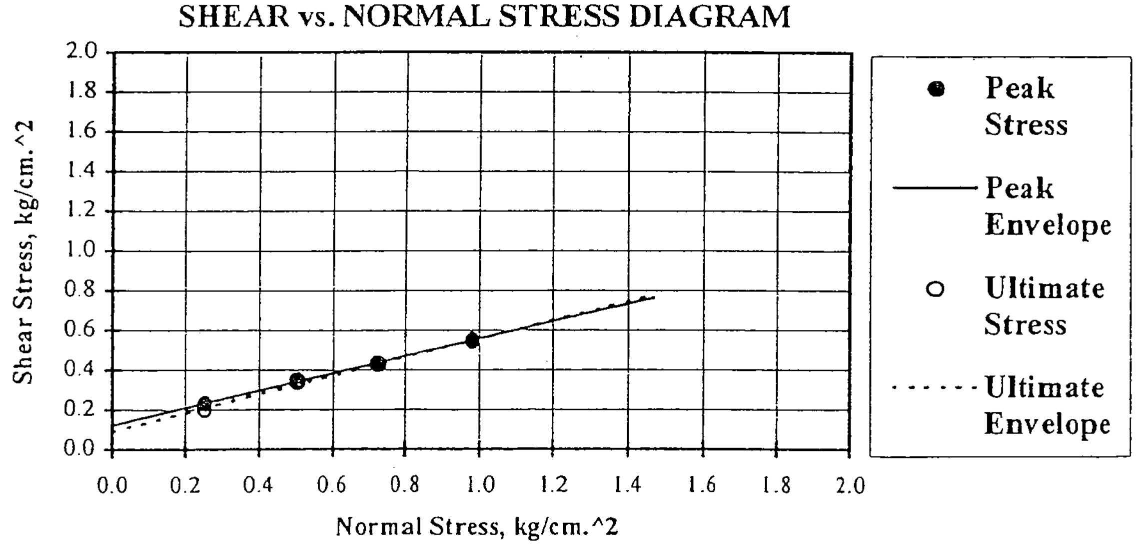
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 103.8 pcf
1 @ 0 - 5 Initial Mosture Content: 11.5 %
Silt Peak Friction Angle (Ø): 24°
Remolded Cohesion (c): 0.122 kg/cm^2 (250 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	103.8	103.8	103.8	103.8	103.8
Moisture Content, %	11.5	11.5	11.5	11.5	11.5
Saturation, %	51	51	51	51	51
At Test					
Moisture Content, %	22.4	21.7	22.3	22.3	22.2
Saturation, %	99	96	98	98	98
Normal Stress, kg/cm^2	0.25	0.51	0.73	0.98	
Peak Stress, kg/cm^2	0.23	0.35	0.44	0.55	
Ultimate Stress, kg/cm^2	0.20	0.34	0.43	0.54	



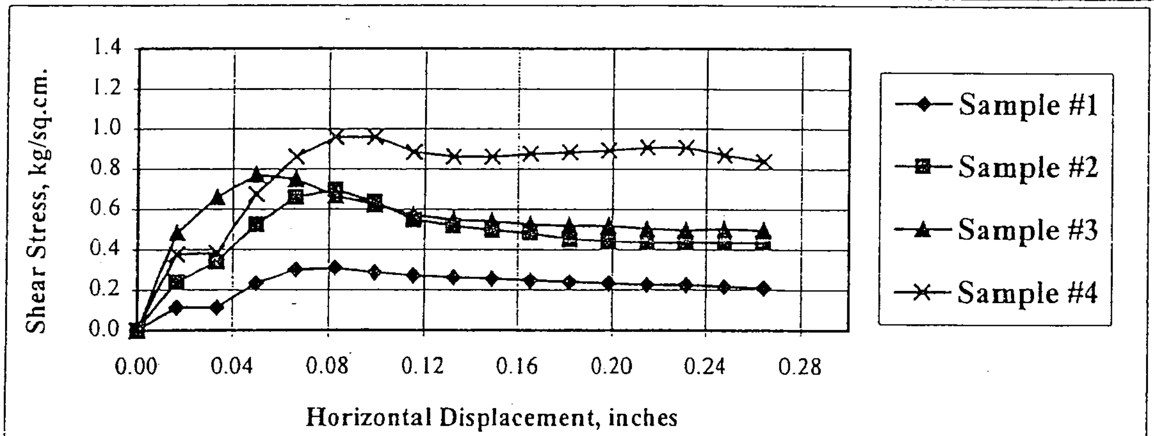
SHEAR vs. NORMAL STRESS DIAGRAM



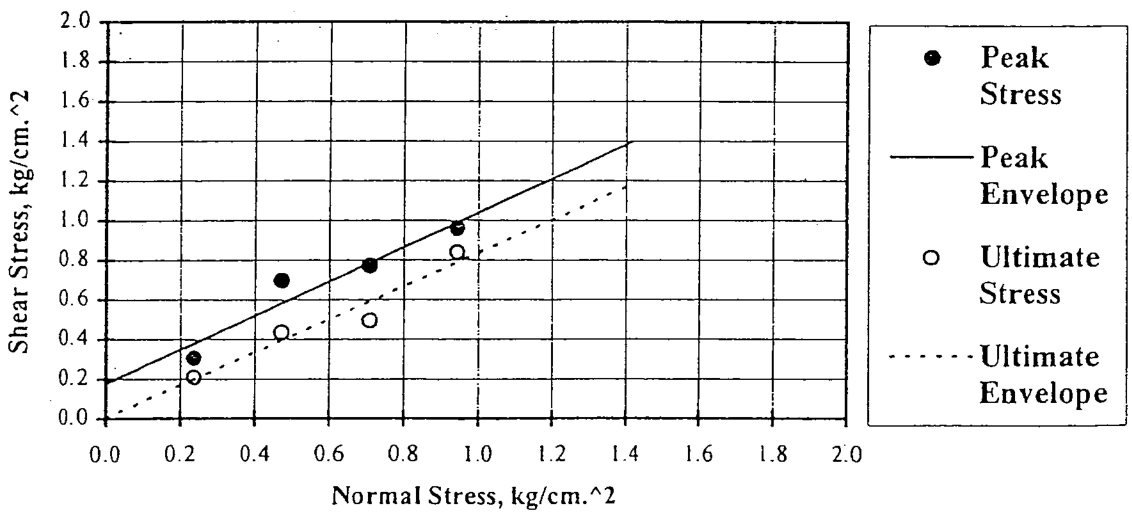
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 108.0 pcf
3 @ 35 Initial Mosture Content: 19.1 %
Clayey Silt Peak Friction Angle (Ø): 41°
Undisturbed Cohesion (c): 0.176 kg/cm^2 (360 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	107.9	108.6	107.4	108.1	108.0
Moisture Content, %	19.1	19.1	19.1	19.1	19.1
Saturation, %	94	96	92	94	94
At Test					
Moisture Content, %	19.9	19.8	19.4	19.9	19.7
Saturation, %	97	99	94	98	97
Normal Stress, kg/cm^2	0.24	0.47	0.71	0.94	
Peak Stress, kg/cm^2	0.31	0.70	0.77	0.96	
Ultimate Stress, kg/cm^2	0.21	0.43	0.49	0.84	



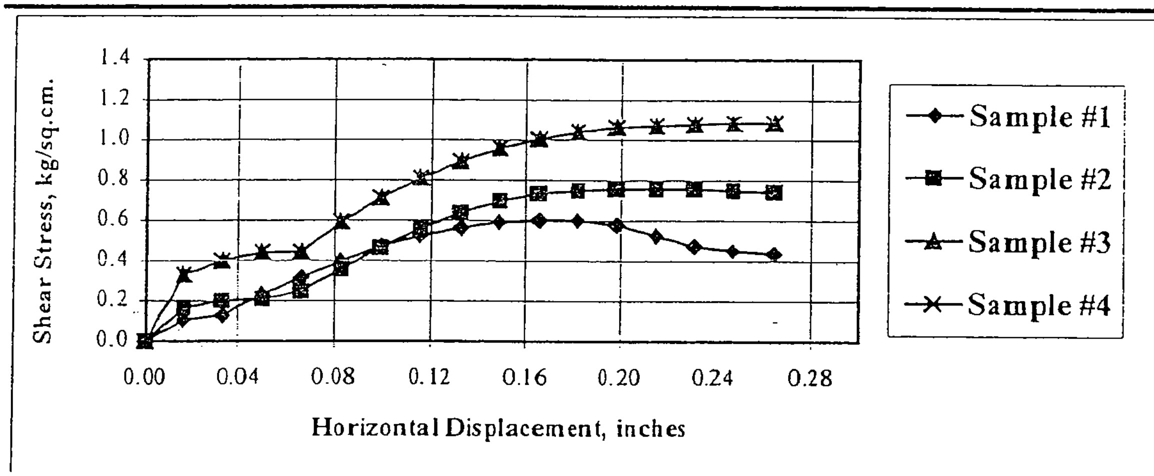
SHEAR vs. NORMAL STRESS DIAGRAM



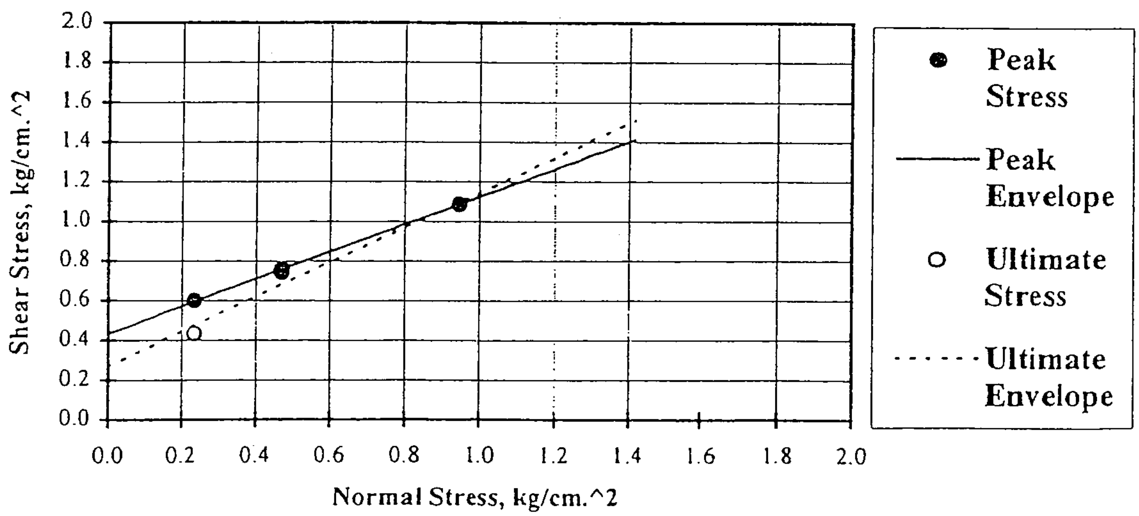
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility	Initial Dry Density: 105.2 pcf
3 @ 2	Initial Mosture Content: 16.8 %
Clay	Peak Friction Angle (Ø): 35°
Undisturbed	Cohesion (c): 0.434 kg/cm^2 (890 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	106.9	109.9	102.0	102.0	105.2
Moisture Content, %	16.8	16.8	16.8	16.8	16.8
Saturation, %	80	87	71	71	77
At Test					
Moisture Content, %	20.3	18.7	21.6	21.6	20.5
Saturation, %	97	97	91	91	94
Normal Stress, kg/cm^2	0.24	0.47	0.94	0.94	
Peak Stress, kg/cm^2	0.60	0.76	1.09	1.09	
Ultimate Stress, kg/cm^2	0.43	0.74	1.09	1.09	



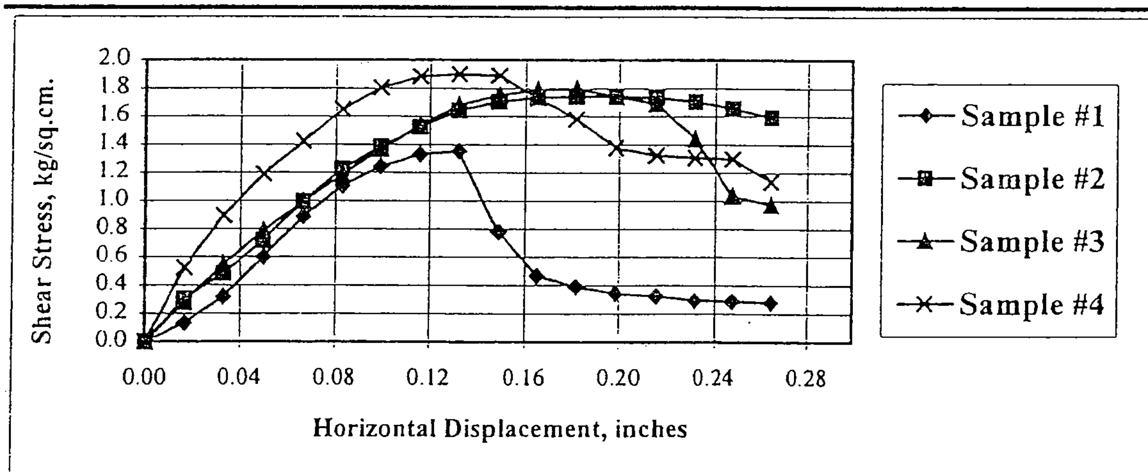
SHEAR vs. NORMAL STRESS DIAGRAM



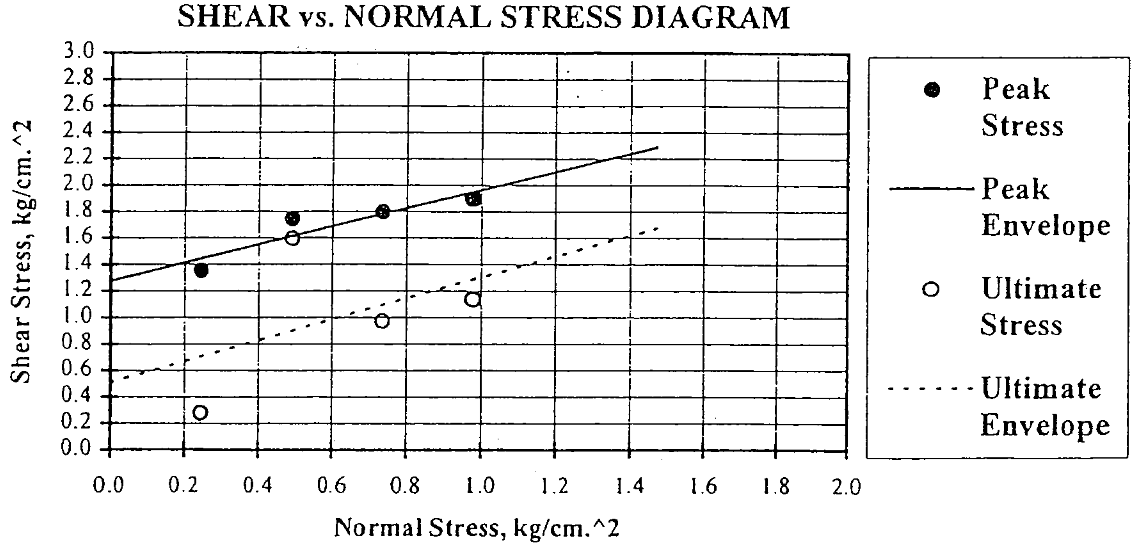
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 116.9 pcf
3 @ 25 Initial Mosture Content: 16.2 %
Clay Peak Friction Angle (Ø): 35°
Undisturbed Cohesion (c): 1.276 kg/cm^2 (2610 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	116.1	116.3	117.3	117.8	116.9
Moisture Content, %	16.2	16.2	16.2	16.2	16.2
Saturation, %	99	100	100	100	102
At Test					
Moisture Content, %	19.0	18.9	19.0	18.7	18.9
Saturation, %	117	116	121	120	119
Normal Stress, kg/cm^2	0.24	0.49	0.73	0.98	
Peak Stress, kg/cm^2	1.35	1.74	1.80	1.90	
Ultimate Stress, kg/cm^2	0.28	1.59	0.97	1.14	



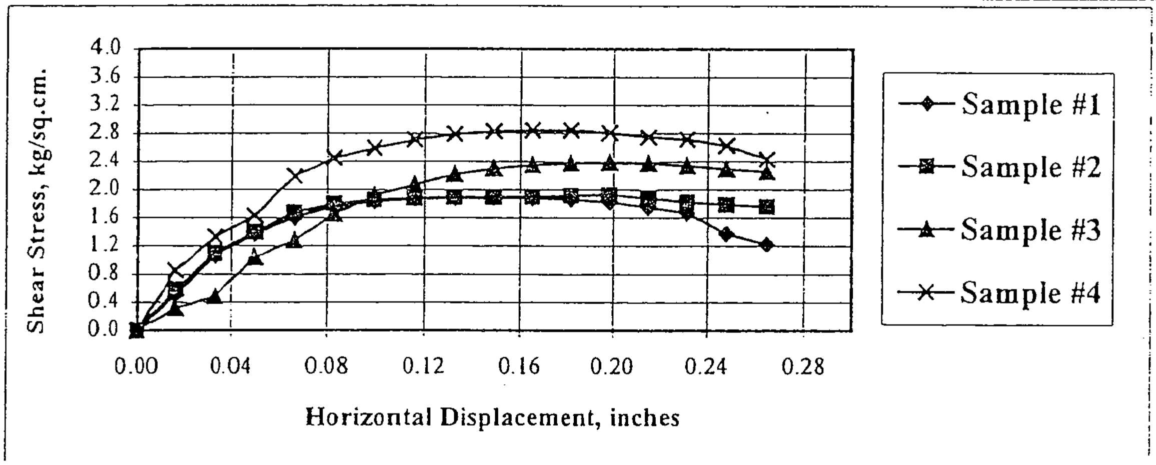
SHEAR vs. NORMAL STRESS DIAGRAM



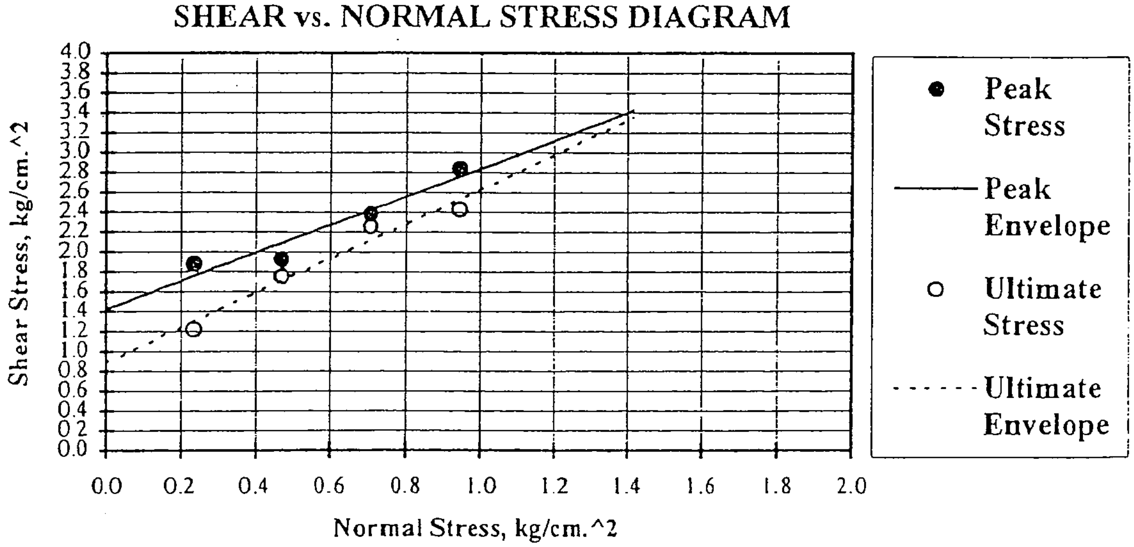
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 118.3 pcf
3 @ 40 Initial Mosture Content: 14.0 %
Silty Clay Peak Friction Angle (Ø): 55°
Undisturbed Cohesion (c): 1.424 kg/cm^2 (2920 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	119.5	117.4	116.9	119.2	118.3
Moisture Content, %	14.0	14.0	14.0	14.0	14.0
Saturation, %	95	89	88	94	91
At Test					
Moisture Content, %	14.0	14.7	15.4	14.7	14.7
Saturation, %	95	94	97	99	96
Normal Stress, kg/cm^2	0.24	0.47	0.71	0.94	
Peak Stress, kg/cm^2	1.88	1.93	2.38	2.84	
Ultimate Stress, kg/cm^2	1.22	1.75	2.26	2.43	



SHEAR vs. NORMAL STRESS DIAGRAM



DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

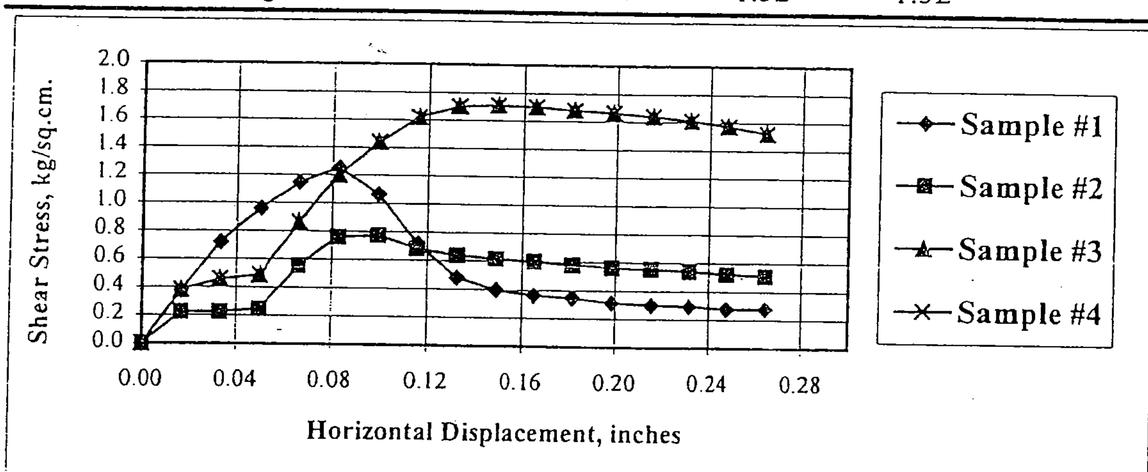
Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 107.7 pcf

3 @ 50 Initial Mosture Content: 19.0 %

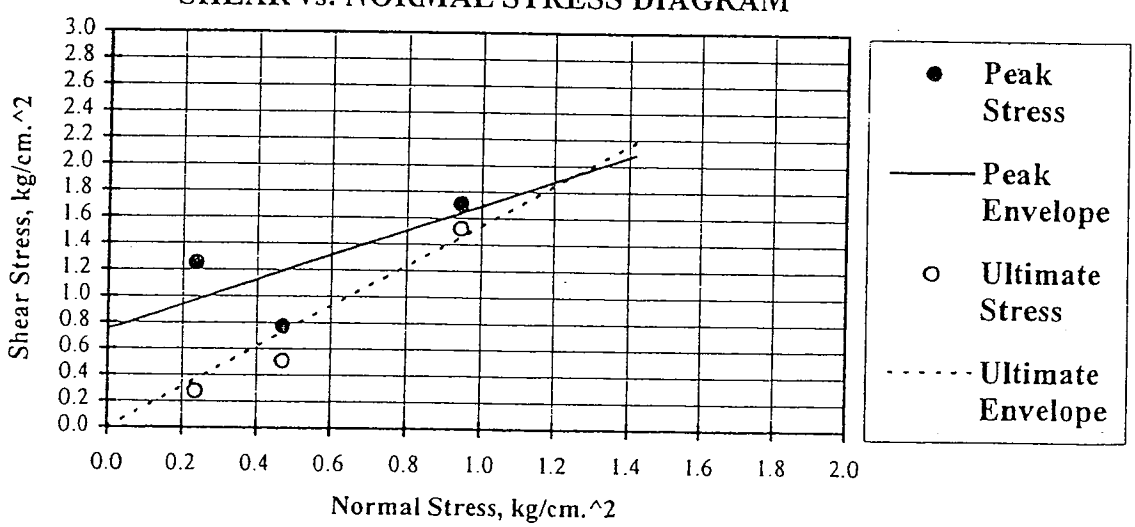
Silty Clay Peak Friction Angle (Ø): 43°

Undisturbed Cohesion (c): 0.748 kg/cm^2 (1530 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	108.4	108.0	107.3	107.3	107.7
Moisture Content, %	19.0	19.0	19.0	19.0	19.0
Saturation, %	95	94	92	92	93
At Test					
Moisture Content, %	19.7	20.2	19.8	19.8	19.9
Saturation, %	98	100	95	95	97
Normal Stress, kg/cm^2	0.24	0.47	0.94	0.94	
Peak Stress, kg/cm^2	1.25	0.77	1.71	1.71	
Ultimate Stress, kg/cm^2	0.28	0.51	1.52	1.52	



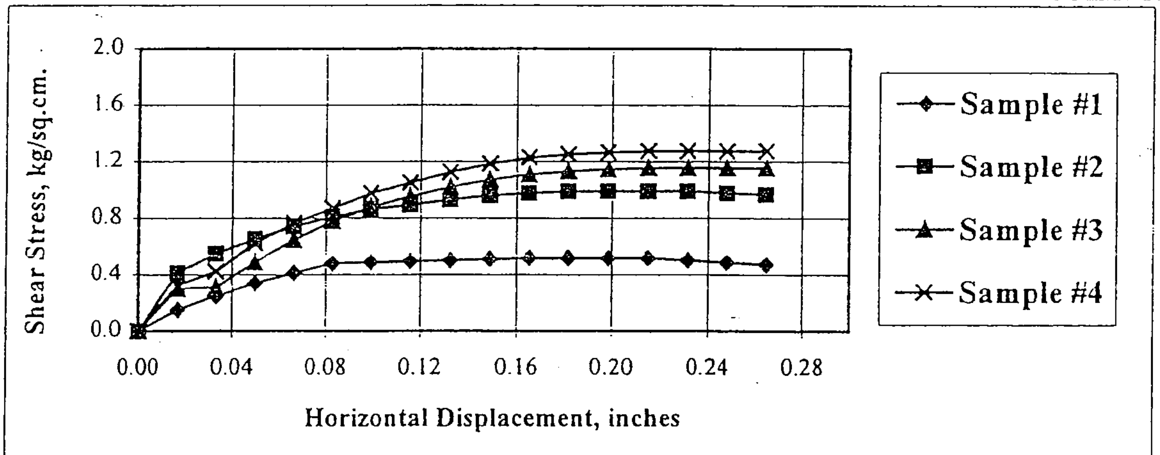
SHEAR vs. NORMAL STRESS DIAGRAM



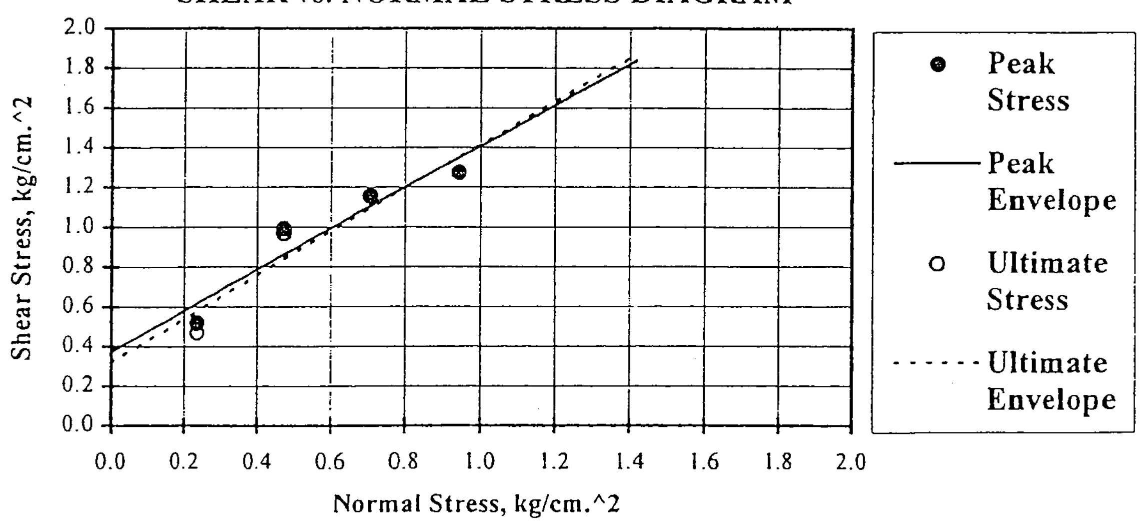
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 110.2 pcf
4 @ 2 Initial Mosture Content: 16.6 %
Silt Peak Friction Angle (Ø): 46°
Undisturbed Cohesion (c): 0.375 kg/cm^2 (770 psf)

Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	109.7	109.7	110.7	110.9	110.2
Moisture Content, %	16.6	16.6	16.6	16.6	16.6
Saturation, %	85	85	88	88	87
At Test					
Moisture Content, %	19.1	18.9	18.3	18.6	18.7
Saturation, %	98	97	97	99	98
Normal Stress, kg/cm^2	0.24	0.47	0.71	0.94	
Peak Stress, kg/cm^2	0.52	0.99	1.15	1.27	
Ultimate Stress, kg/cm^2	0.47	0.97	1.15	1.27	



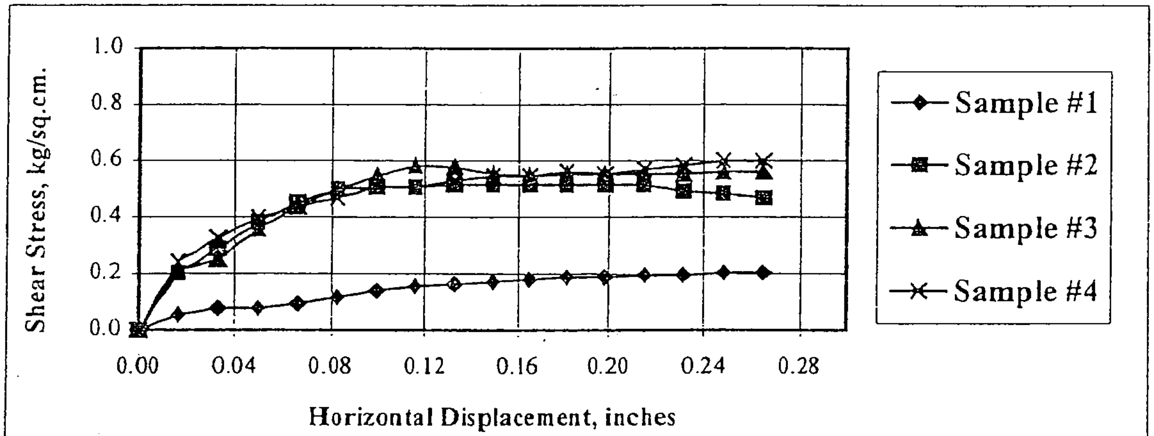
SHEAR vs. NORMAL STRESS DIAGRAM



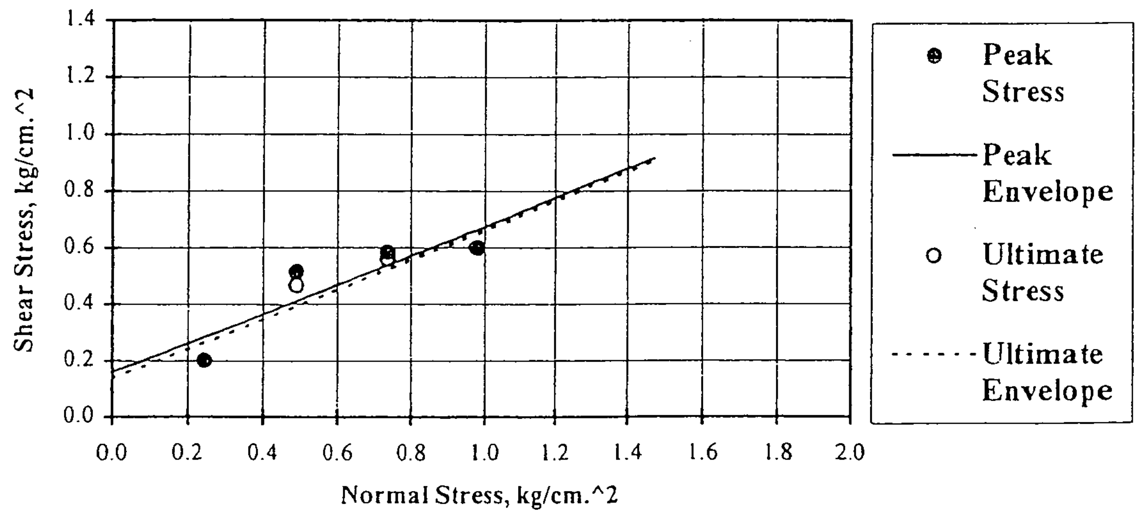
DIRECT SHEAR 080-90 (modified for unconsolidated, undrained conditions)

Santa Paula Memorial Hospital Assisted Living Facility Initial Dry Density: 105.8 pcf
4 @ 15 Initial Mosture Content: 16.3 %
Silt Peak Friction Angle (Ø): 27°
Undisturbed Cohesion (c): 0.160 kg/cm^2 (330 psf)

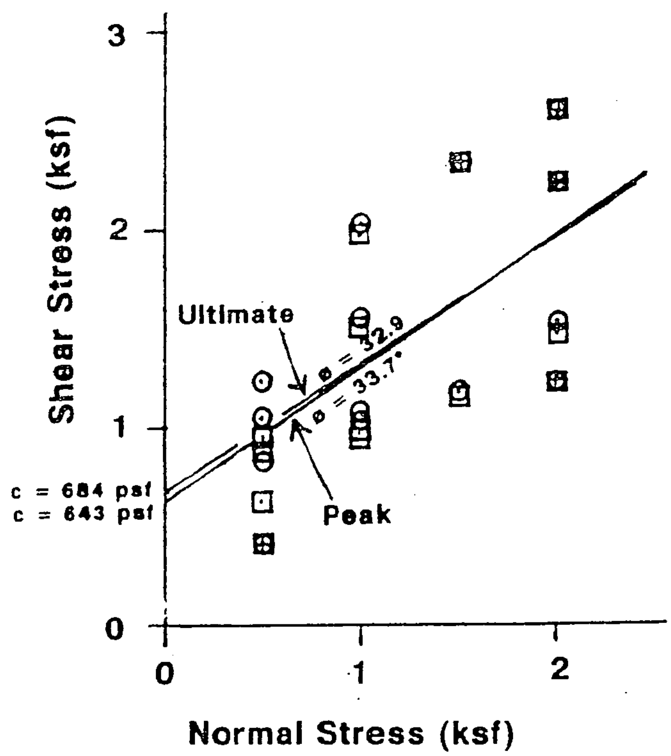
Sample No.	1	2	3	4	Average
Initial					
Dry Density, pcf	103.8	105.9	105.9	107.5	105.8
Moisture Content, %	16.3	16.3	16.3	16.3	16.3
Saturation, %	72	76	76	79	76
At Test					
Moisture Content, %	21.4	19.5	20.1	20.5	20.4
Saturation, %	95	91	94	100	95
Normal Stress, kg/cm^2	0.24	0.49	0.73	0.98	
Peak Stress, kg/cm^2	0.20	0.51	0.58	0.60	
Ultimate Stress, kg/cm^2	0.20	0.47	0.56	0.60	



SHEAR vs. NORMAL STRESS DIAGRAM



SCATTER DIAGRAM OF DIRECT SHEAR DATA FOR TERRACE DEPOSITS

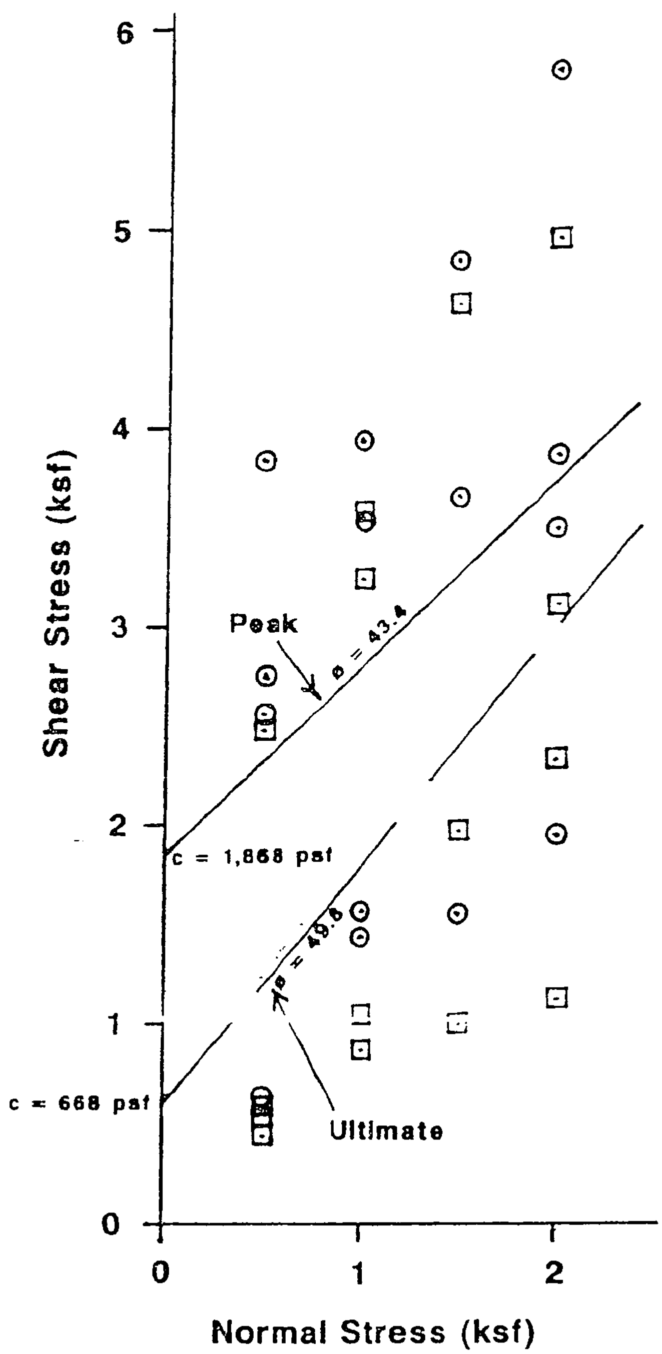


⊙ PEAK VALUES

⊡ ULTIMATE VALUES

ASSISTED LIVING FACILITY	
Earth Systems Southern California	
DATE AUG. 2000	FILE NO. VT22251-02

