SUPPLEMENTAL GEOTECHNICAL REPORT

Responding to

GEOTECHNICAL REVIEW COMMENTS

f o r

HIGHLAND ESTATES

San Mateo County, California

Submitted to: THE CHAMBERLAIN GROUP
San Carlos, California

Prepared by: SOIL FOUNDATION SYSTEMS, INC.
Mountain View, California

November, 1994

F S **SOIL FOUNDATION SYSTEMS, INC.**

Geotechnical Engineers • Engineering Geologists
Soil Mechanics Laboratories

2680 Bayshore Parkway, Suite 411, Mountain View, CA 94043 Tel: (415) 960-6830 Fax: (415) 960-6834

File No. S24-634-2S
November 7, 1994

The Chamberlain Group
P. O. Box 970
San Carlos, California 94070

Attention: Mr. Jack Chamberlain

Subject: HIGHLAND ESTATES
San Mateo County, California
SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
Responding to Geotechnical Review Comments

Gentlemen:

Transmitted herewith is our report responding to peer review comments on our 1993 geotechnical investigation report for the captioned project. The peer reviews were conducted by Earth Systems Consultants and Harlan Tait Associates.

The peer reviews were constructive and useful for focusing once again on the critical geological and geotechnical concerns. This report provides point-by-point responses to the review comments. Additional data is provided to further substantiate the findings and recommendations of the 1993 report. A broader range of alternative measures is presented for mitigation of the geological and geotechnical concerns than were given in the 1993 report.

This supplemental report does not alter the geotechnical conclusion of the 1993 report concerning the feasibility of the proposed project.

Very truly yours,

SOIL FOUNDATION SYSTEMS, INC.

R. Patrick Fain

Darwin Myers
C.E.G. 946

K. C. Sohn
G.E. 795

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INTRODUCTION

The geotechnical investigation report prepared by Soil Foundation Systems, Inc. (SFS) for the captioned project (dated June 10, 1993) was distributed to Earth Systems Consultants (ESC) and Harlan Tait Associates (HTA) for their review and comments. The purpose of this review process is to ensure that all significant geologic, seismic and geotechnical concerns are adequately addressed. We have received written comments from ESC in the letter dated September 27, 1993, and from HTA in the letter dated October 4, 1993. A copy of the report was delivered to the Highlands Community Association via San Mateo County Planning Division.

The comments of ESC and HTA may be broadly divided into the following sections:

Section A — Supplemental Subsurface Exploration
Section B — Geology and Seismicity
Section C — Geotechnical Engineering

Preparation of the responses required clarification of the 1993 report, supplemental analyses and literature review. A complete list of references cited in this supplemental report, as well as those listed in the 1993 report, is provided following the discussion of the pertinent subject matter. Supporting data is presented in the Appendix. This data includes logs of supplemental test pits and borings, supplemental laboratory data, exploratory trench and boring logs of other consultants, reinforced earth slope and retaining wall design analysis by TENSAR Earth Technologies, Inc., and calculation sheets for all supplemental analyses and previous analyses for which calculation sheets were not included in the 1993 report. Plates 1, 2 and 3 (overall geologic map, geologic map of the development area and geologic cross sections, respectively) of the 1993 report are revised, and an isopach map of surficial deposits is presented. The revised Plates 1, 2 and 3 are labeled as 1-REV, 2-REV and 3-REV, respectively, while the isopach map is labeled as Plate 4. These Plates are all provided in the pocket located inside the back cover of this report.

This supplemental report is organized to make point-by-point responses to the technical comments. For clarity, each comment is identified by the reference page number and comment number. Comments are italicized for further clarity.

The comments resulted in further elaboration on techniques available for control of settlement effects on buildings and stabilization of slopes. Section D is provided in this supplemental report to conveniently summarize all the alternative mitigation measures, including those considered in the 1993 report. Note that Section D constitutes supplemental recommendations to the 1993 report.

SECTION A — SUPPLEMENTAL SUBSURFACE EXPLORATION

Earth Systems Consultants comments 4 (page 4) and 9 (page 5), and HTA comment 4 (page 2) indicated a need for supplemental subsurface exploration at the three locations described below:

Location (a): This area is just above the old cut slope behind the Hillsborough West Apartments, to the north and below Boring B-20. ESC indicated the occurrence of a slope instability during construction of a 1.5:1 cut slope during the 1960's in this general vicinity.

Location (b): This area is the colluvium-covered swale on Lots 2 and 3. It was the opinion of ESC that this area may be mantled by landslide deposits.

Location (c): This area is the slope between Borings B-2 and B-8. The reviewers indicated that additional subsurface data was needed for adequate assessment of the stability of the proposed retaining walls and slopes in this area.

On November 9, 1993, Soil Foundation Systems submitted a supplemental exploration plan to the San Mateo County Planning Division, which included a map showing the proposed locations for trenching and/or test borings. Upon approval of the plan by the County, the field work was performed on December 3, 1993, at all three locations, following an on-site meeting attended by representatives of Highlands Community Association; Michael Marangio, a contract biologist for EIP Associates; David Connell of HTA; Jean DeMouthe of ESC; and representatives of The

Chamberlain Group. The exploration plan and procedure were presented at this meeting by K. C. Sohn, project geotechnical engineer of SFS, and Darwin Myers, project geologist of Darwin Myers and Associates. Access to each location was first approved by the biologist.

At location (a), a test boring was drilled to a depth of 21 feet with a Minuteman rig using 3-inch diameter continuous flight augers. This boring location is labeled as BS-1 on Plate 2-REV. During drilling, the hole was kept clear of cuttings by periodically pulling the entire length of the auger out of the hole, and the time taken for each 12 inch advance of auger was recorded. This advance rate is a relative measure of resistance to drilling at the tip. The log of the boring, including the profile of the auger advance rate, is presented on Plate A in Section A of the Appendix.

At location (b), a Minuteman boring (BS-2 on Plate 2-REV) was drilled to a depth of 11 feet in the axis of the swale, near the south corner of Lot 3, and a test pit was excavated on Lot 1 (TP-7). The log of the boring is presented on Plate B, and the trench log is shown on Plate C.

At location (c), two exploratory trenches were excavated (TP-5 and TP-6). Logs of these test pits are presented on Plate C.

Each boring was logged by the project geotechnical engineer during the drilling operation and the test pits were logged by the project geologist. Test pits and borings, along with field procedures, were observed by the geologist from Earth Systems Consultants.

Laboratory testing for the supplemental investigation included a laboratory compaction test and oedometer tests on materials anticipated for use as compacted fill. The results of the laboratory tests are shown on Plates E through I in Section A of the Appendix.

Analysis of the subsurface data and conclusions from the supplemental subsurface exploration are presented in other sections of this report in response to specific comments.

SECTION B — GEOLOGY AND SEISMICITY

ESC Background Comment A, page 2: Evidence of Instability In and Around Site

Natural and man-made slopes within and around the study area have experienced various forms of slope instabilities over the past several decades. Some of these have had little or no effect on developments in the area, while others have had significant impact on roadways, houses and related improvements. For example, public and private property along Ralston Avenue, Polhemus Road, Highway 92, Interstate 280, De Anza Boulevard, Timberlane Way, and other streets in the general site vicinity have experienced various types of slope failure in the past (San Mateo County, 1975).

Response:

Predicting where damaging landslides will occur is important for mitigating losses. The causes of slope failures can span a broad range. In many of the nearby projects that have experienced slope failures, information on the geology of a site was incomplete. An important contributory cause of damage in several of the cited slides was heavy flow of groundwater. The complexity of geology within the Franciscan Assemblage may also have been a factor. In areas of highly sheared, deeply weathered or incompetent rock, stabilization of the weak materials is needed to assure long term stability.

It should also be recognized that standards for geotechnical reports have evolved over the years. There are improved methods of analyzing slopes and new methods of remediating hillside properties, using technology that was not available 20 years ago. Additionally, building and grading codes have been refined over the years. As a consequence, sites that may have been difficult to develop in the earlier decades can now be successfully engineered.

For the Highland Estates project, the subsurface data yielded by previous investigations (United Soil Engineering, 1977; Berlogar, Long & Associates, 1980), in combination with the subsurface work performed by Soil Foundation Systems, Inc., provides a reliable geologic data base upon which to formulate recommendations for site grading, drainage and foundation design. The supplemental subsurface exploration described in the previous section has provided useful additional data to substantiate the geologic interpretation of the site presented in the 1993 report.

ESC Background Comment B, page 2: Slope Instability During Development of Hillsborough West Apartments

Of particular significance to this project are the slope instabilities that occurred behind the Hillsborough West Apartment complex when that project was developed in the 1960's. Initial development plans called for a third building to be placed behind the two existing buildings and a continuous perimeter driveway to surround the complex, generally following the outline of the property boundary. When the cut slopes were created behind the existing buildings, the slopes became unstable to the point that the plans to construct the third building and the perimeter driveway were abandoned. In an attempt to stabilize the slopes, extensive subsurface drains were installed uphill of the cuts in the areas described on Plate 1 in the SFS report as "Disturbed Ground (portion of the site previously graded)" It also appears that the ground was left higher by placing fill at the base of the cut slope to provide additional support and mitigate possible deeper slope failures. Erosion control planting, including numerous pine trees, was implemented in an effort to improve slope stability.

Response:

Prior to issuing the 1993 SFS report, we researched the county files and were unable to locate geotechnical reports for the Hillsborough West Apartments. Moreover, there was no written record of the instability described in the preceding comment.

The personal recollection of the reviewer, in combination with the results of the supplemental study presented in this report, provides data needed to understand the mechanism and geometry of this slope failure. The slope was involved in a toe failure that occurred when the toe excavation exceeded the depth to the contact between the melange and sandstone units. This failure is inferred to have involved sliding of the melange unit on more competent sandstone. Buttressing the toe of the slope, unloading the slope at the top, along with drainage improvements, have controlled mass wasting on the slope. This slope is discussed in detail in later sections of this report (see pages 13, & 56-57).

ESC Geology Comment 1, page 3

The geologic setting (regional geology and bedrock geology) is well described in the SFS report.

Response:

None required.

ESC Geology Comment 2, page 3

The site geology, as depicted on Plates 1 and 2 of the SFS report, is reportedly based on the consultant's interpretation of limited surface exposures and subsurface data obtained from borings and backhoe pits. The SFS report indicates (page 12) that data from a 1980 report by Berlogar, Long & Associates was used in the preparation of the Geologic Maps, Plates 1 and 2, and in preparation of the Geologic Cross Sections, Plate 3. The SFS report also references an earlier report by United Soil Engineering, Inc.

- *The Berlogar test pit logs should be presented in the SFS report, and their locations should be shown on Plate 2 in order to expedite the review.*
- *Surface and subsurface data from the 1977 report by United Soil Engineering, Inc. should also be incorporated into the SFS report.*
- *Each of the previous reports describes slope instabilities in the southeastern portion of the project site. The SFS report should evaluate and discuss the findings from the previous*

reports and the geotechnical reviews pertaining to each report. Sufficient information should be provided by SFS to substantiate differences of opinion or different conclusions from those presented in the previous reports.

Response:

Earlier reports prepared by other consultants for the southern portion of the project site include:

a) A geologic report by United Soil Engineering, Inc., dated October 31, 1977,
b) A geotechnical investigation report by Berlogar, Long & Associates, dated February 11, 1980,
c) A review report by Earth Metrics Incorporated, dated July 2, 1982, of the 1980 Berlogar-Long report, and
d) A review report by Howard-Donley Associates, Inc., dated April 22, 1983, of the 1980 Berlogar-Long report.

The 1977 subsurface investigation program of United Soil Engineering, Inc. included logging of nine test borings ranging from 4 to 21.5 feet in depth. Except for penetration resistance data from 11 points in six holes, no physical data (i.e., laboratory test data) was obtained to substantiate the engineering characteristics of the site. A geologic map showing the boring locations and logs of the borings from the 1977 investigation by United Soil Engineering, Inc. are included in Section B of the Appendix. A primary product of the 1977 investigation was a geologic map of the property that showed most of the site to be within the outcrop belt of sheared Franciscan rock. One boring penetrated a zone of material described as questionable old slide material, terminating at 13 feet below the ground surface. Although no geomorphic evidence of a landslide was reported by United Soil Engineering, the presence of this weak zone was considered possible evidence of an old landslide. However, it was the conclusion of the report that the site was suitable for residential development.

The 1980 investigation of Berlogar, Long & Associates (BLA) consisted of the logging of 49 test pits on the property studied earlier by United Soil Engineering. This study also involved no physical testing of the site materials for determination of engineering characteristics. The test pit locations for this study are shown on Plate 2-REV and the test pit logs have been included in Section B of the Appendix. A primary product of the BLA report was an original geologic map of the property (scale 1″=100′). No landslides were mapped, except for a shallow instability on the over-steepened cut slope along the Ticonderoga Drive frontage of the property. This study also concluded that the site was suitable for residential development and that the landslide postulated by United Soil Engineering was not a landslide, but an area of highly-sheared Franciscan materials. Where materials of this type are encountered during grading operations for the Highland Estates project, they will be overexcavated and engineered to provide for long term stability.

The report of Berlogar, Long and Associates was subject to technical review by two geotechnical consulting firms, Earth Metrics Incorporated and Howard-Donley Associates, retained by the City of San Mateo. Both reviewers concluded that the BLA report was preliminary in scope and recommended a supplemental investigation to provide information on the following specific items:

Earth Metrics Incorporated:

- More precise delineation of the area, depth, and base profile of talus.
- Feasibility of stabilizing the talus deposits.
- More detailed description of the area, depth and base profile of colluvium.
- Description of the extent and composition of artificial fill.
- Precise mapping and examination of identified shallow debris slides and mudflows.
- Delineation of the area of potential soil creep on the geologic map.
- Evaluation of hard rock rippability by seismic refraction survey.
- Examination of the slide in the open space area.

Howard-Donley Associates:

- Rock type distribution and geologic structure, impact of proposed fills on geologic stability of the underlying formation, colluvium thickness, natural springs, and distribution of talus deposits should be evaluated for assessment of slope stability and complete slope stability analyses should be performed.
- Site grading, off-site run-off from the residential area on Cobble Hill Place, and influence of seepage from landscaped areas should be considered for surface and subsurface drainage control designs.
- Grading work should conform to the Uniform Building Code provisions.
- The San Mateo County Seismic Safety Element should be followed.
- Specific foundation recommendations should be provided and geologic cross sections should be shown.

Briefly summarized, the BLA report indicates that the consultant encountered rock at less than 10 feet, overlain by colluvium, except for one bedrock hollow that reportedly had an accumulation of more than 18 feet of talus (BLA TP-35). This test pit does not appear to be representative of conditions on the north-facing slope overlooking the Hillsborough West Apartments. Using the BLA data, San Mateo County Geologist Al Neufeld constructed an isopach map showing the estimated thickness of surface deposits overlying bedrock within the BLA study area. An updated isopach map, incorporating the borehole and test pit data provided by the SFS investigations is presented in Plate 4.

We concur with BLA that the area postulated as a possible old landslide by United Soil Engineering is an area underlain by highly-sheared melange. Two test borings, P-3 and B-4, provide subsurface data in this portion of the site. Approximately 8 to 9 feet of sheared, weak material was penetrated in the borings.

ESC Geology Comment 3, page 3

Several discrepancies are noted when comparing the boring and test pit logs with the geologic units shown on Plate 2 in the SFS report. For example:

a) *The log of Boring B-5 shows angular "talus" (Qc), whereas Plate 2 shows sandstone (Ss) mapped at that location.*

b) *Boring B-7 shows a profile of colluvium (Qc) over melange (Fm) over sandstone (Ss), whereas the map shows sandstone (Ss).*

c) *Boring B-19 shows serpentine (Sp) over graywacke sandstone (Ss), whereas the map shows melange (Fm).*

d) *Borings B-20 through B-24 show colluvium and breccia (identified as melange (Fm) on the logs) over sandstone. The descriptions of the near-surface materials are similar to those given for colluvium (Qc) elsewhere on the site. If these materials are, in fact, colluvium, the map should be revised to reflect colluvium in the area of Borings B-20 through B-24. Additional subsurface exploration may be needed in this area to better define the geologic materials, to address the origin and thickness of the "breccia", and to evaluate the nature of previous landslide movements in this area.*

e) *Test Pits T-1 and T-2 both show colluvium (Qc) to depths greater than 5 and 6 feet, whereas the map shows melange (Fm) at those locations. The colluvium contains large, angular blocks of graywacke sandstone similar to that described in the colluvium on the south side of the prominent drainage swale in the Townhome site.*

f) *Boring P-1 describes 5 feet of fill (Qaf), whereas the map shows colluvium (Qc) at that location.*

Response:

a) In Plates 1 and 2 of the 1993 report, surficial deposits were mapped where their thickness was five feet or greater. Boring B-5 is located on a bedrock nose. Subsurface data (BLA-2, -5, -6, -12, -13, -34, -37, -38, -40, -42; along with T-3, T-4, B-5 and B-7) indicate that the nose consists of extremely hard, blocky sandstone at shallow depths. In B-5 the depth to sandstone was five feet; BLA-6 penetrated 4.5 feet of soil, but was terminated at that depth. The location of the colluvium/sandstone contact has been modified so that it passes through B-5.

b) Logs from 14 subsurface points of geologic observation confirm that B-7 is within an area of extremely hard sandstone. The sheared siltstone in B-7 is interpreted to be a localized occurrence that is too small to constitute a mappable unit.

c) Boring B-19 penetrated four feet of serpentine over sandstone. Due to its limited thickness, the serpentine was not considered a mappable unit. We considered classifying the rock in this portion of the site as sandstone. Because of the association of sandstone with serpentine in B-19, the bedrock was mapped as undifferentiated melange (Fm). We agree that there is some justification for mapping this hillside area as sandstone, but have elected to classify it as melange (Fm).

d) Borings B-21 through B-24 are located in an area that has been disturbed by the activities of man. It is our conclusion that some colluvium was stripped from this area as part of the efforts to stabilize the cut slope behind the Hillsborough West Apartments. The material described as brecciated siltstone and claystone in the borehole logs is the material referred to as sheared siltstone by published mapping of the USGS. It is not colluvium.

e) The thickness of colluvium in test pit T-1 varied from two to more than six feet. Nearby borings (B-22 and B-24) penetrated less than five feet of colluvium. Based on the preponderance of evidence, the project geologist mapped this area as bedrock. The colluvium may exceed five feet in thickness locally, but these areas are considered to be too small to be mappable units.

   The hazard posed by colluvium is its inherent tendency to slump or slough when exposed on cut slopes. Several cut slopes on the project are expected to expose colluvium. Recommended measures to stabilize slopes exposing colluvium were discussed in the 1993 report (see pages 25, 36, 53).

   Test pit T-2 penetrated five feet of colluvium. Just down slope from T-2, boring B-20 penetrated five feet of colluvium overlying melange. It is our conclusion that bedrock is about five feet deep in this area. Note that T-2 is on the flank of a drainage swale that has a mapped extent of 0.5 acres (see Plate 2 of the 1993 report). The depth to rock in the swale is inferred to be greater than five feet; areas outside the swale are inferred to have soils which are typically less than five feet thick.

f) We concur with the comment: P-1 is within the fill area. The map has been revised accordingly.

ESC Geology Comment 4, page 4

[ESC's] *reconnaissance of the site could not confirm some of the outcrop patterns shown on the map. For example:*

a) *Serpentinite was observed as isolated boulders below the fill at the proposed Concord Place extension. The serpentinite shown on Lots 7 through 13 was not visible at the ground surface.*

b) *The geomorphic features on Lots 1 through 4 at the south side of Bunker Hill Drive suggest that the colluvium (Qc) and melange (Fm) mapped at that location may be landslide deposits and/or a denuded landslide scarp. The area that includes the four lots is mapped as possible landslide deposits by San Mateo County (1975) and as "unstable" by Nilsen et al. (1979), as shown on Figure 3 in the SFS report. Further subsurface exploration is recommended in this area in order to further characterize the materials and evaluate their stability. This area may include landslide deposits similar to those mapped adjacent to Lot 4.*

c) *The arcuate feature mapped below Borings B-22 and B-24 has a hummocky surface suggestive of active landsliding and/or deep soil creep. In light of the previous slope instabilities in this area, further subsurface exploration is recommended to characterize the materials and evaluate their stability with respect to the proposed grading in that area. Are the "erosion" features uphill from Boring B-20 similar to those shown below Borings B-22 and B-24? If so, what is the possible impact on the proposed grading for the Townhome development?*

d) *The cut slope on the north side of Ticonderoga Drive exposes interbedded sandstone and siltstone at the location mapped as melange (Fm).*

Response:

a) The isolated serpentine boulders noted during ESC's reconnaissance are interpreted as rock that was too large to be crushed by the earthmoving equipment used to grade the existing Highlands residential development west of the site. Without crushing, the boulders were too large to be used in fills within that 1950s' project. They were disposed of by placing them in the adjacent parcel (Highland Estates site).

According to Plates 1 and 2, Lots 7 through 13 are chiefly in the outcrop belt of serpentine (Sp). Our interpretation of bedrock geology is based on subsurface data from this portion of the site, not outcrop data. Terrain features and vegetation were used to map contacts of the serpentine. The contacts shown are approximate.

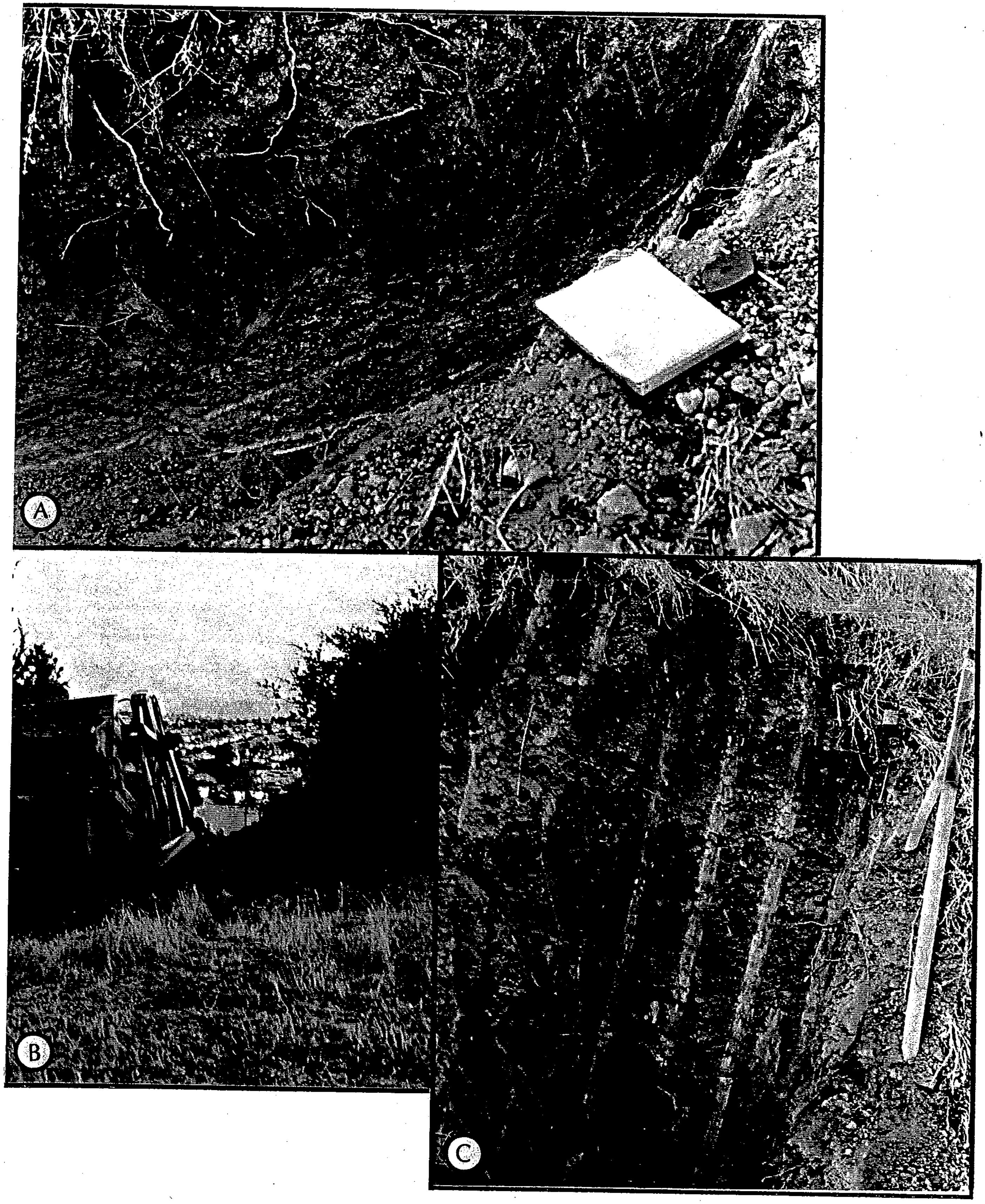
b) Lots 1 through 4 are on the flank of a ridge underlain by competent graywacke sandstone bedrock. Bunker Hill Drive is aligned along the axis of the ridge. This ridge was graded to create nearly level building pads on each flank of the road, and the entire neighborhood was developed in the late 1950s. The area encompassing Lots 1 through 4 was not developed then because it was not practical to create level pad lots having sufficient depth to accommodate the Eichler house plans. Based on interpretation of geomorphic features, reconnaissance mapping, and limited subsurface data (Boring P-6), this area was considered to be underlain by graywacke sandstone at a shallow depth. There is a well-defined drainage swale on Lots 2 and 3. It is overgrown with native vegetation and appears stable; however, Plates 1 and 2 show colluvium (Qc) in the axis of the swale. It is our conclusion that the depth to bedrock is greater than five feet in this area.

In response to ESC's concerns regarding stability in this area, subsurface work was performed in this portion of the site as a part of the supplemental investigation. Test Pit TP-7 was excavated on proposed Lot 1, and a Minuteman boring (BS-2) was situated in the axis of the swale, near the south corner of proposed Lot 3. The test pit, which was excavated on an existing bench, penetrated graywacke sandstone that was so massive that the test pit was only excavated to a depth of 2.5 feet deep. A graded, six-foot high slope above the bench was exposed in the back wall of the test pit. It exposed six feet of stiff fill. The bedrock on the floor of TP-7 was more difficult to rip than materials logged elsewhere on the site. Plate C in Section A of the Appendix presents the log of TP-7 and Figure A, page 12, presents views of field conditions. View A shows the contact of the slightly weathered (gray) sandstone on the lower part of the trench wall. Overlying the gray sandstone is a moderately weathered (yellowish brown) sandstone. View B shows the backhoe exiting the site. In the foreground is the upper graded pad. The backhoe used an existing haul road to access the lower bench. View C shows the stiff, rocky fill exposed in the back wall of the trench. The lath to the right of the trench is three feet long.

The log of BS-2 indicates two feet of colluvium overlying moderately weathered (light brown) siltstone and sandstone. The boring was 11 feet deep, and the material became very hard below nine feet.

In summary, the subsurface data gathered during the supplemental investigation provides strong evidence that proposed Lots 1 through 3 are underlain by competent bedrock. No slide debris is present on these parcels.

c) The hummocky surface appearance of the area below Borings B-22 and B-24 represents man-made features, including slope benches with runoff interceptor ditches, graded roads, and excavations performed to unload the slope immediately above the cut slope constructed during the 1960s. Underdrains were installed in this area during the episode



VIEWS OF TP-7 VICINITY	FIGURE: A	SOIL FOUNDATION SYSTEMS INC.
	DATE: 11-94	JOB NUMBER: S24-634-2S

of the cut slope instability. In response to the ESC comment, Boring BS-1 was drilled as part of the supplemental investigation. This boring, drilled to a depth of 21 feet, was strategically located to correlate with previous borings in the area. When combined with the data from Boring B-23, it provided data to construct a profile of the subsurface strata in the area disturbed by grading when the apartments were constructed. The profile provided confirming information on the instability that occurred along the contact between the melange unit and the underlying sandstone unit. Specifically, the profile indicates that the contact between the two units was exposed in the cut slope, near the toe. At the time of the investigation no physical evidence of subsurface seepage or free water was found in this area. The old underdrain found in Boring B-24 was dry and a piezometer tube installed in Boring B-22 did not collect any free water during the 1993-1994 winter. The material encountered in Boring BS-1 was so dry that water was required to facilitate removal of the cuttings. A geologic cross section through this area, which shows the profile discussed above, is presented in Plate D in Section A of the Appendix. Further discussion relating to stabilization of this area will be presented under "Slope Stability" in the Geotechnical Engineering Section.

The erosion features uphill from Boring B-20 are different from the terrain features discussed above. This area is the upper elevations of a topographic swale and is covered by shallow colluvium. Cut slopes are proposed over this portion of the swale area. The 1993 SFS report recommends removal of the colluvium to mitigate the potential for rockfalls and other instability of the colluvium. Under this approach, the possibility of ravelling of the colluvium immediately above the cut slope is anticipated. Alternatives for mitigating this hazard are discussed under "Slope Stability" in the Geotechnical Engineering Section.

d) The ridge area northwest of Ticonderoga Drive contains large blocks that are chiefly massive graywacke sandstone (see Plates 1 and 2). The sheared rock along Ticonderoga Drive is interpreted as matrix material containing isolated tectonic lenses of graywacke sandstone. Near the upper elevations of Pulgas Ridge, the matrix material is intruded by serpentinite. The USGS maps show the site as sheared rock. Our mapping differentiates between (a) matrix material (i.e., sheared rock), (b) the large blocks of competent sandstone and (c) serpentinite. Published maps were prepared to provide an overview of geologic conditions that generally prevail in San Mateo County. The site geologic map provides the details that are necessary for slope stability analyses.

According to Wentworth et al. (1985), the sheared rocks of the Franciscan Assemblage range from soft to hard, fracture spacing from very close to close, permeability very low to low, and much of the rock and overlying soils are expansive. For our engineering analysis, strength characteristics of the Franciscan unit were based on laboratory analysis of representative samples collected from on-site borings. Because these samples were moderately to slightly weathered, the measured strength characteristics are inferred to be lower than that of fresh rock. We infer that the use of the strength characteristics of weathered rock in the slope stability analyses has tended to yield calculated factors of safety on the conservative side. Based on the results of laboratory tests, it is our conclusion that the soil and soft weathered rock on the site are low in expansion potential.

ESC Geology Comment 5, page 4

The description in the SFS report (page 16) of "Tectonic Lenses" states that the sandstone material beneath the melange matrix in B-22 is interpreted as a "...sheet-like body of sandstone within the melange matrix." If so, does the sandstone surface shown on cross sections GS-1 and GS-2 represent a bedding plane parallel to the ground surface? What are the implications to the overall slope stability of the colluvium and/or melange unit overlying a "sheet-like body of sandstone"? (Compare with west-dipping sandstone (Ss) at GS-6, page 18.)

Response

Boreholes in this portion of the site terminate in competent sandstone. The cross sections correlate sandstone in the lower part of one boring with sandstone in adjacent borings. The inferred continuity of the sandstone is consistent with the subsurface data. The contact of the sandstone with the overlying sheared siltstone melange could be bedding-related, but is more likely a sheared tectonic contact.

The contact between sheared siltstone and sandstone is a potential failure plane when it "daylights" in a cut slope. To mitigate this potential hazard, we have recommended that the proposed reinforced earth slopes and retaining walls be supported on the sandstone (see Figure 8-A, page 41, of the 1993 SFS report). The assessment of overall slope stability in the townhome area considered the potential instability of the colluvium "daylighting" in cut slopes. This concern was discussed in response to the preceding comment.

It should also be recognized that the engineering geologist for the project will examine and map rock exposed in cut slopes and keyways throughout the construction period, as needed. Wherever contacts are characterized by weak rock and/or seepage, measures will be implemented during construction to eliminate these adverse conditions.

ESC Geology Comment 6, page 4

The discussions of geologic cross Sections GS-2 and GS-3 in the SFS report are not consistent with the materials illustrated on the cross sections of those shown on the Geologic Map, Plate 2.

Response:

The interpretation of a vertical dike of serpentine occurring in the western end of GS-2 was not shown on the cross section. GS-2 has been revised to show this contact line at the toe of the artificial fill (Qaf). Section GS-3 is correct, but the colluvium unit (Qc) shown in the lower portion was erroneously described as alluvium in the discussion. However, these discrepancies do not materially alter the assessment of slope stability in the area.

ESC Geology Comment 7, page 5

Section GS-5 shows a cut slope within the "talus"/colluvium that mantles the slope. What is the stability of the proposed cut slope above the retaining wall at this location? What plans are proposed to stabilize the slope if the "talus" is 10 feet thick, as reported by Berlogar?

Response:

The test pits of Berlogar, Long and Associates nearest GS-5 are BLA-35 and BLA-36. The depth to bedrock was approximately 18.5 and 6.5 feet, respectively. It is our determination that the major portion of the slope is underlain by less than 6 feet of colluvium, with the exception of a small area near BLA TP-35, as presented in Plate 4 (Isopach Map of Surficial Deposits). The log of TP-35 is interpreted as evidence that there are local bedrock hollows filled with colluvial deposits. Any bedrock hollows filled with colluvium will be overexcavated during grading operations. Much of the colluvium will be eliminated as a part of the cut slope construction.

ESC Geology Comment 8, page 5

The SFS report states on page 31 that "...no hydrophilic vegetation or other indications of shallow groundwater were found in the proposed single-family lot areas (Lots 15 through 18) or the planned Townhome areas "on this portion of the site." A growth of willows was observed during our reconnaissance in the area behind the Hillsborough West Apartments, near the property line below B-8. The source of water for the willows should be determined. It is assumed that the symbols on Plate 2 for "seepage, deep soil erosion" on the slope above and below B-20 and B-21 are intended to indicate evidence of shallow groundwater.

Response:

An on-site drainage swale (area of proposed Lots 13 and 14) carries year-round flow, emanating from a culvert placed by the developer of the neighborhood upslope. The water carried by this culvert may be a combination of natural springs and irrigation water (due to overwatering of ornamental vegetation). During the dry summer months, runoff exiting the culvert disappears on the floor of the swale. The willows noted by the ESC comment are downslope from boring B-8, and centered in the existing drainage channel. SFS logged borings to bedrock on the alluvial terrace just northeast of the channel (B-9 through B-13), along with a hillside boring just south of the channel (boring B-8). These borings did not encounter groundwater. Surface water was encountered in the upper elevations of the drainage swale near the existing terminus of Concord Place. The probable sources of this water are subdrains and storm drains from the existing development on top of Pulgas Ridge. Seepage from the pampas grass area (i.e., area of proposed Lots 9 and 10) feeds the drainage swale and, consequently, is another source of water.

The symbols on Plate 2 for "seepage, deep soil erosion" indicate a potential for seasonal seepage in the swale areas above and below Borings B-20 and B-21. No evidence of groundwater was found in any of the borings located in this portion of the site, nor was active seepage observed on the ground surface.

ESC Geology Comment 9, page 5

The report is lacking subsurface data between Borings B-8 and B-2 to properly assess slope stability and foundation conditions on the slope south of the Hillsborough West Apartment site.

HTA Comment 4, page 2

We question the practicality of constructing a 25-foot-high retaining wall (Section GS-10) above a 1.5:1 (horizontal to vertical) slope on the adjacent property with essentially no subsurface information in an area where undocumented grading was previously performed.

Response

The supplemental investigation included the logging of two test pits on this slope (TP-5 and TP-6). Both test pits were located on the outboard edge of an existing rough-graded road. This road is located at the top of the 1.5:1 slope which was constructed during the 1960s by cutting the natural slope. The logs of these test pits are presented in Plate C. TP-5 was located in the area of a proposed townhome building. The test pit penetrated 2.5 feet of fill and 1.5 feet of colluvium overlying Franciscan Assemblage sedimentary rock. As the log indicates, the rock consisted of sheared siltstone/shale with graywacke inclusions up to one foot in maximum dimension. The upper 2.5 feet of sheared rock was severely weathered (3a); rock below that depth was moderately weathered (3b). The basal two feet of the test pit exposed slightly weathered graywacke sandstone (5c). Dominant partings in the bedrock possessed dips of 50 to 60 degrees.

TP-6 penetrated up to seven feet of fill and colluvium overlying bedrock. The rock consisted of sheared shale underlain by graywacke sandstone. The contact on these units dipped to the south (into the hill) at 45 degrees. If the fill is excluded, the depth to bedrock is five feet or less at the site of test pits TP-5 and TP-6. Therefore, the proposed reinforced earth walls in this area will be founded on sandstone.

This slope was mapped as colluvium (Qc) in Plate 2 of the 1993 report, with sandstone (Ss) shown outcropping on the ridge crest. The recent test pits indicate that the thickness of colluvium at the areas explored is less than 5 feet, with competent, slightly to moderately weathered sandstone present at a relatively shallow depth. The sandstone in TP-5 and TP-6 is now interpreted as a contiguous unit to the sandstone mapped upslope. The 1993 SFS analyses assumed the rock in this slope to be melange (Fm), which is weaker than the sandstone.

Section GS-10 has been re-analyzed for the profile described above. Sandstone encountered in the supplemental exploration is competent rock, capable of supporting the proposed retaining wall construction. Results of current analyses indicate that the slope will have an adequate factor of safety to support the retaining wall. The pertinent slope stability computations are included in Section C.3 of the Appendix (see pages C.3.48 - C.3.59)

ESC Geology Comment 10, page 5

The SFS report states that they reviewed air photos taken in 1955, 1961, 1983 and 1985. Additional air photos should be reviewed, particularly those taken between 1961 and 1983. This 22 year gap is significant, as it includes the time during which adjacent projects were developed, etc. The report should also discuss those features that are of significance on each photo pair.

Response:

Since the 1993 SFS report was issued, geologic interpretation of 1941, 1956, 1965 and 1970 aerial photographs has been performed. Detailed citations of the photographs are presented in the bibliography. Some of the features seen on the photographs have been previously described (see Response to ESC Background Comment B). Significant features seen on the 1965 photographs include the following:

- The Hillsborough West Apartments had been constructed. There were two small areas on the slope south of the apartments and one behind the apartments that had a freshly graded appearance. It is believed that the grading in these areas was done as a result of previous slope instabilities.

- The floor of the central drainage swale (i.e., Townhouse site) had been cleared of vegetation. The disturbed area was nearly flat, suggestive of minor grading.

- There was no pampas grass on the slope overlooking the central drainage swale (i.e., area of proposed Lots 8 and 9).

- No active sliding was observed within the areas currently planned for development, except for the shallow failures that had occurred on the cut slope along the perimeter of the Hillsborough West Apartments property.

- Two shallow slides were observed on the slope below the water reservoir. These were shallow debris flow slides, and do not affect any of the proposed lots.

ESC Seismicity Comment 11, page 5

The seismic history of the area is not well discussed in the SFS report, and the potential ground shaking characteristics may be understated.

a) *A regional map showing the location of the site in relation to known active faults and previous earthquake epicenters would be useful in this regard.*

b) *A discussion of historical earthquakes and their effects on this or similar nearby sites should be included in the report. Figure 4 in the SFS report indicates that the site area would experience ground shaking intensities of Modified Mercalli 6.5 (sic) for a 1906-type event (M 8.3) on the San Andreas fault. Other references suggest higher ground shaking intensities, and they should be presented for comparison. For example, Nason (1980) reports a Modified Mercalli intensity VIII during the 1906 earthquake for the San Mateo Hills at a location 3 km east of the San Andreas fault. Borcherdt and others (1975) predict that the site area will experience "very strong" ground shaking in a 1906-type earthquake. This corresponds to Grade C on the San Francisco intensity scale, which is comparable to intensity 8+ on the Rossi-Forel scale, and intensity VIII on the Modified Mercalli scale.*

c) *The SFS report cites a peak ground acceleration of 0.5g for a magnitude 7 earthquake on the San Andreas fault, based on attenuation curves by Seed and Idriss (1982). Data from the Loma Prieta Earthquake of 1989 (M 7.1) indicate a peak acceleration of 0.64g at Corralitos, approximately 6.5 miles from the earthquake epicenter (CSMIP, 1989). The report should also discuss predicted peak accelerations for a magnitude 8 to 8.3 event on the San Andreas fault. Several other references are available for estimating peak horizontal accelerations, both for near-field and distant earthquakes. (See 1992 CDMG Open File Report 91-1, "Peak Acceleration from Maximum Credible Earthquakes in California", among others.) The implications of higher peak accelerations should be discussed by the consultant.*

Response:

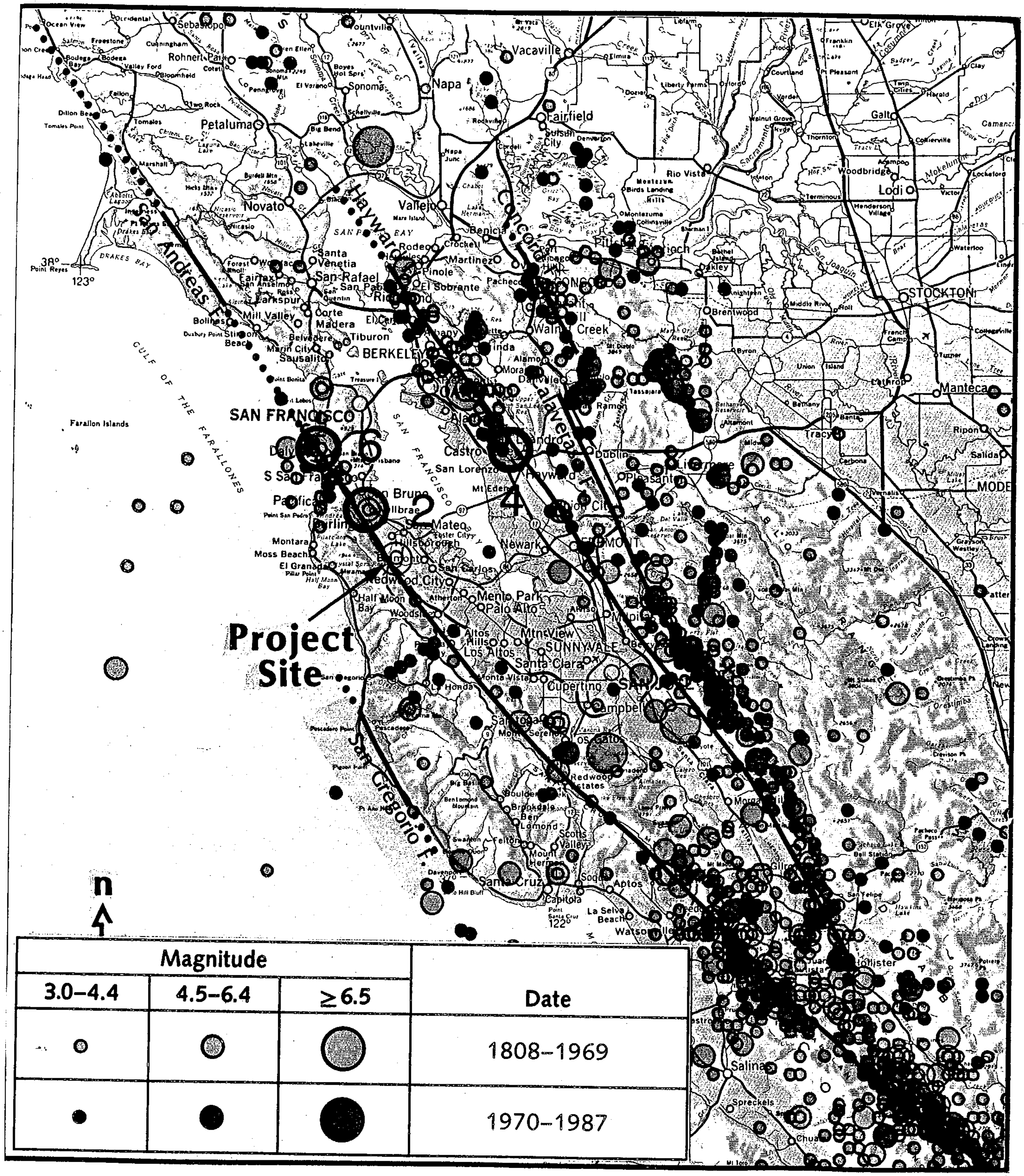
a) Enclosed with this response document is a regional seismicity map which shows the location of the site in relationship to earthquake epicenters and faults (see Figure B, page 20).

b) Notable San Francisco Bay Region earthquakes include the following:

Magnitude	Date	Comments
7.0	June 1838	San Andreas fault. Epicenter inferred to be on the peninsula.
6.8	October 1868	Hayward fault. Surface fault rupture from Warm Springs District of Fremont to Mills College in Oakland
8.3	April 1906	San Francisco. Surface fault rupture occurred on the peninsula; based on amount of fault offset, epicenter generally considered to be in Marin or possibly San Francisco Counties.
7.1	October, 1989	Epicenter in the Loma Prieta area of the Santa Cruz Mountains, just south of the Peninsula segment of the San Andreas fault.

The discussion presented by ESC indicates that several publications have prepared isoseismal maps of ground shaking in San Mateo County associated with the 1906 San Francisco Earthquake. These reports use different intensity scales and differ somewhat in their interpretation. Certainly the project site, along with all properties within one mile of the San Andreas fault, will be subjected to strong ground shaking in the event of a high magnitude earthquake originating on the peninsula segment of this fault. The affected area includes all the development on Pulgas Ridge (i.e., all neighboring subdivisions). Because of improvements in building and grading codes over the years, the susceptibility to damage will be a function of several variables, including type and age of construction, distance from the source of the earthquake, and local site conditions. It would not be surprising if improvements on the site perform better than some older structures that are farther from the San Andreas fault.

One of the most comprehensive assessments of the Bay Area's vulnerability to earthquake shaking is a 1983 report issued by the Association of Bay Area Governments (ABAG), in cooperation with the USGS. According to a USGS intensity map in this



source: modified after Goter(1988)

Scale: 1" = 16mi.

SEISMICITY OF CALIFORNIA, 1808-1987	FIGURE B	SOIL FOUNDATION SYSTEMS INC.
	DATE: 11-94	JOB NUMBER S24-634-2S

report, the Pulgas Hill area, including this site, is in an area subject to "strong" earthquake shaking (San Francisco Intensity D). This correlates with a Modified Mercalli intensity of VII. The damage cost factor for Class 1A buildings (wood-frame dwellings) is 5% at San Francisco Intensity D. Therefore, a residence with a replacement cost of $300,000 could expect to sustain an average cost of $15,000 in earthquake damage. In the event of ground failure or fire, the damage could be more severe. If maximum ground shaking intensities on the site are Modified Mercalli VIII as is suggested by some of the references cited by ESC, the damage cost factor would be 7% for wood-frame dwellings, and a structure valued at $300,000 could expect to sustain, on average, $21,000 in damage.

According to ABAG, the source of the damage is as follows:

Modified Mercalli Intensity	Damage Cost Factor For Wood-Frame Dwellings			
	Finish Component	Structural Component	Chimneys	Total
VII	0.03	4.44	0.07	4.54
VIII	0.60	6.28	0.18	7.06

c) The value of earthquake magnitude used in the 1993 SFS report is for the peninsula segment of the San Andreas fault, taken from a recent USGS publication (Circular 1053, issued in 1990). This publication was generated by a working group of twelve scientists and represents a consensus on this subject. The ESC comment compares a peak ground acceleration of 0.5g for the subject site, as estimated based on the Seed-Idriss attenuation curve for a magnitude 7 earthquake, to a peak ground acceleration of 0.64g that was actually measured at Corralitos during the 1989 Loma Prieta earthquake.

There are a number of publications which provide estimates of the peak bedrock acceleration and attenuation relationships (Joyner and Boore, 1988; Mauchlin and Jones, 1992; Sadigh et al., 1993). Applying these relationships to the subject site for a 7.5 magnitude earthquake on the Peninsula segment of the San Andreas fault typically yields bedrock accelerations in the range of 0.6g to 0.7g, and some forecast accelerations as high as 0.9g. A 1992 CDMG document shows bedrock accelerations of 0.6g in the vicinity of the site.

From the standpoint of geotechnical engineering applications, it is important to distinguish the difference between the peak value from any such empirical attenuation relationship and actual measured values. Attenuation relationships are established through statistical analysis and thus the resulting peak ground acceleration estimates involve certain statistical median values. However, a measured value is absolute. There is a consensus of opinion within the profession that such a measured peak value of ground acceleration is not appropriate for use in analyses because the time duration of the motion is too short for the ground motion to fully respond. In response to the comment, the anticipated peak ground accelerations at the site for higher magnitude earthquakes have been determined, and will be discussed in detail under "Seismic Coefficient" in the Geotechnical Engineering Section.

ESC <u>Seismicity</u> Comment 12, page 6

The SFS report states on page 24 that "...the most critical seismic factor is the effect of the ground acceleration on stability of the proposed slopes." Their evaluation should also include the seismic stability of existing slopes as well as proposed slopes. In a USGS publication titled "Map Showing Slope Stability During Earthquakes in San Mateo County, California," by Wieczorek and others (1985), the eastern slopes of Pulgas Ridge are shown to be in their highest category of susceptibility to landsliding during a major earthquake. It is recommended that the consultant review this publication and evaluate the applicability of their analysis and methodology to this site.

Response:

The existing natural slopes were included in slope stability analyses presented in the 1993 report. The slope cross sections designated as Subsections "a" (e.g., Section GS-1a) in the computer runs include the natural slope and the contiguous cut slopes. The computed factors of safety for these slopes are adequate. Construction of the proposed fills on the lower elevation of these slopes and reconstruction of cut slopes exposing weak materials, as recommended in the geotechnical report, will further improve stability of these slopes.

The map referred to by the comment is presented in Figure C, page 23. According to the Seismic Slope Stability Map, San Mateo County can be divided into four slope stability categories, as follows:

Susceptibility Zone	% of Area Likely to Fail in a Major Earthquake
High (red)	<25%
Moderate (orange)	~15%
Low (yellow)	~ 5%
Very Low (green)	< 3%

EXPLANATION

Susceptibility and percentage of area likely to fail in a major earthquake

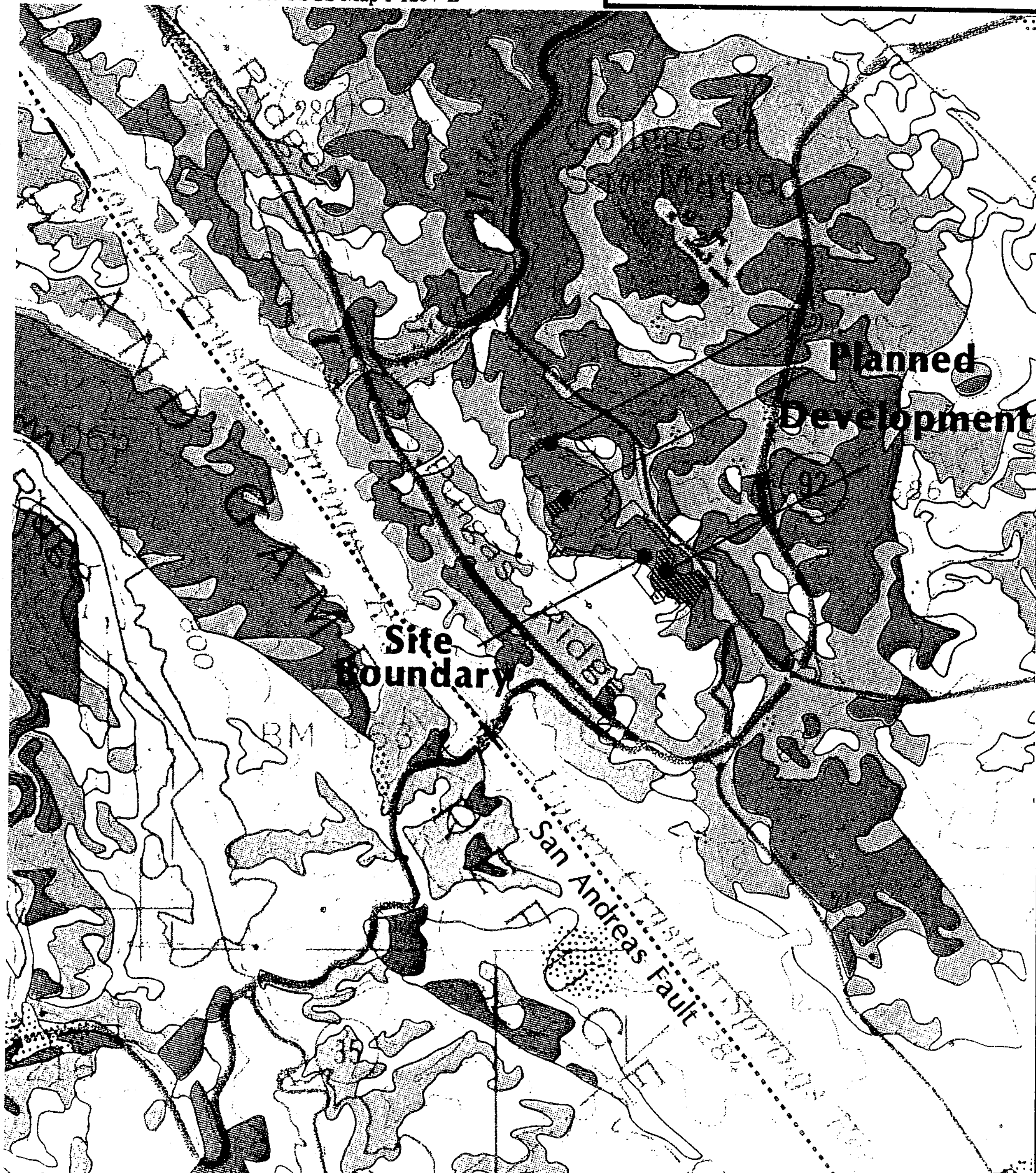
- *High* .... less than or equal to 25%
- *Moderate* .... approximately 15%
- *Low* .... approximately 5%
- *Very low* .... less than 3%

AREA OF LOW, MODERATE, OR HIGH LIQUEFACTION SUSCEPTIBILITY—Data from T. L. Youd and J. B. Perkins (1985)

FAULT ZONE—Approximately located. Data modified from Brabb and Pampeyan (1983). Specific location generalized, therefore should not be used for evaluation of fault-rupture hazard

Source: USGS Map I-1257-E

USGS SLOPE STABILITY DURING EARTHQUAKES		
Scale: 1" = 3000'	FIGURE: C	n
DATE: 11-94		
SOIL FOUNDATION SYSTEMS INC.		
JOB NUMBER: S24-634-2S		



The intent of the USGS map is to "red flag" areas where potential stability problems may exist, so that site specific investigations can be performed. The methodology of the USGS (Wieczorek et al., 1985) assessed seismic stability using limited data from existing maps, reports, and reconnaissance from selected sites in San Mateo County. There is no documentation that any work was performed on Pulgas Ridge or the Highland Estates site by the authors of the USGS map. The study concludes that approximately 30% of the County, including Pulgas Ridge, is in the "highest" susceptibility category.

The referenced USGS method is based on a concept of "critical acceleration", which is defined as follows:

$$a_c = (FS - 1)\ g \sin\theta \qquad [1]$$

where, FS is the computed static factor of safety, g is gravitational acceleration, and θ is the slope angle.

This method would merit application to a specific site only if the static factor of safety and slope angle are representative of the site. One must recognize that the strength parameters and slope angles assigned to each of the subregions included in the USGS study were used to establish a map of regional relative landslide susceptibility during earthquakes, not to assess the stability of a specific site within that subregion. As discussed in the 1993 SFS report, there are areas outside the project area on Pulgas Ridge that are unstable or potentially unstable.

The USGS study provides a context for the site-specific investigation performed by SFS. The intended general usage of the map is not disputed in this response. However, as stated by the authors of the USGS map, it is not appropriate for determining the susceptibility to earthquake-induced landsliding at any specific site due to various limitations.

One test of the validity of the USGS seismic slope stability map is the performance of slopes during historic earthquakes. Although the 1989 Loma Prieta earthquake did not fully test the resistance of hillside areas in San Mateo County, slopes on Pulgas Ridge performed satisfactorily during this earthquake. The effects of the 1906 earthquake, including slides, are analyzed in a USGS publication (Youd and Hoose, 1978) indicating no direct evidence of earthquake-triggered landslides on the flanks of Pulgas Ridge.

The USGS issued a report that documents landslides triggered by the unusually wet January 3-5, 1982 rainstorm (Ellen and Wieczorek, 1988). Rainfall on Pulgas Ridge was approximately 125 mm (4.9 inches). No landslides occurred on the flanks of Pulgas Ridge as a result of this storm, which triggered an estimated 18,000 slides in the San Francisco Bay Region.

ESC Seismicity Comment 13, page 6

Seismic stability analysis reportedly used the El Centro earthquake ground motion records for input. Wieczorek and others suggest that the El Centro earthquake records represent a "lower-bound" event when evaluating landslide movement. They show the Parkfield earthquake to be nearer the "upper-bound" earthquake. For this site, an "upper-bound" earthquake may be more appropriate. Please address.

Response:

The understanding that our seismic stability analyses used the El Centro earthquake ground motion records is incorrect. Several bases were considered for selecting the design seismic coefficient of 0.15 used in the slope stability analyses, as will be detailed in the subsection entitled "Seismic Coefficient", located in the section on Geotechnical Engineering. The Seed-Martin shear-beam solution was one of these bases. The Seed-Martin solution determines the average seismic coefficient of a slope at a site, and it requires the natural period of the slope and strong ground motion records applicable to the site. Seed and Martin developed a graph showing the numerical relationship between the average coefficient and natural period as determined from the shear-beam solution using the El Centro records. According to this graph, the average coefficient for a slope having a natural period of 1.2 seconds (same as Pulgas Ridge) is 0.11. We have linearly scaled this value in proportion to the ratio of peak accelerations to obtain the average coefficient for the subject site. The peak accelerations used for this scaling were 0.35g from the El Centro records and 0.5g for the subject site. Further discussions are presented on pages 47 through 50 of this report regarding the selection of the peak ground accelerations and seismic coefficient for the subject site.

The definition of a "lower bound" or "upper bound" earthquake, as used by Wieczorek et al. (1985), is not directly relevant to establishing seismic coefficients in the manner described above. However, the scaling applied to the above results based on the El Centro records (defined as a lower bound earthquake) may be considered as a conversion to an "upper bound earthquake". Earthquake bounds as used by Wieczorek et al. are defined in terms of slope displacements and critical accelerations that are determined by their method. Therefore, the significance of the earthquake bounds, when concerned with a specific site, should be assessed based on the critical acceleration applicable to the site. A critical acceleration of approximately 0.6g was calculated for the subject site, as is more fully discussed on page 50 of this report. When this value is used in Figure 3 of Wieczorek et al., the anticipated slope displacement is nil under any earthquake within the designated design earthquake bounds.

ESC <u>General Comments and Recommendations</u>, page 8

Comment 1

It is our opinion that the SFS report has not adequately defined the distribution and engineering characteristics of the geologic materials at this site. Additional field and office studies are recommended to address the items discussed in this review.

Response:

The maps and sections have been amended by this supplemental report to reflect the results of the December 1993 subsurface investigation and the editorial comments of ESC.

Comment 2

Some of the references listed in the SFS bibliography were not discussed in the report. The report should also include a discussion of additional references pertinent to the site, including the San Mateo County Seismic Safety Element.

Response:

This report contains discussion of additional pertinent references, including an updated bibliography.

With regard to the San Mateo County Seismic and Safety Element, information in the technical supplement to the element includes the following:

a) The anticipated Modified Mercalli intensities in the Pulgas Ridge area are MM VIII for an M7 earthquake on the San Andreas fault; MM IX for an M8.3 earthquake on the San Andreas fault.

b) At MM VIII, the damage cost factor for one and two story dwellings is 5.7%. At MM IX, the damage cost factor is 15%.

c) At MM VIII, the estimated peak ground acceleration is 0.12g. At MM IX, the estimated peak ground acceleration is 0.25g.

d) Forecasts are made for the mean return prediction (in years) as a function of peak ground accelerations. These forecasts, which are for incorporated cities, are relatively constant countywide. They indicate that a .55g peak acceleration has a return period of approximately 325 years; that .25g has a return period of approximately 87 years.

Comment 3

SFS should review and comment on previous reports. The new SFS report should address the constraints identified in the previous reports, and appropriate recommendations should be made.

Response:

Subsurface data of previous investigations, in combination with the SFS subsurface investigation, were utilized to interpret site geologic conditions, prepare original geologic maps and cross sections, and prepare an isopach map showing the thickness of surficial deposits on site. The SFS interpretation of geology on the site is consistent with all available outcrop and subsurface data.

Geotechnical recommendations are provided which address the specific approach to development proposed by the Chamberlain Group. Special design features have been incorporated into the project to ensure that safety factors for graded slopes meet San Mateo County's requirements.

Comment 4

The geologic map and cross sections should be checked for consistency, and modified as needed as additional subsurface data is developed.

Response:

Based on the December 1993 field work and editorial comments of ESC, the map and sections have been modified.

Comment 5

Revisions and/or additions to the Geotechnical Investigation Report should be submitted to the County for review.

Response:

Concur with the comment.

References Cited for Geology and Seismicity

Association of Bay Area Governments, 1983. "Using Earthquake Intensity and Related Damage to Estimate Earthquake Intensity and Cumulative Damage Potential from Ground Shaking." Earthquake Mapping Project, Working Paper #17.