SCATTER DIAGRAM OF DIRECT SHEAR DATA FOR SAUGUS FORMATION



⊙ PEAK VALUES

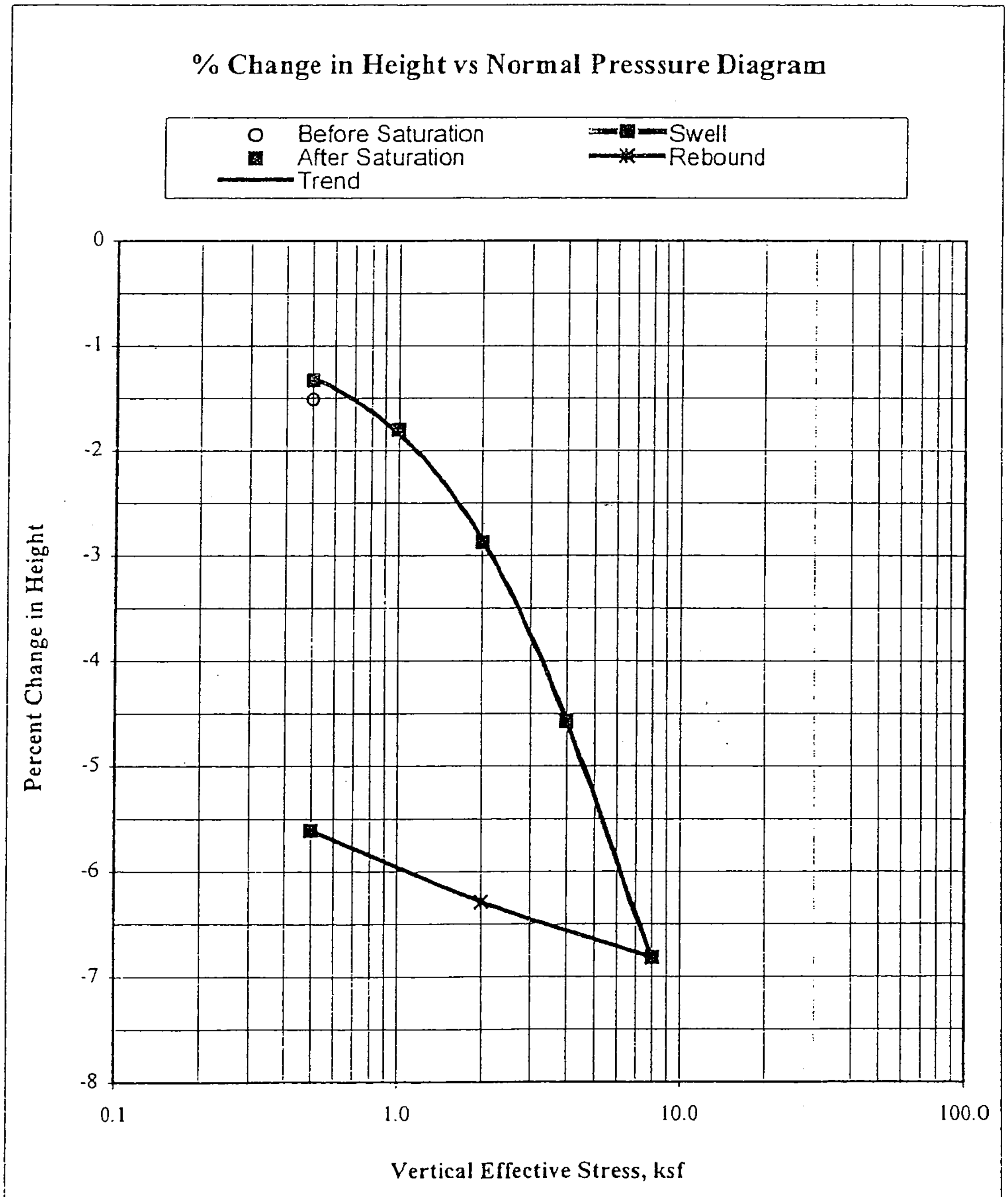
⊡ ULTIMATE VALUES

ASSISTED LIVING FACILITY	
Earth Systems Southern California	
DATE **AUG. 2000**	FILE NO. **VT22251-02**

CONSOLIDATION TEST

ASTM D 2435-90

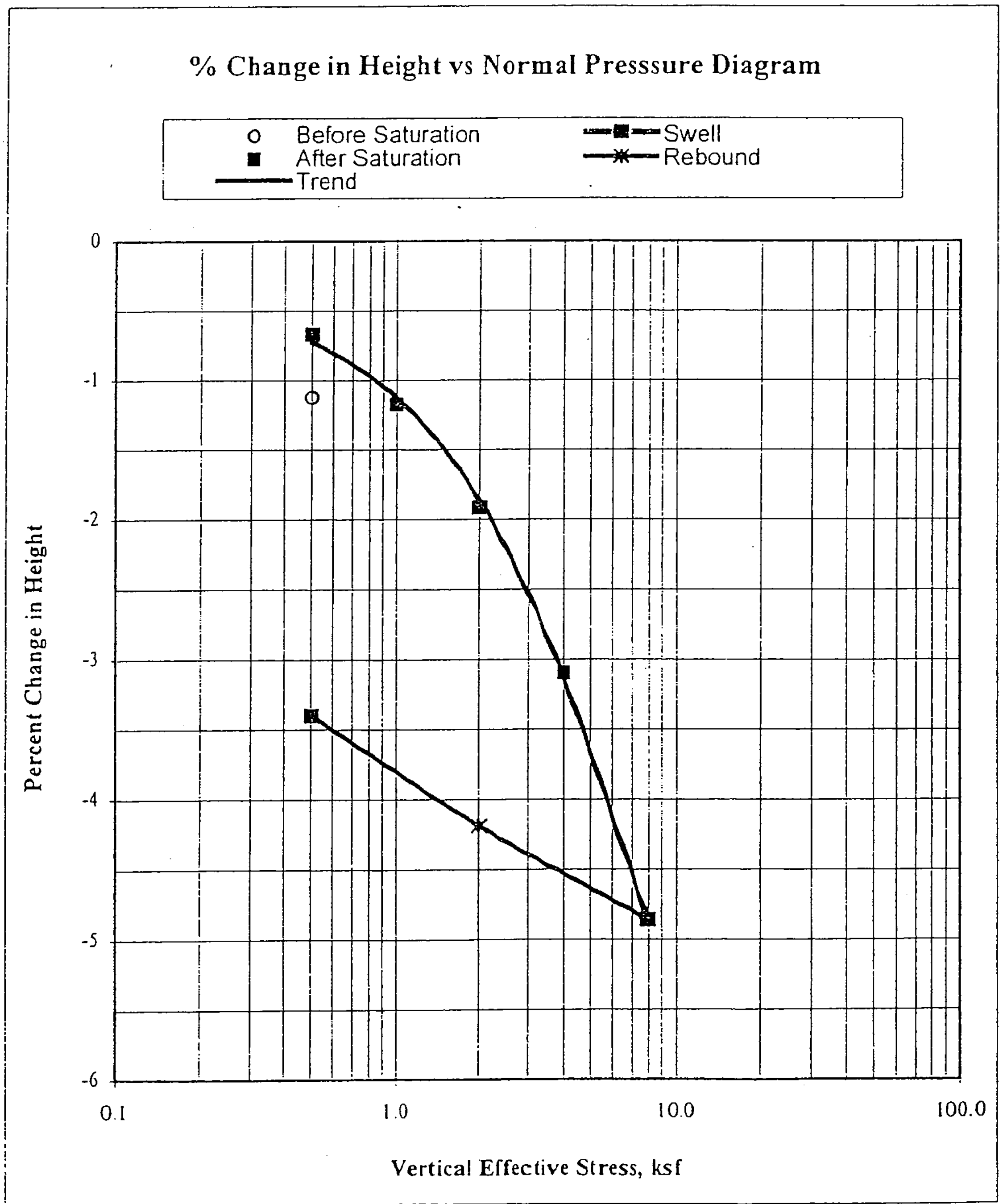
Santa Paula Memorial Hospital Assisted Living Facility	Initial Dry Density: 103.7 pcf
1 @ 5	Initial Moisture, %: 16.6%
Clay	Specific Gravity: 2.67 (assumed
Ring Sample	Initial Void Ratio: 0.607



CONSOLIDATION TEST

ASTM D 2435-90

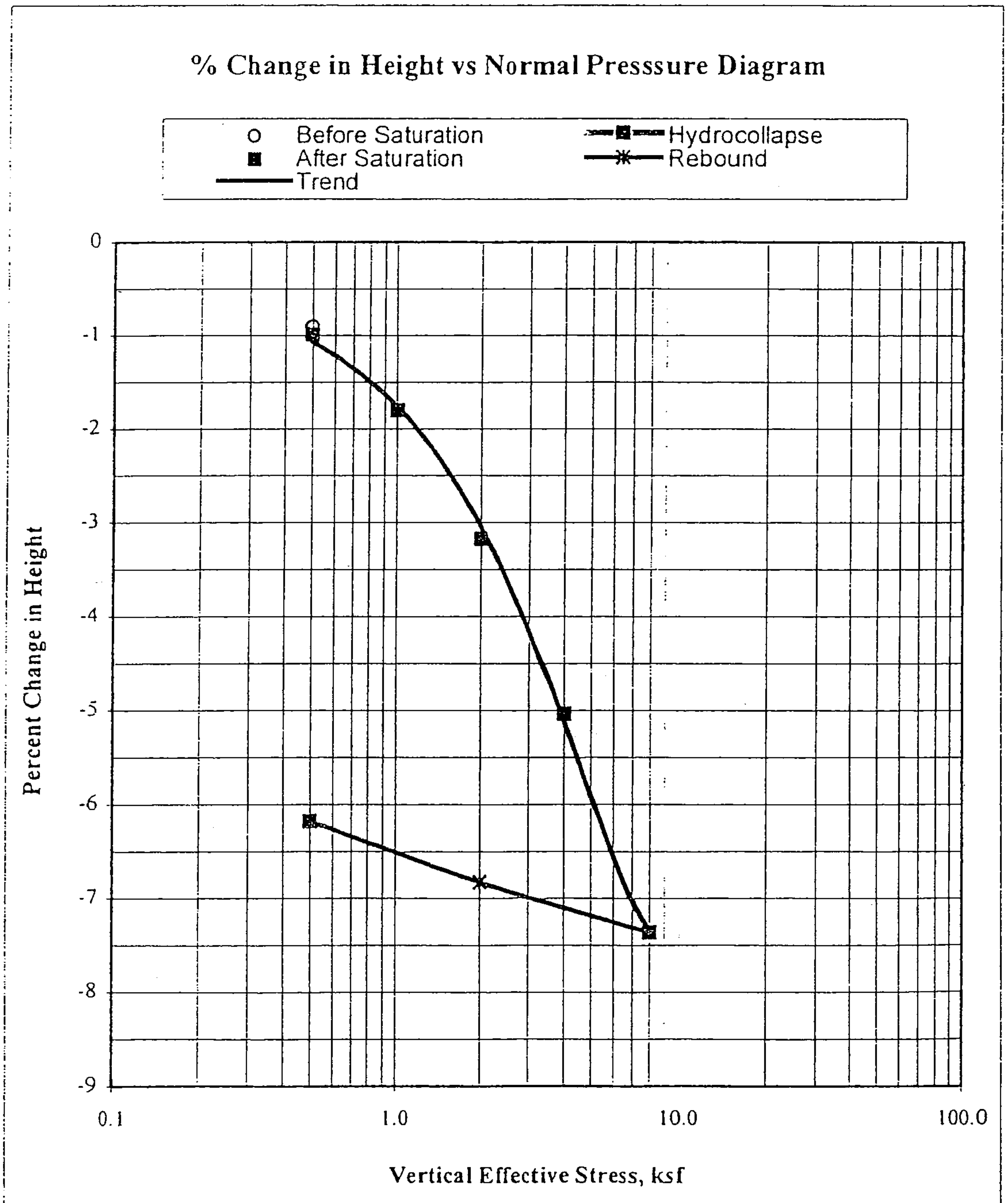
Santa Paula Memorial Hospital Assisted Living Facility	Initial Dry Density: 113.8 pcf
2 @ 5	Initial Moisture, %: 15.6%
Clay	Specific Gravity: 2.67 (assumed
Ring Sample	Initial Void Ratio: 0.465



CONSOLIDATION TEST

ASTM D 2435-90

Santa Paula Memorial Hospital Assisted Living Facility		Initial Dry Density:	108.8 pcf
4 @ 5		Initial Moisture, %:	15.5%
Clay		Specific Gravity:	2.67 (assumed
Ring Sample		Initial Void Ratio:	0.533



APPENDIX D

STABILITY ANALYSES

Computer Program

Stability analyses were performed using the computer program SLOPE/W (Ver. 4.23) by Geo-Slope. The program analyzes slope stability problems by a two-dimensional limit equilibrium methods including Bishop's, Janbu, Morgenstern & Price, and general limit equilibrium (GLE). The particular method used for each analysis is indicated on the output plots.

Soil strength can be modeled in a variety of ways including standard Mohr-Coulomb, bilinear Mohr-Coulomb, and general shear strength relationships. Where materials strengths have anisotropic properties, the program allows the strength to be modeled by introducing a strength function. The function allows the user to define a weighting factor that is applied to the standard Mohr-Coulomb strength depending upon the angle of inclination of the slice base. With this function, anisotropic conditions typically found in bedrock materials can be modeled.

Potential failure surfaces are determined by circular surfaces, block-specified surfaces, or fully-specified surfaces. For circular surfaces, the user provides a grid of radius points and upper and lower bounds for the radius search. The program calculates the factor of safety for each radius grid point and all increments between the upper and lower boundaries. For block-specified surfaces, the user provides two grids of points, one for points of entry and one for points of exit. The program calculates the factor of safety for all possible combinations of surfaces defined by connecting pairs of grid points. For fully-specified surfaces, the user defines a specific failure surface for which the program calculates the factor of safety. The program can also model other factors such as groundwater, earthquake loads, and external loads.

Shear Strengths

The shear strengths used in our analyses were based on direct shear testing and previous experience. The strength values used are summarized In Table D-1 below:

TABLE D-1
Summary of Shear Strengths

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Compacted Fill (Qcaf)	128	250	25
Terrace Deposit (Qt)	128	350	26
Terrace Deposit (Qt) Temporary Cut	128	400	26
Bedrock (TQsa)	130	1500	28

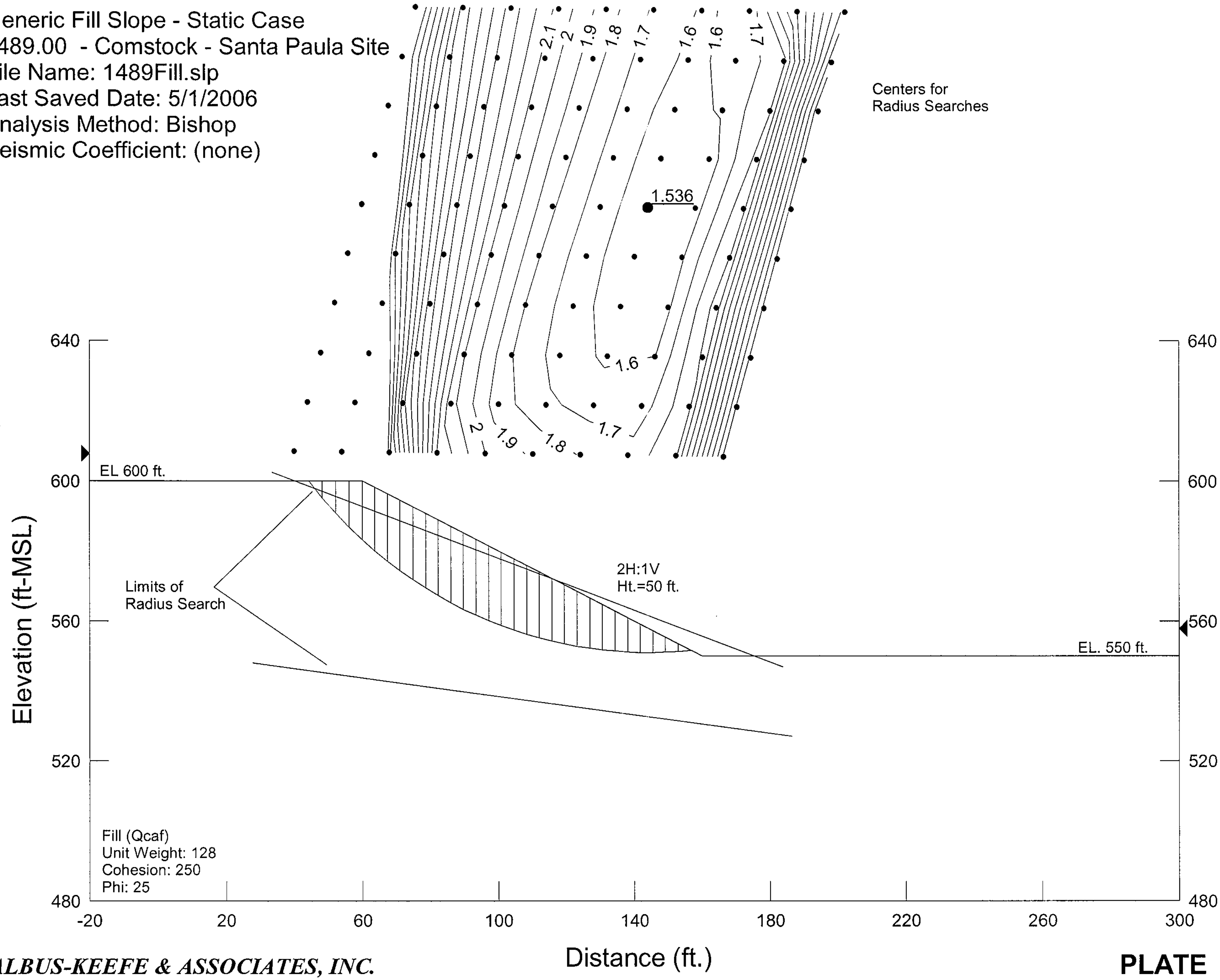
Summary of Results

Results of the analyses are summarized in Table D-2 below. Plots of the results are attached as Plates D-1 through D-6.

TABLE D-2
Summary of Stability Analyses

Section	Search Type	Analysis Method	Plates	Static Factor of Safety	Seismic Factor of Safety
50' Fill Slope 2H:1V	Circular	Bishop	D-1 & D-2	1.54	1.11
C-C'	Circular	Bishop	D-3 & D-4	1.63	1.24
C-C' Detail	Circular	Bishop	D-5 & D-6	1.81	1.38
D-D'	Circular	Bishop	D-7 & D-8	1.81	1.34
F-F'	Circular	Bishop	D-9 & D-10	1.54	1.26
G-G'	Circular	Bishop	D-11 & D-12	1.51	1.10
G-G' Temp. Cut	Circular	Bishop	D-13	1.25	-

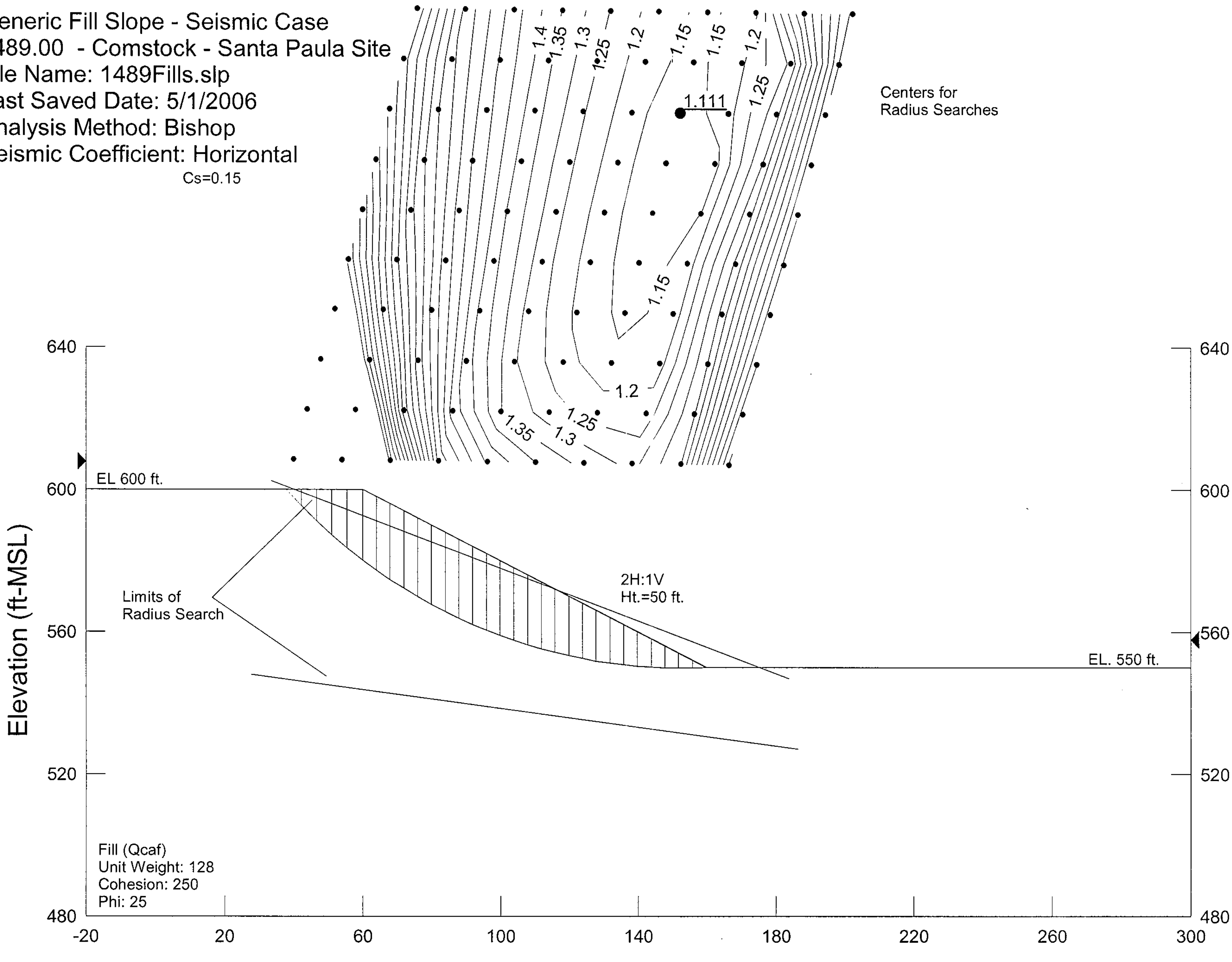
Generic Fill Slope - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489Fill.slp
Last Saved Date: 5/1/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

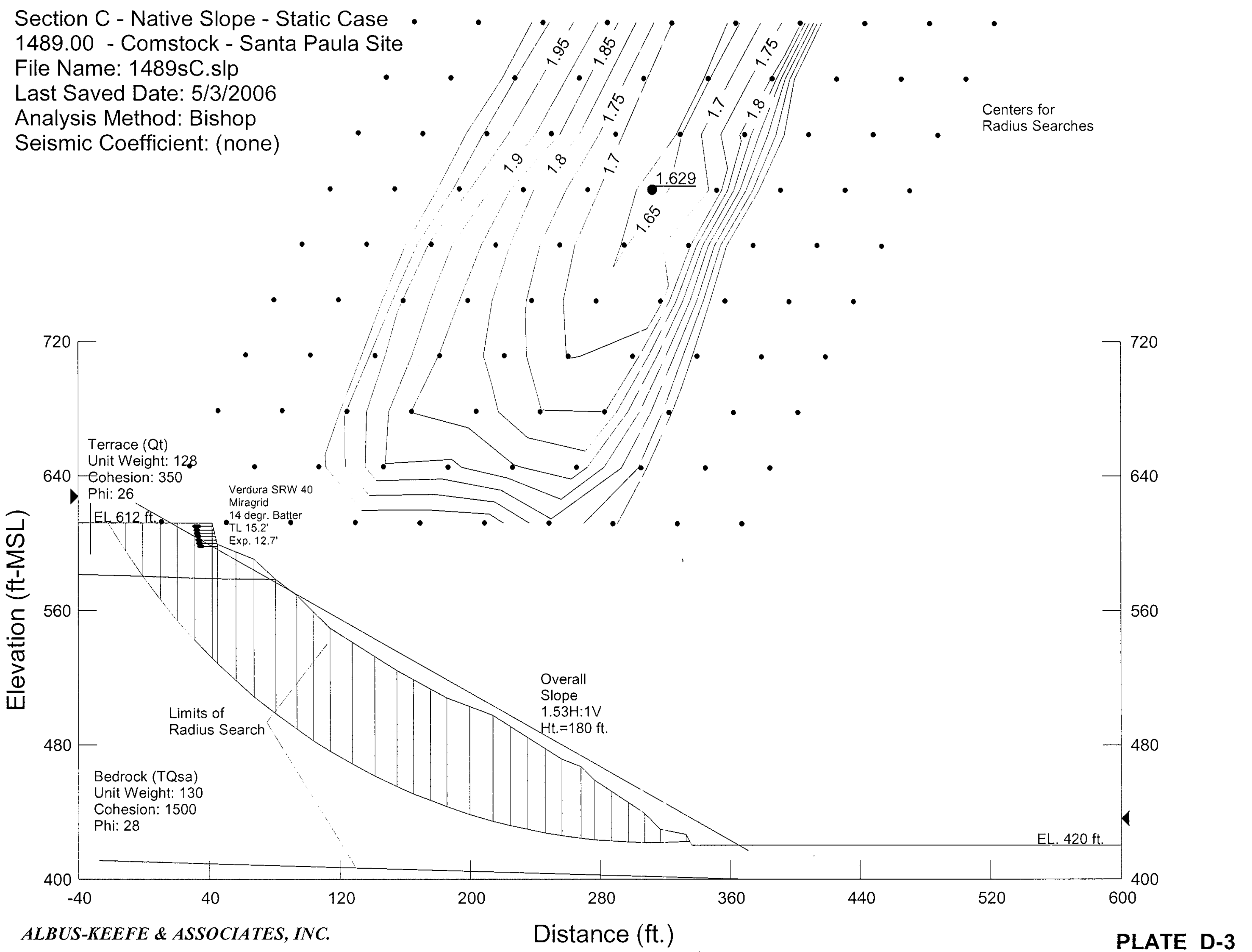
PLATE D–1

Generic Fill Slope - Seismic Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489Fills.slp
Last Saved Date: 5/1/2006
Analysis Method: Bishop
Seismic Coefficient: Horizontal
Cs=0.15



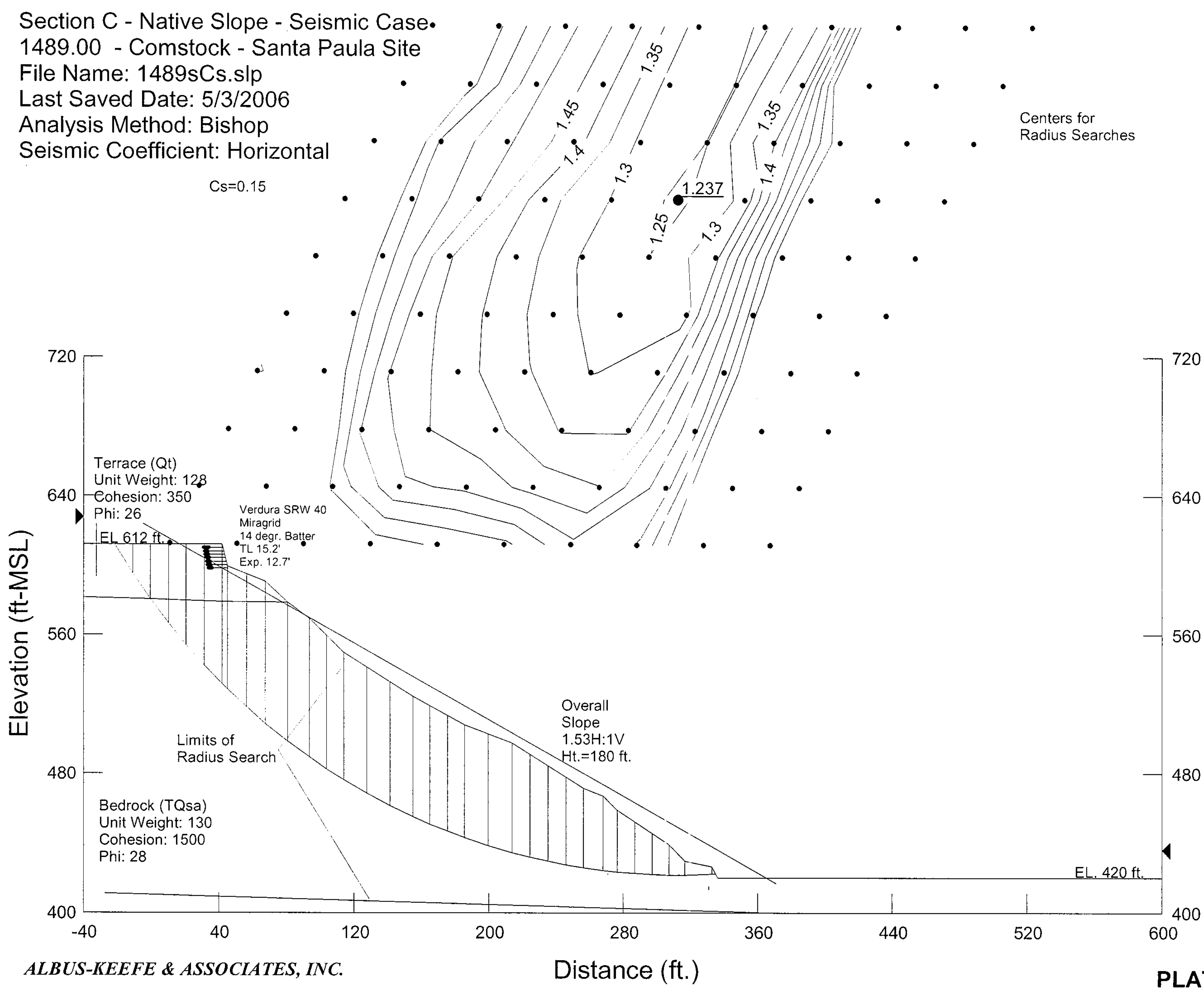
ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D–2



ALBUS-KEEFE & ASSOCIATES, INC.

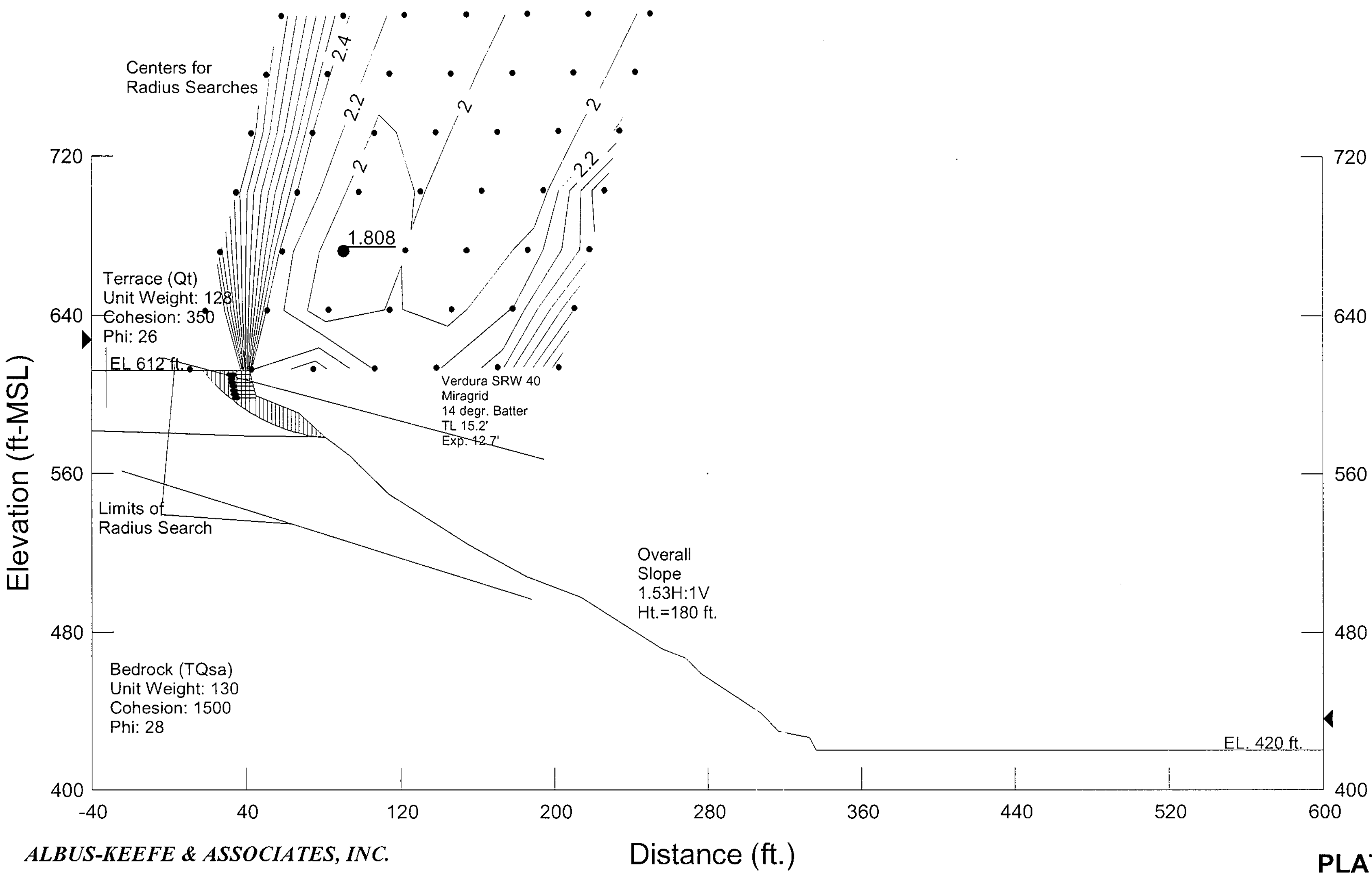
PLATE D-3



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D-4

Section C - Global Segmental Wall - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489sC1.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: (none)

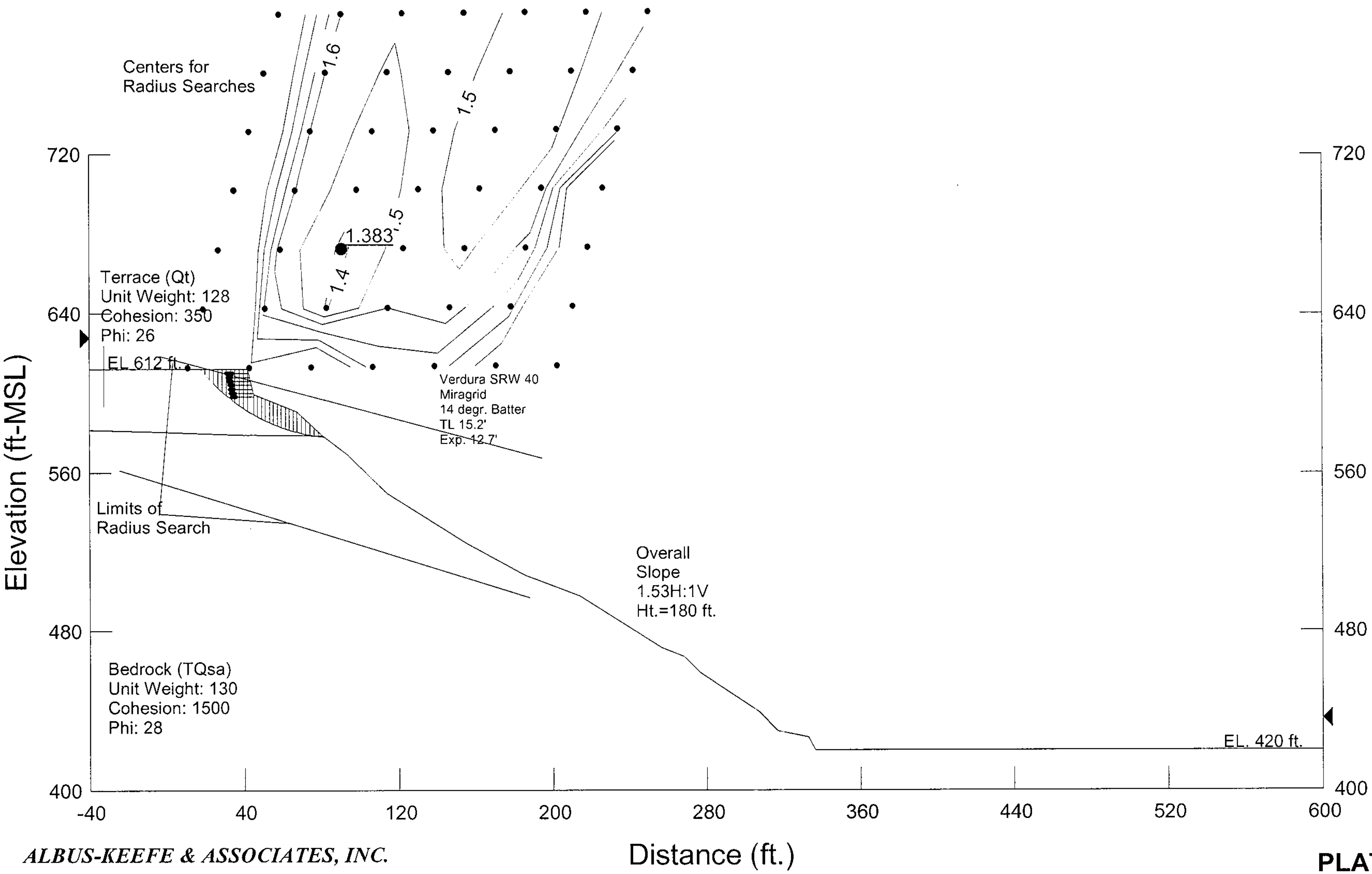


ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D-5

Section C - Global Segmental Wall - Seismic Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489sC1s.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: Horizontal

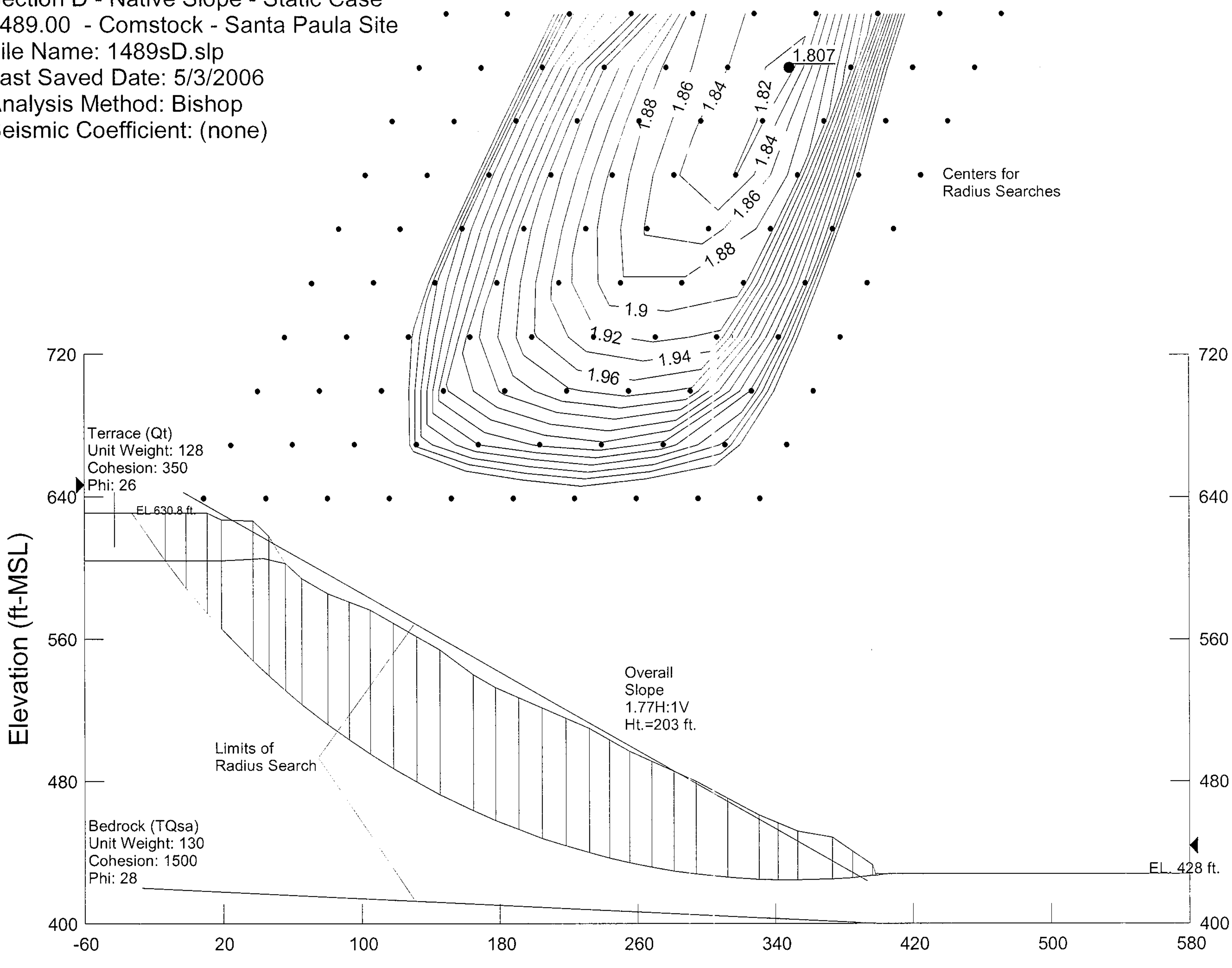
Cs=0.15



ALBUS-KEEFE & ASSOCIATES, INC.

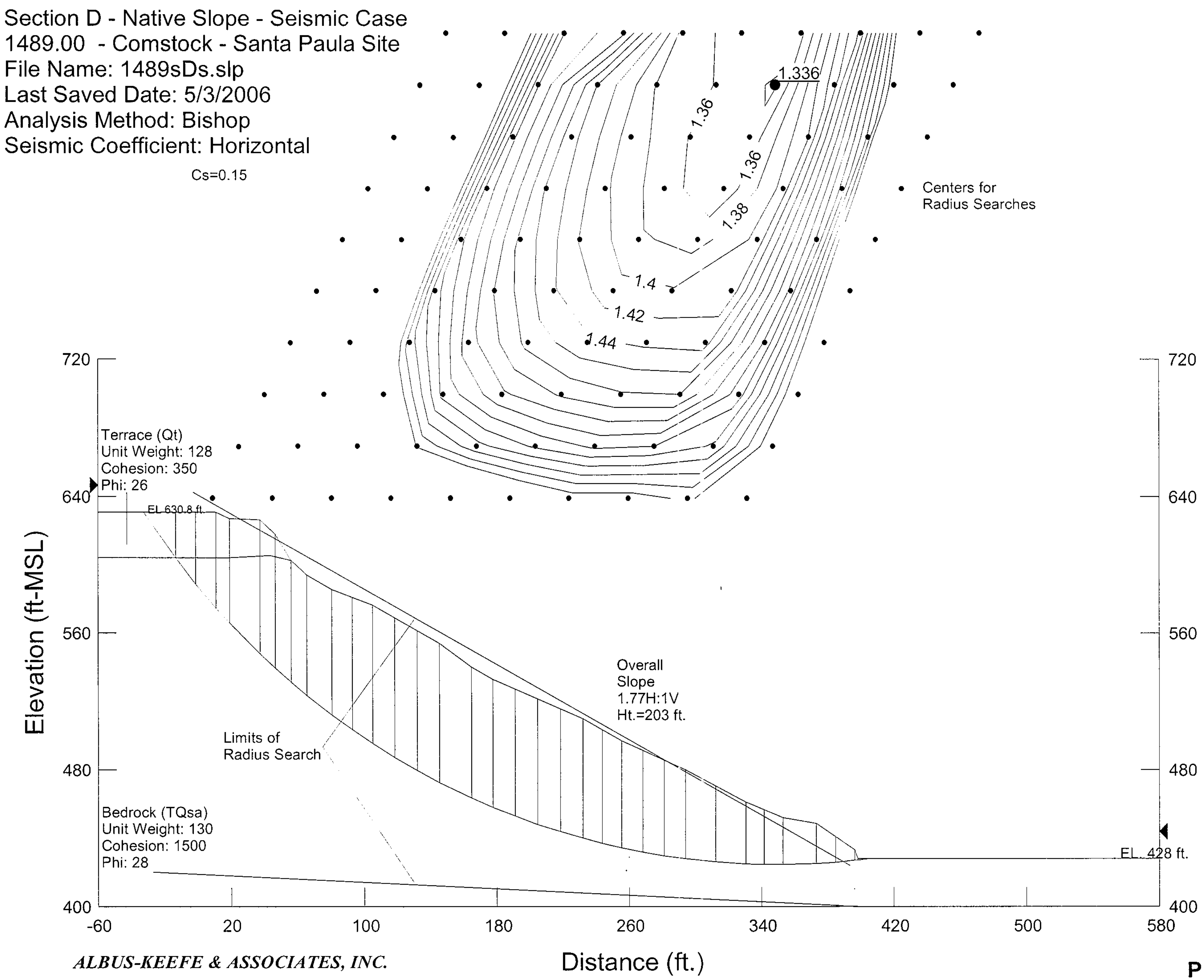
PLATE D-6

Section D - Native Slope - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489sD.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

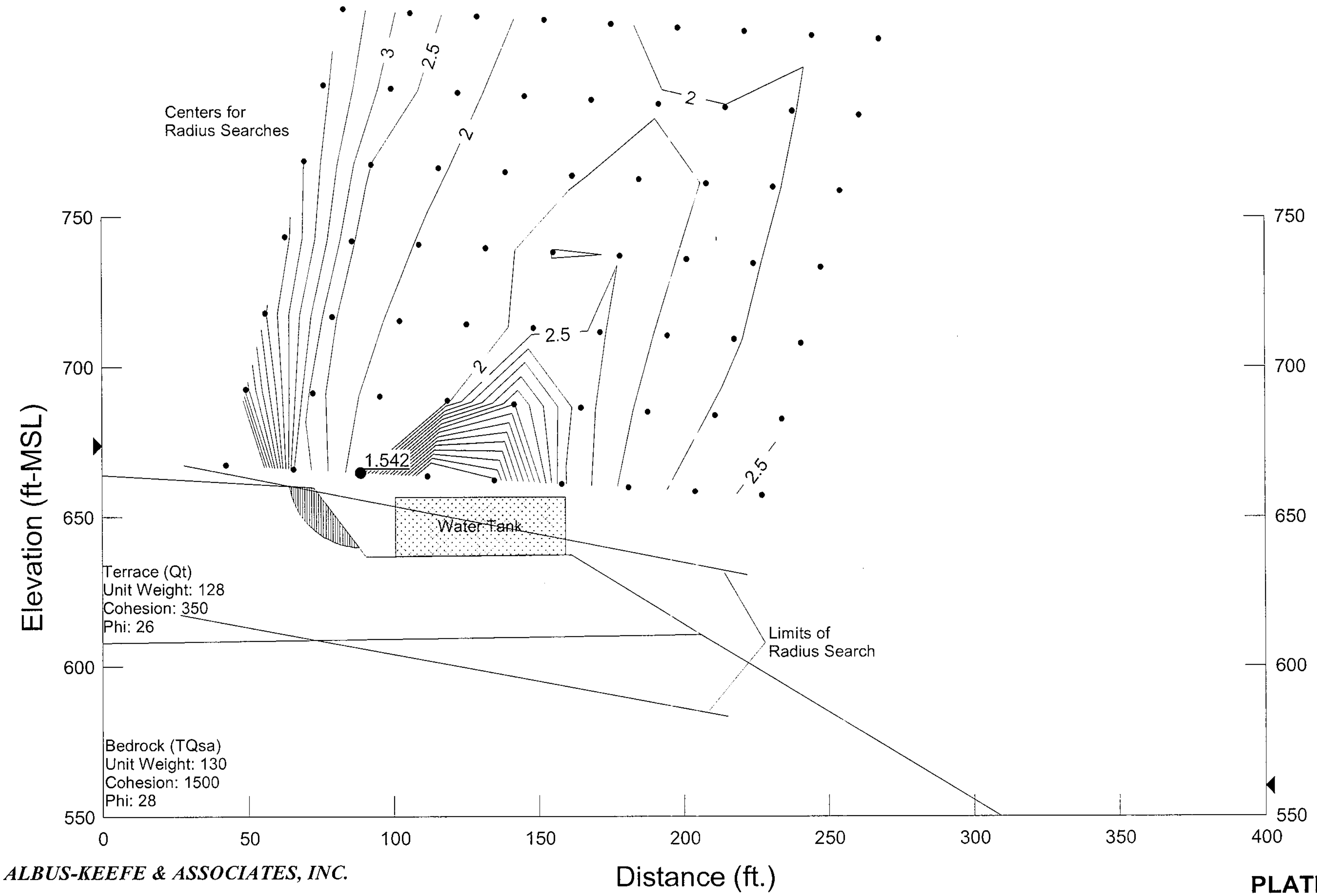
PLATE D-7



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D-8

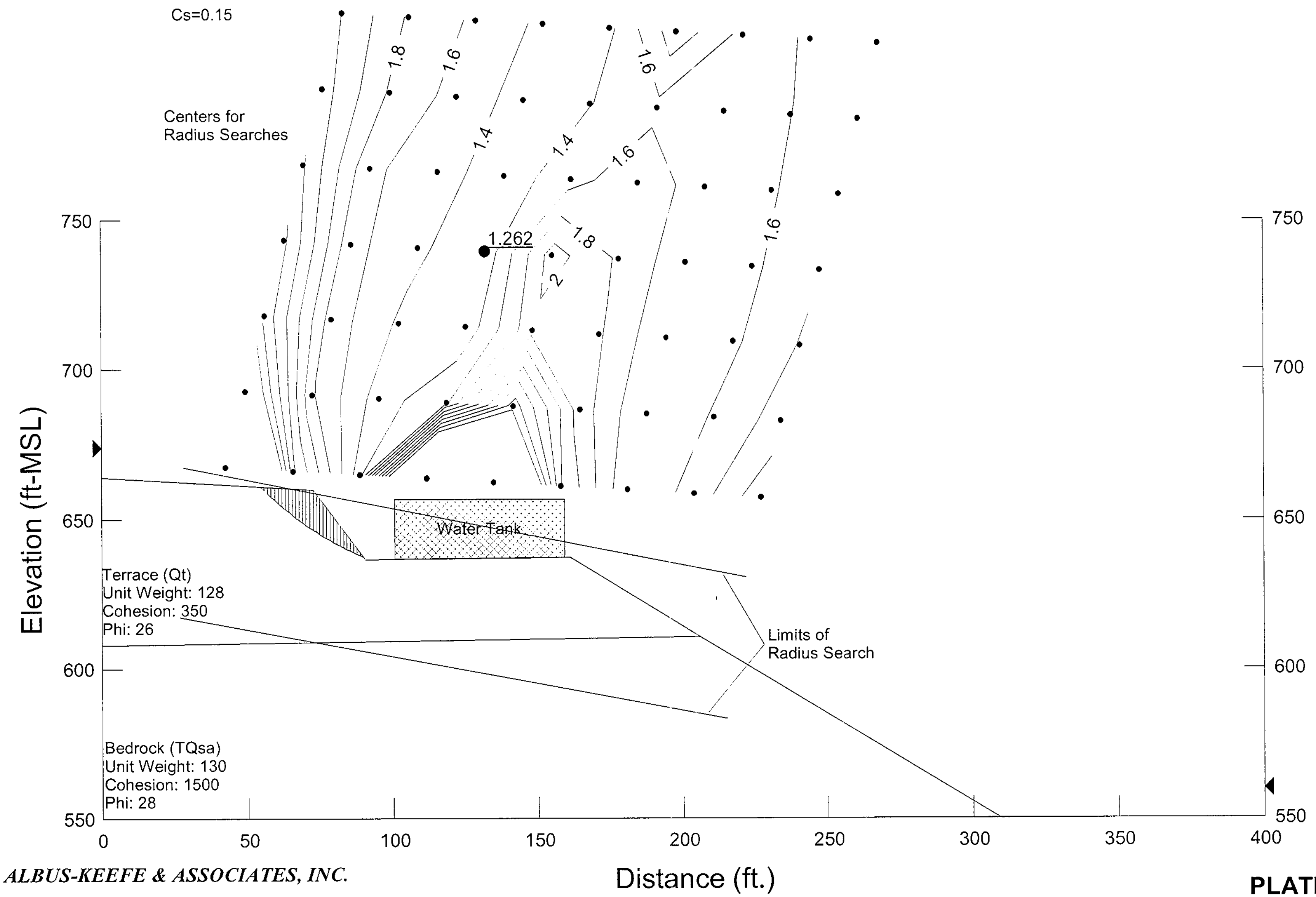
Section F - Tank Slope - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489_F.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D-9

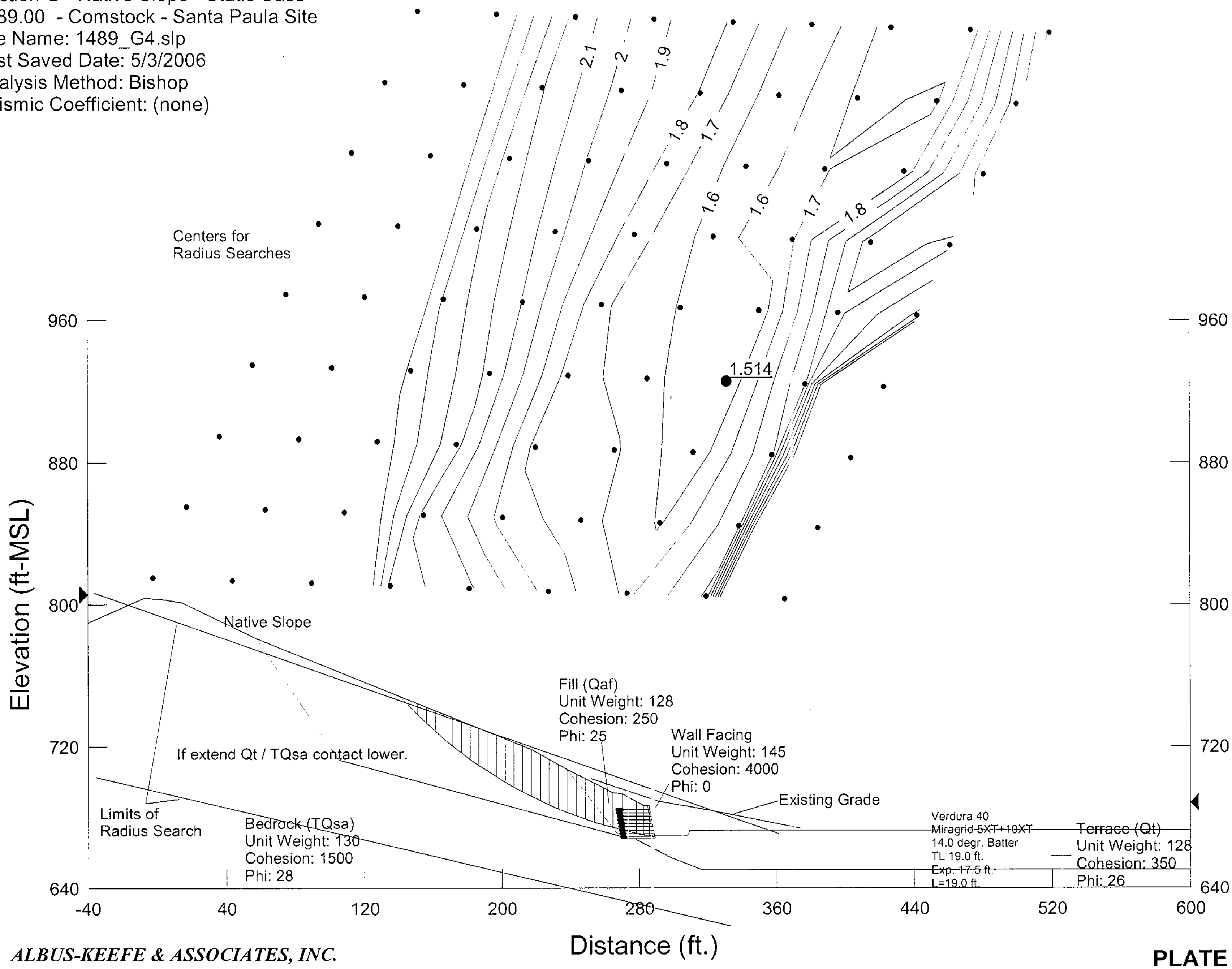
Section F - Tank Slope - Seismic Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489_Fs.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: Horizontal



ALBUS-KEEFE & ASSOCIATES, INC.

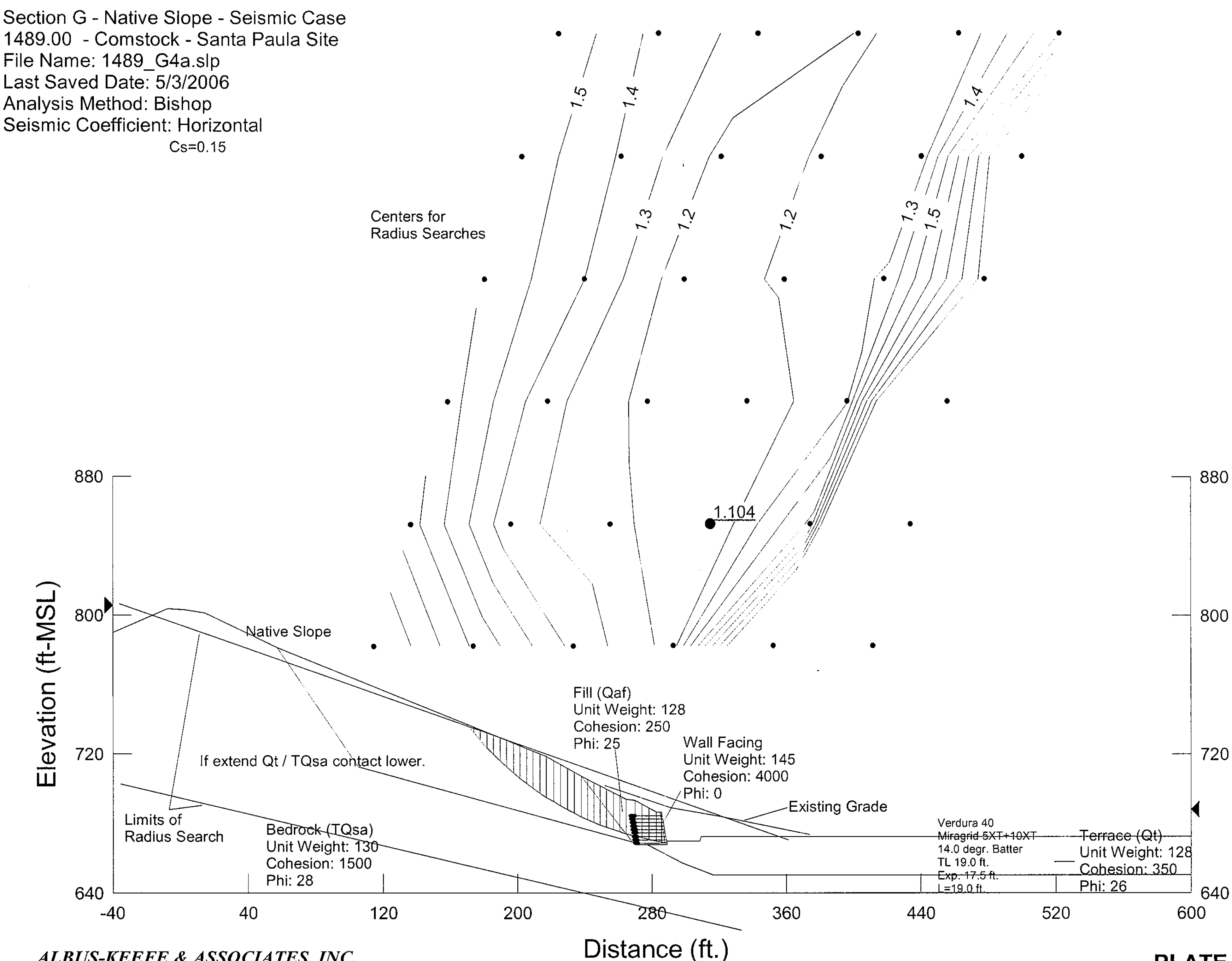
PLATE D-10

Section G - Native Slope - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489_G4.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D-11

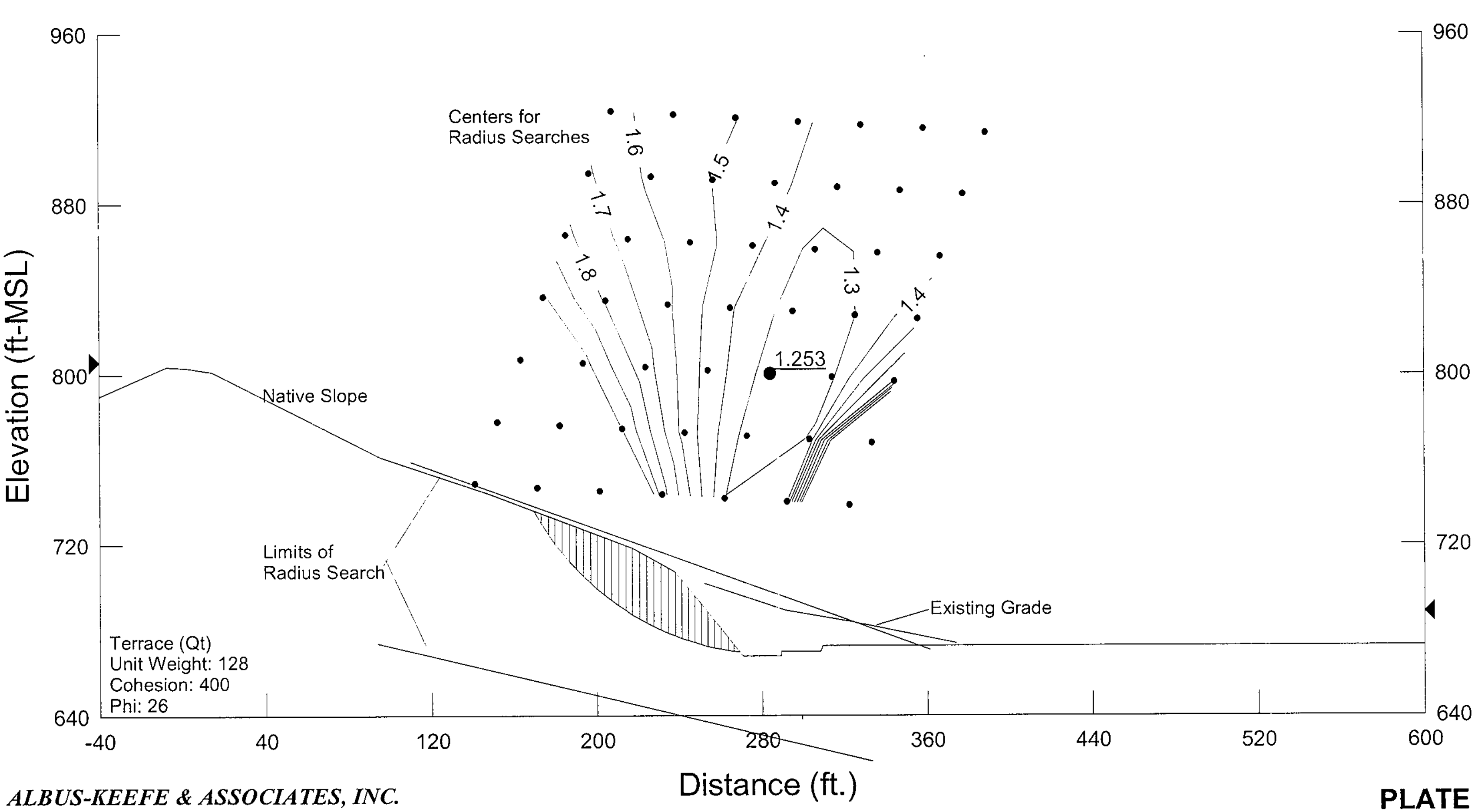


ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D-12

Section G - Native Slope - Temporary Cut
1489.00 - Comstock - Santa Paula Site
File Name: 1489_G3.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: (none)

IF ALL Qt



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE D–13

APPENDIX E

SETTLEMENT AND PILE ANALYSES

SETTLEMENT CALCULATIONS

J.N.	1489.00
Client:	Comstock Homes
Location:	Typical Footing on Qt

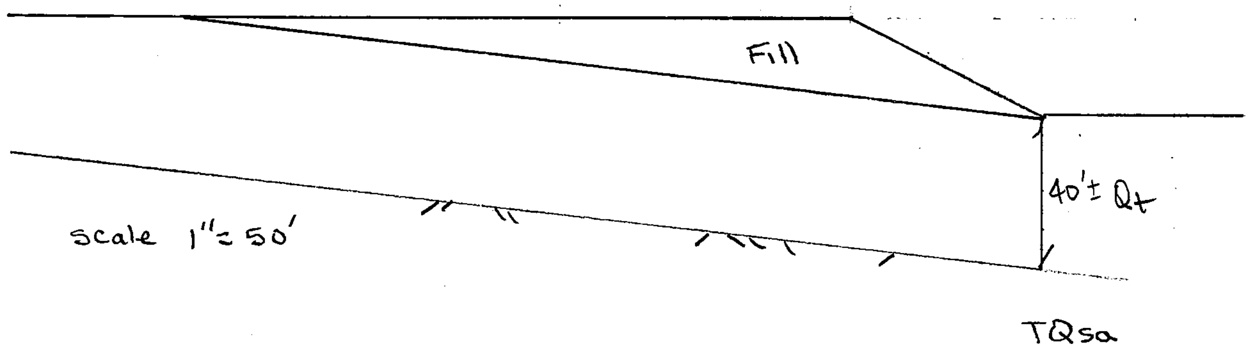
Rectangular Load (NAVFAC DM 7.1-165, Boussinesq)

Soil Density (psf)	125
Sat. Density (psf)	125
Depth to Water (ft)	100
Footing Depth (ft)	1.5
Footing width (ft)	1.25
Footing length (ft)	65
Bearing pressure (psf)	2000
Layer Thickness (ft)	1
Starting Depth (ft)	1.5
Rigidity Factor	0.7
Overconsolidation Pressure (psf)	1875

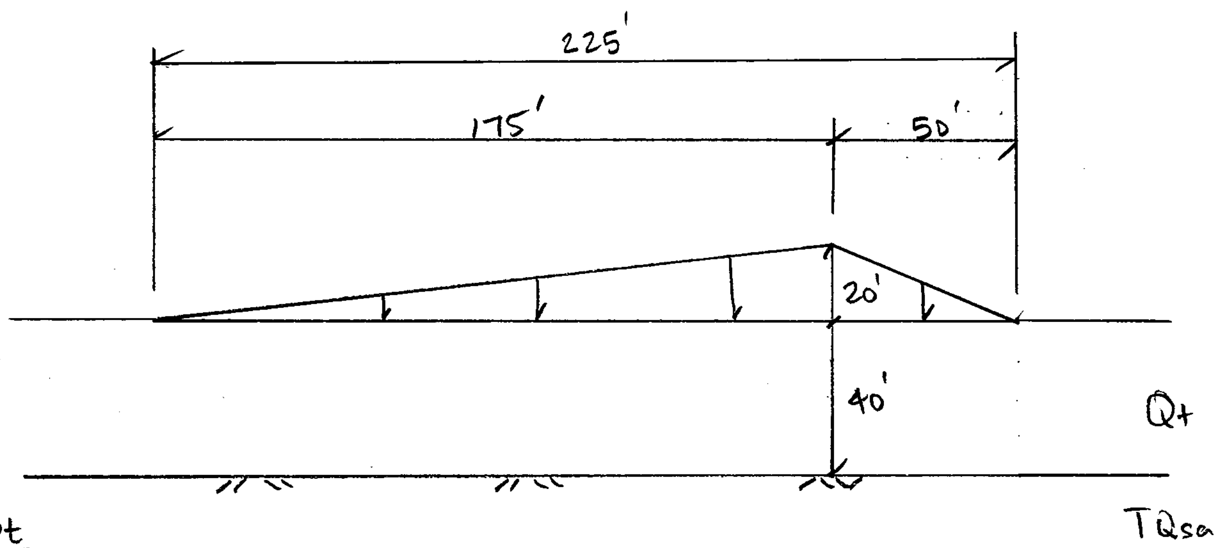
Depth (ft)	Sigma o (psf)	Delta Sigma (psf)	Sigma f (psf)	Pc	Ce	Cc	Sett. (in)	Cumul. Sett. (in)
2	250	814	1064	2125	0.008	0.057	0.04	0.04
3	375	545	920	2250	0.008	0.057	0.03	0.07
4	500	406	906	2375	0.008	0.057	0.02	0.09
5	625	322	947	2500	0.008	0.057	0.01	0.10
6	750	267	1017	2625	0.008	0.057	0.01	0.11
7	875	228	1103	2750	0.008	0.057	0.01	0.11
8	1000	199	1199	2875	0.008	0.057	0.01	0.12

Settlement Analysis @ Deepest Fill

Cross Section at Lot 66



Model at Lot 66



Qt

0-7'	C_c = 0.057	C_r = 0.007	OC + 3000 psf
7-40	C_c = 0.057	C_r = 0.007	OC + 1875 psf

TQsa non-compressible

SETTLEMENT CALCULATIONS

J.N.	1489.00
Client:	Comstock Homes
Location:	20' Fill at Lot 66

Triangular Load (NAVFAC DM 7.1-165, Boussinesq)

Soil Density (psf)	125
Sat. Density (psf)	125
Depth to Water (ft)	100
Width of Load Base (ft)	225
Distance to Peak of Load from Left (	175
Distance to Point from Left (ft)	145
Peak Pressure (psf)	2540
Layer Thickness (ft)	2
Starting Depth (ft)	0
Rigidity Factor	1
Overconsolidation Pressure (psf)	1875

Depth (ft)	Sigma o (psf)	Delta Sigma (psf)	Sigma f (psf)	Pc	Ce	Cc	Sett. (in)	Cumul. Sett. (in)
1	125	2106	2231	2000	0.008	0.057	0.30	0.30
3	375	2105	2480	2250	0.008	0.057	0.21	0.50
5	625	2105	2730	2500	0.008	0.057	0.17	0.67
7	875	2103	2978	2750	0.008	0.057	0.14	0.81
9	1125	2101	3226	3000	0.008	0.057	0.12	0.94
11	1375	2097	3472	3250	0.008	0.057	0.11	1.05
13	1625	2092	3717	3500	0.008	0.057	0.10	1.15
15	1875	2086	3961	3750	0.008	0.057	0.09	1.24
17	2125	2078	4203	4000	0.008	0.057	0.08	1.32
19	2375	2069	4444	4250	0.008	0.057	0.08	1.40
21	2625	2058	4683	4500	0.008	0.057	0.07	1.47
23	2875	2046	4921	4750	0.008	0.057	0.06	1.53
25	3125	2033	5158	5000	0.008	0.057	0.06	1.59
27	3375	2018	5393	5250	0.008	0.057	0.05	1.64
29	3625	2003	5628	5500	0.008	0.057	0.05	1.69
31	3875	1986	5861	5750	0.008	0.057	0.04	1.73
33	4125	1969	6094	6000	0.008	0.057	0.04	1.77
35	4375	1951	6326	6250	0.008	0.057	0.04	1.81
37	4625	1933	6558	6500	0.008	0.057	0.03	1.84
39	4875	1913	6788	6750	0.008	0.057	0.03	1.87

SETTLEMENT CALCULATIONS

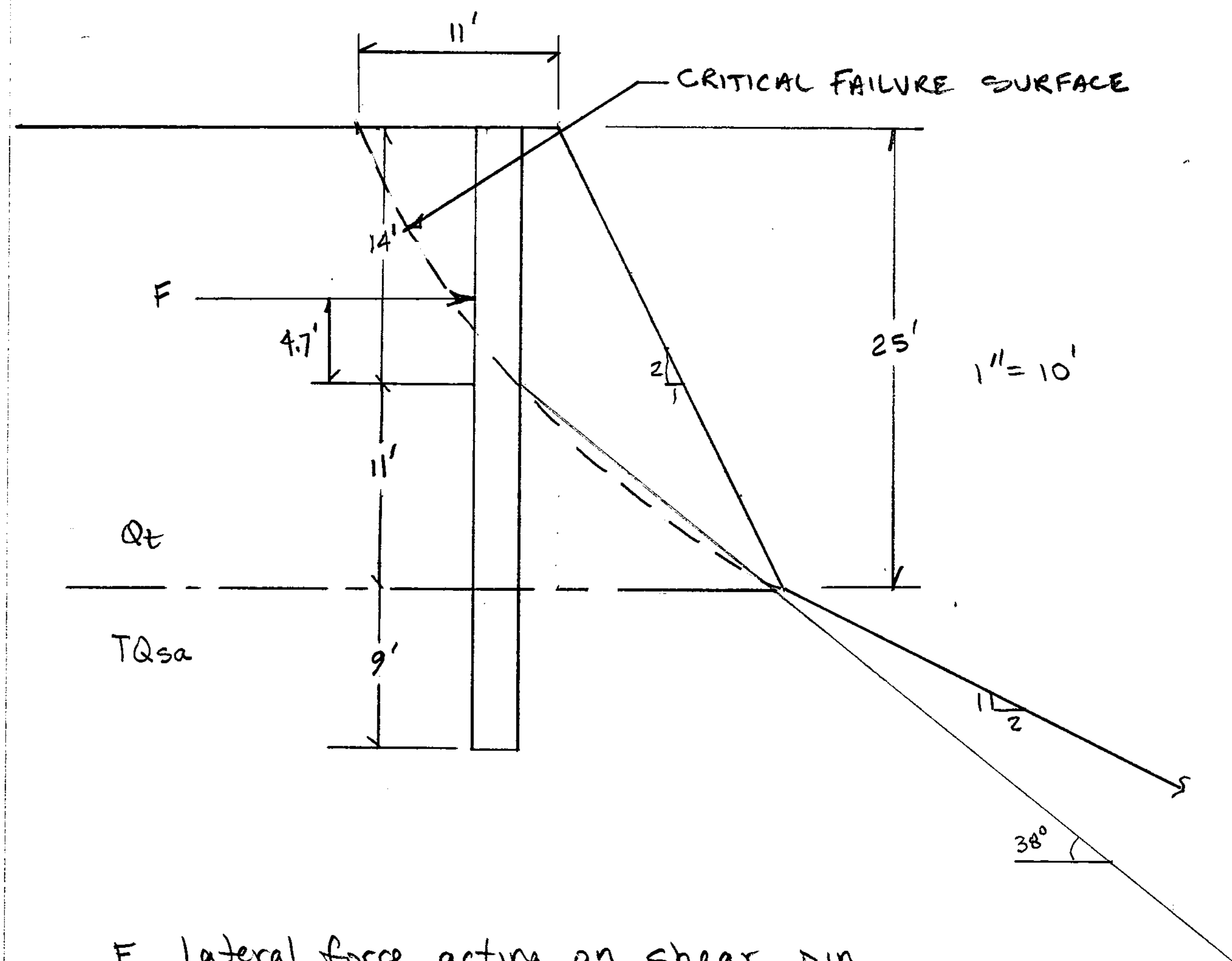
J.N.	1489.00
Client:	Comstock Homes
Location:	20' Fill at Lot 66

Triangular Load (NAVFAC DM 7.1-165, Boussinesq)

Soil Density (psf)	125
Sat. Density (psf)	125
Depth to Water (ft)	100
Width of Load Base (ft)	225
Distance to Peak of Load from Left (	145
Distance to Point from Left (ft)	145
Peak Pressure (psf)	2540
Layer Thickness (ft)	2
Starting Depth (ft)	0
Rigidity Factor	1
Overconsolidation Pressure (psf)	1875

Depth (ft)	Sigma o (psf)	Delta Sigma (psf)	Sigma f (psf)	Pc	Ce	Cc	Sett. (in)	Cumul. Sett. (in)
1	125	2526	2651	2000	0.008	0.057	0.40	0.40
3	375	2494	2869	2250	0.008	0.057	0.29	0.69
5	625	2463	3088	2500	0.008	0.057	0.24	0.93
7	875	2432	3307	2750	0.008	0.057	0.21	1.14
9	1125	2401	3526	3000	0.008	0.057	0.18	1.32
11	1375	2370	3745	3250	0.008	0.057	0.16	1.47
13	1625	2339	3964	3500	0.008	0.057	0.14	1.61
15	1875	2308	4183	3750	0.008	0.057	0.12	1.73
17	2125	2278	4403	4000	0.008	0.057	0.11	1.84
19	2375	2247	4622	4250	0.008	0.057	0.10	1.94
21	2625	2217	4842	4500	0.008	0.057	0.09	2.03
23	2875	2188	5063	4750	0.008	0.057	0.08	2.11
25	3125	2158	5283	5000	0.008	0.057	0.07	2.18
27	3375	2129	5504	5250	0.008	0.057	0.06	2.25
29	3625	2100	5725	5500	0.008	0.057	0.06	2.30
31	3875	2072	5947	5750	0.008	0.057	0.05	2.36
33	4125	2044	6169	6000	0.008	0.057	0.05	2.41
35	4375	2016	6391	6250	0.008	0.057	0.04	2.45
37	4625	1989	6614	6500	0.008	0.057	0.04	2.49
39	4875	1962	6837	6750	0.008	0.057	0.03	2.52

SHEAR PIN DESIGN



F, lateral force acting on shear pin, is greater of unbalance lateral force to obtain slope stability factor of safety ≥ 1.5 or active earth pressure from upper 14' if outter wedge slides away.