Berlogar. Long and Associates, 1980. "Report, Geotechnical Investigation, 11.9 Acre Parcel, Polhemus Road and Ticonderoga Drive, San Mateo County for Ponderosa Homes." (BLA Job #805-10).

Brabb, E. E., and E. H. Pampeyan, 1972. "Preliminary Geologic Map of San Mateo County, California." U.S. Geological Survey, Basic Data Contribution 41.

Brabb, E. E., and E. H. Pampeyan, 1972. "Preliminary Geologic Map of San Mateo County, California." U.S. Geological Survey, Basic Data Contribution 42.

Brabb, E. E., and E. H. Pampeyan, 1972. "Preliminary Geologic Map of San Mateo County, California." U.S. Geological Survey, Basic Data Contribution 43.

Brabb, E. E., and E. H. Pampeyan, 1983. "Geologic Map of San Mateo County, California." U.S. Geological Survey, Map I-1257-A.

Brabb, E. E., E. H. Pampeyan, and M. G. Bonilla, 1972. "Landslide Susceptibility in San Mateo County, California." U.S. Geological Survey, Miscellaneous Field Studies Map MF-0.

California Division of Mines and Geology, 1975. "Recommended Guidelines for Determining the Maximum Credible and the Maximum Probable Earthquakes." CDMG Note No. 43.

California Division of Mines and Geology, 1974. "State of California Special Studies Zones Map, San Mateo 7-1/2-minute quadrangle."

Campbell, K. W., 1987. "Predicting Strong Ground Motion in Utah." in Gori, P. L. and Hays, W. W., eds., Assessment of Regional Earthquake Hazards and Risk Along the Wasatch Front, Utah, U.S. Geological Survey Open-File Report 87-585, Vol. II, Sect. L, 1-50.

Contra Costa County General Plan: 1990-2005.

Earth Metrics, Inc., 1982. "Ponderosa Homes Project." PA 80-86.

Ellen, S.D., and G.F. Wieczorek, Editors, 1988. "Landslides, Floods and Marine Effects of the Storm of January 3-5, 1982, in the San Francisco Bay Region, California." U.S. Geological Survey, Professional Paper 1434.

Goter, Susan, 1988. "Seismicity of California, 1808-1987." U.S. Geological Survey, Open File.

Howard Donley and Associates, 1983. "Geotechnical Report Review - Berlogar, Long Associates Report."

Joyner, W. B. & D. M. Boore, 1988. "Measurement, Characterization, and Prediction of Strong Ground Motion." *Earthquake Engineering and Soil Dynamics II -- Recent Advances in Ground-Motion Evaluation*, ASCE Geotechnical Special Publication No. 20, pp. 43-102.

Mark, R. K., E. B. Newman, and E. E. Brabb, 1988. "Slope Map of San Mateo County, California." U.S. Geologic Survey, Map I-1257-J.

Mauchlin, L. and A. L. Jones, 1992. "Peak Acceleration From Maximum Credible Earthquakes in California (Rock and Stiff Soil Sites)." California Department of Mines and Geology, Open File Report 92-01.

Montgomery, D.R. and R.H. Wright, T. Booth, 1991. "Debris Flow Hazard Mitigation for Colluvium-Filled Swales." AEG Bulletin, Vol. XXVIII, No. 3, pp. 303-323.

Perkins, J. B., 1987. "Maps Showing Cumulative Damage Potential from Earthquake Ground Shaking, San Mateo County, California." U.S. Geologic Survey, Map I-1257-I.

Sadigh, K., C.-Y Chang, N.A. Abrahamson, S.J. Chiou, and M.S. Power, 1993. "Specification of Long-Period Ground Motions: Updated Attenuation Relationships for Rock Site Conditions and Adjustment Factors for Near-Fault Effects." *Proceedings, Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, Applied Technology Council, Palo Alto, California, Report No. ATC 17-1, 59-70.

San Mateo County, 1976. Seismic Safety Elements of the General Plan.

Seed, H. B., and I. M. Idriss, 1982. "Ground Motion and Soil Liquefaction during Earthquakes." Earthquake Engineering Research Institute, Monograph.

United Soil Engineering, 1977. "Geologic Investigation for Part of 11.9 Acres, Northwest of Polhemus Road & Ticonderoga Drive, San Mateo County, California."

USGS, 1990. "Probabilities of Large Earthquakes in the San Francisco Bay Region, California." US Geologic Survey Circular 1053.

Wentworth, C. M., S. Ellen, V. A. Frizzell, and J. Schlocker, 1985. "Map of Hillside Materials and Description of Their Engineering Character, San Mateo County, California." U.S. Geologic Survey, Map I-1257-D.

Wesnousky, S. G., 1986. "Earthquakes, Quaternary Faults and Seismic Hazard in California." *Journal of Geophysical Research,* Vol. 91, No. B12, pp. 12587 - 12631.

Wesson, R. L., et al., 1975. "Faults and Future Earthquakes." *Studies for Seismic Zonation of the San Francisco Bay Region,* U.S. Geological Survey Professional Paper 941-A, pp. 5-30.

Wieczorek, G.F., R.C. Wilson, and E.L. Harp, 1985. "Map Showing Slope Stability During Earthquakes in San Mateo County, California." U.S. Geological Survey Map I-1257-E.

Youd, T.L. and S.N. Hoose, 1978. "Historic Ground Failures in Northern California Triggered by Earthquakes." U.S. Geological Survey Professional Paper 993.

Aerial Photographs

Type	Date	Scale	Flight	Source
Black & White	6-23-70	1:24,000	2946-8-21	San Mateo Cnty.
Black & White	5-11-65	1:12,000	SM 1-209,-210	San Mateo Cnty.
Black & White	5-27-56	1:20,000	2946-8-21	San Mateo Cnty.
Black & White	3-23-41	1:24,000	6660-82,-83	Fairchild Aerial Surveys

SECTION C — GEOTECHNICAL ENGINEERING

Responses are presented in the following subsections to comprehensively discuss each subject matter:

Subsection 1 — Settlement
Subsection 2 — Seismic Coefficient
Subsection 3 — Slope Stability
Subsection 4 — Stability of Reinforced Earth and Piers
Subsection 5 — Miscellaneous Items

Subsection 1 — Settlement (ESC Comment 14, page 6 and HTA Comment 2, page 1)

ESC Comment 14, page 6

Section GS-4 illustrates a proposed Townhome building situated on fill ranging from 25 to 50 feet in thickness. Several of the Townhome buildings will be situated on varying thicknesses of fill and/or cut/fill transitions. Additional information should be provided regarding the amounts of total settlement and differential settlement anticipated at such locations. Estimates of time-rates of settlement should also be provided, and the possible effects of long-term settlement on the proposed structures should be evaluated. Recommendations should be provided to mitigate the effects of these settlements on the proposed structures, the adjacent improvements, and the underground utilities. Settlement of sidehill fills can also result in lateral movement and deformation. The effects of such movements, and recommendations to mitigate them, should be evaluated and discussed in the SFS report.

HTA Comment 2, page 1

Potential for settlement of buildings and other improvements requires further evaluation and discussion due to the large thickness of fill planned over the moderately compressible alluvium west of the Hillsborough West Apartments (Section GS-4) and the planned varying thickness of fill under some buildings. The report should include a discussion of the potential for hydrocompression of fill after construction improvements. Delaying construction until after the first winter may not be long enough under extreme fill conditions.

Response:

The above comments require further discussion and additional information on the following specific items:

a) Potential for hydrocompression settlement of the fill.
b) Amounts of total and differential settlements and time rate of settlement.
c) Potential for lateral deformation of sidehill fills.
d) Effects of long-term settlement on the proposed structures.

The comments also indicate that the recommended delay period may not be long enough under extreme fill conditions.

The 1993 SFS report treated settlement aspects of the site on pages 5, 6, 24, 48, 49, 58 and 59. It was our conclusion that the fill settlement could be mitigated by delaying foundation construction through the first winter cycle in areas where the fill exceeds 10 feet in thickness. The effect of settlement was also considered in the foundation recommendations, together with other site and geotechnical conditions. The calculation sheets for analyses supporting the above conclusion and providing the basis for the foundation recommendations are included in Section C.1 in the Appendix of this report.

Not all details of the analysis and calculations were provided in the 1993 geotechnical report. In response to the comments of the reviewers, plans, cross sections, critical buildings, and calculation sheets, including soil data and various conditions defined for the analysis, are also provided herein.

a) Hydrocompression of Fills

The comment expressed a concern for potential hydrocompression of the proposed canyon fill. The response will first discuss aspects of hydrocompression and the results of additional laboratory studies conducted since the 1993 report was issued.

Hydrocompression is a phenomenon of compression of a soil mass due to wetting, and is recognized as an important contributor to the settlement of compacted fills. In general, collapsible soils, such as loose silts or mixtures of silt and fine sand, are more susceptible to hydrocompression than clays. It is a consensus that clayey soils compacted wet of optimum moisture content generally experience negligible hydrocompression, provided the overburden pressure does not greatly exceed the compaction-induced prestress. The native soils and rocks on the site have clay-like engineering properties when compacted to above 90% relative compaction at a moisture content wet of optimum (per ASTM test method D1557). This characteristic is supported by test data on compacted native materials. The results of oedometer tests indicate the compaction-induced prestress to be approximately equivalent to 40 feet of fill. Therefore, clayey fills 40 feet or less in thickness would be free from the concern of hydrocompression, provided that the fill is properly constructed. However, hydrocompression is a major concern for fills having much greater thickness, as in the case of high earthfill dams.

Hydrocompression of compacted fills has attracted much attention in recent years and, at the same time, has been a subject of controversy within the geotechnical profession. Review of the literature indicates that there is significant disagreement within the profession regarding assessment of hydrocompression of fills. As an example, the Villa Trinidad and Villa Martinique subdivisions in the north San Diego area were constructed during the 1970s by filling canyons with clayey sand to more than 70 feet deep locally. The canyon fills experienced more than 11 inches of settlement within nine years, causing damage to homes built on the fill. The behavior of this fill was investigated by a number of professionals. Brandon et al. (1990) looked at the problem on the basis of hydrocompression and computed the settlement based on the results of laboratory tests that were intended to simulate the hydrocompression process in the field. Their assumptions were that the fill was well constructed, uniformly compacted to an average relative compaction of 92% to 93% and placed on well-prepared ground, without ascertaining whether these assumptions represented the actual field conditions of the fill. Their analysis concluded that hydrocompression accounted for the total settlement experienced by the buildings. However, others who had actual soil data and information on the field condition of the fill, did not accept the conclusion of Brandon et al. Specifically, Spang and Hardman (1992) indicate, "The majority of compacted fills constructed within the San Diego area, of comparable depth, similar materials, and using construction practices common at the time, have performed adequately." On the basis of factual data from more than 100 test borings drilled within Unit 5 of Villa Trinidad, Pradel et al. (1992) cite construction defects as the cause of the settlement. Their findings included insufficient removal of loose topsoil, presence of organic matter, absence of underdrains and inadequate compaction of the fill (80% relative compaction as opposed to 92% to 93% assumed by Brandon et al.), among other defects.

The assessment of hydrocompression is commonly based on the results of oedometer tests performed on soil samples fabricated to represent the compacted fill in the field. Since fill undergoing hydrocompression in the field may not be fully saturated consolidation tests would provide conservative predictions of settlement due to hydrocompression. The 1993 SFS report presented the results of consolidation tests on fabricated samples from two sources, marked as Sample No. 21-A and X1-A (see Plates 38 and 39, Appendix A in that report).

To provide a broader data base for assessment of hydrocompression of compacted fills for the subject project, additional laboratory tests were performed. The soil sample used for this study is designated as Sample No. HC-1, and is a mixture of weathered sandstone (previous Sample No. X1-A) and the siltstone melange from the cliff along Ticonderoga Drive. A laboratory compaction test was performed on this sample, in accordance with the ASTM test method D1557, and the test results are presented on Plate E in Section A of the Appendix. Two series of compression tests were performed in oedometers on fabricated specimens from this sample. In the first series, six specimens were fabricated in three pairs at molding moisture contents of approximately 1.0% below and 1.0% and 2.0% above the optimum moisture content, respectively, for each pair, to approximately 92.0% relative compaction. The 92.0% relative compaction was chosen because the fill in the field would likely be compacted in the range of 90.0% to 95% relative compaction in order to meet the minimum requirement of 90.0% relative compaction. One specimen in each pair was incrementally loaded to 4,300 p.s.f. and the other to 8,600 p.s.f., without submerging the specimens in water. The specimens were kept for 60 minutes under each increment of loading. At the conclusion of the incremental loading, the specimens were inundated with water and continuous readings were taken of the compression for a one-day duration. The intent of this test series was to determine the effect of placement moisture content on hydrocompressibility. The overall test procedure for this series follows the procedure introduced by Brandon, Duncan and Gardner (1990). Results of these tests are presented on Plates F, G and H in Section A of the Appendix. In the other series, two nominally identical specimens were fabricated at a moisture content slightly above the optimum to a relative compaction of 92.7%. Oedometer tests were performed on these specimens, one submerged and the other without being submerged under water. These tests were intended to determine the effect of fill thickness and/or the overburden on hydrocompressibility. This test procedure is similar to the "double oedometer test" described by Lawton et al. (1989). Results of this test series are presented on Plate I in Section A of the Appendix.

Results of the above tests provide the following fill characteristics relating to hydrocompression:

i) The oedometer tests showed a significant amount of instantaneous compression under the surcharge loads before the specimens were submerged in water. This suggests that the fill would undergo a similar instantaneous compression during placement, resulting in an increase in density and strength at depth. The test specimens also experienced volume expansion after being submerged in water. This was thought to be due to relief of the compaction-induced prestress, rather than due to expansiveness of the soil. The fill material tested has a Plasticity Index of less than 10%. The pattern of the volume expansion in terms of the surcharge loads suggests that this effect would be most pronounced in the upper 5 feet ± of the fill.

ii) Test results suggest that hydrocompression would be negligible when the placement moisture content is kept wet of optimum for the height of fills anticipated for the subject project.

Results of the laboratory analyses support the conclusion that hydrocompression is not a significant factor for the proposed fills. This conclusion is consistent with the findings of others. Lawton et al. (1989) performed parametric studies on the hydrocompressibility of a compacted clayey sand. On the basis of these studies, they concluded that compacted fills are free from hydrocompression when the placement moisture content is maintained wet of optimum and the overburden pressure does not greatly exceed the compaction-induced prestress.

b) Settlement Analysis — Amount and Rate of Settlement

b-1) Soil Parameters

The compression ratio for hydrocompression of fills was determined to be 0.017 from the results of the "double oedometer test" presented on Plate I, and 0.012 from the results of the tests presented on Plates F, G and H in Section A of the Appendix. An average value of 0.015 was chosen for analysis.

The rate of secondary compression (or creep) of fills was determined from the results of consolidation tests on Samples 21-A, X1-A and HC-1-D2. The conventional analysis in this branch of soil mechanics assumes that the rate of secondary compression is independent of the layer thickness and consolidation stress, (see Ladd, 1971, among others). However, for fill undergoing self-weight compression, as well as for natural deposits, empirical evidence suggests that the rate of secondary compression increases linearly with the thickness of the fill or deposit (Sohn and Lee, 1994; Janbu et al., 1989). This basis provides larger differential settlements between sections of differing fill thickness. On this basis, a linear regression equation was derived, as shown on page C.1.4 of the calculation sheets in Section C.1 of the Appendix, for determination of the rate of secondary compression in terms of the self-weight pressure (p) at mid-height of the fill. This equation is:

$$C_\alpha = 0.056p + 0.976 \qquad [2]$$

The coefficient of consolidation for the fill was determined to be 0.5 square feet per day from the results of the previous consolidation tests (Samples 21-A and X1-A).

For the alluvium deposit in the main canyon, the soil parameters for settlement analysis were determined from the results of consolidation tests presented on Plates 36 and 37 in the 1993 report. Their values are as follows:

Compression Ratio:	0.075 for the upper 10 feet and 0.065 for the remainder,
Recompression Ratio:	0.030 for the upper 10 feet and 0.025 for the remainder,
Coeff. of Consolidation:	0.035 sq. ft. per day, and
Rate of Secondary Compression:	

$$C_\alpha = 0.0833p + 0.55 \qquad [3]$$

b-2) Selection of Soil Profile and Critical Area for Analysis

Important factors that determined the selection of areas included in the analysis were the amount of fill, variation of the fill and alluvium thickness with distance, as well as the location of proposed improvements and buildings.

Several lines of soil profile were constructed across the canyon fill behind the Hillsborough West Apartments to determine the most critical location of fill settlement for improvements and underground utilities. The most critical soil profile crosses the cul-de-sac. This critical soil profile includes a maximum thickness of 20 feet of alluvium and 40 feet of engineered fill. This profile is shown on page C.1.5 of the calculation sheets.

Critical buildings were selected based on the amount and thickness variation of fills under each building. Figure D is an index plan showing designation of the buildings in the townhome area. Table S-I presents the maximum thickness of underlying fill and the ratio of differential fill thickness to horizontal distance for each building.

TABLE S-I

Building	Max. Fill Thickness	Max. Thickness Difference to Distance Ratio
A	17′	0.25
B	12′	0.40
C, D, E	0	0
F	40′	0.40
G	40′	0.35
H	50′	0.50
I	60′	0.50
J	14′	0.25
K	20′	0.20
L, M	0	0
O	36′	0.35
P	20′	0.20

The maximum thickness shown in Table S-I includes the anticipated overexcavation to remove surface deposits, such as colluvium and alluvium. As indicated by the above Table, the most critical buildings are "H" and "I" in terms of the maximum thickness of fill and the rate of change in fill thickness with distance.

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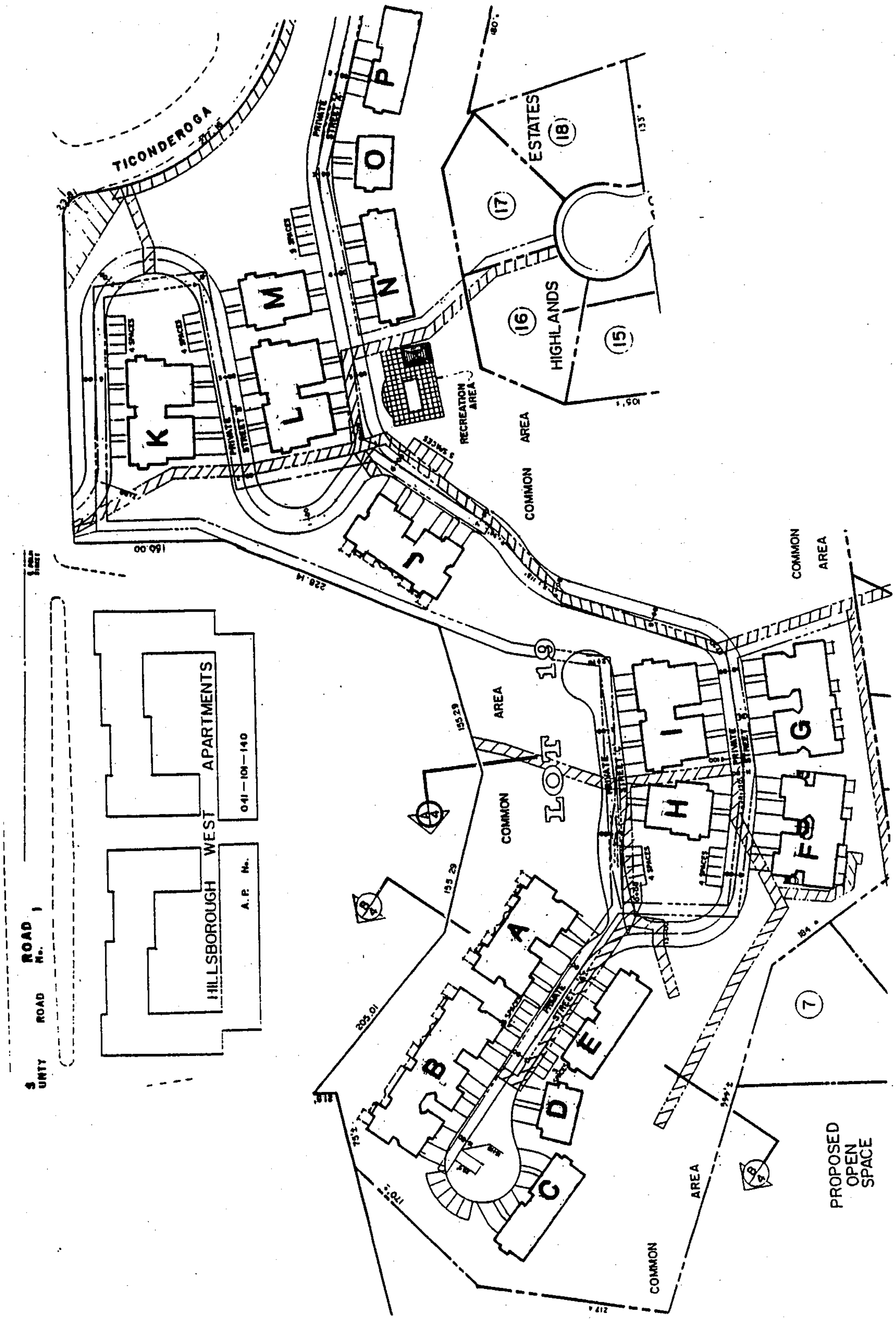


Figure D - Index Plan of Townhome Development

SOIL FOUNDATION SYSTEMS, INC.

Details of the soil profile under and across these buildings are shown on pages C.1.6 through C.1.8 of Section C in the Appendix. Note that Section GS-4 on Plate 3 of the 1993 report was intended for use in slope stability analysis, as discussed in that report. The fill, as shown on Section GS-4, includes the retaining wall backfill. For the purpose of settlement analysis, the retaining wall backfill needs to be separated from that portion of the fill constructed in the initial mass grading. The fill section shown in GS-4 also does not include the overexcavation to remove the surface deposits in preparing the site for filling.

b-3) Methods and Rationales Adopted for Analysis

Both the fill and alluvium were treated as undergoing settlement due to primary consolidation and secondary compression (or creep). The primary consolidation settlement was treated as a stress-strain problem whose characteristics were defined by the compression and recompression ratios presented earlier in this response. The secondary compression settlement was obtained by integrating the rate of secondary compression over the time span of interest. This time span was taken as 30 years from completion of the building, and represents the life of the building.

Alluvium was included in the thickness of compacted fill in the settlement analysis. The alluvium is less than 10 feet thick and transitions into the colluvium at the location of Buildings "H" and "I". At this location, the canyon is narrow and the overexcavation to remove the colluvium, and other excavations necessary for site preparation and underdrain installation, would also remove the alluvium. Geologic Section GS-4 does not depict all of the profile details across the canyon that led to the above consideration. The removal of the alluvium in the above building areas will be ensured during the grading operations.

The foundation settlement analyses considered hydrocompression and secondary compression (creep) of the fill due to self-weight and additional consolidation of the fill under the imposed loading. The imposed loads were considered to consist of the weight of the retaining wall backfill and the weight of the building. Retaining wall backfills that would be placed after completion of the mass grading occur only behind garage retaining walls for the upper level units. A uniformly distributed loading of 700 p.s.f. was assigned to account for the retaining wall backfill over the upper level garage area and 300 p.s.f. over the entire building area to account for the weight of the building.

The fill was treated as saturated for determination of the time-settlement relationship. Since it is unlikely that the fill will ever attain such a condition, this assumption will result in an overestimate of the long-term settlement. As the fill would be provided with extensive underdrains, a two-way drainage condition was adopted in the analysis. In determining the anticipated post-construction settlements, the time to 50% of the imposed loading (the weight of retaining wall backfills and the building weight) was taken as 2.5 years from the beginning of the fill operation. This time span includes, on the average, six months for grading, one year for the delay of foundation construction and one year for time to receive 50% of the imposed loads.

b-4) Results of Analysis and Discussions

Fill settlement attributable to hydrocompression is graphically presented in Figure E.

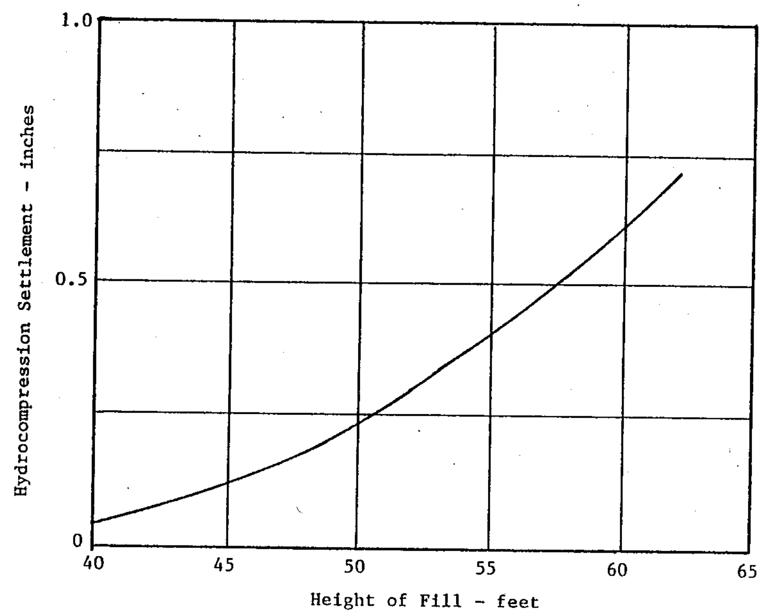
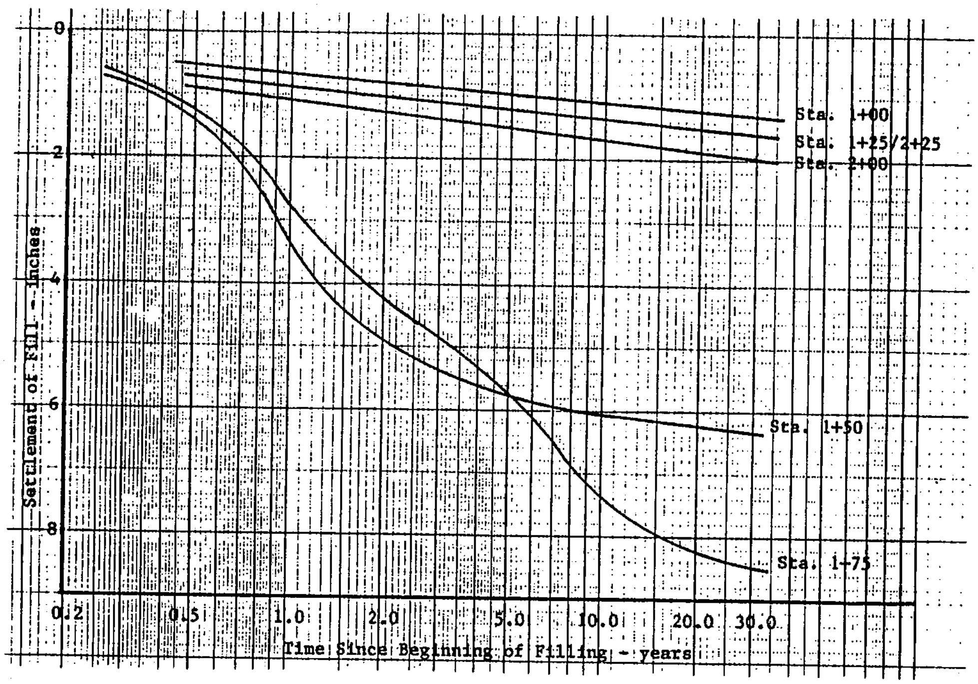


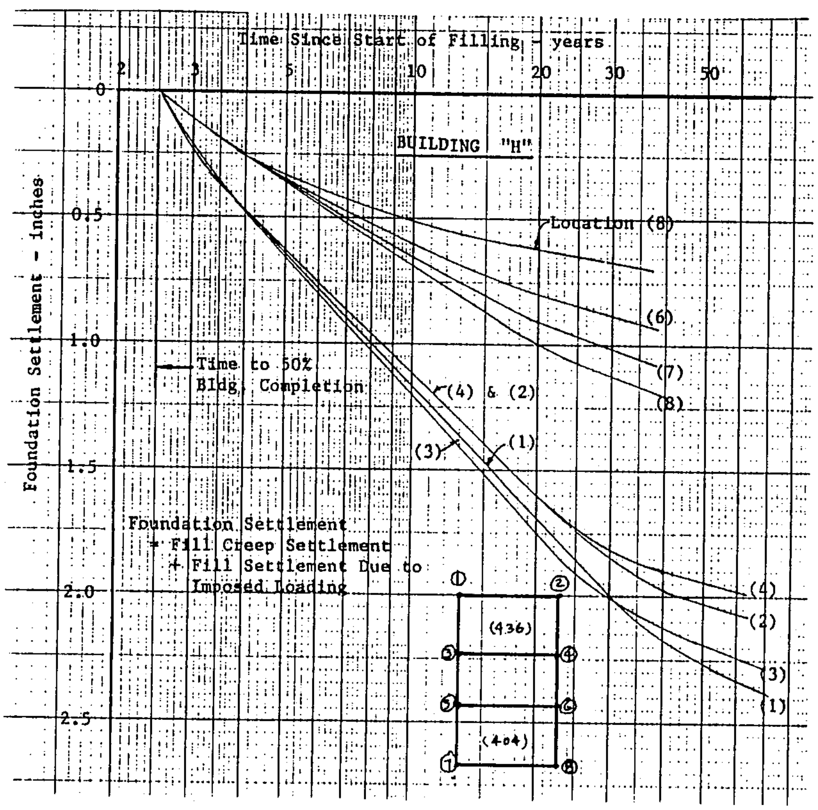
Figure E - Hydrocompression Settlement of Fill

Hydrocompression, in fills consisting of the native materials, is inactive for fill depths of less than 40 feet. For a fill height of 50 feet, the amount of hydrocompression is approximately 1/4 inch. This is the maximum condition anticipated in building areas, including the amount of overexcavation of the surface deposits. Therefore, hydrocompression is insignificant in assessment of the fill settlement for the subject project. However, as evident from the above graph, hydrocompression can be a significant contributor to settlement of fills consisting of material similar to the native soil when the fill becomes more than 70 feet in thickness.

Secondary compression or creep settlement of the fill is significant and accounts for most of the self-weight settlement of the fill under 50 feet in thickness. This is an important factor in assessing the overall time-settlement relationship that determines differential settlements at any given period. When estimating such creep settlement, it is important to consider the dependency of the creep rate on the stress level or thickness of the layer (Sohn and Lee, 1994; Janbu et al., 1989). This method estimates larger differential settlement due to creep in comparison to the conventional method, which assumes the rate of secondary compression to be independent of the layer thickness. Results of settlement analyses for buildings "H" and "I" are presented in Figures F and G, respectively.

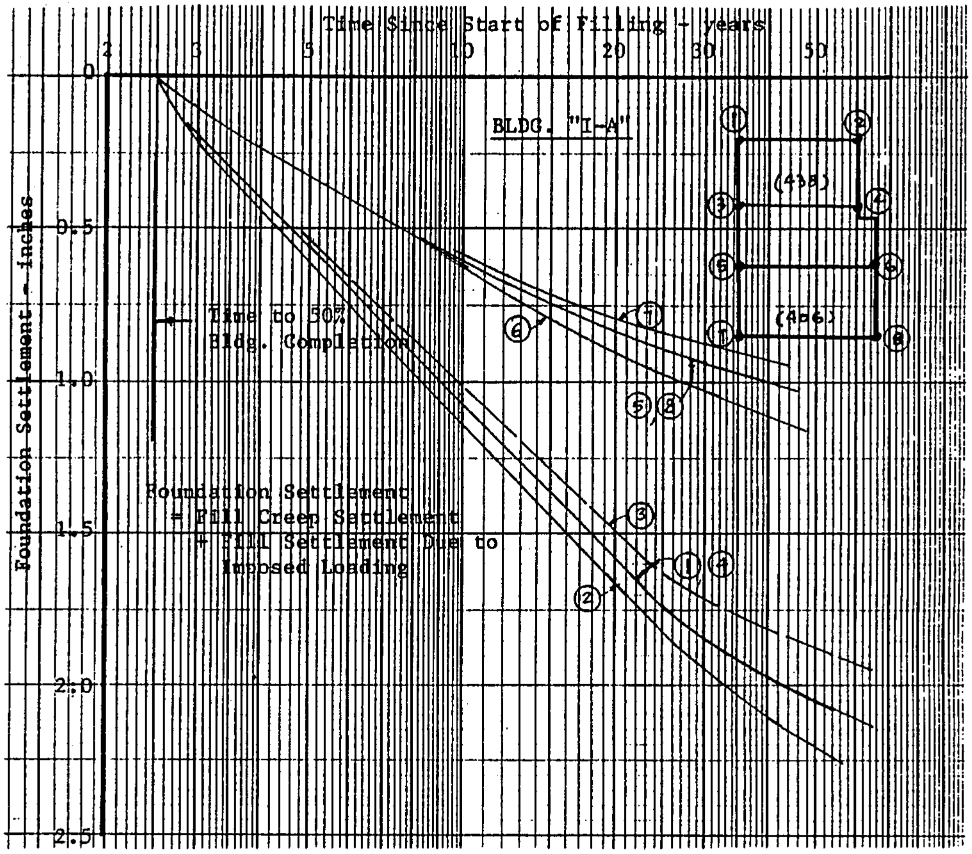


a) Fill Settlement Along Center Line of Cul-de-sac

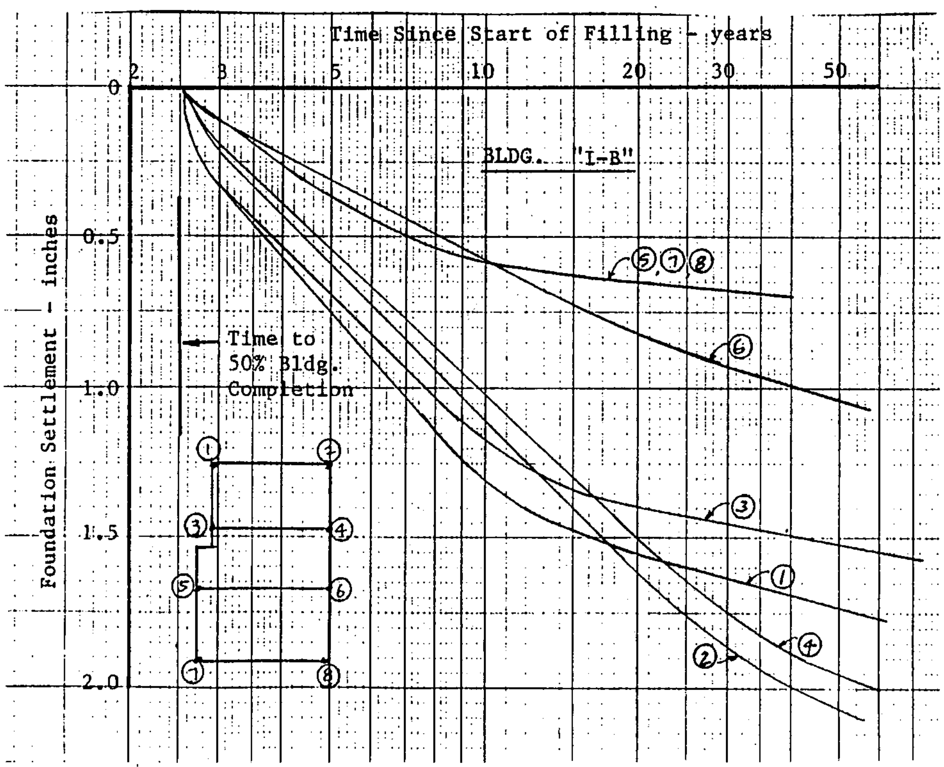


b) Foundation Settlement, Bldg. "H"

Figure F - Amount and Rate of Settlement



a) Foundation Settlement, Bldg. "I-A"



b) Foundation Settlement, Bldg. "I-B"

Figure G - Amount and Rate of Settlement

d) Effect of Long-Term Settlement on the Proposed Structures

The effect of differential settlement on buildings was analyzed in terms of the amount of differential settlement and the corresponding angular distortion of the building. This analysis followed the criteria presented by MacDonald and Skempton (1955) and Grant et al. (1974). The criteria adopted to determine tolerable differential settlements for buildings are as follows:

Angular Distortion (= differential settlement/distance) $\leq$ 1/250 to 1/300
Allowable Differential Settlement $\leq$ 500 x (allowable angular distortion).

The above criteria are for wood-frame and stucco or wood siding construction, and are interpolated from the criteria for brick wall and sheet metal wall construction. According to the above criteria, the allowable differential settlement for the townhomes at critical locations is about 2 inches, whereas the predicted differential settlement is on the order of 1 inch. The predicted angular distortion is also well within the allowable limits.

The potential for lateral deformation of the fill and building distortion due to differential settlement becomes more favorable when the effect of the slow rate of the long-term settlement is introduced in the above assessments. An excellent treatise on the settlement rate effect is found in Grant et al. (1974).

The 1993 SFS report recommended that the proposed townhomes be supported by drilled piers extending a minimum depth of 10 feet into the fill and 5 feet in cut areas. This recommendation is apt to be construed as solely intended for minimizing the foundation settlement. It was provided mainly to account for the variable ground conditions expected to result in the upper several feet due to the complex pattern of ground breaks to be created by the grading scheme beneath the buildings. Variable ground conditions are also expected in the upper 5 ± feet of the fill from rebound (or swell) due to relief of the compaction-induced prestress by wetting and lack of overburden to preserve the prestress. This fill behavior was extensively treated in the previous section dealing with hydrocompression.

Piers embedded in fills may be subjected to downdrag as the fill subsides. This consideration becomes important when the piers penetrate a compressible layer overlain by fill and are embedded in an incompressible stratum. If the pier and fill settle by the same rate and amount, no downdrag would develop. For the 10 foot-deep pier foundation proposed for townhomes in the 1993 report, the condition that would generate the largest negative friction would occur if the piers penetrated 10 feet of fill and then were embedded in bedrock. Whether or not downdrag would be a significant factor will depend on the amount of settlement anticipated within the top 10 feet of the fill, as well as the relative movement between the fill and the pier. Based on extrapolation of the fill settlements shown on page C.1.2 of the calculations, approximately 1/4 inch of settlement is expected from 10 feet of fill within the first 20 years ± of the building life.

As an alternative for control of differential settlements, this supplemental report considers the use of deep piers penetrating the entire fill depth and embedded in bedrock. The issue of downdrag due to fill settlement becomes an important consideration for design of these piers. Further discussions on this alternative and recommendations for design of the piers are presented on page 45 in the following subsection e and in Section D of this report.

Reinforced earth slopes and walls will undergo settlement due to self-weight compression in the same manner as discussed for compacted fills. The actual thickness of the fill for reinforced earth slopes and walls, as measured on a vertical plane, is not as great as for the canyon fill. The reinforced earth structures primarily support streets which are better able to accommodate long-term settlement. When roadway construction is delayed one year from completion of the earth structure, the amount of creep settlement expected from a 40-foot thick fill during the first ten-year performance of the pavement is less than 1/2 inch. This is not a significant amount to affect performance of the streets.

e) Settlement Mitigation Measures

Various measures were discussed in this supplemental report, as well as in the 1993 report, aimed at either reducing settlements or minimizing the effects of settlement on structures. Settlement analysis for a setting such as the subject site is inevitably affected by numerous factors. Consequently, the analysis can at best yield approximate predictions. Therefore, it is prudent to consider all possible practical means of reducing the settlement and lessening the effects of settlement, regardless of the magnitude of settlement predicted by analysis. This section is a summary of all the measures that have been treated in this report and the 1993 report.

i) Measures for Reducing Settlement

Special measures recommended for the reduction of settlement include overexcavation of the alluvial deposits in the drainage swale behind the Hillsborough West Apartments and placement of fills at wet of optimum moisture contents. Overexcavation of the alluvium will extend to a minimum depth of 10 feet. The proposed overexcavation will eliminate the entire depth of alluvium in building areas and greatly reduce differential settlement across the area of the proposed cul-de-sac. Placing compacted fills at higher moisture contents has been shown to be advantageous for minimizing the potential for hydrocompression settlement. These measures will be undertaken in addition to routine practices such as ground preparation, deep sidehill benching, overexcavation of weak materials and installation of underdrains, among others, during grading operations.

ii) Measures for Minimizing the Effect of Settlement

Structures can be adversely affected by post-construction settlements. The 1993 report recommended that foundation construction be postponed through the first winter upon completion of the site grading as a method for reducing the post-construction settlement. The recommended drainage control measures and adequate slope maintenance practices are also intended to minimize post-construction settlement.

Differential settlements can affect the structural stability of a given building during its design life, including the foundation. The estimated maximum differential settlement for the townhome building underlain by the largest variation in fill thickness is on the order of one inch. For the proposed size and type of townhome construction, differential settlements of less than two inches are not expected to cause significant structural distress. Therefore, the anticipated differential settlement is well within structurally tolerable limits. The 1993 report recommended supporting the townhomes on piers 10 feet deep in fills and five feet deep in cuts. This foundation design was not intended as a means of eliminating differential settlement. This recommendation considered the nonuniform ground conditions that result from the grading scheme and hydrological factors within the upper several feet.

While the anticipated differential settlement is acceptable from an engineering perspective, for functional reasons it may be desirable to further reduce differential settlement, depending on the criteria of the individual occupants. For the type of foundation considered for townhomes in the 1993 report, it would be possible to further reduce differential settlement at critical locations. This may be accomplished by locating the building so that it is underlain by a smaller variation in fill thickness or by structurally separating the individual units that are contained in one contiguous building in the current plan. The effects of differential settlement can also be reduced by structurally designing the foundation to withstand the differential settlement. The critical buildings are "F", "G", "H" and "I", as designated in Figure D. Anticipated foundation settlements at various locations beneath buildings "H" and "I" were shown in Figures F and G, respectively. The extreme measure for control of settlement effects would be the total elimination of differential settlement. This would require deep piers penetrating through the fills and embedded in bedrock. Foundation design considerations for deep piers will be provided in Section D of this report.

In terms of minimizing the effects of post-construction settlements, it will be recommended that all underground pipelines crossing areas underlain by fill of varying thickness be provided with flexible connection joints.

f) References Cited for Settlement Analysis

Brandon, T. L., J. M. Duncan, and W. S. Gardner, 1990. "Hydrocompression Settlement of Deep Fills." *Journal of Geotechnical Engineering*, ASCE, Vol. 116, No. 10, pp. 1536-1548.

Grant, R., J. T. Christian, and E. H. Vanmarcke, 1974. "Differential Settlement of Buildings." *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 100, No. GT 9, pp. 973-991.

Janbu, N., G. Svanϕ, and S. Christensen, 1989. "Backcalculated Creep Rates from Case Records." *Proceedings*, 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, Vol. 3, pp. 1809-1812.

Ladd, C. C., 1971. "Settlement Analyses for Cohesive Soils." M. I. T. Research Report R71-2, Soils Publication 272.

Lawton, E. C., R. J. Fragaszy, and J. H. Hardcastle, 1989. "Collapse of Compacted Clayey Sand." *Journal of Geotechnical Engineering*, ASCE, Vol. 115, No. 9, pp. 1252-1266.

Lawton, E. C., R. J. Fragaszy, and M. D. Hetherington, 1992. "Review of Wetting-Induced Collapse in Compacted Soil." *Journal of Geotechnical Engineering*, ASCE, Vol. 118, No. 9, pp. 1376-1394.

Lee, K. L. & C. K. Shen, 1969. "Horizontal Movements Related to Subsidence." *Journal of Geotechnical Engineering*, ASCE, Vol. 100, No. 1.

MacDonald, D. H., and A. W. Skempton, 1955. "A Survey of Comparisons Between Calculated and Observed Settlements of Structures on Clay." *Proceedings*, Institution of Civil Engineers, London, England, p. 318.

Pradel, D., G. Raad, and R. G. Harter, 1992. "Discussion on Hydrocompression Settlement of Deep Fills." *Journal of Geotechnical Engineering*, ASCE, Vol. 118, No. 6, pp. 954-956.

Sohn, K. C., and S. Lee, 1994. "A Method for Prediction of Long Term Settlement of Sanitary Landfill." *Proceedings*, First International Congress on Environmental Geotechnics, Edmonton, Alberta, Canada, pp. 807-812.

Spang, W., and S. L. Hardman, 1992. "Discussion on Hydrocompression Settlement of Deep Fills." *Journal of Geotechnical Engineering*, ASCE, Vol. 118, No. 6, pp. 952-954.

Subsection 2 — Seismic Coefficient (ESC Comments 11 & 17 and HTA Comment 3-c)

ESC Comment 11, page 6

- *The SFS report cites a peak ground acceleration of 0.5g for a magnitude 7 earthquake on the San Andreas fault, based on attenuation curves by Seed and Idriss (1982). Data from the Loma Prieta Earthquake of 1989 (M 7.1) indicate a peak acceleration of 0.64g at Corralitos, approximately 6.5 miles from the earthquake epicenter (CSMIP, 1989). The report should also discuss predicted peak accelerations for a magnitude 8 to 8.3 event on the San Andreas fault. Several other references are available for estimating peak horizontal accelerations, both for near-field and distant earthquakes. (See 1992 DMG Open File Report 92-1, "Peak Acceleration from Maximum Credible Earthquakes in California," among others.) The implications of higher peak accelerations should be discussed by the consultant.*

ESC Comment 17, page 7

The SFS report indicates that a seismic coefficient of 0.15 was used in the stability analyses. While this value is commonly used to evaluate the stability of slopes during seismic shaking, it may be more appropriate to utilize a higher coefficient for this site because of its proximity to the San Andreas fault (see paragraphs 11 and 13 above), and its history of slope instabilities.

HTA Comment 3-c, page 2

In our opinion, a seismic coefficient of 0.15 for stability analysis is low considering the proximity to the San Andreas fault.

Response:

The slope stability analyses presented in the 1993 SFS report were based on a seismic coefficient of 0.15. The comments suggest that a higher seismic coefficient should be used.

This issue needs to be viewed in a broader context. As elaborated in the famous lecture by Professor Ralph Peck, entitled "Pitfalls of Overconservatism in Geotechnical Engineering" (1975 Martin S. Kapp Lecture), arbitrary bases introduced in each step of a multi-disciplinary process for the development of a design parameter lead to overconservatism which "can defeat its purpose and lead to wastefulness and less satisfactory solutions than if reasonable risks were accepted." Citing seismic stability of Arctic pipelines as an example, Professor Peck explains overconservatism as the compounding of conservatism introduced by the geologist in geologic evaluation, the seismologist in estimating the recurrence probability and magnitude of the earthquake, the earthquake engineer in predicting the ground motions, the geotechnical engineer in determining the soil strength parameters and in stability analysis, and finally the structural dynamicist in determining the behavior of the pipeline during the earthquake.

The seismic coefficient is only one of many factors considered in the slope stability analysis. Justification for the parameters used in the geotechnical analyses was presented in various parts of the 1993 report. As indicated in the Earth Systems comment, a seismic coefficient of 0.15 is commonly used for slope stability analyses. Higher values of seismic coefficient would be justified in situations where there is a concern of strength degradation that would result from development of high pore pressure or liquefaction (Marcuson, 1981). The 1993 report presented discussions indicating that liquefaction potential is not a significant concern for the site. The effect of the site's proximity to the fault should be assessed on the basis of recognized attenuation relationships for earthquake ground motions, together with other critical factors that determine stability of slopes at the site. It was the conclusion of the 1993 report that a coefficient of 0.15 is reasonable to use in the slope stability analyses, based on the discussions presented below.

As indicated in the 1993 SFS report, the undrained static strengths of the deposits were used to model dynamic strengths. This approach is conservative (Makdisi and Seed, 1978). The 1993 report recommended that the allowable supporting capacity of structures resting on soils, such as bearing capacity of foundations and passive earth pressures, be increased by 1/3 to include seismic loading. This is in conformity with the current standards of practice in geotechnical engineering. Similarly, increasing the soil strength in the seismic slope stability analyses is justifiable. However, this was not done in the analyses for the 1993 report. Professor Ishihara (1985) indicates that soil deposits exhibit significant increases in strength under dynamic loading. This increase in the cohesion value under dynamic load can range from 2.4 times the static value for volcanic clays to 1.6 to 1.9 times for fills consisting of 60% sand and gravel and 40% silt and clay. Further, he cites several case histories in Japan where the actual slope performance during an earthquake was better predicted when the appropriately adjusted static strength was used in the analysis. In developing a ground response relationship for earthquakes at soft soil sites, Idriss (1991) suggests that the ratio of undrained shear strength to the effective overburden pressure may be increased by a factor of 1.3 to 1.5 to account for the effect of the rate of loading. The above discussion leads to the conclusion that it is quite reasonable to increase the static strength by a factor of 1.5 for sites underlain by stiff soils, such as the Highland Estates site. In summary, a significant degree of conservatism was introduced into the seismic slope stability analysis by using unfactored static strength for dynamic strength.

The Seed-Martin shear-beam solution (Seed & Martin, 1966) was applied to assess the seismic coefficient. Seed and Martin computed average seismic coefficients for a wide range of natural periods by using the El Centro earthquake records. It is possible to obtain the average seismic coefficient for the subject site by scaling the Seed-Martin computational results in terms of the ratio of peak ground accelerations at both sites. This type of scaling is reasonable for the intended purpose (Matasovic, 1991). This scaling method was previously discussed on page 25 of this report. To perform the scaling, the natural period and peak ground accelerations for the subject site must be determined. The natural period for the site was determined to be 1.2

References Cited in Discussion of Seismic Coefficient:

Idriss, I. M., 1985. "Evaluation of Seismic Risk in Engineering Practice." Theme Lecture, *Proceedings*, Eleventh International Conference for Soil Mechanics and Foundation Engineering, San Francisco, U.S.A., Vol. 1, pp. 255-320.

Idriss, I. M., 1991. "Earthquake Ground Motions at Soil Sites." *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. III, pp. 2265-2275.

Ishihara, K, 1985. "Stability of Natural Deposits During Earthquakes." Theme Lecture, *Proceedings*, Eleventh International Conference for Soil Mechanics and Foundation Engineering, San Francisco, U.S.A., Vol. 1, pp. 321-376.

Makdisi, F. I. and H. B. Seed, 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT7, pp. 849-867.

Marcuson III, W. F., 1981. "Earth Dams and Stability of Slopes Under Dynamic Loads." Moderators Report, *Proceedings*, International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla, Rolla, Missouri, Vol. 3, p. 1175.

Marcuson III, W. F., M. E. Hynes, and A. G. Franklin, 1992. "Seismic Stability and Permanent Deformation Analysis: The Last 25 Years." Invited Lecture, *Proceedings*, Specialty Conference on Stability and Performance of Slopes and Embankments-II, ASCE, Vol. 1, pp. 552-592.

Matasovic, N, 1991. "Selection of Method for Seismic Slope Stability Analysis." *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla, Rolla, Missouri, Vol. II, pp. 1057-1062.

Mauchlin, L. and A. L. Jones, 1992. "Peak Acceleration From Maximum Credible Earthquakes in California (Rock and Stiff Soil Sites)." California Department of Mines and Geology, Open File Report 92-01.

Peck, R. B., 1975. "Pitfalls of Overconservatism in Geotechnical Engineering." *First Martin S. Kapp Lecture*, Foundation and Soil Mechanics Group Metropolitan Section, ASCE, pp. 1-10.

In response to ESC's Seismic Comment 12, it was indicated that the USGS methodology for assessing regional seismic slope stability in San Mateo County (Wieczorek et al., 1985) would merit application to a specific site provided the computed static factor of safety and the slope angle for the site are used. The USGS method is based on a concept of "critical acceleration" (see Eq. [1] for definition). When applied to slope GS-7b, the USGS method results in a critical acceleration of 0.5957g. When this acceleration is compared to the acceleration computed from the 1991 Idriss attenuation relationship, it corresponds to an earthquake event between M=7.0 and 7.5. Further, when it is compared to the peak ground acceleration of 0.5g used in our slope stability analysis, the corresponding factor of safety under seismic conditions becomes 1.2 (=0.5957/0.5). However, the computed factor of safety under seismic loading, using a coefficient of 0.15g, was 0.921 for slope GS-7b. This indicates that the basis used in the 1993 report for seismic slope stability analysis is conservative.

The slope stability analyses presented in the 1993 report provided an assessment of global stability. The strength of the deposits, the seismic coefficient, and all other factors considered in the analyses were used for that purpose. The 1993 report recognizes that the site is underlain by complex geologic units, characterized by pockets of weak zones or discontinuities. However, any attempt to account for the adverse effect of such features on slope stability by arbitrarily choosing higher values of seismic coefficient is in our view unsound and inappropriate. Rather, the localized adverse geologic features, where encountered during grading, should be corrected by using accepted methods. Recommendations are provided in the report to ensure this end.

In citing the shortcomings existent in current methodologies, Marcuson (1992) recommended that professionals in the field "Step back and look at the big picture." The various points presented above must be included in this picture. All factors affecting slope performance, including the seismic coefficient, should be considered as a whole when assessing slope stability. The analyses and discussions presented above clearly demonstrate that a seismic coefficient of 0.15 is appropriate for the subject project when other critical factors affecting slope stability have been properly weighed. The above analysis also shows that injecting overconservatism in the selection of the seismic coefficient without allowing an appropriate adjustment to other critical factors that are already conservative would result in wastefulness and more extensive site grading than is necessary.

References Cited in Discussion of Seismic Coefficient:

Idriss, I. M., 1985. "Evaluation of Seismic Risk in Engineering Practice." Theme Lecture, *Proceedings*, Eleventh International Conference for Soil Mechanics and Foundation Engineering, San Francisco, U.S.A., Vol. 1, pp. 255-320.

Idriss, I. M., 1991. "Earthquake Ground Motions at Soil Sites." *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. III, pp. 2265-2275.

Ishihara, K, 1985. "Stability of Natural Deposits During Earthquakes." Theme Lecture, *Proceedings*, Eleventh International Conference for Soil Mechanics and Foundation Engineering, San Francisco, U.S.A., Vol. 1, pp. 321-376.

Makdisi, F. I. and H. B. Seed, 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT7, pp. 849-867.

Marcuson III, W. F., 1981. "Earth Dams and Stability of Slopes Under Dynamic Loads." Moderators Report, *Proceedings*, International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla, Rolla, Missouri, Vol. 3, p. 1175.

Marcuson III, W. F., M. E. Hynes, and A. G. Franklin, 1992. "Seismic Stability and Permanent Deformation Analysis: The Last 25 Years." Invited Lecture, *Proceedings*, Specialty Conference on Stability and Performance of Slopes and Embankments-II, ASCE, Vol. 1, pp. 552-592.

Matasovic, N, 1991. "Selection of Method for Seismic Slope Stability Analysis." *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla, Rolla, Missouri, Vol. II, pp. 1057-1062.

Mauchlin, L. and A. L. Jones, 1992. "Peak Acceleration From Maximum Credible Earthquakes in California (Rock and Stiff Soil Sites)." California Department of Mines and Geology, Open File Report 92-01.

Peck, R. B., 1975. "Pitfalls of Overconservatism in Geotechnical Engineering." *First Martin S. Kapp Lecture*, Foundation and Soil Mechanics Group Metropolitan Section, ASCE, pp. 1-10.

Seed, H. B., and G. R. Martin, 1966. "The Seismic Coefficient in Earth Dam Design." *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, Vol. 92, No. SM3, pp. 25-58.

Wieczorek, G. F., R. C. Wilson, and E. L. Harp, 1985. "Map Showing Slope Stability During Earthquakes in San Mateo County, California." U.S. Geological Survey Map I-1257-E.

Subsection 3 — Slope Stability (ESC Comments 16, 17 & 19, and HTA Comment 3)

HTA Comment 3-a, page 1

Stability analysis was difficult to evaluate because locations of failure surfaces and material properties were not shown on the cross section.

Response:

Cross sections showing the critical failure circle and the material properties for each of the ten sections analyzed for the 1993 report are provided in Section C.3 of the Appendix.

HTA Comment 3-b, page 2

Non-circular surfaces appear to have not been analyzed. Non-circular analysis may be more appropriate for the natural material layering and bedding shown on some of the cross sections.

Response:

Infinite slope geometry was considered in previous analyses. However, the infinite slope results and supporting calculations were not included in the report because circular geometry presents more critical conditions when earthquake forces are considered. In response to the comments, calculations using infinite slope geometry are provided in Section C.3 of the Appendix for cross sections GS-1b, GS-4b, GS-5, GS-7b and GS-10.

HTA Comments 3-d, page 2

It is unclear whether buttressing beyond the property line in Section GS-6 will be allowed and, if not, how will that slope be stabilized?

ESC Comment 19, page 8

Cross section GS-6 shows alternative grading schemes at the toe of the slope. The stability of each alternative should be evaluated and the safety factors presented for each.

cemented, weathered rocks, such as the materials present on the site, are susceptible to disturbance when the samples are obtained by driving the sampler under impact loading. This disturbance results primarily from the destruction of the cementation effect by the impact loading. Therefore, the in-place strength of such materials is underestimated by laboratory strength tests. The strength profile on page 27 of the report represents the typical trend of the strength for the Fm unit on the site, and should be considered conservative.

ii) It is noted that the strength parameters utilized in the analyses were determined from the results of direct shear tests on samples that were brought to saturation prior to testing. Saturation of the samples was intentionally done to account for the reduction in strength due to an increase in moisture in the field. The analysis also considered that compacted fills would be loosened in the upper several feet due to relief of the compaction-induced prestress. This phenomenon was discussed in detail earlier in the subsection entitled "Hydrocompression". This effect was found to be limited to essentially the upper five feet, and therefore it was our conclusion that this effect is insignificant for the overall stability of slopes underlain by thick fills.

iii) This comment requests clarification of the computational method used to determine the required stabilizing forces for those slopes involved in the recommended stabilization schemes. The force computations were presented on pages B124 through B128, Appendix B, of the 1993 SFS report. An equation was derived providing the required resisting force in terms of the overturning moment, circle radius and the deficiency in the factor of safety. This equation is presented at the top of the calculation sheet. The computation is presented in a tabular form. The first column, Computation Line Number, refers to the circle number in the computer printout pertaining to the selected slope. The overturning moment, circle radius and computed factor of safety are taken from this line of the computer printout. The circle tangent elevations are additionally needed to determine the resisting force profile, and these are found by adding the y-coordinate (vertical) of the circle center to the circle radius. Therefore, the computation to determine the required resisting force in the slope is an extension of the slope stability analysis for that particular slope.

ESC Comment 17, page 7

The SFS report indicates that a seismic coefficient of 0.15 was used in the stability analyses. While this value is commonly used to evaluate the stability of slopes during seismic shaking, it may be more appropriate to utilize a higher coefficient for this site because of its proximity to the San Andreas fault (see paragraphs 11 and 13 above), and its history of slope instabilities.

HTA Comment 3-c, page 2

In our opinion, a seismic coefficient of 0.15 for stability analysis is low considering the proximity to the San Andreas fault.

Response:

The seismic coefficient of 0.15 used in the previous analyses was derived for a magnitude 7 earthquake. The preceding subsection, entitled "Seismic Coefficient", presented our selected values of seismic coefficient for higher magnitude earthquakes, ranging up to 8.5. These values were 0.20 for M=7.5, 0.21 for M=8.0 and 0.22 for M=8.5. Additional stability analyses were performed using these values. In these supplemental analyses, the undrained strengths were adjusted to estimate the dynamic strengths. As indicated by Ishihara (1985), factors for adjusting the cohesion portion of the undrained strength range from 1.6 to more than 2.0 for a wide range of soil types. Idriss (1991) indicates an increase in the undrained strength of 30% to 50% under dynamic loading at soft soil sites. For the current analyses a factor of 1.5 was applied to the cohesive component, and no increase was allowed for the frictional component of strength. Sections GS-1b, GS-4a, GS-5, GS-7b and GS-10 were included in the analyses. Computer runs for these analyses are included in Section C.3 of the Appendix.

Results of the analyses are presented below in Table S-III. Compared to the results of the previous analyses, the factor of safety for GS-1b, GS-4a and GS-10 is greater or essentially the same when higher seismic coefficients and undrained shear strengths adjusted for dynamic strengths are used.

TABLE S-III

Section	Factor of Safety			Previous Analyses
	Seismic Coefficient			
	0.20	0.21	0.22	
GS-1b	1.23	1.20	1.17	1.11
GS-4a	1.12	1.09	1.06	1.10
GS-5	1.05	1.02	0.98	1.10
GS-7b	0.88	0.85	0.82	0.92
GS-10	1.09	1.05	1.02	1.02

However, contrary to the other cross sections, GS-7b shows lower factors of safety for magnitude 7.5 or greater, as compared to the case of using M=7.0. This slope was determined to require stabilization based on the previous analysis. Therefore, comparison needs to be made in terms of the required stabilizing forces. The required stabilizing forces and the corresponding factors of safety are shown below in Table S-IV.

TABLE S-IV

Depth Below Reinforced Earth	Required Resisting Force for Section GS-7b, kips			
	Seismic Coefficient 0.20	0.21	0.22	Previous Recommendation
5 feet	175 (1.45)	232 (1.09)	266 (0.95)	220 (1.15)
10 feet	158 (1.56)	199 (1.24)	258 (0.96)	215 (1.15)

Note: Numbers in ( ) are factors of safety.