Per stability analysis, F = 3,850 lbs/ft for FS = 1.5 (see attached plot, Plates E-7 & E-8)

For active earth pressure:

use 40 pcf

$$F = \frac{40 \cdot 14^2}{2} = 3{,}920 \text{ lbs/ft}$$

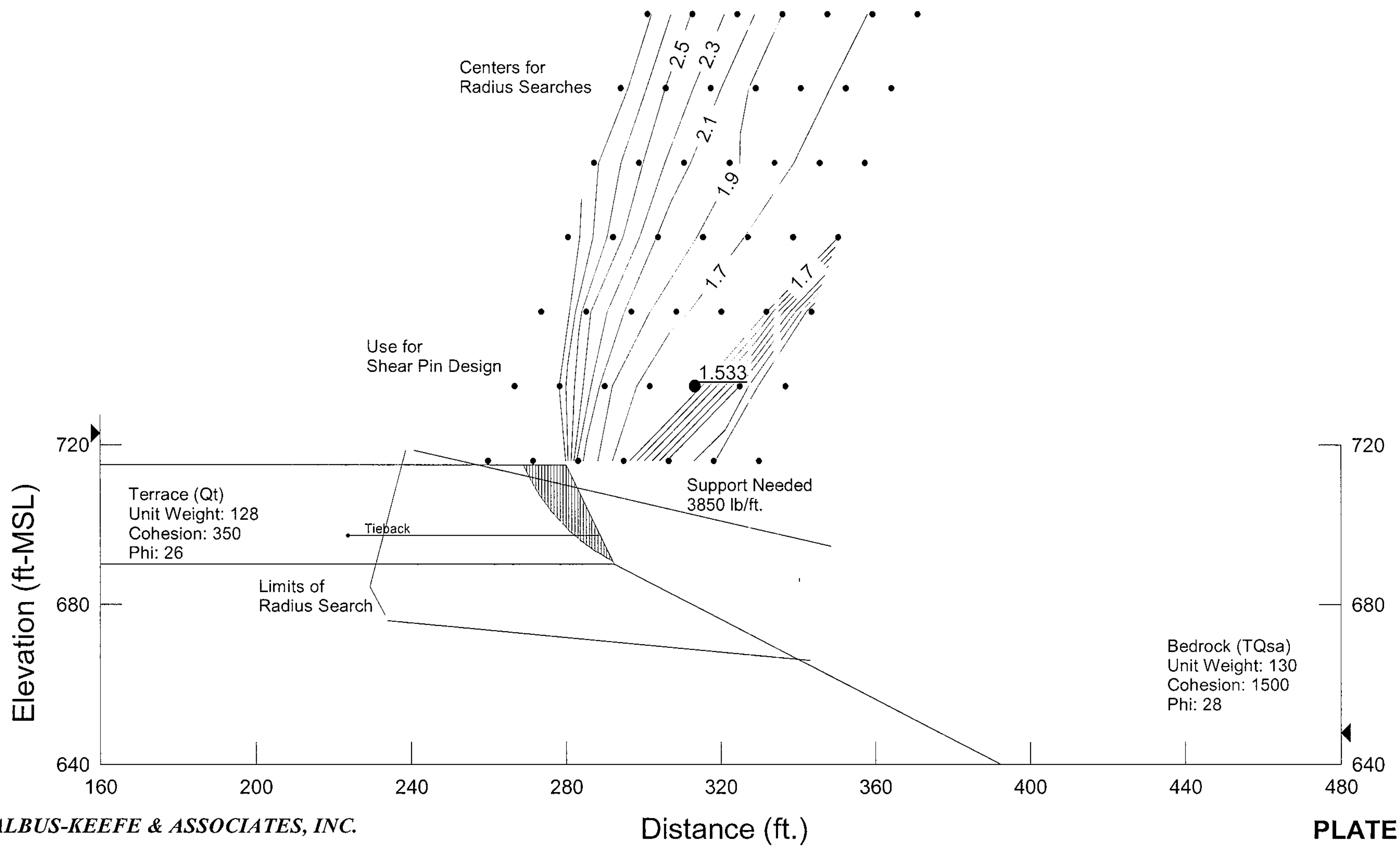
active pressure is greater ∴ use 3,920 lbs/ft

For pile spacing of 10' c/c

Horz. load per pile, F_p = $3920 \cdot 10 \approx 40^k$

For 30" concrete pile, 34' long, deflection is 0.8" OK
(see attached Plate E-9 for deflection analysis)

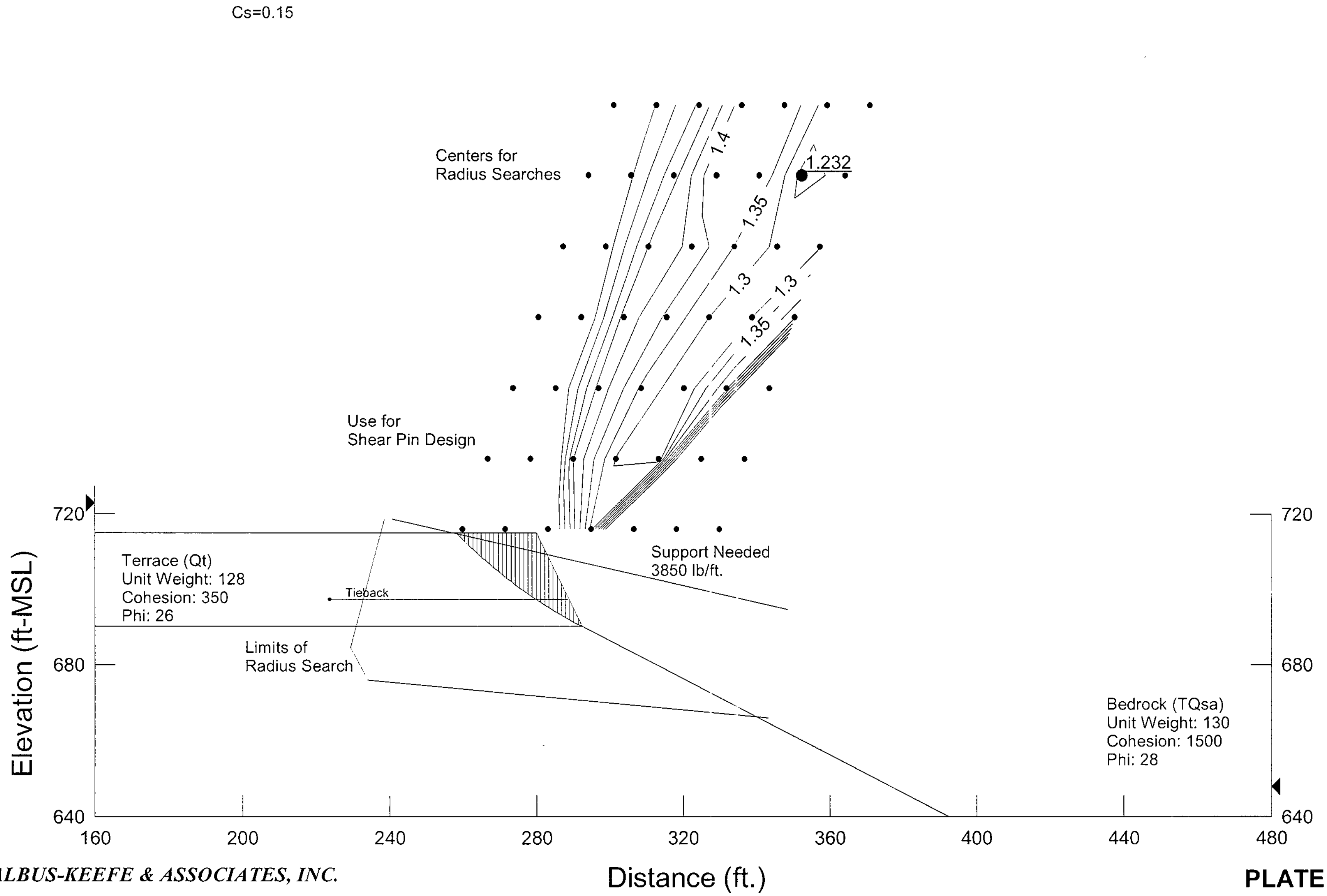
Shear Pin Design - Required Support - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489_SP1.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE E-7

Shear Pin Design - Required Support - Seismic Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489_SP1s.slp
Last Saved Date: 5/3/2006
Analysis Method: Bishop
Seismic Coefficient: Horizontal



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE E-8

Deflection of Pile Adjacent Slope (Poulos 1976)

J.N.	1489.00
Client:	Comstock Homes
Description:	Bluff Shear Pins

Pile Diameter (ft)	2.50
Pile Length (ft)	20.00
Soil Modulus (psi)	2484.00
Unit weight (pcf)	128.00
Phi (deg.)	28.00
Pile Modulus (psi)	3.00E+06
Distance from Slope (ft)	1.00
Slope Angle from horz (deg)	38.00
H (kips)	40.00
Hu (kips)	141.74
H/Hu	0.28
Eccentricity (ft)	5.00

SPT (ML)	45	E (psi) =	2484
SPT (SP)	0	E (psi) =	0
SPT (SW)	0	E (psi) =	0
SPT (GP/GW)	0	E (psi) =	0
Qc (TONS/FT^2)	0	E (psi) =	0
Rel. den. (%)	75	E (psi) =	2967

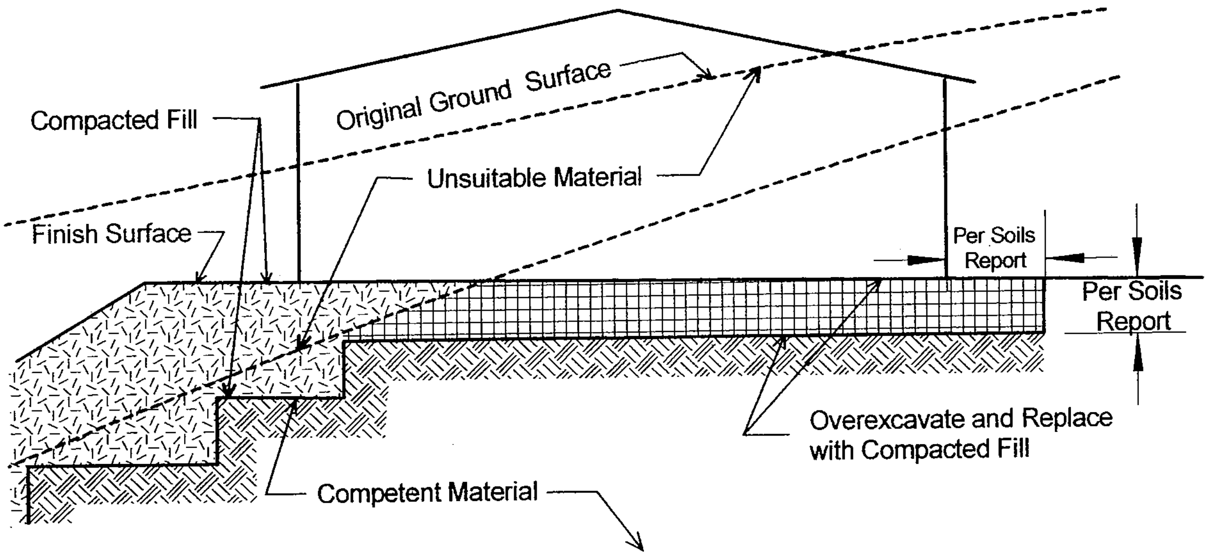
Moment of Inertia (ft^4)	1.92
Kr	0.01447
L/d	8.00
x/d	0.40
IpH	4.00
CpH	1.40
IpM	8.00
CpM	1.50
Fp	0.70

Deflection (in.)	0.8243

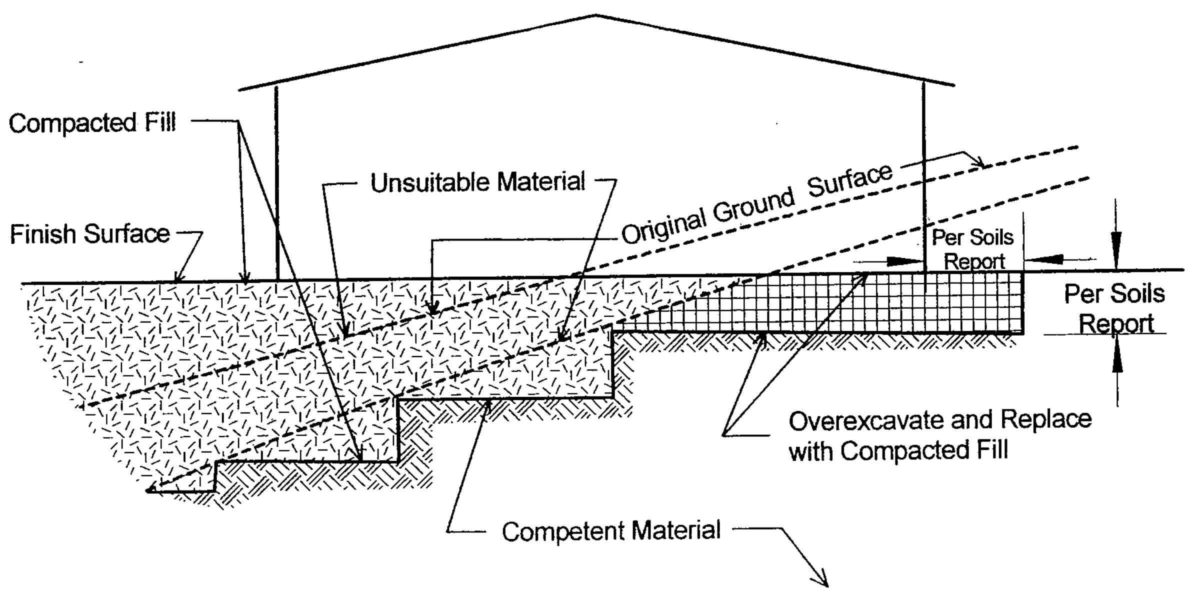
APPENDIX F

TYPICAL GRADING DETAIL

CUT TRANSITION LOT



CUT/FILL TRANSITION LOT

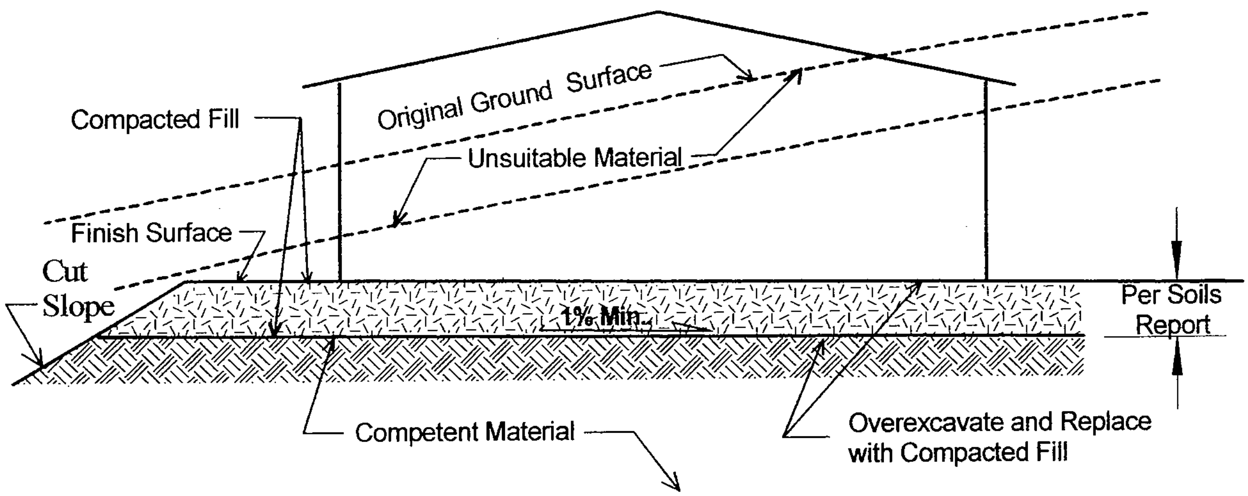


TRANSITION LOT CAPPING

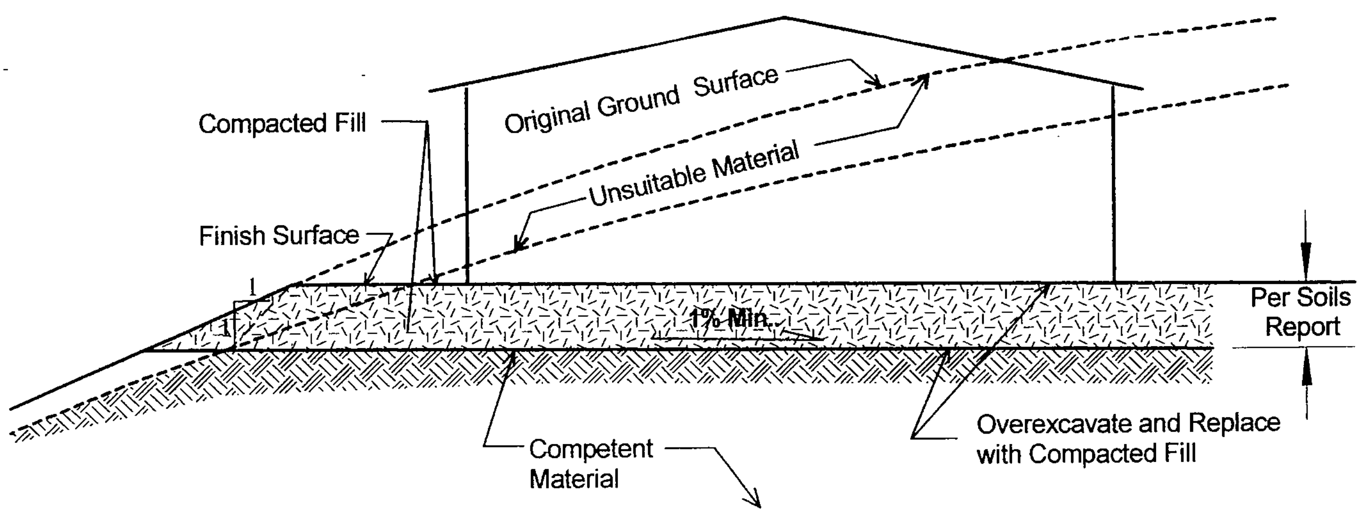
Not To Scale
See attached soils report for specific recommendations.

Detail No. 18

CUT LOT



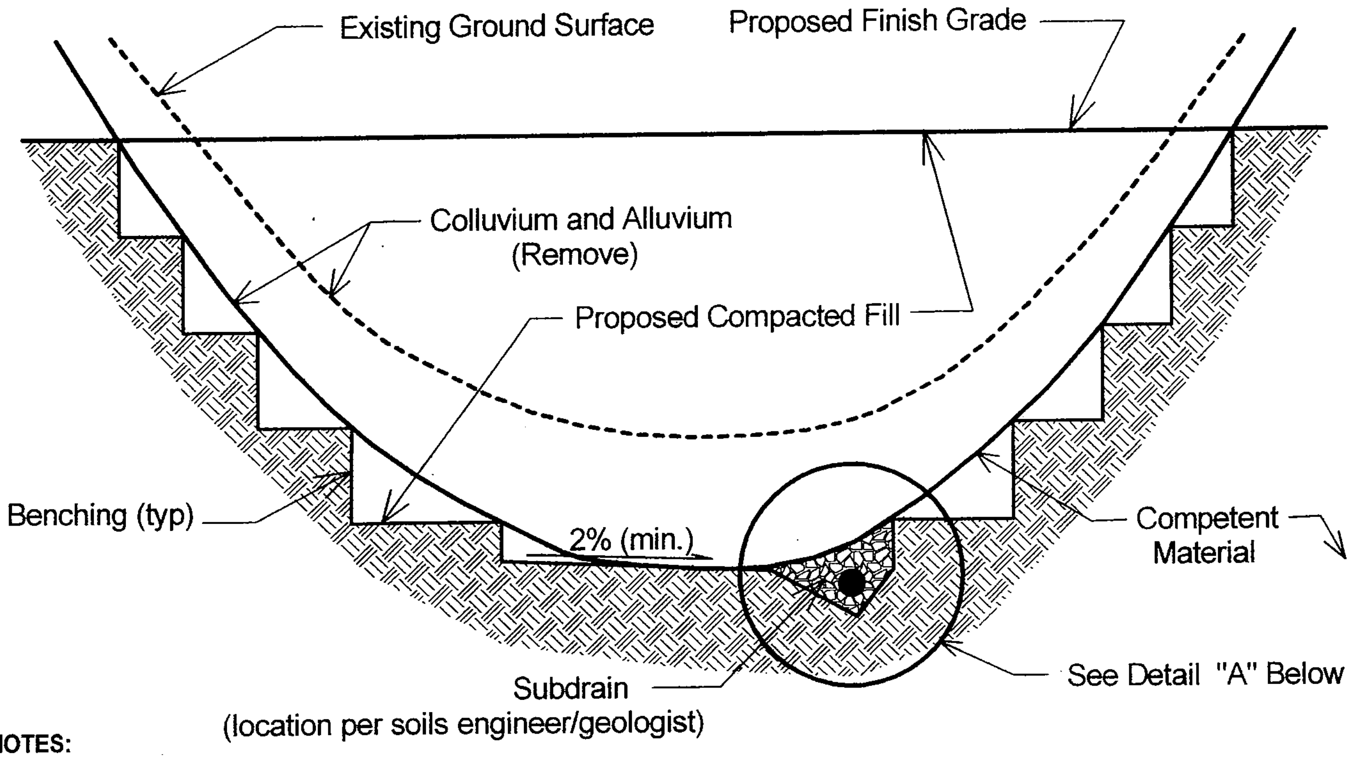
DAYLIGHT CUT LOT



CUT LOT CAPPING

Not To Scale
See attached soils report for specific recommendations.

Detail No. 17



NOTES:

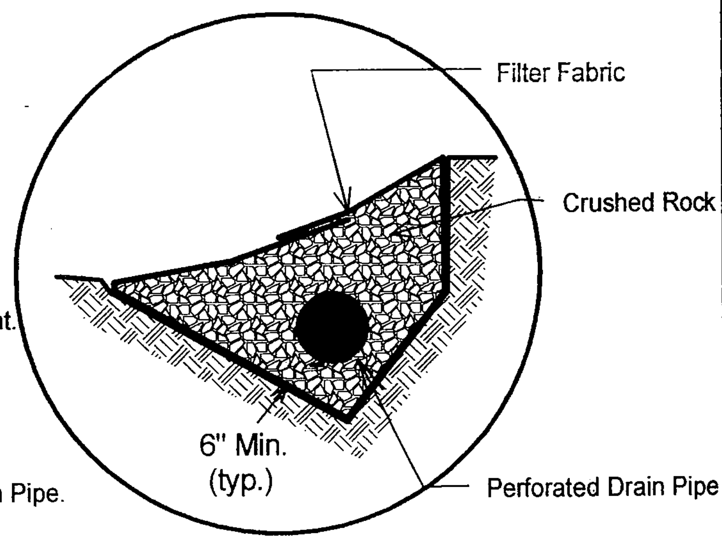
Perforated Drain Pipe should be at least 6 inches in diameter, or 8 inches for runs of more than 500 feet, consisting of either Shedule 40 PVC or SDR 35.
A min. of 8 perforations per linear foot should be provided along the bottom of pipe. The perforations should be in two rows 120 degrees apart, and have diameters of 3/8".
Upstream ends should be provided with a cap. The pipe should slope at a min. 1% gradient toward Outlet Pipes.
Glue all joints.

Outlet Pipe should equal the diameter of the perforated pipe, consisting of either shedule 40 PVC or SDR 35. The pipe should slope at a min. 1% gradient toward the Discharge Point.
Backfill around Outlet Pipe should consist of onsite soils.
Glue all joints.

Crushed Rock should conform to the Standard Specifications for Public Works Construction, Section 200-1.2, for 3/4".
Provide at least 9 cubic feet per lineal foot of Perforated Drain Pipe.
Provide at least 6 inches of gravel below Perforated Pipe.

Filter Fabric should consist of Mirifi 140N or equivelent.
Ends should overlap at least 12 inches.

CALTRANS Class II Permeable Filter Material can be used in lieu of Crushed Rock encased in Filter Fabric



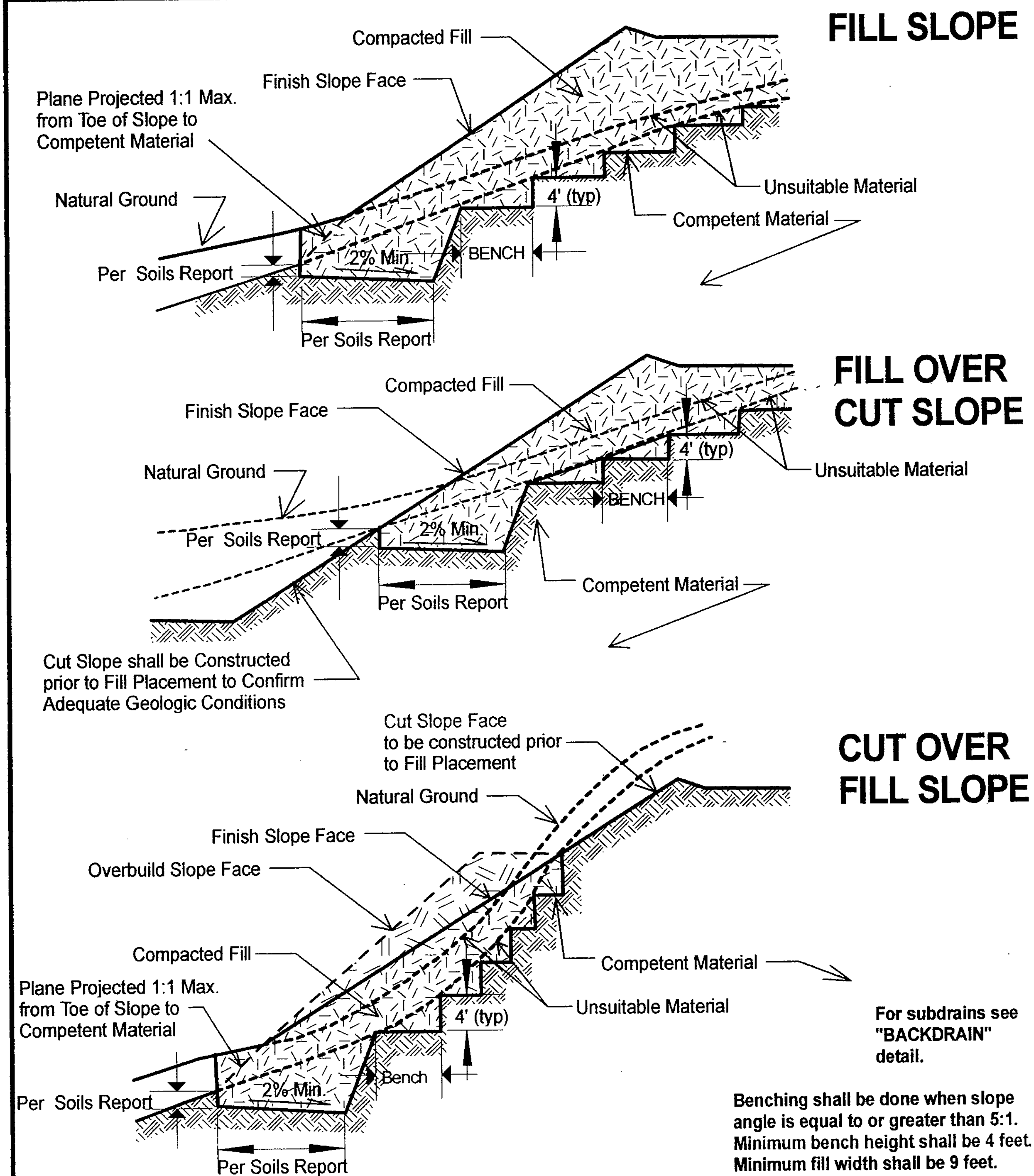
DETAIL "A"

CANYON SUBDRAIN

Not To Scale
See attached soils report for specific recommendations.

Detail No. 7b

ALBUS-KEEFE & ASSOCIATES, INC.



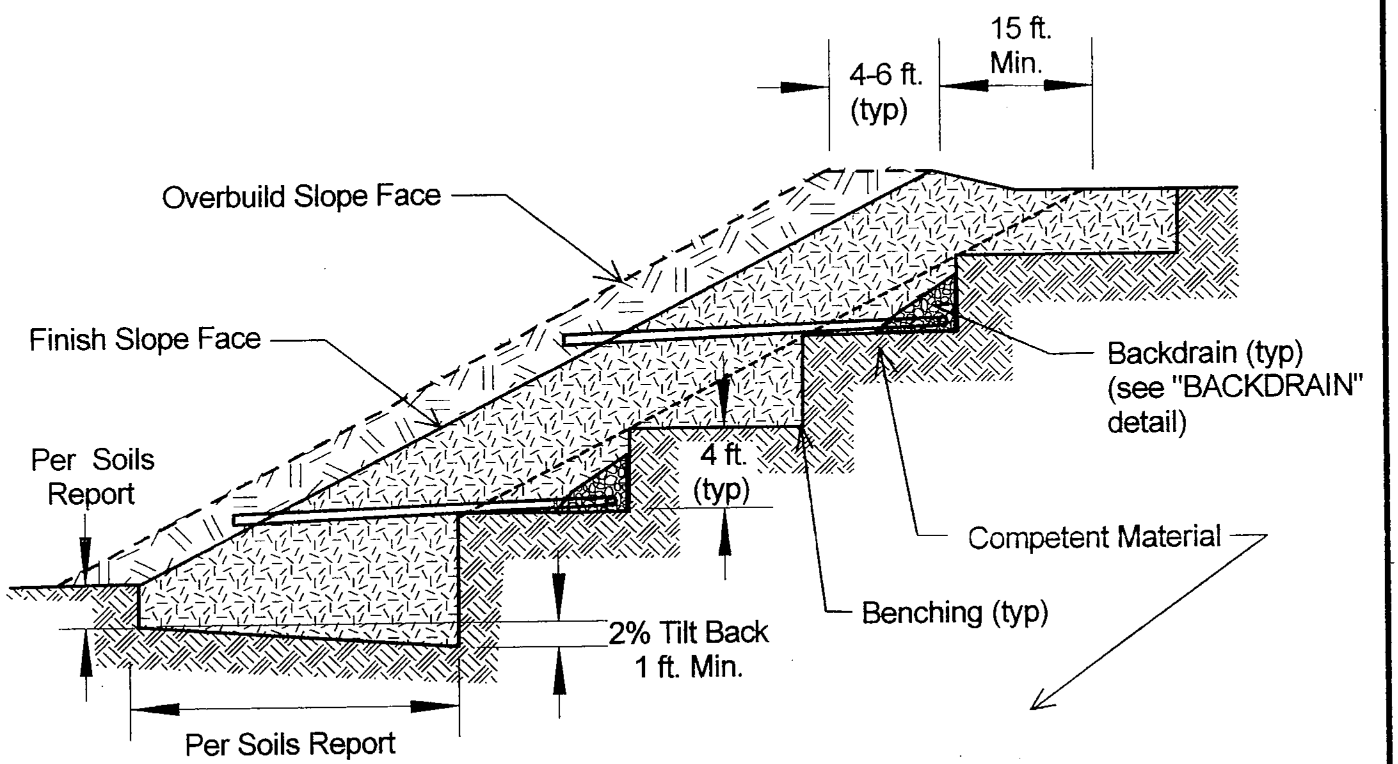
For subdrains see "BACKDRAIN" detail.

Benching shall be done when slope angle is equal to or greater than 5:1. Minimum bench height shall be 4 feet. Minimum fill width shall be 9 feet.

FILL SLOPE CONSTRUCTION

Not To Scale
See attached soils report for specific recommendations.

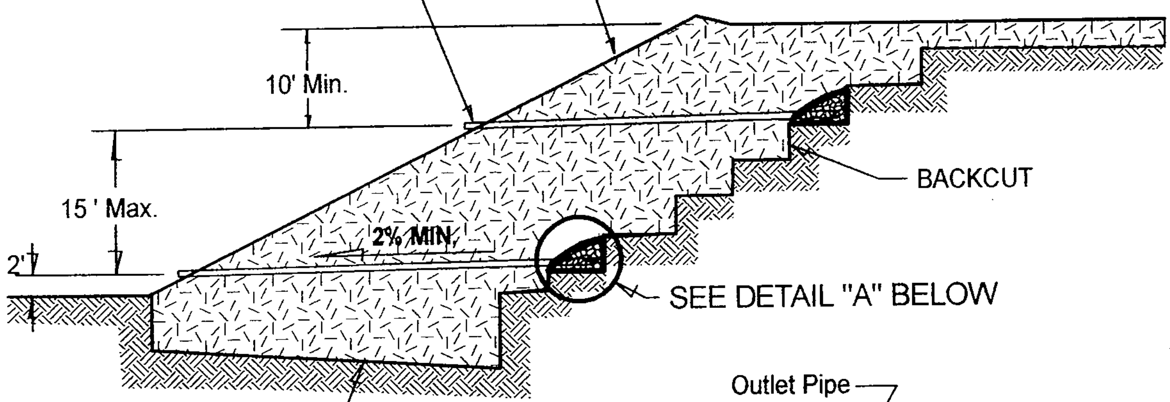
Detail No. 1

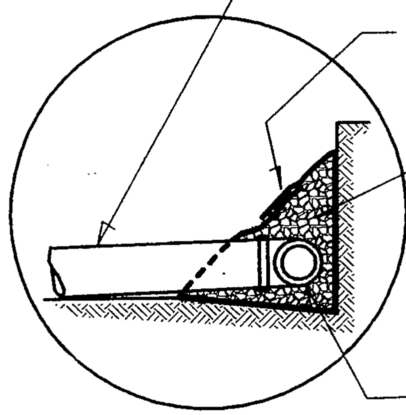


BUTTRESS/STABILIZATION FILL

Not To Scale
See attached soils report for specific recommendations.

Detail No. 3a





DETAIL "A"

NOTES:

Perforated Drain Pipe should be at least 4 inches in diameter consisting of either Shedule 40 PVC or SDR 35. A min. of 8 perforations per linear foot should be provided along the bottom of pipe. Upstream ends should be provided with a cap. The pipe should slope at a min. 1% gradient toward Outlet Pipes.

Outlet Pipe should be at least 4 inches in diameter consisting of either shedule 40 PVC or SDR 35. The pipe should slope at a min. 2% gradient toward slope face. Backfill around Outlet Pipe should consist of onsite soils. Provide Outlet Pipe for each 100 feet of Perforated Drain Pipe. Glue all joints. Extend Outlet Pipe 1 foot beyond Finish Slope Face.

Crushed Rock should conform to the Standard Specifications for Public Works Construction, Section 200-1.2, for 3/4". Provide at least 4 cubic feet per lineal foot of Perforated Drain Pipe. Provide at least 4 inches of gravel below perforated pipe.

Filter Fabric should consist of Mirifi 140N or equivelent. Ends should overlap at least 12 inches.

CALTRANS Class II Permeable Filter Material can be used in lieu of Crushed Rock encased in Filter Fabric.

BACKDRAIN

Not To Scale
See attached soils report for specific recommendations.

Detail No. 8a

APPENDIX C

ALBUS-KEEFE AND ASSOCIATES

Recommendations for Stabilizations of Upper Easterly Bluff
Proposed 75-Lot Residential Development
Western Terminus of 10th Street, Santa Paula, California
August 31, 2006

7290.200.101
March 10, 2008
Revised May 23, 2008

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

August 31, 2006
J.N.: 1489.00

Ms Tiffany Sukay
Comstock Homes
321 12th Street, Suite 200
Manhattan Beach, California 90266

Subject: **Recommendations for Stabilizations of Upper Easterly Bluff, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California**

References: *Second Response to city of Santa Paula Review, dated July 25, 2006, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California*, by Albus-Keefe & Associates, Inc., dated July 31, 2006

Supplemental Geotechnical Investigation and Rough Grading Plan Review, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated May 3, 2006

Dear Ms Sukay:

The upper portion of the easterly bluff has been recognized as not providing a factor of safety greater than 1.5 against gross failure. This condition was discussed in our referenced Supplemental Geotechnical Investigation Report. Subsequent responses to the city of Santa Paula have lead us to recommend removing the terrace deposits that comprise the upper bluff and replacing the natural slope with a 1 to 1 fill slope reinforced with geogrids. While a general feasibility evaluation was performed for our referenced Second Response, this document provides more supporting analyses and details on the recommended mitigation.

The grading plans have been revised by DRC to reflect the removal and replacement of the slope down to at least the daylight of the terrace deposits. The plan depicting these revisions has been attached herein as Plate 1. We have performed slope stability analyses of the conditions along the bluff using the computer program Slope/W. Details of the program and methodology were provided in our referenced Supplemental Geotechnical Investigation.

Three cross sections have been prepared to depict conditions along the bluff and were used for stability analyses. The analyses conservatively assume a weathered bedrock zone on the slope face below the terrace deposit that is 10 feet deep. Significant field mapping augmented by hand augers at selected locations suggest the thickness is probably no more than 7 feet. Shear strengths for compacted fill and intact bedrock were established in our referenced Supplemental Geotechnical Investigation

Report and used herein. Shear strengths for the weathered bedrock were based on reducing the cohesion of intact bedrock to a value equivalent to a soil material. Strengths used in our analyses are summarized below.

Summary of Shear Strengths

Material	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Compacted Fill (Qcaf)	128	250	25
Weathered Bedrock (TQsa)	128	300	28
Bedrock (TQsa)	130	1500	28

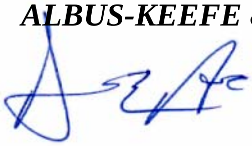
Our analysis indicates that global stability can be achieved with the installation of Mirifi 10XT geogrids having lengths that are approximately equal to the height of the 1 to 1 fill slope. Factors of safety for static and seismic cases are greater than 1.5 and 1.1, respectively. Results of our analyses are provided in Plate A-1 through A-6.

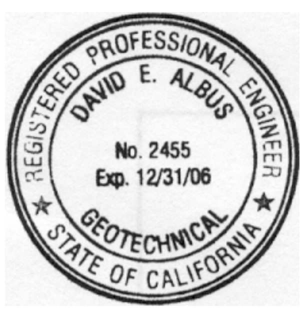
Facing stability will require the installation of additional geogrids. We are recommending the use of Mirifi 2XT geogrids that are embedded into the slope face 8 feet then wrap the outer slope face with a minimum embedment of 3 feet. These geogrids should be provided every 2.5 feet vertically. Our proposed layout is provided on Plate 1 and a schematic of the system is provided on Plate 2.

We appreciate this opportunity to be of continued service to you. If you have any questions regarding this correspondence, please feel free to call.

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

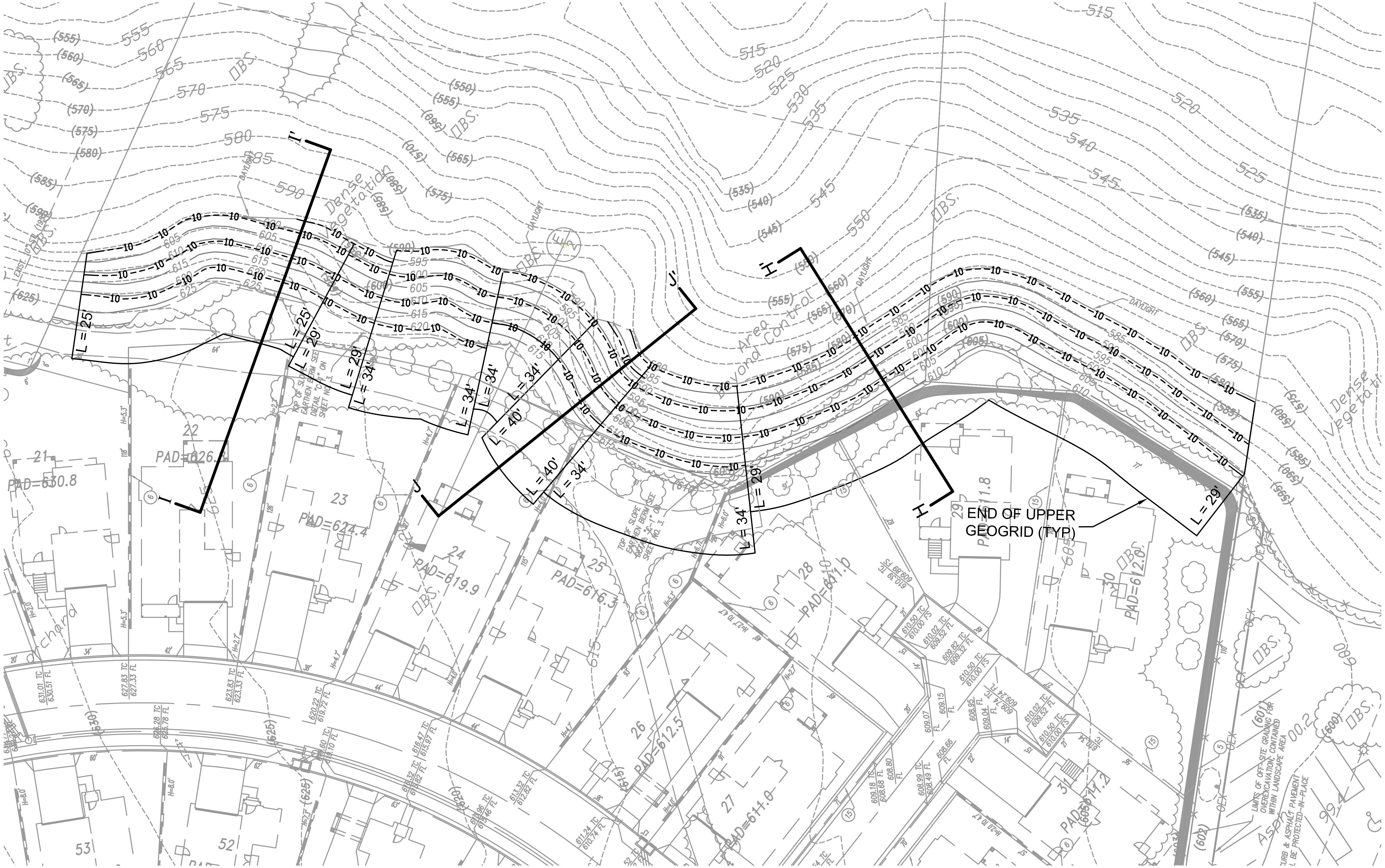


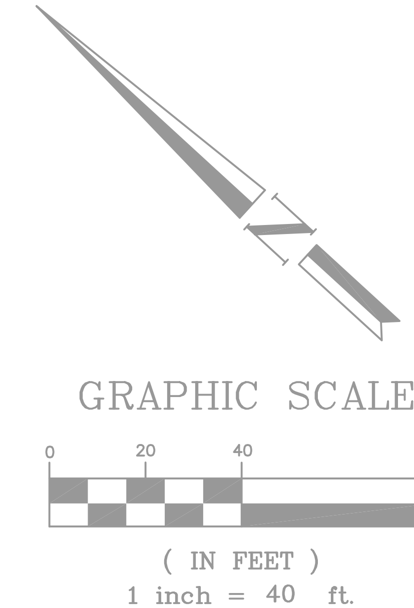


David E. Albus
Principal Engineer
GE 2455

Enclosures:

Plate 1- Geogrid Layout Map
Plate 2- Schematic Layout of Geogrid Slope
Plates A-1 through A-6- Stability Analyses for Geogrid Slope





EXPLANATION

- - -10- - -10- - Miragrid 10XT Geogrid

L = 40' - Length of Geogrid

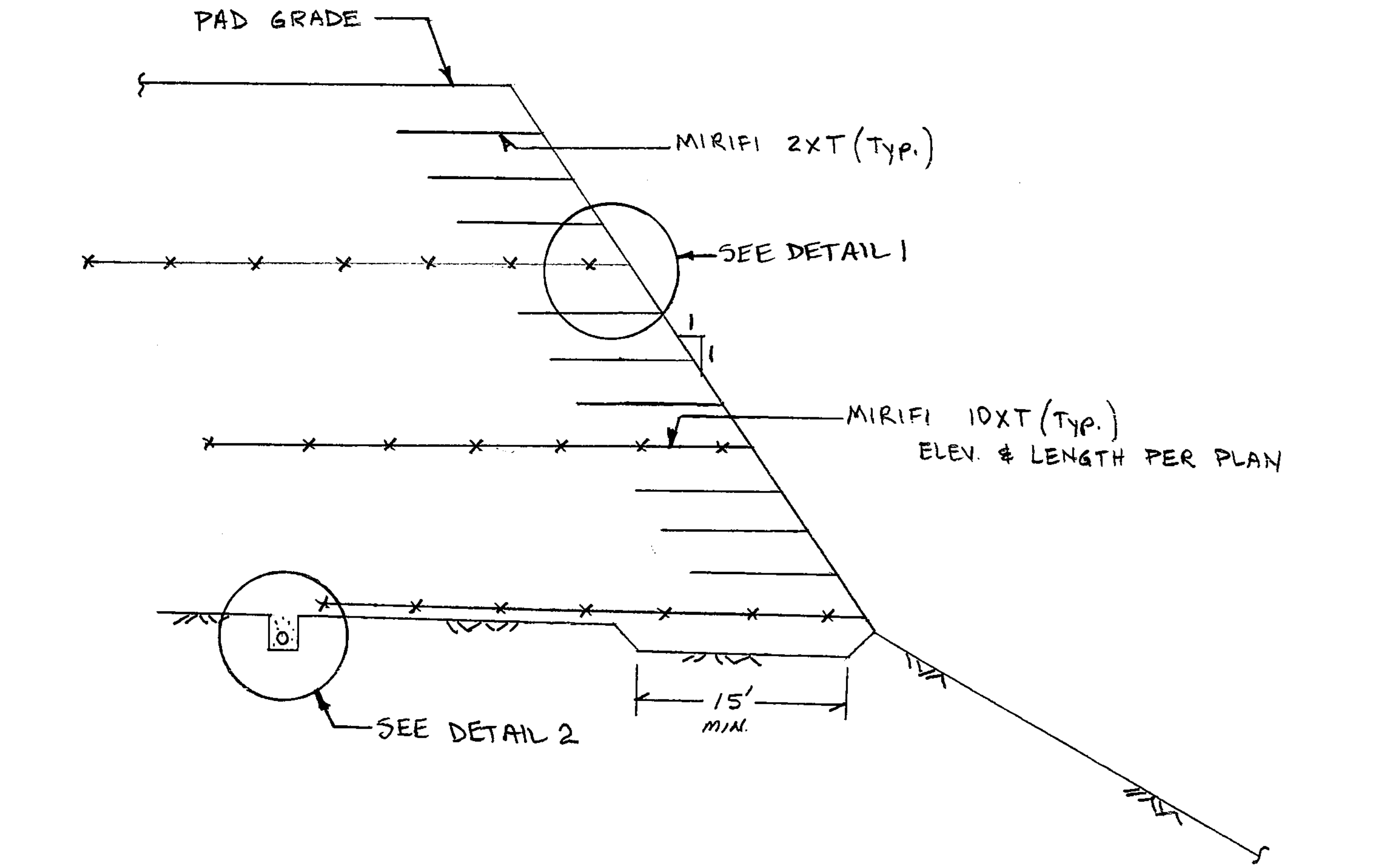
J J' - Geologic Cross-Section

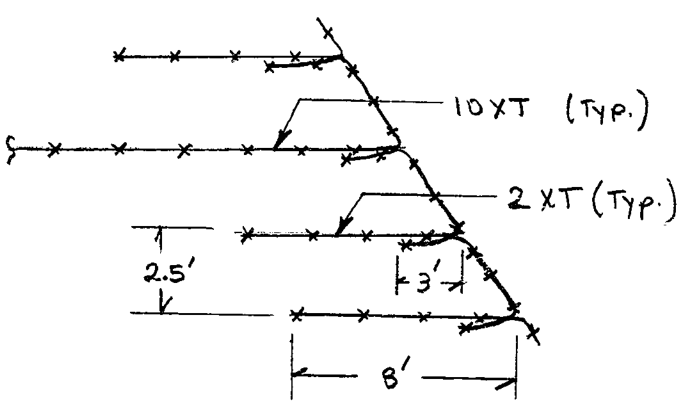
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOGRID SLOPE EXHIBIT

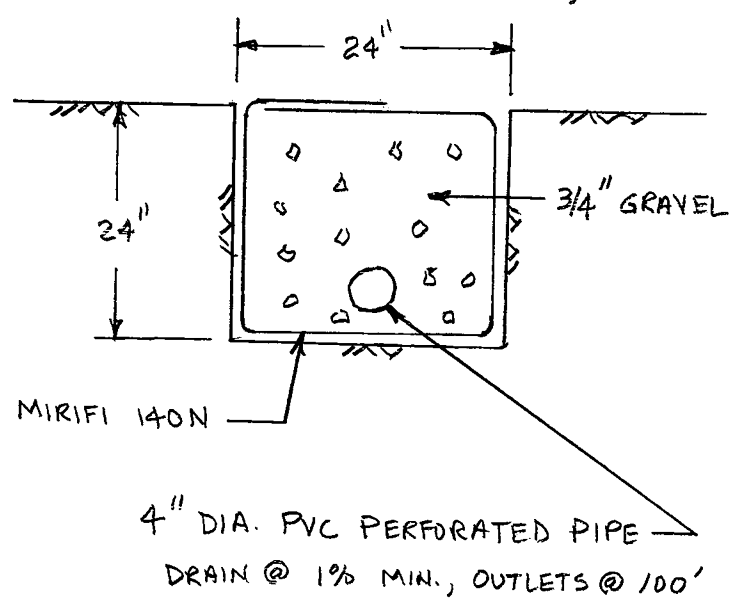
Job No.: 1489.00	Date: 08/31/06	Plate: 1

SCHEMATIC LAYOUT OF GEOGRID SLOPE





DETAIL 1



DETAIL 2

APPENDIX A

STABILITY ANALYSES

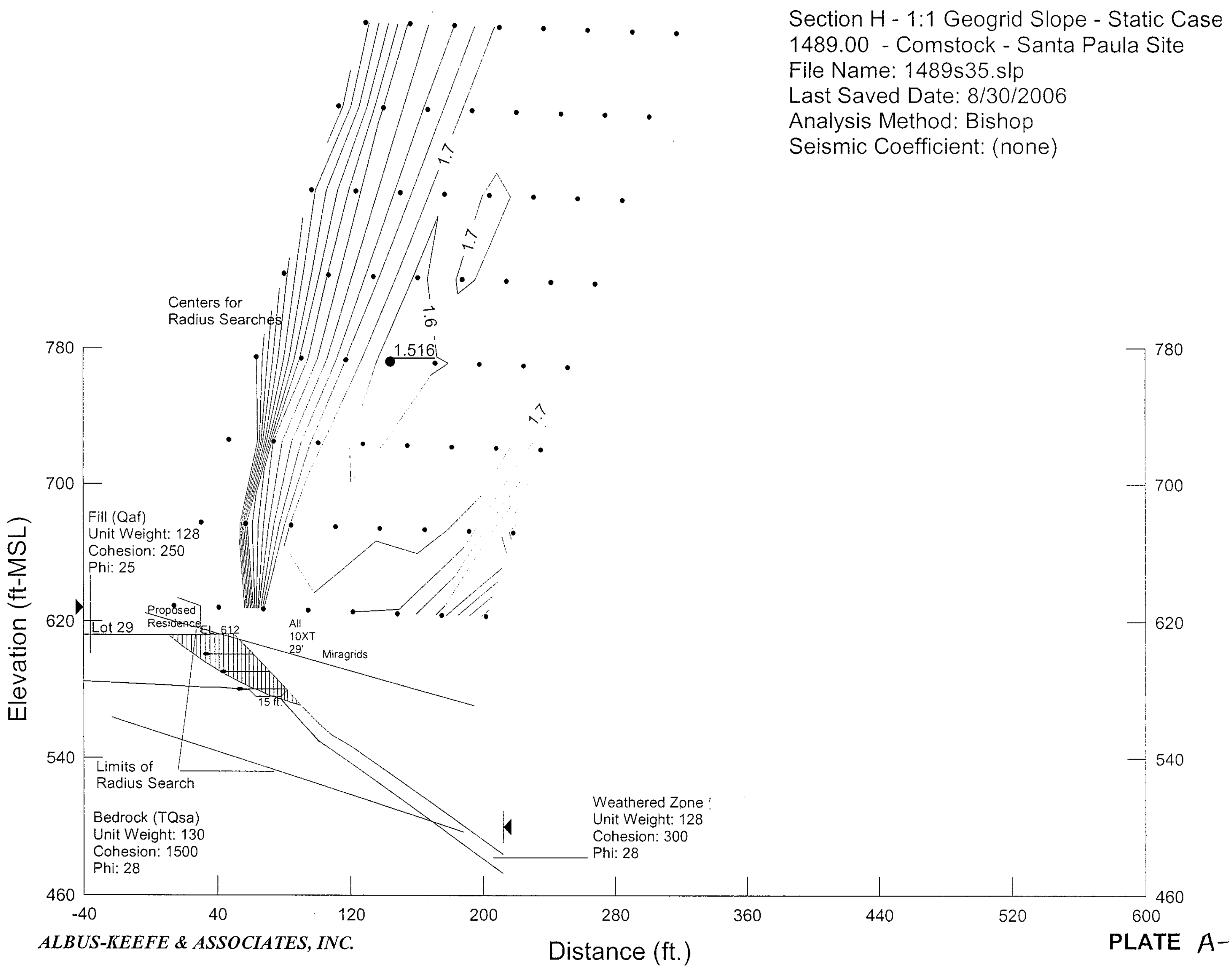


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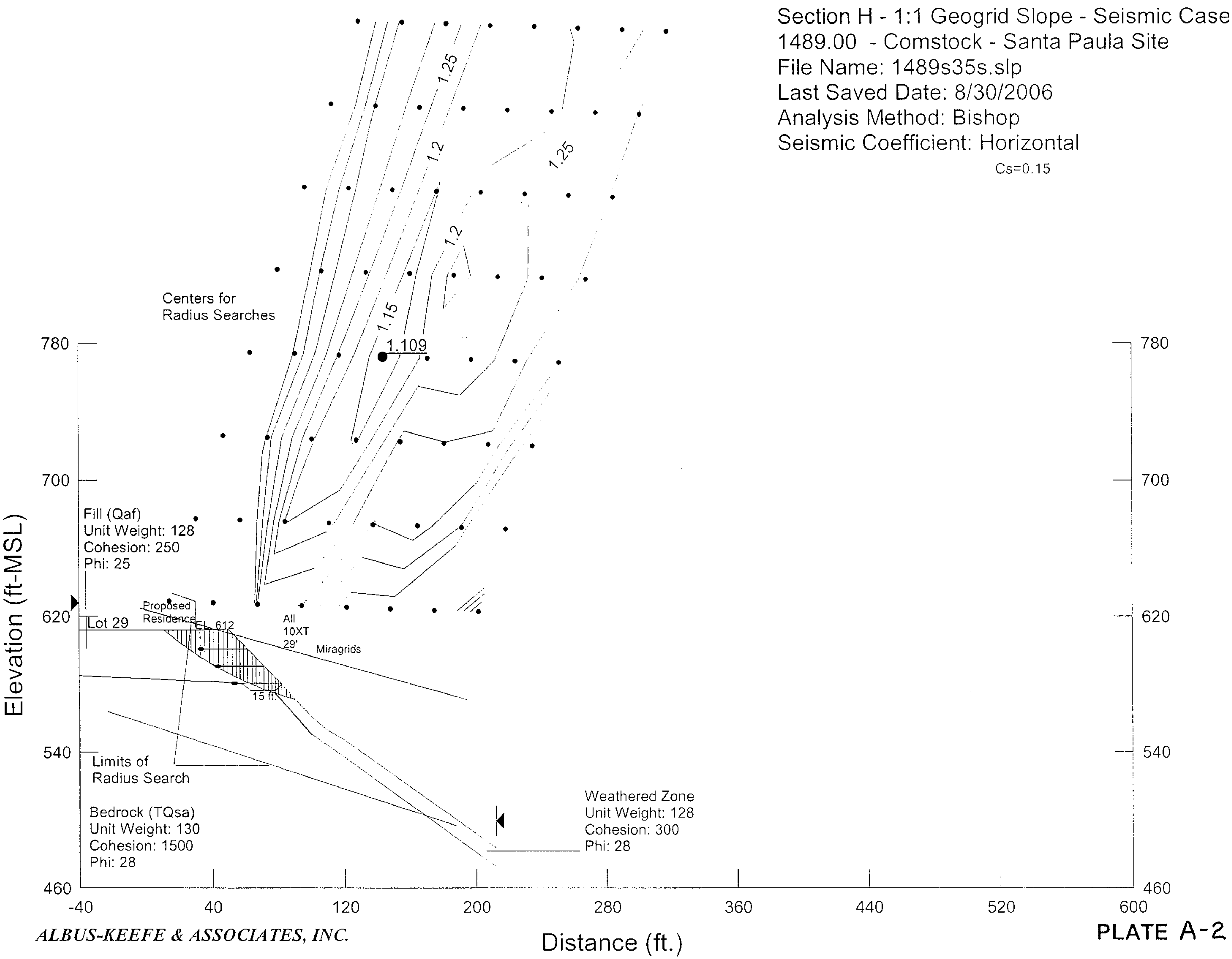
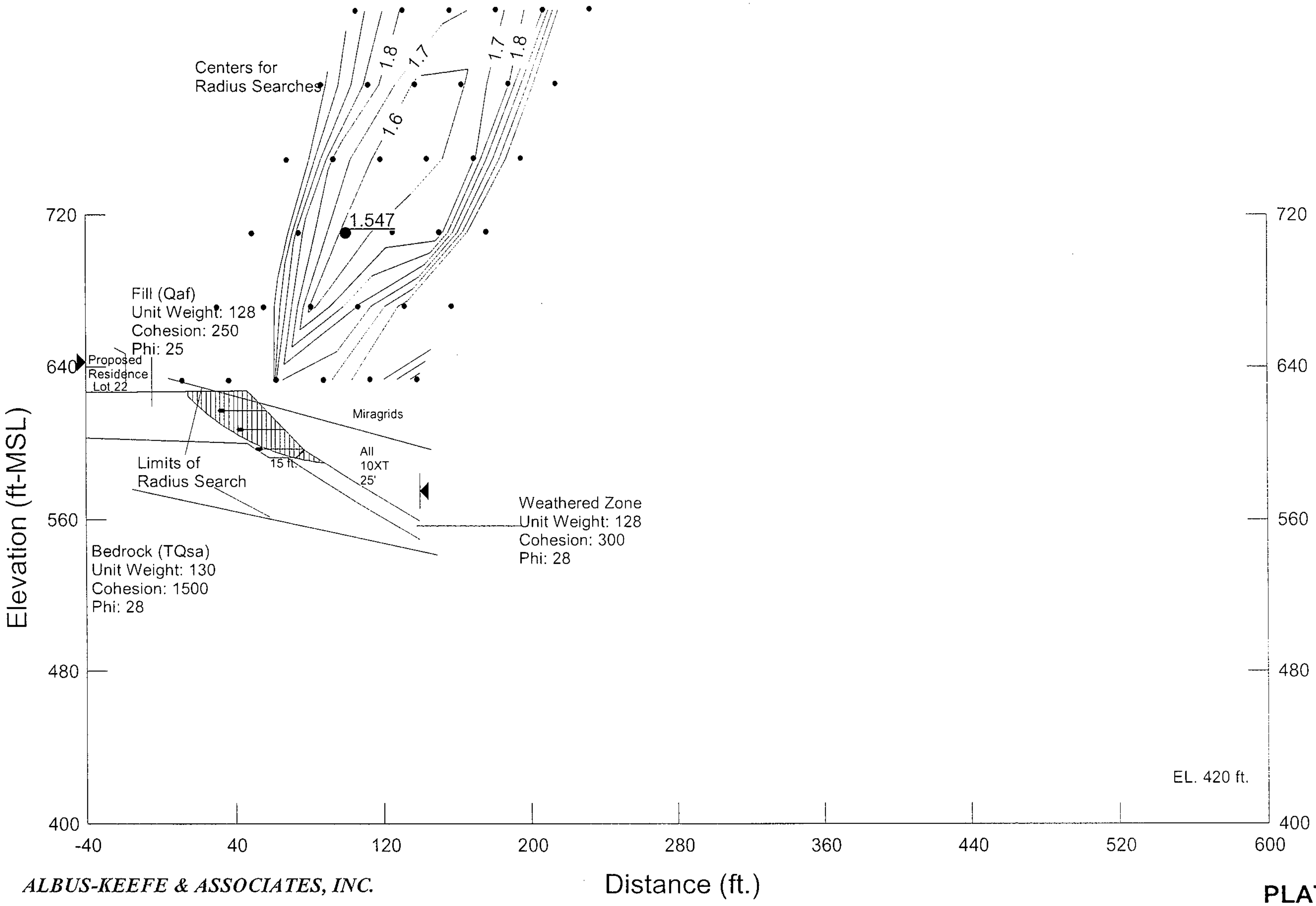
PLATE A-1



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE A-2

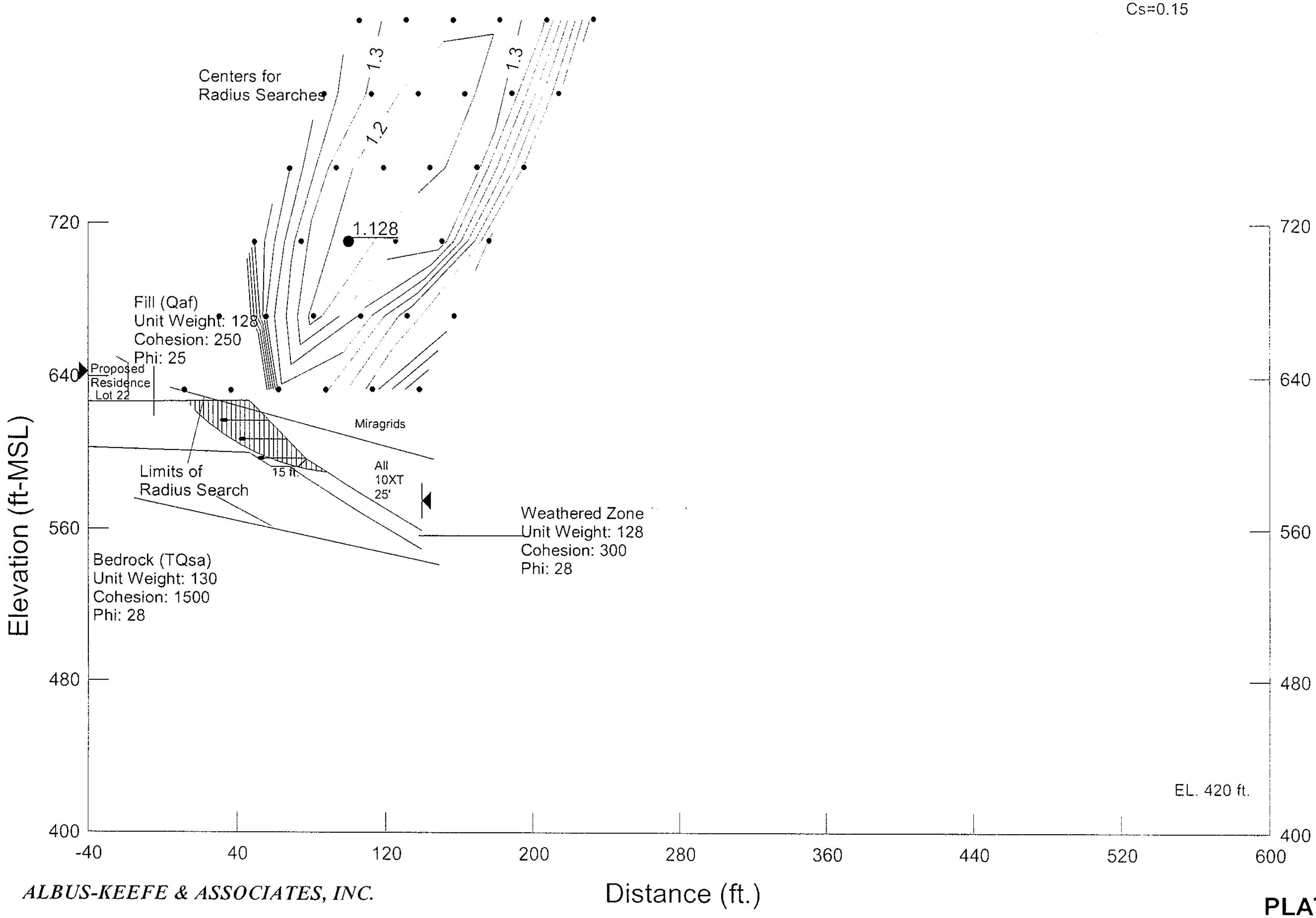
Section I - 1:1 Geogrid Slope - Static Case
1489.00 - Comstock - Santa Paula Site
File Name: 1489s36.slp
Last Saved Date: 8/30/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE A-3

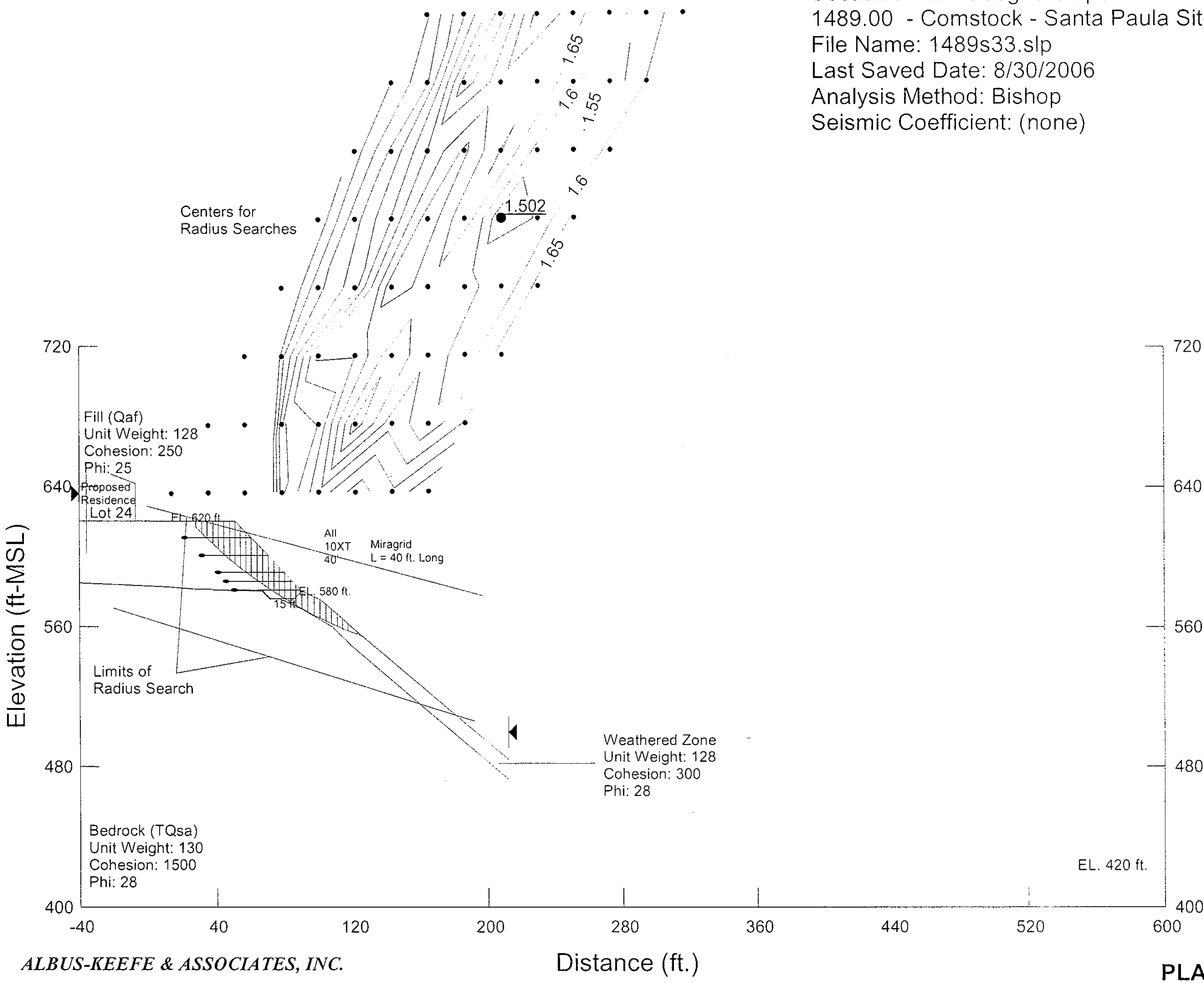
Section I - 1:1 Geogrid Slope - Seismic Cas
1489.00 - Comstock - Santa Paula Site
File Name: 1489s36s.slp
Last Saved Date: 8/30/2006
Analysis Method: Bishop
Seismic Coefficient: Horizontal
Cs=0.15



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE A-4

Section J - 1:1 Geogrid Slope - Static Cas
1489.00 - Comstock - Santa Paula Site
File Name: 1489s33.slp
Last Saved Date: 8/30/2006
Analysis Method: Bishop
Seismic Coefficient: (none)



ALBUS-KEEFE & ASSOCIATES, INC.

PLATE A-5

Section J - 1:1 Geogrid Slope - Seismic Cas
1489.00 - Comstock - Santa Paula Site
File Name: 1489s33s.slp
Last Saved Date: 8/30/2006
Analysis Method: Bishop
Seismic Coefficient: Horizontal
Cs=0.15

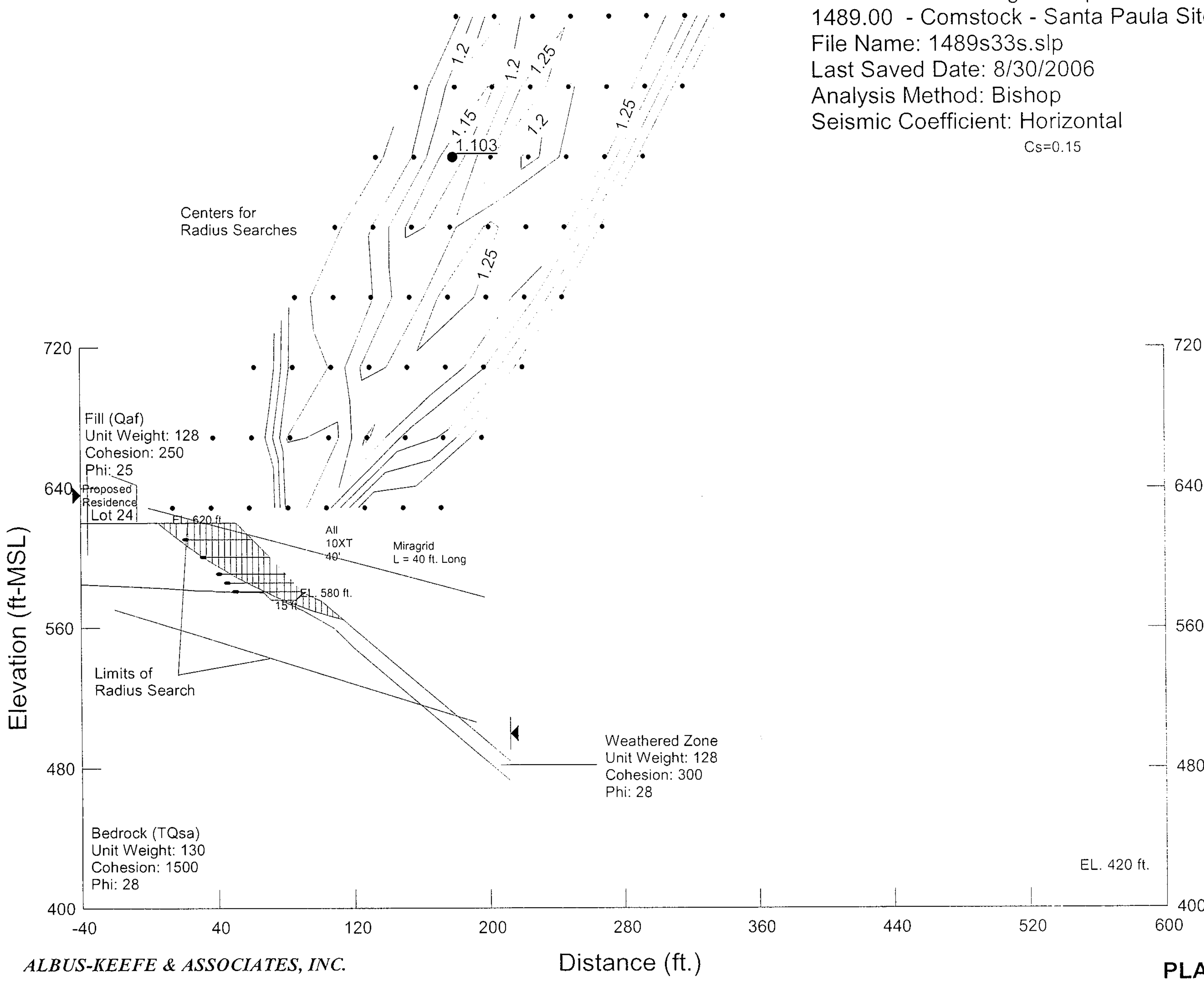


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PLATE A-6

APPENDIX D

KANE GEOTECH INC.

Santa Paula Debris Flow Investigation
Ventura County, California
July 3, 2007

7290.200.101
March 10, 2008
Revised May 23, 2008

Comstock Homes

Santa Paula Debris Flow Investigation

Ventura County, California

July 3, 2007

Job No. GT06-24

Prepared for:
Comstock Homes
321 12th Street
Manhattan Beach, CA 90266

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Comstock Homes

Santa Paula Debris Flow Investigation

Ventura County, California

TABLE OF CONTENTS

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Tel: 209-472-1822 Fax: 209-472-0802
www.kanegeotech.com

Comstock Homes

Santa Paula Debris Flow Investigation

Ventura County, California

Job No. GT06-24

1. INTRODUCTION

The project consists in determining the design input required for debris flow barriers and slope stabilization below a proposed development of single/ two-story family homes. The project site is located in Santa Paula, Ventura County, California, Figure 1. The family homes will be constructed near the ridge of a slope, above and to the west of the existing family homes. A geotechnical investigation by Albus-Keefe & Associates, (Albus, 2006) indicated that the potential for shallow debris flows exists on the slope. The geotechnical consultant has recommended the construction of a debris flow mitigation system at the base of the slope.

1.1 Purpose

The purpose of this report is to present the analysis of the site and design requirements for barriers utilizing Geobrugg UX/VX Debris Flow Protection System technology. As part of the construction process a debris flow barrier will be required.

1.2 Scope of Work

The scope of services provided by KANE GeoTech, Inc. were the following:

1. *Site visit and data collection.* KANE GeoTech, Inc. personnel made site visit(s) to the subject property and collected the data necessary to design a debris flow barrier for the site.

2. *Analysis and Design.* We reviewed existing reports and our own field data to determine the energies and barrier heights necessary to produce a debris flow design that will mitigate the debris flow hazard at the site.

Figure 1. Project Site in Santa Paula, Ventura County, California.

3. *Report and Engineering Drawings*. Upon receiving approval for the recommended systems, KANE GeoTech, Inc. will provide design drawings stamped by a licensed California civil engineer experienced in soil and rock slope stability and debris flow mitigation. These plans will be suitable for obtaining bids for construction of the barrier.

2. SITE DESCRIPTION

2.1 General

The project site is in Santa Paula, Ventura, California, latitude 34°22'05.96"N, longitude W119°03'54.90". The elevation is about 570-ft MSL at top of slope. The site is located in the City of Santa Paula, Ventura County, California, and is in the Transverse Ranges geomorphic province. The ground surface slopes downward from the proposed development and contains three relatively steep east-west trending swales. The slope below the proposed development decreases from about 0.5H:1.0V near the crest to about 2.0 H:1V at the base. The slope is covered with vegetation including grasses, shrubs, and trees, Figure 2. The top of the ridge is the area of proposed developed with homes lining the ridge. Total relief averages approximately 125-ft from the top of the ridge to the bottom of the slope, which is developed with single family homes. The project site is down-slope from the proposed development.