For GS-7b, the resisting force recommended in the 1993 report is adequate for a seismic coefficient up to 0.21, which is equivalent to an 8.0 magnitude event. These results are favorable given the low probability of occurrence for higher magnitude earthquakes.

Based on the results of the analyses and the discussions presented above, it is concluded that the slope stabilization measures recommended in the 1993 report are adequate for seismic events as large as magnitude 8.0.

Slope Protection During Overexcavation Along Project Boundary

Overexcavation to substantial depths into the natural slope is required at several locations to improve long-term stability of reinforced earth retaining walls and constructed slopes. There are instances where the overexcavation will occur along the project boundary. In these instances, it may become necessary to implement measures for protecting the adjacent property during performance of the overexcavation.

Three areas have been identified that may require protection of the adjacent property during performance of the overexcavation. These areas are designated as "A" through "C" in Figure H, page 58.

Two alternative grading schemes have been proposed for Area "A". One scheme uses reinforced earth retaining walls with the existing drainage ditch left open, as in its current state, while the other involves the construction of a 2:1 fill slope over the ditch. No special overexcavation is necessary for the 2:1 slope alternative. The 1993 report recommended that the reinforced earth section be extended deep into the natural slope. It was estimated that

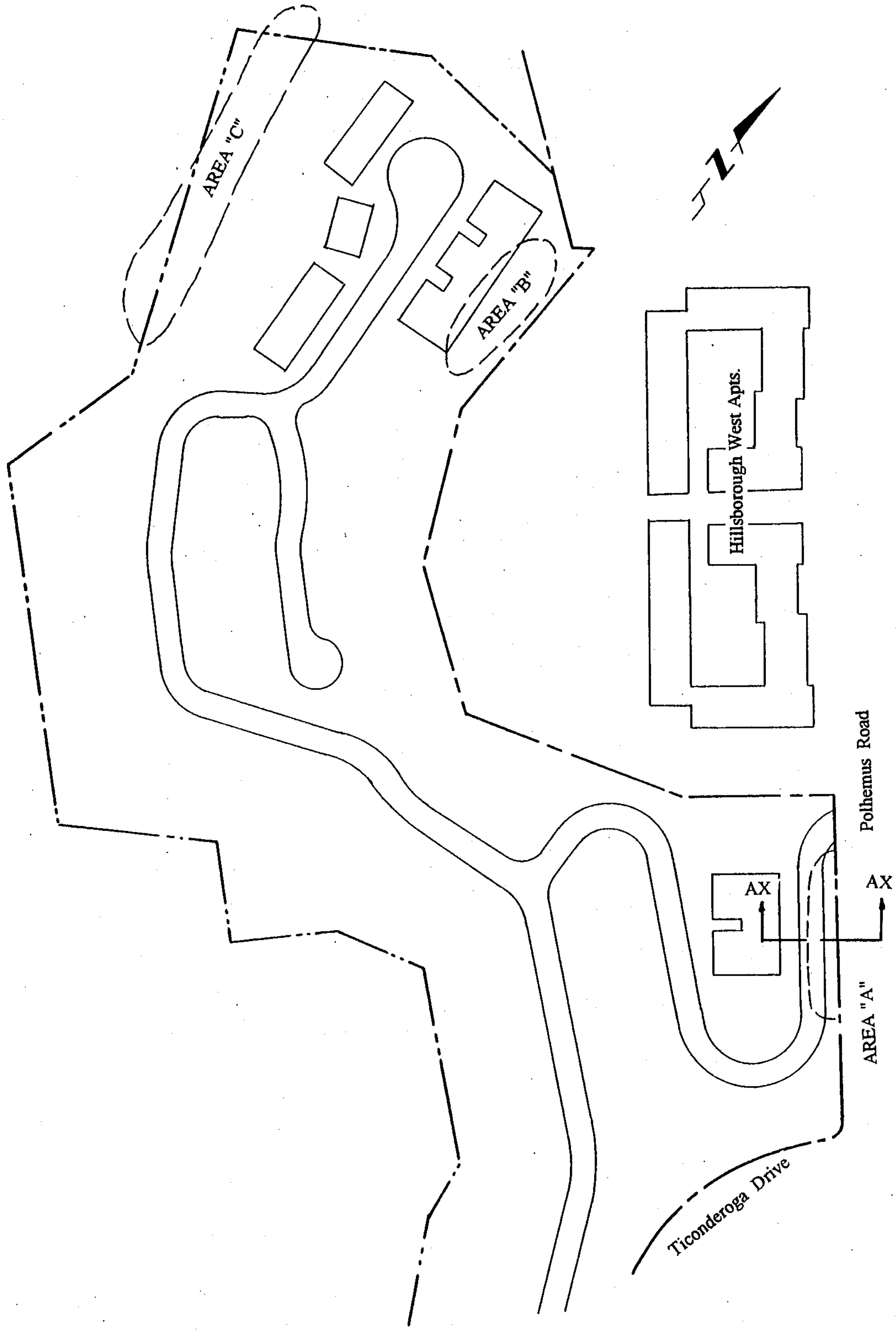


Figure H - Areas of Potential Instability During Overexcavation

approximately 15 feet of overexcavation might be required, or approximately 10 feet as measured from the base of the ditch. Approximately 7 feet of clearance is available between the retaining wall and the property line. This will provide a 0.5:1 unsupported cut along the property line side of the excavation. Area "A" is underlain by 10 feet of overburden soil which becomes stiff below 5 feet. The ground below the retaining wall slopes to the ditch at 3:1. An unsupported temporary excavation should be feasible, provided care is exercised during excavation to avoid excessive disturbance to the cut bank. In the event of potential instability of the cut bank, it would be feasible to protect the slope by providing spaced concrete piers near the base of the excavation. The alternative measures for performing overexcavation in Area "A" are illustrated below in Figure I.

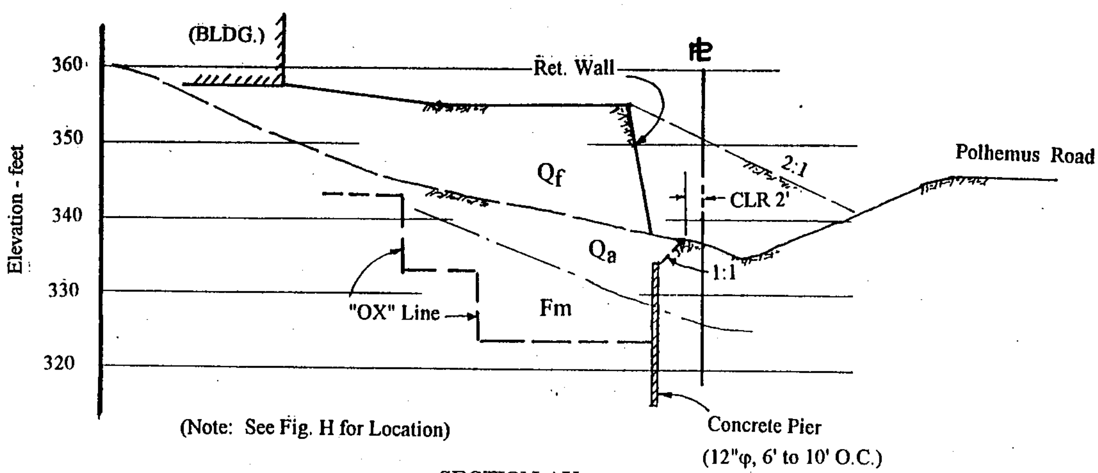


Figure I - Overexcavation in Area "A"

Area "B" is the old cut slope behind the Hillsborough West Apartments. Reinforced earth slopes are proposed in this area. Based on subsurface data from the supplemental exploration, overexcavation of approximately 15 feet is anticipated to remove the material above the failure plane formed by the 1960s instability. The proposed toe line of the slope is approximately 8 feet away from the property line at its nearest point, and the length of the area over which the toe comes within 15 feet of the property line is approximately 50 feet. Since the slope requiring protection is downhill from the excavation and the excavation will unload the slope, the overall stability of the lower slope should be favorable during the overexcavation. However, the excavation face could experience local slumping. In our opinion, temporary stability of the excavation face should be favorable at an approximate slope of 1:1. In segments where the physical constraints require a steeper slope, the lower portion could be supported by drilled spaced piers. Although these piers are intended for temporary support of the cut, they would provide an additional benefit to the post-construction stability of the overall slope. The slope protection measures for Area "B" are illustrated in Figure J, page 60.

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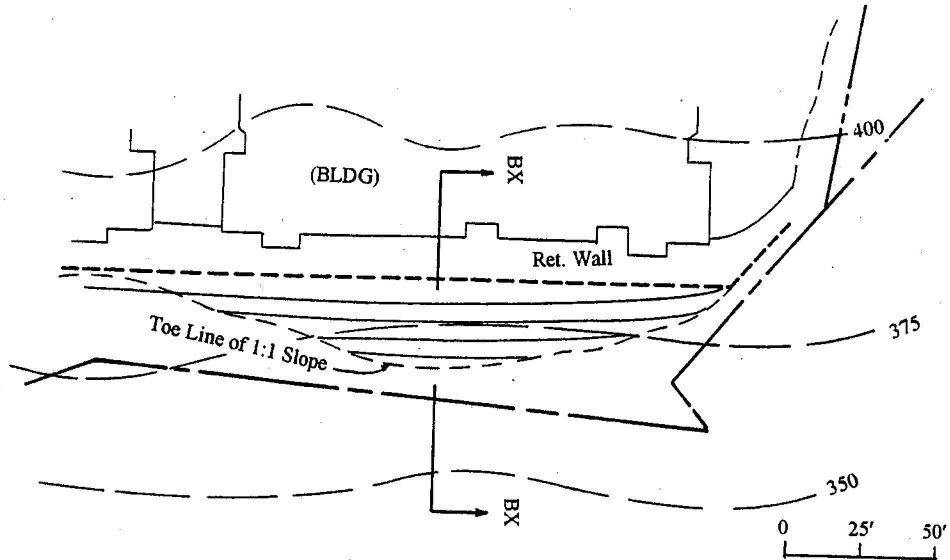
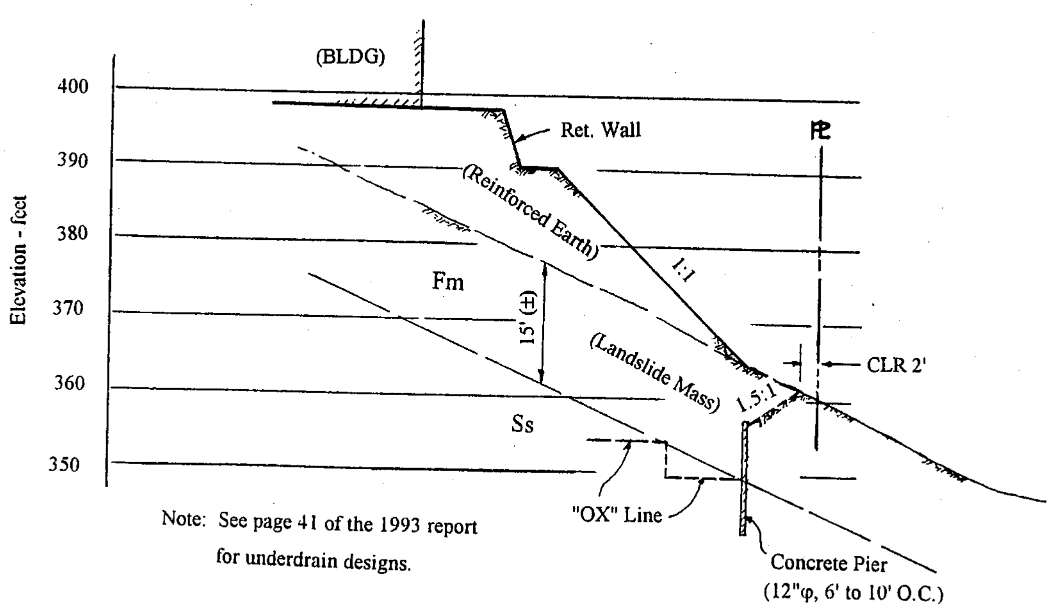
November 7, 1994



PLAN — AREA "B"



SECTION BX

Figure J - Overexcavation in Area "B"

Area "C" encompasses the proposed cut slopes upslope of the northerly portion of the townhome development. Overexcavation is anticipated to remove cobbles and loose colluvium exposed on the cut slope in the upper elevations of Sections GS-1 and GS-2. The comments of ESC suggested that the potential ravelling or slumping of the surficial deposits during overexcavation near the proposed limits of grading work should be addressed and mitigation measures provided.

The isopach map of surface deposits, Plate 4, indicates that the surface deposits over this area vary from 4 feet to 6 feet in thickness. Ravelling or slumping of the surface deposits is likely. However, since the surface deposits are not thick, this occurrence is not expected to extensively undermine the natural slope above the overexcavation. The property boundary of the existing subdivision upslope is more than 200 feet away from the proposed cut slopes. If the excavation experiences extensive slumping of the upper slope, then drilled concrete piers should be used for protection of the cut slope, as in Areas "A" and "B". Figure K, page 62, presents the features discussed.

The anticipated performance of excavations at three critical locations and general alternatives for stabilizing the excavation were discussed above. In settings similar to the above cases, the most effective and practical methods for performing deep excavations are usually determined based on the field conditions encountered during the excavation. It is commonly the responsibility of the contractor to protect the excavation. The measures discussed above are intended to indicate the feasibility of performing the required overexcavations without endangering the adjoining properties.

References Cited for Slope Stability

Felio, Guy Y., 1988. "User's Manual for BISTAT, A Microcomputer Program for Slope Stability Using the Simplified Bishop Method and Stochastic Analysis." Civil Engineering Department, University of California, Los Angeles.

Idriss, I. M., 1991. "Earthquake Ground Motions at Soil Sites." *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. III, pp. 2265-2275.

Ishihara, K., 1985. "Stability of Natural Deposits During Earthquakes." *Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering,* San Francisco, Vol. 1, pp. 255-320.

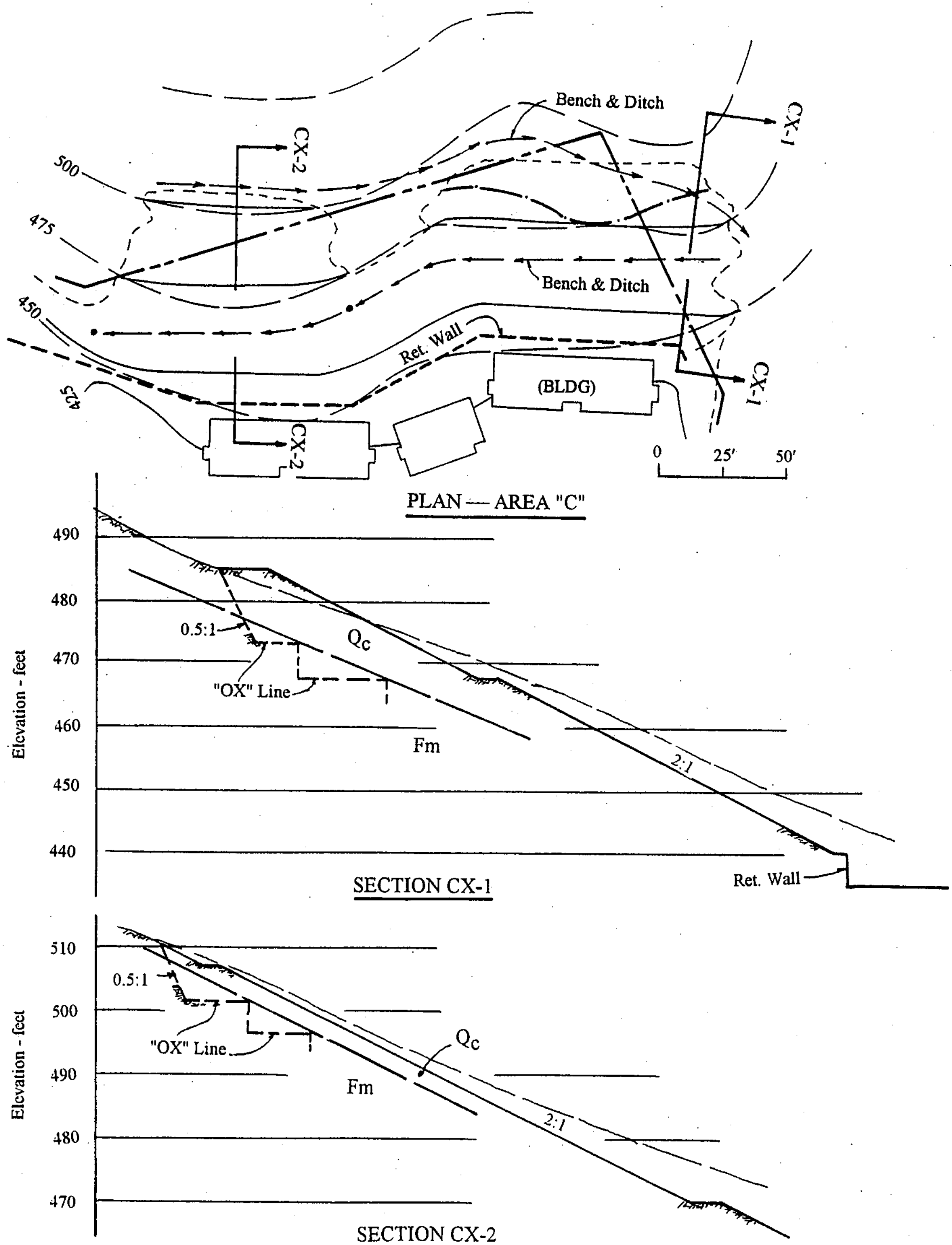


Figure K - Overexcavation in Area "C"

Subsection 4 - Stability of Reinforced Earth and Piers (ESC Comment 18, and HTA Comment 5)

ESC Comment 18, page 8

The reinforced earth walls and slopes may deform during seismic loading. The possibility of deformation of the reinforced earth structures during an earthquake, and the possible impact on adjacent buildings and improvements should be evaluated and discussed by the geotechnical consultant. The report presents several case histories relating to the use of reinforced earth construction. Similar discussions or other types of substantiating information would be useful in evaluating the performance of pier-supported reinforced earth embankments.

HTA Comment 5, page 2

Further documentation of the use of piers under reinforced earth slopes should be provided. Do the piers support the weight of the soil above or do they just provide lateral resistance? If the material underlying the fill settles, will the fill push the piers down or settle around the pier tops, and what would be the effect of either?

Response:

The comments may be broadly divided into the following categories to formulate our responses:

a) Deformation of the reinforced earth walls and slopes, and its effect on adjacent buildings,
b) Action of piers installed to reinforce slopes, and
c) Case histories of the use of piers in slope stabilization.

a) Deformation of Reinforced Earth Structures

Reinforced earth slopes and retaining walls normally experience settlement and lateral deformation, as do any other constructed fill slopes. The previous discussions relating to hydrocompression of fill are equally applicable to reinforced earth slopes. The maximum thickness of reinforced fill is less than 40 feet. As indicated in the previous discussion of settlement, hydrocompression of fills consisting of the native soils is negligible when the fill is less than approximately 40 feet in thickness. As in regular fills, reinforced earth fills would also experience creep settlement. As shown on page C.1.2 of the settlement calculation sheets in Section C.1 of the Appendix, the creep settlement of a 40-foot thick fill is less than one inch over a period of 30 years. This does not represent a significant amount of post-construction settlement.

Reinforced earth construction is a sound, well-accepted technology, as discussed in the 1993 report (pages 32 through 35). In his state-of-the-art lecture entitled "Static Stability and

Deformation Analysis", presented at the 1992 ASCE specialty conference on "Stability and Performance of Slopes and Embankments-II", Professor J. M. Duncan states:

> "Research and field studies during the past 25 years have developed a solid basis for analysis and design of reinforced earth slopes and embankments. Based on the research findings, and experience with field installations, it is possible now to design with confidence slopes and embankments that are reinforced with geotextiles, geogrids, and steel mesh."

As a part of the work for the 1993 report, TENSAR Earth Technologies, Inc., an internationally recognized manufacturer of geogrids for earth reinforcing elements and designer of reinforced earth slopes and retaining walls, developed a preliminary design for the reinforced earth slopes and retaining walls of cross sections GS-1, GS-6 and GS-7. The design summary and supporting calculations (computer printouts) provided by TENSAR are included in Section C.4 of the Appendix. As shown by the TENSAR design summary, both the reinforced slopes and retaining walls take up a large portion of the overall fill slope. Both the reinforced slopes and walls are essentially flexible gravity walls having a large width-to-height ratio, with no rigid footings. The proposed slope stabilization plan calls for placing the reinforced earth section on a wide level base formed by cutting into bedrock or overexcavating the overburden materials to maximize the resistance against lateral translation and to minimize settlement. This stabilization plan also uses drilled piers. The use of drilled piers is intended to improve the global stability of the slope during seismic loading; however, these piers will also provide an interlocking effect between the reinforced section and the natural slope beneath, thus further improving the resistance of the reinforced section against lateral translation. The reinforced section will be designed to attain self-stability against lateral translation and overturning, and the reinforcement will be designed to control the lateral deformation under static and seismic loading within the reinforced section. Plans will be prepared for the reinforced earth slope and retaining wall construction and included in the final grading plan package.

In assessing the effect of deformation of the reinforced section on structures, it is necessary to consider the tolerance of the structure to deformation. For the subject project, streets will be directly supported on top of the reinforced earth walls or slopes, with the exception of the area behind the Hillsborough West Apartments. The majority of the case histories presented in the 1993 report on the use and performance of reinforced earth construction pertained to the support of highways. Pavements are more tolerable to deformation than rigid foundations. The cited case histories amply demonstrated that reinforced earth construction is suitable for the support of pavements. The 1993 geotechnical report recommended drilled piers passing through the reinforced earth sections for the support of the townhome buildings behind the Hillsborough West Apartments. The intent of this recommendation is to minimize the effect of deformations in the reinforced earth sections on the adjacent buildings, in addition to minimizing the structural loads being transmitted to the reinforced earth sections.

b) Action of Piers in Slope Stabilization

The use of drilled piers in the slope is not intended to distribute the weight of the reinforced earth slope or retaining wall into the slope. The purpose of the piers is to increase shearing resistance within the natural slope to a level necessary to attain the desired factor of safety. In essence, the piers are reinforcing elements. In the proposed plan, the piers are located more or less directly below the center of the most critical potential failure circle. Consequently, the pier resistance will be mobilized predominately in the lateral direction, and the vertical component of the pier thrust will be negligible.

Piers proposed for slope reinforcement would be embedded deep into the bedrock. Therefore, if the ground underlying the fill settles, the fill will settle more than the piers. On this basis, it may be argued that the piers will be pushed down (loaded) by the fill and simultaneously the fill will settle around the pier top, causing negative friction on the shaft. Such loading could be important when the piers are intended to provide vertical support. However, these piers are not intended for vertical support. The overburden soil that would settle will be removed in the base key excavation in which the piers will be located. Therefore, the ground settlement around the piers would be small and, consequently, the resulting negative friction on the piers would be insignificant.

In the above discussions, two sources of vertical loads were identified to act on the pier, although they are insignificant. To complete the response on the effect of vertical loads, it should be noted that these loads affect the flexural requirement of laterally loaded piers. As discussed in the 1993 report, piers are introduced in the slope stabilization method to improve the factor of safety of the slope under seismic loading. Since the compression process of the bedrock would be short under the imposed weight of the reinforced earth structures, it is reasonable to ignore the ground settlement leading to the vertical loads discussed above. Therefore, the piers may be treated as being subjected to lateral loads only.

The concept of using drilled piers to reinforce slopes is neither unique nor new. It is derived from the use of drilled piers for landslide stabilization, which dates back over a century (Fukuoka, 1977; Leventhal and Mostyn, 1987). The following discussion provides citations that are relevant to the rationale for pier actions in slope reinforcement, as conceived in the 1993 SFS geotechnical report.

Lateral loads are transmitted to piers when they are used for either slope reinforcing or landslide stabilization. Under these circumstances, the lateral loads on the piers are induced by horizontal deformation or displacement of the soil mass. This type of laterally loaded pier is commonly known as a "passive pier". Drilled piers are also used to support horizontal loads, as in the case of "flag poles". In this case, the imposed lateral load causes the soil mass to deform or displace laterally. Piers subjected to this type of action are commonly known as "active piers". To correctly define the actions of a pier it is important to distinguish between active and passive piers. In the case of active piers, there is no horizontal movement in the soil mass when piers are not present, and the soil resistance against the pier is developed by deflection, rotation and translation of the pier into the soil mass. An example of solutions for active piers is the solution introduced by Matlock and Reese (1960) and Reese (1977), employing the concept of p-y curves.

In the case of slope reinforcement or landslide stabilization, the analysis must determine the limiting pier spacing required to avoid failure of the soil mass between the piers, and the resulting thrust from the moving soil mass that will be transmitted to the piers. Wang and Yen (1974) developed a solution for determination of the pier spacing and the force on the pier for a single row of piers used for landslide stabilization in an infinite slope. Ito and Matsui (1975) introduced solutions based on the theories of plastic deformation and plastic flow for determination of the lateral forces acting on a single row of piers, in terms of the pier spacing and diameter. One of their basic assumptions was a plane strain condition in the vertical direction. Ito, Matsui & Hong (1981), Ito, Matsui & Hong (1982) and Matsui, Hong & Ito (1982) applied the Ito-Matsui solution to actual design problems dealing with passive pier systems and extended the solution to include a multiple-row pier system. However, the basic assumptions introduced in the derivation of the original solution for a single row of piers were carried over into the extension of the solution dealing with a multiple-row system. The Wang-Yen solution is not applicable to the subject slopes because it assumes an infinite slope condition and unrestrained slide movement above the slide plane. The Ito-Matsui assumption of a plane strain condition in the vertical direction is not met for the subject slopes because of the presence of a reinforced earth structure over the soil mass and the increase in stiffness of the soil mass with depth. The boundary conditions of the problem closely resemble a tunnel heading in squeezing ground, or the ground below lagging in deep excavations supported by a soldier pile-lagging system. Solutions are available for these situations that estimate the thrust in terms of the size of the opening, the overburden pressure and shear strength of the soil (Sohn and Sohn, 1990, 1993; Davis, 1968).

The boundary condition of the problem can be more simply, yet quite adequately represented by a rigid plastic wedge enclosed by the plane containing the base of the reinforced earth at the top, the critical circle plane at the base and the vertical plane containing the first row of piers. The solution derived for this boundary condition is presented on calculation sheet page C.4.37 in Section C.4 of the Appendix. The curved surface of the wedge was assumed to be planar in the derivation. This is a conservative assumption, because the curved surface of the wedge base provides a larger cross sectional area of the wedge. Further, friction or shearing resistance on the planes forming the top and base of the wedge was neglected; shearing resistance was assumed to be active only on the two cross sections at the inner edge of two adjacent piers. The required pier spacing using the conditions described above for Slope GS-7b, which requires the largest stabilizing force, are presented below:

Pier Diameter, d, feet:	0.5	1.0	1.5	2.0
Edge-to-Edge Spacing in Each Row:	18.2d	9.6d	6.8d	5.3d

The above pier sizes are most commonly used in slope stabilization work. The maximum edge-to-edge spacing of five pier diameters recommended in the 1993 report was based on these results and consideration of the various solutions discussed above. Our 1993 recommendation is clearly conservative.

Determination of the structural capacity of passive piers is complicated by the difficulty of including the effect of the ground displacement that would occur in the absence of the pier on the pier-soil interaction (De Beer, 1977). Fukuoka (1977) introduced solutions for piers subject to forces from a moving landslide mass by modelling the ground above and below the slide plane in terms of the coefficient of horizontal subgrade reaction. The Fukuoka solutions do not explicitly consider the effect of ground displacement on the soil-pier interaction. Viggiani (1981) also developed solutions for piers subject to lateral loads from moving landslide masses. Viggiani's solutions are based on the concept of a yield value of pier-soil interaction. The yield value is established in terms of a bearing capacity factor, which is adjusted to account for the effect of the ground displacement. The Viggiani solutions are based on various assumed failure modes of the pier-soil system and are limited to cohesive soils. Bowles (1988) presented a numerical analysis of a laterally loaded pile by treating it as a beam resting on an elastic foundation characterized by the coefficient of horizontal subgrade reaction. The coefficient of horizontal subgrade reaction in the Bowles solution is derived in terms of the bearing capacity factors for rigid foundations. The Bowles analysis utilizes the finite element method, and his source program is available in the cited reference. The Reese solution (1977) for laterally loaded piles also treats the pile as a linearly elastic beam resting on an elastic medium. The soil resistance against the laterally loaded pile is represented by a line load whose variation along the pile is described by a set of p-y curves. To solve the pile problem it is necessary to predict a set of p-y curves. A suggested procedure for establishing p-y curves in stiff clay is available (Reese and Welch, 1975), and will be given further treatment in Section D of this report. The solutions discussed above essentially treat the pier as an active pier interacting with the ground.

As discussed above, rigorous solutions currently available for laterally loaded piers have limitations when applied to passive piers. The most critical limitation is that the solutions overestimate the capacity of passive piers. In the 1993 report, this factor led to modelling the multiple-row pier-soil system as an integral unit undergoing direct shear. This type of failure mode was recognized for a multiple-row pier system in slope stabilization as early as 1936 (see Fukuoka, 1977). The direct shear model is a simplistic representation of the multiple-row pier-soil system. The pier design should be based on this model when the model provides more conservative results than other rigorous solutions. The Bowles solution was selected for this comparison. The coefficient of horizontal subgrade reaction used in the analysis was determined based on the method suggested by Bowles, and is expressed by the following equation:

$$k_s = 790 + 16d + 110z + 845q_s \qquad [6]$$

where, k_s: coefficient of horizontal subgrade reaction, k.c.f.,
d: diameter of pier, feet,
z: depth to the potential failure plane below the original ground surface, feet, and
q_s: surcharge at the original ground surface due to the reinforced earth, k.s.f.

Using Bowles finite element source program, pier loads that would result from the required stabilizing force for Section GS-7b were computed for pier sizes of 6 inches, 8 inches, 12 inches and 18 inches. Of the pier loads computed by this method, bending moment governs the design. Therefore, the amount of reinforcing steel required for the maximum bending moment was determined by treating the pier as a reinforced concrete pier. This steel amount was compared with the steel required for the direct shear model. Table S-V (below) presents this comparison in terms of the ratio of steel required for the direct shear model to that required for bending moment.

TABLE S-V

Pier Size	Number of Rows						
	4	5	6	7	8	10	15
6"	-	1.13	-	-	-	1.11	1.08
8"	-	1.23	-	-	-	1.16	1.18
12"	-	1.21	1.21	1.20	1.18	1.16	-
18"	1.28	-	1.24	-	1.20	-	1.17

The comparisons shown above indicate that the direct shear model requires an approximately 10% to 25% heavier pier design than the rigorous solution of Bowles. Given the limitations of the various rigorous solutions, this conservatism is considered to be appropriate. Therefore, it is our conclusion that the basis for the criteria recommended in the 1993 report for determination of pier capacity is reasonable. Our recommended geotechnical criteria for use in the rigorous solution of Reese or Bowles are summarized in Section D of this report.

All pertinent computations, including the finite element analysis computer runs in support of the pier capacity comparisons shown above, are included in Section C.4 of the Appendix.

c) Case Histories of Use of Piers in Slope Stabilization

ESC requested case histories relating to the performance of pier-supported reinforced earth embankments. As stated earlier, the piers are intended to reinforce the slope but not to support the reinforced earth structures. Several case histories are presented below where piers were used to reinforce or stabilize slopes. In each of these projects SFS investigated the site and provided the geotechnical design analysis.

- Slope on Celeo Lane, Santa Clara County, CA

Celeo Lane is located above Hamilton Road in the east San Jose foothills. Grading for a row of residential lots along the lower side of Celeo Lane involved fill slopes of more than 20 feet in height, constructed on sloping natural ground. Large movements of the slope had occurred on one lot, causing distress to the house and cracking in the driveway slabs. SFS investigated stability of the slope on this lot and designed a shear-pin system consisting of drilled piers to stabilize the slope. The owner planned to build a wooden deck extending over the slope. Except for the limited number of piers required for support of the deck, reinforcement in the piers was limited to within several feet of the slide plane. The piers were installed in multiple-rows and staggered across the entire width of the lot. The work was done in the mid-1970s. Slopes on several lots on either side of this lot were involved in sliding during the 1981-1982 storms and the homes on these lots were completely destroyed. The lot that had been stabilized with the shear-pin system was the lone survivor of this disaster. The engineering work for this slope was cited by Channel 11, a local television station in San Jose, as an example of engineering excellence during their telecast of the slope failure.

- Monterey Bay Estates Percolation Pond, Marina, CA

An approximately 25-foot high slope was required for construction of a percolation pond within spatial constraints. Two parallel retaining walls of 5 to 6 feet in height were proposed to provide acceptable slope ratios between, above and below the walls. However, the global stability of the slope-wall system was considered to be marginal. Drilled piers, spaced 6 feet on centers, were used to improve the stability. The site is underlain by dune sand deposits. The work was completed three weeks before the Loma Prieta earthquake, and the slope performed satisfactorily through this seismic event.

- Stonegate Ridge Unit 5, South San Francisco, CA

Development was proposed in the proximity of the property boundary. A landslide crossed the property boundary and extended some distance over the downhill slope of the adjoining property. Removal of the slide debris within the project site was impractical without encroaching upon the adjoining property. A system of drilled piers was used to contain the slide mass within the project site. The construction was completed before the Loma Prieta earthquake, and the slope performance has been satisfactory.

- Slope Stabilization at Sugarloaf Condominium Project, San Mateo, CA

Parking lot construction at the higher elevations of a ridge slope required stabilization of a landslide below. The proposed toe of the slope was located near the property line. Complete removal of the slide debris at and near the toe was not practical due to spatial constraints. Drilled piers were used to reinforce the toe section of the slope. This work was completed in the early part of 1990, and the slope has been performing satisfactorily.

- Reported Case Histories in Literature

Landslide stabilization and slope reinforcement using drilled piers has been extensively reported in engineering literature. Over the years these case histories have contributed to a better understanding of the factors affecting the soil-pier system, and led to the development of improved and tested design technologies. For example, Pearlman et al. (1992) cite two recent case histories where small diameter, (5 to 9 inches in diameter) slender piers installed in multiple-row patterns were an effective and practical method for landslide stabilization. As discussed earlier, the studies for this supplemental report included small size piers, in the range of 6 to 8 inches in diameter. It was found that these small piers, reinforced with bundles of 2 to 3 #9 re-bars or a single steel pipe, could be used in most cases for the proposed slope reinforcing within the subject project.

References Cited for Reinforced Earth and Piers:

Bowles, Joseph E., 1988. *Foundation Analysis and Design, 4/e*. McGraw-Hill Book Co., NY.

Davis, E. H., M. J. Gunn, R. J. Mair, and H. N. Seneviratne, 1980. "The Stability of Shallow Tunnels and Underground Openings in Cohesive Material." *Geotechnique*, Vol. 30, No. 4, pp. 397-416.

De Beer, E., 1977. "Piles Subjected to Static Lateral Loads." State-of-the-Art Report, *Proceedings*, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan.

De Beer, E and R. Carpenter, 1977. "Discussion on Methods to Estimate Lateral Force Acting on Stabilizing Piles; Tomio Ito and Tamotsu Matsui." *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 17, No. 1, pp. 68-82.

Duncan, James M., 1992. "Slope Stabilization Using In Situ Earth Reinforcements." *Proceedings*, Specialty Conference on Stability and Performance of Slopes and Embankments-II, ASCE, Vol. 2.

Fukuoka, M., 1977. "The Effects of Horizontal Loads on Piles Due to Landslides." State-of-the-Art Report, *Proceedings*, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan.

Ito, Tomio and Tamotsu Matsui, 1975. "Methods to Estimate Lateral Force Acting on Stabilizing Piles." *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 15, No. 4, pp. 43-59.

Ito, Tomio and Tamotsu Matsui, 1978. "Discussion Closure on Methods to Estimate Lateral Force Acting on Stabilizing Piles." *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 18, No. 2, pp. 41-44.

Ito, Tomio, Tamotsu Matsui, and Won Pyo Hong, 1981. "Design Method for Stabilizing Piles Against Landslide — One Row of Piles." *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 21, No. 1.

Ito, Tomio, Tamotsu Matsui, and Won Pyo Hong, 1982. "Extended Design Method for Multi-Row Stabilizing Piles Against Landslide." *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 22, No. 1.

Leventhal, Andrew R. and Garry R. Mostyn, 1987. "Slope Stabilization Techniques and Their Application." *Proceedings*, Extension Course on Slope Stability and Stabilization, Sydney, Australia, pp. 183-230.

Matlock, H. and L. C. Reese, 1960. "Generalized Solutions for Laterally Loaded Piles." *Journal of Geotechnical Engineering*, ASCE, Vol. 86, SM 5, Oct., pp. 63-91.

Matsui, Tamotsu, Won Pyo Hong and Tomio Ito, 1982. "Earth Pressures on Piles in a Row Due to Lateral Soil Movements." *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 22, No. 2.

Pearlman, Seth L., Bradley D. Campbell and James L. Withiam, 1992. "Slope Stabilization Using In Situ Earth Reinforcements." *Proceedings*, Specialty Conference on Stability and Performance of Slopes and Embankments-II, ASCE, Vol. 2, pp. 1333-1348.

Reese, L. C., 1977. "Laterally Loaded Piles: Program Documentation." *Journal of Geotechnical Engineering*, ASCE, Vol. 103, No. GT4, pp. 289-305.

Reese, L. C. and K. C. Welch, 1975. "Lateral Loading of Deep Foundations in Stiff Clay." *Journal of Geotechnical Engineering*, ASCE, Vol. 101, No. GT7, pp. 633-649.

Sohn, J. and K. C. Sohn, 1990. "Prediction of Soil Behavior and Face Stability in Deep Excavations Supported by Soldier Pile Walls." *Proceedings*, First Korea-Japan Joint Geotechnical Seminar on Excavation and Tunneling in Urban Areas, Seoul, Korea, pp. 145-161.

Sohn, K. C. and J. Sohn, 1993. "Stability of Tunnel Working Face in Squeezing Ground." *Proceedings*, National Conference on Geotechnical Engineering and Tunneling Technology, Korean Geotechnical Society, Seoul, Korea, pp. 9-14.

TENSAR Earth Technologies, Inc., 1993. "Preliminary Design, Highland Terrace Project, San Mateo County, California, Project No. S93204."

Wang W. L. and B. C. Yen, 1974. "Soil Arching in Slopes." ASCE, *Journal of the Geotechnical Engineering Division*, Vol. 100, GT1, pp. 67-78.

Viggiani, C., 1981. "Ultimate Lateral Load on Piles Used to Stabilize Landslides." *Proceedings*, Tenth International Conference on Soil Mechanics and Foundation Engineering, Vol. 3.

Subsection 5 - Miscellaneous Items (ESC Comment 15, page 7 & HTA Comment 1, page 1)

ESC Comment 15, page 1

The SFS report states (page 36) that the "...colluvium will be removed during the grading operation and used in engineered fill." The distribution, thickness and characteristics of the colluvium must be determined as accurately as possible to allow the project Civil Engineer to estimate earthwork quantities, including areas to be undercut and reworked where colluvium is present. The report should address the implications of reworking significant quantities of colluvium and the possible impact on the earthwork operations and earthwork quantities. Will the over-size cobbles and boulders found within the colluvium be suitable for use in engineered fill? The report recommends (page 50) the use of on-site materials having a friction angle of 30 degrees and a unit weight of 130 pcf for reinforced earth embankments. Table II, page 28, indicates that compacted fill will have a friction angle of 20 degrees. Are their sufficient quantities of suitable materials on-site to produce fill for the reinforced earth? If not, what alternatives are recommended to achieve the specified stability.

HTA Comment 1, page 1

Brian Kangus Foulk (BKF) is not clear whether their earthwork quantities include overexcavation of colluvial materials as recommended in the Soil Foundation Systems, Inc. (SFS) report. BKF states that no allowance was made for shrinkage, whereas SFS recommends that an average shrinkage factor of five percent be used. Fill shrinkage could have a large impact on earthwork balance, and error in either cut or fill quantities could result in the hauling of materials onto or off the site.

Response:

Both comments are concerned with the anticipated earthwork quantities for site grading, including the required overexcavation for removal of colluvium. Brian Kangas Foulk presented estimated earthwork quantities for site grading, excluding the colluvium overexcavation, in their submittal to the County, dated June 28, 1993. Their computation of earthwork quantities does not include shrinkage of the material due to compaction. It was and still is our conclusion that the shallow bedrock condition in the proposed cut areas would result in little or no shrinkage. Based on this conclusion, the BKF earthwork computation was performed before the 1993 report was issued. However, in view of the fact that an excess of material would be better than a shortage, a shrinkage factor of 5% was recommended in the 1993 report, with a provision to designate an area for dirt balancing.

Using the dirt quantity computation provided by BKF in June 1993, and a shrinkage factor of 5%, there would be a shortage of 8,300 yards in the as-placed volume. It is expected that BKF would make the necessary dirt quantity adjustment in the final construction drawing phase to balance the dirt quantities.

This response report includes an isopach map showing the distribution and thickness of the colluvium. When the isopach map data is applied to the proposed grading plan, the average thickness of the colluvium to be removed is less than 3 feet, and the colluvium removal is limited only to small areas at four locations. The isopach map has been provided to BKF for the purpose of estimating the quantity of colluvium to be overexcavated.

Removal of colluvium is required in cut areas and on natural slopes in the immediate vicinity of the top edge of cut slopes. As discussed earlier in response to ESC Geology Comment 7, much of the colluvium would be removed as a part of the required cuts, and the remaining colluvium is not significant in depth. Overexcavation and restoration of cut slopes is routinely done in hillside grading, and such operations are not a major undertaking when performed during mass grading. Locally the colluvium is cobbly. The cobbles can be crushed with heavy bulldozers and compactors, as was confirmed by backhoe trenching and auger drilling. The crushed cobbles, when mixed with on-site soils, is an excellent source of fill material. Numerous trenches excavated by Berlogar-Long as well as our trenches and auger borings, located in the proposed major cut area over the southern ridge of the townhome site, did not encounter boulders. However, the proposed cuts may produce some oversize boulders in certain locales. These boulders may be used in deep fills provided that the boulders are placed with a sufficient spacing between them to allow passage of the compaction equipment. Detailed recommendations for handling oversize boulders in deep fills are presented in the 1993 report.

For the previous investigation, four samples were collected from the proposed cut areas and direct shear tests were performed on specimens fabricated from these samples. Results of these direct shear tests were presented in Table IV, page A38 of the 1993 report. Friction angles from these test results were all greater than 30 degrees. A unit weight of 130 p.c.f. in wet density is approximately equivalent to 92% relative compaction. Therefore, the materials generated from the proposed cut areas should be suitable for construction of reinforced earth structures.

SECTION D — SUMMARY OF ALTERNATIVE MITIGATION MEASURES

The ultimate purpose of the 1993 report and this supplemental report was identification of potential hazards and determination of alternative measures to adequately mitigate the hazards. Geotechnical hazard mitigation measures encompass two broad areas: settlement and slope stability.

Settlement Mitigation Measures

Settlement mitigation measures are divided into two categories; one aimed at reducing settlement and the other aimed at minimizing the effect of settlement on structures.

1. Various recommendations were provided in the 1993 report for reduction of settlements. These included seepage control measures (Section 5.4), earthwork requirements for site grading (Section 5.5), surface drainage and erosion control measures (Section 5.10) and utility trench backfill requirements (Section 5.11), among others. In addition to the above, the following measures are recommended for the reduction of settlements:

a) The alluvium deposits in the drainage swale behind the Hillsborough West Apartments shall be overexcavated to a minimum depth of 10 feet.

b) All compacted fills shall be placed at moisture contents not less than 1% wet of the optimum moisture content as determined by ASTM test method D1557-90.

2. The effect of differential settlement on buildings is viewed from two criteria; one concerned with structural stability and the other involving functionality. Excessive differential settlement results in structural distress, in the form of cracking in walls or distortion in parts of the frame. The anticipated maximum differential settlement for the townhomes is not excessive in this regard. Differential settlement can also result in tilting of a building that is structurally stable. This consequence may cause functional discomfort to some occupants, even though the effect may be insignificant. It is possible to reduce or control the differential settlement effects on the townhomes.

a) As a means of reducing the effect of post-construction differential settlement on buildings, the 1993 report recommended delaying foundation construction at least through the first winter following completion of the site grading in areas where the fill is more than 10 feet thick. In this regard, scheduling the construction sequence to allow for longer periods before commencing the foundation work in critical areas would further reduce the post-construction settlement.

b) In the current plan several townhomes are critically located in terms of the variable fill thickness causing differential settlement. These buildings are designated as "F", "G", "H" and "I" in Figure D, page 37 of this report. Differential settlements beneath these buildings could be further reduced by relocating the buildings to areas underlain by smaller variations in fill thickness. Also, the effect of the settlement would be reduced if the multiple unit buildings were structurally separated into smaller buildings.

c) The effects of differential settlement on the buildings listed above can be further controlled by structurally stiffening the foundation grade beams and the connected retaining walls to withstand the differential settlements. For this purpose, the graphs presented in Figures F and G may be used to obtain the design values for differential settlement between various parts of the building.

d) The foundation construction considered in the 1993 report for the townhomes was not intended as a means of eliminating differential settlements. When more complete control of the differential settlement effects is desired, deep piers may be used. These piers should penetrate the fill and be embedded at least 5 feet into bedrock. These piers would be subjected to negative friction due to the relative movement between the pier and the subsiding fill. The following geotechnical criteria are recommended for the design of these piers:

Allowable Bearing Capacity: $q_a = 5500 + 250\ D$, p.s.f. [7a]

Negative skin friction: $F_{ns} = 15\ d\ D_f^2$, pounds [7b]

where, D : pier embedment, feet
D_f : fill thickness, feet
d : pier diameter, feet.

Slope Stabilization Measures

The 1993 report identified several slopes whose stability is considered marginal during intense ground shaking due to earthquakes, and alternative measures were recommended in that report to improve stability of these slopes. As with settlement mitigation, many recommendations were given to improve long-term stability of all slopes. In the 1993 report, these included slope designs (Section 5.2), seepage control (Section 5.4), earthwork requirements for site grading (Section 5.6), surface drainage control and proper maintenance of slopes (Section 5.10), and special slope stabilization measures, among others.

There are two instances where slope stabilization measures are required. One concerns the long-term stability of the marginally stable slopes. The other concerns protection of the adjacent properties during overexcavation of the natural slopes along the property boundary. In light of new findings from the supplemental exploration and the extensive discussions presented in both the 1993 report and this supplemental report concerning slope stabilization measures, the following summary of recommendations is provided:

Long-Term Stability of Slopes

1. Discussions and recommendations for alternative slope stabilization measures are presented in Section 4.6.5 (pages 39 through 46) and Section 5.3 (pages 51 through 53) in the 1993 report. Slope stabilization will utilize overexcavation of the overburden soil and/or weak bedrock materials to competent bedrock in all cases. The slopes will be reconstructed with either compacted fill or reinforced earth. The 1993 report determined several slopes in the townhome area that require additional reinforcing. For these slopes, the 1993 report recommended the use of drilled concrete piers in multiple-row arrangements.

2. Although the basic stabilization measures remain the same, the results of the supplemental exploration indicate that the extent of overexcavation for slope GS-1 behind the Hillsborough West Apartments would be greater than that shown on page 41 of the 1993 report. The actual overexcavation for the area represented by Slope GS-1 shall extend into the stable bedrock below all existing or potential slide planes, which will be located by the project geotechnical engineer during overexcavation of the slope.

3. Structural requirements of piers in multiple-row arrangements shall be analyzed by the three methods presented below. In view of the limitations inherent in each of the methods, it is recommended that the pier design be based on the heaviest requirements from these analyses.

a) Treat piers as being subjected to a direct shearing at all depths for mobilizing the required stabilizing forces presented on page 52 of the 1993 report. The 1993 report should be referred to for further details concerning the use of this method.

b) The Bowles method (1988), or any modification thereof, with the coefficient of horizontal subgrade reaction determined from the following:

$$k_s = 790 + 16d + 110z + 845q_s \qquad [6]$$

where, k_s: coefficient of horizontal subgrade reaction, k.c.f.,
d: diameter of pier, feet,
z: depth to the potential failure plane below original ground surface, feet, and
q_s: surcharge at the original ground surface due to the reinforced earth, k.s.f.

c) The Reese method (1977), or any modification thereof, may be used. The following p-y curves are recommended for the strength characteristics of the deposits into which the pier will be embedded:

$$E_S = \frac{p}{y} = \frac{k_s d}{1.6}, \quad \text{for } y \geq 0.2d \qquad [8]$$

where,
- p: soil reaction, pounds per foot,
- y: pier deflection, feet
- d: pier diameter, feet, and
- k_S: coefficient of horizontal subgrade reaction from Eq. 6, k.c.f.

4. Based on the results of the supplemental subsurface exploration, it is concluded that concrete piers recommended in the 1993 report to reinforce the slope GS-10 in the townhome area (pages 46 and 52) and the slope in the swale area on Lot 2 (page 52) are not required.

5. As elaborated and recommended in Section 5.12 (page 61) of the 1993 report, supplemental geotechnical investigation should be performed during the grading operations when the site becomes readily accessible to exploratory equipment.

Protection of Adjacent Properties During Overexcavation

6. Three critical areas were previously identified on Figure H (page 53) that would require special attention to protect the adjacent properties during overexcavation of the natural slopes along the property boundary. Several alternatives that could be implemented for protection of the adjacent properties were generally described on Figures I through K. Overexcavation of the natural slopes at these three locations should be performed in close consultation with the project geotechnical engineer to ensure that the adjacent properties are not adversely affected during the overexcavation. The contractor shall submit a complete plan to the project geotechnical engineer for review prior to commencing the overexcavation, outlining the sequence of the excavation and any and all slope bracing measures to be provided.

Authority of Supplemental Recommendations

All supplemental recommendations presented in this report are subject to the same limitations and conditions of the 1993 report as stated in Section 6, page 62, of said report.

In the event of conflicting recommendations in the 1993 report and this supplemental report, the recommendations given in the supplemental report shall govern.

File No. S24-634-2S
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APPENDIX

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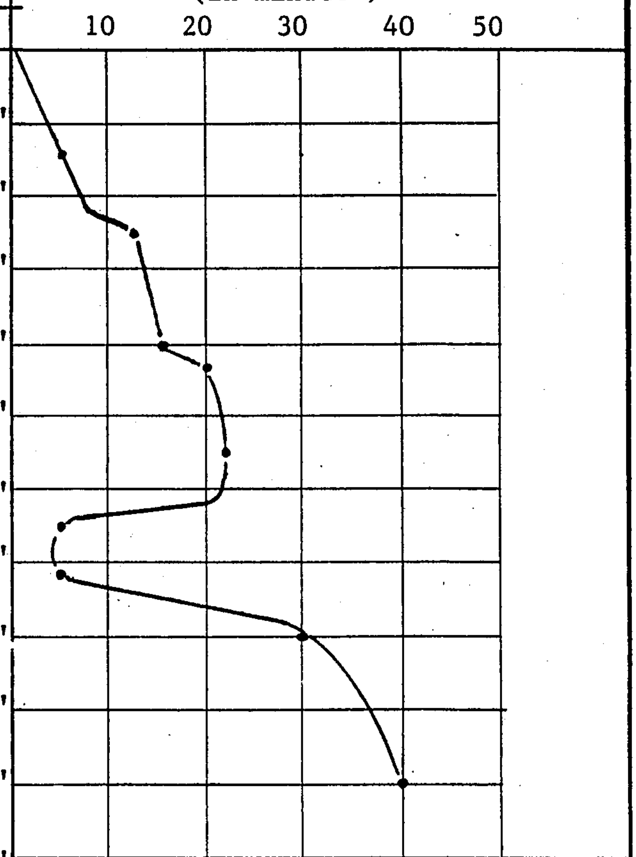
EXPLORATORY BORING LOG

Location: Site #1
Boring No. BS-1
Date Drilled: 12/3/93
Equipment: Minuteman; 3" diameter continuour flight augers

Sample Number	Depth, feet	Boring Log	Unified Soil Classification System Symbols	Description	Standard Penetration Test, blows/foot
	2, 4			Colluvium: clayey Silt with sandstone fragments, light brown (Qc)	
	6			Siltstone breccia, gray-green, firm, dry (Fm)	
	8, 10, 12			Siltstone/sandstone breccia, tan to medium brown, very firm, dry; (Fm) uniform resistance to drilling	
	14			very easy drilling from 13' to 15'; rock fragments	
	16, 18, 20			Sandstone/siltstone; gray-brown, very hard, fine grained, dry	
				Bottom at 21 feet	

Drilling Time per 12" Advance

(in minutes)



Notes: Small amount of water was periodically poured into the hole to facilitate removal of cuttings.

Plate A - Log of Test Boring, BS-1

SOIL FOUNDATION SYSTEMS, INC.

File No. S24-634-2S
November 7, 1994

EXPLORATORY BORING LOG

Location: Site #3
Boring No. BS-2
Date of Drilling: 12/3/93
Equipment: Minuteman; 3" diameter continuous flight augers

Drilling Time
12-inch Advance - minutes

Sample Number	Depth, feet	Boring Log	Unified Soil Classification System Symbols	Description	Standard Penetration Test, blows/foot
	2			Colluvium: dark brown clayey Silt, sandy (Qc)	
	4 6			Siltstone; light brown, dry, firm (Fm)	
	8 10			Sandstone; fine grained, dry, fairly hard (Ss) becoming very hard at 9'	
				Bottom at 11 feet	

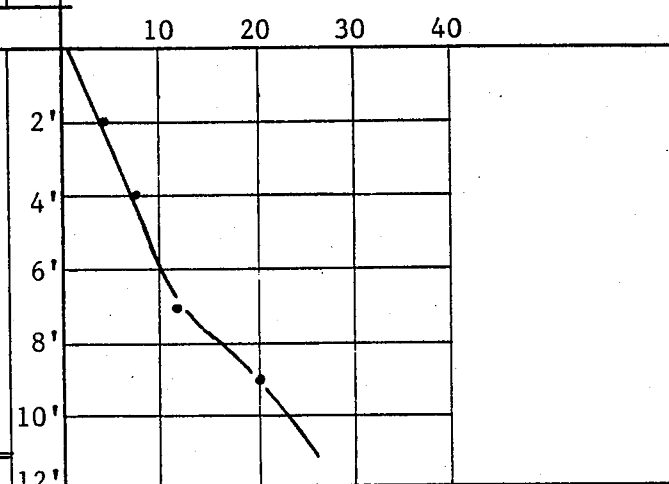
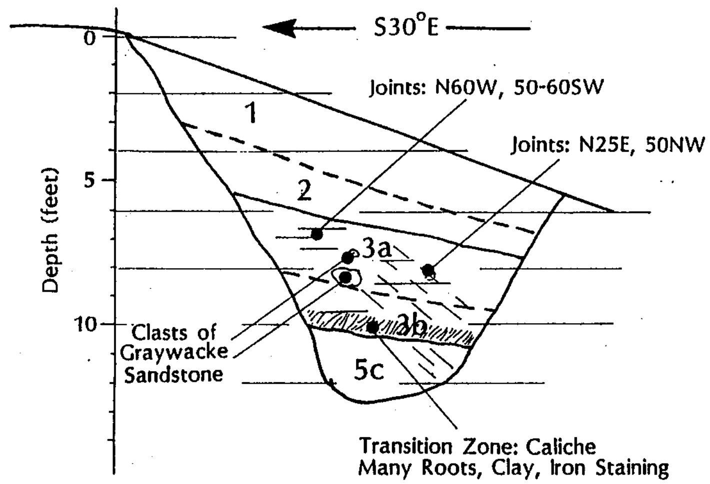


Plate B - Log of Test Boring, BS-2

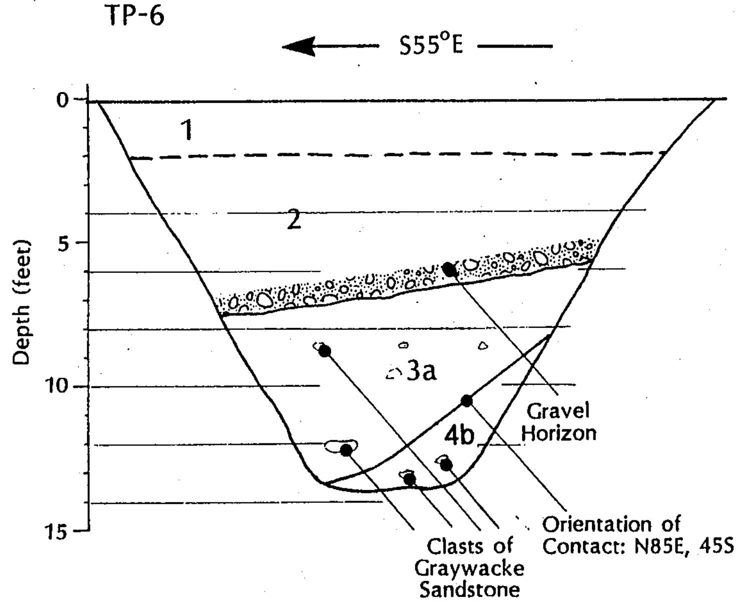
SOIL FOUNDATION SYSTEMS, INC.

File No. S24-634-2S
November 7, 1994

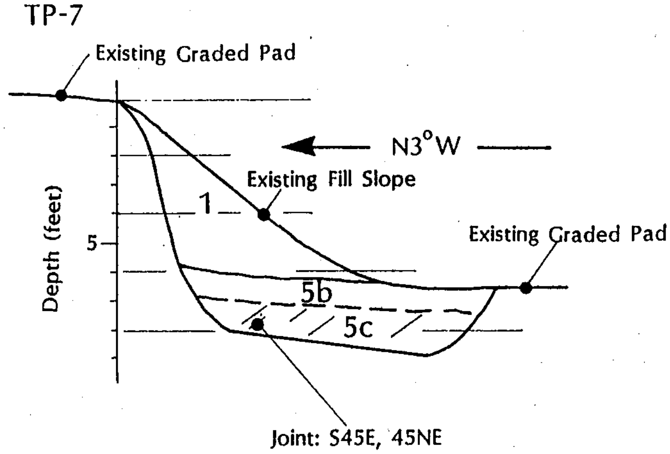
TP-5



TP-6



TP-7



Lithological Units

1 Fill. Clayey sand with angular clasts of graywacke; dark yellowish brown (10YR4/2) damp, many roots; minor disseminated caliche, chunks of glass, loose to firm.

2 Colluvium. Clayey, sandy silt with graywacke inclusions up to 18″ in maximum dimension. Unconsolidated and unsorted; moderate yellowish brown, damp; many roots; caliche staining at basal contact; firm.

3a Sheared Siltstone/Shale. With graywacke inclusions; severely weathered; dark yellowish brown (10YR4/2), many roots, minor caliche, moist to slightly moist, stiff to very stiff. Small partings dip in direction of slope at 25° ±5°.

3b Sheared Siltstone/Shale. With graywacke inclusions, moderately weathered, grayish red purple (5RP4/2), many roots, minor caliche; slightly moist, stiff to very stiff.

4b Sandstone. Clayey, moderately to severely weathered; light yellowish brown (6/4) arkostic (dull feldspars weathering to clay; mica flakes?) slightly moist, friable, medium compact to compact.

5b Graywacke Sandstone. Moderately weathered; dark yellowish brown (10YR4/2), damp, roots, iron staining and minor caliche, fine- to medium-grained, compact,(hard to rip).

5c Graywacke Sandstone. Slightly weathered, light olive gray (5Y 5/2), with dark yellowish brown (10YR4/2) staining on joint planes; joints typically spaced at 6″ to 12″; damp, massive, very compact, (very hard to rip).

Symbols

Geologic Contact (dashed where approximate)

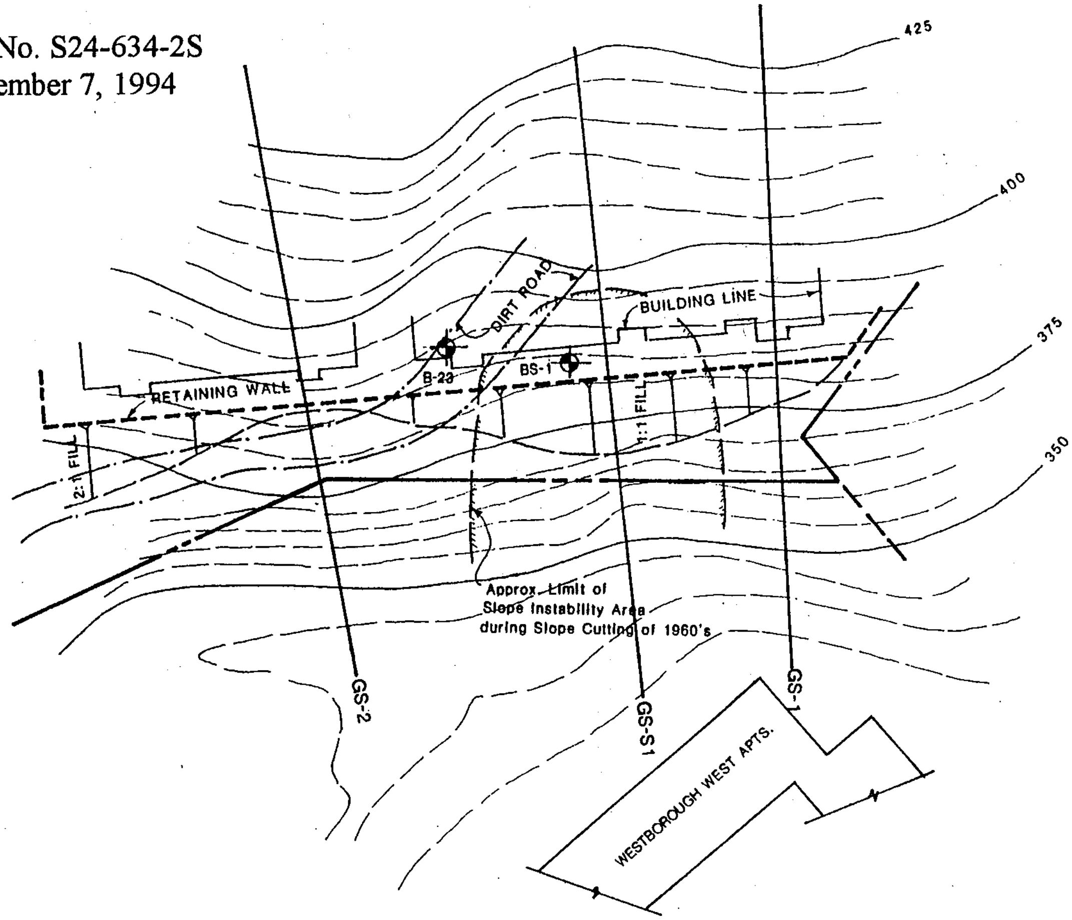
Gravel Horizon

Joint Set

Test pits logged by Darwin Myers, CEG 946, on 3 December 1993

Plate C - Log of Test Pits TP-5 Through TP-7

SOIL FOUNDATION SYSTEMS, INC.



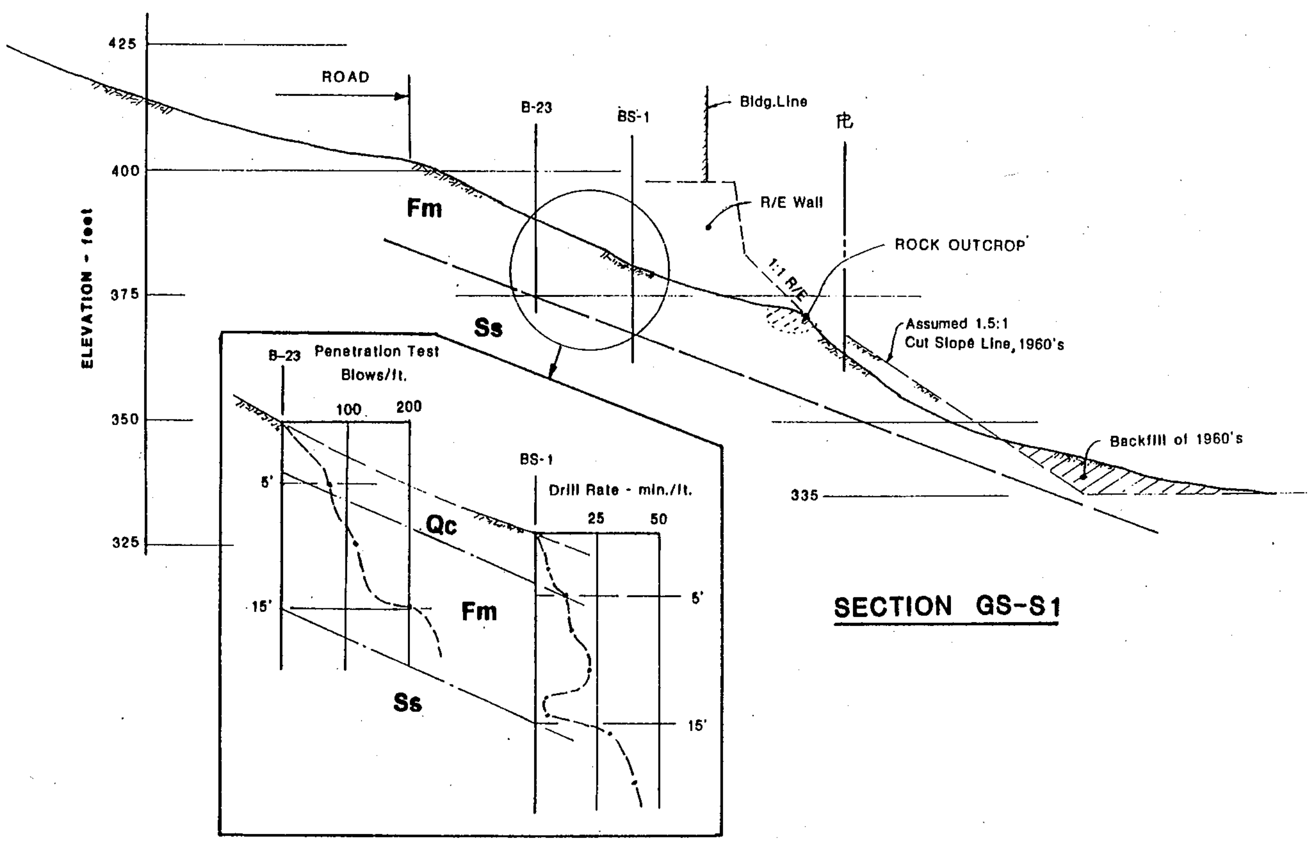
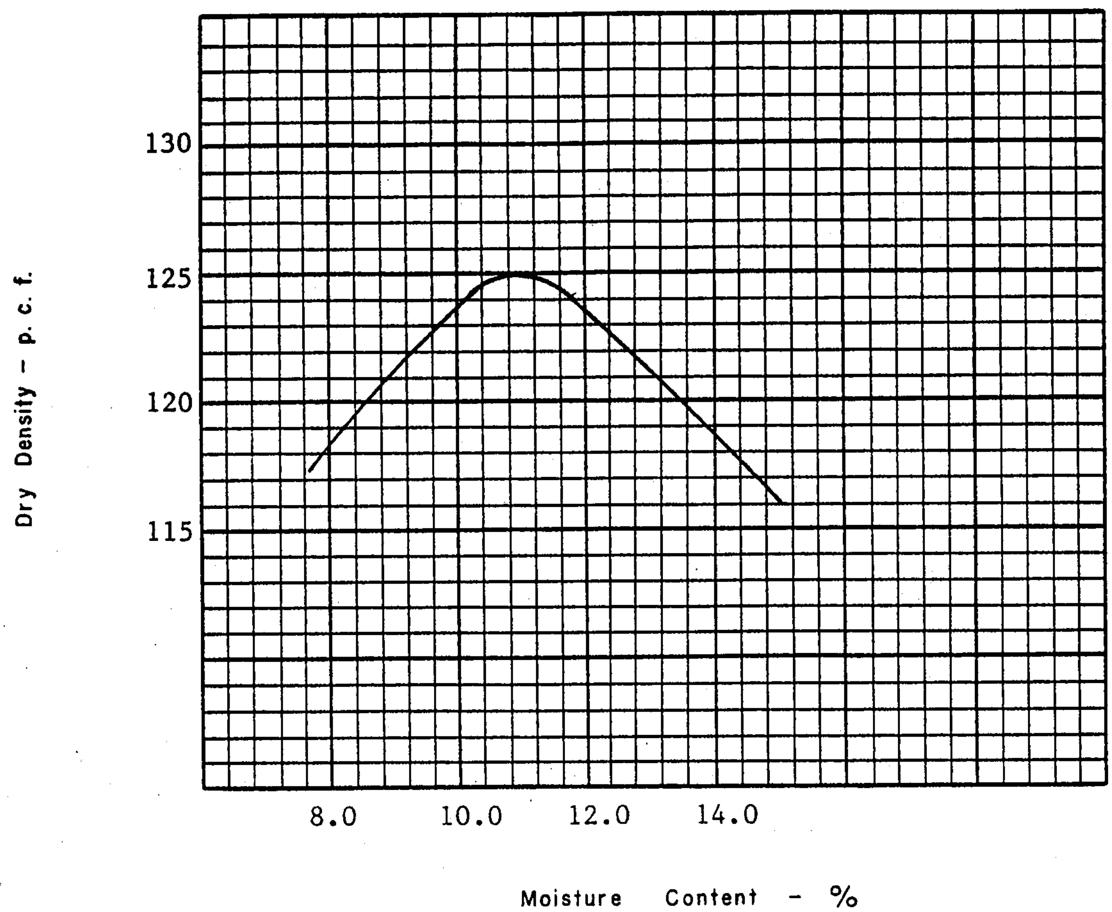


Plate D - Area of Previous Slope Instability

SOIL FOUNDATION SYSTEMS, INC.

LABORATORY COMPACTION TESTS



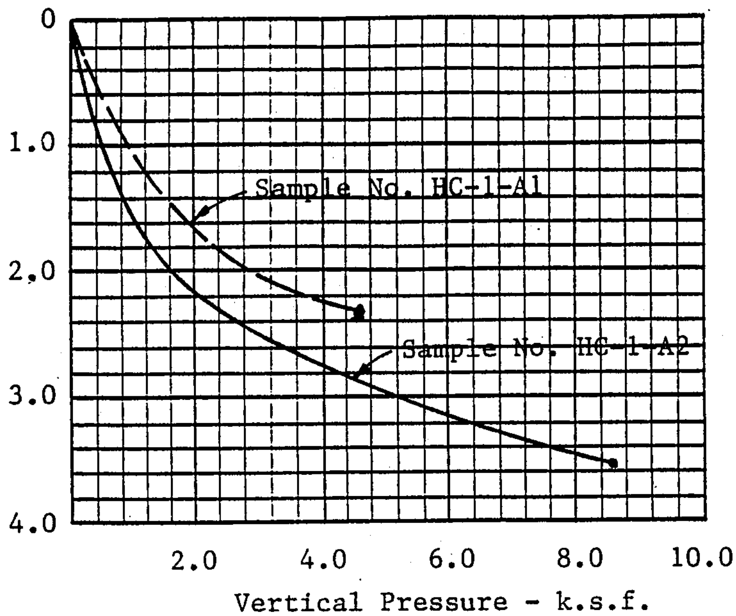
Sample Number	HC-1		
Description	Mixture of tan brown weathered Sandstone and melonge		
Test Procedure	ASTM D1557-90		
Max. Dry Density, p. c. f.	125.0		
Opt. Moisture Content, %	11.0		

Plate F - Results of Laboratory Compaction Test

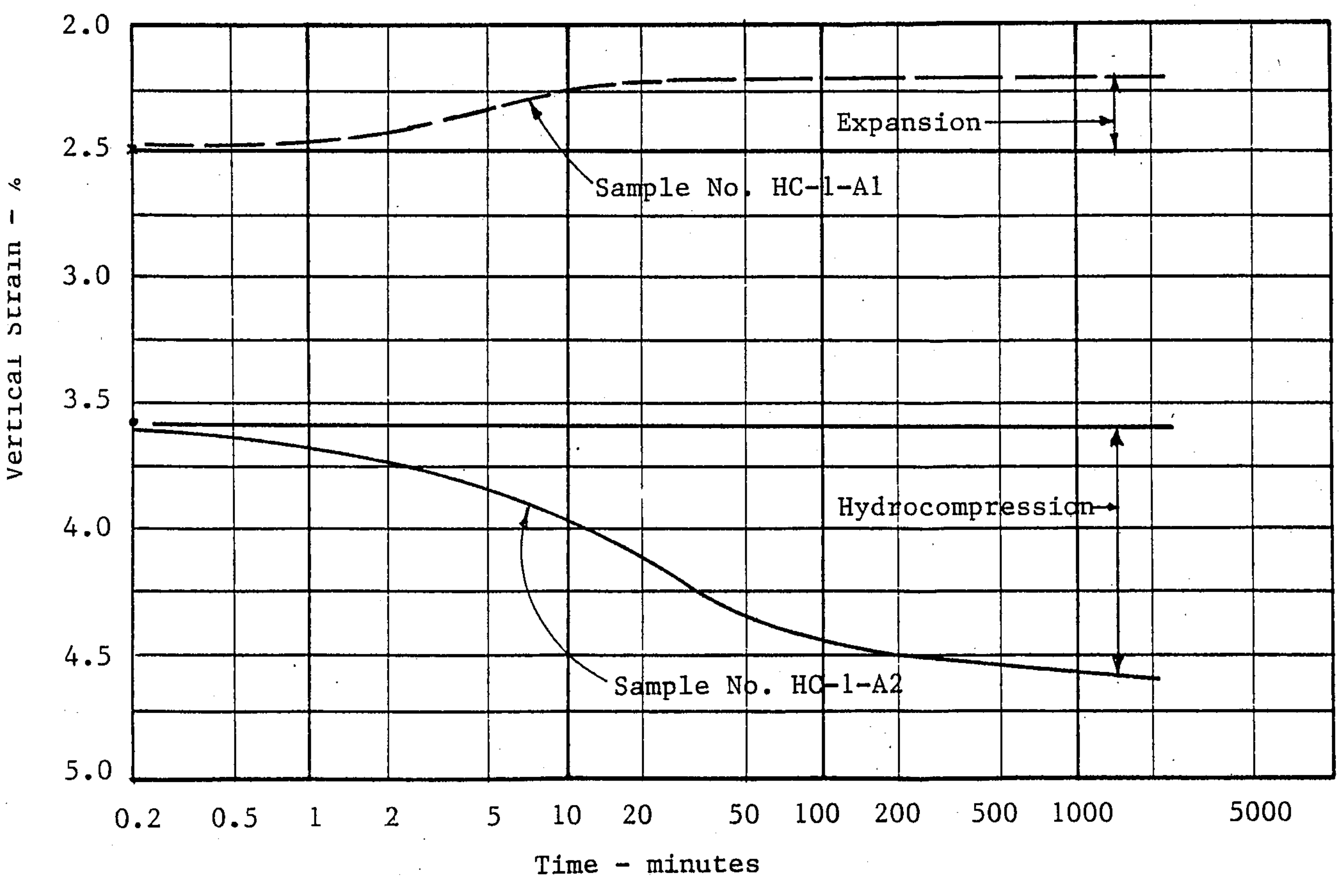
File No. S24-634-2S
November 7, 1994

Specimen:	HC-1-A1	HC-1-A2
Modling w/c, %:	10.2	10.2
Final w/c: %:	19.3	19.0
Dry Density, pcf:	114.7	114.5
Rel. Compaction:	91.8	91.7%
Final Loading, ksf:	4.3	8.6

Note: Loading time for each load increment = 60 minutes

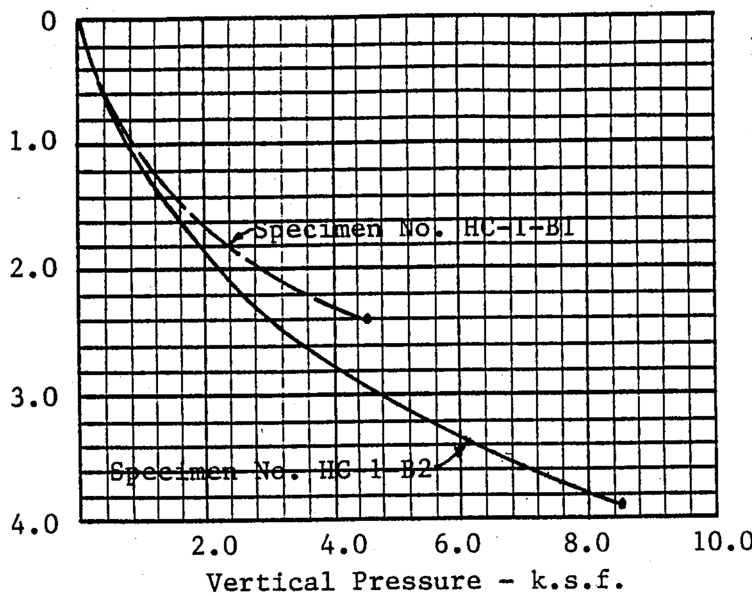


a) Verical Pressure vs. Strain w/o
Specimens submerged in water



b) Vertical Strain vs. Time since submerging specimens in water

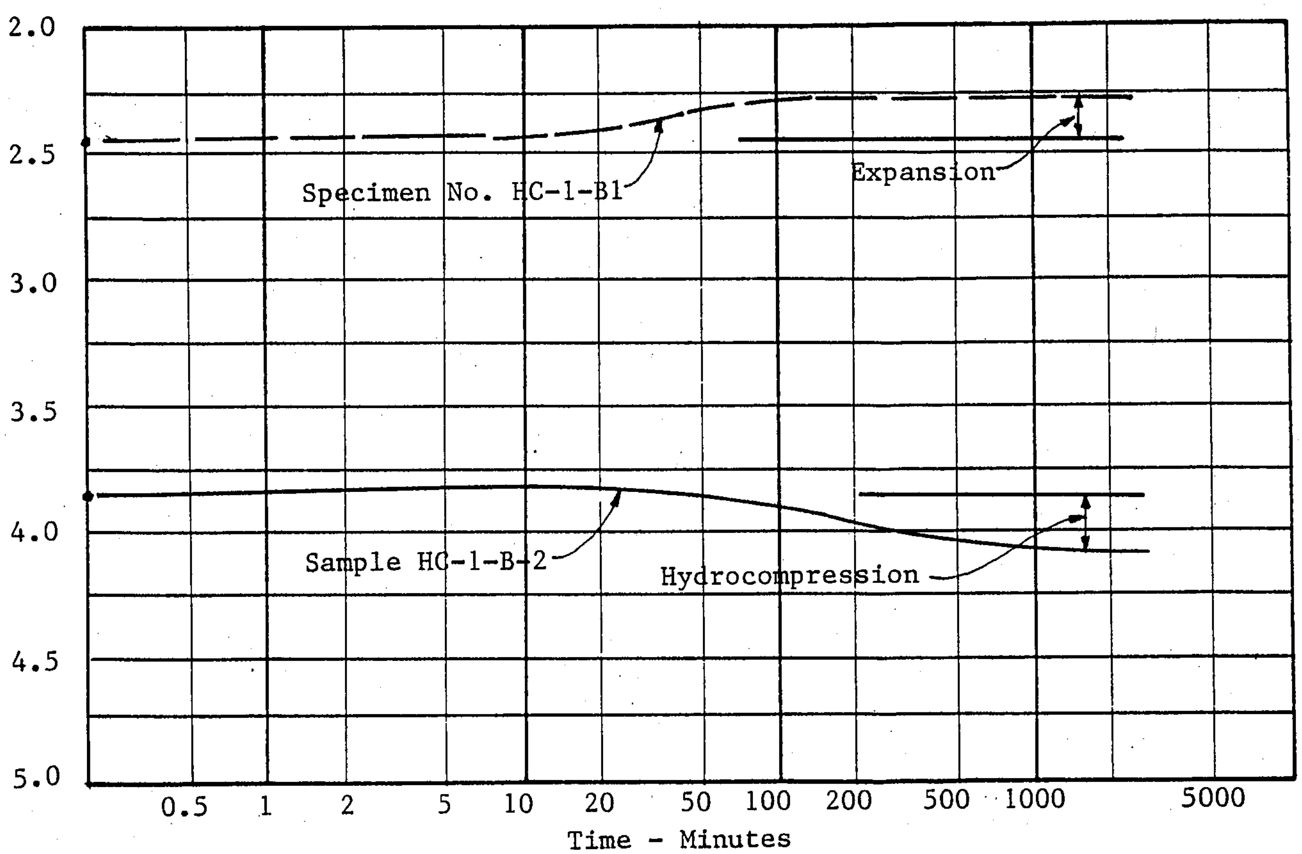
File No. S24-634-2S
November 7, 1994



a) Vertical Pressure vs. Strain w/o Specimens submerged in water

Specimens:		HC-1-B1	HC-1-B2
Molding w/c, %	:	11.8	11.8
Final w/c, %	:	16.6	16.1
Dry Density, pcf	:	114.4	114.7
Rel. Compaction, %	:	91.5	91.8
Final Loading, ksf	:	4.3	8.6

Note: Loading time for each load increment = 60 minutes

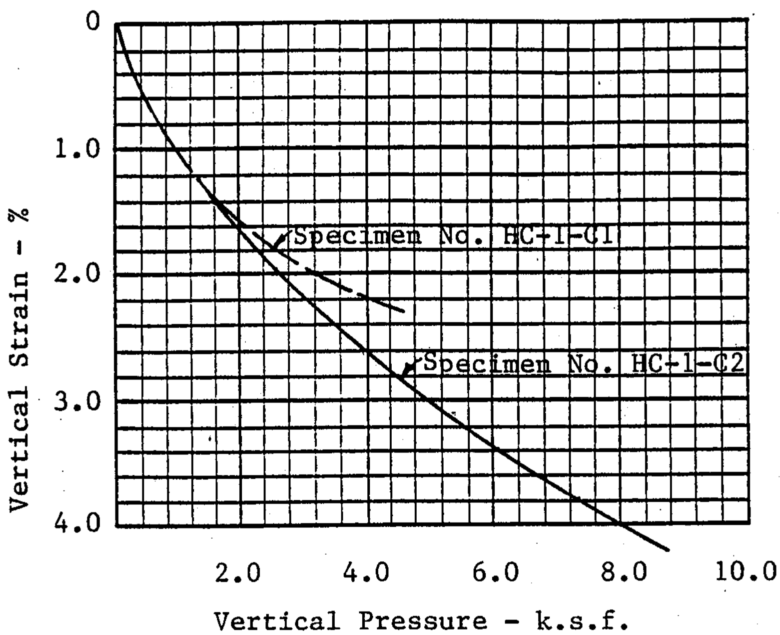


b) Vertical Strain vs. Time since submerging specimens in water

Plate H - Oedometer Test Results

SOIL FOUNDATION SYSTEMS, INC.

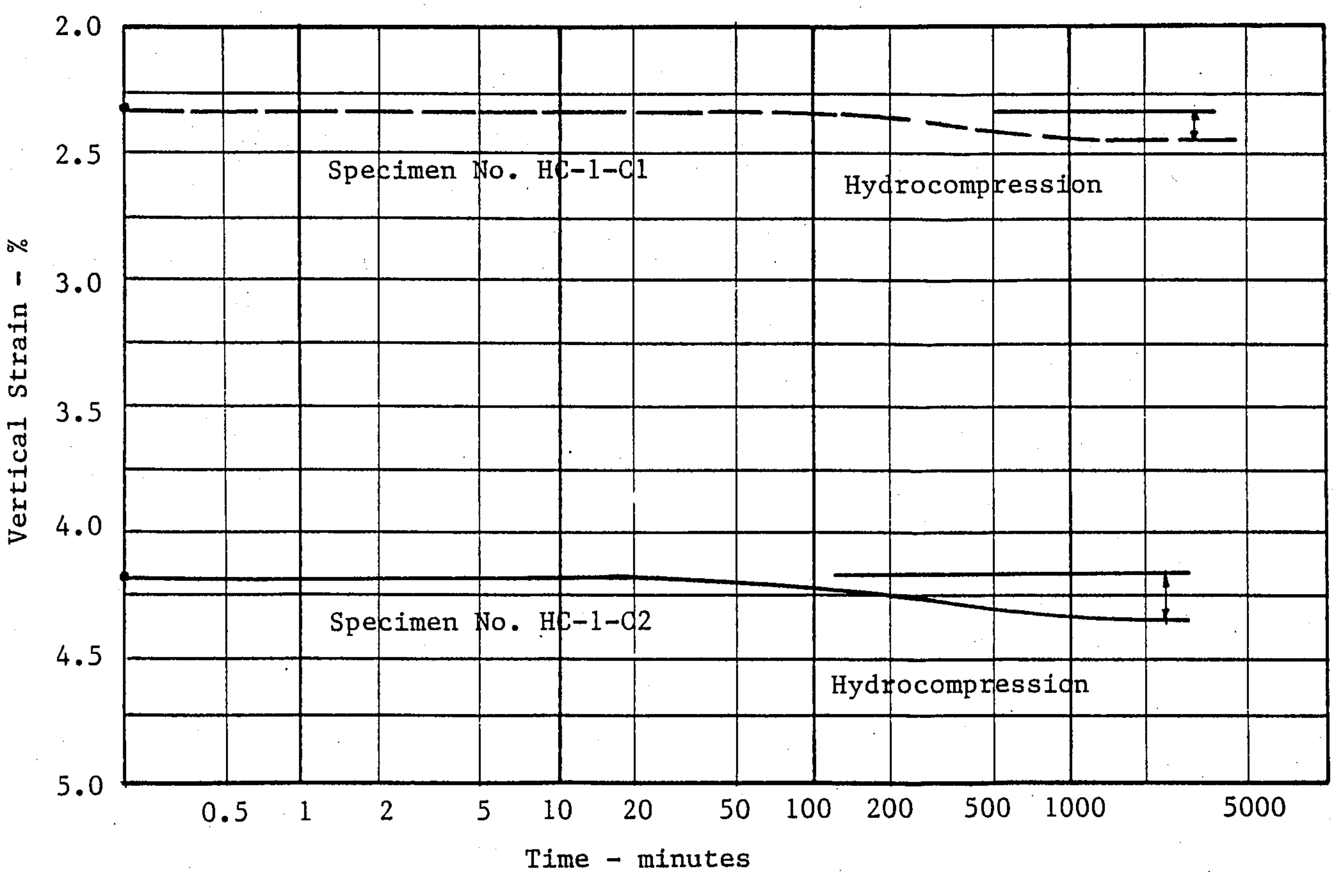
File No. S24-634-2S
November 7, 1994



a) Vertical Pressure vs. Strain w/o Specimens submerged in water

Specimen:		HC-1-C1	HC-1-C2
Molding w/c, %	:	13.7	13.7
Final w/c, %	:	16.8	16.3
Dry Density, pcf	:	114.8	114.9
Rel. Compaction, %	:	91.8	91.8
Final Loading, ksf	:	4.3	8.6

Note: Loading time for each load increment = 60 minutes



b) Vertical Strain vs. Time since submerging specimens in water

File No. S24-634-2S
November 7, 1994

Specimen No.	HC-1-D1	HC-1-D2
Molding Moisture Content, %:	11.4	11.4
Final Moisture Content, % :	-	15.4
Dry Density, p.c.f. :	114.7	114.9
Relative Compaction:, % :	92.6	92.7

Note: Loading time for each load increment:
Testing on as-compacted = 60 minutes
Testing on soaked = 24 hours.

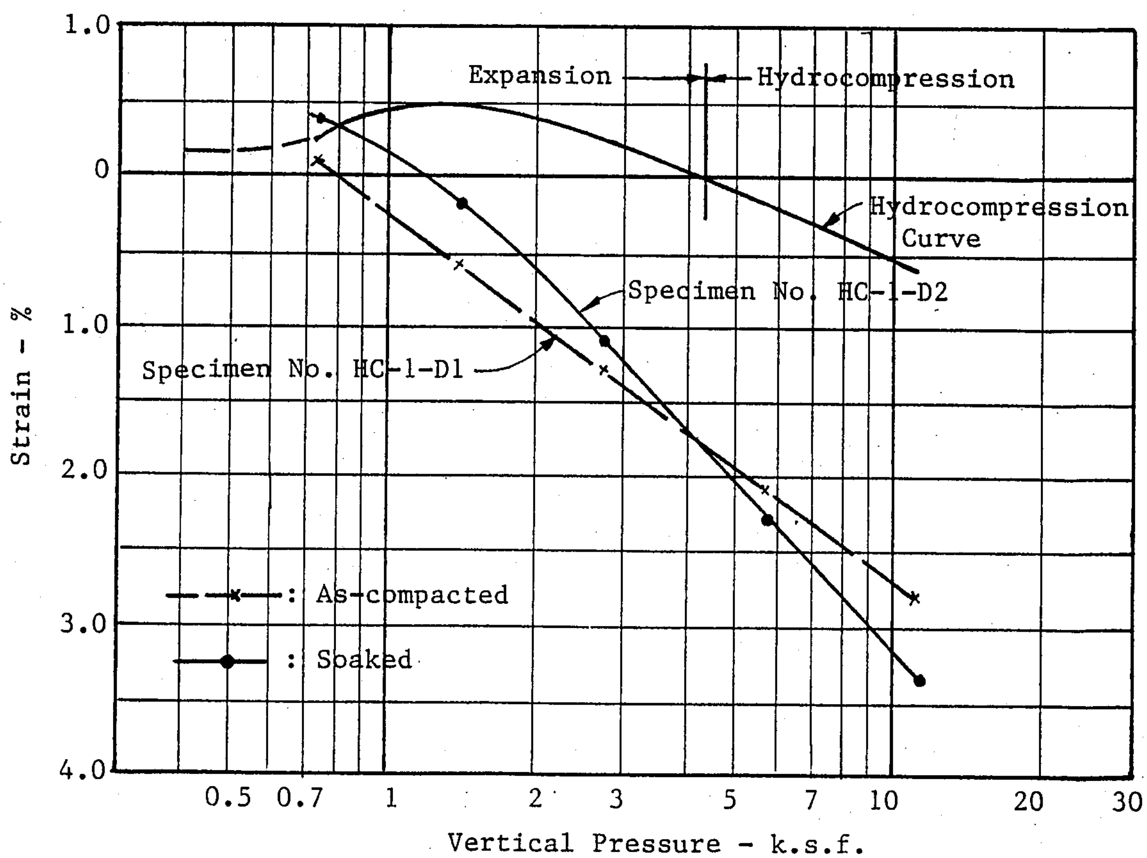
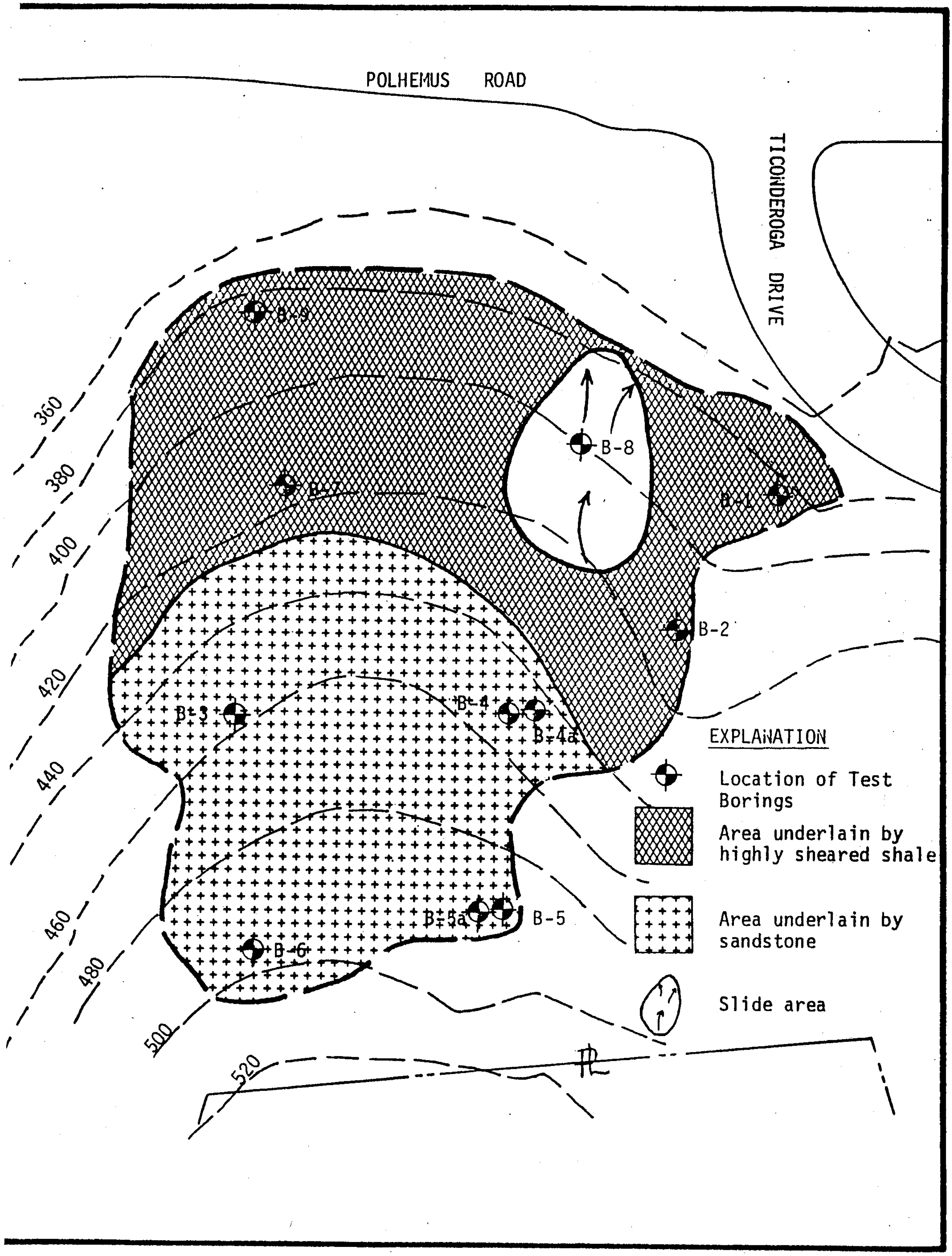


Plate J - Double-Oedometer Test Results

SOIL FOUNDATION SYSTEMS, INC.

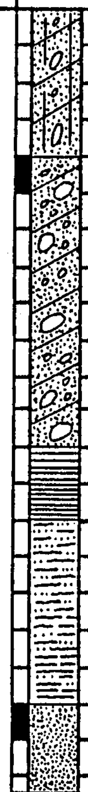


ig. 1 - Site Plan

United Soil Engineering, Inc.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-1
Date Drilled: 10-18-77		

Job No. 7-1967-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Medium brown sandy clayey SILT with scattered gravel; residual soil, dry; SM stiff from 4'
		28					5		Yellowish-brown, very stiff, slightly damp, sandy gravelly CLAY; CL
							10		
							15		Gray-green to dark gray highly sheared shale with rounded shear pebbles (1/4 to 1" in diameter) and very dense fragments of sand-stone (gray, calcite - veined) below 14'
		49					20		Light gray-brown, slightly damp, highly weathered & sheared *
									Boring terminated @ 21½'

Remarks: * = sandstone - some shear pebbles @ ½' maximum

Figure 3 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-2
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Light to medium brown gravelly silty SAND; changing color to gray-brown; SM
		70+					5		Highly sheared fragments of sand-stone & dark gray, yellow-green sheared shale; hard rock fragments @ 6 to 7½'
							10		
									Fairly smooth drilling form 7½' to 10½'. rocky from 10½' to 15'; Dark gray to medium brown & yellow-orange fragments of sand-stone & chert in matrix of clayey sand;
		21					15		
									Boring terminated @ 16½'

Remarks:

Figure 4 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-3
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Light yellow, light orange, and light tan (dry) colluvium;
		70+					5		composed of angular sandstone block in sandy SILT to silty sand matrix; dry through out; grades to in-place, tan-weathered (graywacke) sandstone.
							10		
									Boring terminated @ 10'

Remarks:

Figure 5 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-4
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Light tan and yellow (dry) colluvium (rock fragments to 1' in silty sandy matrix) rock
									Boring terminated @ 4'

Remarks:

* refusal in rock

Figure 6 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-4a
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Light tan and yellow (dry) rocky colluvium (rock fragments to 1' in silty sand matrix)
							5		
									(smoother) occasional rock fragments, weathered, dry sandstone
							10		

Remarks:

Figure 7 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-5
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Medium brown colluvium (angular sandstone blocks to 8" in sandy silt to silty sand matrix) - very dense @ 2'
									Boring terminated @ 4'

Remarks:

Figure 8a - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-5a
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Medium brown colluvium (angular sandstone blocks to 8 " in *
									sandstone block
							5		Light blue-gray fractured sandstone; fresh to slightly weathered; gradational to
							10		Fresh graywacke sandstone (very slow, steady, rough drilling)
									Boring terminated @ 11'

Remarks: * = sandy silt to silty sand matrix (very dense @ 2')

Figure 8b - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-6
Date Drilled: 10-18-77		

Job No. 7-1967-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Medium brown, dry, blocky sandstone fragments to 1'6" in sandy clayey silt matrix (colluvium) grades to
							5		Light yellow brown weathered sandstone, some chert & shale, gritstone;
									Gray fresh sandstone;
									Boring terminated @ 8'

Remarks:

Figure 9 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-7
Date Drilled: 10-18-77		

Job No. 7-1967-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Medium brown to light orange to light gray-green, highly weathered sandstone (dry, slightly gravelly & clayey silty SAND; SM
		28					5		Black, gray, dark moroon & gray-green highly sheared shale with greenstone (?) sheared lenticular bodies, some sheared pebbles to ½" maximum.
		26					10		(metabasalt (?) in tip)
		70+					15		
							20		Hightly sheared dark gray to orange, slightly damp claystone, shale and chert. possible old slide plane (clayey sand in sampler)
									Boring terminated @ 21½'

Remarks:

Figure 10 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-8
Date Drilled: 10-18-77		

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "φ" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 7-1967-S1 DESCRIPTION
									Medium brown, dry to slightly damp gravelly clayey SILT;
		16					5		Dark gray to blue gray and yellow-brown, damp to moist CLAY (highly sheared shale?) probable slide material; CL
		14					10		Light orange to brown clayey gravelly SAND & sandy gravelly CLAY (old slide material?) CL
							15		Gray-green and orange, highly sheared shale with some rock fragments, some chert·
							20		
									Boring terminated @ 20'

Remarks:

Figure 11 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Logged By: TN	EXPLORATORY BORING LOG	Hole No. B-9
Date Drilled: 10-18-77		Job No. 7-1967-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "ø" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Medium browm, dry gravelly clayey SILT; changing color to yellow-brown
		48					5		
									Medium to dark gray & gray-green slightly damp, highly sheared shale with red-brown streaks; ocassional layers or zones of light sandy CLAY and clayey sand with rock fragments; sheared dark gray shale; dry to slightly damp through out.
							10		
							15		
							20		
									Boring terminated @ 21'

Remarks:

Figure 12 - Log of Test Boring

UNITED SOIL ENGINEERING, INC.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-1	0-6	Fill: heterogeneous mixture of sandy clay and gravelly clay, brown and light brown, damp, medium stiff, (W < PL), some gravels to 6" across; a 5/8" diameter cable at 4'; base marked by 2" to 3" brown organic material.
	6-7½	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	7½-11	Subsoil: gravelly clay, brown, damp (W < PL), medium to low plasticity, gravels > 4" across comprise approximately 50 percent of this material, and percentage increasing with depth to possible bedrock at the bottom of the test pit.
		Total depth 11 feet; no free groundwater.
TP-2	0-1½	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1½-3	Subsoil: sandy clay with gravel, light brown, damp (W < PL), medium plasticity; increasing gravels with depth, fragments of sandstone commonly 1' to 3' across.
		Total depth 3 feet; no free groundwater.
TP-3	0-1½	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1½-6½	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W < PL), fragments of sandstone generally < 3" across.
		Total depth 6½'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-4	0-3½	Soil: sandy clay, brown, slightly damp (W < PL), medium to low plasticity, soft, with gravelly clay 3" thick at the base; contact with underlying subsoil approximately 25° downhill, no shearing observed.
	3½-6	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W < PL), fragments of sandstone generally <3" across.
		Total depth 6 feet; no free groundwater.
TP-5	0-2	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-4½	Subsoil: sandy clay to gravelly clay, light brown, slightly damp (W < PL), gravels generally <3" across.
	4½-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'.
		Total depth 5 feet; no free groundwater.
TP-6	0-4½	Soil?: silty gravel, dark brown, moist (W ≥ PL), fragments of sandstone generally 6" across; very hard digging.
		Total depth 4½ feet; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-7	0-6½	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	6½-10½	Talus: sandy gravel with minor clay, light brown, fragments of sandstone 6" to 1' across in sandy matrix, generally loose.
	10½-12	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 12 feet; no free groundwater.
TP-8	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-5	Colluvium?: clayey sand, brownish orange, damp (W<PL), friable.
	5-7½	Landslide shear zone?: clay to sandy clay, dark gray, moist (W≥PL), stiff, high plasticity.
	7½-10½	Colluvium?: clayey sand as above between 2 and 5 feet.
		Total depth 10½'; no free groundwater.
TP-9	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-5	Colluvium?: clayey sand, brownish orange, damp (W<PL), friable.
	5-8	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 8'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-10	0-1½	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1½-4	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'.
		Total depth 4'; no free groundwater.
TP-11	0-1	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1-3½	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'.
		Total depth 3½'; no free groundwater.
TP-12	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-4	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'.
		Total depth 4'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-13	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-3½	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'.
		Total depth 3½'; no free groundwater.
TP-14	0-4½	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	4½-6	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	6-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across.
		Total depth 7'; no free groundwater.
TP-15	0-2½	Fill?: sandy clay, mottled dark brown and reddish-brown, slightly damp (W<PL), medium stiff to stiff, medium plasticity, layered structure (horizontal).
	2½-4	Soil: sandy clay, dark brown, damp (W<PL), firm to 3½', low plasticity; medium stiff below 3½'.
	4-5½	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	5½-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across.
		Total depth 7'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-16	0-6	Fill: sandy clay as in TP-15; mottled light and dark brown, damp (W≤PL), and soft in top 2'; dark gray to dark brown, firm to very stiff, slightly damp (W<PL) below 2'.
	6-9	Soil: sandy clay, dark brown, damp (W<PL), firm to 7', low plasticity; medium stiff below 7'.
	9-10½	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	10½-11½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 11½'; no free groundwater.
TP-17	0-2	Soil: silty clay with minor sand, gray, damp to moist (W<PL), medium stiff to stiff, high plasticity, soft in top foot.
	2-5½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Damp from 2 to 3½' (W<PL), slightly damp (W≤PL) below 3½'; large block of very fractured but hard greenstone at 5'.
		Total depth 5½'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-18	0-1½	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1½-2	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W<PL), fragments of sandstone generally ≤3" across.
	2-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 1½'.
		Total depth 5'; no free groundwater.
TP-19	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-2	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	2-5	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across.
		Total depth 5'; no free groundwater.
TP-20	0-1½	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1½-5	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 5'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-21	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-2	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	2-6½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across, damp (W<PL).
		Total depth 6½'; no free groundwater.
TP-22	0-2½	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	2½-3	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	3-8½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across.
		Total depth 8½'; no free groundwater.
TP-23	0-4	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	4-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, a 6" steel pipe 5½' deep headed toward a man-hole.
		Total depth 7'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-24	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-6	Colluvium: sandy clay, dark brown, slightly damp (W<PL), medium stiff to stiff, medium to high plasticity.
	6-9	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 9'; no free groundwater.
TP-25	0-4	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	4-8½	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'.
		Total depth 8½'; no free groundwater.
TP-26	0-4	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	4-11	Colluvium: sandy clay, dark brown, slightly damp (W<PL), medium stiff to stiff, medium to high plasticity, with increasing gravel to bottom.
		Total depth 11'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-27	0-3	Soil: sandy clay, dark brown, damp (W<PL), firm to 2', low plasticity; medium stiff below 2'.
	3-5	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	5-10½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, moist to very moist (W>PL).
		Total depth 10½'; no free groundwater.
TP-28	0-3	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	3-6	Colluvium: sandy clay, dark brown, slightly damp (W<PL), medium stiff to stiff, medium to high plasticity, with dispersed gravel and layers of gravel.
	6-11	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, slightly damp (W<PL); contact with overlying colluvium is oriented downhill about 23 degrees and is distinct, no shearing observed.
		Total depth 11'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-29	0-2	Soil: sandy clay, dark brown, damp (W<PL), medium stiff to 1½, low plasticity.
	2-5	Colluvium?: sandy clay, dark brown, slightly damp (W<PL), firm to stiff, medium to high plasticity.
	5-8½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across, slightly damp (W<PL).
		Total depth 8½'; no free groundwater.
TP-30	0-2	Soil: sandy clay, dark brown, damp (W<PL), firm to 1, low plasticity; medium stiff below 1'.
	2-3½	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W<PL), fragments of sandstone generally <3" across.
	3½-5½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across, slightly damp (M<PL).
		Total depth 5½'; no free groundwater.
TP-31	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'.
		Total depth 5'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-32	0-1½	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1½-6	Bedrock: contact between sandstone and Franciscan sheared rock, sandstone to west.
		Total depth 6'; no free groundwater.
TP-33	0-1½	Soil: sandy clay, dark brown, damp (W<PL), firm to 1', low plasticity; medium stiff below 1'.
	1½-9	Colluvium: sandy clay, light brown, moist (W>PL), medium to high plasticity; gray, with common organic material below 6', low plasticity.
	9-12½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across.
		Total depth 12½'; no free groundwater.
TP-34	0-1½	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1½-4	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 1½'.
		Total depth 4'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-35	0-2½	Soil: gravelly silt with some clay, abundant organic material; dark brown, slightly damp (W<PL), low plasticity, soft; gravels to 1' across.
	2½-8½	Colluvium: gravelly clay, light brown, slightly damp (W<PL), medium to low plasticity, medium stiff.
	8½-18½	Talus: sandy gravel with minor clay, light brown, slightly damp, very loose; gravels all subangular sandstone commonly 6" to 8" across, but some 2' to 3'.
		Total depth 18½'; no free groundwater.
TP-36	0-3	Soil: gravelly silt with some clay, abundant organic material; dark brown, slightly damp (W<PL), low plasticity, soft; gravels to 1' across.
	3-6½	Colluvium and Talus: sandy gravel with minor clay to gravelly clay.
	6½-8½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, slightly damp (W<PL).
		Total depth 8½'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-37	0-3	Soil: gravelly silt with some clay, abundant organic material; dark brown, slightly damp (W<PL), low plasticity, soft; gravels to 1' across.
	3-12	Talus: sandy gravel with minor clay, light brown, slightly damp, very loose; gravels all subangular sandstone commonly 6" to 8" across, but some 2' to 3'.
TP-38	0-3	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	3-4	Colluvium: gravelly clay, light brown, slightly damp (W<PL), medium to low plasticity, medium stiff.
	4-6	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 1½'.
		Total depth 6'; no free groundwater.
TP-39	0-2½	Soil: sandy clay with minor gravel.
	2½-9	Bedrock: sheared sandstone, probably intermediate between sandstone as in TP-18 and Franciscan sheared rock as in TP-7.
		Total depth 9'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-46	0-3	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	3-9	Colluvium: sandy clay with minor gravel.
	9-11	Bedrock (in place?): sandstone, fine- to medium grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'.
		Total depth 11'; no free groundwater.
TP-47	0-4½	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	4½-6	Alluvium: sandy clay with large gravel up to 3' across, light brown, moist (W≥PL), high plasticity.
	6-12	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1' across.
		Total depth 12'; free groundwater at 6'.

Berlogar, Long & Associates

Job No. 805-10

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-48	0-2½	Soil: sandy clay with gravel.
	2½-7	Talus: sandy clay with large gravel up to 3' across.
	7-9	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1' across.
		Total depth 9'; no free groundwater.
TP-49	0-2	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	2-6	Talus: sandy clay with large gravels.
	6-8	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1' across.
		Total depth 8'; no free groundwater.

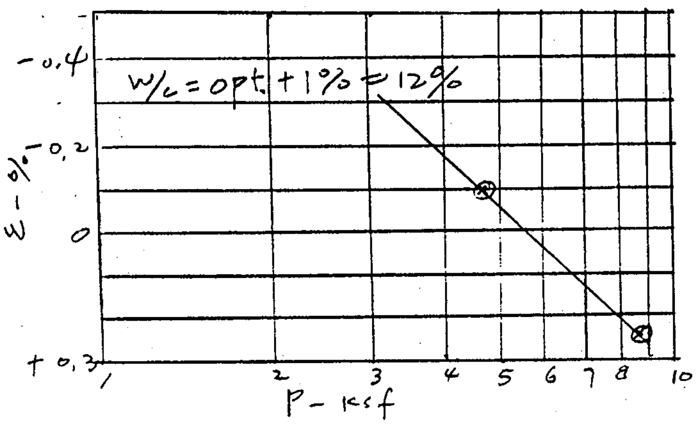
Berlogar, Long & Associates

File No. S24-634-2S
November 7, 1994

SETTLEMENT ANALYSIS

SOIL DATA : FILL – Sample 21-A : $t_{90} = 2.5$ min. ; X1-A : $t_{90} = 2.5$ min } $C_v = 0.05$ in²/min $= 0.5$ ft²/day

$C_R = 0.017$ – from Hydrocompression Curve on PLATE S-4



$C_R = \frac{1}{100}(0.25 + 0.10) \times \frac{1}{0.301} \doteq 0.012$
from Hydrocompression Test Series A, B & C.

Use $C_R = 0.015$

C_R: Comp. ratio
C_{RR}: Recompression Ratio

Qa — $H_{max} = 20'$ (after OX) $= H_1(10') + H_2(10')$

Sample 9-2 : H_1: $C_R = 0.075$, $C_{RR} = 0.030$ $P_p = 2.5$ ksf
9-3 : H_2: $C_R = 0.065$, $C_{RR} = 0.025$ $P_p = 2.0$ ksf
$C_v = 0.035$ ft²/day

1] SELF-WEIGHT COMPRESSION OF FILL

a) Hydrocompression

NOTE: No hydrocompression for $H_f \leq 35'$ (See Response Report P.)

$P_i = 35' \times 130 = 4550$ psf

H_f, ft:	40	45	50	55	60
$P_f = \frac{1}{2}(H_f + 35') \times 130$:	4875	5200	5525	5850	6175
$H = H_f - 35'$, ft :	5	10	15	20	25
$\rho_{Hc} = 0.015 H \times \log(P_f/P_i)$, in. :	0.03	0.10	0.23	0.40	0.60

NOTE: C_v not applicable to Hydrocompression — Assume $\rho_{Hc} = at$
$a = \rho_{Hc}/5$ yrs. For $H_f = 60'$, $a = 0.12''$/yr. — May be neglected !!

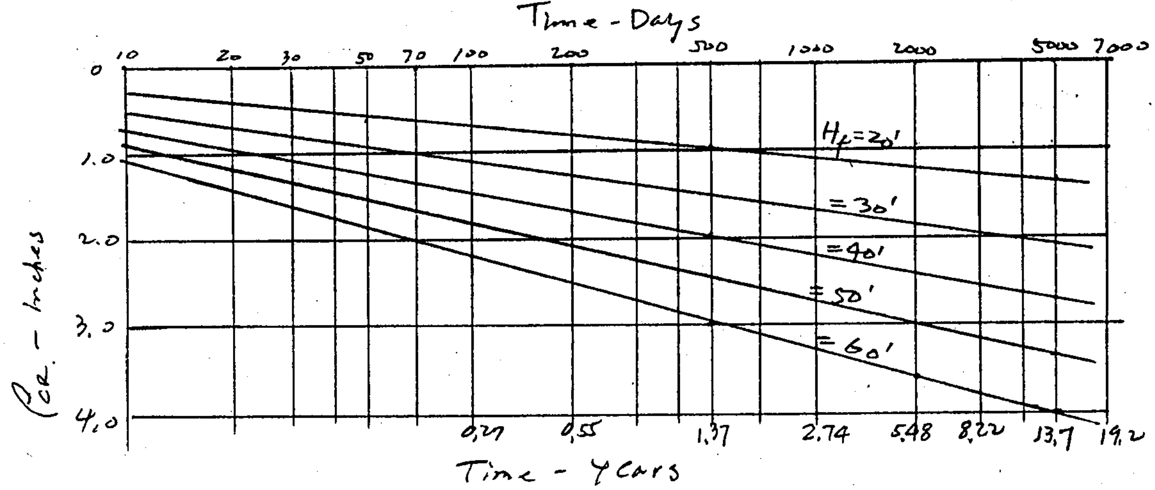
2

b) Creep

i) per Janbu, et al (See Report for Ref.) : $\frac{d\rho_{cr}}{dt} = \dot{\rho}_{cr} = \frac{H}{Rt}$

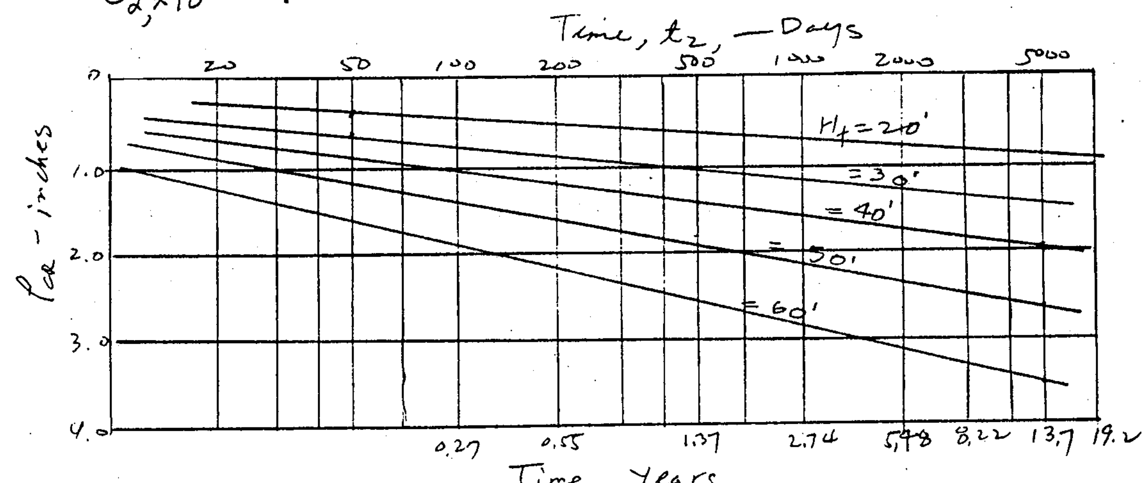
R = creep resistance : 1000 ~ 2000 for stiff clay. – Say R = 1500

$\rho_{cr} = \int \dot{\rho}_{cr}\, dt = \frac{H_f}{R} \ln(t_2/t_1)$, $t_1 = 1$ day @ completion of filling



ii) Use $C_\alpha = f(p)$, $\rho_{cr} = C_\alpha H_f \log(t)$

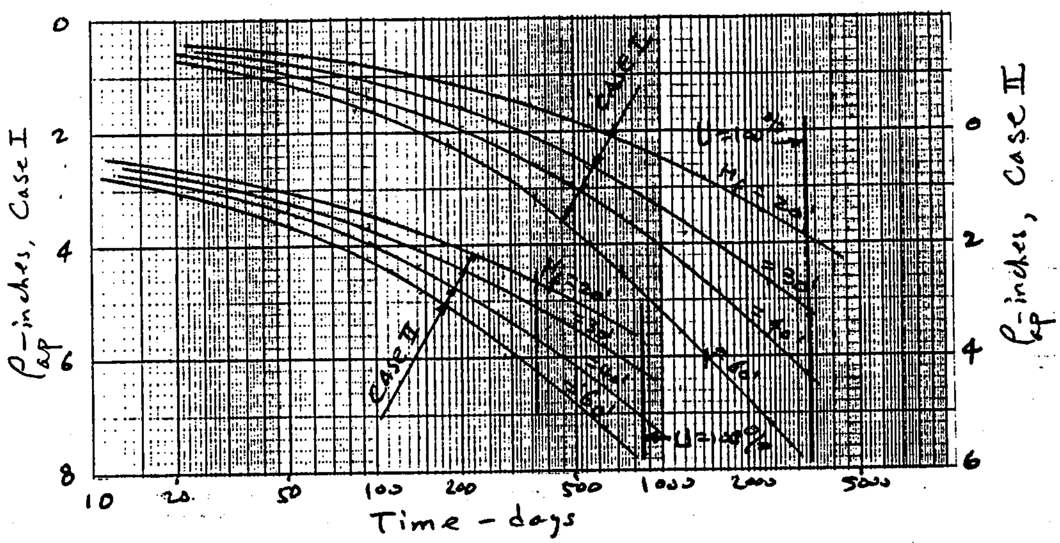
H_f, ft	:	20	30	40	50	60
p @ $\frac{1}{2}H_f$, psf	:	1,300	1,950	2600	3250	3900
$C_\alpha \times 10^{-3}$	:	1.05	1.09	1.12	1.16	1.31



t_2: Ellapsed Time since Completion of Fill

NOTE: R = 2000 w/ Jambu would give good agreement w/ results using C_α.

4



SECONDARY COMPRESSION

$P_c = 130(H_f + \frac{1}{2}H)$

$\rho_s = C_\alpha H \, Log(t_2/t_p)$ — $t_p = 3500$ days – Case I; $t_p = 860$ days – Case II

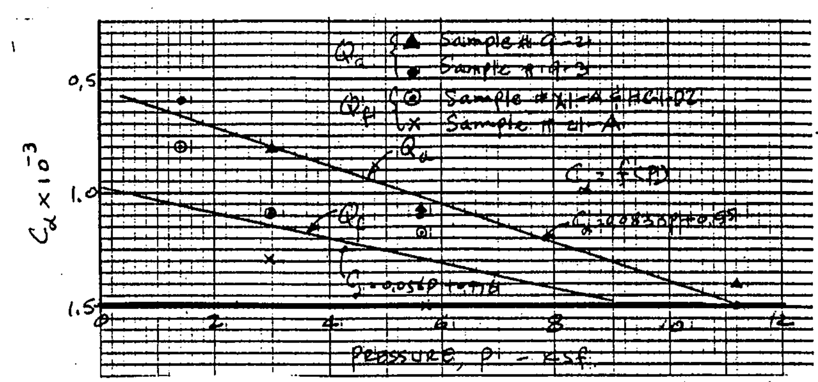
	H_f, ft	20	30	40	50	60
CASE I:	P_c, psf.	3,900	5200	6500	7800	9100
	$C_\alpha \times 10^{-3}$	0.87	0.98	1.09	1.20	1.31
CASE II:	P_c, psf.	3250	4550	5850	7150	8450
	$C_\alpha \times 10^{-3}$	0.82	0.93	1.04	1.15	1.25

Case I : $H_f = 60'$, $C_\alpha = 1.31 \times 10^{-3}$ $t_p = 3,500$ days $t_2 = 30$ yrs ($\doteq 11,000$ days)

$\rho_s = 1.31 \times 10^{-3} \times 20' \times 12 \times Log(11,000/3,500) = \underline{\underline{0.16''}}$

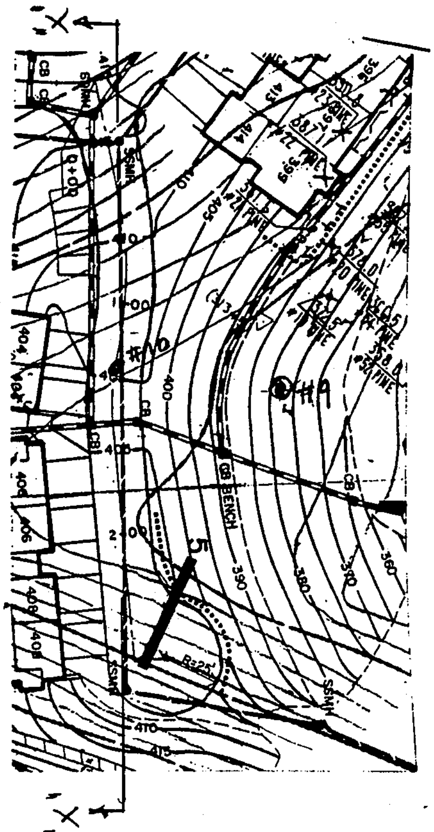
NOTE: $\rho_s = 0.16''$ – 30-yr. Life Time creep settlement under 60' of fill for H=20' case – most critical case.

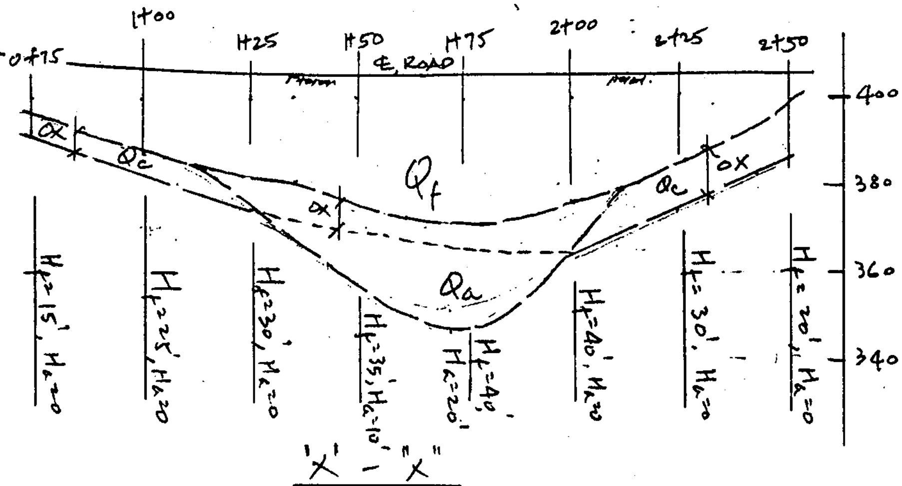
$\therefore$ Creep settlement of Qa may be neglected.