2.2 General Geology

The project site is located in the southern Transverse Ranges geologic province. Geologic materials consist of sandstone, siltstone, and claystone of the Saugus Formation capped by terrace deposits. The terrace deposits contain numerous rounded sandstone cobbles up to 1.5-ft in the largest dimension. These cobbles are readily visible in the exposed slopes, Figure 3. Surficial soil is silty sand and soft. The soil is composed of silt, sandy silt, fine-grained

Figure 2. Oblique view of slopes at the site.

sand, and fine gravel. There are no boulder-size materials on the site.

The site is not in an Alquist-Priolo Zone, although it is within 30-km of six faults zones, Table 1. These faults are capable of producing strong ground motions that could trigger a debris flow under certain conditions.

Figure 3. Terrace materials at project site.

3. SITE EVALUATION

KANE GeoTech, Inc. personnel visited the site on July 6, 2006 and June 4, 2007. The site was evaluated by physically traversing the property. Photographs were taken of the slope and slope features. Details and data were recorded on field map and field book.

TABLE 1 Faults That Could Trigger Debris Flows

Fault Name	Distance From Fault (km)
Oakridge Fault	2.5
Ventura-Pitas Point Fault	5
Simi-Santa Rosa Fault	5
San Cayetano Fault	12.5
Red Mountain Fault	27
Mission Ridge-Arroyo Parida-Santa Ana Fault	30
Santa Ynez Fault (east segment)	30

After evaluating the site it was determined that three locations at the site have the potential to produce debris flow hazard. Chutes were labeled A, B, and C. The chute locations can be found in Appendix A.

All chutes terminated close to the homes located at the base of the terrace. Figures 4 and 5 show the chutes as viewed from below.

4. DEBRIS FLOW BARRIER DESIGN

4.1 Background

A Geobrugg Debris Flow Protection System (Roth, 2003) was chosen for the project site. Geobrugg is one of the few manufacturers of debris flow protection systems and has been involved in substantial research regarding debris flow mitigation (Duffy and Peilia, 1999). The basic debris flow protection system consists of a rockfall protection system modified to resist the velocities and energies of aerial loads unique to debris flows.

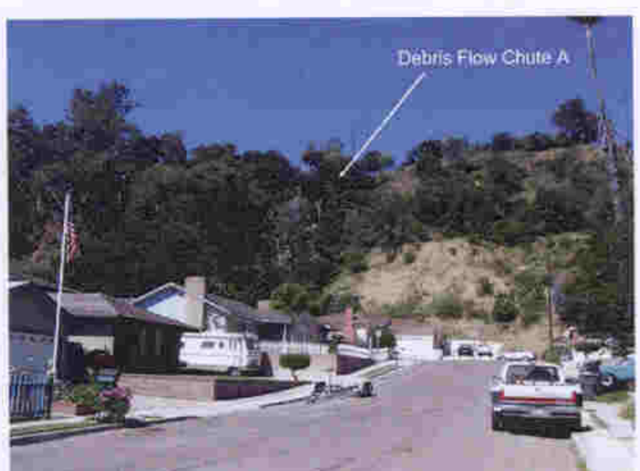

Figure 4. Debris flow Chute A.

Support ropes transfer impact loads on the ring nets to the ground. As in a conventional rockfall protection system, impact energy is absorbed by the net braking elements in the support ropes. In addition, the ring net barrier in the system allows the passage of water and fine particles, eliminating the need to consider any bulking factor when determining barrier height.

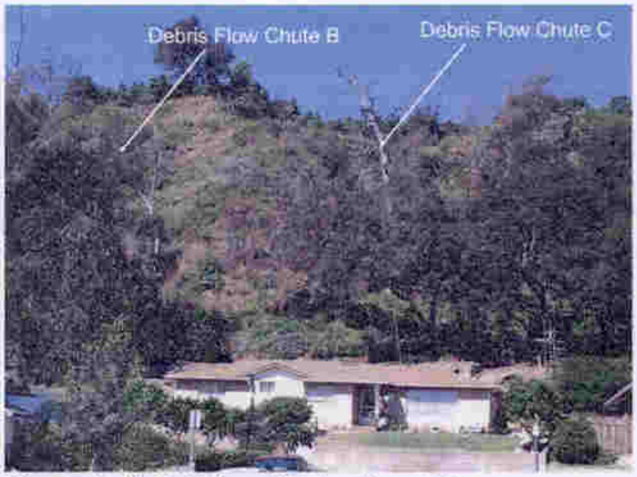

Figure 5. Debris flow Chutes B and C.

4.2 Potential Debris Flow Volumes

Existing methods for determining debris flow volumes are meant for large watersheds and large-scale structures such as basins and bridges impacted by timber impacts (Gatwood, et al., 2000; Bradley, et. al., 2005). They are unsuitable for small debris flow basins and chutes.

Conventional debris flow barrier design is based on field observations (Duffy and Peilia, 1999) and full-scale testing in controlled situations (De Natale, et al., 1996; Muraishi and Sano, 1997). Other work related to the design of debris flow protection systems includes Mitzuyama, et al. (1992) and Rickenmann (1999, 2001), and PWRI (1988).

Geobrugg (2003) has developed a design methodology for chutes suitable for its debris flow barrier systems. The first step in design is to determine the total debris flow volume. Then the resulting peak discharge is calculated. From the peak discharge, the flow velocity can be estimated. Once the mass and velocity are known, the design energy can be determined. Finally, the design height is calculated. It should be noted that debris flows tend to be linear features so that after an initial dynamic impact, additional surges add only a quasi-static load to the net, instead of a dynamic load. In addition, the debris material already impacted and dewatered on the net serves to absorb some of the energy of subsequent surges. The result is that much of the debris flow material is not against the net, resulting in decreased energy absorption and height requirements. An example calculation and the actual debris flow protection system values for the project are given in Appendix B.

Limits of potential debris flow areas were estimated by Albus-Keefe & Associates, Inc. (Keefe, 2007) based on field observations and topographic expression from a LIDAR topographic map provided by FUGRO West, Inc. According to Albus-Keefe, isolated surficial failures within the defined debris flow chutes vary in thickness from a few inches to several feet. For barrier fence design purposes, Albus-Keefe has suggested that a 3-ft thick column of soil could fail within the entire area of Chutes A, B, and C. Since it is unlikely that the entire area of any one chute would fail during a single event, the volumes of soil determined for barrier fence design should be a conservative estimate that takes into account multiple events with varying thicknesses (Albus-Keefe, 2007). The total cumulative potential volumes of the debris flows used in the analyses are given in Table 2.

TABLE 2 Debris Flow Chutes and Estimated Volumes

Chute	Soil	Area (ft^2)	Total Volume (ft^3)
A	Silt with 1'x1'x0.5' Cobbles	5,032	15,096
B	Silt with 1'x1'x0.5' Cobbles	10,793	32,379
C	Silt With 1'x1'x0.5' Cobbles	5,705	17,115

5. RESULTS

Debris flow analyses, Appendix B, indicated that each of the three debris flow chutes can be mitigated with the installation of a single barrier for a total of three barriers. The analyses indicated that barriers should be placed near the mouth of the chutes just inside the property line, Figure 6. These barriers can be either Geobrugg UX (Figure 7) or VX (Figure 8) barrier depending the shape of the chutes at the finalized barrier locations.

6. CONCLUSIONS AND RECOMMENDATIONS

The installation of Geobrugg Debris Flow Protection Systems as described in this report will meet the debris flow protection requirements of the proposed project.

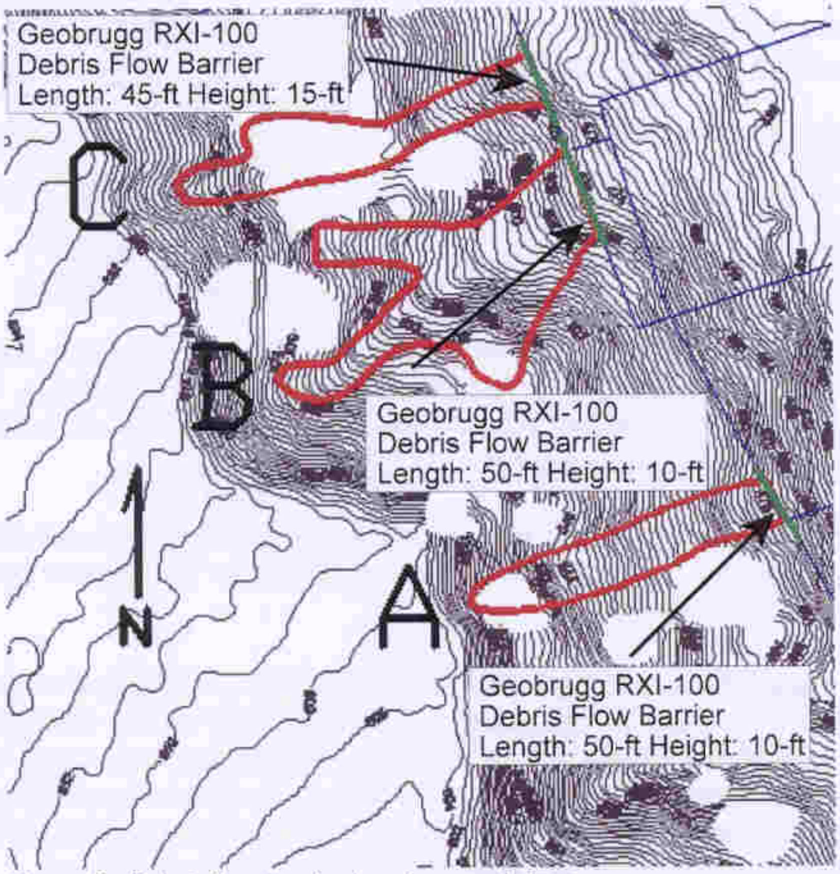

Figure 6. Debris flow barrier locations and sizes.

Figure 7. Geobrugg UX debris flow barrier (Source: Geobrugg Fatzer, AG).

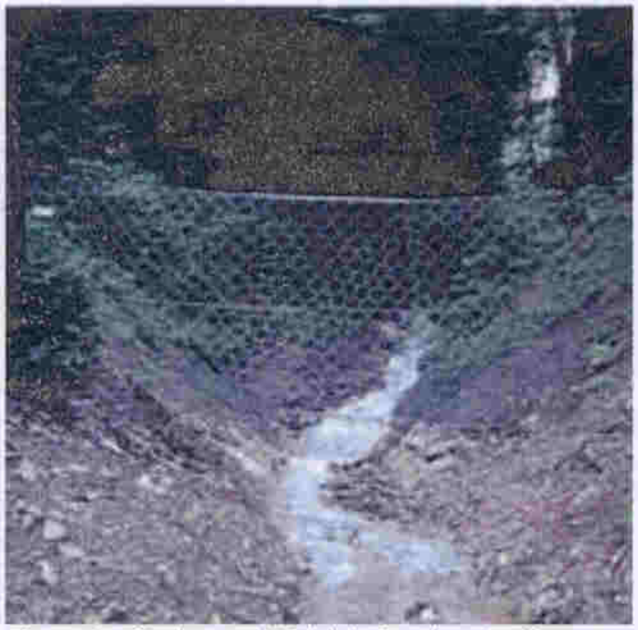

Figure 8. Geobrugg VX debris flow barrier (Source: Geobrugg Fatzer, AG).

1. *Construction recommendations.* All work should be constructed by competent construction personnel. Qualified installers should be contacted to obtain more accurate pricing. For this particular project we recommend the following installers both of whom has significant experience in installing Geobrugg Debris Flow Protection Systems We recommend the following contractors:

   AIS Construction Company
   P.O. Box 239
   Carpenteria, CA 93014
   Tel 805-684-4344
   Fax 85-566-0109
   www.aisconstruction.com

   Hi-Tech Rockfall Construction, Inc.
   P.O. Box 674
   Forest Grove, OR 97116
   Tel 503-357-6508
   Fax 503-357-7323
   www.hitechrockfall.com

   On approval by City of Santa Paula officials, KANE GeoTech, Inc. will supply shop drawings, stamped by a Civil Engineer registered in California, for the purpose of obtaining bids for the construction. These drawings will incorporate the results of the analyses described in this report.

2. *Cost estimate.* We estimate the installed cost of the recommended system to be as shown in Table 3.The total cost of the project in its current configuration will be approximately $158,500.

3. *Maintenance.* As a minimum, the fence and the area behind it should be inspected annually and repaired as necessary. Clean out may not be necessary as the barriers are capable of storing the entire predicted debris flow volumes. In any event, a full barrier will mitigate any additional movement of upper slope material by significantly changing the gradient and removing the energy from any subsequent flows. Follow the manufacturer's maintenance recommendations and procedures.

TABLE 3 Calculated Debris Flow Design Parameters and Estimated Installed Costs

Chute	Length ft	Height ft (m)	Energy ft-tons	Geobrugg Debris Flow Protection System	Unit Cost (Ft)	Estimated Cost
A	50	10	192	RXI-100	$1,000	$50,000
B	50	10	369	RXI-100	$1,000	$50,000
C	45	15	236	RXI-100	$1,300	$58,500
						$158,500

NOTE: Design energies and barrier heights are used, along with the location of the debris flow mitigation structures, to determine type of mitigation used. Reported design energies and bounce heights alone do not solely determine the mitigation strategy or debris flow protection system type.

7. REFERENCES

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Duffy, J. D., and Peilia, A. (1999). "Case Studies on Debris and Mudslide Barrier Systems in California."

De Natale, J. S. et al. (1996). "Response of the Geobrugg Cable Net System to Debris Flow Loading." Report, California Polytechnic State University, San Luis Obispo, California.

Gatwood, E., Pedersen, J., and Casey, K. (2000) "Los Angeles District Method for Prediction of Debris Yield" U.S. Army Corps of Engineers, Los Angeles District, February 1992, Updated February 2000.

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Keefe, P. (2007). Personal Communication. Via email, June 12, 2007.

Muraishi, H. and Sano, S. (1997). "Full Scale Rockfall Test of Ring Barrier and Components."

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Rickenmann, D. (1999). "Empirical Relationships for Debris Flows." *Natural Hazards*, 19(1), 47-77.

Rickenmann, D. (2001). "Estimation of Debris Flow Impact on Flexible Wire Rope Barriers." Birmensdorf, interner Bericht, unver ffentlicht.

PWRI (1988). "Technical Standard for Measures Against Debris Flows (Draft)." Ministry of Construction, Japan.

Roth, A. (2003). "VX/UX Protection System Against Debris Flow." Design Concept, Geobrugg Protection Systems, Romanshorn, Switzerland.

8. LIMITATIONS

Debris flow and rockfall are sporadic and unpredictable. Causes range from human construction to environmental (weather, earthquakes) effects. Because of the multiplicity of factors affecting rockfall dynamics, debris flow and rockfall are not, and cannot be, exact sciences that guarantee the safety of individuals and property. However, by the application of sound engineering principles to a predictable range of geodynamics, the risk of injury and property loss can be substantially reduced by the use of properly designed barriers in identified risk areas. Inspection and maintenance of barriers is necessary to insure that the desired protection level is not degraded by impact damage exceeding the design limits of a particular system or by corrosion from pollution or other man-made factors.

The analyses, conclusions and recommendations contained in this report are based on the site conditions observed by KANE GeoTech, Inc. personnel and derived from the information provided to KANE GeoTech, Inc. by others. If there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we urge that our report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only for the project and site studied. This report should not be used after three years.

Our professional services were performed, our findings obtained, and our recommendations proposed in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. Findings and statements of professional opinion do not constitute a guarantee or warranty, expressed or implied.

REGISTERED PROFESSIONAL ENGINEER
WILLIAM F. KANE
C 055714
Exp. 12-31-08
CIVIL
STATE OF CALIFORNIA

***KANE* GeoTech, Inc.**

William F. Kane, PhD, PG, PE
California Licensed Civil Engineer No. 55714

APPENDIX A

DEBRIS FLOW CHUTE AND BARRIER LOCATION MAP

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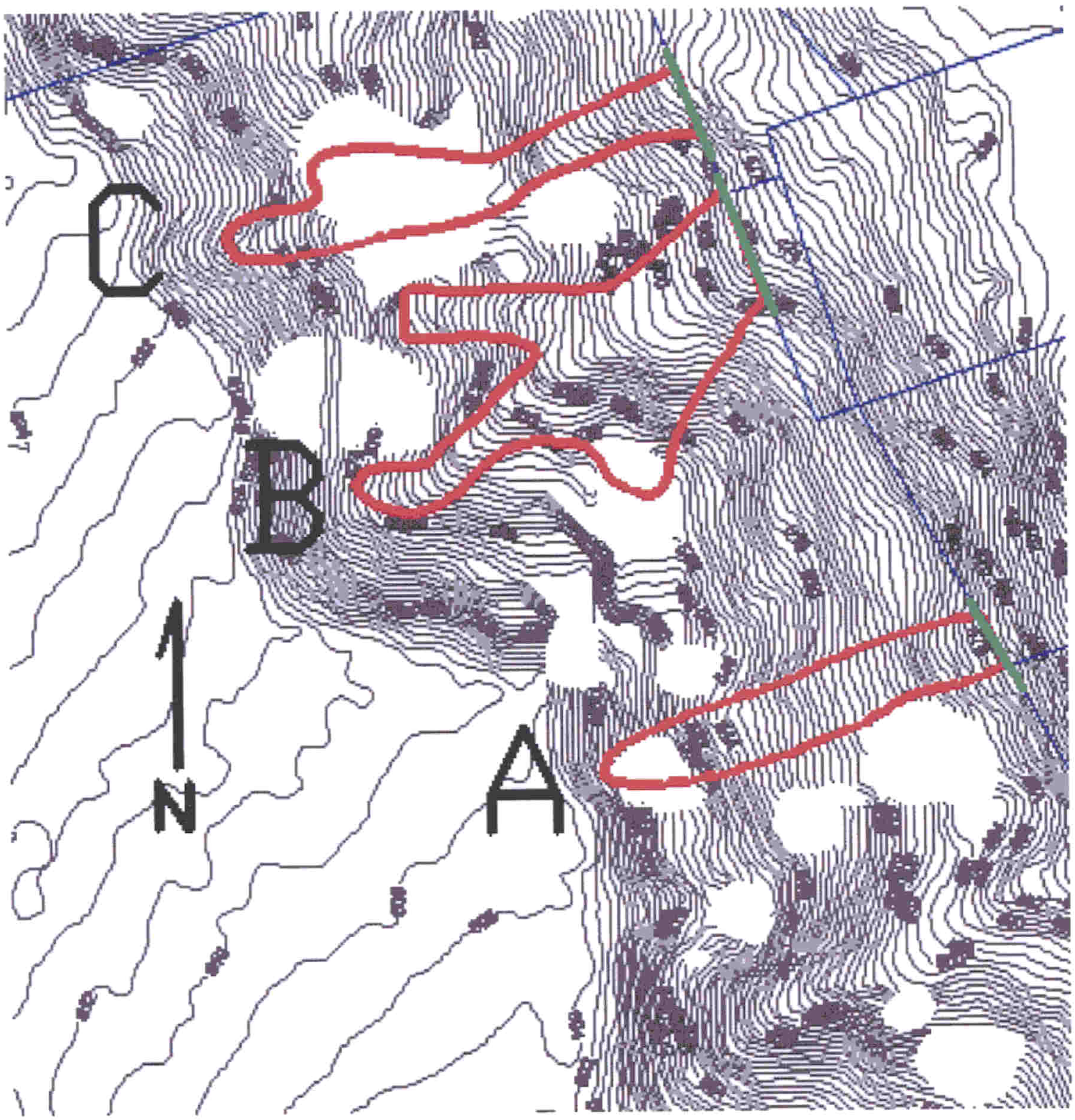
PLATE A-1 Debris Flow Chute and Barrier Locations

KANE GeoTech, Inc.
Geoengineering Consultants

Comstock Homes
Debris Flow Barrier
Ventura County, California



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APPENDIX B

DEBRIS FLOW PROTECTION SYSTEM CALCULATIONS

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GT06-24 ~ SANTA PAULA DEBRIS FLOW 7-28-06 /WFK 1/2

SAMPLE CALCULATION - DEBRIS FLOW CHUTE A

- ROTH, A. (2003) "VX/UX PROTECTION SYSTEMS AGAINST DEBRIS FLOW." FATZER AG, GEOBRUGG PROTECTION SYSTEMS, ROMANSHORN, SWITZERLAND

- ALBUS-KEEFE, INC (2006) PERSONAL COMMUNICATIONS

SLOPE < = 21° $\gamma = 120^{pcf}$ $T_{impact} = 2.0$ (GRANULAR) (FAST FLOWING)

THICKNESS = $4^{ft.}$ = 1.22^{m} FLOW WIDTH = 40^{ft} = 12.16^{m}

BULKING FACTOR = 0 (DEWATERED BY BARRIER) NET STORAGE = 30%

- ESTIMATED VOLUME 970^{m^3} (KGT CALCULATIONS)

- PEAK DISCHARGE - GRANULAR MATERIAL - COBBLES, ETC

$$Q_p = 0.135 V_{DF}^{0.78} \quad \text{(EQN 1)}$$
$$= (0.135)(970)^{0.78}$$
$$\underline{\underline{Q_p = 28.84 \, m^3/s}} \quad \text{RANGE } 5^{m^3/s} - 30^{m^3/s} \text{ OK} \checkmark$$

- VELOCITY

$$v = 2.1 \, Q_p^{0.33} S^{0.33} \quad \text{(EQN 3)}$$
$$v = (2.1)(28.84)^{0.33} [\tan(21°)]^{0.33}$$
$$\underline{\underline{v = 4.64 \, m/s}} \quad \text{RANGE } 2^{m/s} - 6^{m/s} \text{ OK} \checkmark$$

- MASS

$$m_d = \rho_d Q_p T_{imp} \quad \text{(EQN 6)}$$
$$= [(120)(16.02)](28.84)(2)$$
$$\underline{\underline{m_d = 110,884^{kg}}}$$

- KINETIC ENERGY

$$KE = \tfrac{1}{2} m_d v^2 \quad \text{(EQN 7)}$$

GT06-24 ~ SANTA PAULA DEBRIS FLOW 7-28-06 / WFK 2/2

$$KE = \frac{(0.5)(110,884)(4.64)^2}{1000}$$

$$\boxed{KE = 1,194^{kJ} = 442^{ft\text{-}tons}}$$

- BARRIER HEIGHT (NET STORAGE = 30%)

$$H_o = \left\{\left(\frac{32}{9}\right)\left(\frac{V_{DF}}{b}\right)\tan\left[\arctan S_o - \arctan\left(\frac{2}{3}S_o\right)\right]\right\}^{1/2} \quad \text{(EQN 22)}$$

$$H_o = \left\{\left(\frac{32}{9}\right)\left[\frac{(1-0.3)(970)}{12.16}\right]\tan\left[\arctan(\tan 21^\circ) - \arctan\left(\frac{2}{3}\tan 21^\circ\right)\right]\right\}^{1/2}$$

$$\boxed{H_o = 4.81^{m} = 15.7^{ft}}$$

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Debris Flow Calculations

Reference: Roth, A. (2003). "VX/UX Protection Systems Against Debris Flow." Fatzer, A.G., Geobrugg Protection Systems, Romanshorn, Switzerland

PROJECT NO. GT06-24 **PROJECT NAME:** Santa Paula Debris Flow Investigation

DESCRIPTION: Debris Flow Chute A

Input Parameters		Calculated Values	
Debris Flow Area	5,032 ft^2	Debris Flow Area	467 m^2
Thickness of Mobilized Material	3.0 ft	Thickness of Mobilized Material	0.91 m
Unit Weight (γ)	120 pcf	Unit Weight	1,922 kg/m^3
Slope Angle	21 °	Slope Gradient (S)	0.38
Flow Width (b)	42 ft	Flow Width (b)	12.8 m
Bulking Factor	0%		
Deformed Net Capacity	30%		
Time of Impact (T_imp)	2.5 s		

Debris Flow Analysis

Debris Flow Volume		
Volume	427 m^3	Range: 100-m^3 to 1,000-m^3
Peak Discharge		
Granular Material	15.2 m^3/s	Range: 5 m^3/s to 30 m^3/s
Fine-Grained Material	22.5 m^3/s	Range: 5 m^3/s to 30 m^3/s
Velocity		
Velocity	3.76 m/s	Range: 2 m/s to 6 m/s
Kinetic Energy		
Mass	73,152 kg	
Design Energy	**517 KJ**	**192 ft-tons**
Design Height		
Design Height	**3.1 m**	**10.2 ft**

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Debris Flow Calculations

Reference: Roth, A. (2003). "VX/UX Protection Systems Against Debris Flow." Fatzer, A.G., Geobrugg Protection Systems, Romanshorn, Switzerland

PROJECT NO. GT06-24 **PROJECT NAME:** Santa Paula Debris Flow Investigation

DESCRIPTION: Debris Flow Chute B

Input Parameters		**Calculated Values**	
Debris Flow Area	10,793 ft^2	Debris Flow Area	1003 m^2
Thickness of Mobilized Material	3.0 ft	Thickness of Mobilized Material	0.91 m
Unit Weight (γ)	120 pcf	Unit Weight	1,922 kg/m^3
Slope Angle	18 °	Slope Gradient (S)	0.32
Flow Width (b)	82 ft	Flow Width (b)	25.0 m
Bulking Factor	0%		
Deformed Net Capacity	30%		
Time of Impact (T_imp)	2 s		

Debris Flow Analysis

Debris Flow Volume		
Volume	917 m^3	Range: 100-m^3 to 1,000-m^3
Peak Discharge		
Granular Material	27.6 m^3/s	Range: 5 m^3/s to 30 m^3/s
Fine-Grained Material	41.2 m^3/s	Range: 5 m^3/s to 30 m^3/s
Velocity		
Velocity	4.33 m/s	Range: 2 m/s to 6 m/s
Kinetic Energy		
Mass	106,123 kg	
Design Energy	**995 KJ**	**369 ft-tons**
Design Height		
Design Height	**3.0 m**	**10.0 ft**

KANE GeoTech, Inc.
P.O. Box 7526
Stockton, CA 95267-0526
Tel: 209-472-1822 Fax: 209-472-0802 Email: william.kane@kanegeotech.com

Debris Flow Calculations

Reference: Roth, A. (2003). "VX/UX Protection Systems Against Debris Flow." Fatzer, A.G., Geobrugg Protection Systems, Romanshorn, Switzerland

PROJECT NO. GT06-24 **PROJECT NAME:** Santa Paula Debris Flow Investigation

DESCRIPTION: Debris Flow Chute C

Input Parameters

Debris Flow Area	5,705 ft^2
Thickness of Mobilized Material	3.0 ft
Unit Weight (γ)	120 pcf
Slope Angle	30 °
Flow Width (b)	35 ft
Bulking Factor	0%
Deformed Net Capacity	30%
Time of Impact (T_imp)	2 s

Calculated Values

Debris Flow Area	530 m^2
Thickness of Mobilized Material	0.91 m
Unit Weight	1,922 kg/m^3
Slope Gradient (S)	0.58
Flow Width (b)	10.7 m

Debris Flow Analysis

Debris Flow Volume		
Volume	485 m^3	Range: 100-m^3 to 1,000-m^3
Peak Discharge		
Granular Material	16.8 m^3/s	Range: 5 m^3/s to 30 m^3/s
Fine-Grained Material	24.9 m^3/s	Range: 5 m^3/s to 30 m^3/s
Velocity		
Velocity	4.44 m/s	Range: 2 m/s to 6 m/s
Kinetic Energy		
Mass	64,541 kg	
Design Energy	**637 KJ**	**236 ft-tons**
Design Height		
Design Height	**4.2 m**	**13.8 ft**

APPENDIX E

ALBUS-KEEFE AND ASSOCIATES

Rough Grading Report for Phase 1 Construction Area
Lots 26 through 35 and 44 through 50, Associated Streets and Slopes
Tract 5606, Santa Paula, California
October 23, 2007

7290.200.101
March 10, 2008
Revised May 23, 2008

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

October 23, 2007
J.N.: 1489.00

Ms. Tiffany Sukay
Comstock Homes
321 12th Street, Suite 200
Manhattan Beach, California 90266

Subject: **Rough Grading Report for Phase 1 Construction Area, Lots 26 through 35 and 44 through 50, Associated Streets and Slopes, Tract 5606, City of Santa Paula, California.**

Dear Ms. Sukay:

We are pleased to present to you our report of rough grading services for the subject site. This report presents a summary of our geotechnical observation and testing services we provided during site rough grading operations for the subject lots as well as our conclusions and recommendations pertaining to future site development based on the as-graded site conditions.

We appreciate this opportunity to be of service to you. If you have any questions regarding the contents of this report, please do not hesitate to call.

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

Patrick M. Keefe

Patrick M. Keefe
Principal Engineering Geologist

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REPORT

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MAP

1.0 INTRODUCTION

1.1 PURPOSE

This report presents a summary of geotechnical consulting services provided by ***Albus-Keefe & Associates, Inc.*** during rough grading of Lots 26 through 35 and 44 through 50, associated interior streets and slopes within Tract 5606. Conclusions and recommendations relative to future site development are also discussed herein. Rough grading under the purview of this report was accomplished from August 30, 2006, through September 28, 2007. The purpose of the rough grading was to create building pads for single-family residential development. The limits of rough grading performed under the purview of this report are shown on the enclosed Plot Plan, Plate 1.

Geotechnical consulting services provided by this firm during the completion of rough grading operations for Lots 37-40 (model lots) has been previously reported in the referenced report dated February 14, 2007. At the time of this report, continued rough grading operations are being performed in other areas of Tract 5606. As such, a summary of geotechnical consulting services for the balance of the site will be reported at a later date.

1.2 PROJECT PLAN AND JURISDICTION

The layout of Tract 5606 is shown on the plans entitled "Rough Grading Plan, Tract No. 5606, City of Santa Paula", prepared by Development Resource Consultants, Inc. Sheet 9 of these plans is used as the base map and enclosed herein as Plate 1(Plot Plan).

Rough grading operations summarized herein were performed under the jurisdiction of the City of Santa Paula, California.

1.3 SCOPE OF SERVICES

Albus-Keefe & Associates, Inc. has provided geotechnical consulting services as described below:

- Reviewed referenced geotechnical reports by this firm and others for the subject site and adjacent sites.
- Provided observation during clearing and grubbing operations.
- Provided observation during removal of unsuitable earth materials.
- Provided observation of fill keys and overexcavation bottoms.
- Provided observation during placement of subdrains and backdrains.

- Provided observation and field testing during scarification, moisture conditioning and recompaction of exposed earth materials within removal and overexcavation bottoms, and during fill placement within the site.

- Provided observation and field testing during fill slope construction and placement of geogrid.

- Provided observation and field testing during construction and backfill of segmental retaining walls.

- Provided laboratory testing of earth materials encountered during rough grading operations.

- Preparation of this report summarizing our observations, results of field and laboratory testing, and opinions and recommendations relative to future development of the site.

2.0 SUMMARY OF ROUGH GRADING OPERATIONS

Albus-Keefe & Associates, Inc. performed field observation and testing services during rough grading operations for the site. J&S Excavating, Inc., performed the rough grading summarized in this report except for soils and materials related to the segmental retaining walls and geogrid-reinforced slope. Construction of the segmental retaining walls and construction of the geogrid-reinforced 1:1 slope was performed by Soil Retention. Observations by our staff and discussions with the contractors indicate that the work was performed as discussed in the following sections.

2.1 SITE PREPARATION

The subject site was cleared of deleterious debris, trees, and other vegetation prior to rough grading operations. The deleterious materials were generally disposed of offsite.

Within the limits of rough grading, all existing non-engineered fill materials, topsoil, and the upper 5 to 7 feet of the older colluvial materials were removed to expose competent older colluvium and terrace deposits considered suitable for support of the proposed engineered fill, structures or related improvements. The approximate elevations of the removal bottoms are shown on the Plot Plan, Plate 1.

2.2 FILL PLACEMENT

Prior to placement of engineered fill materials, the exposed ground was scarified to a depth of approximately 6 inches, moisture conditioned to a relatively uniform moisture content near or slightly above optimum, and then compacted.

Following preparation of the exposed ground surface, fill was placed in lifts up to approximately 6 inches in thickness; moisture conditioned to a relatively uniform moisture content near or slightly above optimum, and then mechanically compacted. Mechanical compaction was achieved by using a rubber-tired dozer (Caterpillar 824/834) as well as by wheel rolling with loaded scrapers. Each lift was placed in a similar manner. Fill materials were derived from removal and cut areas within the development and imported from an off-site source. Prior to placement of fill on surfaces inclined steeper than approximately 5:1 (h:v), near-vertical benches were cut into competent earth materials within the adjacent ascending terrain.

Rocks greater than 12 inches in maximum dimension were generated during the rough grading operations. The majority of oversize rocks were disposed of off site. However, a relatively minor amount of oversized rocks, less than 3 feet in maximum dimension, were scattered within the engineered fill materials at depths greater than 10 feet below finish grade and 10 feet horizontally from finished slope faces. The oversized rocks were mixed with granular materials and spread throughout the fill to eliminate nesting.

The approximate limits of compacted fill placed under the purview of this report are shown on the Plot Plan, Plate 1. The maximum depths of compacted fill placed within the level portions of the building pads are listed in Table C-1, Appendix C.

2.3 LOT CAPPING

All cut lots and the cut and shallow fill portions of cut-to-fill transition lots were overexcavated a minimum of 3 feet below the proposed pad grades and replaced with compacted fill. The overexcavation generally extended across the entire lot. Overexcavations were generally graded to slope at approximately 1 percent toward the adjacent street or deeper fill.

2.4 EAST BLUFF STABILIZATION

The reviewer for the city of Santa Paula requested that a complete exposure of the bedrock near the easterly bluff be made and mapped to substantiate our stability analysis in our supplemental geotechnical investigation report. Therefore, a temporary 1:1 backcut was created at the rear of Lots 21-25 and 28-30 to expose bedrock.

Following observations of the geologic conditions by this office and the city reviewer, a keyway was excavated in the bluff. The keyway has a width ranging from 25 to 35 feet and a depth of at least 2 feet into competent bedrock or terrace deposits. A backdrain was provided at the heel of the keyway in accordance with the recommendation of our referenced geotechnical investigation report.

The slope was reconstructed at a ratio of 1:1 with select granular sand fill material and geogrids in conformance with our recommendations previously approved by the city reviewer. Mirifi 10XT Geogrids were installed from the slope face and into the slope a horizontal distance approximately equal to the slope height. These geogrids were placed at pre-determined elevations based on our stability analysis previously reviewed and approved by the city reviewer. Mirifi 2XT geogrids were placed between the Mirifi 10XT geogrids every 2.5 feet vertically. Each geogrid was placed on the fill to provide an embedment of 8 feet into the slope with additional geogrid left at the slope face. Following placement of approximately 2.5 feet of fill, the additional geogrid was wrapped over the outer slope face and embedded into the slope face with a minimum of 3 feet.

In order to mitigate piping of granular soils through the geogrids, cohesive onsite soils were placed on the outer 2 to 4 feet of the slope face. In addition, Verdura wall blocks were utilized every 2.5 feet to aid in constructing the slope at a ratio of 1:1. Compaction on the outer edge was achieved with hand operated compaction equipment. Compaction within the inner select granular sand fill material was achieved with a steel-wheeled vibratory compactor.

2.5 SEGMENTAL RETAINING WALLS

Geogrid-reinforced segmental retaining walls were constructed during site grading along the eastern boundary of Lots 34 and 35 and along the southeastern boundary of Lots 30, 31, and 33 (Wall "B"). This firm provided full-time observation and testing during segmental wall construction. Our services consisted of observation of foundation excavations and backdrain systems for segmental retaining walls, observation of facing block placement, observation of geogrid installation, and observation and compaction testing of select wall backfill.

Drain pipe, consisting of 4-inch-diameter perforated P.V.C. schedule 40 or SDR 35 pipe, was placed within the drain rock of the segmental retaining wall. The drain pipe was placed as low as possible to provide outlet above adjacent grades. Drain pipes were set to drain at a minimum gradient of 1 percent toward outlets. Outlets were constructed of 4-inch-diameter solid P.V.C. pipe and were generally spaced to outlet every 100 feet.

2.6 INCOMPLETE WORK

At the time of preparing this report, a masonry retaining wall proposed at the rear of Lots 32 and 33 was not constructed. The wall will vary from 2 to 6 feet in height. At this location, a temporary slope has been constructed to provide support for the pads.

3.0 AS-GRADED GEOLOGIC CONDITIONS

3.1 GEOLOGIC OBSERVATIONS

Periodic geologic observations were made during rough grading to compare the anticipated and as-graded geologic conditions. The geologic conditions that were mapped are relatively similar to the anticipated conditions and are shown on the enclosed Plot Plan, Plate 1. A detailed description of the geologic units encountered within the limits of this report is discussed in the following section.

3.2 GEOLOGIC UNITS

3.2.1 Non-Engineered Artificial Fill

Non-engineered artificial fill associated with previous agricultural activities were present locally throughout much of the site. These fill material were typically damp to very moist, soft to stiff or medium dense to dense, and consisted of clayey silt, silty clay, silt, silty sand with clay, and sandy silt with clay and locally contained rock fragments. Non-engineered artificial fill was completed removed during site grading.

3.2.2 Older Colluvium (Qcolo)

Older colluvium (previously referred to as the upper terrace deposits in our referenced report dated May 3, 2006) underlies the majority of the site under the purview of this report. The older colluvial materials consist primarily of silt, clayey silt, sandy silt, silty clay, and sandy clay that are various shades of brown. These deposits are typically damp to very moist and stiff to hard. The upper 5 to 7 feet of these materials was generally weathered and was removed during site grading to expose competent older colluvium.

3.2.3 Terrace Deposits (Qt)

Late Pleistocene-age stream terrace deposits (previously referred to by this firm as the lower portion of the terrace deposits) were mapped throughout the bluff top excavation. This unit consists of poorly- to locally-well-stratified gravels, cobbles, and boulders in a matrix of clayey sand, silty sand and sand. These materials were generally observed to be damp to moist and dense to very dense. The base of the terrace deposits is generally slightly inclined to the southeast. Some local scour features were noted at the base of the unit, particularly where the underlying bedrock unit is comprised of granular materials. These materials were exposed in the backcut created during the east bluff stabilization.

3.2.4 Bedrock – Saugus Formation (TQsa)

Plio-Pleistocene-age Saugas Formation underlies the entire project area. The Saugus Formation contains non-marine sediments that consist of massive to thickly-bedded clayey siltstone, sandy siltstone, silty sandstone, sandstone, and conglomerate interbeds, generally 1 foot to 6 feet in thickness and thin clay seams, typically ¼ inch or less.

The bedrock units were observed to be light brown, reddish brown, pale olive-gray to olive-brown in color, soft to moderately hard, damp to moist, slightly to moderately weathered, and locally contain some calcium carbonate mineralization along joints.

Where observed, bedding within the Saugus Formation is massive to thinly bedded, but often indistinct with gradational contacts. Cross bedding and scour features were also observed.

A number of clay seams were noted to be tectonically sheared, polished and locally striated in the down dip direction. We attribute these features to flexural slip during rapid uplift and folding of the bedrock in the region. Bedrock materials were exposed in the backcut created during the east bluff stabilization.

3.3 GEOLOGIC STRUCTURE

3.3.1 Bedding

Bedding plane surfaces within the sandstone units are generally gradational to moderately well developed while bedding plane surfaces within the siltstone units are well developed and distinct where in contact with the sandstone. Bedding structure in the Saugus Formation is relatively uniform. The preponderance of the bedding attitudes observed throughout Tract 5606 strike N63ºE ± 20º and dip 48º ± 11º to the southeast.

3.3.2 Jointing

The jointing observed on site was typically high angled, non-planar, discontinuous, tight, and lined with calcium carbonate and/or iron oxidation staining. Joint attitudes observed within the bedrock along the bluff top generally strike north to south and northwest to southeast with moderate to vertical dip angles, mainly dipping toward the east.

3.4 LANDSLIDES

No landslides were identified within or adjacent to the subject lots during the rough grading operations.

3.5 GROUNDWATER

Groundwater was not encountered within the limits of this report.

4.0 FIELD TESTING

The in-place density of fill materials was determined in accordance with ASTM D1556 (6-inch sand cone), ASTM D2937 (drive-cylinder), and ASTM D2922/D3017 (nuclear gauge). The results of field density tests were compared to the maximum density determined in accordance with ASTM D1557-02 to evaluate relative compaction. Where test results indicated a relative compaction less than 90%, the limit of the area of substandard fill was determined, the fill materials were then moisture conditioned, if needed, and recompacted until subsequent testing resulted in a relative compaction of 90% or greater. Field density tests were taken at a frequency of at least one test per 1000 cubic yards of fill placement or one test for every two vertical feet of fill placement, whichever occurred first. Surface density tests were taken upon achieving rough pad grade. The results of field density tests are presented in Appendix A, on Table A. As previously noted, rough grading operations within Lots 37-40 were performed concurrently with the rough grading operations for the balance of Tract 5606. As such, the test numbers on Table A are not in a sequential order. Only the tests conducted within the limits of time report are included on Table A. The approximate test locations are shown on the enclosed Plot Plan, Plate 1.

5.0 LABORATORY TESTING

Representative samples of the onsite soils were collected and tested in the laboratory during the rough grading operations. Laboratory tests included maximum dry density and optimum moisture content, sand equivalence (SE), direct shear, expansion index, soluble sulfate content, Atterberg Limits, and corrosion series. Descriptions of the laboratory tests are presented below. Pertinent test values are presented within Appendix B.

Maximum dry density and optimum moisture content tests were performed on selected samples in general conformance with ASTM D-1557-02. Pertinent test results are presented within Table B-1.

Sand equivalence tests were performed on selected samples obtained from the select granular soils used as backfill for the geogrid-reinforced slope and the segmental retaining wall. These tests were performed in accordance with California Test Method 217. Pertinent test results are presented within Table B-2.

Direct shear tests were performed on selected samples obtained from the select granular soils used as backfill for the segmental retaining wall. These tests were performed in accordance with ASTM D 3080-80. The samples were remolded to 90 percent of maximum dry density and 2 percentage points over optimum. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plates B-1 through B-3.

Expansion Index tests were performed on representative earth materials encountered near finish pad grades during rough grading operations. Expansion Index testing was completed in accordance with Uniform Building Code (U.B.C.) Standard 18-2. Test results are presented in Appendix B, on Table B-3.

Soluble sulfate tests were performed on representative earth materials encountered near finish pad grades during rough grading operations. Soluble sulfate tests were completed in accordance with California Test Method No. 417. The test results are presented in Appendix B, on Table B-3.

Atterberg Limits were performed on a selected soil sample in accordance with ASTM D4318-93. Plastic Index value is presented on Table B-3.

Corrosion analyses, which include chloride content, minimum resistivity, and pH, were performed on a selected sample. The tests were performed in accordance with California Test Method (CTM) 422, CTM 643 and CTM 643, respectively. The test results are included in Table B-3.

6.0 CONCLUSIONS

6.1 WORK COMPLIANCE

Earthwork carried out under the observation and testing by ***Albus-Keefe & Associates, Inc***. was performed in substantial conformance with the project plans and specifications, the grading codes of the City of Santa Paula, and applicable portions of the project geotechnical requirements. ***Albus-Keefe & Associates, Inc.*** is not responsible for line and grade. Rough grading work for the site has been observed and tested in a manner consistent with the standard of care currently exercised by members of the profession practicing in the same general locality under similar conditions

6.2 FUTURE SITE DEVELOPMENT

From a geotechnical point of view, the rough graded configurations of the subject lots are considered suitable for future residential development as currently proposed, provided that the recommendations presented herein and in our referenced geotechnical reports are implemented during future grading and construction.

6.3 GEOLOGIC HAZARDS

6.3.1 Ground Rupture

No active faults are known to project through the site nor does the site lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. No active or potentially active faults were observed during grading or during earlier geotechnical investigations within the limits of the subject lots. Therefore, the potential for ground rupture due to fault displacement beneath the site is considered low.

6.3.2 Ground Shaking

The site is located in a seismically active area that has historically been affected by moderate to occasionally high levels of ground motion. The site lies in close proximity to several active faults; therefore, during the life of the proposed development, the property will probably experience moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the southern California region. Structural designs should consider the potential for high ground accelerations as discussed in the referenced geotechnical reports as well as the requirements of the CBC presented in Section 7.2 herein.

6.3.3 Liquefaction

Based on the as-graded site conditions, the potential for liquefaction at the site is considered very small.

6.3.4 Landslides

No landslides were identified within or adjacent to the subject lots during the rough grading operations.

6.3.5 Seiche and Tsunami

The site is elevated more than 1000 feet above sea level and is located a substantial distance from significant body of water. As such, the potential for hazards related to seiche and tsunami is considered remove.

6.4 SETTLEMENT

Based on the anticipated relatively light foundation loads, total and differential settlement is not anticipated to exceed 1 inch and ½-inch over 30 feet, respectively. The estimated magnitudes of settlement are considered within tolerable limits for the proposed structures.

6.5 GROUNDWATER

Adverse effects from groundwater or seepage are not anticipated at the subject site provided that future surface water is controlled to limit excessive subsurface infiltration from irrigation or concentrated runoff.

6.6 SLOPE STABILITY

The fill slopes constructed to support the subject lots are considered grossly and surficially stable provided that the recommendations presented in this report are implemented and maintained at all times.

Natural slopes located beyond the limits of grading are considered grossly stable. These slopes are mantled with varying thicknesses of colluvial soils that may be prone to sloughing during periods of rain.

6.7 SELECT MATERIALS FOR GEOGRID SLOPE AND SEGMENTAL WALLS

Laboratory testing of select samples for backfill used in construction of the geogrid-reinforced slope and segmental walls indicate they substantially meet the project requirements. Results of direct shear tests indicate the ultimate friction angle of soils used in backfill of the segmental retaining walls ranged from 31 to 33 degrees thereby exceeding the design value of 30.5 degrees. Results of sand equivalence tests indicate the select materials used for both the segmental retaining wall and the geogrid slope ranged from 28 to 65 with an average value of 45. The target value specified by the city reviewer was 30. While one test result indicates a value slightly below the specified value, the average was significantly over the specified value. We conclude the select materials used are in substantial conformance with the project requirements and will perform as intended.

7.0 RECOMMENDATIONS

7.1 POST GRADING CONSIDERATIONS

7.1.1 Site Drainage

Positive drainage devices, such as sloping concrete flatwork, graded swales, and/or area drains, should be provided around the new construction to collect and direct all surface water to a suitable discharge area. No rain or excess water should be allowed to pond in yard areas or near structures including homes, retaining walls, or segmental walls.

7.1.2 Irrigation Considerations

Excessive irrigation water can be detrimental to the performance of proposed site development. Water applied in excess of the needs of vegetation will tend to percolate into the ground. Such percolation can lead to nuisance seepage and shallow perched groundwater. Seepage can form on slope faces, on the faces of retaining walls, in streets, or other low-lying areas. These conditions could lead to adverse effects such as the formation of stagnant water that breeds insects, distress or damage of trees, surface erosion, slope instability, discoloration and salt buildup on wall faces, and premature failure of pavement. Excessive watering can also lead to elevated vapor emissions within homes that can damage flooring finishes or lead to mold growth inside the home.

Key factors that can help mitigate the potential for adverse effects of over watering include the judicious use of water for irrigation, use of irrigation systems that are appropriate for the type of vegetation and geometric configuration of the planted area, the use of soil amendments to enhance moisture retention, use of low-water demand vegetation, regular use of appropriate fertilizers, and seasonal adjustments of irrigation systems to match vegetation needs for water. A landscape architect or other knowledgeable professional should provide specific recommendations. Future homebuyers should be made aware of these issues and consequences.

7.1.3 Utility Trench Backfill

Trench excavations may be cut vertically up to a height of 4 feet provided that no adverse geologic conditions or surcharging of the excavations are present. Trench excavations greater than 4 feet in height should be laid back at a maximum gradient of 1:1 (h:v). Excavations that expose loose granular soils prone to sloughing and caving should be laid back to a flatter gradient or where practical, hydraulic shoring with appropriate lagging may be utilized. The project geologist or soil engineer should observe all trench excavations to provide specific recommendations. All trench excavations should conform to the requirements of CAL OSHA.

All utility trench backfill should be compacted to at least 90 percent of the laboratory standard. Trench backfill should be brought to relatively uniform moisture content of 100 to 135 percent of optimum, placed in lifts no greater than 18 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. The project geotechnical consultant should perform density testing, along with probing, to verify adequate compaction.

Within shallow trenches (less than 18 inches deep) where pipes may be damaged by heavy compaction equipment, imported clean sand having a Sand Equivalent of 30 or greater may be utilized. The sand should be placed in the trench, thoroughly watered, and then compacted with a vibratory compactor.

Where utility trenches are proposed parallel to any building footing (interior and/or exterior trenches), the bottom of the trench should not be located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base. For utility trenches located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base or crossing footing trenches, concrete or slurry should be used as trench backfill.

7.1.4 Re-Certification of Pads

Building pads will tend to become desiccated and weathered over time. If homes are not constructed on pads within approximately 6 months following completion of rough grading, the pads should be re-evaluated by the project geotechnical consultant to confirm they are still suitable for their intended use. Pads that become overly desiccated or eroded by rainfall may require some remedial earthwork to restore proper moisture and compaction near the surface prior to home construction.

7.2 SEISMIC DESIGN PARAMETERS

The closest known Type A active fault to the site is the San Andreas fault located approximately 52.0 km northeast of the site. The closest known Type B fault to the site is the onshore segment of the Oak Ridge fault located approximately 2.7 km west of the site. For design of the project in accordance with Chapter 16 of the 2001 CBC, seismic design factors are provided in Table 7.0.

TABLE 7.1

Parameter	Value
Seismic Zone Factor, Z	0.4
Soil Profile Type, S	S_D
Near Source Factor, Na	1.2
Near Source Factor, Nv	1.5
Seismic Coefficient, Ca	0.54
Seismic Coefficient, Cv	0.96

7.3 SETTLEMENT CONSIDERATIONS

Foundations should be designed to tolerate total and differential settlements of up to 1 inch and ½-inch over 30 feet, respectively.

7.4 SOIL EXPANSION

Testing of typical site soils within the subject lots were performed during recent rough grading operations. Laboratory test results for the subject lots indicate a Low expansion potential (CBC Table 18-1-B). Specific recommendations are provided in the following sections.

7.5 POST-TENSIONED FOUNDATIONS

7.5.1 General

The recommendations presented herein for foundations and slabs on grade are based on soils with a Medium expansion potential. ***Based on as-graded site conditions and results of laboratory testing conducted during rough grading operations, previous recommendations for foundation design and construction presented in the referenced reports remain applicable. For convenience, the recommendations are reiterated herein.***

7.5.2 Allowable Bearing Value

A bearing value of 1,500 pounds per square foot may be used for continuous beams founded at a minimum depth of 12 inches below the lowest adjacent grade. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for wind and seismic forces.

7.5.3 Lateral Resistance

A passive earth pressure of 250 pounds per square foot per foot of depth up to a maximum value of 1000 pounds per square foot may be used to determine lateral bearing for beams. A coefficient of friction of 0.30 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for wind and seismic forces.

The above values are based on beams placed directly against competent native soils or compacted fill. In the case where beam sides are formed, all backfill against the beams should be compacted to at least 90 percent of the laboratory standard.

7.5.4 Foundation Setbacks

At the rear of Lots 28-30, the upper 30 to 35 feet of the easterly bluff was removed and replaced with select granular soils reinforced with geogrids. This condition is anticipated to provide a more stable condition as well as reduce the potential magnitude of lateral fill extension (lateral movement of slopes due to expansive soils). In consideration of these improvements, the bottom outer edge of foundations for residential structures located adjacent a top of slope should be setback from the slope face a horizontal distance of at least 1/3 the height of the slope. The horizontal distance should not be less than 7 feet but need not exceed 20 feet.

The bottom outer edge of foundations located adjacent the top of segmental retaining walls should be setback from the wall such that the bottoms of footings are founded below a 1 to 1 (H:V) plane projected up from the base of segmental walls. The horizontal distance should not be less than 5 feet from the top of wall.

The above setbacks may be accomplished through the use of deepened footings or caissons below the foundation. If caissons are required, this office should provide specific recommendations.

7.5.5 Beam Dimensions

Perimeter edge beams for both one-story and two-story structures should be founded at a minimum depth of 15 inches below the lowest adjacent final ground surface. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs.

7.5.6 Slab on Grade

The thickness of the floor slabs should be determined by the project structural engineer with consideration of the requirements of CBC 1816; however, we recommend a minimum slab thickness of 4.5 inches.

All dwelling area floor slabs constructed on-grade should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 10-mil Visqueen or equivalent. A minimum of two (2) inches of clean sand having an SE of at least 30 should be placed over the membrane to promote uniform curing of the concrete. This vapor barrier system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Pre-saturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage floor slab areas should be thoroughly moistened to achieve a moisture content that is at least 110 percent over the optimum moisture content. This moisture content should penetrate to a minimum depth of 12 inches below the bottoms of the slabs.

Design in accordance with 2001 CBC Section 1816, may be based on the following parameters:

TABLE 7.2
Post-Tensioned Design Parameters

Parameter	Value
% Clay (portion passing No. 200 sieve)	50
Plastic Index	25
Plastic Limit	20
Clay Type	Montmorillonite
Depth to Constant Soil Suction (feet)	5
Constant Soil Suction (pF)	3.6
Velocity of Moisture Flow (in./mo.)	0.5
Subgrade Modulus (pci)	150

Values for e_m may be estimated from Figure 18-III-14 of the CBC based on the selected Thornthwaite moisture index. Although the CBC indicates a Thornthwaite index of –20, consideration should be given to non-climatic factors such as irrigation practices that could affect the assumed value. Values for y_m may utilize Table 18-III based on the parameters provided in the table above and the estimated e_m. Using a Thornthwaite index of –20, the e_m and y_m values are summarized below:

Edge Lift Moisture Variation Distance, e_m	2.6 feet
Edge Lift, y_m	0.316 inches
Center Lift Moisture Variation Distance, e_m	5.3 feet
Center Lift, y_m	1.360 inches

7.6 FOOTING OBSERVATIONS

All footing trenches should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedments recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. All loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

7.7 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 7 feet in each direction. Cold joints should be keyed or provided with dowels spaced 18 inches on center. Special jointing detail should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to a moisture content of at least 120 percent of optimum to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete.

The geotechnical consultant should observe and verify the density and moisture content of subgrade soils prior to pouring concrete to ensure that the required compaction and pre-moistening recommendations have been met.

Drainage from flatwork areas should be directed to local area drains and/or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. The concrete flatwork should also be sloped at a minimum gradient of 2% away from building foundations and masonry walls.

7.8 CORROSION POTENTIAL

Laboratory testing for soluble sulfate content was completed on representative samples collected near rough grades. The test results indicate onsite soils contain less than 0.10% soluble sulfate. We recommend that the procedures provided in C.B.C. Section 1904.3 and Table 19-A-4, for concrete exposed to sulfate-containing solutions be followed for **Negligible** Sulfate Exposure.

Testing for chloride levels in site soils does not indicate a corrosive environment to metals. However, site soils do indicate a minimum resistivity less than 2000 ohm-cm. As such, site soils are corrosive to metals. Structures fabricated from metal should have appropriate corrosion protection if they will be in contact with site soils. Under such conditions, a corrosion specialist should provide specific recommendations.

7.9 SEGMENTAL RETAINING WALL LIMITATIONS

Segmental walls were constructed with geogrids to within as shallow as 2 feet of finish grade. The depths of the existing geogrids will allow for the installation of turf and typical yard planting such as shrubs and small trees. Planting of trees that require excavations greater than 2 feet in depth should be reviewed by this office prior to planting. No trees should be planted such that the ultimate drip lines of the mature tree will extend beyond the wall face. Root systems from trees placed in close proximity to the wall could cause deterioration and/or displacement of the geogrids and facing units.

Any improvements constructed above the geogrids or within a 1 to 1 plane projected up from the bottom of the walls may impact performance of the wall system. No yard improvements should be constructed within this zone without specific recommendations provided by this office or other geotechnical consultant familiar with segmental retaining walls. We recommend that future buyers of these properties be advised of these special conditions.

Proposed precise grading will provide positive drainage away from the segmental retaining walls and this condition <u>must</u> be maintained. Alteration of drainage in areas above the segmental walls could create conditions that are detrimental to the long-term performance of these walls. Future buyers of these properties should be advised of the critical nature of maintaining positive drainage away from these walls.

It is imperative that the existing geogrids placed during the wall construction be protected in place. Cutting or removal of the geogrids could create conditions that would be detrimental to the long-term performance of the walls. Future buyers of these properties should be advised of the critical nature of maintaining the integrity and position of the geogrids.

7.10 GEOGRID SLOPE LIMITATIONS

The slope at the rear of Lots 28 through 30 was constructed with geogrid reinforcement. Geogrids were placed as shallow as 2 feet from finish grade. The depths of the existing geogrids will allow for the installation of turf and typical yard planting such as shrubs and trees. However, no trees should be planted within 5 feet of the top of slope to avoid damaging shallow geogrids.

Any improvements constructed within 3 feet of the top of slope or deeper than 8 feet below grade anywhere within the rear yards may impact the geogrids. No yard improvements should be constructed within 3 feet of the top of slope or deeper than 8 feet without specific recommendations provided by this office or other geotechnical consultant familiar with geogrid-reinforced slopes. We recommend that future buyers of these properties be advised of these special conditions.

7.11 PLAN REVIEWS AND CONSTRUCTION SERVICES

We recommend ***Albus-Keefe & Associates, Inc*.,** be engaged to review the foundation plans prior to construction. This is to verify that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. If we are not provided the opportunity to review these documents, we take no responsibility for misinterpretation of our recommendations.

We recommend that a geotechnical consultant be retained to provide soil engineering services during future post-grading construction of the project. These services are to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly, the project geotechnical consultant should review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appears to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

8.0 CLOSURE

This report has been prepared for the exclusive use of **Comstock Homes.** Professional judgments presented in this report are based on evaluations of the technical information gathered, on construction procedures reported by others, and on our general experience in the field of geotechnical engineering. Our engineering work and judgments rendered meet the standard of care of our profession at this time and locale. We do not guarantee or warranty the performance of the project in any respect.

This report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

We hope that this report fulfills the current needs of the project. If you have any questions, or require additional information, please contact the undersigned.

Respectfully submitted,

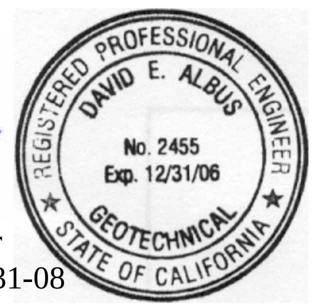
ALBUS-KEEFE & ASSOCIATES, INC.

Michael Putt
Project Geologist
CEG 2341

Reviewed by:

Patrick M. Keefe
Principal Engineering Geologist
CEG 2022

David E. Albus
Principal Engineer
GE 2455, exp 12-31-08



REFERENCES

Geotechnical Reports

Due Diligence Geotechnical Investigation, 16.17 Acre Parcel, Santa Paul Memorial Hospital Property, City of Santa Paula, California, by Geolabs – Westlake Village, dated March 21, 2005.

Grading Plan Review, 16.17 Acre Parcel, Ridgeview at Vista Glen, Santa Paul Memorial Hospital Property, City of Santa Paula, California, by Geolabs – Westlake Village, dated August 10, 2005.

Supplemental Geotechnical Investigation and Rough Grading Plan Review, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated May 3, 2006 (J.N.: 1489.00).

Oversize Rock Placement Criteria, Tract 5606, Santa Paula, California, by Albus-Keefe & Associates, Inc., dated May 3, 2006 (J.N.: 1489.00).

Engineering Analyses for Segmental Retaining Walls, Ridgeview at Vista Glen, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated May 10, 2006 (J.N.: 1489.00).

Response to City of Santa Paula Review, dated May 23, 2006, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated June 28, 2006 (J.N.: 1489.00).

Second Response to City of Santa Paula Review, dated July 25, 2006, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated July 31, 2006 (J.N.: 1489.00).

Supplemental Engineering Analyses for Segmental Retaining Walls (Delta 1), Ridgeview at Vista Glen, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated August 10, 2006 (J.N.: 1489.00).

Supplemental Information No. 2, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated August 22, 2006 (J.N.: 1489.00).

Yielding of Backfill Materials for Segmental Retaining Walls, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated September 19, 2006 (J.N.: 1489.00).

Evaluation of Bulge in Segmental Wall B, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated November 2, 2006 (J.N.: 1489.00).

Rough Grading Report for Lots 37 through 40 (Model Lots), Tract 5606, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated February 14, 2007 (J.N.: 1489.00).

REFERENCES (cont.)

Modification of Building Setbacks, Lots 23 through 30, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated March 12, 2007 (J.N.: 1489.00).

Geotechnical Foundation Recommendations for Screen Walls at Lots 1 and 40, Ridgeview at Vista Glen, Western Terminus of 10th Street, City of Santa Paula, California a, by Albus-Keefe & Associates, Inc., dated June 28, 2007 (J.N.: 1489.00).

Geotechnical Evaluation of Debris Flow Chutes, Tract 5606, Proposed 75-Lot Residential Development, Western Terminus of 10th Street, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated July 9, 2007 (J.N.: 1489.00).

Clearing of Loose Soil Below Lots 21-25 and 28-30 Generated During Rough Grading, Tract No. 5606, City of Santa Paula, California, by Albus-Keefe & Associates, Inc., dated October 2, 2007 (J.N.: 1489.00).

<u>Plans</u>

Rough Grading Plan by Development Resource Consultants, Inc., Sheets 5 through 11 of 16, dated April 10, 2006, Scale: 1"=20'.

Segmental Retaining Wall Plans, Ridgeview at Vista Glen, City of Santa Paula, by Albus-Keefe & Associates, Inc., dated August 10, 2006.

SEE SHEET NO. 8 FOR CONTINUATION

SEE SHEET NO. 5

SEE SHEET NO. 10 FOR CONTINUATION

CONSTRUCTION NOTES:

1. CONSTRUCT INTERCEPTOR DRAIN AT TOP OF SLOPE PER DETAIL "D" ON SHEET NO. 3.
2. CONSTRUCT TERRACE DRAIN PER DETAIL "C-2" ON SHEET NO. 3.
3. CONSTRUCT 9.0' WIDE "HILLSIDE" TRAIL PER DETAIL ON SHEET NO. 4.
4. CONSTRUCT DOWNDRAIN PER DETAIL ON SHEET NO. 3.
5. CONSTRUCT "VERDURA" MSE WALL PER SOIL RETENTION SYSTEMS PRODUCT RECOMMENDATIONS AND SEPARATE PLAN AND PERMIT.
6. CONSTRUCT MASONRY RETAINING WALL PER SEPARATE PLAN AND PERMIT.
7. CONSTRUCT CONCRETE 3' WIDE x 1.0' DEEP "V-DITCH" PER DETAIL ON SHEET NO. 4.
8. CONSTRUCT SPLASH WALL PER DETAIL ON SHEET NO. 4.
9. CONSTRUCT TERRACE DRAIN / DOWNDRAIN TRANSITION PER DETAIL ON SHEET NO. 3.
10. INSTALL 4" PVC SCH 40 DRAIN PIPE, SEE DETAIL ON SHEET NO. 3.
11. INSTALL 6" DIA. DRAIN INLET, NDS OR APPROVED EQUAL, SEE DETAIL ON SHEET NO. 3.
12. CORE DRILL CURB FOR 3" PVC PIPE, SEE DETAIL ON SHEET NO. 3.
13. CONSTRUCT REINFORCED CONCRETE STAIRWAY PER A.P.W.A. STD. PLAN NO. 640-1, AND DIMENSIONS AS SHOWN ON SHEET NO. 7.
14. CONSTRUCT MASONRY BLOCK WALL WITH TUBULAR STEEL ON TOP, SEE LANDSCAPE ARCHITECT'S PLAN FOR DESIGN AND CONTRUCTION DETAILS.
15. CONSTRUCT GARDEN WALL WITH FENCING ATOP .
16. PIPE CONNECTION TO EXISTING STORM DRAIN PER APWA STD. PLAN NO. 335-1, CASE 1.
17. P.V.C. PIPE CONNECTION, SEE DETAIL ON SHEET 3.
18. INSTALL 8" PVC SCH 40 DRAIN PIPE.

NOTE TO CONTRACTOR:

1. SEE APPROVED RETAINING WALL PLAN FOR WALL CONSTRUCTION DETAILS.
2. TEMPORARY EXCAVATION: IN THE EVENT SLOT-CUTTING IS REQUIRED A DETAIL WILL BE PROVIDED BY THE SOILS ENGINEER.
3. REFER TO THE APPROVED STORM DRAIN PLAN FOR STORM DRAIN CONSTRUCTION DETAILS.
4. SOUTHERN CALIFORNIA EDISON (S.C.E.) TRANSMISSION AND DISTRIBUTION LINES ARE NOT REQUIRED TO BE UNDERGROUND AND SHALL BE PROTECTED IN PLACE.

EXPLANATION

(LOCATIONS APPROXIMATE)

GEOLOGIC UNITS:

- Qt - Terrace Deposits
- Qcolo - Older Colluvium
- TQsa - Bedrock: Saugus Formation

LINES AND SYMBOLS:

- 52 - Bedding Attitude
- 80 - Joint Attitude
- Vertical Joint Attitude
- ? - Geologic Contact (querried where uncertain)
- 583, 4 - Subdrain: indicating end (T), outlet (circle), and top of pipe elevations in feet
- Fill Key
- VW185 - Field Density Test (See Table A-1 For Test Code Explanation)
- 588 - Excavation Bottom Elevation (in feet)
- F E - Limits of This Report (F = fill, E = existing)
- NAP - Not a Part of this Report

Note: Geologic Conditions Depicted at Cut and/or Removal Bottom Grades

CITY OF SANTA PAULA
PUBLIC WORKS DEPARTMENT
970 VENTURA STREET, SANTA PAULA, CALIFORNIA 93060
PHONE NO.: (805) 933-4212; FAX NO.: (805) 525-6660

SITE IMPROVEMENTS
ROUGH GRADING PLAN

PREPARED BY:
DRC Development Resource Consultants, Inc.
Civil Engineering - Land Surveying - Land Planning
8175 EAST KAISER BOULEVARD
ANAHEIM HILLS, CA 92808 (714) 685-6860

R. C. E.: 67577 EXPIRES: 06-30-07 DATE

REGISTERED PROFESSIONAL ENGINEER
ONOFRE RAMIREZ
No. 67577
Exp. 06-30-07
CIVIL
STATE OF CALIFORNIA

REV.	SYM.	DESCRIPTION	R.C.E.	DATE

ACCEPTED BY: ______ PUBLIC WORKS DIRECTOR/CITY ENGINEER DATE

RECOMMENDED BY: ______ ASSISTANT CITY ENGINEER DATE

DESIGNED BY:	DRAWN BY:	CHECKED BY:	DRAWING NUMBER
ORS	ORS	DRL	
DATE: 04/10/06	DATE: 04/10/06	DATE: 04/10/06	SHEET 9 OF 16

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

PLOT PLAN

Job No.: 1489.00	Date: 10/23/07	Plate: 1

APPENDIX A

SUMMARY OF FIELD DENSITY TEST RESULTS

TABLE A

SUMMARY OF FIELD DENSITY TEST RESULTS

Test Number	Test Date	Test Type (*)	Location (Northing) (Tract)	(Easting) (Lot)	Elev./ Depth (ft.)	Moist. Content (%)	Dry Density (pcf)	Max. Dry Density (pcf)	Opt. Moist. (%)	Max. Curve No.	Rel. Comp. (%)	Req. Comp. (%)	Pass/ Fail (P/F)
13	08/31/06	N	11640	5375	604.0	11.5	113.1	122.0	12.5	B	93	90	P
14	08/31/06	N	11750	5365	607.0	11.2	112.5	122.0	12.5	B	92	90	P
15	08/31/06	N	11670	5290	608.0	12.5	111.3	122.0	12.5	B	91	90	P
16	08/31/06	N	11775	5345	610.0	12.8	111.7	122.0	12.5	B	92	90	P
17	09/01/06	N	11685	5375	610.0	13.2	109.5	122.0	12.5	B	90	90	P
18	09/01/06	N	11780	5290	612.0	12.9	113.5	122.0	12.5	B	93	90	P
20	09/01/06	N	11750	5340	612.0	12.6	112.7	122.0	12.5	B	92	90	P
21	09/01/06	N	11610	5307	606.0	14.4	111.0	122.0	12.5	B	91	90	P
40	09/06/06	N	11795	5295	614.0	15.3	112.7	122.0	12.0	B	92	90	P
43	09/06/06	N	11500	5315	603.0	12.6	113.7	122.0	12.5	B	93	90	P
44	09/06/06	N	11570	5355	605.0	13.7	112.0	122.0	12.5	B	92	90	P
45	09/06/06	N	11515	5350	606.0	13.5	111.7	122.0	12.5	B	92	90	P
48	09/07/06	N	11435	5340	600.0	16.4	111.2	122.0	12.5	B	91	90	P
53	09/08/06	N	11445	5325	602.0	12.9	113.5	122.0	12.5	B	93	90	P
VW 97	09/14/06	N	11465	5165	607.0	15.8	109.3	131.0	9.0	D	83	90	F
VW 97 A	09/18/06	N	11465	5165	607.0	12.3	119.6	131.0	9.0	D	91	90	P
VW 98	09/14/06	N	11540	5160	607.0	14.7	111.7	131.0	9.0	D	85	90	F
VW 98 A	09/18/06	N	11540	5160	607.0	10.9	121.2	131.0	9.0	D	93	90	P
VW 115	09/18/06	N	11455	5170	607.0	11.1	121.4	131.0	9.0	D	93	90	P
VW 116	09/18/06	N	11495	5165	608.0	12.4	121.4	131.0	9.0	D	93	90	P
VW 117	09/18/06	N	11525	5165	608.0	11.5	121.5	131.0	9.0	D	93	90	P
VW 118	09/18/06	N	11480	5165	608.0	11.0	121.5	131.0	9.0	D	93	90	P
VW 119	09/18/06	N	11615	5150	609.0	15.3	115.7	131.0	9.0	D	88	90	F
VW 119 A	09/19/06	N	11495	5165	609.0	10.1	125.3	131.0	12.5	D	96	90	P
VW 142	09/20/06	D	11415	5385	604.0	9.1	122.3	131.0	9.0	D	93	90	P
154	09/22/06	N	11420	5330	604.0	12.9	110.5	122.0	12.5	B	91	90	P
VW 159	09/25/06	D	11680	5150	610.0	13.5	110.8	122.0	12.5	B	91	90	P

* See last page of this table for explanations.

ALBUS-KEEFE & ASSOCIATES, INC.

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SUMMARY OF FIELD DENSITY TEST RESULTS

Test Number	Test Date	Test Type (*)	Location (Northing) (Tract)	Location (Easting) (Lot)	Elev./ Depth (ft.)	Moist. Content (%)	Dry Density (pcf)	Max. Dry Density (pcf)	Opt. Moist. (%)	Max. Curve No.	Rel. Comp. (%)	Req. Comp. (%)	Pass/ Fail (P/F)
VW 160	09/25/06	N	11515	5160	610.0	12.5	116.2	122.0	12.5	B	95	90	P
VW 161	09/25/06	N	11620	5150	612.0	10.2	122.1	131.0	9.0	D	93	90	P
VW 162	09/25/06	D	11510	5165	612.0	10.5	120.2	131.0	9.0	D	92	90	P
VW 163	09/25/06	N	11555	5160	614.0	9.6	123.1	131.0	9.0	D	94	90	P
VW 164	09/25/06	N	11585	5155	614.0	9.0	122.8	131.0	9.0	D	94	90	P
VW 165	09/25/06	N	11720	5135	616.0	9.8	121.9	131.0	9.0	D	93	90	P
VW 166	09/26/06	N	11455	5165	616.0	9.3	120.7	131.0	9.0	D	92	90	P
VW 167	09/26/06	N	11595	5150	618.0	8.4	120.5	131.0	9.0	D	92	90	P
VW 168	09/26/06	N	11645	5150	618.0	9.6	119.7	131.0	9.0	D	91	90	P
VW 169	09/26/06	N	11715	5140	618.0	9.1	126.2	131.0	9.0	D	96	90	P
VW 170	09/26/06	N	11450	5170	620.0	10.3	123.4	131.0	9.0	D	94	90	P
VW 171	09/26/06	N	11525	5160	620.0	8.1	121.2	131.0	9.0	D	93	90	P
VW 172	09/26/06	N	11485	5160	622.0	7.7	120.1	131.0	9.0	D	92	90	P
VW 173	09/27/06	N	11740	5130	620.0	7.5	121.3	131.0	9.0	D	93	90	P
VW 175	09/27/06	N	11775	5115	624.0	9.0	119.5	131.0	9.0	D	91	90	P
VW 176	09/27/06	N	11770	5115	626.0	9.2	122.2	131.0	9.0	D	93	90	P
VW 177	09/27/06	N	11780	5110	628.0	9.5	124.9	131.0	9.0	D	95	90	P
VW 178	09/27/06	N	11785	5100	630.0	10.9	120.0	131.0	9.0	D	92	90	P
VW 188	09/29/06	N	11400	5385	605.0	8.4	120.7	131.0	9.0	D	92	90	P
197	10/03/06	N	11405	5125	623.0	13.9	110.6	122.0	12.5	B	91	90	P
202	10/04/06	N	11605	5150	618.0	12.9	113.0	122.0	12.5	B	93	90	P
203	10/04/06	N	11620	5140	620.0	12.6	111.7	122.0	12.5	B	92	90	P
204	10/04/06	N	11590	5140	622.0	14.2	109.5	122.0	12.5	B	90	90	P
205	10/04/06	N	11640	5135	624.0	13.7	111.1	122.0	12.5	B	91	90	P
FG 206	10/04/06	N	11630	5145	623.0	12.9	111.7	122.0	12.5	B	92	90	P
207	10/05/06	N	11690	5125	625.0	12.9	106.9	122.0	12.5	B	88	90	F
207 A	10/05/06	N	11690	5125	625.0	11.9	112.1	122.0	12.5	B	92	90	P

* See last page of this table for explanations.

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208	10/05/06	N	11550	5145	626.0	12.8	112.6	122.0	12.5	B	92	90	P
209	10/05/06	N	11715	5120	626.0	14.6	110.5	122.0	12.5	B	91	90	P
210	10/05/06	N	11725	5115	627.0	15.3	114.1	122.0	12.5	B	94	90	P
211	10/05/06	N	11765	5100	627.0	13.7	115.4	122.0	12.5	B	95	90	P
212	10/05/06	N	11530	5145	628.0	13.5	112.9	122.0	12.5	B	93	90	P
214	10/05/06	N	11515	5150	630.0	14.0	111.8	122.0	12.5	B	92	90	P
215	10/05/06	N	11710	5110	630.0	15.3	114.3	122.0	12.5	B	94	90	P
216	10/06/06	N	11575	5135	631.0	12.6	112.5	122.0	12.5	B	92	90	P
219	10/06/06	N	11510	5110	633.0	14.7	106.0	122.0	12.5	B	87	90	F
219 A	10/06/06	N	11510	5110	633.0	15.2	109.8	122.0	12.5	B	90	90	P
220	10/06/06	N	11470	5135	634.0	14.5	111.0	122.0	12.5	B	91	90	P
221	10/06/06	N	11555	5115	635.0	14.3	113.2	122.0	12.5	B	93	90	P
222	10/06/06	N	11715	5105	636.0	13.9	112.2	122.0	12.5	B	92	90	P
223	10/06/06	N	11615	5115	637.0	13.4	112.9	122.0	12.5	B	93	90	P
224	10/09/06	N	11590	5115	638.0	15.6	112.4	122.0	12.5	B	92	90	P
233	10/10/06	N	11610	5105	646.0	12.6	116.0	122.0	12.5	B	95	90	P
234	10/10/06	N	11700	5100	647.0	15.3	115.7	122.0	12.5	B	95	90	P
243	10/11/06	S	11520	5230	603.0	12.9	112.9	122.0	12.5	B	93	90	P
244	10/11/06	N	11560	5205	605.0	13.7	107.2	122.0	12.5	B	88	90	F
244 A	10/11/06	S	11560	5205	605.0	14.5	115.1	122.0	12.5	B	94	90	P
245	10/19/06	N	11645	5260	605.0	15.1	109.5	122.0	12.5	B	90	90	P
246	10/19/06	S	11655	5200	607.0	13.6	110.3	122.0	12.5	B	90	90	P
247	10/19/06	S	11700	5200	608.0	15.0	112.0	122.0	12.5	B	92	90	P
248	10/19/06	S	11740	5205	610.0	12.5	111.0	122.0	12.5	B	91	90	P
249	10/20/06	S	11790	5205	612.0	11.9	114.7	122.0	12.5	B	94	90	P
250	10/20/06	N	11835	5160	617.0	12.6	113.5	122.0	12.5	B	93	90	P
253	10/23/06	N	11468	5254	604.0	13.2	112.6	122.0	12.5	B	92	90	P

* See last page of this table for explanations.

ALBUS-KEEFE & ASSOCIATES, INC.