Determination of C_α used in Analysis

3] SETTLEMENT OF FILL ACROSS SWALE @ ℄ OF ROAD (BELOW BLDGS.)





NOTES: AVG. PROFILE BETWEEN ℄ OF ROAD & BORING #9.

Q_f: { ① Hydrocompression, $\rho_{Hc} \doteq 0$
② Creep, $\rho_{cR} \gg 0$

Q_a { ① Primary Consolidation, $\rho_{ap} \gg 0$ —(take from pg. 4)
② Secondary Compression, $\rho_{as} \doteq 0$

$(\rho_{ap} - \rho_{fc})$ = fill settlement corrected for construction perio[d]

ρ_f : fill settlement after filling.

$\rho_f \doteq \rho_{cR} + (\rho_{ap} - \rho_{fc})$ $\qquad \rho_{cR} = C_\alpha H_f \, Log(t/200)$ for $t \geq 400$ days.
t: time since beginning of filling

		50	100	200	400	800	1600	3200		
	t, days:	50	100	200	400	800	1600	3200		
	yrs:	.137	.274	.548	1.10	2.20	4.40	8.8	18	3[illegible]
Sta. 1+50, $H_f=35'$	ρ_{ap}:	1.3"	1.9	2.6	3.5	4.5	4.5	4.5	4.5	4.5
	$(\rho_{ap}-\rho_{fc})$:		0.4	1.04	2.6	3.5	4.5	4.5	4.5	4.5
	ρ_{cR}:		0.25	0.5	1.1	1.3	1.5	1.6	1.8	1.[illegible]
	$\rho_{cR}+\rho_{fc}$:		0.7	1.5	3.7	4.8	6.0	6.1	6.3	6.4
Sta. 1+75, $H_f=40'$	ρ_{ap}:	1.0	1.4	2.0	2.7	3.7	4.8	6.3	6.3	6.
	ρ_{fc}:		0.3	0.8	2.0	2.7	3.7	5.2	6.3	6.
	ρ_{cR}:		0.3	0.6	1.2	1.5	1.7	1.9	2.0	2.[illegible]
	$\rho_{cR}+\rho_{fc}$:		0.6	1.4	3.2	4.3	5.4	7.1	8.3	8.
Sta. 1+25	$\rho_{cR}+(\rho_{fc}=0)$		0.2	0.4	0.9	1.1	1.25	1.4	1.5	1.[illegible]
Sta. 2+00	"		0.25	0.5	1.1	1.3	1.5	1.6	1.8	1.9
Sta. 1+00	"		0.2	0.3	0.7	0.9	1.0	1.1	1.2	1.[illegible]
Sta. 2+25	"		0.2	0.4	0.9	1.1	1.25	1.4	1.5	1.6

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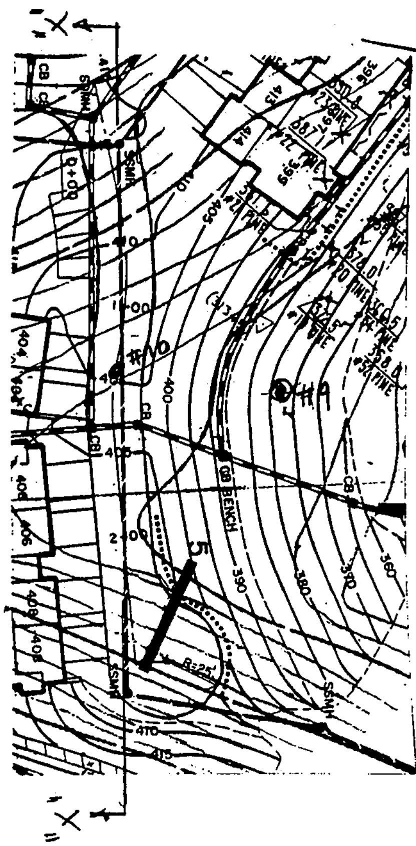
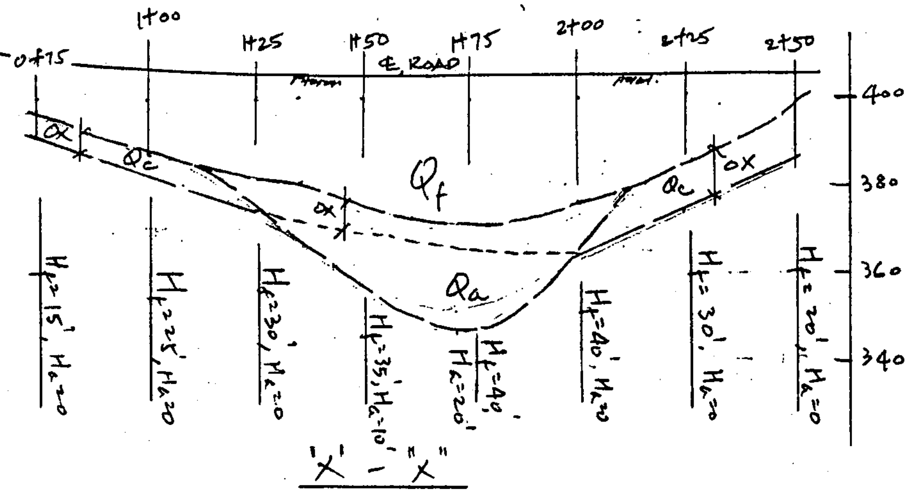
3] SETTLEMENT OF FILL ACROSS SWALE @ ℄ OF ROAD (BELOW BLDGS.)





'X' - "X"

NOTE: AVG. PROFILE BETWEEN ℄ OF ROAD & BORING #9.

Q_f: ① Hydrocompression, $\rho_{Hc} \doteq 0$
② Creep, $\rho_{CR} \gg 0$

Q_a ① Primary Consolidation, $\rho_{ap} \gg 0$ — (take from pg. 4)
② Secondary Compression, $\rho_{as} \doteq 0$

$(\rho_{ap} - \rho_{fc})$ = fill settlement corrected for construction perio[d]

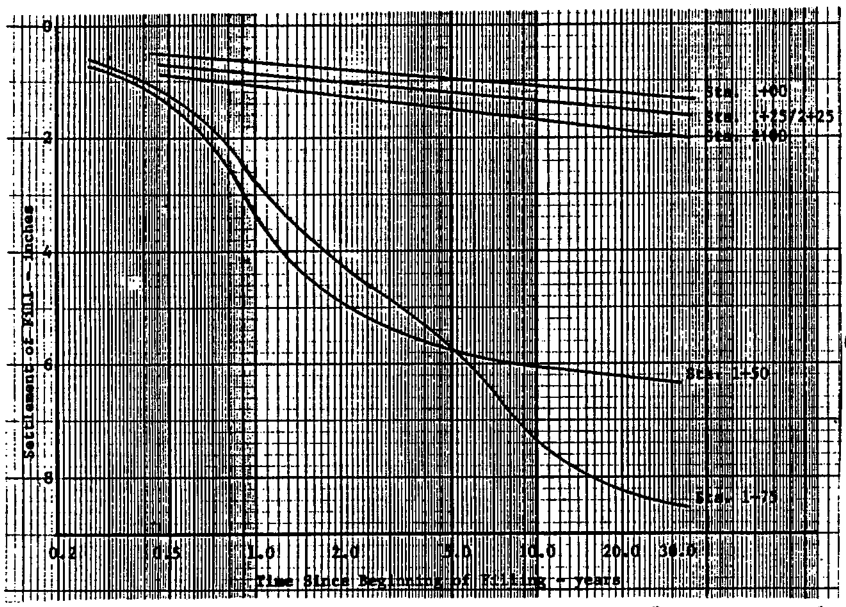
ρ_f : fill settlement after filling.

$\rho_f \doteq \rho_{CR} + (\rho_{ap} - \rho_{fc})$ $\qquad \rho_{CR} = C_\alpha H_f \operatorname{Log}(t/200)$ for $t \geq 400$ days.

t : time since beginning of filling

	t, days:	50	100	200	400	800	1600	3200		
	yrs:	.137	.274	.548	1.10	2.20	4.40	8.8	18	3
Sta. 1+50, H_f=35'	ρ_{ap}:	1.3"	1.9	2.6	3.5	4.5	4.5	4.5	4.5	4.
	$(\rho_{ap} - \rho_{fc})$:		0.4	1.04	2.6	3.5	4.5	4.5	4.5	4.5
	ρ_{CR}:		0.25	0.5	1.1	1.3	1.5	1.6	1.8	1.
	$\rho_{CR} + \rho_{fc}$:		0.7	1.5	3.7	4.8	6.0	6.1	6.3	6.4
Sta. 1+75, H_f=40'	ρ_{ap}:	1.0	1.4	2.0	2.7	3.7	4.8	6.3	6.3	6.
	ρ_{fc}:		-0.3	0.8	2.0	2.7	3.7	5.2	6.3	6.
	ρ_{CR}:		0.3	0.6	1.2	1.5	1.7	1.9	2.0	2.
	$\rho_{CR} + \rho_{fc}$:		0.6	1.4	3.2	4.3	5.4	7.1	8.3	8.
Sta. 1+25	$\rho_{CR} + \rho_{fc}(=0)$		0.2	0.4	0.9	1.1	1.25	1.4	1.5	1.
Sta. 2+00	"		0.25	0.5	1.1	1.3	1.5	1.6	1.8	1.9
Sta. 1+00	"		0.2	0.3	0.7	0.9	1.0	1.1	1.2	1.
Sta. 2+25	"		0.2	0.4	0.9	1.1	1.25	1.4	1.5	1.6

(6)



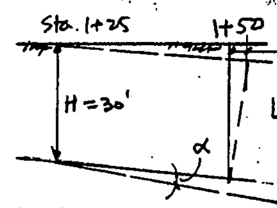
Summary of Fill settlement Along "X"-"X"

Compute Horizontal Movement @ Sta. 1+25 & Sta. 2+00:

(Assume Critical time span for A.C. pavement @ Road = $t = 2 \sim 5$ yrs.)

	1+25	1+50	1+75	2+00
ρ_{f-2}	1.0"	4.9"	1.3"	4.3"
ρ_{f-5}	1.2"	5.8"	1.5"	5.8"
	0.2"	0.9"	0.2"	1.5"

$\Delta\rho_f = 0.9 - 0.2 = 0.7''$ 1.5 − 0.2 = 1.3"



$m = K\alpha H$. K = 0.75

Sta. 1+25: $\alpha = 0.7/25'$; $m = (0.75 \times 0.7/25) \times 30 = 0.6''$. $\varepsilon_t \doteq 0.2\%$ } very Small

Sta 2+00: $\alpha = 1.3/25$ $m = (0.75 \times 1.3/25) \times 40 = 1.6''$ $\varepsilon_t = 0.3\%$

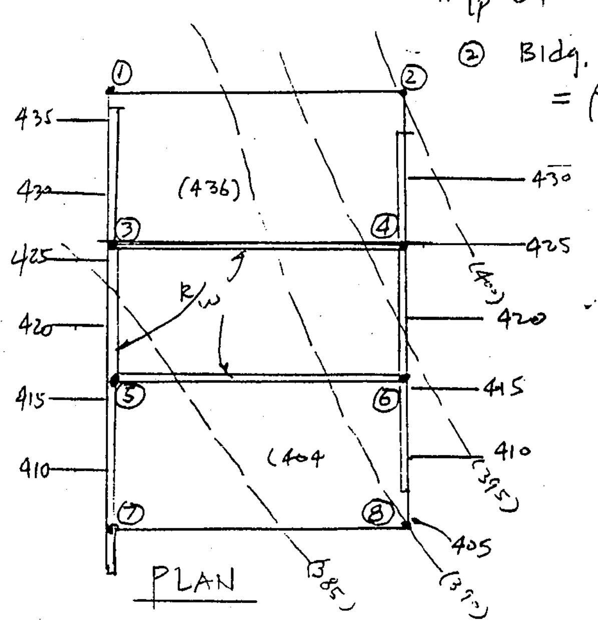
4] FOUNDATION SETTLEMENT

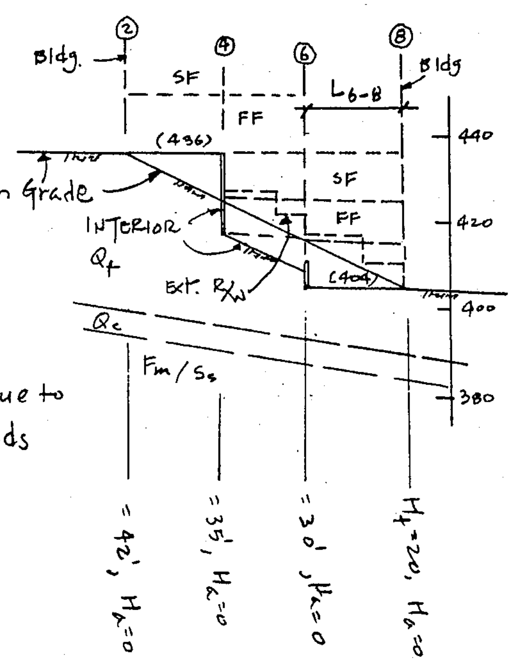
BLDG. "H"

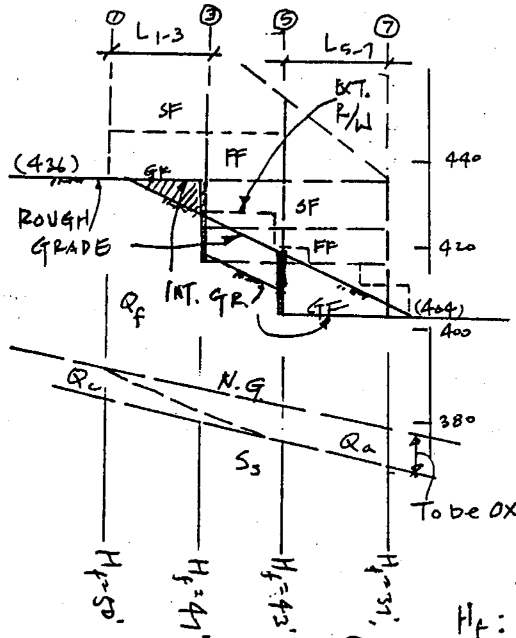
NOTE: ① All Q_a & Q_c under bldgs. Will be OK'd.
∴ $\rho_p = 0$ for BLDG.

② Bldg. Settlement $= \rho_{cR} + \rho_e$,

ρ_e: Settlement due to imposed Loads







Compute ρ_ℓ:

Loading: ①, ②, ③ & ④ — Other Locations

	①, ②, ③ & ④	Other Locations
Ret. Wall Back Fill –	700 psf.	~~0~~
Structure	300 "	300
$\Delta p =$	1,000 psf	300 psf

For Q_f, $C_c = 0.03$ — Data on Samples HC-1-D2

$$\rho_\ell = 0.36 H_f \times \text{Log}\left(\frac{\Delta p + 65 H_f}{65 H_f}\right), \quad H_f \text{ in feet.}$$

Take $(\rho_\ell)_{field} = \mathcal{U}\rho_\ell = 0.7\,\rho_\ell$

		①	②	③	④	⑤	⑥	⑦	⑧
H_f:		50'	40'	45'	35'	45'	30'	40'	20'
$(\times 0.7)$ $\frac{H}{o}$ $(\rho_\ell)_{field}$:		1.5"	1.4"	1.5"	1.4"	0.5"	0.5"	0.5"	0.4"
$U, \% =$	$t_\ell = 0.5$ yr.	11%	14	13	16	13	19	14	28
	$t_\ell = 1.0$	16	21	18	24	18	27	21	41
	$t_\ell = 2$	23	28	25	34	25	37	28	58
	$t_\ell = 5$	37	45	41	52	41	60	45	82
	$t_\ell = 10$	51	62	57	72	57	79	62	98
	$t = 30$	82	92	88	96	88	98	92	100

Combine ρ_{cr} & $(\rho_\ell)_{field}$: t: time since start of filling

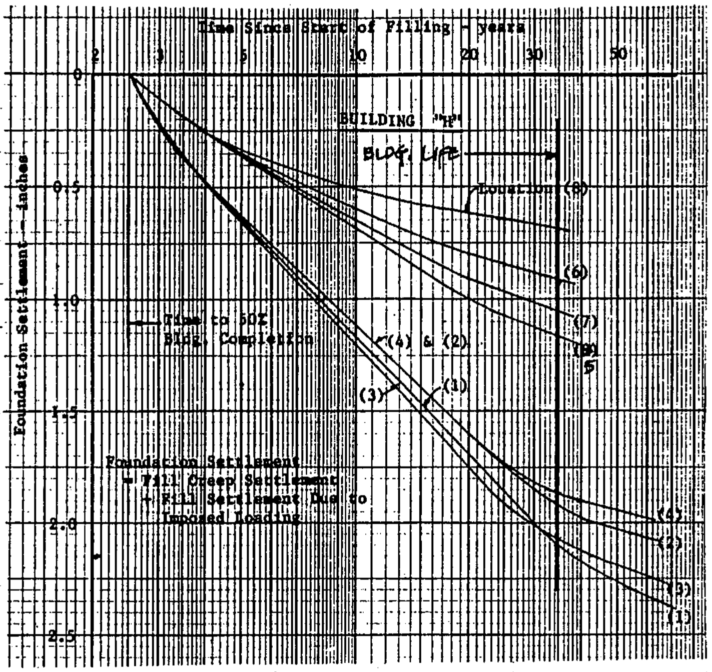
$\rho_{cr} = C_\alpha H_f \,\text{Log}\left(\frac{t}{2.5}\right)$ $\quad \rho_{cr} = 0$ @ $t = 0.5$ yr, $(\rho_\ell)_{field} = \mathcal{U}\rho_\ell$, $\mathcal{U} = 0.7$

$\rho_\ell = 0$ @ $t = 2.5$ yrs. $= t_{fill}(=1\text{ yr}) + t_{delay}(1\text{ yr}) + \frac{t_\ell}{2}$

$\rho_{cr} + \mathcal{U}\rho_\ell$

t:	3	5	10	20	40 (yrs)
①	0.25	0.68	1.15	1.68	2.22
②	0.30	0.65	1.10	1.59	1.98
③	0.30	0.65	1.15	1.75	2.15
④	0.22	0.66	1.07	1.51	1.90
⑤	0.10	0.35	0.65	1.02	1.20
⑥	0.18	0.39	0.62	0.80	0.95
⑦	0.10	0.33	0.60	0.92	1.08
⑧	0.17	0.33	0.52	0.67	0.75

(8)



Differential Settlement :

$t = 35$ YRS. for Bldg. Life.

For ① ~ ④ :

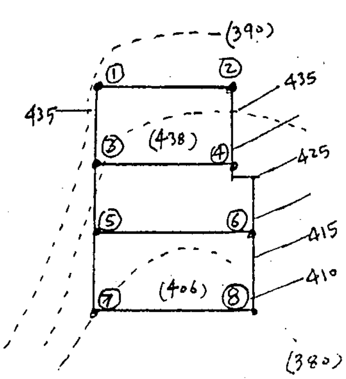
	L_{1-2}	L_{3-4}
$\Delta\rho$:	0.2"	0.2"
L =	42'	42'
Δ/L =	4×10^{-4}	4×10^{-4}

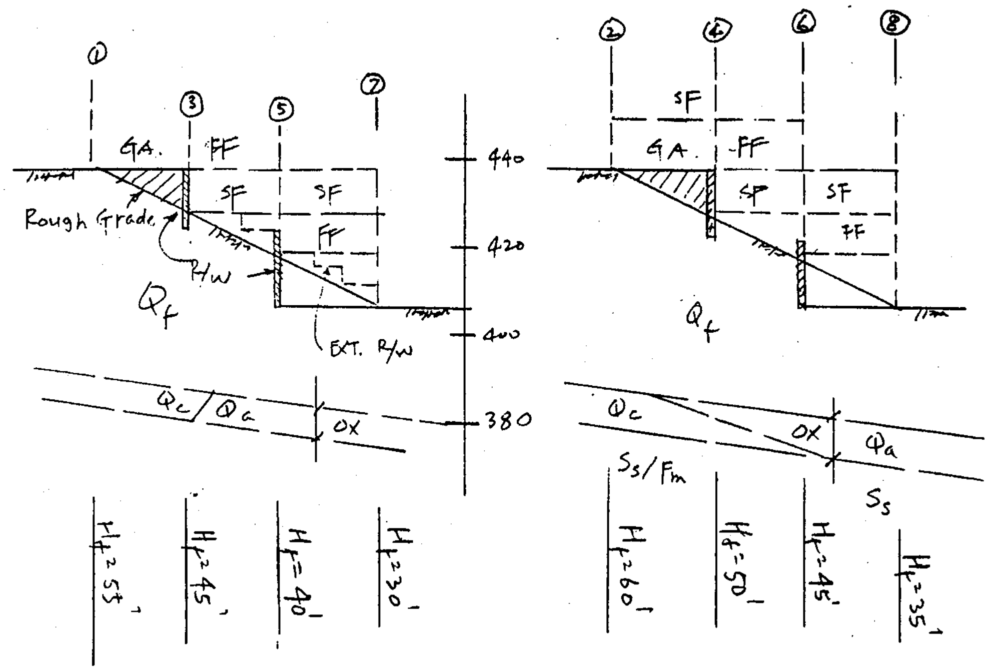
	L_{3-5}	L_{4-6}
$\Delta\rho$ =	0.85"	0.90"
L =	20'	20'
Δ/L =	0.0035	0.0038

$(\Delta/L)_{allow}$ for wood-frame residential $\doteqdot 1/250 \sim 1/300 \doteqdot 0.0038$

$(\Delta\rho)_{allow} \doteqdot 500(\Delta/L)_{allow} = 1.9" > 0.9"$ OK

BLDG. "I-A"





Loading : $\Delta p = 1000$ psf @ ①~④

$\Delta p = 300$ " @ others

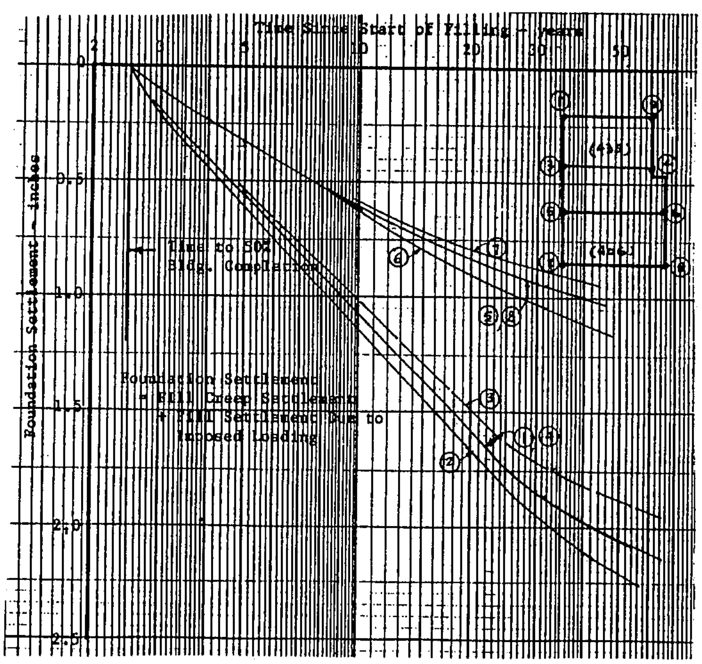
Compute $\mu\rho_\ell$: $\mu = 0.7$

	1	2	3	4	5	6	7	8
H_f :	55	60	45	50	40	45	30	35
$\mu\rho_\ell$:	1.5"	1.5	1.4	1.5	0.5"	0.5	0.5	0.5
U% = t_ℓ = 0.5 yr.	10	9	13	11	14	13	19	16
= 1.0	14	13	18	16	21	18	27	24
= 2	21	18	25	23	28	25	37	34
= 5	33	29	41	37	45	41	60	52
= 10	45	41	57	51	62	57	79	72
= 30	77	73	88	82	92	88	98	96
= 50	90	87	96	94	98	96	100	100

$t_{2.5} = t_{\ell\, 0.5}$; $t_{3.0} = t_{\ell\, 1\,yr.}$ $t - t_\ell = 2$ yrs.

Compute $\rho_{CR} + \mu\rho_\ell$:

	1		2		3		4		5		6		7		8	
t	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$	ρ_{CR}	$\mu\rho_\ell$
3	0.06	0.15	0.07	0.15	0.05	0.20	0.06	0.20	0.04	0.05	0.05	0.05	0.03	0.07	0.04	0.07
5	0.25	0.30	0.28	0.30	0.19	0.40	0.21	0.4	0.16	0.15	0.19	0.15	0.12	0.20	0.14	0.20
10	0.50	0.55	0.57	0.55	0.37	0.65	0.42	0.65	0.32	0.25	0.37	0.25	0.24	0.32	0.27	0.32
20	0.74	0.85	0.82	0.85	0.56	0.95	0.63	0.95	0.49	0.37	0.56	0.37	0.35	0.45	0.42	0.45
40	0.82	1.17	0.95	1.17	0.62	1.20	0.70	1.20	0.54	0.47	0.62	0.47	0.40	0.5	0.47	0.5
	2.00		2.10		1.82											



Differential Settlement :

	L3-5	L4-6
$\Delta\rho$ =	0.8"	0.75"
L =	20'	20'
Δ/L =	0.0033	0.0031

$(\Delta/L) < (\Delta/L)_{allow} = 0.0038$ ok

$(\Delta\rho)_{max} \doteq 500(\Delta/L)$ allow $= 1.9" > 0.8"$ ok

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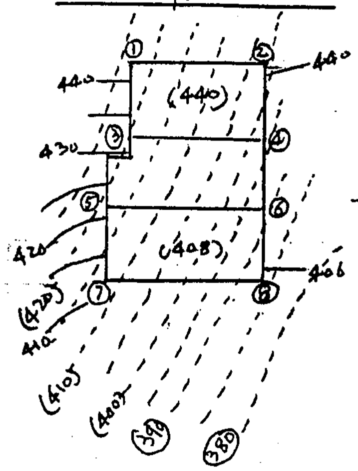
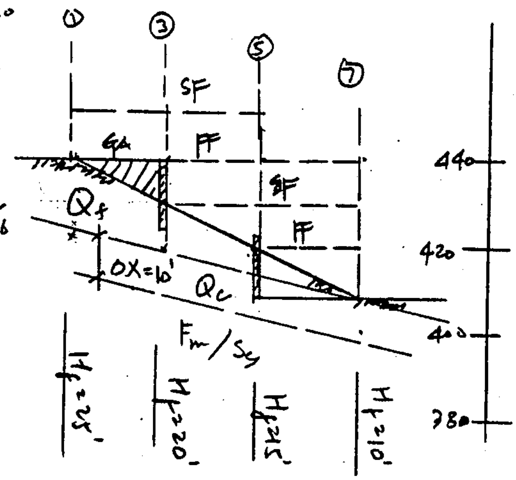
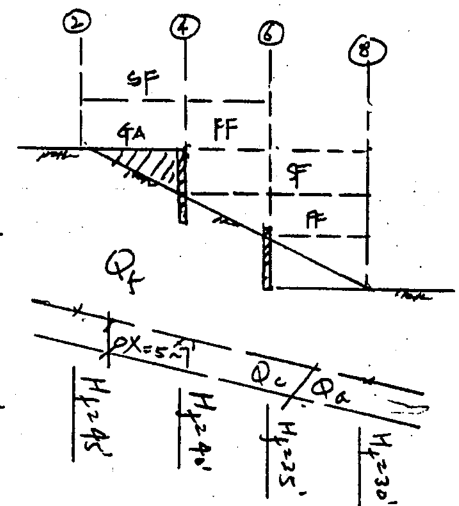
BLDG. "I-B"







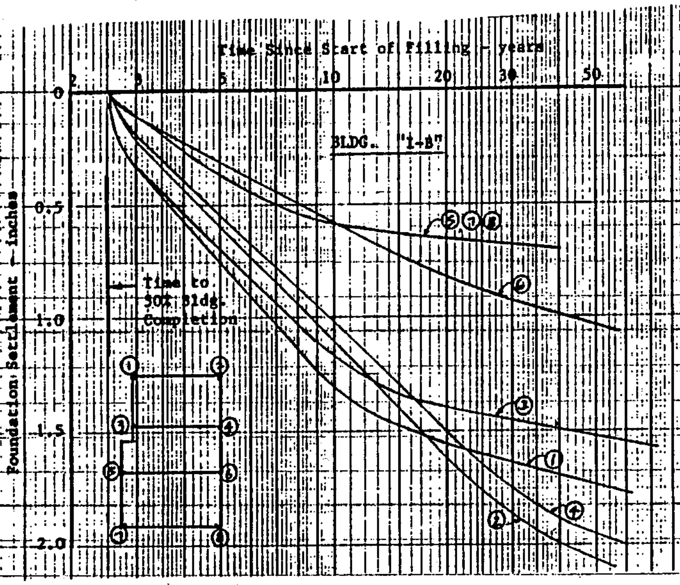
Compute $\mu \rho_c$: $\mu = 0.7$ $\Delta p = 1000$ psf @ ①, ②, ③ & ④ ; $\Delta p = 300$ psf @ others.

	①	②	③	④	⑤	⑥	⑦	⑧
H_f, ft. :	25	45	20	40	15	35	10	30
Δp :	1000	1000	1000	1000	300	300	300	300
$\mu \rho_c$:	1.3"	1.4	1.2	1.4	0.4	0.5	0.4	0.5
U = t=0.5 yr :	24%	13	28	14	36	16	57	19
t=1.0 :	32%	18	41	21	53	24	77	27
t=2.0 :	45%	25	58	28	73	34	94	37
t=5.0 :	72	41	82	45	95	52	100	60
t=10 :	90	57	98	62	100	72	100	79
t=30 :	100	88	100	92	100	96	100	98

$\rho_{cR} + \mu \rho_c$: $\rho_{cR} = C_\alpha H_f \operatorname{Log}(t/2.5)$, t in yrs. since beginning of filling

t =	3	5	10	20	40
①	0.35	0.75	1.35	1.55	1.70
②	0.20	0.60	1.10	1.60	2.0
③	0.35	0.70	1.20	1.40	1.50
④	0.20	0.60	1.00	1.50	1.90
⑤	0.10	0.4	0.55	0.65	0.70
⑥	0.10	0.3	0.60	0.80	1.00
⑦	0.10	0.30	0.50	0.55	0.60
⑧	0.10	0.30	0.60	0.65	0.70

(11)

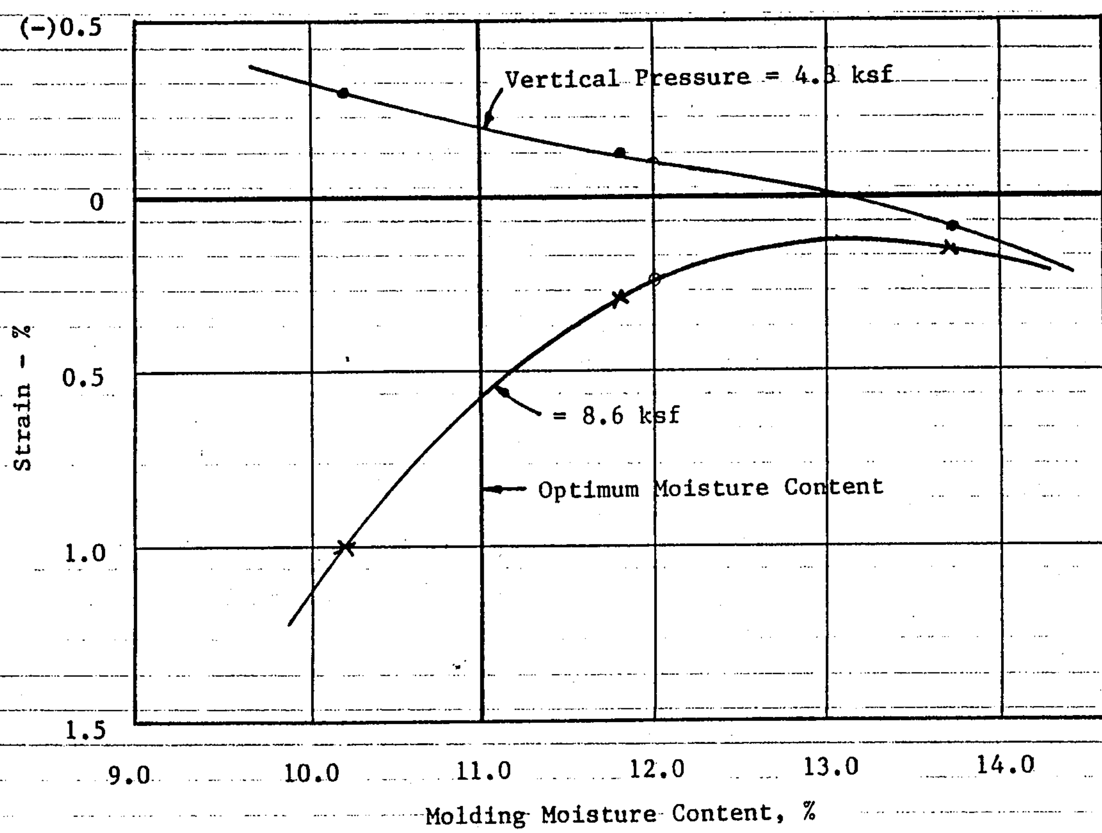


Differential Settlement:
(@ $t = 35$ yrs)

	L_{3-4}	L_{3-5}	L_{4-6}
$\Delta\rho =$	0.34"	0.77"	0.87"
$L =$	40'	20'	20'
$\Delta/L =$	0.0007	0.0032	0.0036

$\Delta/L = 0.0036 \leq (\Delta/L)_{allow} = 0.0038$ ok

$(\Delta\rho)_{allow} = 500\,(\Delta/L)_{allow} = 1.9$ OK



Estimation of Compaction-Induced Prestress

PEAK GROUND ACCELERATION

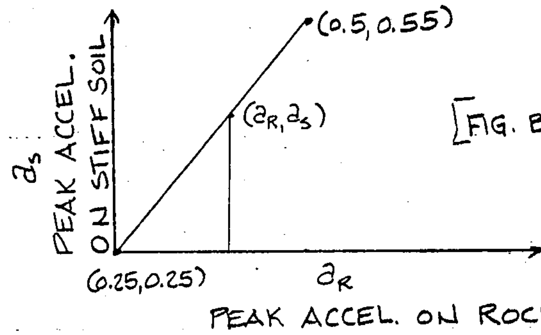
ATTENUATION RELATIONSHIP

$$\ln a = \ln \alpha_{(m)} - \beta_{(m)} \ln (R+20)$$ — IDRISS, 1985

USE R=1.5 Km

MAGNITUDE	$\alpha(m)$	$\beta(m)$	a
7.0	91.7	1.63	0.617
7.5	49.8	1.41	0.658
8.0	28.5	1.21	0.696
8.5	15.9	1.01	0.717

CORRECTION FOR STIFF SOIL SITE



[FIG. B-2, IDRISS, 1985]

$$(a_s - 0.25) : (a_R - 0.25) = 0.25 : 0.3$$

$$0.25 a_R - 0.0625 = 0.3 a_s - 0.075$$

$$a_s = \frac{1}{0.3}(0.25 a_R + 0.0125)$$

MAGNITUDE	PEAK ACCELERATION ON ROCK	PEAK ACCELERATION ON STIFF SOIL
7.0	0.617	0.56
7.5	0.658	0.59
8.0	0.696	0.63
8.5	0.717	0.64

HIGHLAND ESTATES

PEAK GROUND ACCELERATION

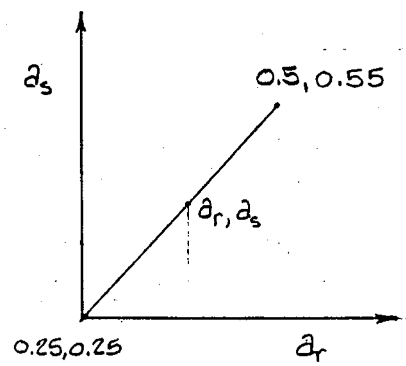
ATTENUATION RELATIONSHIP; IDRISS, 1991

$$\ln a_r = -0.05 + e^{[3.477 - 0.284M]} - \ln(R+20)\, e^{[2.475 - 0.286M]}$$

FOR ROCK SITES, $M > 6.0$

$R \doteq 1.5\,\text{km}$

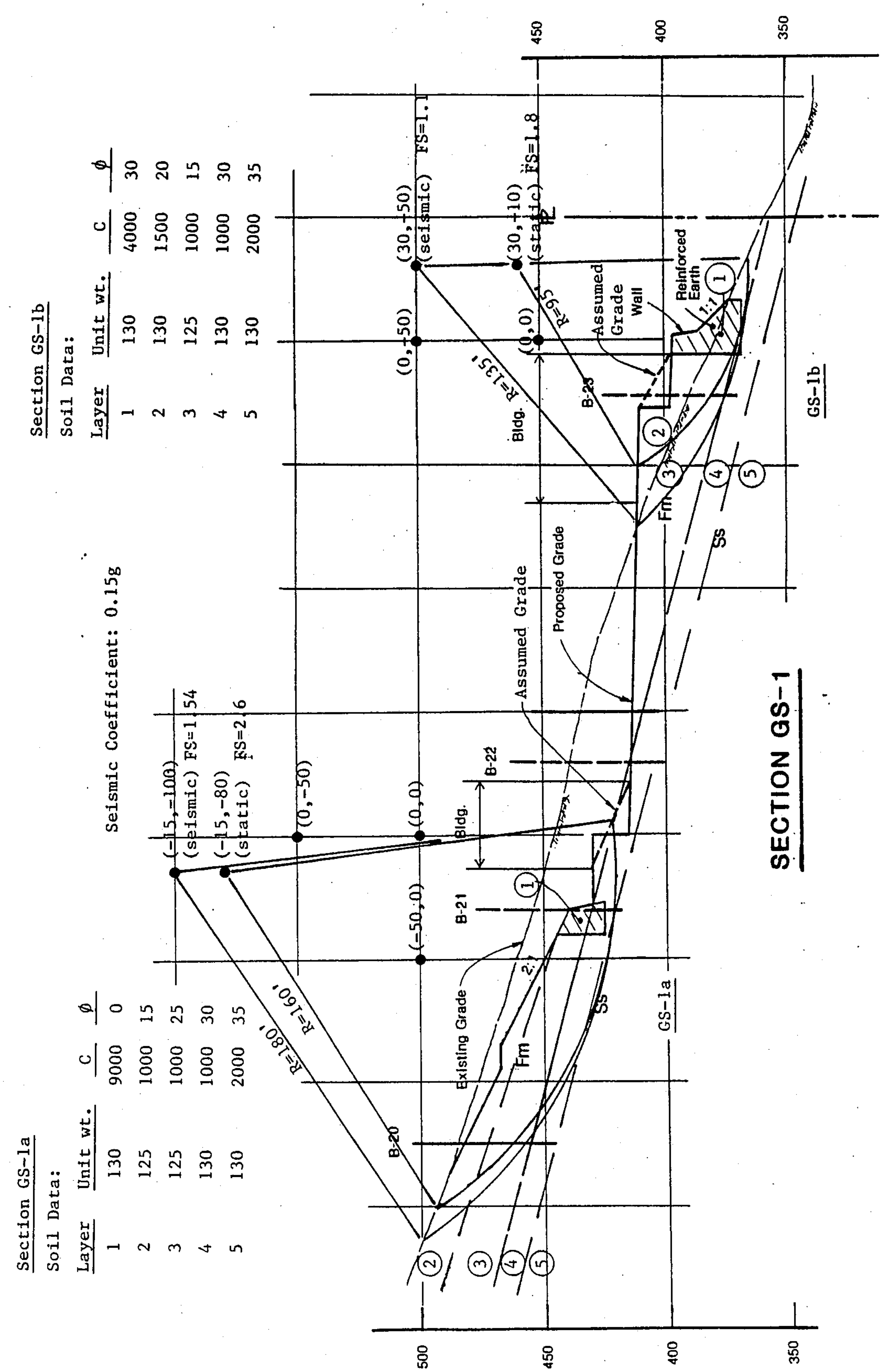
CORRECTION FOR STIFF SOIL SITE; FIG. B-2, IDRISS, 1985



$$(a_s - 0.25) : (a_r - 0.25) = 0.25 : 0.3$$

$$a_s = \frac{1}{0.3}(0.25 a_r + 0.0125)$$

MAGNITUDE	PEAK ACCELERATION ON ROCK	PEAK ACCELERATION ON STIFF SOIL
7.0	0.582	0.527
7.5	0.624	0.562
8.0	0.662	0.593
8.5	0.697	0.623



Section GS-1a — Soil Data:

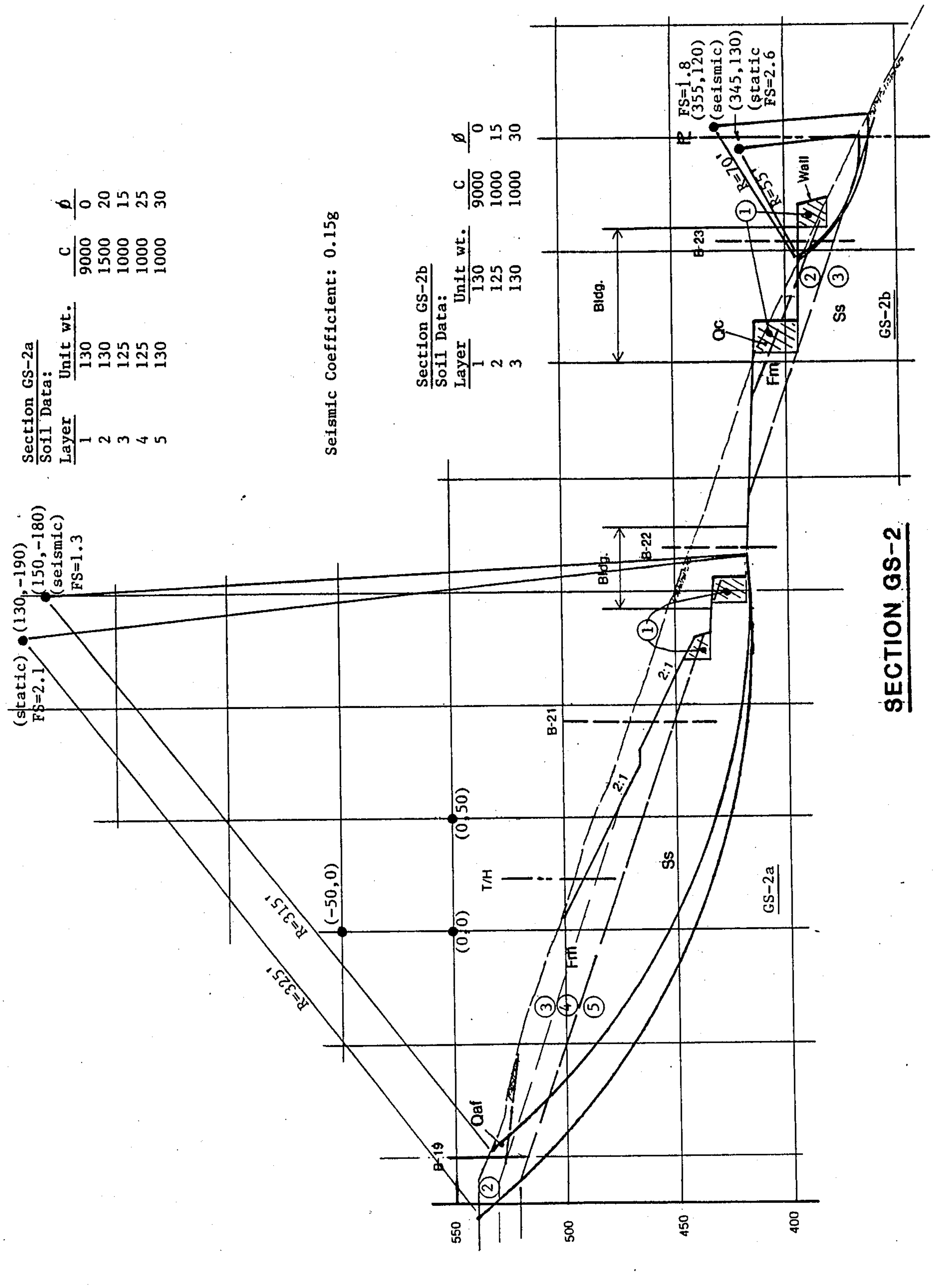
Layer	Unit wt.	C	ϕ
1	130	9000	0
2	125	1000	15
3	125	1000	25
4	130	1000	30
5	130	2000	35

Section GS-1b — Soil Data:

Layer	Unit wt.	C	ϕ
1	130	4000	30
2	130	1500	20
3	125	1000	15
4	130	1000	30
5	130	2000	35

GS-1: Cross Section Showing Critical Cirlces

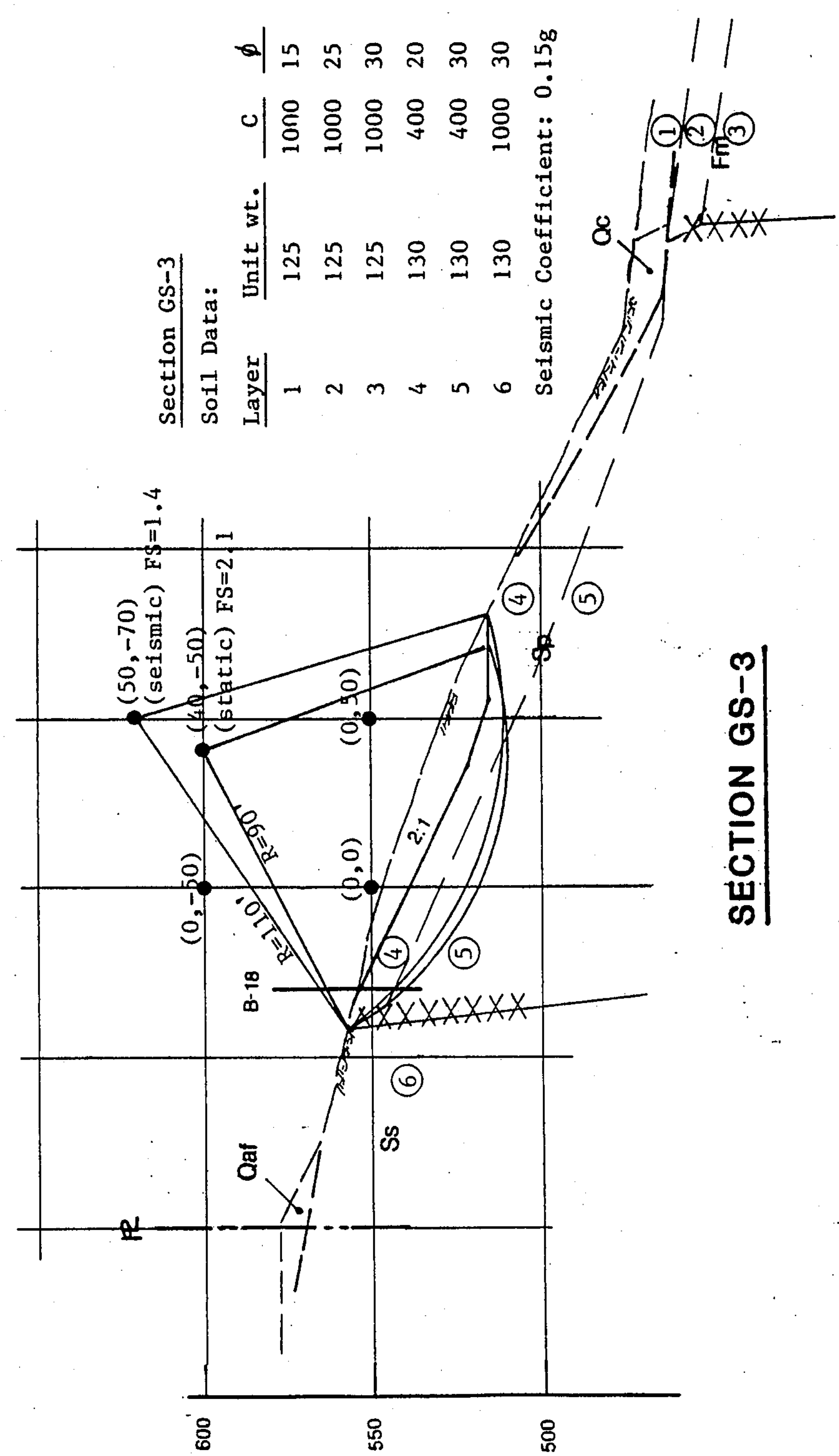
File No. S24-634-2S
November 7, 1994



GS-2: Cross Section Showing Critical Circles

SOIL FOUNDATION SYSTEMS, INC.

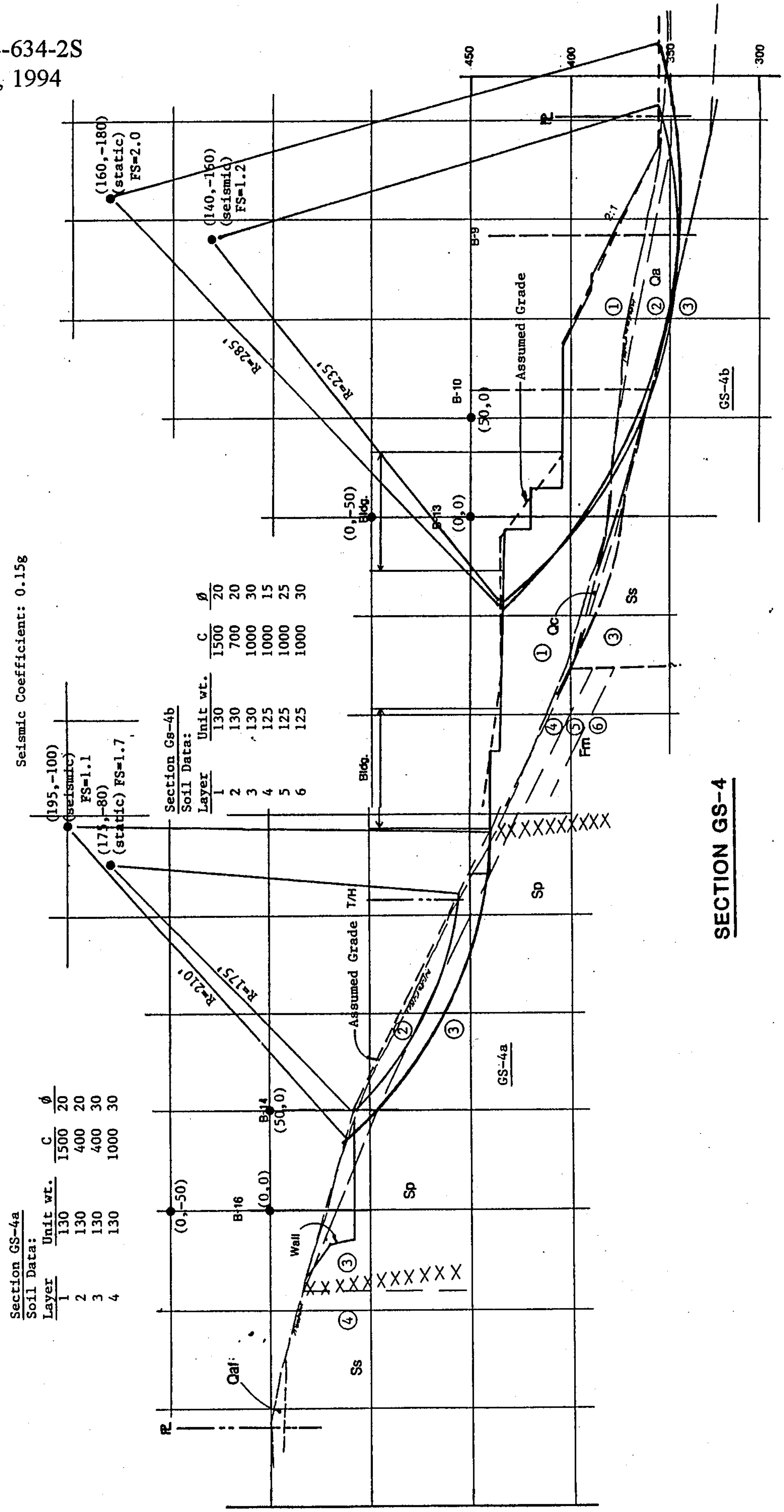
File No. S24-634-2S
November 7, 1994



GS-3: Cross Section Showing Critical Circles

SOIL FOUNDATION SYSTEMS, INC.

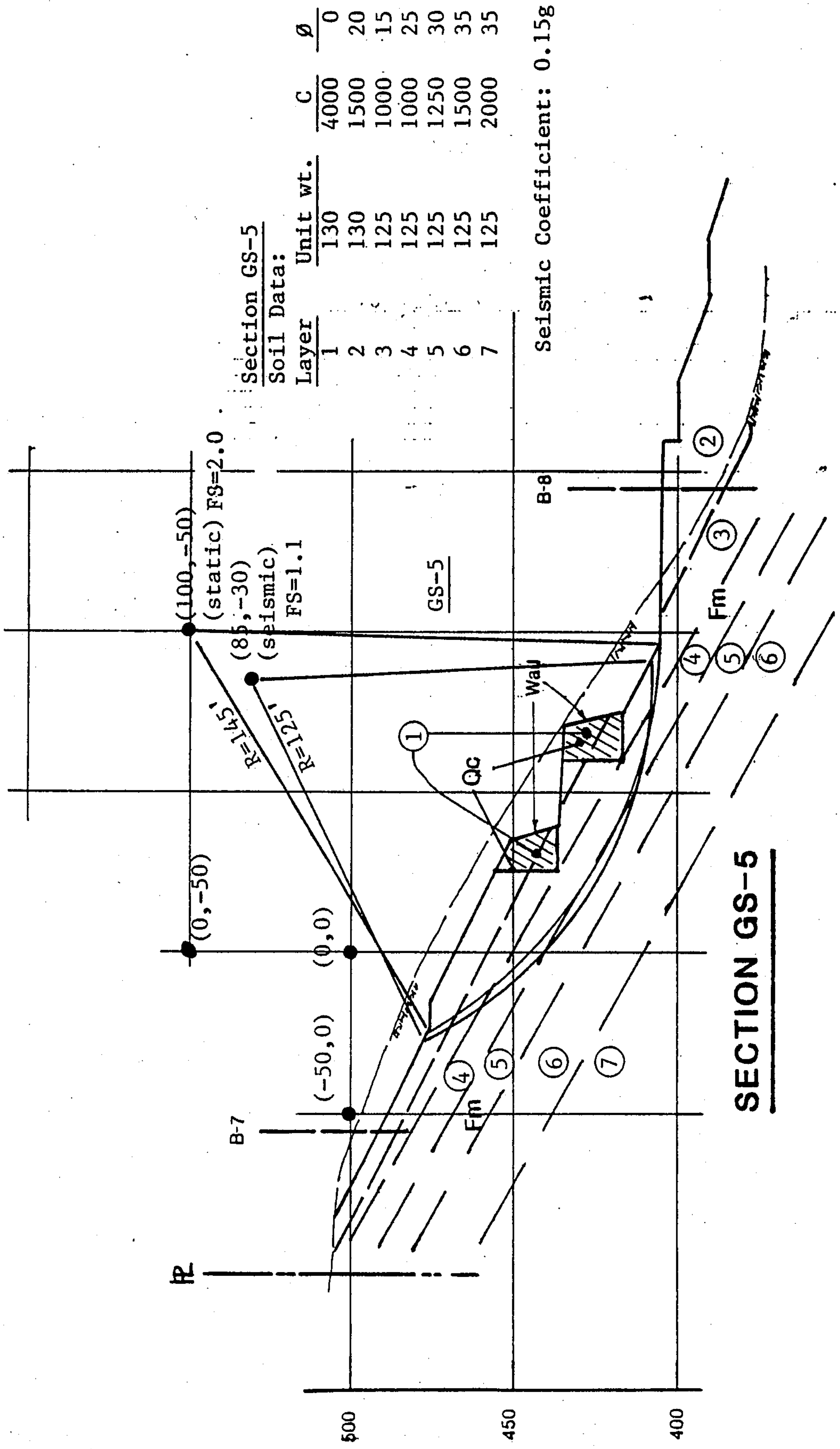
File No. S24-634-2S
November 7, 1994



GS-4: Cross Section Showing Critical Circles

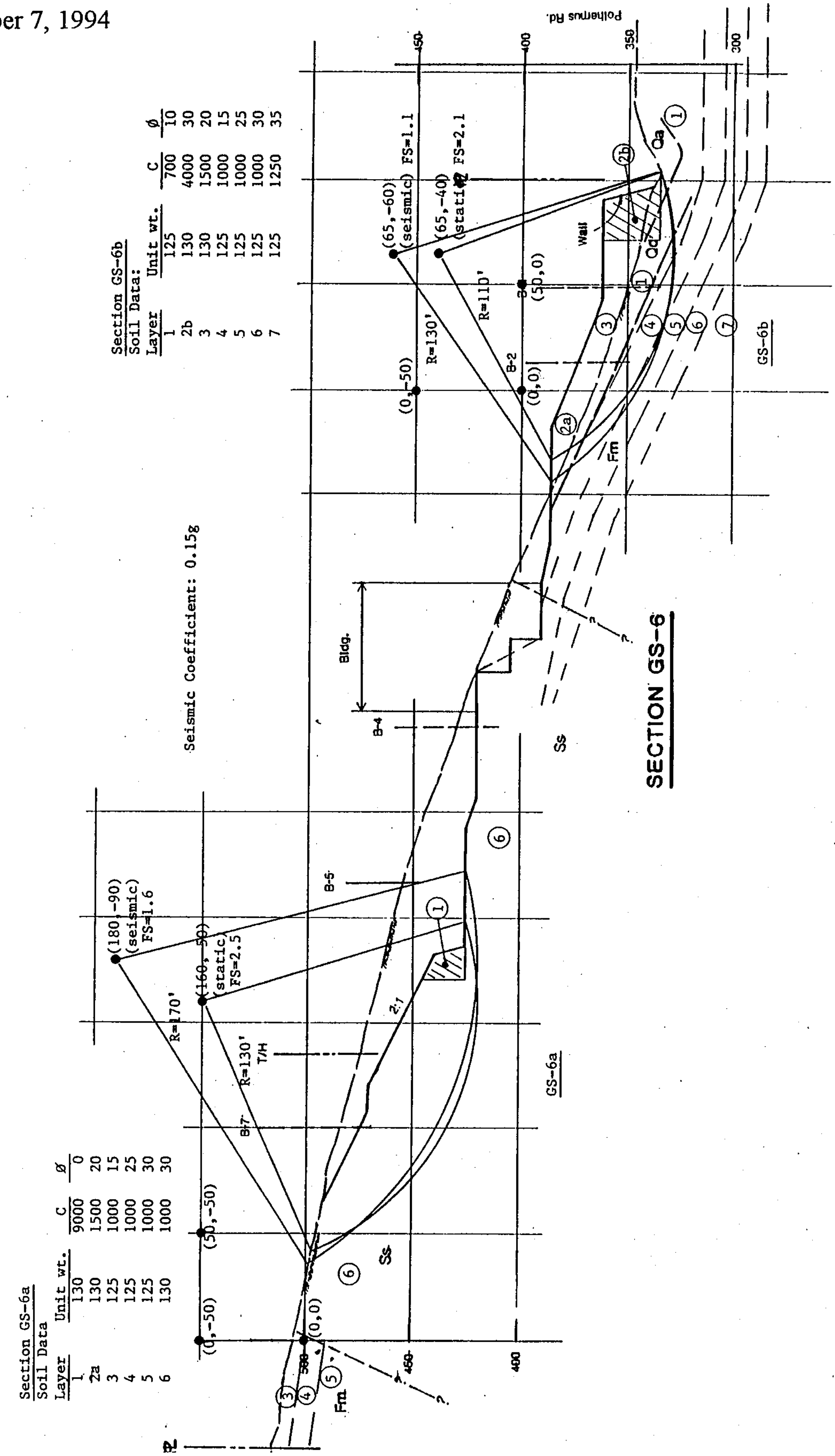
SOIL FOUNDATION SYSTEMS, INC.

File No. S24-634-2S
November 7, 1994



GS-5: Cross Section Showing Critical Circles

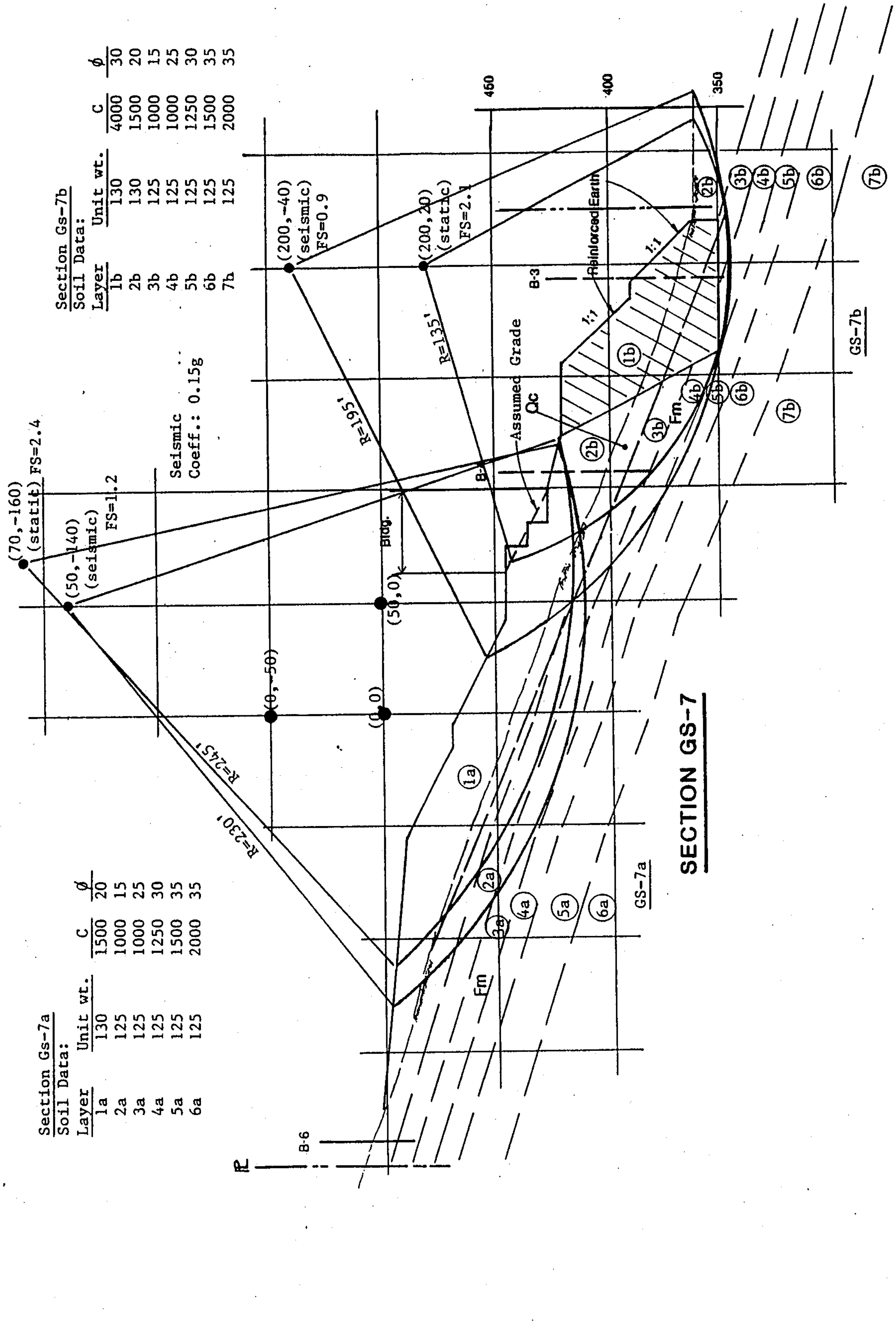
SOIL FOUNDATION SYSTEMS, INC.



GS-6: Cross Section Showing Critical Circles

SOIL FOUNDATION SYSTEMS, INC.

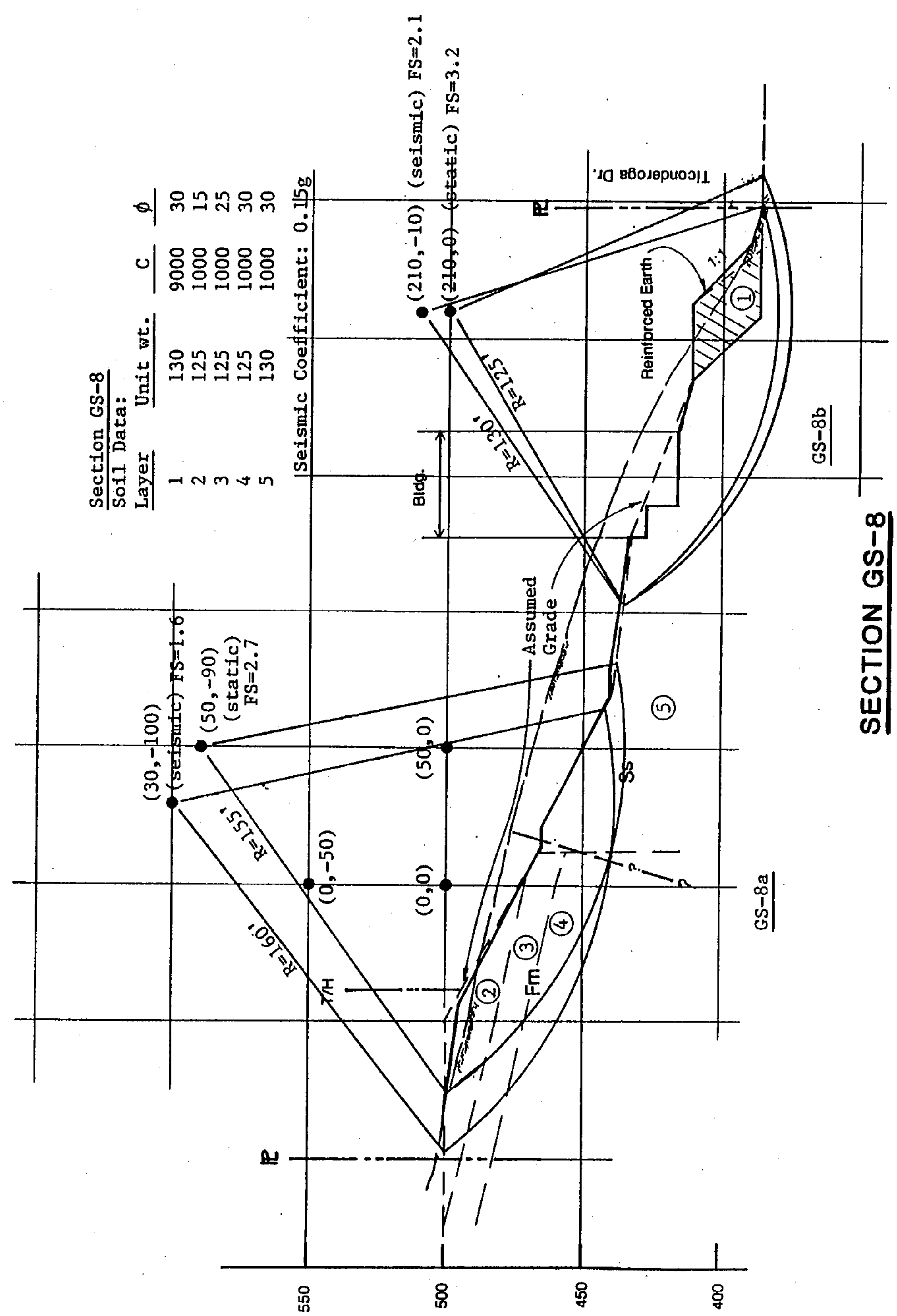
File No. S24-634-2S
November 7, 1994



GS-7: Cross Section Showing Critical Circles

SOIL FOUNDATION SYSTEMS, INC.

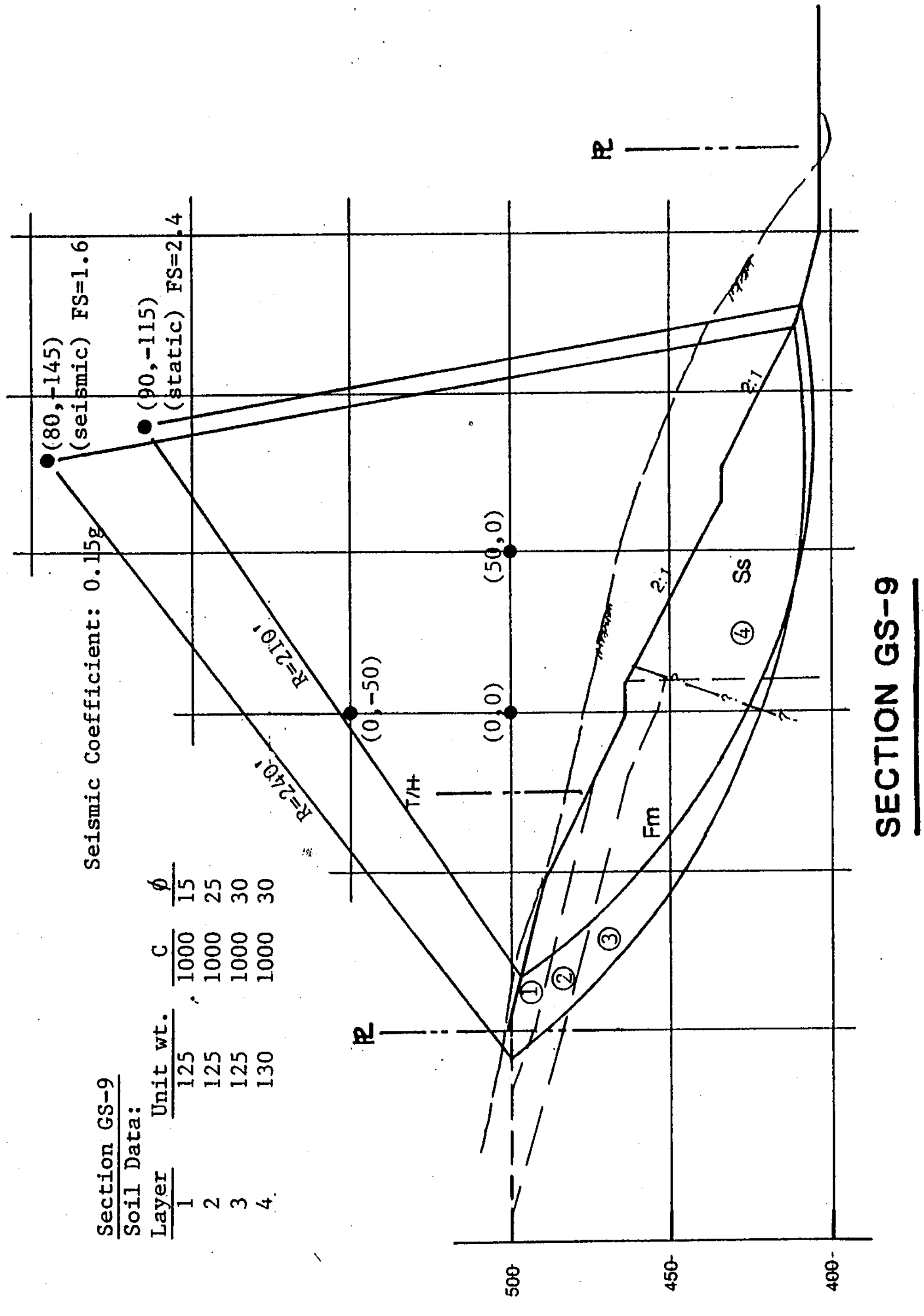
File No. S24-634-2S
November 7, 1994



GS-8: Cross Section Showing Critical Circles

SOIL FOUNDATION SYSTEMS, INC.

File No. S24-634-2S
November 7, 1994



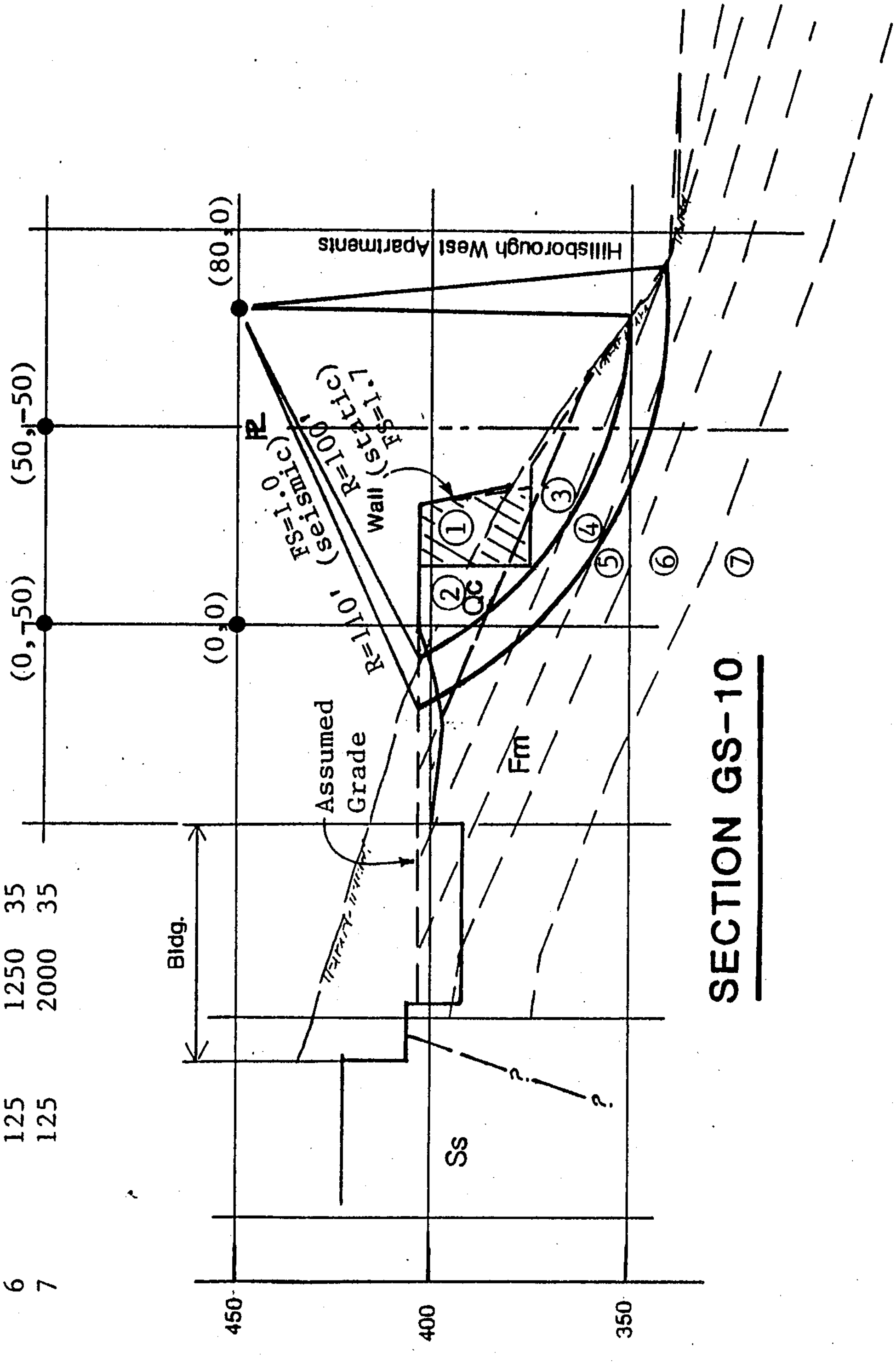
GS-9: Cross Section Showing Critical Circles

File No. S24-634-2S
November 7, 1994

Section GS-10
Soil Data:

Layer	Unit wt.	C	Ø
1	130	4000	30
2	130	1500	20
3	125	1000	15
4	125	1000	25
5	125	1000	30
6	125	1250	35
7	125	2000	35

Seismic Coefficient: 0.15g



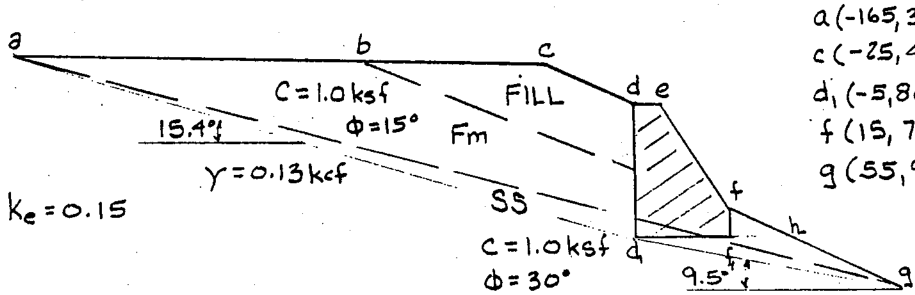
GS-10: Cross Section Showing Critical Circles

SOIL FOUNDATION SYSTEMS, INC.

PROJECT:	HIGHLAND ESTATES	Calculation By:	RPF	Date:	
		Checked By:		Date:	
FILE NO.	634	Sheet No.		of	

SLOPE STABILITY ANALYSIS — FAILURE ALONG STRATIFICATION

SECTION GS-1b



$k_e = 0.15$

a(-165,36) ; b(-75,38)
c(-25,40) ; d(-5,53)
d₁(-5,80) ; e(2,53)
f(15,75) ; f₁(15,80)
g(55,90) ; h(28.3,80)

$F_R = W\cos\alpha \tan\phi + c\ell$ $F_D = W\sin\alpha + k_e W\cos\alpha + P_a$ $P_a = \frac{1}{2}h^2(EFW)$

EFW = 70 pcf

FAILURE ALONG ad_1g

$acdd_1$: VoL = 3070, W = 399.1 $defgd_1$: VoL = 597, W = 77.6

WEDGE	W	α	ℓ	c	Φ	F_R	F_D STATIC	F_D SEISMIC
1	399.1	15.4	165.9	1.0	30	388	106	164
2	77.6	9.5	60.8	1.0	30	44	13	24
						432	119	188
						FS =	3.6	2.3

FAILURE ALONG dd_1h

$defhd_1$: VoL = 430.3, W = 55.9 $P_a = 25.5$, IGNORE PASSIVE AT TOE

$\alpha = 0°$ c = 1.0 ksf

$\ell = 33.3'$ $\phi = 30°$

F_R	F_D STATIC	F_D SEISMIC
66	26	34
FS =	2.5	1.9

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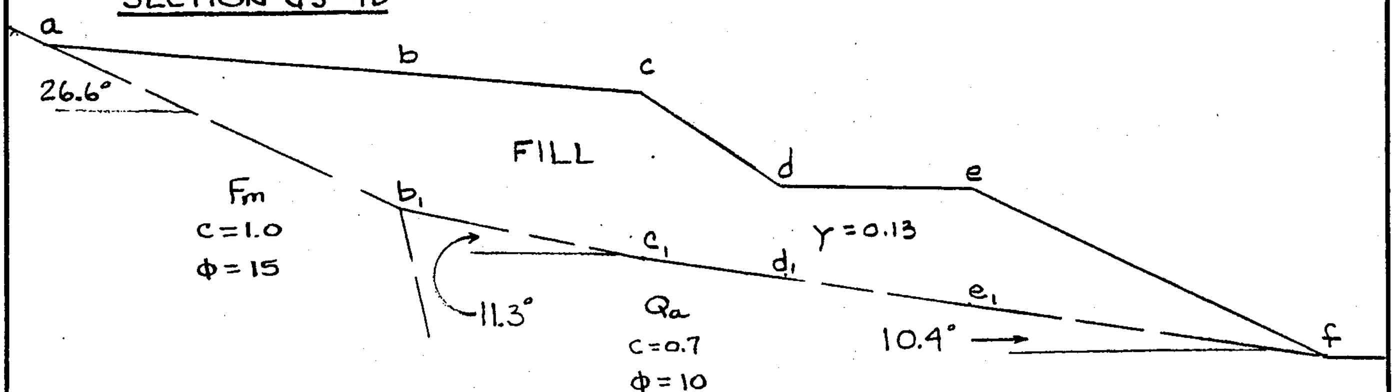
PROJECT:	HIGHLAND ESTATES	Calculation By:	RPF	Date:	
		Checked By:		Date:	
FILE NO.	634	Sheet No.		of	

SLOPE STABILITY ANALYSIS – FAILURE ALONG STRATIFICATION

SECTION GS-4b



a (-180,0); b (-80,10); b_1 (-80,50);   k=0.15
c (-5,15); c_1 (-5,65); d (35,45);
d_1 (35,73); e (85,45); e_1 (85,80); f (185,100)

FAILURE ALONG a b, c, f   (ASSUME NO INCREASE IN Q_a STRENGTH DUE TO COMPRESSION BY FILL)

abb_1: VOL = 2000
W = 260

bcc_1b_1 VOL = 3375
W = 439

$cdefc_1$ VOL = 4450
W = 579

$$F_R = W \cos\alpha \tan\phi + c\ell$$

$$F_D = W \sin\alpha + kW\cos\alpha$$

WEDGE	W	α	ℓ	c	φ	F_R	F_D STATIC	F_D SEISMIC
1	260	26.6	111.8	1.0	15	174	116	151
2	439	11.3	76.5	0.7	10	152	86	151
3	579	10.4	193.2	0.7	10	294	105	190
						620	308	492
						FS =	2.0	1.3

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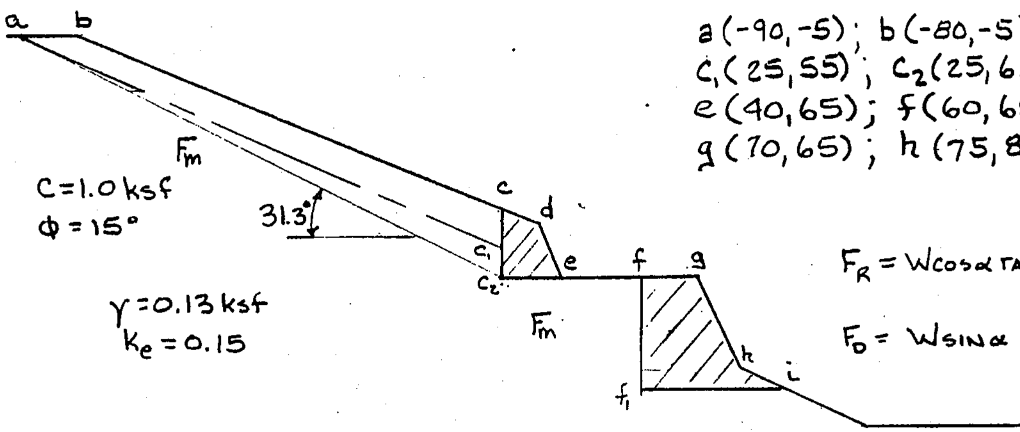
PROJECT:	HIGHLAND ESTATES	Calculation By:	RPF	Date:	
		Checked By:		Date:	
FILE NO.		Sheet No.		of	

SLOPE STABILITY ANALYSIS – FAILURE ALONG STRATIFICATION

SECTION GS-5



C = 1.0 ksf
φ = 15°

γ = 0.13 ksf
k_e = 0.15

a(-90,-5); b(-80,-5); c(25,45);
c_1(25,55); c_2(25,65); d(35,50);
e(40,65); f(60,65); f_1(60,90);
g(70,65); h(75,85); i(85,90)

$F_R = W\cos\alpha \tan\phi + c\ell$

$F_D = W\sin\alpha + k_e W\cos\alpha + P_a$

FAILURE ALONG ac_2e

$abcc_2$: VOL = 1400, W = 182 $cdec_2$: VOL = 212.5, W = 27.6

WEDGE	W	α	ℓ	c	φ	F_R	F_D STATIC	F_D SEISMIC
1	182	31.3	134.6	1.0	15	176	95	118
2	27.6	0	15	1.0	15	22	0	4
						198	95	122
						FS =	2.1	1.6

FAILURE ALONG cc_2e

$cdec_2$: VOL = 212.5, W = 27.6 P_a = 14

α = 0 C = 1.0 ksf

ℓ = 15' φ = 15°

F_R	F_D STATIC	F_D SEISMIC
22	14	18
FS =	1.6	1.2

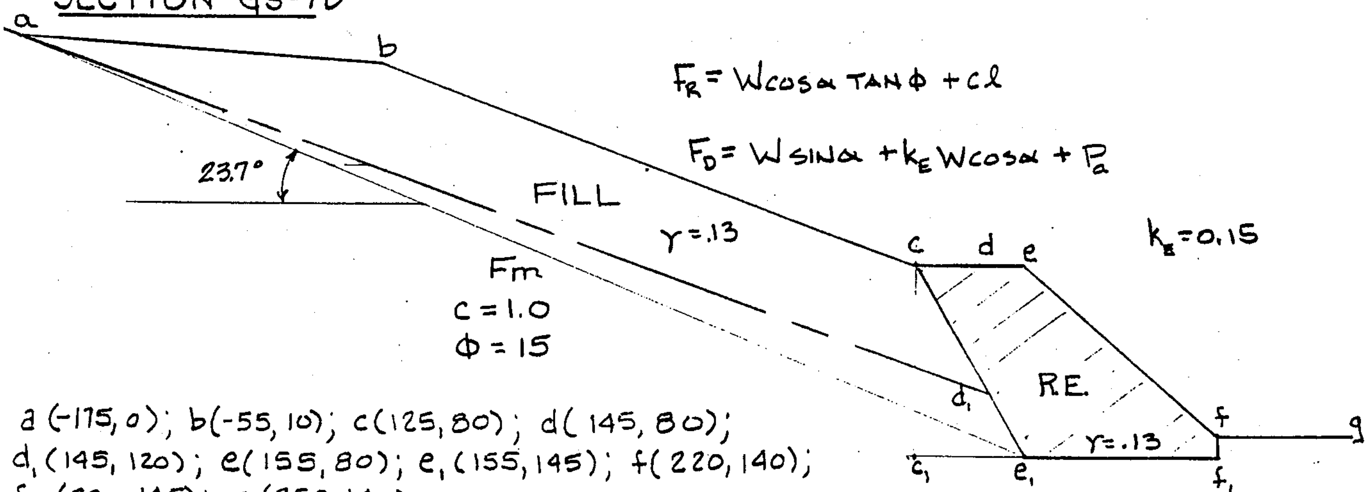
PROJECT:	HIGHLAND ESTATES	Calculation By:	RPF	Date:	
		Checked By:		Date:	
FILE NO.	634	Sheet No.		of	

SLOPE STABILITY ANALYSIS — FAILURE ALONG STRATIFICATION

SECTION GS-76

$F_R = W\cos\alpha \tan\phi + cl$

$F_D = W\sin\alpha + k_E W\cos\alpha + P_a$



a(-175, 0); b(-55, 10); c(125, 80); d(145, 80); d_1(145, 120); e(155, 80); e_1(155, 145); f(220, 140); f_1(220, 145); g(250, 140)

FAILURE ALONG ae_1f_1

(OMIT PASSIVE RESISTANCE AT TOE, AND PIERS BELOW REINFORCED EARTH)

$abce_1$: VOL = 11850, W = 1540.5 $ceff_1e_1$: VOL = 3250, W = 422.5

WEDGE	W	α	l	c	ϕ	F_R	F_D STATIC	F_D SEISMIC
1	1540.5	23.7	330	1.0	15	708	619	831
2	422.5	0	65	1.0	15	178	0	63
						886	619	894
						FS =	1.4	1.0

FAILURE ALONG cc_1f_1

cc_1f_1fe: VOL = 4225, W = 549

$\alpha = 0$, $l = 95'$, P_a = 148 kip/'

F_R	F_D STATIC	F_D SEISMIC
242	148	230
FS =	1.6	1.1

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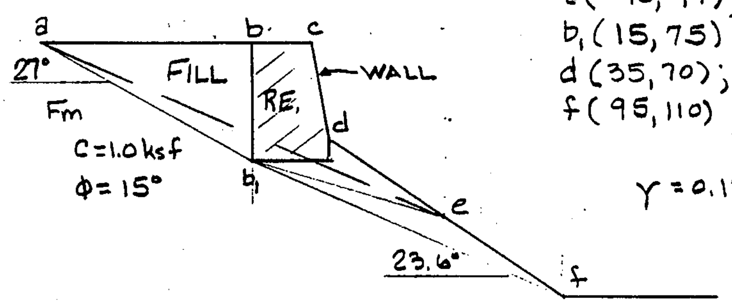
PROJECT:	HIGHLAND ESTATES	Calculation By:	RPF	Date:	
		Checked By:		Date:	
FILE NO.	634	Sheet No.		of	

SLOPE STABILITY ANALYSIS — FAILURE ALONG STRATIFICATION

SECTION GS-10



a(-40, 47); b(15, 47);
b_1(15, 75); c(30, 47);
d(35, 70); e(65, 90);
f(95, 110)

$\gamma = 0.13$ $k_e = 0.15$

$F_R = W \cos\alpha \tan\phi + c\ell$ $F_D = W \sin\alpha + k_e W \cos\alpha + P_a$

FAILURE ALONG ab_1f (IGNORE PIERS BELOW REINF. EARTH SECTION)

abb_1: VOL = 770, W = 100.1 $bcdfb_1$: VOL = 1002.5, W = 130.3

WEDGE	W	α	ℓ	c	ϕ	F_R	F_D STATIC	F_D SEISMIC
abb_1	100.1	27.0	61.7	1.0	15	85.6	45.4	58.8
bfb_1	130.3	23.6	87.3	1.0	15	119.3	52.2	70.1
						205	98	129
						FS =	2.1	1.6

FAILURE ALONG bb_1f (IGNORE PIERS)

$bcdefb_1$: VOL = 1002.5, W = 130.3, P_a = 27.4 kip/'

$\alpha = 23.6°$ C = 1.0 ksf

$\ell = 87.3'$ $\phi = 15°$

F_R	F_D STATIC	F_D SEISMIC
119.3	79.6	97.5
FS =	1.5	1.2

Problem Title: Highland Estates: Profile GS-6b

User's Name:P. F.
Date: 06-07-1993

SECTION GS-6b

(2:1 SLOPE ALTERNATIVE)

STATIC

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 11

Section	X	Y-crk.	Y-grd.
1	-110.0	10.0	10.0
2	-85.0	10.0	10.0
3	-60.0	15.0	15.0
4	-20.0	15.0	15.0
5	40.0	38.0	38.0
6	85.0	38.0	38.0
7	98.0	44.0	44.0
8	107.0	48.0	48.0
9	123.0	55.0	55.0
10	133.0	53.0	53.0
11	300.0	53.0	53.0

CIRCLE DATA

Coordinates of first circle (X,Y): 50 -140
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 5
Y-direction: 5

Elevation of upper-most tangent: 70
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

No seismic analysis
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	1500.0	20.0	130.0
2	700.0	10.0	125.0
3	1000.0	15.0	125.0
4	1000.0	25.0	125.0
5	1000.0	30.0	125.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer / layer number 1	2	3	4	5
1	-110.0	10.0	10.0	10.0	10.0	200.0
2	-85.0	10.0	10.0	10.0	20.0	200.0
3	-60.0	15.0	15.0	25.0	35.0	200.0
4	-20.0	23.0	32.0	42.0	52.0	200.0
5	40.0	50.0	60.0	70.0	80.0	200.0
6	85.0	67.0	80.0	90.0	100.0	200.0
7	98.0	73.0	85.0	98.0	108.0	200.0
8	107.0	77.0	90.0	100.0	110.0	200.0
9	123.0	68.0	81.0	91.0	101.0	200.0
10	133.0	62.0	72.0	82.0	92.0	200.0
11	300.0	62.0	72.0	82.0	92.0	200.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	210.0	50.0	-140.0	4.482	0.29780D+08	0.13348D+09	247.4
2	215.0	50.0	-140.0	4.574	0.34923D+08	0.15974D+09	269.5
3	220.0	50.0	-140.0	4.676	0.40866D+08	0.19108D+09	290.2
4	190.0	50.0	-120.0	4.489	0.25113D+08	0.11272D+09	232.0
5	195.0	50.0	-120.0	4.506	0.30361D+08	0.13681D+09	257.2
6	200.0	50.0	-120.0	4.656	0.35417D+08	0.16489D+09	277.3
7	170.0	50.0	-100.0	4.388	0.20967D+08	0.92000D+08	219.0
8	175.0	50.0	-100.0	4.428	0.25912D+08	0.11475D+09	244.2
9	180.0	50.0	-100.0	4.659	0.30067D+08	0.14007D+09	263.7
10	150.0	50.0	-80.0	4.396	0.17193D+08	0.75573D+08	205.9
11	155.0	50.0	-80.0	4.470	0.21170D+08	0.94627D+08	226.0
12	160.0	50.0	-80.0	4.509	0.25213D+08	0.11369D+09	248.6
13	130.0	50.0	-60.0	4.287	0.13668D+08	0.58590D+08	193.1
14	135.0	50.0	-60.0	4.412	0.16878D+08	0.74467D+08	208.1
15	140.0	50.0	-60.0	4.490	0.20536D+08	0.92196D+08	230.4
16	210.0	70.0	-140.0	4.225	0.26755D+08	0.11304D+09	244.7
17	215.0	70.0	-140.0	4.185	0.32510D+08	0.13606D+09	269.2
18	220.0	70.0	-140.0	4.430	0.38035D+08	0.16850D+09	290.3
19	190.0	70.0	-120.0	4.201	0.22858D+08	0.96016D+08	230.3
20	195.0	70.0	-120.0	4.046	0.27745D+08	0.11227D+09	254.0
21	200.0	70.0	-120.0	4.653	0.32845D+08	0.15281D+09	276.4
22	170.0	70.0	-100.0	4.020	0.19411D+08	0.78038D+08	217.1
23	175.0	70.0	-100.0	4.088	0.23693D+08	0.96851D+08	238.3
24	180.0	70.0	-100.0	4.567	0.27637D+08	0.12621D+09	260.0
25	150.0	70.0	-80.0	3.877	0.16086D+08	0.62366D+08	204.5
26	155.0	70.0	-80.0	4.066	0.19763D+08	0.80358D+08	224.4
27	160.0	70.0	-80.0	4.106	0.23408D+08	0.96112D+08	243.4
28	130.0	70.0	-60.0	3.591	0.12618D+08	0.45308D+08	191.0
29	135.0	70.0	-60.0	4.144	0.15657D+08	0.64880D+08	210.1
30	140.0	70.0	-60.0	3.814	0.19242D+08	0.73396D+08	228.4
31	210.0	90.0	-140.0	3.834	0.23427D+08	0.89818D+08	240.4
32	215.0	90.0	-140.0	3.616	0.28688D+08	0.10373D+09	262.4
33	220.0	90.0	-140.0	4.091	0.33513D+08	0.13711D+09	285.5
34	190.0	90.0	-120.0	3.783	0.19760D+08	0.74757D+08	229.0
35	195.0	90.0	-120.0	3.471	0.24644D+08	0.85531D+08	250.4
36	200.0	90.0	-120.0	3.997	0.29484D+08	0.11785D+09	270.7
37	170.0	90.0	-100.0	3.711	0.16330D+08	0.60600D+08	217.1
38	175.0	90.0	-100.0	3.141	0.20493D+08	0.64360D+08	237.7

39	180.0	90.0	-100.0	3.742	0.24791D+08	0.92774D+08	257.4
40	150.0	90.0	-80.0	3.733	0.12872D+08	0.48053D+08	204.5
41	155.0	90.0	-80.0	3.199	0.16633D+08	0.53204D+08	224.4
42	160.0	90.0	-80.0	3.544	0.20352D+08	0.72121D+08	243.4
43	130.0	90.0	-60.0	4.309	0.94426D+07	0.40691D+08	188.6
44	135.0	90.0	-60.0	3.409	0.12554D+08	0.42792D+08	210.1
45	140.0	90.0	-60.0	3.623	0.16117D+08	0.58388D+08	228.4
46	210.0	110.0	-140.0	3.892	0.18555D+08	0.72225D+08	240.4
47	215.0	110.0	-140.0	3.570	0.23360D+08	0.83388D+08	262.4
48	220.0	110.0	-140.0	3.875	0.28457D+08	0.11028D+09	283.4
49	190.0	110.0	-120.0	4.475	0.14772D+08	0.66110D+08	229.0
50	195.0	110.0	-120.0	3.621	0.19741D+08	0.71474D+08	250.4
51	200.0	110.0	-120.0	3.642	0.24326D+08	0.88605D+08	270.7
52	170.0	110.0	-100.0	4.833	0.11257D+08	0.54401D+08	213.2
53	175.0	110.0	-100.0	3.893	0.15429D+08	0.60059D+08	237.7
54	180.0	110.0	-100.0	3.755	0.19978D+08	0.75013D+08	257.4
55	150.0	110.0	-80.0	5.083	0.85464D+07	0.43438D+08	193.9
56	155.0	110.0	-80.0	4.053	0.11567D+08	0.46883D+08	219.1
57	160.0	110.0	-80.0	3.885	0.15274D+08	0.59346D+08	242.5
58	130.0	110.0	-60.0	5.741	0.63099D+07	0.36228D+08	174.2
59	135.0	110.0	-60.0	4.513	0.87198D+07	0.39349D+08	198.6
60	140.0	110.0	-60.0	4.329	0.11404D+08	0.49370D+08	221.3
61	210.0	130.0	-140.0	7.176	0.11544D+08	0.82838D+08	231.8
62	215.0	130.0	-140.0	4.862	0.16154D+08	0.78542D+08	261.5
63	220.0	130.0	-140.0	4.217	0.21356D+08	0.90063D+08	283.4
64	190.0	130.0	-120.0	8.254	0.88919D+07	0.73397D+08	213.1
65	195.0	130.0	-120.0	5.674	0.12765D+08	0.72431D+08	242.3
66	200.0	130.0	-120.0	4.695	0.17023D+08	0.79920D+08	268.7
67	170.0	130.0	-100.0	7.814	0.72174D+07	0.56399D+08	194.1
68	175.0	130.0	-100.0	6.337	0.97691D+07	0.61911D+08	222.8
69	180.0	130.0	-100.0	5.006	0.13265D+08	0.66396D+08	248.7
70	150.0	130.0	-80.0	7.903	0.57014D+07	0.45060D+08	174.8
71	155.0	130.0	-80.0	6.436	0.78015D+07	0.50208D+08	203.0
72	160.0	130.0	-80.0	5.522	0.10182D+08	0.56224D+08	228.3
73	130.0	130.0	-60.0	7.496	0.47917D+07	0.35920D+08	160.3
74	135.0	130.0	-60.0	7.085	0.60923D+07	0.43165D+08	182.9
75	140.0	130.0	-60.0	5.726	0.78327D+07	0.44853D+08	207.5

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	175.0	90.0	-100.0	3.141
2	155.0	90.0	-80.0	3.199
3	135.0	90.0	-60.0	3.409
4	195.0	90.0	-120.0	3.471
5	160.0	90.0	-80.0	3.544
6	215.0	110.0	-140.0	3.570
7	130.0	70.0	-60.0	3.591
8	215.0	90.0	-140.0	3.616
9	195.0	110.0	-120.0	3.621
10	140.0	90.0	-60.0	3.623

Problem Title: Highland Estates: Slope No. GS-1b

User's Name:P. F.
Date: 03-04-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 11

SECTION GS-1b

SEISMIC 0.2g

COHESION INCREASE

Section	X	Y-crk.	Y-grd.
1	-170.0	35.0	35.0
2	-75.0	40.0	40.0
3	-25.0	40.0	40.0
4	-5.0	55.0	55.0
5	-5.0	55.0	55.0
6	9.0	55.0	55.0
7	10.0	65.0	65.0
8	20.0	80.0	80.0
9	55.0	95.0	95.0
10	90.0	110.0	110.0
11	200.0	120.0	120.0

CIRCLE DATA

Coordinates of first circle (X,Y): 20 -60
Intervals of circle coordinates
X-direction: 10
Y-direction: 10
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 80
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .2
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	30.0	130.0
5	3000.0	35.0	130.0

FILE

X	elevation to bottom of layer — layer number 1	2	3	4	5
-170.0	35.0	35.0	35.0	45.0	300.0
-75.0	40.0	40.0	60.0	70.0	300.0
-25.0	40.0	60.0	75.0	85.0	300.0
-5.0	55.0	70.0	80.0	90.0	300.0
-5.0	80.0	80.0	80.0	90.0	300.0
9.0	80.0	80.0	82.0	92.0	300.0
10.0	80.0	80.0	83.0	93.0	300.0
20.0	80.0	80.0	85.0	95.0	300.0
55.0	95.0	95.0	95.0	105.0	300.0
90.0	110.0	110.0	110.0	110.0	300.0
200.0	120.0	120.0	120.0	140.0	300.0

radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
140.0	20.0	-60.0	2.199	0.20264D+08	0.44563D+08	108.7
145.0	20.0	-60.0	1.352	0.35864D+08	0.48481D+08	128.8
150.0	20.0	-60.0	1.423	0.44239D+08	0.62931D+08	147.5
130.0	20.0	-50.0	2.163	0.18306D+08	0.39588D+08	104.7
135.0	20.0	-50.0	1.339	0.32215D+08	0.43132D+08	124.6
140.0	20.0	-50.0	1.348	0.41732D+08	0.56265D+08	142.9
120.0	20.0	-40.0	2.129	0.16387D+08	0.34894D+08	100.9
125.0	20.0	-40.0	1.327	0.27849D+08	0.36948D+08	120.1
130.0	20.0	-40.0	1.338	0.37425D+08	0.50059D+08	138.3
140.0	30.0	-60.0	2.220	0.17980D+08	0.39922D+08	98.2
145.0	30.0	-60.0	1.311	0.30024D+08	0.39367D+08	119.0
150.0	30.0	-60.0	1.339	0.40054D+08	0.53633D+08	138.7
130.0	30.0	-50.0	2.194	0.16105D+08	0.35329D+08	94.5
135.0	30.0	-50.0	1.234	0.27264D+08	0.33639D+08	115.1
140.0	30.0	-50.0	1.329	0.36001D+08	0.47828D+08	134.3
120.0	30.0	-40.0	2.177	0.14204D+08	0.30928D+08	90.6
125.0	30.0	-40.0	1.278	0.23802D+08	0.30430D+08	111.1
130.0	30.0	-40.0	1.322	0.31981D+08	0.42270D+08	129.9
140.0	40.0	-60.0	2.365	0.14863D+08	0.35145D+08	87.4
145.0	40.0	-60.0	1.603	0.22579D+08	0.36185D+08	108.4
150.0	40.0	-60.0	1.313	0.34188D+08	0.44905D+08	129.3
130.0	40.0	-50.0	2.519	0.12663D+08	0.31895D+08	83.5
135.0	40.0	-50.0	1.602	0.20072D+08	0.32146D+08	104.4
140.0	40.0	-50.0	1.309	0.30483D+08	0.39887D+08	125.3
120.0	40.0	-40.0	2.543	0.10888D+08	0.27683D+08	79.5
125.0	40.0	-40.0	1.615	0.17496D+08	0.28260D+08	100.3
130.0	40.0	-40.0	1.294	0.28055D+08	0.36307D+08	121.1

CRITICAL CIRCLES

radius	X-center	Y-center	F.S.
135.0	30.0	-50.0	1.234
125.0	30.0	-40.0	1.278
130.0	40.0	-40.0	1.294
140.0	40.0	-50.0	1.309
145.0	30.0	-60.0	1.311
150.0	40.0	-60.0	1.313
130.0	30.0	-40.0	1.322
125.0	20.0	-40.0	1.327
140.0	30.0	-50.0	1.329

Problem Title: Highland Estates: Slope No. GS-1b

User's Name: P. F.
Date: 03-04-1994

SECTION GS-1b

SEISMIC 0.21g

COHESION INCREASE

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 11

Section	X	Y-crk.	Y-grd.
1	-170.0	35.0	35.0
2	-75.0	40.0	40.0
3	-25.0	40.0	40.0
4	-5.0	55.0	55.0
5	-5.0	55.0	55.0
6	9.0	55.0	55.0
7	10.0	65.0	65.0
8	20.0	80.0	80.0
9	55.0	95.0	95.0
10	90.0	110.0	110.0
11	200.0	120.0	120.0

CIRCLE DATA

Coordinates of first circle (X,Y): 20 -60
Intervals of circle coordinates
X-direction: 10
Y-direction: 10
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 80
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .21
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	30.0	130.0
5	3000.0	35.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer layer number 1	2	3	4	5
1	-170.0	35.0	35.0	35.0	45.0	300.0
2	-75.0	40.0	40.0	60.0	70.0	300.0
3	-25.0	40.0	60.0	75.0	85.0	300.0
4	-5.0	55.0	70.0	80.0	90.0	300.0
5	-5.0	80.0	80.0	80.0	90.0	300.0
6	9.0	80.0	80.0	82.0	92.0	300.0
7	10.0	80.0	80.0	83.0	93.0	300.0
8	20.0	80.0	80.0	85.0	95.0	300.0
9	55.0	95.0	95.0	95.0	105.0	300.0
10	90.0	110.0	110.0	110.0	110.0	300.0
11	200.0	120.0	120.0	120.0	140.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	140.0	20.0	-60.0	2.152	0.20692D+08	0.44520D+08	108.7
2	145.0	20.0	-60.0	1.310	0.36882D+08	0.48324D+08	128.8
3	150.0	20.0	-60.0	1.380	0.45450D+08	0.62736D+08	147.5
4	130.0	20.0	-50.0	2.116	0.18687D+08	0.39548D+08	104.7
5	135.0	20.0	-50.0	1.298	0.33116D+08	0.42989D+08	124.6
6	140.0	20.0	-50.0	1.307	0.42897D+08	0.56075D+08	142.9
7	120.0	20.0	-40.0	2.084	0.16722D+08	0.34857D+08	100.9
8	125.0	20.0	-40.0	1.288	0.28595D+08	0.36839D+08	120.1
9	130.0	20.0	-40.0	1.297	0.38454D+08	0.49882D+08	138.3
10	140.0	30.0	-60.0	2.173	0.18347D+08	0.39873D+08	98.2
11	145.0	30.0	-60.0	1.274	0.30805D+08	0.39258D+08	119.0
12	150.0	30.0	-60.0	1.300	0.41115D+08	0.53461D+08	138.7
13	130.0	30.0	-50.0	2.148	0.16427D+08	0.35283D+08	94.5
14	135.0	30.0	-50.0	1.200	0.27966D+08	0.33546D+08	115.1
15	140.0	30.0	-50.0	1.291	0.36938D+08	0.47671D+08	134.3
16	120.0	30.0	-40.0	2.133	0.14482D+08	0.30886D+08	90.6
17	125.0	30.0	-40.0	1.244	0.24396D+08	0.30341D+08	111.1
18	130.0	30.0	-40.0	1.285	0.32798D+08	0.42129D+08	129.9
19	140.0	40.0	-60.0	2.316	0.15153D+08	0.35096D+08	87.4
20	145.0	40.0	-60.0	1.566	0.23066D+08	0.36111D+08	108.4
21	150.0	40.0	-60.0	1.278	0.35040D+08	0.44767D+08	129.3
22	130.0	40.0	-50.0	2.470	0.12894D+08	0.31852D+08	83.5
23	135.0	40.0	-50.0	1.565	0.20496D+08	0.32078D+08	104.4
24	140.0	40.0	-50.0	1.273	0.31227D+08	0.39762D+08	125.3
25	120.0	40.0	-40.0	2.495	0.11081D+08	0.27645D+08	79.5
26	125.0	40.0	-40.0	1.579	0.17854D+08	0.28200D+08	100.3
27	130.0	40.0	-40.0	1.258	0.28750D+08	0.36177D+08	121.1

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	135.0	30.0	-50.0	1.200
2	125.0	30.0	-40.0	1.244
3	130.0	40.0	-40.0	1.258
4	140.0	40.0	-50.0	1.273
5	145.0	30.0	-60.0	1.274
6	150.0	40.0	-60.0	1.278
7	130.0	30.0	-40.0	1.285
8	125.0	20.0	-40.0	1.288
9	140.0	30.0	-50.0	1.291
10	130.0	20.0	-40.0	1.297

Problem Title: Highland Estates: Slope No. GS-1b

User's Name:P. F.
Date: 03-04-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

SECTION 1-b

COHESION INCREASE
SEISMIC 0.22g

GEOMETRY

Number of Sections : 11

Section	X	Y-crk.	Y-grd.
1	-170.0	35.0	35.0
2	-75.0	40.0	40.0
3	-25.0	40.0	40.0
4	-5.0	55.0	55.0
5	-5.0	55.0	55.0
6	9.0	55.0	55.0
7	10.0	65.0	65.0
8	20.0	80.0	80.0
9	55.0	95.0	95.0
10	90.0	110.0	110.0
11	200.0	120.0	120.0

CIRCLE DATA

Coordinates of first circle (X,Y): 10 -70
Intervals of circle coordinates
X-direction: 10
Y-direction: 10
Number of intervals
X-direction: 5
Y-direction: 9

Elevation of upper-most tangent: 80
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .22
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	30.0	130.0
5	3000.0	35.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4	5
1	-170.0	35.0	35.0	35.0	45.0	300.0
2	-75.0	40.0	40.0	60.0	70.0	300.0
3	-25.0	40.0	60.0	75.0	85.0	300.0
4	-5.0	55.0	70.0	80.0	90.0	300.0
5	-5.0	80.0	80.0	80.0	90.0	300.0
6	9.0	80.0	80.0	82.0	92.0	300.0
7	10.0	80.0	80.0	83.0	93.0	300.0
8	20.0	80.0	80.0	85.0	95.0	300.0
9	55.0	95.0	95.0	95.0	105.0	300.0
10	90.0	110.0	110.0	110.0	110.0	300.0
11	200.0	120.0	120.0	120.0	140.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	150.0	10.0	-70.0	1.853	0.30076D+08	0.55723D+08	123.3
2	155.0	10.0	-70.0	1.317	0.47226D+08	0.62198D+08	142.0
3	160.0	10.0	-70.0	1.436	0.63736D+08	0.91556D+08	160.1
4	140.0	10.0	-60.0	1.827	0.27378D+08	0.50022D+08	119.3
5	145.0	10.0	-60.0	1.317	0.42191D+08	0.55571D+08	137.7
6	150.0	10.0	-60.0	1.424	0.58212D+08	0.82904D+08	155.6
7	130.0	10.0	-50.0	1.827	0.23655D+08	0.43226D+08	115.2
8	135.0	10.0	-50.0	1.304	0.38152D+08	0.49737D+08	133.3
9	140.0	10.0	-50.0	1.406	0.49804D+08	0.70009D+08	150.9
10	120.0	10.0	-40.0	1.862	0.19095D+08	0.35561D+08	110.9
11	125.0	10.0	-40.0	1.300	0.33231D+08	0.43192D+08	128.7
12	130.0	10.0	-40.0	1.387	0.42675D+08	0.59205D+08	146.0
13	110.0	10.0	-30.0	2.102	0.16541D+08	0.34774D+08	106.5
14	115.0	10.0	-30.0	1.291	0.29331D+08	0.37856D+08	124.0
15	120.0	10.0	-30.0	1.367	0.36134D+08	0.49401D+08	141.0
16	100.0	10.0	-20.0	2.075	0.14508D+08	0.30109D+08	102.3
17	105.0	10.0	-20.0	1.279	0.25647D+08	0.32809D+08	119.1
18	110.0	10.0	-20.0	1.361	0.31747D+08	0.43199D+08	135.8
19	90.0	10.0	-10.0	2.050	0.12529D+08	0.25681D+08	97.9
20	95.0	10.0	-10.0	1.270	0.22124D+08	0.28089D+08	114.2
21	100.0	10.0	-10.0	1.298	0.28716D+08	0.37270D+08	130.3
22	80.0	10.0	0.0	2.118	0.10467D+08	0.22168D+08	93.2
23	85.0	10.0	0.0	1.210	0.19424D+08	0.23506D+08	109.1
24	90.0	10.0	0.0	1.298	0.24472D+08	0.31775D+08	124.8
25	70.0	10.0	10.0	2.130	0.85070D+07	0.18119D+08	88.2
26	75.0	10.0	10.0	1.214	0.15951D+08	0.19360D+08	103.7
27	80.0	10.0	10.0	1.317	0.19683D+08	0.25917D+08	119.0
28	150.0	20.0	-70.0	1.864	0.23810D+08	0.44388D+08	112.7
29	155.0	20.0	-70.0	1.282	0.41920D+08	0.53750D+08	133.0
30	160.0	20.0	-70.0	1.352	0.51288D+08	0.69322D+08	151.9
31	140.0	20.0	-60.0	2.106	0.21121D+08	0.44476D+08	108.7
32	145.0	20.0	-60.0	1.271	0.37899D+08	0.48168D+08	128.8
33	150.0	20.0	-60.0	1.340	0.46660D+08	0.62543D+08	147.5
34	130.0	20.0	-50.0	2.072	0.19068D+08	0.39508D+08	104.7
35	135.0	20.0	-50.0	1.260	0.34017D+08	0.42846D+08	124.6
36	140.0	20.0	-50.0	1.268	0.44062D+08	0.55884D+08	142.9
37	120.0	20.0	-40.0	2.041	0.17057D+08	0.34819D+08	100.9
38	125.0	20.0	-40.0	1.252	0.29341D+08	0.36730D+08	120.1
39	130.0	20.0	-40.0	1.259	0.39484D+08	0.49710D+08	138.3
40	110.0	20.0	-30.0	2.086	0.14986D+08	0.31260D+08	96.8
41	115.0	20.0	-30.0	1.239	0.25929D+08	0.32124D+08	115.8
42	120.0	20.0	-30.0	1.271	0.33958D+08	0.43161D+08	133.4

42	120.0	20.0	-30.0	1.271	0.33958D+08	0.43161D+08	133.4
43	100.0	20.0	-20.0	2.068	0.13004D+08	0.26894D+08	92.6
44	105.0	20.0	-20.0	1.173	0.23555D+08	0.27629D+08	111.4
45	110.0	20.0	-20.0	1.267	0.29612D+08	0.37532D+08	128.5
46	90.0	20.0	-10.0	2.065	0.11016D+08	0.22751D+08	88.2
47	95.0	20.0	-10.0	1.222	0.19301D+08	0.23583D+08	106.7
48	100.0	20.0	-10.0	1.273	0.24757D+08	0.31512D+08	123.6
49	80.0	20.0	0.0	2.449	0.86666D+07	0.21223D+08	83.6
50	85.0	20.0	0.0	1.224	0.16195D+08	0.19818D+08	101.8
51	90.0	20.0	0.0	1.275	0.20923D+08	0.26670D+08	118.4
52	70.0	20.0	10.0	2.499	0.68141D+07	0.17029D+08	78.7
53	75.0	20.0	10.0	1.284	0.12379D+08	0.15893D+08	96.6
54	80.0	20.0	10.0	1.281	0.17688D+08	0.22661D+08	112.8
55	150.0	30.0	-70.0	2.083	0.20819D+08	0.43362D+08	101.9
56	155.0	30.0	-70.0	1.254	0.34977D+08	0.43852D+08	123.1
57	160.0	30.0	-70.0	1.273	0.46537D+08	0.59248D+08	143.0
58	140.0	30.0	-60.0	2.128	0.18715D+08	0.39823D+08	98.2
59	145.0	30.0	-60.0	1.239	0.31585D+08	0.39149D+08	119.0
60	150.0	30.0	-60.0	1.263	0.42176D+08	0.53289D+08	138.7
61	130.0	30.0	-50.0	2.104	0.16750D+08	0.35237D+08	94.5
62	135.0	30.0	-50.0	1.167	0.28669D+08	0.33454D+08	115.1
63	140.0	30.0	-50.0	1.255	0.37875D+08	0.47515D+08	134.3
64	120.0	30.0	-40.0	2.090	0.14761D+08	0.30844D+08	90.6
65	125.0	30.0	-40.0	1.211	0.24989D+08	0.30252D+08	111.1
66	130.0	30.0	-40.0	1.249	0.33614D+08	0.41988D+08	129.9
67	110.0	30.0	-30.0	2.195	0.12549D+08	0.27546D+08	86.5
68	115.0	30.0	-30.0	1.209	0.21916D+08	0.26497D+08	106.9
69	120.0	30.0	-30.0	1.245	0.29554D+08	0.36795D+08	125.5
70	100.0	30.0	-20.0	2.492	0.10344D+08	0.25773D+08	82.3
71	105.0	30.0	-20.0	1.241	0.18181D+08	0.22569D+08	102.6
72	110.0	30.0	-20.0	1.184	0.26794D+08	0.31735D+08	121.0
73	90.0	30.0	-10.0	2.720	0.81015D+07	0.22037D+08	77.8
74	95.0	30.0	-10.0	1.292	0.13926D+08	0.17995D+08	98.0
75	100.0	30.0	-10.0	1.245	0.21833D+08	0.27178D+08	116.2
76	80.0	30.0	0.0	2.807	0.64252D+07	0.18036D+08	73.2
77	85.0	30.0	0.0	1.654	0.10971D+08	0.18152D+08	93.3
78	90.0	30.0	0.0	1.274	0.17922D+08	0.22834D+08	111.2
79	70.0	30.0	10.0	3.544	0.45499D+07	0.16125D+08	68.2
80	75.0	30.0	10.0	1.730	0.86245D+07	0.14924D+08	88.2
81	80.0	30.0	10.0	1.352	0.13749D+08	0.18593D+08	105.8
82	150.0	40.0	-70.0	2.160	0.17742D+08	0.38329D+08	91.1
83	155.0	40.0	-70.0	1.470	0.26433D+08	0.38863D+08	112.2
84	160.0	40.0	-70.0	1.248	0.40132D+08	0.50087D+08	133.3
85	140.0	40.0	-60.0	2.269	0.15443D+08	0.35047D+08	87.4
86	145.0	40.0	-60.0	1.530	0.23552D+08	0.36037D+08	108.4
87	150.0	40.0	-60.0	1.243	0.35892D+08	0.44628D+08	129.3
88	130.0	40.0	-50.0	2.423	0.13126D+08	0.31808D+08	83.5
89	135.0	40.0	-50.0	1.530	0.20919D+08	0.32011D+08	104.4
90	140.0	40.0	-50.0	1.240	0.31971D+08	0.39637D+08	125.3
91	120.0	40.0	-40.0	2.449	0.11274D+08	0.27607D+08	79.5
92	125.0	40.0	-40.0	1.545	0.18212D+08	0.28139D+08	100.3
93	130.0	40.0	-40.0	1.224	0.29446D+08	0.36046D+08	121.1
94	110.0	40.0	-30.0	3.040	0.88208D+07	0.26811D+08	75.3
95	115.0	40.0	-30.0	1.758	0.14422D+08	0.25356D+08	96.1
96	120.0	40.0	-30.0	1.238	0.24744D+08	0.30631D+08	116.8
97	100.0	40.0	-20.0	3.640	0.65188D+07	0.23728D+08	70.9
98	105.0	40.0	-20.0	2.103	0.11705D+08	0.24614D+08	91.7
99	110.0	40.0	-20.0	1.299	0.20101D+08	0.26112D+08	112.4
100	90.0	40.0	-10.0	3.946	0.49235D+07	0.19428D+08	66.3
101	95.0	40.0	-10.0	2.204	0.94592D+07	0.20848D+08	87.0
102	100.0	40.0	-10.0	1.344	0.16512D+08	0.22200D+08	107.7
103	80.0	40.0	0.0	4.645	0.35426D+07	0.16455D+08	61.3
104	85.0	40.0	0.0	2.520	0.71633D+07	0.18049D+08	82.1
105	90.0	40.0	0.0	1.629	0.13132D+08	0.21395D+08	102.8
106	70.0	40.0	10.0	5.775	0.22451D+07	0.12965D+08	53.7
107	75.0	40.0	10.0	3.464	0.48127D+07	0.16670D+08	76.8
108	80.0	40.0	10.0	1.844	0.10049D+08	0.18535D+08	97.6
109	150.0	50.0	-70.0	2.964	0.12330D+08	0.36550D+08	79.5
110	155.0	50.0	-70.0	1.939	0.20099D+08	0.38968D+08	99.1
111	160.0	50.0	-70.0	1.302	0.32974D+08	0.42936D+08	122.9

111	160.0	50.0	-70.0	1.302	0.32974D+08	0.42936D+08	122.9
112	140.0	50.0	-60.0	3.495	0.97798D+07	0.34178D+08	75.6
113	145.0	50.0	-60.0	2.055	0.17427D+08	0.35812D+08	94.9
114	150.0	50.0	-60.0	1.536	0.27888D+08	0.42847D+08	119.0
115	130.0	50.0	-50.0	3.837	0.81771D+07	0.31372D+08	71.6
116	135.0	50.0	-50.0	2.084	0.15065D+08	0.31398D+08	90.5
117	140.0	50.0	-50.0	1.553	0.24539D+08	0.38112D+08	115.0
118	120.0	50.0	-40.0	4.003	0.66995D+07	0.26818D+08	67.4
119	125.0	50.0	-40.0	2.288	0.12356D+08	0.28266D+08	85.8
120	130.0	50.0	-40.0	1.625	0.20485D+08	0.33292D+08	110.8
121	110.0	50.0	-30.0	4.328	0.51996D+07	0.22504D+08	62.9
122	115.0	50.0	-30.0	3.221	0.93134D+07	0.29995D+08	80.7
123	120.0	50.0	-30.0	1.465	0.17652D+08	0.25853D+08	106.5
124	100.0	50.0	-20.0	5.229	0.37248D+07	0.19477D+08	58.0
125	105.0	50.0	-20.0	3.622	0.70850D+07	0.25664D+08	74.8
126	110.0	50.0	-20.0	1.821	0.13821D+08	0.25167D+08	102.0
127	90.0	50.0	-10.0	6.668	0.23104D+07	0.15406D+08	52.5
128	95.0	50.0	-10.0	4.186	0.51966D+07	0.21751D+08	67.4
129	100.0	50.0	-10.0	1.920	0.11212D+08	0.21528D+08	97.3
130	80.0	50.0	0.0	8.853	0.11924D+07	0.10556D+08	36.6
131	85.0	50.0	0.0	5.208	0.34453D+07	0.17943D+08	60.7
132	90.0	50.0	0.0	2.827	0.75305D+07	0.21290D+08	92.3
133	70.0	50.0	10.0	9.685	0.66627D+06	0.64525D+07	22.0
134	75.0	50.0	10.0	6.516	0.19256D+07	0.12547D+08	46.3
135	80.0	50.0	10.0	3.551	0.48924D+07	0.17372D+08	86.1