TABLE A

SUMMARY OF FIELD DENSITY TEST RESULTS

Test Number	Test Date	Test Type (*)	Location (Northing) (Tract)	Location (Easting) (Lot)	Elev./ Depth (ft.)	Moist. Content (%)	Dry Density (pcf)	Max. Dry Density (pcf)	Opt. Moist. (%)	Max. Curve No.	Rel. Comp. (%)	Req. Comp. (%)	Pass/ Fail (P/F)
254	10/23/06	N	11545	5225	604.0	14.0	114.2	122.0	12.5	B	94	90	P
255	10/23/06	N	11850	5115	625.0	14.1	113.2	122.0	12.5	B	93	90	P
256	10/24/06	S	11590	5200	606.0	14.5	112.4	122.0	12.5	B	92	90	P
257	10/24/06	N	11630	5220	608.0	15.2	112.0	122.0	12.5	B	92	90	P
258	10/24/06	S	11675	5210	610.0	15.3	110.8	122.0	12.5	B	91	90	P
259	10/25/06	N	11735	5185	612.0	14.3	116.1	122.0	12.5	B	95	90	P
260	10/25/06	N	11770	5180	614.0	14.8	114.8	122.0	12.5	B	94	90	P
FS 275	10/27/06	N	11570	5140	630.0	15.4	112.2	122.0	12.5	B	92	90	P
FS 276	10/27/06	N	11660	5110	640.0	12.8	109.7	122.0	12.5	B	90	90	P
FS 277	10/27/06	N	11705	5120	630.0	13.3	112.5	122.0	12.5	B	92	90	P
317	12/20/06	N	11795	5185	616.0	15.1	113.8	122.0	12.5	B	93	90	P
318	12/20/06	N	11820	5145	623.0	15.3	113.9	122.0	12.5	B	93	90	P
323	12/20/06	N	11785	5160	618.0	13.1	110.5	122.0	12.5	B	91	90	P
378	01/22/07	N	11665	5520	575.0	15.6	114.4	122.0	12.5	B	94	90	P
379	01/22/07	N	11665	5517	577.0	13.9	112.7	122.0	12.5	B	92	90	P
380	01/22/07	N	11670	5515	579.0	14.7	113.0	122.0	12.5	B	93	90	P
381	01/23/07	N	11675	5515	581.0	14.0	116.2	122.0	12.5	B	95	90	P
382	01/23/07	N	11705	5508	580.0	16.1	112.1	122.0	12.5	B	92	90	P
383	01/23/07	N	11735	5487	580.0	15.3	111.4	122.0	12.5	B	91	90	P
384	01/23/07	N	11765	5465	583.0	14.0	111.8	122.0	12.5	B	92	90	P
FG 406	01/25/07	N	11425	5345	606.0	11.8	111.5	122.0	12.5	B	91	90	P
FG 407	01/25/07	N	11475	5340	606.3	14.5	112.1	122.0	12.5	B	92	90	P
466	03/13/07	N	11765	5520	585.0	10.3	119.5	131.0	9.5	D	91	90	P
467	03/14/07	N	11747	5468	586.0	9.7	123.5	131.0	9.5	D	94	90	P
468	03/15/07	N	11782	5450	587.0	10.0	120.8	131.0	9.5	D	92	90	P
469	03/16/07	N	11735	5532	588.0	11.6	127.3	131.0	9.5	D	97	90	P
470	03/19/07	N	11765	5495	587.0	10.9	122.2	131.0	9.5	D	93	90	P

* See last page of this table for explanations.

ALBUS-KEEFE & ASSOCIATES, INC.

TABLE A

SUMMARY OF FIELD DENSITY TEST RESULTS

Test Number	Test Date	Test Type (*)	Location (Northing) (Tract)	Location (Easting) (Lot)	Elev./ Depth (ft.)	Moist. Content (%)	Dry Density (pcf)	Max. Dry Density (pcf)	Opt. Moist. (%)	Max. Curve No.	Rel. Comp. (%)	Req. Comp. (%)	Pass/ Fail (P/F)
471	03/20/07	N	11711	5538	590.0	11.0	120.0	131.0	9.5	D	92	90	P
472	03/21/07	N	11778	5473	589.0	11.4	124.9	131.0	9.5	D	95	90	P
473	03/22/07	N	11720	5520	592.0	10.2	121.3	131.0	9.5	D	93	90	P
485	04/02/07	N	11770	5465	591.0	10.2	121.5	131.0	9.5	D	93	90	P
486	04/02/07	N	11810	5405	593.0	10.5	121.4	131.0	9.5	D	93	90	P
487	04/02/07	N	11695	5530	594.0	9.9	119.6	131.0	9.5	D	91	90	P
490	04/04/07	N	11760	5480	596.0	10.0	124.4	131.0	9.5	D	95	90	P
491	04/04/07	N	11695	4595	596.0	11.4	127.4	131.0	9.5	D	97	90	P
492	04/04/07	N	11750	5420	598.0	10.5	123.5	131.0	9.5	D	94	90	P
493	04/05/07	N	11716	5492	599.0	11.3	124.9	131.0	9.5	D	95	90	P
499	04/09/07	N	11650	5495	600.0	11.2	122.3	131.0	9.5	D	93	90	P
500	04/09/07	N	11745	5492	601.0	10.6	125.0	131.0	9.5	D	95	90	P
501	04/09/07	N	11845	5390	603.0	9.5	121.8	131.0	9.5	D	93	90	P
VW 502	04/09/07	N	11630	5505	601.0	11.3	123.7	131.0	9.5	D	94	90	P
VW 506	04/10/07	N	11595	5455	603.0	10.6	123.4	131.0	9.5	D	94	90	P
510	04/11/07	N	11768	5405	603.0	11.2	120.8	131.0	9.5	D	92	90	P
VW 511	04/11/07	N	11560	5400	605.0	9.9	122.2	131.0	9.5	D	93	90	P
VW 512	04/11/07	N	11580	5435	607.0	9.7	122.3	131.0	9.5	D	93	90	P
VW 518	04/13/07	N	11615	5480	609.0	9.6	123.3	131.0	9.5	D	94	90	P
VW 519	04/13/07	N	11575	5420	611.0	9.2	121.0	131.0	9.5	D	92	90	P
520	04/13/07	N	11735	5431	605.0	9.1	123.7	131.0	9.5	D	94	90	P
521	04/16/07	N	11780	5415	607.0	10.7	122.3	131.0	9.5	D	93	90	P
522	04/16/07	N	11815	5385	612.0	11.0	125.1	131.0	9.5	D	95	90	P
523	04/16/07	N	11850	5375	612.0	10.3	123.1	131.0	9.5	D	94	90	P
550	04/25/07	N	11415	5215	606.0	9.2	127.4	131.0	9.5	D	97	90	P
551	04/25/07	N	11415	5205	608.0	9.0	120.8	131.0	9.5	D	92	90	P
552	04/25/07	S	11420	5195	610.0	9.5	122.6	131.0	9.5	D	94	90	P

* See last page of this table for explanations.

ALBUS-KEEFE & ASSOCIATES, INC.

TABLE A

SUMMARY OF FIELD DENSITY TEST RESULTS

Test Number	Test Date	Test Type (*)	Location (Northing) (Tract)	Location (Easting) (Lot)	Elev./ Depth (ft.)	Moist. Content (%)	Dry Density (pcf)	Max. Dry Density (pcf)	Opt. Moist. (%)	Max. Curve No.	Rel. Comp. (%)	Req. Comp. (%)	Pass/ Fail (P/F)
553	04/25/07	N	11420	5190	612.0	10.9	123.5	131.0	9.5	D	94	90	P
554	04/25/07	N	11425	5180	614.0	11.8	124.9	131.0	9.5	D	95	90	P
555	04/25/07	N	11435	5175	616.0	9.0	122.4	131.0	9.5	D	93	90	P
556	04/25/07	N	11450	5170	618.0	9.1	122.2	131.0	9.5	D	93	90	P
557	04/26/07	N	11455	5165	620.0	9.9	120.0	131.0	9.5	D	92	90	P
558	04/26/07	S	11460	5165	622.0	10.5	121.3	131.0	9.5	D	93	90	P
559	04/26/07	N	11465	5160	624.0	10.6	123.5	131.0	9.5	D	94	90	P
564	04/25/07	N	11730	5380	606.0	15.3	112.5	122.0	12.5	B	92	90	P
565	04/25/07	N	11675	5435	604.0	13.8	110.8	122.0	12.5	B	91	90	P
566	04/25/07	N	11625	5445	606.0	14.5	109.5	122.0	12.5	B	90	90	P
567	04/25/07	N	11595	5410	608.0	13.7	111.1	122.0	12.5	B	91	90	P
573	04/26/07	N	11660	5455	609.0	16.0	110.9	122.0	12.5	B	91	90	P
574	04/26/07	N	11685	5490	610.0	15.3	111.7	122.0	12.5	B	92	90	P
575	04/26/07	N	11615	5420	610.0	15.6	113.8	122.0	12.5	B	93	90	P
576	04/26/07	N	11725	5405	609.0	14.7	115.8	122.0	12.5	B	95	90	P
577	04/26/07	N	11725	5455	611.0	14.9	115.5	122.0	12.5	B	95	90	P
584	04/30/07	N	11430	5285	601.0	13.8	111.9	122.0	12.5	B	92	90	P
585	04/30/07	N	11555	5280	604.0	14.9	109.6	122.0	12.5	B	90	90	P
586	04/30/07	N	11740	5270	608.0	15.2	112.9	122.0	12.5	B	93	90	P
VW 588	05/01/07	N	11645	5110	633.0	10.2	122.3	131.0	9.5	D	93	90	P
VW 589	05/01/07	N	11640	5110	635.0	9.6	121.8	131.0	9.5	D	93	90	P
VW 590	05/01/07	N	11640	5115	637.0	11.3	121.3	131.0	9.5	D	93	90	P
VW 591	05/01/07	N	11635	5113	639.0	10.1	123.4	131.0	9.5	D	94	90	P
VW 592	05/01/07	N	11653	5110	641.0	14.9	117.5	122.0	12.5	B	96	90	P
603	05/10/07	N	11400	5160	610.0	10.1	121.4	131.0	9.0	D	93	90	P
604	05/10/07	N	11405	5180	607.0	9.7	121.5	131.0	9.0	D	93	90	P
605	05/11/07	N	11400	5210	608.0	14.9	111.1	122.0	12.5	B	91	90	P

* See last page of this table for explanations.

ALBUS-KEEFE & ASSOCIATES, INC.

TABLE A

SUMMARY OF FIELD DENSITY TEST RESULTS

Test Number		Test Date	Test Type (*)	Location (Northing) (Tract)	(Easting) (Lot)	Elev./ Depth (ft.)	Moist. Content (%)	Dry Density (pcf)	Max. Dry Density (pcf)	Opt. Moist. (%)	Max. Curve No.	Rel. Comp. (%)	Req. Comp. (%)	Pass/ Fail (P/F)
FG	654	06/14/07	N	11705	5330	611.0	12.9	111.8	122.0	12.5	B	92	90	P
FG	655	06/14/07	N	11745	5315	612.5	13.7	113.8	122.0	12.5	B	93	90	P
FG	656	06/14/07	N	11725	5400	611.0	13.5	114.6	122.0	12.5	B	94	90	P
FG	657	06/14/07	N	11705	5450	611.8	13.0	113.6	122.0	12.5	B	93	90	P
FG	658	06/14/07	N	11660	5475	612.0	14.6	114.5	122.0	12.5	B	94	90	P
FG	659	06/14/07	N	11460	5230	606.2	13.8	114.2	122.0	12.5	B	94	90	P
FG	660	06/14/07	N	11520	5215	606.8	12.7	116.2	122.0	12.5	B	95	90	P
FG	661	06/14/07	N	11550	5200	607.3	12.7	114.1	122.0	12.5	B	94	90	P
FG	662	06/14/07	N	11600	5220	608.0	13.3	110.1	122.0	12.5	B	90	90	P
FG	663	06/14/07	N	11645	5200	609.1	12.4	115.5	122.0	12.5	B	95	90	P
FG	664	06/14/07	N	11685	5205	610.5	12.2	114.3	122.0	12.5	B	94	90	P
RW	695	08/13/07	N	11715	5210	612.0	12.7	121.8	131.0	9.0	D	93	90	P
RW	696	08/13/07	N	11710	5180	613.0	15.1	111.8	122.0	12.5	B	92	90	P
RW	697	08/13/07	S	11763	5212	615.0	11.9	120.5	131.0	9.0	D	92	90	P
RW	698	08/14/07	N	11755	5170	617.0	12.2	123.1	131.0	9.0	D	94	90	P
RW	699	08/14/07	N	11750	5150	619.0	14.6	112.3	122.0	12.5	B	92	90	P
RW	709	08/20/07	N	11775	5320	614.0	16.0	112.7	122.0	12.5	B	92	90	P
VW	717	09/11/07	S	11550	5385	602.0	10.8	121.3	131.0	9.0	D	93	90	P
VW	718	09/11/07	N	11510	5370	604.0	11.7	121.4	131.0	9.0	D	93	90	P
VW	719	09/11/07	N	11540	5375	606.0	10.6	119.1	131.0	9.0	D	91	90	P
VW	720	09/12/07	N	11555	5390	608.0	10.0	119.5	131.0	9.0	D	91	90	P
	729	09/28/07	N	11612	5384	608.0	11.8	110.1	122.0	12.5	B	90	90	P
	730	09/28/07	N	11788	5388	609.0	12.2	114.4	122.0	12.5	B	94	90	P
	731	09/28/07	N	11797	5383	611.0	10.3	110.5	122.0	12.5	B	91	90	P
	732	09/28/07	N	11765	5385	611.0	10.2	110.7	122.0	12.5	B	91	90	P

* See last page of this table for explanations.

TABLE A

EXPLANATION OF TEST CODES

PREFIX CODE DESIGNATION FOR TEST NUMBERS

ET-	Exploratory Trench	SG-	Segmental Wall	VW-	Vedura Wall
FG-	Finish Grade	SR-	Slope Repair		
OG-	Original Ground	RG-	Rough Grade		
SF-	Slope Face	RW-	Retaining Wall		

CODES FOLLOWING THE TEST NUMBER

A: Subsequent retest of failed density test after fill reconditioning and recompaction.
R: Failed material removed.

TEST TYPE

D: Drive-Cylinder Test per ASTM D2937
S: Sand Cone Test per ASTM D1556
N: Nuclear Gauge Test per ASTM D2922 and D3017

MAX. CURVE NO.

B, D: Corresponds to Max Curve Designation listed in Table B-1 (represents the laboratory maximum dry density/optimum moisture content for a representative fill material encountered during grading).

APPENDIX B

SUMMARY OF LABORATORY TEST RESULTS

TABLE B-1
Maximum Dry Density & Optimum Moisture

Max. Curve No.	Description	Max. Dry Density (pcf)	Optimum Moisture (%)
B	Reddish-Brown Sandy Silt with Clay (ML)	122.0	12.5
D	Import - Yellowish-Brown Sand (SP)	131.0	9.0

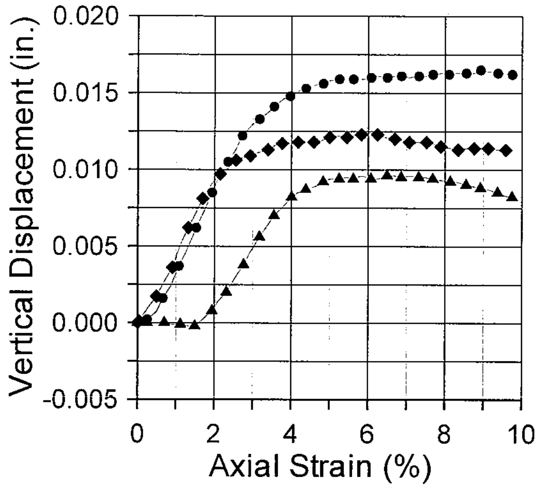
TABLE B-2
Sand Equivalence Tests

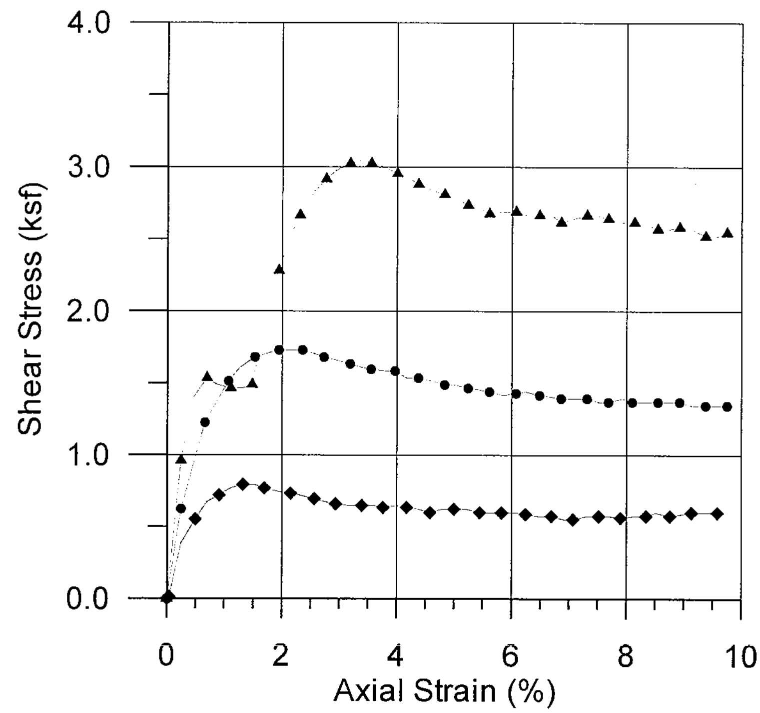
Sample No.	Description	SE
4	Import-Slightly Silty Sand (SM-SP)	35
5	Import – Well Grade Sand (SW)	65
6	Import – Well Grade Sand (SW)	53
7	Import- Silty Sand (SM-SP)	28

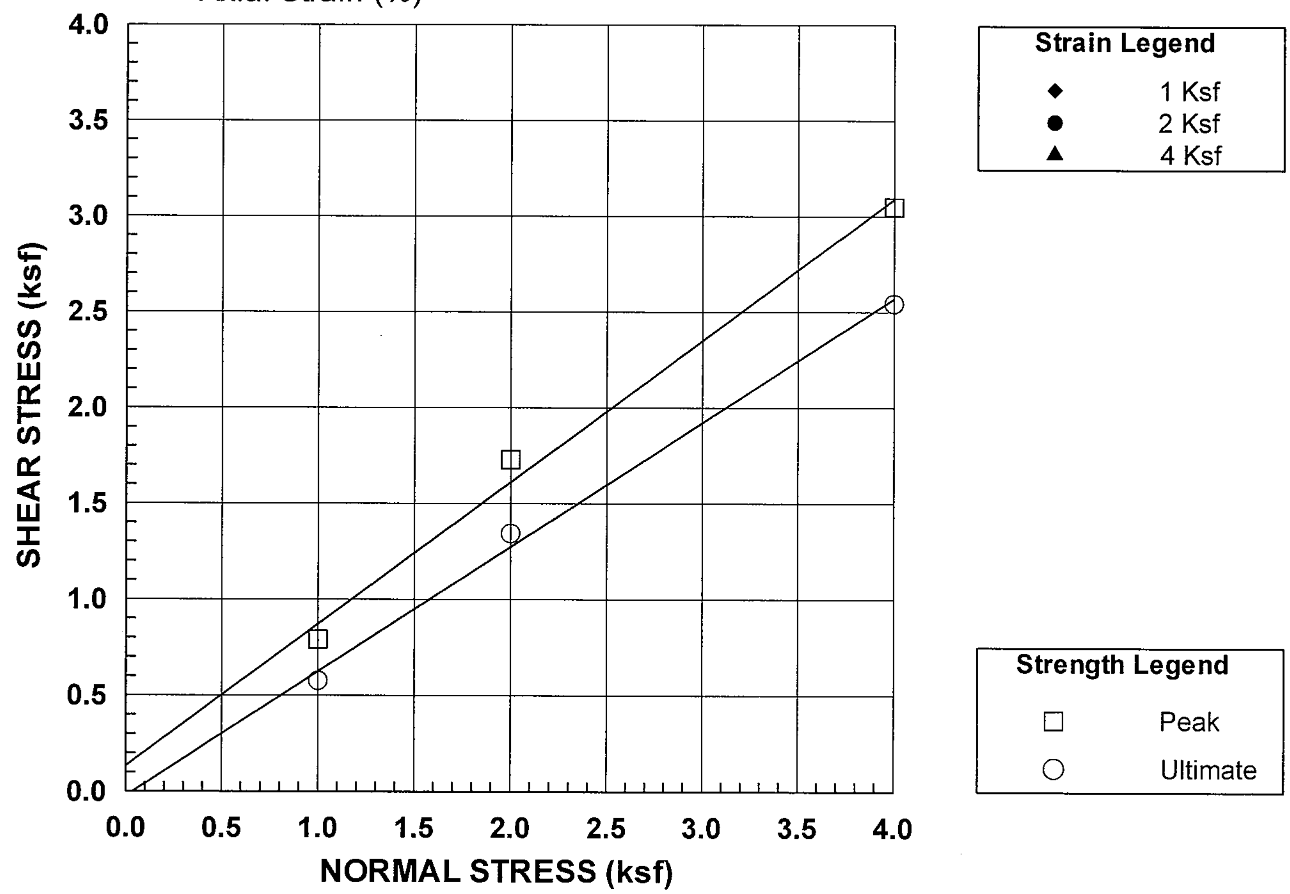
TABLE B-3
Summary of Expansion, Atterberg Limits, Sulfate and Corrosion Testing

	Test Results							
Building Pads	**Expansion Index**	**Expansion Potential**	**Plasticity Index**	**Soluble Sulfate Content**	**Sulfate Exposure**	**Minimum Resistivity (ohm-cm)**	**pH**	**Chloride Content (ppm)**
26-29	33	Low	_	0.002%	Negligible	-	-	-
30-34	47	Low	16.7	0.002%	Negligible	-	-	-
35	35	Low	14.0	0.015%	Negligible	1,100	7.6	280
44-50	46	Low	-	0.004%	Negligible	-	-	-

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	0.792	1.728	3.048
Peak Displacement (in)	0.032	0.0465	0.0815
Ultimate Stress (ksf)	0.576	1.344	2.544
Ultimate Displacement (in)	0.242	0.247	0.247
Initial Dry Density	117.9	117.9	117.9
Initial Moisture Content (%)	9.007	9.007	9.007
Strain Rate (in/min)	0.050		







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
#13 Wall B 595'	Remolded - Saturated	Silty Sand (SM)

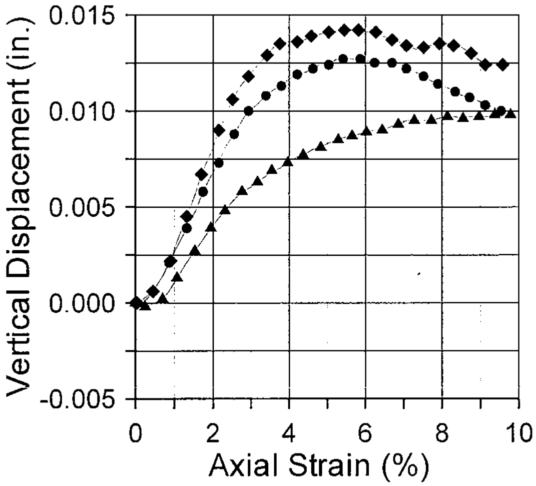
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

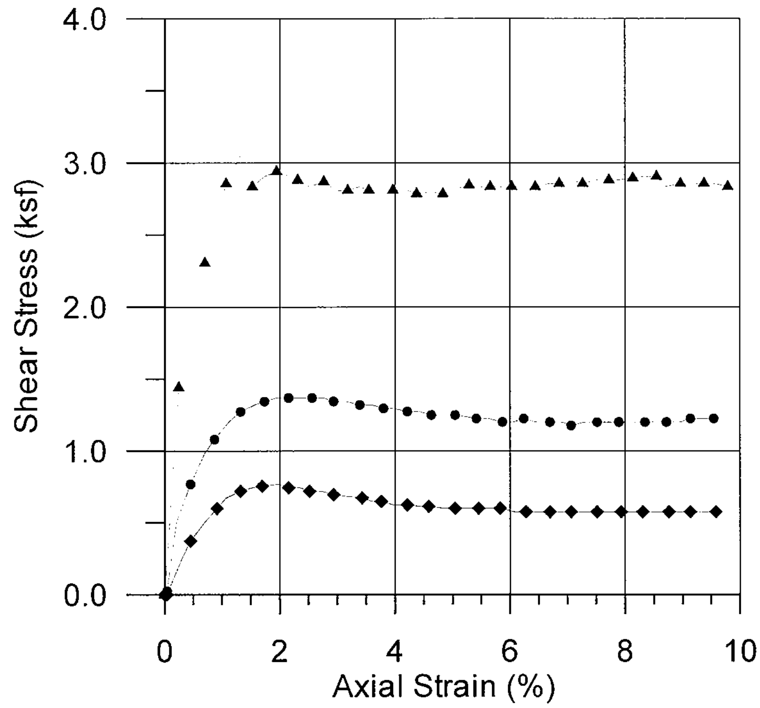
DIRECT SHEAR

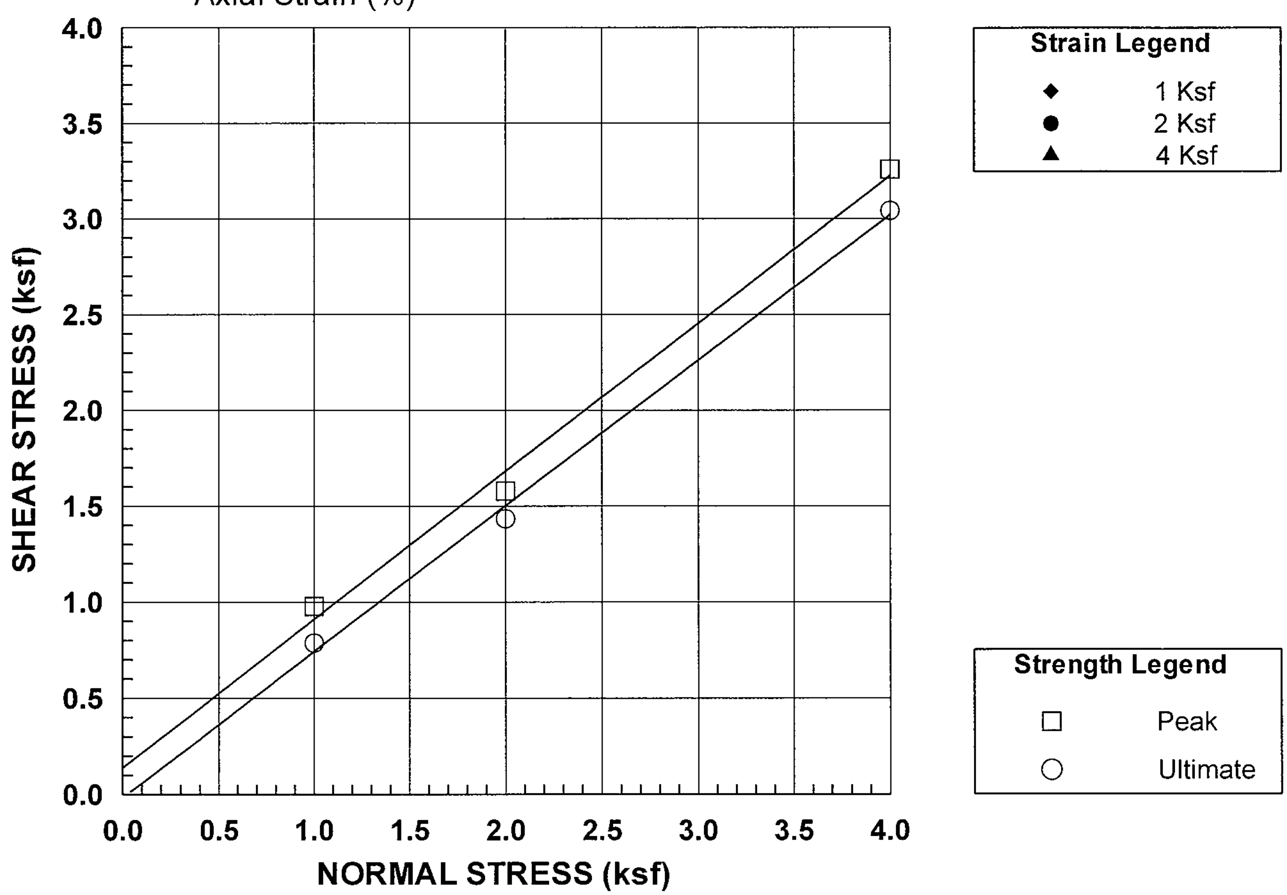
Job No: 1489.00

Plate No: B-1

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	0.978	1.578	3.258
Peak Displacement (in)	0.0475	0.0465	0.032
Ultimate Stress (ksf)	0.786	1.434	3.042
Ultimate Displacement (in)	0.242	0.241	0.246
Initial Dry Density	117.9	117.9	117.9
Initial Moisture Content (%)	9.024	9.024	9.024
Strain Rate (in/min)	0.050		







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
#14 Wall B 600'	Remolded - Saturated	Silty Sand (SM)

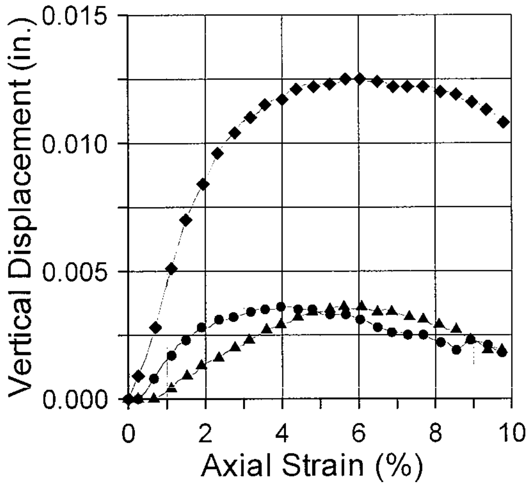
ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

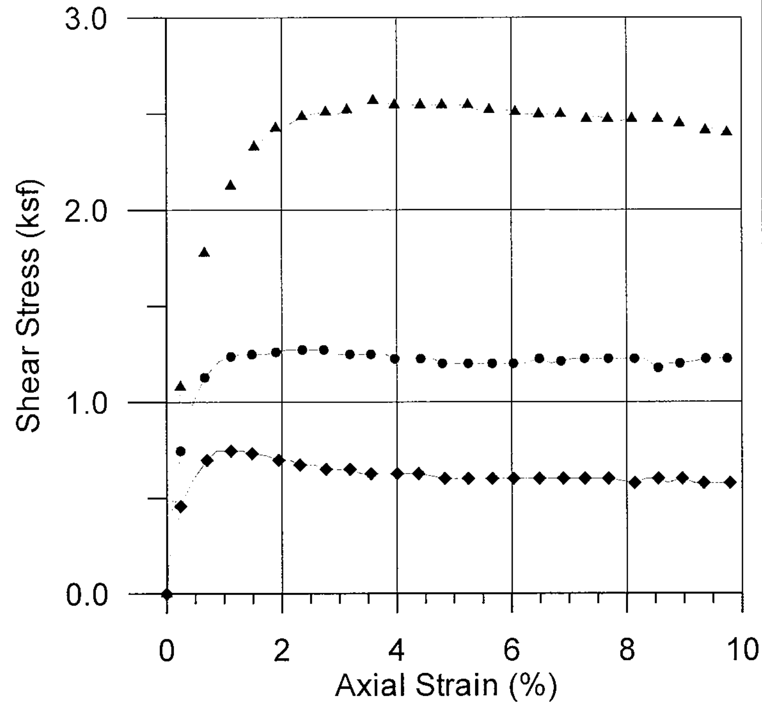
DIRECT SHEAR

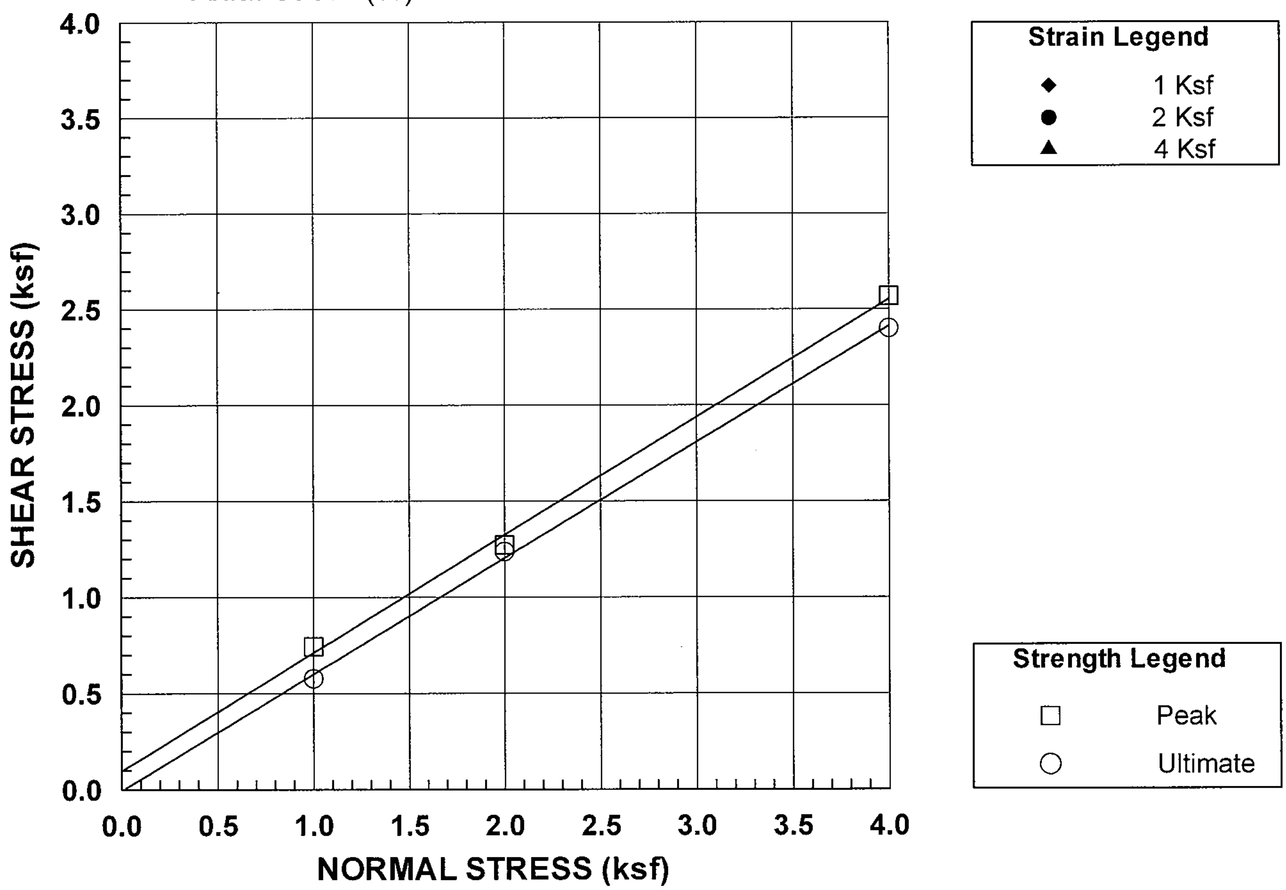
Job No: 1489.00

Plate No: B-2

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	0.744	1.272	2.568
Peak Displacement (in)	0.021	0.051	0.087
Ultimate Stress (ksf)	0.576	1.236	2.4
Ultimate Displacement (in)	0.246	0.247	0.247
Initial Dry Density	117.4	116.5	116.5
Initial Moisture Content (%)	9.995	10.86	10.86
Strain Rate (in/min)		0.050	







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
#18 Wall B 595'	Remolded - Saturated	Silty Sand (SM)

ALBUS-KEEFE & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	**DIRECT SHEAR**	**Job No: 1489.00**
		Plate No: B-3

APPENDIX C

MAXIMUM DEPTH OF FILL BENEATH LEVEL PAD AREAS

Table C-1
Maximum Depth of Fill Beneath Level Pad Areas

Lot Number	Depth of Fill (ft)	Lot Number	Depth of Fill (ft)
26	8	35	8
27	7	44	5
28	27	45	5
29	38	46	5
30	41	47	6
31	13	48	5
32	7	49	5
33	8	50	8
34	8		

APPENDIX F

CONTECH CONSTRUCTION PRODUCTS, INC.

Detention System Maintenance Guide

7290.200.101
March 10, 2008
Revised May 23, 2008

MAINTENANCE

Underground storm water detention and retention systems should be inspected at regular intervals and maintained when necessary to ensure optimum performance. The rate at which the system collects pollutants will depend more heavily on site activities than the size or configuration of the system.

Inspection

Inspection is the key to effective maintenance and is easily performed. CONTECH recommends ongoing quarterly inspections of the accumulated sediment. Sediment deposition and transport may vary from year to year and quarterly inspections will help insure that systems are cleaned out at the appropriate time. Inspections should be performed more often in the winter months in climates where sanding operations may lead to rapid accumulations, or in equipment washdown areas. It is very useful to keep a record of each inspection. A sample inspection log is included for your use.

Systems should be cleaned when inspection reveals that accumulated sediment or trash is clogging the discharge orifice. CONTECH suggests that all systems be designed with an access/inspection manhole situated at or near the inlet and the outlet orifice. Should it be necessary to get inside the system to perform maintenance activities, all appropriate precautions regarding confined space entry and OSHA regulations should be followed.

Cleaning

Maintaining an underground detention or retention system is easiest when there is no flow entering the system. For this reason, it is a good idea to schedule the cleanout during dry weather.

Accumulated sediment and trash can typically be evacuated through the manhole over the outlet orifice. If maintenance is not performed as recommended, sediment and trash may accumulate in front of the outlet orifice. Manhole covers should be securely seated following cleaning activities.

INSPECTION & MAINTENANCE LOG

__" Diameter System			Location: Anywhere, USA		
Date	**Depth of Sediment**	**Accumulated Trash**	**Maintenance Performed**	**Maintenance Personnel**	**Comments**
12/01/99	2"	None	Removed Sediment	B. Johnson	Installed
03/01/00	1"	Some	Removed Sediment and Trash	B. Johnson	Swept parking lot
06/01/00	0"	None	None		
09/01/00	0"	Heavy	Removed Trash	S. Riley	
12/01/00	1"	None	Removed Sediment	S. Riley	
4/01/01	0"	None	None	S. Riley	
04/15/01	2"	Some	Removed Sediment and Trash	ACE Environmental Services	

SAMPLE