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	135.0	30.0	-50.0	1.167
2	105.0	20.0	-20.0	1.173
3	110.0	30.0	-20.0	1.184
4	115.0	30.0	-30.0	1.209
5	85.0	10.0	0.0	1.210
6	125.0	30.0	-40.0	1.211
7	75.0	10.0	10.0	1.214
8	95.0	20.0	-10.0	1.222
9	85.0	20.0	0.0	1.224
10	130.0	40.0	-40.0	1.224

Problem Title: Highland Estates: GS-4a

User's Name: P.F.
Date: 03-04-1994

SECTION GS-4a
SEISMIC 0.29
COHESION INCREASE

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 8

Section	X	Y-crk.	Y-grd.
1	-200.0	0.0	0.0
2	-100.0	0.0	0.0
3	-40.0	15.0	15.0
4	-40.0	15.0	15.0
5	20.0	40.0	40.0
6	50.0	40.0	40.0
7	190.0	110.0	110.0
8	350.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 185 -110
Intervals of circle coordinates
X-direction: 10
Y-direction: 10
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .2
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	2250.0	20.0	130.0
2	600.0	20.0	130.0
3	600.0	30.0	130.0
4	1500.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4
1	-200.0	0.0	0.0	0.0	300.0
2	-100.0	0.0	0.0	0.0	300.0
3	-40.0	15.0	15.0	15.0	300.0
4	-40.0	15.0	15.0	300.0	300.0
5	20.0	40.0	40.0	300.0	300.0
6	50.0	40.0	55.0	300.0	300.0
7	190.0	110.0	120.0	300.0	300.0
8	350.0	175.0	185.0	300.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	215.0	185.0	-110.0	1.142	0.44779D+08	0.51146D+08	166.5
2	220.0	185.0	-110.0	1.120	0.61291D+08	0.68653D+08	185.3
3	225.0	185.0	-110.0	1.318	0.78245D+08	0.10316D+09	233.1
4	205.0	185.0	-100.0	1.146	0.39989D+08	0.45847D+08	162.7
5	210.0	185.0	-100.0	1.115	0.55707D+08	0.62103D+08	181.4
6	215.0	185.0	-100.0	1.308	0.71997D+08	0.94194D+08	226.0
7	195.0	185.0	-90.0	1.153	0.35454D+08	0.40867D+08	158.8
8	200.0	185.0	-90.0	1.122	0.49566D+08	0.55634D+08	177.4
9	205.0	185.0	-90.0	1.309	0.65469D+08	0.85689D+08	221.1
10	215.0	195.0	-110.0	1.185	0.33925D+08	0.40215D+08	155.4
11	220.0	195.0	-110.0	1.128	0.49290D+08	0.55610D+08	175.3
12	225.0	195.0	-110.0	1.482	0.67017D+08	0.99312D+08	229.3
13	205.0	195.0	-100.0	1.224	0.28698D+08	0.35122D+08	151.5
14	210.0	195.0	-100.0	1.133	0.44067D+08	0.49934D+08	171.4
15	215.0	195.0	-100.0	1.489	0.60929D+08	0.90709D+08	224.5
16	195.0	195.0	-90.0	1.291	0.23571D+08	0.30440D+08	147.5
17	200.0	195.0	-90.0	1.141	0.39063D+08	0.44579D+08	167.4
18	205.0	195.0	-90.0	1.512	0.54152D+08	0.81867D+08	219.6
19	215.0	205.0	-110.0	circle does not intercept slope			
20	220.0	205.0	-110.0	1.177	0.37325D+08	0.43935D+08	164.2
21	225.0	205.0	-110.0	1.574	0.55714D+08	0.87678D+08	227.8
22	205.0	205.0	-100.0	circle does not intercept slope			
23	210.0	205.0	-100.0	1.214	0.32126D+08	0.38995D+08	160.3
24	215.0	205.0	-100.0	1.603	0.49764D+08	0.79780D+08	223.0
25	195.0	205.0	-90.0	circle does not intercept slope			
26	200.0	205.0	-90.0	circle does not intercept slope			
27	205.0	205.0	-90.0	1.653	0.43587D+08	0.72065D+08	218.2

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	210.0	185.0	-100.0	1.115
2	220.0	185.0	-110.0	1.120
3	200.0	185.0	-90.0	1.122
4	220.0	195.0	-110.0	1.128
5	210.0	195.0	-100.0	1.133
6	200.0	195.0	-90.0	1.141
7	215.0	185.0	-110.0	1.142
8	205.0	185.0	-100.0	1.146
9	195.0	185.0	-90.0	1.153
10	220.0	205.0	-110.0	1.177

Problem Title: Highland Estates: GS-4a

User's Name:P.F.
Date: 03-04-1994

SECTION GS-4a
- SEISMIC 0.21g
- COHESION INCREASE

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 8

Section	X	Y-crk.	Y-grd.
1	-200.0	0.0	0.0
2	-100.0	0.0	0.0
3	-40.0	15.0	15.0
4	-40.0	15.0	15.0
5	20.0	40.0	40.0
6	50.0	40.0	40.0
7	190.0	110.0	110.0
8	350.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 185 -110
Intervals of circle coordinates
X-direction: 10
Y-direction: 10
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .21
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	2250.0	20.0	130.0
2	600.0	20.0	130.0
3	600.0	30.0	130.0
4	1500.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4
1	-200.0	0.0	0.0	0.0	300.0
2	-100.0	0.0	0.0	0.0	300.0
3	-40.0	15.0	15.0	15.0	300.0
4	-40.0	15.0	15.0	300.0	300.0
5	20.0	40.0	40.0	300.0	300.0
6	50.0	40.0	55.0	300.0	300.0
7	190.0	110.0	120.0	300.0	300.0
8	350.0	175.0	185.0	300.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	215.0	185.0	-110.0	1.115	0.45687D+08	0.50931D+08	166.5
2	220.0	185.0	-110.0	1.093	0.62538D+08	0.68356D+08	185.3
3	225.0	185.0	-110.0	1.289	0.79820D+08	0.10285D+09	233.1
4	205.0	185.0	-100.0	1.119	0.40798D+08	0.45650D+08	162.7
5	210.0	185.0	-100.0	1.088	0.56839D+08	0.61827D+08	181.4
6	215.0	185.0	-100.0	1.279	0.73443D+08	0.93911D+08	226.0
7	195.0	185.0	-90.0	1.125	0.36171D+08	0.40690D+08	158.8
8	200.0	185.0	-90.0	1.096	0.50556D+08	0.55398D+08	177.4
9	205.0	185.0	-90.0	1.279	0.66782D+08	0.85428D+08	221.1
10	215.0	195.0	-110.0	1.157	0.34605D+08	0.40055D+08	155.4
11	220.0	195.0	-110.0	1.102	0.50267D+08	0.55373D+08	175.3
12	225.0	195.0	-110.0	1.449	0.68362D+08	0.99070D+08	229.3
13	205.0	195.0	-100.0	1.196	0.29255D+08	0.34990D+08	151.5
14	210.0	195.0	-100.0	1.106	0.44939D+08	0.49718D+08	171.4
15	215.0	195.0	-100.0	1.456	0.62151D+08	0.90485D+08	224.5
16	195.0	195.0	-90.0	1.263	0.24014D+08	0.30339D+08	147.5
17	200.0	195.0	-90.0	1.114	0.39837D+08	0.44384D+08	167.4
18	205.0	195.0	-90.0	1.480	0.55211D+08	0.81684D+08	219.6
19	215.0	205.0	-110.0	circle does not intercept slope			
20	220.0	205.0	-110.0	1.150	0.38060D+08	0.43758D+08	164.2
21	225.0	205.0	-110.0	1.539	0.56842D+08	0.87490D+08	227.8
22	205.0	205.0	-100.0	circle does not intercept slope			
23	210.0	205.0	-100.0	1.185	0.32769D+08	0.38837D+08	160.3
24	215.0	205.0	-100.0	1.568	0.50776D+08	0.79611D+08	223.0
25	195.0	205.0	-90.0	circle does not intercept slope			
26	200.0	205.0	-90.0	circle does not intercept slope			
27	205.0	205.0	-90.0	1.617	0.44487D+08	0.71914D+08	218.2

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	210.0	185.0	-100.0	1.088
2	220.0	185.0	-110.0	1.093
3	200.0	185.0	-90.0	1.096
4	220.0	195.0	-110.0	1.102
5	210.0	195.0	-100.0	1.106
6	200.0	195.0	-90.0	1.114
7	215.0	185.0	-110.0	1.115
8	205.0	185.0	-100.0	1.119
9	195.0	185.0	-90.0	1.125
10	220.0	205.0	-110.0	1.150

Problem Title: Highland Estates: GS-4a

User's Name:P.F.
Date: 03-04-1994

SECTION GS-4a
SEISMIC 0.22g
COHESION INCREASE

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 8

Section	X	Y-crk.	Y-grd.
1	-200.0	0.0	0.0
2	-100.0	0.0	0.0
3	-40.0	15.0	15.0
4	-40.0	15.0	15.0
5	20.0	40.0	40.0
6	50.0	40.0	40.0
7	190.0	110.0	110.0
8	350.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 185 -110
Intervals of circle coordinates
X-direction: 10
Y-direction: 10
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .22
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	2250.0	20.0	130.0
2	600.0	20.0	130.0
3	600.0	30.0	130.0
4	1500.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4
1	-200.0	0.0	0.0	0.0	300.0
2	-100.0	0.0	0.0	0.0	300.0
3	-40.0	15.0	15.0	15.0	300.0
4	-40.0	15.0	15.0	300.0	300.0
5	20.0	40.0	40.0	300.0	300.0
6	50.0	40.0	55.0	300.0	300.0
7	190.0	110.0	120.0	300.0	300.0
8	350.0	175.0	185.0	300.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	215.0	185.0	-110.0	1.088	0.46595D+08	0.50716D+08	166.5
2	220.0	185.0	-110.0	1.067	0.63784D+08	0.68059D+08	185.3
3	225.0	185.0	-110.0	1.260	0.81395D+08	0.10255D+09	233.1
4	205.0	185.0	-100.0	1.092	0.41608D+08	0.45455D+08	162.7
5	210.0	185.0	-100.0	1.062	0.57971D+08	0.61552D+08	181.4
6	215.0	185.0	-100.0	1.250	0.74889D+08	0.93629D+08	226.0
7	195.0	185.0	-90.0	1.098	0.36888D+08	0.40513D+08	158.8
8	200.0	185.0	-90.0	1.070	0.51546D+08	0.55163D+08	177.4
9	205.0	185.0	-90.0	1.251	0.68095D+08	0.85168D+08	221.1
10	215.0	195.0	-110.0	1.131	0.35286D+08	0.39895D+08	155.4
11	220.0	195.0	-110.0	1.076	0.51244D+08	0.55136D+08	175.3
12	225.0	195.0	-110.0	1.418	0.69706D+08	0.98828D+08	229.3
13	205.0	195.0	-100.0	1.169	0.29812D+08	0.34858D+08	151.5
14	210.0	195.0	-100.0	1.081	0.45811D+08	0.49502D+08	171.4
15	215.0	195.0	-100.0	1.424	0.63373D+08	0.90263D+08	224.5
16	195.0	195.0	-90.0	1.236	0.24458D+08	0.30238D+08	147.5
17	200.0	195.0	-90.0	1.088	0.40610D+08	0.44189D+08	167.4
18	205.0	195.0	-90.0	1.448	0.56269D+08	0.81502D+08	219.6
19	215.0	205.0	-110.0	circle does not intercept slope			
20	220.0	205.0	-110.0	1.123	0.38795D+08	0.43582D+08	164.2
21	225.0	205.0	-110.0	1.506	0.57969D+08	0.87303D+08	227.8
22	205.0	205.0	-100.0	circle does not intercept slope			
23	210.0	205.0	-100.0	1.158	0.33412D+08	0.38679D+08	160.3
24	215.0	205.0	-100.0	1.534	0.51788D+08	0.79442D+08	223.0
25	195.0	205.0	-90.0	circle does not intercept slope			
26	200.0	205.0	-90.0	circle does not intercept slope			
27	205.0	205.0	-90.0	1.581	0.45386D+08	0.71764D+08	218.2

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	210.0	185.0	-100.0	1.062
2	220.0	185.0	-110.0	1.067
3	200.0	185.0	-90.0	1.070
4	220.0	195.0	-110.0	1.076
5	210.0	195.0	-100.0	1.081
6	200.0	195.0	-90.0	1.088
7	215.0	185.0	-110.0	1.088
8	205.0	185.0	-100.0	1.092
9	195.0	185.0	-90.0	1.098
10	220.0	205.0	-110.0	1.123

Problem Title: Highland Estates: Profile GS-5

User's Name:RPF
Date: 03-04-1994

GENERAL DATA

SECTION GS-5
SEISMIC 0.2g
COHESION INCREASE

UNITS

Unit Weight of Water: 62.4

GEOMETRY

Number of Sections : 13

Section	X	Y-crk.	Y-grd.
1	-200.0	-5.0	-5.0
2	-80.0	-5.0	-5.0
3	25.0	45.0	45.0
4	25.0	45.0	45.0
5	35.0	50.0	50.0
6	40.0	65.0	65.0
7	60.0	65.0	65.0
8	60.0	65.0	65.0
9	70.0	65.0	65.0
10	71.0	85.0	85.0
11	71.0	85.0	85.0
12	95.0	95.0	95.0
13	250.0	100.0	100.0

CIRCLE DATA

Coordinates of first circle (X,Y): 70 -160
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 85
Tangent interval: 10
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .2
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES
SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	25.0	125.0
5	1700.0	30.0	125.0
6	2250.0	35.0	125.0
7	3000.0	35.0	125.0

Section Number	X	elevation to bottom of layer — layer number 1	2	3	4	5	6	7
1	-200.0	-5.0	-5.0	-5.0	-5.0	-5.0	-5.0	300.0
2	-80.0	-5.0	0.0	10.0	20.0	30.0	50.0	300.0
3	25.0	45.0	55.0	65.0	75.0	85.0	105.0	300.0
4	25.0	65.0	65.0	65.0	75.0	85.0	105.0	300.0
5	35.0	65.0	65.0	70.0	80.0	90.0	110.0	300.0
6	40.0	65.0	65.0	73.0	83.0	93.0	113.0	300.0
7	60.0	65.0	65.0	85.0	95.0	105.0	125.0	300.0
8	60.0	90.0	90.0	90.0	95.0	105.0	125.0	300.0
9	70.0	90.0	90.0	90.0	98.0	108.0	128.0	300.0
10	71.0	90.0	90.0	90.0	100.0	110.0	130.0	300.0
11	71.0	85.0	85.0	90.0	100.0	110.0	130.0	300.0
12	95.0	95.0	95.0	100.0	110.0	120.0	140.0	300.0
13	250.0	100.0	175.0	185.0	195.0	205.0	225.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	245.0	70.0	-160.0	1.270	0.16389D+09	0.20807D+09	217.8
2	255.0	70.0	-160.0	1.179	0.23574D+09	0.27787D+09	256.3
3	265.0	70.0	-160.0	1.173	0.34642D+09	0.40648D+09	318.0
4	225.0	70.0	-140.0	1.222	0.14493D+09	0.17707D+09	209.4
5	235.0	70.0	-140.0	1.140	0.20518D+09	0.23385D+09	247.4
6	245.0	70.0	-140.0	1.208	0.28667D+09	0.34619D+09	306.8
7	205.0	70.0	-120.0	1.228	0.11659D+09	0.14315D+09	200.7
8	215.0	70.0	-120.0	1.131	0.17241D+09	0.19495D+09	238.2
9	225.0	70.0	-120.0	1.192	0.25099D+09	0.29921D+09	295.1
10	245.0	90.0	-160.0	1.237	0.11878D+09	0.14691D+09	197.8
11	255.0	90.0	-160.0	1.085	0.19667D+09	0.21331D+09	238.6
12	265.0	90.0	-160.0	1.264	0.26978D+09	0.34110D+09	315.7
13	225.0	90.0	-140.0	1.262	0.96943D+08	0.12231D+09	189.4
14	235.0	90.0	-140.0	1.054	0.17241D+09	0.18172D+09	229.9
15	245.0	90.0	-140.0	1.290	0.22738D+09	0.29339D+09	304.6
16	205.0	90.0	-120.0	1.306	0.76781D+08	0.10027D+09	180.4
17	215.0	90.0	-120.0	1.054	0.13949D+09	0.14703D+09	220.9
18	225.0	90.0	-120.0	1.200	0.20201D+09	0.24241D+09	293.0
19	245.0	110.0	-160.0	1.539	0.69733D+08	0.10734D+09	177.3
20	255.0	110.0	-160.0	1.056	0.13853D+09	0.14634D+09	217.5
21	265.0	110.0	-160.0	1.355	0.21010D+09	0.28477D+09	313.4
22	225.0	110.0	-140.0	1.665	0.51143D+08	0.85139D+08	159.5
23	235.0	110.0	-140.0	1.070	0.11548D+09	0.12360D+09	208.7
24	245.0	110.0	-140.0	1.355	0.17739D+09	0.24041D+09	302.3
25	205.0	110.0	-120.0	1.761	0.37477D+08	0.65999D+08	141.7
26	215.0	110.0	-120.0	1.117	0.88960D+08	0.99365D+08	192.7
27	225.0	110.0	-120.0	1.350	0.14977D+09	0.20212D+09	290.8

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	235.0	90.0	-140.0	1.054
2	215.0	90.0	-120.0	1.054
3	255.0	110.0	-160.0	1.056
4	235.0	110.0	-140.0	1.070
5	255.0	90.0	-160.0	1.085
6	215.0	110.0	-120.0	1.117
7	215.0	70.0	-120.0	1.131
8	235.0	70.0	-140.0	1.140
9	265.0	70.0	-160.0	1.173
10	255.0	70.0	-160.0	1.179

Problem Title: Highland Estates: Profile GS-5

User's Name:RPF
Date: 03-04-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.4

SECTION GS-5
SEISMIC 0.21g
COHESION INCREASE

GEOMETRY

Number of Sections : 13

Section	X	Y-crk.	Y-grd.
1	-200.0	-5.0	-5.0
2	-80.0	-5.0	-5.0
3	25.0	45.0	45.0
4	25.0	45.0	45.0
5	35.0	50.0	50.0
6	40.0	65.0	65.0
7	60.0	65.0	65.0
8	60.0	65.0	65.0
9	70.0	65.0	65.0
10	71.0	85.0	85.0
11	71.0	85.0	85.0
12	95.0	95.0	95.0
13	250.0	100.0	100.0

CIRCLE DATA

Coordinates of first circle (X,Y): 70 -160
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 85
Tangent interval: 10
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .21
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	25.0	125.0
5	1700.0	30.0	125.0
6	2250.0	35.0	125.0
7	3000.0	35.0	125.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4	5	6	7
1	-200.0	-5.0	-5.0	-5.0	-5.0	-5.0	-5.0	300.0
2	-80.0	-5.0	0.0	10.0	20.0	30.0	50.0	300.0
3	25.0	45.0	55.0	65.0	75.0	85.0	105.0	300.0
4	25.0	65.0	65.0	65.0	75.0	85.0	105.0	300.0
5	35.0	65.0	65.0	70.0	80.0	90.0	110.0	300.0
6	40.0	65.0	65.0	73.0	83.0	93.0	113.0	300.0
7	60.0	65.0	65.0	85.0	95.0	105.0	125.0	300.0
8	60.0	90.0	90.0	90.0	95.0	105.0	125.0	300.0
9	70.0	90.0	90.0	90.0	98.0	108.0	128.0	300.0
10	71.0	90.0	90.0	90.0	100.0	110.0	130.0	300.0
11	71.0	85.0	85.0	90.0	100.0	110.0	130.0	300.0
12	95.0	95.0	95.0	100.0	110.0	120.0	140.0	300.0
13	250.0	100.0	175.0	185.0	195.0	205.0	225.0	300.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	245.0	70.0	-160.0	1.229	0.16832D+09	0.20694D+09	217.8
2	255.0	70.0	-160.0	1.140	0.24224D+09	0.27612D+09	256.3
3	265.0	70.0	-160.0	1.132	0.35682D+09	0.40392D+09	318.0
4	225.0	70.0	-140.0	1.182	0.14890D+09	0.17602D+09	209.4
5	235.0	70.0	-140.0	1.102	0.21081D+09	0.23226D+09	247.4
6	245.0	70.0	-140.0	1.167	0.29487D+09	0.34413D+09	306.8
7	205.0	70.0	-120.0	1.190	0.11965D+09	0.14235D+09	200.7
8	215.0	70.0	-120.0	1.094	0.17702D+09	0.19358D+09	238.2
9	225.0	70.0	-120.0	1.152	0.25814D+09	0.29736D+09	295.1
10	245.0	90.0	-160.0	1.201	0.12174D+09	0.14620D+09	197.8
11	255.0	90.0	-160.0	1.048	0.20202D+09	0.21175D+09	238.6
12	265.0	90.0	-160.0	1.225	0.27708D+09	0.33928D+09	315.7
13	225.0	90.0	-140.0	1.226	0.99304D+08	0.12172D+09	189.4
14	235.0	90.0	-140.0	1.018	0.17718D+09	0.18035D+09	229.9
15	245.0	90.0	-140.0	1.251	0.23332D+09	0.29188D+09	304.6
16	205.0	90.0	-120.0	1.269	0.78645D+08	0.99797D+08	180.4
17	215.0	90.0	-120.0	1.019	0.14321D+09	0.14597D+09	220.9
18	225.0	90.0	-120.0	1.161	0.20746D+09	0.24096D+09	293.0
19	245.0	110.0	-160.0	1.502	0.71247D+08	0.10700D+09	177.3
20	255.0	110.0	-160.0	1.024	0.14200D+09	0.14546D+09	217.5
21	265.0	110.0	-160.0	1.314	0.21558D+09	0.28337D+09	313.4
22	225.0	110.0	-140.0	1.626	0.52218D+08	0.84904D+08	159.5
23	235.0	110.0	-140.0	1.037	0.11842D+09	0.12282D+09	208.7
24	245.0	110.0	-140.0	1.315	0.18191D+09	0.23929D+09	302.3
25	205.0	110.0	-120.0	1.720	0.38279D+08	0.65823D+08	141.7
26	215.0	110.0	-120.0	1.083	0.91189D+08	0.98779D+08	192.7
27	225.0	110.0	-120.0	1.309	0.15366D+09	0.20119D+09	290.8

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	235.0	90.0	-140.0	1.018
2	215.0	90.0	-120.0	1.019
3	255.0	110.0	-160.0	1.024
4	235.0	110.0	-140.0	1.037
5	255.0	90.0	-160.0	1.048
6	215.0	110.0	-120.0	1.083
7	215.0	70.0	-120.0	1.094
8	235.0	70.0	-140.0	1.102
9	265.0	70.0	-160.0	1.132
10	255.0	70.0	-160.0	1.140

Problem Title: Highland Estates: Profile GS-5

User's Name:RPF
Date: 03-04-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.4

SECTION GS-5

SEISMIC 0.22g

COHESION INCREASE

GEOMETRY

Number of Sections : 13

Section	X	Y-crk.	Y-grd.
1	-200.0	-5.0	-5.0
2	-80.0	-5.0	-5.0
3	25.0	45.0	45.0
4	25.0	45.0	45.0
5	35.0	50.0	50.0
6	40.0	65.0	65.0
7	60.0	65.0	65.0
8	60.0	65.0	65.0
9	70.0	65.0	65.0
10	71.0	85.0	85.0
11	71.0	85.0	85.0
12	95.0	95.0	95.0
13	250.0	100.0	100.0

CIRCLE DATA

Coordinates of first circle (X,Y): 70 -160
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 3
Y-direction: 3

Elevation of upper-most tangent: 85
Tangent interval: 10
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .22
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	25.0	125.0
5	1700.0	30.0	125.0
6	2250.0	35.0	125.0
7	3000.0	35.0	125.0

IL PROFILE

ction number	X	elevation to bottom of layer, layer number 1	2	3	4	5	6	7
1	-200.0	-5.0	-5.0	-5.0	-5.0	-5.0	-5.0	300.0
2	-80.0	-5.0	0.0	10.0	20.0	30.0	50.0	300.0
3	25.0	45.0	55.0	65.0	75.0	85.0	105.0	300.0
4	25.0	65.0	65.0	65.0	75.0	85.0	105.0	300.0
5	35.0	65.0	65.0	70.0	80.0	90.0	110.0	300.0
6	40.0	65.0	65.0	73.0	83.0	93.0	113.0	300.0
7	60.0	65.0	65.0	85.0	95.0	105.0	125.0	300.0
8	60.0	90.0	90.0	90.0	95.0	105.0	125.0	300.0
9	70.0	90.0	90.0	90.0	98.0	108.0	128.0	300.0
10	71.0	90.0	90.0	90.0	100.0	110.0	130.0	300.0
11	71.0	85.0	85.0	90.0	100.0	110.0	130.0	300.0
12	95.0	95.0	95.0	100.0	110.0	120.0	140.0	300.0
13	250.0	100.0	175.0	185.0	195.0	205.0	225.0	300.0

ESULTS

ircle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	245.0	70.0	-160.0	1.192	0.17274D+09	0.20582D+09	217.8
2	255.0	70.0	-160.0	1.103	0.24873D+09	0.27438D+09	256.3
3	265.0	70.0	-160.0	1.093	0.36723D+09	0.40143D+09	318.0
4	225.0	70.0	-140.0	1.145	0.15287D+09	0.17498D+09	209.4
5	235.0	70.0	-140.0	1.066	0.21644D+09	0.23067D+09	247.4
6	245.0	70.0	-140.0	1.129	0.30307D+09	0.34209D+09	306.8
7	205.0	70.0	-120.0	1.154	0.12271D+09	0.14155D+09	200.7
8	215.0	70.0	-120.0	1.058	0.18162D+09	0.19222D+09	238.2
9	225.0	70.0	-120.0	1.114	0.26528D+09	0.29552D+09	295.1
10	245.0	90.0	-160.0	1.167	0.12470D+09	0.14550D+09	197.8
11	255.0	90.0	-160.0	1.014	0.20738D+09	0.21019D+09	238.6
12	265.0	90.0	-160.0	1.187	0.28437D+09	0.33748D+09	315.7
13	225.0	90.0	-140.0	1.191	0.10167D+09	0.12112D+09	189.4
14	235.0	90.0	-140.0	0.984	0.18195D+09	0.17898D+09	229.9
15	245.0	90.0	-140.0	1.214	0.23926D+09	0.29037D+09	304.6
16	205.0	90.0	-120.0	1.234	0.80510D+08	0.99329D+08	180.4
17	215.0	90.0	-120.0	0.986	0.14694D+09	0.14491D+09	220.9
18	225.0	90.0	-120.0	1.125	0.21291D+09	0.23951D+09	293.0
19	245.0	110.0	-160.0	1.466	0.72762D+08	0.10665D+09	177.3
20	255.0	110.0	-160.0	0.994	0.14547D+09	0.14457D+09	217.5
21	265.0	110.0	-160.0	1.276	0.22105D+09	0.28198D+09	313.4
22	225.0	110.0	-140.0	1.589	0.53292D+08	0.84670D+08	159.5
23	235.0	110.0	-140.0	1.006	0.12135D+09	0.12205D+09	208.7
24	245.0	110.0	-140.0	1.277	0.18643D+09	0.23816D+09	302.3
25	205.0	110.0	-120.0	1.680	0.39080D+08	0.65647D+08	141.7
26	215.0	110.0	-120.0	1.051	0.93418D+08	0.98194D+08	192.7
27	225.0	110.0	-120.0	1.271	0.15755D+09	0.20026D+09	290.8

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	235.0	90.0	-140.0	0.984
2	215.0	90.0	-120.0	0.986
3	255.0	110.0	-160.0	0.994
4	235.0	110.0	-140.0	1.006
5	255.0	90.0	-160.0	1.014
6	215.0	110.0	-120.0	1.051
7	215.0	70.0	-120.0	1.058
8	235.0	70.0	-140.0	1.066
9	265.0	70.0	-160.0	1.093
10	255.0	70.0	-160.0	1.103

Problem Title: Highland Estates: Profile GS-7b

User's Name:RPF
Date: 03-04-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.4

SECTION GS-7b

SEISMIC 0.2g

COHESION INCREASE

GEOMETRY

Number of Sections : 14

Section	X	Y-crk.	Y-grd.
1	-175.0	0.0	0.0
2	-55.0	10.0	10.0
3	40.0	55.0	55.0
4	62.0	55.0	55.0
5	100.0	75.0	75.0
6	120.0	77.0	77.0
7	145.0	80.0	80.0
8	152.0	80.0	80.0
9	155.0	80.0	80.0
10	160.0	85.0	85.0
11	185.0	108.0	108.0
12	220.0	140.0	140.0
13	220.0	140.0	140.0
14	500.0	140.0	140.0

CIRCLE DATA

Coordinates of first circle (X,Y): 160 -60
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 4
Y-direction: 4

Elevation of upper-most tangent: 150
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .2
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	25.0	125.0
5	1850.0	30.0	125.0
6	2000.0	35.0	125.0
7	3000.0	35.0	125.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer / layer number 1	2	3	4	5	6	7
1	-175.0	0.0	0.0	0.0	10.0	20.0	40.0	300.0
2	-55.0	10.0	42.0	52.0	62.0	72.0	92.0	300.0
3	40.0	55.0	80.0	90.0	100.0	110.0	130.0	300.0
4	62.0	55.0	90.0	100.0	110.0	120.0	140.0	300.0
5	100.0	75.0	105.0	115.0	125.0	135.0	155.0	300.0
6	120.0	80.0	115.0	125.0	135.0	145.0	165.0	300.0
7	145.0	125.0	125.0	135.0	145.0	155.0	175.0	300.0
8	152.0	135.0	135.0	135.0	145.0	155.0	175.0	300.0
9	155.0	140.0	140.0	140.0	148.0	158.0	178.0	300.0
10	160.0	150.0	150.0	150.0	150.0	160.0	180.0	300.0
11	185.0	150.0	150.0	150.0	160.0	170.0	190.0	300.0
12	220.0	150.0	150.0	160.0	170.0	180.0	200.0	300.0
13	220.0	140.0	150.0	160.0	170.0	180.0	200.0	300.0
14	500.0	140.0	230.0	240.0	250.0	260.0	280.0	400.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	210.0	160.0	-60.0	1.068	0.30917D+09	0.33033D+09	311.1
2	215.0	160.0	-60.0	0.897	0.37911D+09	0.34017D+09	337.6
3	220.0	160.0	-60.0	0.948	0.41599D+09	0.39438D+09	361.8
4	190.0	160.0	-40.0	1.289	0.22659D+09	0.29202D+09	289.0
5	195.0	160.0	-40.0	0.915	0.30561D+09	0.27968D+09	314.5
6	200.0	160.0	-40.0	0.959	0.34698D+09	0.33270D+09	338.0
7	170.0	160.0	-20.0	1.296	0.18616D+09	0.24119D+09	268.4
8	175.0	160.0	-20.0	0.978	0.25123D+09	0.24563D+09	291.0
9	180.0	160.0	-20.0	0.963	0.28759D+09	0.27695D+09	313.6
10	150.0	160.0	0.0	1.323	0.14632D+09	0.19361D+09	247.9
11	155.0	160.0	0.0	0.992	0.19808D+09	0.19654D+09	266.9
12	160.0	160.0	0.0	0.964	0.23452D+09	0.22612D+09	288.5
13	210.0	180.0	-60.0	1.144	0.24547D+09	0.28092D+09	298.6
14	215.0	180.0	-60.0	0.882	0.31821D+09	0.28060D+09	325.6
15	220.0	180.0	-60.0	0.912	0.36703D+09	0.33480D+09	350.4
16	190.0	180.0	-40.0	1.149	0.19709D+09	0.22638D+09	276.9
17	195.0	180.0	-40.0	0.882	0.26641D+09	0.23499D+09	303.1
18	200.0	180.0	-40.0	0.911	0.31011D+09	0.28251D+09	327.1
19	170.0	180.0	-20.0	1.409	0.14006D+09	0.19742D+09	254.9
20	175.0	180.0	-20.0	0.898	0.21256D+09	0.19080D+09	280.1
21	180.0	180.0	-20.0	0.916	0.25544D+09	0.23409D+09	303.4
22	150.0	180.0	0.0	1.380	0.11479D+09	0.15844D+09	232.8
23	155.0	180.0	0.0	0.912	0.16382D+09	0.14948D+09	256.7
24	160.0	180.0	0.0	0.921	0.20579D+09	0.18950D+09	279.1
25	210.0	200.0	-60.0	1.303	0.17427D+09	0.22699D+09	284.7
26	215.0	200.0	-60.0	0.920	0.25862D+09	0.23780D+09	312.4
27	220.0	200.0	-60.0	0.947	0.29872D+09	0.28302D+09	337.8
28	190.0	200.0	-40.0	1.591	0.12903D+09	0.20524D+09	263.4
29	195.0	200.0	-40.0	0.918	0.21248D+09	0.19514D+09	290.3
30	200.0	200.0	-40.0	0.969	0.24312D+09	0.23565D+09	314.9
31	170.0	200.0	-20.0	1.577	0.10714D+09	0.16891D+09	246.9
32	175.0	200.0	-20.0	0.932	0.16854D+09	0.15710D+09	269.1
33	180.0	200.0	-20.0	0.974	0.20035D+09	0.19508D+09	291.7
34	150.0	200.0	0.0	1.579	0.85010D+08	0.13425D+09	232.8
35	155.0	200.0	0.0	0.943	0.13373D+09	0.12612D+09	254.0
36	160.0	200.0	0.0	0.968	0.16552D+09	0.16027D+09	273.6
37	210.0	220.0	-60.0	1.522	0.13638D+09	0.20761D+09	272.6
38	215.0	220.0	-60.0	0.969	0.19721D+09	0.19112D+09	297.7
39	220.0	220.0	-60.0	0.980	0.24296D+09	0.23806D+09	323.9
40	190.0	220.0	-40.0	2.013	0.92953D+08	0.18711D+09	260.1
41	195.0	220.0	-40.0	1.232	0.14609D+09	0.17995D+09	283.1
42	200.0	220.0	-40.0	0.968	0.20739D+09	0.20073D+09	304.2
43	170.0	220.0	-20.0	2.019	0.75144D+08	0.15172D+09	242.7
44	175.0	220.0	-20.0	1.268	0.11478D+09	0.14556D+09	269.1
45	180.0	220.0	-20.0	1.019	0.15702D+09	0.16002D+09	289.5
46	150.0	220.0	0.0	2.403	0.55469D+08	0.13328D+09	219.6
47	155.0	220.0	0.0	1.314	0.91341D+08	0.11999D+09	245.1
48	160.0	220.0	0.0	1.028	0.12607D+09	0.12966D+09	268.6

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	215.0	180.0	-60.0	0.882
2	195.0	180.0	-40.0	0.882
3	215.0	160.0	-60.0	0.897
4	175.0	180.0	-20.0	0.898
5	200.0	180.0	-40.0	0.911
6	220.0	180.0	-60.0	0.912
7	155.0	180.0	0.0	0.912
8	195.0	160.0	-40.0	0.915
9	180.0	180.0	-20.0	0.916
10	195.0	200.0	-40.0	0.918

Problem Title: Highland Estates: Profile GS-7b

User's Name:RPF
Date: 03-04-1994

SECTION GS-7b
SEISMIC 0.21g
COHESION INCREASE

GENERAL DATA

UNITS

Unit Weight of Water: 62.4

GEOMETRY

Number of Sections : 14

Section	X	Y-crk.	Y-grd.
1	-175.0	0.0	0.0
2	-55.0	10.0	10.0
3	40.0	55.0	55.0
4	62.0	55.0	55.0
5	100.0	75.0	75.0
6	120.0	77.0	77.0
7	145.0	80.0	80.0
8	152.0	80.0	80.0
9	155.0	80.0	80.0
10	160.0	85.0	85.0
11	185.0	108.0	108.0
12	220.0	140.0	140.0
13	220.0	140.0	140.0
14	500.0	140.0	140.0

CIRCLE DATA

Coordinates of first circle (X,Y): 160 -60
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 4
Y-direction: 4

Elevation of upper-most tangent: 150
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .21
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES
SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	25.0	125.0
5	1850.0	30.0	125.0
6	2000.0	35.0	125.0
7	3000.0	35.0	125.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4	5	6	7
1	-175.0	0.0	0.0	0.0	10.0	20.0	40.0	300.0
2	-55.0	10.0	42.0	52.0	62.0	72.0	92.0	300.0
3	40.0	55.0	80.0	90.0	100.0	110.0	130.0	300.0
4	62.0	55.0	90.0	100.0	110.0	120.0	140.0	300.0
5	100.0	75.0	105.0	115.0	125.0	135.0	155.0	300.0
6	120.0	80.0	115.0	125.0	135.0	145.0	165.0	300.0
7	145.0	125.0	125.0	135.0	145.0	155.0	175.0	300.0
8	152.0	135.0	135.0	135.0	145.0	155.0	175.0	300.0
9	155.0	140.0	140.0	140.0	148.0	158.0	178.0	300.0
10	160.0	150.0	150.0	150.0	150.0	160.0	180.0	300.0
11	185.0	150.0	150.0	150.0	160.0	170.0	190.0	300.0
12	220.0	150.0	150.0	160.0	170.0	180.0	200.0	300.0
13	220.0	140.0	150.0	160.0	170.0	180.0	200.0	300.0
14	500.0	140.0	230.0	240.0	250.0	260.0	280.0	400.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	210.0	160.0	-60.0	1.031	0.31909D+09	0.32883D+09	311.1
2	215.0	160.0	-60.0	0.862	0.39166D+09	0.33770D+09	337.6
3	220.0	160.0	-60.0	0.912	0.42952D+09	0.39180D+09	361.8
4	190.0	160.0	-40.0	1.247	0.23341D+09	0.29097D+09	289.0
5	195.0	160.0	-40.0	0.880	0.31566D+09	0.27769D+09	314.5
6	200.0	160.0	-40.0	0.922	0.35839D+09	0.33058D+09	338.0
7	170.0	160.0	-20.0	1.253	0.19185D+09	0.24035D+09	268.4
8	175.0	160.0	-20.0	0.941	0.25957D+09	0.24429D+09	291.0
9	180.0	160.0	-20.0	0.926	0.29714D+09	0.27522D+09	313.6
10	150.0	160.0	0.0	1.280	0.15078D+09	0.19302D+09	247.9
11	155.0	160.0	0.0	0.955	0.20464D+09	0.19549D+09	266.9
12	160.0	160.0	0.0	0.927	0.24240D+09	0.22474D+09	288.5
13	210.0	180.0	-60.0	1.106	0.25291D+09	0.27978D+09	298.6
14	215.0	180.0	-60.0	0.848	0.32853D+09	0.27874D+09	325.6
15	220.0	180.0	-60.0	0.877	0.37901D+09	0.33258D+09	350.4
16	190.0	180.0	-40.0	1.111	0.20296D+09	0.22555D+09	276.9
17	195.0	180.0	-40.0	0.849	0.27511D+09	0.23344D+09	303.1
18	200.0	180.0	-40.0	0.876	0.32030D+09	0.28065D+09	327.1
19	170.0	180.0	-20.0	1.369	0.14381D+09	0.19691D+09	254.9
20	175.0	180.0	-20.0	0.864	0.21942D+09	0.18962D+09	280.1
21	180.0	180.0	-20.0	0.881	0.26388D+09	0.23259D+09	303.4
22	150.0	180.0	0.0	1.340	0.11789D+09	0.15803D+09	232.8
23	155.0	180.0	0.0	0.880	0.16894D+09	0.14863D+09	256.7
24	160.0	180.0	0.0	0.886	0.21254D+09	0.18835D+09	279.1
25	210.0	200.0	-60.0	1.266	0.17885D+09	0.22641D+09	284.7
26	215.0	200.0	-60.0	0.887	0.26676D+09	0.23659D+09	312.4
27	220.0	200.0	-60.0	0.913	0.30818D+09	0.28147D+09	337.8
28	190.0	200.0	-40.0	1.552	0.13202D+09	0.20483D+09	263.4
29	195.0	200.0	-40.0	0.887	0.21909D+09	0.19423D+09	290.3
30	200.0	200.0	-40.0	0.936	0.25064D+09	0.23448D+09	314.9
31	170.0	200.0	-20.0	1.538	0.10962D+09	0.16857D+09	246.9
32	175.0	200.0	-20.0	0.901	0.17363D+09	0.15640D+09	269.1
33	180.0	200.0	-20.0	0.940	0.20654D+09	0.19413D+09	291.7
34	150.0	200.0	0.0	1.541	0.86952D+08	0.13398D+09	232.8
35	155.0	200.0	0.0	0.912	0.13768D+09	0.12558D+09	254.0
36	160.0	200.0	0.0	0.935	0.17060D+09	0.15947D+09	273.6
37	210.0	220.0	-60.0	1.483	0.13969D+09	0.20716D+09	272.6
38	215.0	220.0	-60.0	0.938	0.20300D+09	0.19043D+09	297.7
39	220.0	220.0	-60.0	0.947	0.25046D+09	0.23711D+09	323.9
40	190.0	220.0	-40.0	1.974	0.94669D+08	0.18686D+09	260.1
41	195.0	220.0	-40.0	1.197	0.14991D+09	0.17940D+09	283.1
42	200.0	220.0	-40.0	0.935	0.21377D+09	0.19994D+09	304.2
43	170.0	220.0	-20.0	1.978	0.76584D+08	0.15151D+09	242.7
44	175.0	220.0	-20.0	1.233	0.11773D+09	0.14516D+09	269.1
45	180.0	220.0	-20.0	0.987	0.16159D+09	0.15949D+09	289.5
46	150.0	220.0	0.0	2.361	0.56388D+08	0.13312D+09	219.6
47	155.0	220.0	0.0	1.277	0.93678D+08	0.11965D+09	245.1
48	160.0	220.0	0.0	0.996	0.12975D+09	0.12923D+09	268.6

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	215.0	180.0	-60.0	0.848
2	195.0	180.0	-40.0	0.849
3	215.0	160.0	-60.0	0.862
4	175.0	180.0	-20.0	0.864
5	200.0	180.0	-40.0	0.876
6	220.0	180.0	-60.0	0.877
7	195.0	160.0	-40.0	0.880
8	155.0	180.0	0.0	0.880
9	180.0	180.0	-20.0	0.881
10	160.0	180.0	0.0	0.886

Problem Title: Highland Estates: Profile GS-7b

User's Name:RPF
Date: 03-04-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.4

SECTION GS-7b

SEISMIC 0.22g

COHESION INCREASE

GEOMETRY

Number of Sections : 14

Section	X	Y-crk.	Y-grd.
1	-175.0	0.0	0.0
2	-55.0	10.0	10.0
3	40.0	55.0	55.0
4	62.0	55.0	55.0
5	100.0	75.0	75.0
6	120.0	77.0	77.0
7	145.0	80.0	80.0
8	152.0	80.0	80.0
9	155.0	80.0	80.0
10	160.0	85.0	85.0
11	185.0	108.0	108.0
12	220.0	140.0	140.0
13	220.0	140.0	140.0
14	500.0	140.0	140.0

CIRCLE DATA

Coordinates of first circle (X,Y): 160 -60
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 4
Y-direction: 4

Elevation of upper-most tangent: 150
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .22
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	15.0	125.0
4	1500.0	25.0	125.0
5	1850.0	30.0	125.0
6	2000.0	35.0	125.0
7	3000.0	35.0	125.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3	4	5	6	7
1	-175.0	0.0	0.0	0.0	10.0	20.0	40.0	300.0
2	-55.0	10.0	42.0	52.0	62.0	72.0	92.0	300.0
3	40.0	55.0	80.0	90.0	100.0	110.0	130.0	300.0
4	62.0	55.0	90.0	100.0	110.0	120.0	140.0	300.0
5	100.0	75.0	105.0	115.0	125.0	135.0	155.0	300.0
6	120.0	80.0	115.0	125.0	135.0	145.0	165.0	300.0
7	145.0	125.0	125.0	135.0	145.0	155.0	175.0	300.0
8	152.0	135.0	135.0	135.0	145.0	155.0	175.0	300.0
9	155.0	140.0	140.0	140.0	148.0	158.0	178.0	300.0
10	160.0	150.0	150.0	150.0	150.0	160.0	180.0	300.0
11	185.0	150.0	150.0	150.0	160.0	170.0	190.0	300.0
12	220.0	150.0	150.0	160.0	170.0	180.0	200.0	300.0
13	220.0	140.0	150.0	160.0	170.0	180.0	200.0	300.0
14	500.0	140.0	230.0	240.0	250.0	260.0	280.0	400.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	210.0	160.0	-60.0	0.995	0.32900D+09	0.32735D+09	311.1
2	215.0	160.0	-60.0	0.829	0.40422D+09	0.33526D+09	337.6
3	220.0	160.0	-60.0	0.879	0.44306D+09	0.38927D+09	361.8
4	190.0	160.0	-40.0	1.207	0.24023D+09	0.28993D+09	289.0
5	195.0	160.0	-40.0	0.847	0.32572D+09	0.27575D+09	314.5
6	200.0	160.0	-40.0	0.888	0.36979D+09	0.32850D+09	338.0
7	170.0	160.0	-20.0	1.213	0.19754D+09	0.23953D+09	268.4
8	175.0	160.0	-20.0	0.907	0.26790D+09	0.24298D+09	291.0
9	180.0	160.0	-20.0	0.892	0.30668D+09	0.27353D+09	313.6
10	150.0	160.0	0.0	1.240	0.15524D+09	0.19243D+09	247.9
11	155.0	160.0	0.0	0.921	0.21120D+09	0.19448D+09	266.9
12	160.0	160.0	0.0	0.893	0.25028D+09	0.22339D+09	288.5
13	210.0	180.0	-60.0	1.070	0.26035D+09	0.27865D+09	298.6
14	215.0	180.0	-60.0	0.817	0.33886D+09	0.27690D+09	325.6
15	220.0	180.0	-60.0	0.845	0.39100D+09	0.33040D+09	350.4
16	190.0	180.0	-40.0	1.076	0.20883D+09	0.22474D+09	276.9
17	195.0	180.0	-40.0	0.817	0.28381D+09	0.23191D+09	303.1
18	200.0	180.0	-40.0	0.844	0.33049D+09	0.27882D+09	327.1
19	170.0	180.0	-20.0	1.331	0.14757D+09	0.19641D+09	254.9
20	175.0	180.0	-20.0	0.833	0.22627D+09	0.18847D+09	280.1
21	180.0	180.0	-20.0	0.849	0.27232D+09	0.23111D+09	303.4
22	150.0	180.0	0.0	1.303	0.12099D+09	0.15762D+09	232.8
23	155.0	180.0	0.0	0.849	0.17406D+09	0.14780D+09	256.7
24	160.0	180.0	0.0	0.854	0.21930D+09	0.18726D+09	279.1
25	210.0	200.0	-60.0	1.231	0.18342D+09	0.22582D+09	284.7
26	215.0	200.0	-60.0	0.856	0.27490D+09	0.23540D+09	312.4
27	220.0	200.0	-60.0	0.881	0.31764D+09	0.27995D+09	337.8
28	190.0	200.0	-40.0	1.514	0.13500D+09	0.20443D+09	263.4
29	195.0	200.0	-40.0	0.857	0.22570D+09	0.19333D+09	290.3
30	200.0	200.0	-40.0	0.904	0.25816D+09	0.23333D+09	314.9
31	170.0	200.0	-20.0	1.501	0.11210D+09	0.16822D+09	246.9
32	175.0	200.0	-20.0	0.871	0.17873D+09	0.15571D+09	269.1
33	180.0	200.0	-20.0	0.908	0.21272D+09	0.19320D+09	291.7
34	150.0	200.0	0.0	1.504	0.88893D+08	0.13372D+09	232.8
35	155.0	200.0	0.0	0.883	0.14163D+09	0.12505D+09	254.0
36	160.0	200.0	0.0	0.903	0.17568D+09	0.15870D+09	273.6
37	210.0	220.0	-60.0	1.445	0.14301D+09	0.20671D+09	272.6
38	215.0	220.0	-60.0	0.909	0.20878D+09	0.18975D+09	297.7
39	220.0	220.0	-60.0	0.916	0.25796D+09	0.23620D+09	323.9
40	190.0	220.0	-40.0	1.936	0.96384D+08	0.18661D+09	260.1
41	195.0	220.0	-40.0	1.163	0.15374D+09	0.17885D+09	283.1
42	200.0	220.0	-40.0	0.905	0.22016D+09	0.19916D+09	304.2
43	170.0	220.0	-20.0	1.939	0.78023D+08	0.15129D+09	242.7
44	175.0	220.0	-20.0	1.200	0.12068D+09	0.14477D+09	269.1
45	180.0	220.0	-20.0	0.957	0.16616D+09	0.15897D+09	289.5
46	150.0	220.0	0.0	2.320	0.57307D+08	0.13296D+09	219.6
47	155.0	220.0	0.0	1.243	0.96016D+08	0.11931D+09	245.1
48	160.0	220.0	0.0	0.965	0.13343D+09	0.12882D+09	268.6

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	195.0	180.0	-40.0	0.817
2	215.0	180.0	-60.0	0.817
3	215.0	160.0	-60.0	0.829
4	175.0	180.0	-20.0	0.833
5	200.0	180.0	-40.0	0.844
6	220.0	180.0	-60.0	0.845
7	195.0	160.0	-40.0	0.847
8	180.0	180.0	-20.0	0.849
9	155.0	180.0	0.0	0.849
10	160.0	180.0	0.0	0.854

Problem Title: Highland Estates: Profile No. 10

User's Name:P. F.
Date: 09-12-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GS-10 (REVISED)

STATIC

GEOMETRY

Number of Sections : 9

Section	X	Y-crk.	Y-grd.
1	-100.0	47.0	47.0
2	-35.0	47.0	47.0
3	15.0	47.0	47.0
4	15.0	47.0	47.0
5	30.0	47.0	47.0
6	35.0	70.0	70.0
7	40.0	75.0	75.0
8	95.0	110.0	110.0
9	200.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 50 -30
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 5
Y-direction: 5

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

No seismic analysis
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	1500.0	20.0	130.0
3	1000.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3
1	-100.0	47.0	47.0	150.0
2	-35.0	47.0	47.0	150.0
3	15.0	47.0	70.0	150.0
4	15.0	75.0	75.0	150.0
5	30.0	75.0	75.0	150.0
6	35.0	75.0	75.0	150.0
7	40.0	75.0	75.0	150.0
8	95.0	110.0	110.0	150.0
9	200.0	115.0	115.0	150.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	135.0	50.0	-30.0	2.708	0.24422D+08	0.66124D+08	161.5
2	140.0	50.0	-30.0	2.693	0.29320D+08	0.78956D+08	175.6
3	145.0	50.0	-30.0	2.682	0.34753D+08	0.93220D+08	189.5
4	115.0	50.0	-10.0	2.558	0.19823D+08	0.50716D+08	151.6
5	120.0	50.0	-10.0	2.570	0.24154D+08	0.62069D+08	165.3
6	125.0	50.0	-10.0	2.587	0.28952D+08	0.74909D+08	178.9
7	95.0	50.0	10.0	2.441	0.15670D+08	0.38252D+08	140.6
8	100.0	50.0	10.0	2.472	0.19241D+08	0.47561D+08	153.9
9	105.0	50.0	10.0	2.507	0.23225D+08	0.58236D+08	167.1
10	75.0	50.0	30.0	2.344	0.11275D+08	0.26425D+08	127.4
11	80.0	50.0	30.0	2.393	0.14007D+08	0.33522D+08	140.2
12	85.0	50.0	30.0	2.443	0.17090D+08	0.41756D+08	152.8
13	55.0	50.0	50.0	2.287	0.69485D+07	0.15892D+08	104.6
14	60.0	50.0	50.0	2.358	0.89823D+07	0.21181D+08	116.8
15	65.0	50.0	50.0	2.427	0.11334D+08	0.27512D+08	128.9

16	135.0	70.0	-30.0	2.273	0.22800D+08	0.51819D+08	145.6
17	140.0	70.0	-30.0	2.270	0.28299D+08	0.64229D+08	160.4
18	145.0	70.0	-30.0	2.323	0.33364D+08	0.77500D+08	182.3
19	115.0	70.0	-10.0	2.164	0.17999D+08	0.38957D+08	136.2
20	120.0	70.0	-10.0	2.171	0.22394D+08	0.48609D+08	150.6
21	125.0	70.0	-10.0	2.208	0.27627D+08	0.60992D+08	170.5
22	95.0	70.0	10.0	2.094	0.13913D+08	0.29130D+08	125.9
23	100.0	70.0	10.0	2.056	0.17704D+08	0.36397D+08	139.9
24	105.0	70.0	10.0	2.094	0.22195D+08	0.46478D+08	157.5
25	75.0	70.0	30.0	2.046	0.93734D+07	0.19182D+08	113.7
26	80.0	70.0	30.0	2.003	0.12283D+08	0.24605D+08	127.0
27	85.0	70.0	30.0	2.043	0.15658D+08	0.31983D+08	142.1
28	55.0	70.0	50.0	2.548	0.49816D+07	0.12692D+08	92.1
29	60.0	70.0	50.0	2.001	0.73515D+07	0.14708D+08	104.7
30	65.0	70.0	50.0	2.042	0.96830D+07	0.19774D+08	117.1
31	135.0	90.0	-30.0	2.114	0.17917D+08	0.37878D+08	127.1
32	140.0	90.0	-30.0	2.023	0.23472D+08	0.47496D+08	143.0
33	145.0	90.0	-30.0	2.188	0.28778D+08	0.62979D+08	179.0
34	115.0	90.0	-10.0	2.080	0.13253D+08	0.27572D+08	117.9
35	120.0	90.0	-10.0	2.027	0.17805D+08	0.36095D+08	133.6
36	125.0	90.0	-10.0	2.080	0.23730D+08	0.49352D+08	167.4
37	95.0	90.0	10.0	2.481	0.90688D+07	0.22499D+08	107.9
38	100.0	90.0	10.0	1.979	0.13081D+08	0.25893D+08	123.4
39	105.0	90.0	10.0	2.143	0.16847D+08	0.36096D+08	154.7
40	75.0	90.0	30.0	3.752	0.45622D+07	0.17116D+08	96.3
41	80.0	90.0	30.0	2.583	0.77280D+07	0.19963D+08	111.4
42	85.0	90.0	30.0	2.221	0.10989D+08	0.24403D+08	139.6
43	55.0	90.0	50.0	circle does not intercept slope			
44	60.0	90.0	50.0	4.289	0.25138D+07	0.10782D+08	90.2
45	65.0	90.0	50.0	3.207	0.54256D+07	0.17401D+08	114.9
46	135.0	110.0	-30.0	2.754	0.10184D+08	0.28053D+08	102.6
47	140.0	110.0	-30.0	2.108	0.15586D+08	0.32848D+08	121.7
48	145.0	110.0	-30.0	2.363	0.21471D+08	0.50729D+08	175.3
49	115.0	110.0	-10.0	4.025	0.55995D+07	0.22538D+08	92.1
50	120.0	110.0	-10.0	2.729	0.99831D+07	0.27248D+08	112.0
51	125.0	110.0	-10.0	2.487	0.15204D+08	0.37805D+08	164.0
52	95.0	110.0	10.0	9.925	0.94547D+06	0.93840D+07	23.9
53	100.0	110.0	10.0	4.389	0.50527D+07	0.22175D+08	101.3
54	105.0	110.0	10.0	3.253	0.10192D+08	0.33157D+08	151.6
55	75.0	110.0	30.0	circle does not intercept slope			
56	80.0	110.0	30.0	circle does not intercept slope			
57	85.0	110.0	30.0	F.S. greater than 10			
57	85.0	110.0	30.0	%15.557	0.53962D+06	0.83950D+07	24.5
58	55.0	110.0	50.0	circle does not intercept slope			
59	60.0	110.0	50.0	circle does not intercept slope			
60	65.0	110.0	50.0	7.197	0.10150D+07	0.73053D+07	86.4
61	135.0	130.0	-30.0	9.394	0.14003D+07	0.13155D+08	23.1
62	140.0	130.0	-30.0	6.699	0.48715D+07	0.32635D+08	88.5
63	145.0	130.0	-30.0	3.993	0.11603D+08	0.46325D+08	171.2
64	115.0	130.0	-10.0	circle does not intercept slope			
65	120.0	130.0	-10.0	F.S. greater than 10			
65	120.0	130.0	-10.0	%14.598	0.54117D+06	0.79000D+07	16.1
66	125.0	130.0	-10.0	6.658	0.45104D+07	0.30032D+08	101.5
67	95.0	130.0	10.0	circle does not intercept slope			
68	100.0	130.0	10.0	circle does not intercept slope			
69	105.0	130.0	10.0	circle does not intercept slope			
70	75.0	130.0	30.0	circle does not intercept slope			
71	80.0	130.0	30.0	circle does not intercept slope			
72	85.0	130.0	30.0	circle does not intercept slope			
73	55.0	130.0	50.0	circle does not intercept slope			
74	60.0	130.0	50.0	circle does not intercept slope			
75	65.0	130.0	50.0	circle does not intercept slope			

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	100.0	90.0	10.0	1.979
2	60.0	70.0	50.0	2.001
3	80.0	70.0	30.0	2.003
4	140.0	90.0	-30.0	2.023
5	120.0	90.0	-10.0	2.027
6	65.0	70.0	50.0	2.042
7	85.0	70.0	30.0	2.043
8	75.0	70.0	30.0	2.046
9	100.0	70.0	10.0	2.056
10	125.0	90.0	-10.0	2.080

Problem Title: Highland Estates: Profile No. 10

User's Name:P. F.
Date: 09-12-1994

GENERAL DATA

GS-10 (REVISED)
· SEISMIC: 0.2g
· STRENGTH INCREASE ALLOWED

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 9

Section	X	Y-crk.	Y-grd.
1	-100.0	47.0	47.0
2	-35.0	47.0	47.0
3	15.0	47.0	47.0
4	15.0	47.0	47.0
5	30.0	47.0	47.0
6	35.0	70.0	70.0
7	40.0	75.0	75.0
8	95.0	110.0	110.0
9	200.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 40 -40
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 5
Y-direction: 5

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .2
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3
1	-100.0	47.0	47.0	150.0
2	-35.0	47.0	47.0	150.0
3	15.0	47.0	70.0	150.0
4	15.0	75.0	75.0	150.0
5	30.0	75.0	75.0	150.0
6	35.0	75.0	75.0	150.0
7	40.0	75.0	75.0	150.0
8	95.0	110.0	110.0	150.0
9	200.0	115.0	115.0	150.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	145.0	40.0	-40.0	1.931	0.46386D+08	0.89551D+08	173.5
2	150.0	40.0	-40.0	1.924	0.55289D+08	0.10638D+09	187.6
3	155.0	40.0	-40.0	1.922	0.65046D+08	0.12500D+09	201.5
4	125.0	40.0	-20.0	1.910	0.37925D+08	0.72432D+08	163.8
5	130.0	40.0	-20.0	1.914	0.45349D+08	0.86819D+08	177.5
6	135.0	40.0	-20.0	1.927	0.53153D+08	0.10243D+09	191.1
7	105.0	40.0	0.0	1.856	0.30141D+08	0.55929D+08	153.1
8	110.0	40.0	0.0	1.888	0.35579D+08	0.67176D+08	166.5
9	115.0	40.0	0.0	1.917	0.41609D+08	0.79748D+08	179.7
10	85.0	40.0	20.0	1.914	0.21082D+08	0.40347D+08	140.9
11	90.0	40.0	20.0	1.944	0.25546D+08	0.49670D+08	153.8
12	95.0	40.0	20.0	1.971	0.30632D+08	0.60376D+08	166.6
13	65.0	40.0	40.0	1.952	0.13980D+08	0.27284D+08	124.3
14	70.0	40.0	40.0	1.997	0.17316D+08	0.34578D+08	136.5
15	75.0	40.0	40.0	2.036	0.21146D+08	0.43045D+08	148.7

16	145.0	60.0	-40.0	1.726	0.42015D+08	0.72500D+08	158.4
17	150.0	60.0	-40.0	1.724	0.50720D+08	0.87427D+08	172.9
18	155.0	60.0	-40.0	1.736	0.60415D+08	0.10490D+09	189.6
19	125.0	60.0	-20.0	1.691	0.34209D+08	0.57845D+08	149.1
20	130.0	60.0	-20.0	1.697	0.41552D+08	0.70513D+08	163.3
21	135.0	60.0	-20.0	1.704	0.49918D+08	0.85067D+08	178.1
22	105.0	60.0	0.0	1.607	0.27466D+08	0.44148D+08	139.0
23	110.0	60.0	0.0	1.634	0.33282D+08	0.54390D+08	152.8
24	115.0	60.0	0.0	1.659	0.39780D+08	0.65975D+08	166.4
25	85.0	60.0	20.0	1.614	0.19188D+08	0.30969D+08	127.6
26	90.0	60.0	20.0	1.683	0.23348D+08	0.39298D+08	140.9
27	95.0	60.0	20.0	1.703	0.28592D+08	0.48706D+08	154.1
28	65.0	60.0	40.0	1.675	0.11913D+08	0.19960D+08	112.1
29	70.0	60.0	40.0	1.734	0.15120D+08	0.26211D+08	124.7
30	75.0	60.0	40.0	1.750	0.19131D+08	0.33486D+08	137.1
31	145.0	80.0	-40.0	1.543	0.36811D+08	0.56791D+08	141.2
32	150.0	80.0	-40.0	1.557	0.44790D+08	0.69747D+08	156.6
33	155.0	80.0	-40.0	1.661	0.53880D+08	0.89486D+08	186.2
34	125.0	80.0	-20.0	1.575	0.27614D+08	0.43485D+08	132.3
35	130.0	80.0	-20.0	1.564	0.34877D+08	0.54561D+08	147.4
36	135.0	80.0	-20.0	1.571	0.45456D+08	0.71422D+08	175.0
37	105.0	80.0	0.0	1.604	0.20950D+08	0.33597D+08	122.6
38	110.0	80.0	0.0	1.511	0.27219D+08	0.41130D+08	137.4
39	115.0	80.0	0.0	1.639	0.32723D+08	0.53646D+08	162.8
40	85.0	80.0	20.0	1.663	0.13989D+08	0.23262D+08	111.9
41	90.0	80.0	20.0	1.625	0.18678D+08	0.30348D+08	126.2
42	95.0	80.0	20.0	1.598	0.24491D+08	0.39146D+08	149.0
43	65.0	80.0	40.0	2.322	0.68141D+07	0.15821D+08	97.7
44	70.0	80.0	40.0	1.834	0.10615D+08	0.19470D+08	111.2
45	75.0	80.0	40.0	1.803	0.14932D+08	0.26916D+08	130.7
46	145.0	100.0	-40.0	1.577	0.26716D+08	0.42141D+08	120.4
47	150.0	100.0	-40.0	1.538	0.35163D+08	0.54066D+08	137.5
48	155.0	100.0	-40.0	1.689	0.44361D+08	0.74929D+08	182.6
49	125.0	100.0	-20.0	2.162	0.16606D+08	0.35895D+08	111.3
50	130.0	100.0	-20.0	1.599	0.25415D+08	0.40648D+08	128.4
51	135.0	100.0	-20.0	1.829	0.32661D+08	0.59737D+08	171.6
52	105.0	100.0	0.0	2.924	0.96873D+07	0.28325D+08	101.5
53	110.0	100.0	0.0	1.878	0.16304D+08	0.30620D+08	118.7
54	115.0	100.0	0.0	1.895	0.24106D+08	0.45676D+08	159.6
55	85.0	100.0	20.0	4.547	0.41760D+07	0.18986D+08	90.5
56	90.0	100.0	20.0	3.092	0.83321D+07	0.25764D+08	107.9
57	95.0	100.0	20.0	2.214	0.14510D+08	0.32129D+08	146.2
58	65.0	100.0	40.0	circle does not intercept slope			
59	70.0	100.0	40.0	3.970	0.22094D+07	0.87721D+07	69.5
60	75.0	100.0	40.0	3.625	0.65209D+07	0.23639D+08	128.3
61	145.0	120.0	-40.0	3.498	0.96371D+07	0.33715D+08	90.5
62	150.0	120.0	-40.0	2.286	0.18398D+08	0.42050D+08	112.9
63	155.0	120.0	-40.0	2.463	0.28923D+08	0.71226D+08	178.6
64	125.0	120.0	-20.0	7.279	0.37146D+07	0.27040D+08	76.6
65	130.0	120.0	-20.0	3.480	0.97441D+07	0.33910D+08	102.6
66	135.0	120.0	-20.0	2.546	0.18879D+08	0.48065D+08	167.9
67	105.0	120.0	0.0	F.S. greater than 10			
67	105.0	120.0	0.0	%17.883	0.29662D+06	0.53044D+07	12.5
68	110.0	120.0	0.0	7.423	0.16465D+07	0.12221D+08	27.0
69	115.0	120.0	0.0	4.417	0.90549D+07	0.39992D+08	156.2
70	85.0	120.0	20.0	circle does not intercept slope			
71	90.0	120.0	20.0	circle does not intercept slope			
72	95.0	120.0	20.0	F.S. greater than 10			
72	95.0	120.0	20.0	%52.654	0.63637D+05	0.33508D+07	8.8
73	65.0	120.0	40.0	circle does not intercept slope			
74	70.0	120.0	40.0	circle does not intercept slope			
75	75.0	120.0	40.0	F.S. greater than 10			
75	75.0	120.0	40.0	%14.020	0.73617D+06	0.10321D+08	82.6

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	110.0	80.0	0.0	1.511
2	150.0	100.0	-40.0	1.538
3	145.0	80.0	-40.0	1.543
4	150.0	80.0	-40.0	1.557
5	130.0	80.0	-20.0	1.564
6	135.0	80.0	-20.0	1.571
7	125.0	80.0	-20.0	1.575
8	145.0	100.0	-40.0	1.577
9	95.0	80.0	20.0	1.598
10	130.0	100.0	-20.0	1.599

Problem Title: Highland Estates: Profile No. 10

User's Name: P. F.
Date: 09-12-1994

GENERAL DATA

GS-10 (REVISED)

- SEISMIC: 0.21g
- STRENGTH INCREASE ALLOWED

UNITS

Unit Weight of Water: 62.43

GEOMETRY

Number of Sections : 9

Section	X	Y-crk.	Y-grd.
1	-100.0	47.0	47.0
2	-35.0	47.0	47.0
3	15.0	47.0	47.0
4	15.0	47.0	47.0
5	30.0	47.0	47.0
6	35.0	70.0	70.0
7	40.0	75.0	75.0
8	95.0	110.0	110.0
9	200.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 40 -40
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 5
Y-direction: 5

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .21
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3
1	-100.0	47.0	47.0	150.0
2	-35.0	47.0	47.0	150.0
3	15.0	47.0	70.0	150.0
4	15.0	75.0	75.0	150.0
5	30.0	75.0	75.0	150.0
6	35.0	75.0	75.0	150.0
7	40.0	75.0	75.0	150.0
8	95.0	110.0	110.0	150.0
9	200.0	115.0	115.0	150.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	145.0	40.0	-40.0	1.886	0.47380D+08	0.89380D+08	173.5
2	150.0	40.0	-40.0	1.881	0.56463D+08	0.10619D+09	187.6
3	155.0	40.0	-40.0	1.879	0.66416D+08	0.12478D+09	201.5
4	125.0	40.0	-20.0	1.867	0.38712D+08	0.72290D+08	163.8
5	130.0	40.0	-20.0	1.872	0.46279D+08	0.86655D+08	177.5
6	135.0	40.0	-20.0	1.886	0.54227D+08	0.10225D+09	191.1
7	105.0	40.0	0.0	1.815	0.30756D+08	0.55811D+08	153.1
8	110.0	40.0	0.0	1.848	0.36283D+08	0.67043D+08	166.5
9	115.0	40.0	0.0	1.877	0.42408D+08	0.79600D+08	179.7
10	85.0	40.0	20.0	1.876	0.21465D+08	0.40265D+08	140.9
11	90.0	40.0	20.0	1.907	0.26002D+08	0.49575D+08	153.8
12	95.0	40.0	20.0	1.933	0.31173D+08	0.60268D+08	166.6
13	65.0	40.0	40.0	1.915	0.14217D+08	0.27229D+08	124.3
14	70.0	40.0	40.0	1.960	0.17607D+08	0.34514D+08	136.5

15	75.0	40.0	40.0	1.999	0.21498D+08	0.42972D+08	148.7
16	145.0	60.0	-40.0	1.689	0.42833D+08	0.72325D+08	158.4
17	150.0	60.0	-40.0	1.688	0.51689D+08	0.87230D+08	172.9
18	155.0	60.0	-40.0	1.701	0.61551D+08	0.10468D+09	189.6
19	125.0	60.0	-20.0	1.656	0.34842D+08	0.57699D+08	149.1
20	130.0	60.0	-20.0	1.663	0.42306D+08	0.70344D+08	163.3
21	135.0	60.0	-20.0	1.670	0.50817D+08	0.84874D+08	178.1
22	105.0	60.0	0.0	1.574	0.27968D+08	0.44022D+08	139.0
23	110.0	60.0	0.0	1.601	0.33875D+08	0.54247D+08	152.8
24	115.0	60.0	0.0	1.626	0.40476D+08	0.65814D+08	166.4
25	85.0	60.0	20.0	1.583	0.19509D+08	0.30877D+08	127.6
26	90.0	60.0	20.0	1.653	0.23713D+08	0.39199D+08	140.9
27	95.0	60.0	20.0	1.673	0.29036D+08	0.48592D+08	154.1
28	65.0	60.0	40.0	1.646	0.12088D+08	0.19903D+08	112.1
29	70.0	60.0	40.0	1.706	0.15331D+08	0.26147D+08	124.7
30	75.0	60.0	40.0	1.722	0.19400D+08	0.33410D+08	137.1
31	145.0	80.0	-40.0	1.509	0.37506D+08	0.56608D+08	141.2
32	150.0	80.0	-40.0	1.525	0.45604D+08	0.69540D+08	156.6
33	155.0	80.0	-40.0	1.628	0.54824D+08	0.89271D+08	186.2
34	125.0	80.0	-20.0	1.543	0.28087D+08	0.43347D+08	132.3
35	130.0	80.0	-20.0	1.534	0.35460D+08	0.54399D+08	147.4
36	135.0	80.0	-20.0	1.540	0.46255D+08	0.71227D+08	175.0
37	105.0	80.0	0.0	1.573	0.21293D+08	0.33488D+08	122.6
38	110.0	80.0	0.0	1.482	0.27661D+08	0.40994D+08	137.4
39	115.0	80.0	0.0	1.610	0.33233D+08	0.53507D+08	162.8
40	85.0	80.0	20.0	1.633	0.14197D+08	0.23185D+08	111.9
41	90.0	80.0	20.0	1.597	0.18950D+08	0.30255D+08	126.2
42	95.0	80.0	20.0	1.571	0.24855D+08	0.39035D+08	149.0
43	65.0	80.0	40.0	2.287	0.68965D+07	0.15772D+08	97.7
44	70.0	80.0	40.0	1.807	0.10746D+08	0.19417D+08	111.2
45	75.0	80.0	40.0	1.776	0.15119D+08	0.26853D+08	130.7
46	145.0	100.0	-40.0	1.544	0.27202D+08	0.41987D+08	120.4
47	150.0	100.0	-40.0	1.506	0.35779D+08	0.53881D+08	137.5
48	155.0	100.0	-40.0	1.657	0.45109D+08	0.74740D+08	182.6
49	125.0	100.0	-20.0	2.127	0.16840D+08	0.35812D+08	111.3
50	130.0	100.0	-20.0	1.568	0.25839D+08	0.40506D+08	128.4
51	135.0	100.0	-20.0	1.796	0.33183D+08	0.59598D+08	171.6
52	105.0	100.0	0.0	2.888	0.97912D+07	0.28277D+08	101.5
53	110.0	100.0	0.0	1.848	0.16528D+08	0.30540D+08	118.7
54	115.0	100.0	0.0	1.864	0.24451D+08	0.45574D+08	159.6
55	85.0	100.0	20.0	4.499	0.42139D+07	0.18960D+08	90.5
56	90.0	100.0	20.0	3.055	0.84169D+07	0.25712D+08	107.9
57	95.0	100.0	20.0	2.181	0.14699D+08	0.32065D+08	146.2
58	65.0	100.0	40.0	circle does not intercept slope			
59	70.0	100.0	40.0	3.919	0.22358D+07	0.87619D+07	69.5
60	75.0	100.0	40.0	3.582	0.65887D+07	0.23599D+08	128.3
61	145.0	120.0	-40.0	3.459	0.97346D+07	0.33676D+08	90.5
62	150.0	120.0	-40.0	2.250	0.18649D+08	0.41958D+08	112.9
63	155.0	120.0	-40.0	2.422	0.29351D+08	0.71101D+08	178.6
64	125.0	120.0	-20.0	7.219	0.37435D+07	0.27023D+08	76.6
65	130.0	120.0	-20.0	3.442	0.98391D+07	0.33866D+08	102.6
66	135.0	120.0	-20.0	2.503	0.19170D+08	0.47982D+08	167.9
67	105.0	120.0	0.0	F.S. greater than 10			
67	105.0	120.0	0.0	%17.788	0.29811D+06	0.53029D+07	12.5
68	110.0	120.0	0.0	7.378	0.16553D+07	0.12213D+08	27.0
69	115.0	120.0	0.0	4.354	0.91734D+07	0.39941D+08	156.2
70	85.0	120.0	20.0	circle does not intercept slope			
71	90.0	120.0	20.0	circle does not intercept slope			
72	95.0	120.0	20.0	F.S. greater than 10			
72	95.0	120.0	20.0	%52.485	0.63836D+05	0.33505D+07	8.8
73	65.0	120.0	40.0	circle does not intercept slope			
74	70.0	120.0	40.0	circle does not intercept slope			
75	75.0	120.0	40.0	F.S. greater than 10			
75	75.0	120.0	40.0	%13.697	0.75326D+06	0.10318D+08	82.6

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	110.0	80.0	0.0	1.482
2	150.0	100.0	-40.0	1.506
3	145.0	80.0	-40.0	1.509
4	150.0	80.0	-40.0	1.525
5	130.0	80.0	-20.0	1.534
6	135.0	80.0	-20.0	1.540
7	125.0	80.0	-20.0	1.543
8	145.0	100.0	-40.0	1.544
9	130.0	100.0	-20.0	1.568
10	95.0	80.0	20.0	1.571

Problem Title: Highland Estates: Profile No. 10

User's Name:P. F.
Date: 09-12-1994

GENERAL DATA

UNITS

Unit Weight of Water: 62.43

GS-10 (REVISED)
• SEISMIC: 0.22g
• STRENGTH INCREASE ALLOWED

GEOMETRY

Number of Sections : 9

Section	X	Y-crk.	Y-grd.
1	-100.0	47.0	47.0
2	-35.0	47.0	47.0
3	15.0	47.0	47.0
4	15.0	47.0	47.0
5	30.0	47.0	47.0
6	35.0	70.0	70.0
7	40.0	75.0	75.0
8	95.0	110.0	110.0
9	200.0	115.0	115.0

CIRCLE DATA

Coordinates of first circle (X,Y): 40 -40
Intervals of circle coordinates
X-direction: 20
Y-direction: 20
Number of intervals
X-direction: 5
Y-direction: 5

Elevation of upper-most tangent: 105
Tangent interval: 5
Number of tangents: 3

CONTROL DATA/ANALYSIS OPTIONS

Seismic coefficient: .22
Number of slices: 10

Total stress analysis

Soil parameters defined in SOIL PROPERTIES

SOIL PROPERTIES

Layer Number	Cohesion	Friction Angle	Unit Weight
1	4000.0	30.0	130.0
2	2250.0	20.0	130.0
3	1500.0	30.0	130.0

SOIL PROFILE

Section Number	X	elevation to bottom of layer, layer number 1	2	3
1	-100.0	47.0	47.0	150.0
2	-35.0	47.0	47.0	150.0
3	15.0	47.0	70.0	150.0
4	15.0	75.0	75.0	150.0
5	30.0	75.0	75.0	150.0
6	35.0	75.0	75.0	150.0
7	40.0	75.0	75.0	150.0
8	95.0	110.0	110.0	150.0
9	200.0	115.0	115.0	150.0

RESULTS

circle	radius	X-center	Y-center	F.S.	ROVER	RESIST	ARCL
1	145.0	40.0	-40.0	1.844	0.48375D+08	0.89211D+08	173.5
2	150.0	40.0	-40.0	1.839	0.57637D+08	0.10599D+09	187.6
3	155.0	40.0	-40.0	1.837	0.67786D+08	0.12456D+09	201.5
4	125.0	40.0	-20.0	1.827	0.39498D+08	0.72148D+08	163.8
5	130.0	40.0	-20.0	1.832	0.47210D+08	0.86492D+08	177.5
6	135.0	40.0	-20.0	1.846	0.55300D+08	0.10207D+09	191.1
7	105.0	40.0	0.0	1.775	0.31372D+08	0.55693D+08	153.1
8	110.0	40.0	0.0	1.809	0.36986D+08	0.66911D+08	166.5
9	115.0	40.0	0.0	1.839	0.43208D+08	0.79453D+08	179.7
10	85.0	40.0	20.0	1.839	0.21848D+08	0.40182D+08	140.9
11	90.0	40.0	20.0	1.870	0.26457D+08	0.49482D+08	153.8
12	95.0	40.0	20.0	1.897	0.31714D+08	0.60162D+08	166.6
13	65.0	40.0	40.0	1.880	0.14454D+08	0.27174D+08	124.3
14	70.0	40.0	40.0	1.925	0.17897D+08	0.34451D+08	136.5

15	75.0	40.0	40.0	1.963	0.21851D+08	0.42899D+08	148.7
16	145.0	60.0	-40.0	1.653	0.43650D+08	0.72150D+08	158.4
17	150.0	60.0	-40.0	1.653	0.52658D+08	0.87024D+08	172.9
18	155.0	60.0	-40.0	1.666	0.62687D+08	0.10445D+09	189.6
19	125.0	60.0	-20.0	1.622	0.35474D+08	0.57553D+08	149.1
20	130.0	60.0	-20.0	1.630	0.43060D+08	0.70176D+08	163.3
21	135.0	60.0	-20.0	1.637	0.51715D+08	0.84682D+08	178.1
22	105.0	60.0	0.0	1.542	0.28470D+08	0.43897D+08	139.0
23	110.0	60.0	0.0	1.570	0.34469D+08	0.54105D+08	152.8
24	115.0	60.0	0.0	1.595	0.41173D+08	0.65653D+08	166.4
25	85.0	60.0	20.0	1.552	0.19831D+08	0.30787D+08	127.6
26	90.0	60.0	20.0	1.624	0.24079D+08	0.39101D+08	140.9
27	95.0	60.0	20.0	1.644	0.29481D+08	0.48478D+08	154.1
28	65.0	60.0	40.0	1.618	0.12264D+08	0.19846D+08	112.1
29	70.0	60.0	40.0	1.678	0.15542D+08	0.26083D+08	124.7
30	75.0	60.0	40.0	1.695	0.19669D+08	0.33334D+08	137.1
31	145.0	80.0	-40.0	1.477	0.38202D+08	0.56426D+08	141.2
32	150.0	80.0	-40.0	1.494	0.46419D+08	0.69334D+08	156.6
33	155.0	80.0	-40.0	1.597	0.55767D+08	0.89057D+08	186.2
34	125.0	80.0	-20.0	1.513	0.28561D+08	0.43209D+08	132.3
35	130.0	80.0	-20.0	1.505	0.36043D+08	0.54237D+08	147.4
36	135.0	80.0	-20.0	1.510	0.47055D+08	0.71034D+08	175.0
37	105.0	80.0	0.0	1.543	0.21635D+08	0.33380D+08	122.6
38	110.0	80.0	0.0	1.454	0.28104D+08	0.40859D+08	137.4
39	115.0	80.0	0.0	1.582	0.33743D+08	0.53368D+08	162.8
40	85.0	80.0	20.0	1.604	0.14404D+08	0.23108D+08	111.9
41	90.0	80.0	20.0	1.569	0.19223D+08	0.30162D+08	126.2
42	95.0	80.0	20.0	1.543	0.25219D+08	0.38925D+08	149.0
43	65.0	80.0	40.0	2.253	0.69789D+07	0.15722D+08	97.7
44	70.0	80.0	40.0	1.780	0.10878D+08	0.19364D+08	111.2
45	75.0	80.0	40.0	1.750	0.15307D+08	0.26789D+08	130.7
46	145.0	100.0	-40.0	1.511	0.27688D+08	0.41834D+08	120.4
47	150.0	100.0	-40.0	1.475	0.36395D+08	0.53696D+08	137.5
48	155.0	100.0	-40.0	1.626	0.45857D+08	0.74551D+08	182.6
49	125.0	100.0	-20.0	2.092	0.17075D+08	0.35729D+08	111.3
50	130.0	100.0	-20.0	1.537	0.26263D+08	0.40365D+08	128.4
51	135.0	100.0	-20.0	1.764	0.33706D+08	0.59460D+08	171.6
52	105.0	100.0	0.0	2.853	0.98950D+07	0.28229D+08	101.5
53	110.0	100.0	0.0	1.818	0.16752D+08	0.30460D+08	118.7
54	115.0	100.0	0.0	1.834	0.24796D+08	0.45472D+08	159.6
55	85.0	100.0	20.0	4.453	0.42519D+07	0.18934D+08	90.5
56	90.0	100.0	20.0	3.018	0.85018D+07	0.25660D+08	107.9
57	95.0	100.0	20.0	2.149	0.14888D+08	0.32002D+08	146.2
58	65.0	100.0	40.0	circle does not intercept slope			
59	70.0	100.0	40.0	3.869	0.22622D+07	0.87517D+07	69.5
60	75.0	100.0	40.0	3.539	0.66565D+07	0.23560D+08	128.3
61	145.0	120.0	-40.0	3.421	0.98321D+07	0.33636D+08	90.5
62	150.0	120.0	-40.0	2.215	0.18900D+08	0.41866D+08	112.9
63	155.0	120.0	-40.0	2.383	0.29780D+08	0.70977D+08	178.6
64	125.0	120.0	-20.0	7.159	0.37724D+07	0.27006D+08	76.6
65	130.0	120.0	-20.0	3.405	0.99341D+07	0.33821D+08	102.6
66	135.0	120.0	-20.0	2.461	0.19461D+08	0.47900D+08	167.9
67	105.0	120.0	0.0	F.S. greater than 10			
67	105.0	120.0	0.0	%17.695	0.29961D+06	0.53014D+07	12.5
68	110.0	120.0	0.0	7.334	0.16642D+07	0.12205D+08	27.0
69	115.0	120.0	0.0	4.293	0.92919D+07	0.39890D+08	156.2
70	85.0	120.0	20.0	circle does not intercept slope			
71	90.0	120.0	20.0	circle does not intercept slope			
72	95.0	120.0	20.0	F.S. greater than 10			
72	95.0	120.0	20.0	%52.317	0.64035D+05	0.33501D+07	8.8
73	65.0	120.0	40.0	circle does not intercept slope			
74	70.0	120.0	40.0	circle does not intercept slope			
75	75.0	120.0	40.0	F.S. greater than 10			
75	75.0	120.0	40.0	%13.390	0.77036D+06	0.10315D+08	82.6

CRITICAL CIRCLES

circle	radius	X-center	Y-center	F.S.
1	110.0	80.0	0.0	1.454
2	150.0	100.0	-40.0	1.475
3	145.0	80.0	-40.0	1.477
4	150.0	80.0	-40.0	1.494
5	130.0	80.0	-20.0	1.505
6	135.0	80.0	-20.0	1.510
7	145.0	100.0	-40.0	1.511
8	125.0	80.0	-20.0	1.513
9	130.0	100.0	-20.0	1.537
10	105.0	60.0	0.0	1.542

TENSAR® Earth Technologies, Inc.

3000 Corporate Center Drive
Suite 370
Morrow, Georgia 30260
404 • 968 • 7600

March 16, 1993

Mr. Steve Miller
Tensar Earth Technologies, Inc.
1250 Aviation Avenue, Suite 200-E
San Jose, California 95110

Preliminary Design
Highland Terrace Project
San Mateo County, California
Project No. S93204

Dear Mr. Miller:

Tensar Earth Technologies, Inc. is pleased to present our preliminary design for the **TENSAR** reinforced slope and retaining walls for the above project. The following design information is considered preliminary and a final design should be performed prior to construction. Information for our design of the reinforced slope and retaining walls was based on cross sections provided to us on March 5, 1993. Our **"TENSLO1"** and **"TENSWAL"** computer programs were used to perform the analyses.

The following paragraphs provides the project information, design methods, and results of our analyses. In addition, our recommended preliminary design for the reinforced slope and retaining walls including geogrid type, length and spacing are presented.

Please note that the design information contained in this report has been developed based on specific mechanical properties of Tensar materials. These properties have been developed based on extensive testing and experience with existing installations. The results of our analyses should not be used for other materials.

PROJECT INFORMATION

Based on cross section information provided to us on March 5, 1993, the project will consist of a **Tensar** reinforced slope (Section 7) and two **Tensar**/Keystone reinforced retaining walls (Sections 1 and 6). Based on our telephone conversation on March 12, 1993 several schemes are anticipated at this point in time. These schemes are presented below.

SECTION 7

The cross section information indicates a 50 feet high 1H:1V (horizontal:vertical) slope with an approximate 10 feet high retaining wall at the toe of the slope. TET proposes to replace this approach with a 30 to 60 feet high 1H:1V reinforced slope or a 37.5 feet 2H:1V slope with a 24.67 feet reinforced retaining wall at the toe of the slope. Computer model sections of 30 and 60 feet high 1H:1V slopes are included in the appendix.

SECTION 6

Information for this section indicates that a 24 feet high reinforced retaining wall will be constructed. The wall will have level backfill and level toe condition in front of the wall.

SECTION 1

Information for this section indicates that a 27.33 feet high reinforced retaining wall will be constructed. This wall will also have level backfill, but will have an approximate 10 feet high 2H:1V slope at the toe of the wall. A computer model section (for global stability analysis) is included in the appendix.

SOIL PARAMETERS

Based upon our conversation on March 12, 1993, the following soil parameters were utilized to perform the analyses. These soil parameters should be verified by the geotechnical engineer, prior to a final design.

DESCRIPTION	ϕ (deg)	C (psf)	γ (pcf)
Reinforced Soils	30	0*	115
Retained Soils	30	0*	115
Foundation Soils	30	500	115

Maximum Allowable Bearing Pressure = 5000 psf
Seismic acceleration = 0.15 g (1H:1V slope only)

* - 200 psf cohesion was utilized in analyses of the 1H:1V slope (section 7) and in the global analyses of the 27.33 feet high wall (section 1).

A uniform surcharge of 250 psf at the top of the slope and walls was used to model roadway loading conditions and construction traffic loading during installation. It was assumed that the foundation loads from the structure above section 1 will not influence the stability of this wall. It was also assumed that adequate drainage provisions will be provided so that hydrostatic forces will not affect the stability of the slope and walls.

DESIGN METHODOLOGY AND RESULTS

Our **"TENSLO1"** and **"TENSWAL"** computer programs was used to perform the analyses. **"TENSLO1"** considers circular failure surfaces and uses the modified Bishop's Method of slices to determine the factor of safety. The program incorporates the effects of Tensar geogrid reinforcing in the calculation of the factor of safety by including an additional resisting force.

Tensar UX1400HT was used as the primary reinforcement for the 1H:1V slope, while UX1500HT and UX1600HT were used as the primary reinforcement for the retaining walls. Safe allowable working strengths of 1524, 2571, and 3286 pounds per linear foot (plf) were used in the design calculations for UX1400HT, UX1500HT, and UX1600HT, respectively. Due to short term loading conditions present during a seismic event, the allowable working strength of UX1400HT was increased by 1/3 to 2030 plf. The magnitude of available geogrid tensile force depends on the design strength of the geogrid and the length of geogrid extending behind the potential failure plane. A soil-geogrid interaction coefficient of 0.7 was assumed in calculating pull-out strength.

RESULTS

The results of our analyses for the 1H:1V slope are presented in the attached computer graphic outputs and a typical section. The results indicate that the global stability is adequate based on a static safety factor equal to or exceeding 1.5 and a seismic factor of safety equal to or exceeding 1.10. We have also included our static input and output files for the 60 feet section. The results of our analyses for the 2H:1V slope and 24 feet high retaining wall for section 7 are presented in the attached **"TENSWAL"** computer outputs and a graphical representation showing the geogrid layout.

The results of our analyses for the retaining wall for section 6 are presented in the attached **"TENSWAL"** computer outputs and a graphical representation showing the geogrid layout.

The results of our analyses for the retaining wall for section 1 are presented in the attached **"TENSWAL"** computer outputs and the computer graphic output for global stability.

It should be noted that intermediate (surficial) reinforcement is also recommended for the 1H:1V slope to resist erosion and aid in compaction at the slope face. The typical section for the 1H:1V slope depicts this reinforcement.

GEOGRID QUANTITY ESTIMATE

Provided in the table below is a geogrid quantity estimate for each section and design approach based on the results of our analyses. The estimate is a neat quantity for the primary reinforcement only and does not take into account any excess for waste.

SECTION	**7**			**6**	**1**
PROPOSED DESIGN APPROACH	**60' SLOPE**	**30' SLOPE**	**SLOPE/ WALL COMBO.**	**24.0 FEET WALL**	**27.33 FEET WALL**
GEOGRID QUANTITY (FT^2/FT)	674.5 UX1400HT	144 UX1400HT	261 UX1600HT	54 UX1600HT	95 UX1600HT
			174 UX1500HT	54 UX1500HT	76 UX1500HT
MAXIMUM EMBEDMENT (FT)	55.0	24.0	43.5	13.5	19.0

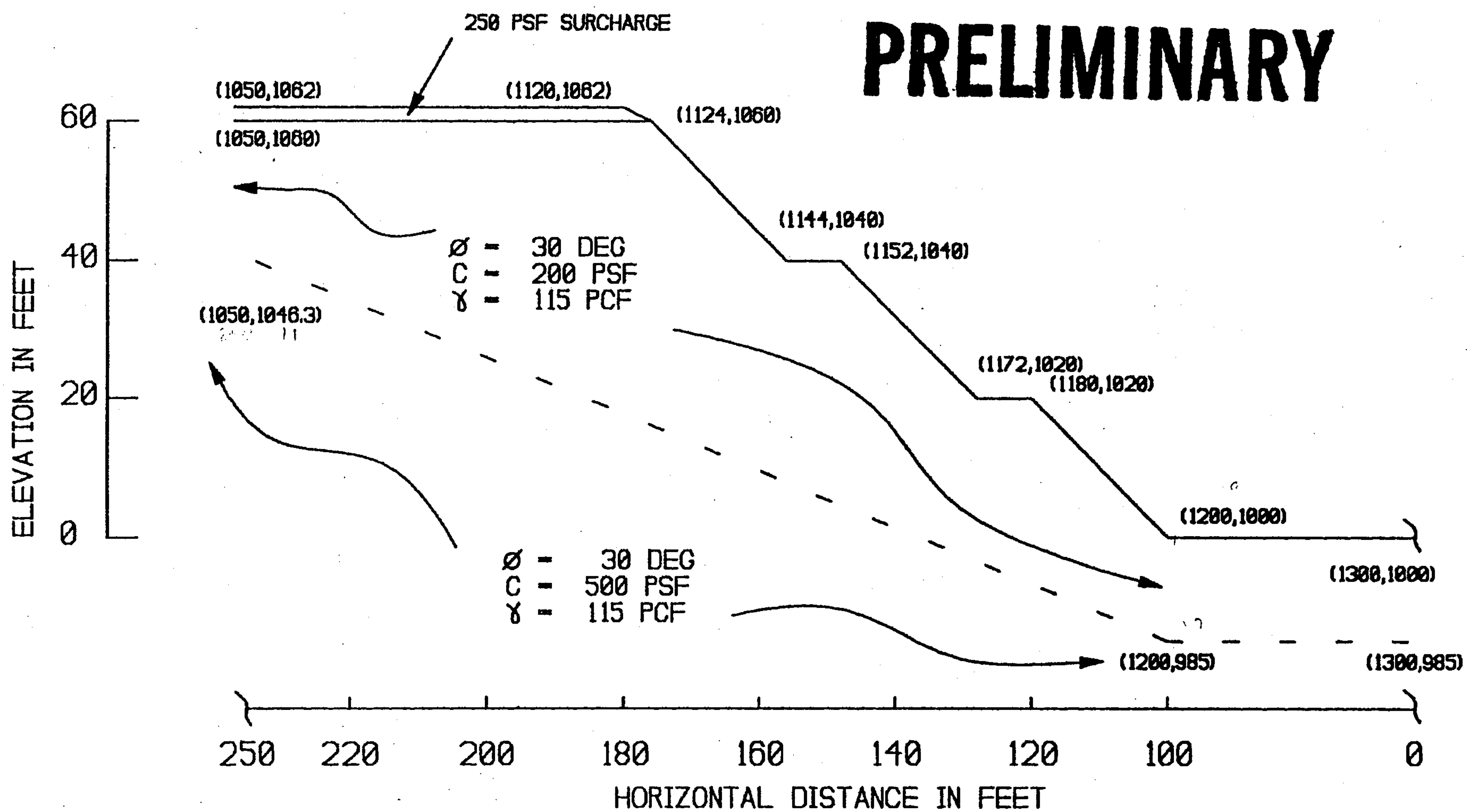
We appreciate the opportunity to provide design assistance on this project and look forward to working with you towards the successful completion of this project. Please do not hesitate to contact us if you have any questions.

Respectfully yours,

Michael S. Warren

Michael S. Warren
Design Engineer

Appendix
Distribution: addressee (2)



Tensar	TENSAR EARTH TECHNOLOGIES, INC. 3000 CORPORATE CENTER DR. STE. 370 MORROW, GA 30260 TEL: (404) 968-7600	03/15/93 ISSUED FOR REVIEW	COMPUTER MODEL SECTION SECTION 7 HIGHLAND TERRACE CALIFORNIA
			PROJECT # S93204 | DRAWING # 1 OF 3

msw 3/13/93 1 of 32

Highland Terrace
60' 1H:1V slope
Static input
Page 1

```
HIGHLAND TERRACE$S93204$SECT. 7; 60' 1H:1V$03/12/93$1
0.
0.0     0.
1.      1050.   1062.
2.      1120.   1062.
3.      1124.   1060.
4.      1144.   1040.
5.      1152.   1040.
6.      1172.   1020.
7.      1180.   1020.
8.      1200.   1000.
9.      1300.   1000.
10.     1050.   1060.
11.     1050.   1046.3
12.     1200.   985.
13.     1300.   985.
14.     1225.   980.
999.
8.
1.      2.      3.
2.      3.      3.
3.      4.      1.
4.      5.      1.
5.      6.      1.
6.      7.      1.
7.      8.      1.
8.      9.      1.
10.     3.      1.
11.     12.     2.
12.     13.     2.
999.
1.      .115    .200    30.     1.1     0.0
2.      .115    .500    30.     1.1     0.0
3.      .125    .050    10.     1.1     0.0
99.
2.      0.      0.0624
1050.   980.
1300.   980.
15.     .05
1002.   1157.5  1197.8  1.524   1.      0.7     1.5
1006.   1147.5  1193.8  1.524   1.      0.7     1.5
1010.   1137.5  1189.8  1.524   1.      0.7     1.5
1014.   1131.   1185.8  1.524   1.      0.7     1.5
1018.   1127.   1181.8  1.524   1.      0.7     1.5
1022.   1120.   1169.8  1.524   1.      0.7     1.5
1026.   1116.   1165.8  1.524   1.      0.7     1.5
1030.   1112.   1161.8  1.524   1.      0.7     1.5
1034.   1108.   1157.8  1.524   1.      0.7     1.5
1038.   1104.   1153.8  1.524   1.      0.7     1.5
1042.   1102.   1141.8  1.524   1.      0.7     1.5
1046.   1098.   1137.8  1.524   1.      0.7     1.5
```

MSW | 3/13/93 | 2 of

1050.	1094.	1133.8	1.524	1.	0.7	1.5
1054.	1090.	1129.8	1.524	1.	0.7	1.5
1058.	1086.	1125.8	1.524	1.	0.7	1.5
20.						
990.						
1010.						
10.						
SEARCH						
1200.	1145.	8.	8.			

MSW 3/13/93

TENSLO1 PROGRAM - VERSION 1.1 :
SLOPE STABILITY BY THE MODIFIED BISHOP METHOD.
TENSAR GEOGRID REINFORCEMENT CAN BE INCLUDED.
PROGRAM BY THE TENSAR CORPORATION, APRIL 1986.

HIGHLAND TERRACE$S93204$SECT. 7; 60' 1H:1V$03/12/93$1

*** UNITS - FEET AND KIPS

NO SEISMIC FORCES

SOIL INFORMATION:

NO.	GAMMA	CBAR	TAN(PHIBAR)	RU	RC
1	.115	.200	.577	1.10	.00
2	.115	.500	.577	1.10	.00
3	.125	.050	.176	1.10	.00

LINE ARRAY:

NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SLOPE	SOIL	TOPLINE
1	1050.00	1062.00	1120.00	1062.00	.0000	3	*
2	1120.00	1062.00	1124.00	1060.00	-.5000	3	*
3	1124.00	1060.00	1144.00	1040.00	-1.0000	1	*
4	1144.00	1040.00	1152.00	1040.00	.0000	1	*
5	1152.00	1040.00	1172.00	1020.00	-1.0000	1	*
6	1172.00	1020.00	1180.00	1020.00	.0000	1	*
7	1180.00	1020.00	1200.00	1000.00	-1.0000	1	*
8	1200.00	1000.00	1300.00	1000.00	.0000	1	*
9	1050.00	1060.00	1124.00	1060.00	.0000	1	
10	1050.00	1046.30	1200.00	985.00	-.4087	2	
11	1200.00	985.00	1300.00	985.00	.0000	2	

PHREATIC SURFACE COORDINATES:

NO.	X-COORD.	Y-COORD.
1	1050.00	980.00
2	1300.00	980.00

UNIT WEIGHT OF FLUID= .0624 KIP/FT3

TENSAR GEOGRID PLACEMENT:

LAYER	ELEVATION	X-LEFT	X-RIGHT
1	1002.00	1157.50	1197.80
2	1006.00	1147.50	1193.80
3	1010.00	1137.50	1189.80
4	1014.00	1131.00	1185.80
5	1018.00	1127.00	1181.80

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6	1022.00	1120.00	1169.80
7	1026.00	1116.00	1165.80
8	1030.00	1112.00	1161.80
9	1034.00	1108.00	1157.80
10	1038.00	1104.00	1153.80
11	1042.00	1102.00	1141.80
12	1046.00	1098.00	1137.80
13	1050.00	1094.00	1133.80
14	1054.00	1090.00	1129.80
15	1058.00	1086.00	1125.80

TENSAR GEOGRID STRENGTH DATA:

LAYER	ULT STREN KIPS/FT	WRKG STREN KIPS/FT	MU	FS-PULL OUT
1	1.52	1.52	.70	1.5
2	1.52	1.52	.70	1.5
3	1.52	1.52	.70	1.5
4	1.52	1.52	.70	1.5
5	1.52	1.52	.70	1.5
6	1.52	1.52	.70	1.5
7	1.52	1.52	.70	1.5
8	1.52	1.52	.70	1.5
9	1.52	1.52	.70	1.5
10	1.52	1.52	.70	1.5
11	1.52	1.52	.70	1.5
12	1.52	1.52	.70	1.5
13	1.52	1.52	.70	1.5
14	1.52	1.52	.70	1.5
15	1.52	1.52	.70	1.5

DERATING CONSTANT FOR NEAR-TANGENT GEOGRIDS= .05

APPROXIMATELY 20. SLICES WILL BE USED AT RMAX.
THE MINIMUM TANGENT ELEVATION FOR ANY FAILURE CIRCLE IS 990.00
THE MAXIMUM TANGENT ELEVATION FOR ANY FAILURE CIRCLE IS 1010.00

THERE ARE 10 INCREMENTS BETWEEN TANGENT LEVELS

AUTOMATIC SEARCH FOR CRITICAL CIRCLE:
INITIAL X = 1200.00, INITIAL Y = 1145.00.
DELX = 8.00, DELY = 8.00.

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X = 1200.00 , Y = 1145.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
155.00	1.850	1.850	1.850	30735.4	.000	26
153.00	1.777	1.777	1.777	28529.0	.000	26
151.00	1.697	1.697	1.697	26354.3	.000	25
149.00	1.609	1.609	1.600	24212.5	.009	23
147.00	1.529	1.529	1.486	22091.2	.042	21
145.00	1.502	1.502	1.421	20001.2	.081	18
143.00	1.539	1.539	1.413	17971.5	.126	16
146.50	1.525	1.525	1.473	21566.3	.052	21
146.00	1.528	1.528	1.460	21042.3	.067	20
145.50	1.519	1.519	1.446	20521.1	.073	20
144.50	1.509	1.509	1.419	19492.0	.090	17
144.00	1.516	1.516	1.417	18980.6	.099	17
143.50	1.530	1.530	1.415	18470.0	.115	16
141.00	1.613	1.613	1.406	16037.0	.207	16
139.00	1.622	1.622	1.399	14196.9	.223	16
137.00	1.635	1.635	1.393	12457.3	.243	15
135.00	1.652	1.652	1.390	10796.7	.263	15

THE LOWEST FACTOR OF SAFETY WAS 1.502 AT R = 145.00.

X = 1192.00 , Y = 1145.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
155.00	1.918	1.918	1.918	32759.0	.000	26
153.00	1.856	1.856	1.856	30555.8	.000	25
151.00	1.793	1.793	1.793	28374.4	.000	25
149.00	1.727	1.727	1.727	26236.0	.000	23
147.00	1.657	1.657	1.653	24118.9	.004	21
145.00	1.548	1.548	1.523	22034.6	.025	18
143.00	1.547	1.547	1.514	20003.6	.033	17
141.00	1.570	1.570	1.504	18072.3	.066	17
144.50	1.545	1.545	1.521	21519.3	.024	18
144.00	1.553	1.553	1.518	21009.8	.035	18
143.50	1.550	1.550	1.516	20500.7	.034	17
142.50	1.553	1.553	1.511	19512.2	.042	17
142.00	1.568	1.568	1.509	19026.6	.059	17
141.50	1.565	1.565	1.506	18546.6	.058	17
139.00	1.586	1.586	1.494	16231.4	.092	17
137.00	1.607	1.607	1.485	14482.5	.122	16
135.00	1.690	1.690	1.476	12814.3	.214	16

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THE LOWEST FACTOR OF SAFETY WAS 1.545 AT R = 144.50.

X = 1208.00 , Y = 1145.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
155.00	1.797	1.797	1.797	28248.0	.000	27
153.00	1.703	1.703	1.703	26045.2	.000	26
151.00	1.607	1.607	1.554	23877.6	.053	23
149.00	1.605	1.605	1.492	21725.6	.113	22
147.00	1.627	1.627	1.430	19605.1	.196	21
150.50	1.601	1.601	1.538	23337.4	.062	23
150.00	1.604	1.604	1.523	22798.8	.081	23
149.50	1.599	1.599	1.507	22262.0	.092	23
148.50	1.608	1.608	1.476	21193.2	.132	22
148.00	1.606	1.606	1.461	20662.4	.145	22
147.50	1.616	1.616	1.446	20133.2	.171	22
145.00	1.550	1.550	1.336	17514.4	.214	17
143.00	1.561	1.561	1.331	15498.9	.230	15
146.50	1.616	1.616	1.415	19079.7	.201	21
146.00	1.604	1.604	1.399	18555.7	.205	21
145.50	1.591	1.591	1.382	18034.2	.209	20
144.50	1.553	1.553	1.335	17000.5	.218	17
144.00	1.555	1.555	1.333	16492.8	.222	17
143.50	1.557	1.557	1.332	15995.3	.225	15
141.00	1.579	1.579	1.328	13576.0	.251	15
139.00	1.600	1.600	1.327	11752.7	.274	15
137.00	1.635	1.635	1.329	10028.1	.306	15
135.00	1.679	1.679	1.337	8402.7	.342	13

THE LOWEST FACTOR OF SAFETY WAS 1.550 AT R = 145.00.

X = 1200.00 , Y = 1153.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
163.00	1.887	1.887	1.887	33081.7	.000	27
161.00	1.816	1.816	1.816	30766.0	.000	26
159.00	1.739	1.739	1.739	28478.1	.000	25
157.00	1.650	1.650	1.650	26225.0	.000	24
155.00	1.558	1.558	1.520	23987.6	.037	21
153.00	1.521	1.521	1.457	21781.6	.064	18
151.00	1.534	1.534	1.449	19636.2	.085	17
154.50	1.548	1.548	1.508	23433.6	.040	21
154.00	1.535	1.535	1.495	22880.6	.040	20

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153.50	1.520	1.520	1.481	22330.5	.039	20
152.50	1.519	1.519	1.455	21237.0	.064	18
152.00	1.527	1.527	1.453	20692.8	.074	17
151.50	1.528	1.528	1.451	20161.4	.077	17
149.00	1.565	1.565	1.441	17599.6	.123	16
147.00	1.626	1.626	1.435	15652.2	.191	16
145.00	1.658	1.658	1.429	13802.3	.228	16
143.00	1.671	1.671	1.424	12056.9	.247	15

THE LOWEST FACTOR OF SAFETY WAS 1.519 AT R = 152.50.

X = 1200.00 , Y = 1137.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
147.00	1.814	1.814	1.814	28379.6	.000	26
145.00	1.738	1.738	1.738	26290.3	.000	26
143.00	1.654	1.654	1.654	24234.3	.000	25
141.00	1.571	1.571	1.542	22198.4	.029	22
139.00	1.529	1.529	1.452	20203.2	.076	20
137.00	1.529	1.529	1.387	18223.2	.142	17
140.50	1.537	1.537	1.493	21696.9	.044	22
140.00	1.527	1.527	1.479	21195.1	.048	22
139.50	1.527	1.527	1.466	20694.8	.061	22
138.50	1.524	1.524	1.439	19705.7	.085	20
138.00	1.529	1.529	1.426	19209.2	.103	19
137.50	1.539	1.539	1.412	18715.5	.128	19
135.00	1.583	1.583	1.378	16305.0	.205	17
133.00	1.592	1.592	1.371	14476.7	.221	16
131.00	1.601	1.601	1.364	12749.0	.237	15
129.00	1.620	1.620	1.360	11099.2	.260	15
127.00	1.641	1.641	1.357	9539.5	.283	15

THE LOWEST FACTOR OF SAFETY WAS 1.524 AT R = 138.50.

X = 1192.00 , Y = 1137.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
147.00	1.880	1.880	1.880	30409.9	.000	26
145.00	1.817	1.817	1.817	28309.5	.000	25
143.00	1.751	1.751	1.751	26256.6	.000	24
141.00	1.683	1.683	1.683	24225.7	.000	23
139.00	1.608	1.608	1.602	22229.0	.006	21
137.00	1.525	1.525	1.487	20250.9	.039	18

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135.00	1.538	1.538	1.477	18333.9	.061	17
138.50	1.582	1.582	1.577	21732.7	.006	22
138.00	1.572	1.572	1.545	21233.9	.027	20
137.50	1.523	1.523	1.495	20742.4	.028	19
136.50	1.522	1.522	1.484	19764.6	.038	18
136.00	1.518	1.518	1.481	19283.9	.037	18
135.50	1.527	1.527	1.479	18808.7	.048	18
133.00	1.556	1.556	1.466	16513.1	.090	17
131.00	1.589	1.589	1.455	14780.7	.134	16
129.00	1.662	1.662	1.446	13127.1	.216	16
127.00	1.667	1.667	1.437	11558.8	.230	16

THE LOWEST FACTOR OF SAFETY WAS 1.518 AT R = 136.00.

X = 1208.00 , Y = 1137.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
147.00	1.764	1.764	1.764	25895.2	.000	27
145.00	1.679	1.679	1.663	23813.6	.015	25
143.00	1.665	1.665	1.528	21748.4	.137	23
141.00	1.655	1.655	1.465	19713.5	.190	22
139.00	1.610	1.610	1.402	17709.5	.207	21
137.00	1.533	1.533	1.305	15738.9	.228	16
135.00	1.548	1.548	1.302	13824.8	.246	15
138.50	1.599	1.599	1.387	17213.1	.212	21
138.00	1.588	1.588	1.370	16725.8	.217	19
137.50	1.575	1.575	1.353	16231.4	.222	19
136.50	1.536	1.536	1.304	15252.2	.233	16
136.00	1.540	1.540	1.303	14771.6	.237	16
135.50	1.542	1.542	1.302	14297.1	.240	16
133.00	1.571	1.571	1.299	12014.0	.271	15
131.00	1.598	1.598	1.301	10298.7	.298	15
129.00	1.643	1.643	1.306	8679.6	.337	14
127.00	1.703	1.703	1.321	7143.1	.382	13

THE LOWEST FACTOR OF SAFETY WAS 1.533 AT R = 137.00.

X = 1192.00 , Y = 1153.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
163.00	1.957	1.957	1.957	35107.4	.000	27
161.00	1.897	1.897	1.897	32788.4	.000	25
159.00	1.835	1.835	1.835	30503.0	.000	26

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157.00	1.772	1.772	1.772	28239.4	.000	24
155.00	1.703	1.703	1.703	26007.5	.000	22
153.00	1.623	1.623	1.609	23809.0	.014	21
151.00	1.583	1.583	1.551	21675.1	.031	17
149.00	1.597	1.597	1.542	19633.1	.055	17
152.50	1.588	1.588	1.566	23267.1	.022	19
152.00	1.578	1.578	1.556	22724.8	.021	18
151.50	1.586	1.586	1.554	22200.4	.032	17
150.50	1.580	1.580	1.549	21155.7	.031	17
150.00	1.577	1.577	1.547	20642.3	.030	17
149.50	1.587	1.587	1.544	20134.7	.042	17
147.00	1.600	1.600	1.533	17685.5	.067	17
145.00	1.624	1.624	1.525	15831.1	.099	17
143.00	1.635	1.635	1.516	14071.8	.119	16

THE LOWEST FACTOR OF SAFETY WAS 1.577 AT R = 150.00.

X = 1208.00 , Y = 1153.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
163.00	1.832	1.832	1.832	30598.0	.000	27
161.00	1.742	1.742	1.742	28284.6	.000	27
159.00	1.649	1.649	1.628	25994.4	.022	24
157.00	1.589	1.589	1.521	23732.1	.068	23
155.00	1.570	1.570	1.460	21503.4	.110	21
153.00	1.541	1.541	1.368	19296.0	.173	17
151.00	1.581	1.581	1.364	17160.0	.217	16
154.50	1.570	1.570	1.445	20949.0	.125	21
154.00	1.560	1.560	1.430	20395.8	.130	21
153.50	1.553	1.553	1.413	19845.0	.141	20
152.50	1.568	1.568	1.367	18753.1	.201	17
152.00	1.576	1.576	1.366	18211.8	.209	16
151.50	1.577	1.577	1.365	17682.7	.213	16
149.00	1.593	1.593	1.359	15141.5	.235	15
147.00	1.611	1.611	1.357	13209.8	.255	15
145.00	1.639	1.639	1.357	11380.9	.281	15
143.00	1.674	1.674	1.362	9650.6	.312	14

THE LOWEST FACTOR OF SAFETY WAS 1.541 AT R = 153.00.

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X = 1198.00 , Y = 1145.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
155.00	1.866	1.866	1.866	31275.9	.000	26
153.00	1.795	1.795	1.795	29078.4	.000	26
151.00	1.721	1.721	1.721	26903.9	.000	25
149.00	1.635	1.635	1.635	24766.2	.000	24
147.00	1.540	1.540	1.504	22642.4	.036	21
145.00	1.511	1.511	1.445	20551.1	.065	18
143.00	1.528	1.528	1.437	18525.0	.092	16
146.50	1.533	1.533	1.491	22117.0	.041	21
146.00	1.520	1.520	1.479	21592.9	.041	20
145.50	1.512	1.512	1.466	21071.3	.046	20
144.50	1.509	1.509	1.443	20035.2	.066	18
144.00	1.519	1.519	1.441	19525.1	.078	18
143.50	1.521	1.521	1.439	19015.6	.081	17
141.00	1.560	1.560	1.428	16588.1	.132	16
139.00	1.634	1.634	1.421	14744.8	.214	16
137.00	1.646	1.646	1.414	12994.3	.231	16
135.00	1.658	1.658	1.408	11340.6	.249	15

THE LOWEST FACTOR OF SAFETY WAS 1.509 AT R = 144.50.

X = 1202.00 , Y = 1145.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
155.00	1.836	1.836	1.836	30153.1	.000	26
153.00	1.758	1.758	1.758	27948.5	.000	26
151.00	1.672	1.672	1.672	25779.5	.000	25
149.00	1.581	1.581	1.549	23631.5	.032	24
147.00	1.538	1.538	1.470	21512.6	.068	22
145.00	1.509	1.509	1.398	19426.7	.111	17
143.00	1.573	1.573	1.391	17391.5	.181	16
146.50	1.537	1.537	1.456	20987.7	.080	21
146.00	1.525	1.525	1.442	20471.0	.083	20
145.50	1.523	1.523	1.428	19947.8	.096	19
144.50	1.523	1.523	1.396	18909.8	.127	17
144.00	1.528	1.528	1.394	18399.1	.134	17
143.50	1.542	1.542	1.393	17889.3	.149	16
141.00	1.602	1.602	1.384	15460.2	.218	16
139.00	1.612	1.612	1.378	13624.3	.233	16
137.00	1.629	1.629	1.374	11884.4	.255	15
135.00	1.651	1.651	1.373	10230.7	.279	15

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THE LOWEST FACTOR OF SAFETY WAS 1.509 AT R = 145.00.

X = 1200.00 , Y = 1147.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
157.00	1.859	1.859	1.859	31317.0	.000	27
155.00	1.786	1.786	1.786	29087.4	.000	26
153.00	1.707	1.707	1.707	26883.8	.000	25
151.00	1.618	1.618	1.613	24714.9	.005	23
149.00	1.537	1.537	1.495	22565.1	.042	21
147.00	1.508	1.508	1.430	20446.2	.077	18
145.00	1.536	1.536	1.422	18389.7	.114	16
148.50	1.524	1.524	1.482	22032.9	.042	21
148.00	1.532	1.532	1.469	21501.7	.064	20
147.50	1.521	1.521	1.455	20973.3	.066	20
146.50	1.508	1.508	1.428	19923.2	.080	18
146.00	1.518	1.518	1.425	19413.0	.093	17
145.50	1.524	1.524	1.424	18895.3	.100	16
143.00	1.594	1.594	1.415	16427.4	.179	16
141.00	1.628	1.628	1.408	14560.6	.220	16
139.00	1.641	1.641	1.402	12788.2	.239	16
137.00	1.657	1.657	1.398	11111.5	.259	15

THE LOWEST FACTOR OF SAFETY WAS 1.508 AT R = 147.00.

X = 1200.00 , Y = 1143.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
153.00	1.841	1.841	1.841	30145.6	.000	26
151.00	1.767	1.767	1.767	27971.1	.000	26
149.00	1.686	1.686	1.686	25825.5	.000	25
147.00	1.596	1.596	1.587	23708.2	.009	22
145.00	1.523	1.523	1.478	21617.4	.045	21
143.00	1.501	1.501	1.412	19562.6	.089	17
141.00	1.541	1.541	1.405	17553.4	.136	16
144.50	1.531	1.531	1.465	21099.7	.066	21
144.00	1.521	1.521	1.452	20583.0	.069	20
143.50	1.519	1.519	1.437	20069.0	.081	20
142.50	1.509	1.509	1.410	19052.5	.099	17
142.00	1.525	1.525	1.408	18548.3	.117	17
141.50	1.534	1.534	1.406	18050.1	.128	17
139.00	1.609	1.609	1.397	15646.7	.212	16
137.00	1.616	1.616	1.390	13833.4	.226	16

MSW | 3/13/93 | 12 of 32

135.00	1.631	1.631	1.384	12117.5	.247	15
133.00	1.649	1.649	1.381	10482.2	.268	15

THE LOWEST FACTOR OF SAFETY WAS 1.501 AT R = 143.00.

X = 1198.00 , Y = 1143.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
153.00	1.857	1.857	1.857	30687.8	.000	26
151.00	1.786	1.786	1.786	28518.7	.000	26
149.00	1.710	1.710	1.710	26372.0	.000	25
147.00	1.623	1.623	1.623	24263.1	.000	24
145.00	1.537	1.537	1.495	22168.2	.042	21
143.00	1.503	1.503	1.437	20106.0	.066	18
141.00	1.531	1.531	1.428	18106.5	.103	16
144.50	1.525	1.525	1.483	21650.1	.042	21
144.00	1.512	1.512	1.470	21133.3	.042	20
143.50	1.521	1.521	1.457	20618.9	.065	20
142.50	1.513	1.513	1.434	19597.2	.079	18
142.00	1.511	1.511	1.432	19094.4	.079	18
141.50	1.522	1.522	1.429	18603.8	.093	17
139.00	1.583	1.583	1.419	16197.4	.164	16
137.00	1.628	1.628	1.412	14381.0	.216	16
135.00	1.640	1.640	1.405	12656.3	.235	16
133.00	1.653	1.653	1.400	11025.5	.253	15

THE LOWEST FACTOR OF SAFETY WAS 1.503 AT R = 143.00.

X = 1202.00 , Y = 1143.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
153.00	1.827	1.827	1.827	29565.1	.000	26
151.00	1.749	1.749	1.749	27388.8	.000	26
149.00	1.661	1.661	1.661	25248.3	.000	25
147.00	1.559	1.559	1.517	23129.7	.042	23
145.00	1.538	1.538	1.462	21039.0	.076	22
143.00	1.520	1.520	1.390	18980.4	.130	17
141.00	1.587	1.587	1.383	16973.8	.205	16
144.50	1.530	1.530	1.448	20527.4	.083	20
144.00	1.530	1.530	1.434	20010.0	.096	20
143.50	1.532	1.532	1.419	19494.1	.113	19
142.50	1.526	1.526	1.388	18470.8	.138	17
142.00	1.544	1.544	1.386	17967.3	.158	17

MSW | 3/13/93 | 13 OF 32

141.50	1.574	1.574	1.384	17469.7	.190	17
139.00	1.596	1.596	1.376	15070.3	.221	16
137.00	1.606	1.606	1.369	13267.5	.237	15
135.00	1.626	1.626	1.366	11545.1	.260	15
133.00	1.649	1.649	1.365	9916.6	.284	15

THE LOWEST FACTOR OF SAFETY WAS 1.520 AT R = 143.00.

X = 1200.00 , Y = 1141.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
151.00	1.832	1.832	1.832	29556.3	.000	26
149.00	1.757	1.757	1.757	27419.2	.000	25
147.00	1.675	1.675	1.675	25297.1	.000	25
145.00	1.588	1.588	1.573	23205.6	.016	23
143.00	1.526	1.526	1.469	21143.7	.057	21
141.00	1.503	1.503	1.404	19116.0	.099	17
139.00	1.565	1.565	1.396	17135.4	.168	16
142.50	1.525	1.525	1.456	20633.2	.069	21
142.00	1.525	1.525	1.443	20123.8	.082	20
141.50	1.515	1.515	1.428	19622.9	.087	19
140.50	1.521	1.521	1.402	18613.1	.119	17
140.00	1.531	1.531	1.399	18116.2	.132	17
139.50	1.540	1.540	1.397	17625.2	.142	17
137.00	1.603	1.603	1.388	15256.5	.215	16
135.00	1.611	1.611	1.381	13470.1	.229	16
133.00	1.627	1.627	1.376	11777.9	.251	15
131.00	1.646	1.646	1.373	10167.7	.272	15

THE LOWEST FACTOR OF SAFETY WAS 1.503 AT R = 141.00.

X = 1198.00 , Y = 1141.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
151.00	1.847	1.847	1.847	30100.0	.000	26
149.00	1.776	1.776	1.776	27959.5	.000	26
147.00	1.700	1.700	1.700	25841.2	.000	25
145.00	1.611	1.611	1.611	23760.1	.000	24
143.00	1.529	1.529	1.487	21694.2	.042	21
141.00	1.503	1.503	1.428	19661.0	.076	18
139.00	1.540	1.540	1.419	17688.1	.121	16
142.50	1.517	1.517	1.474	21183.3	.042	21
142.00	1.516	1.516	1.462	20673.8	.055	20

MSW | 3/13/93 | 14 OF 32

141.50	1.515	1.515	1.448	20166.6	.067	20
140.50	1.505	1.505	1.425	19159.4	.080	18
140.00	1.516	1.516	1.423	18669.9	.093	17
139.50	1.523	1.523	1.420	18178.4	.102	17
137.00	1.614	1.614	1.410	15806.8	.204	16
135.00	1.622	1.622	1.403	14017.3	.219	16
133.00	1.635	1.635	1.396	12318.4	.239	16
131.00	1.649	1.649	1.391	10710.6	.258	15

THE LOWEST FACTOR OF SAFETY WAS 1.503 AT R = 141.00.

X = 1202.00 , Y = 1141.00 :

RADIUS	FS GEOGRIDS AT WORK STRESS	FS GEOGRIDS AT ULT STRESS	FS WITHOUT GEOGRIDS	DRIVING MOMENT: KIP-FT/FT	DELTA FS DUE TO GEOGRIDS	# SLICES
151.00	1.818	1.818	1.818	28977.7	.000	26
149.00	1.739	1.739	1.739	26829.9	.000	26
147.00	1.650	1.650	1.650	24717.1	.000	25
145.00	1.553	1.553	1.509	22627.2	.044	23
143.00	1.536	1.536	1.453	20571.1	.082	21
141.00	1.526	1.526	1.381	18534.2	.144	17
139.00	1.582	1.582	1.374	16556.3	.208	16
142.50	1.536	1.536	1.440	20059.2	.096	20
142.00	1.541	1.541	1.426	19549.2	.115	20
141.50	1.544	1.544	1.411	19040.7	.133	19
140.50	1.547	1.547	1.379	18031.9	.168	17
140.00	1.576	1.576	1.377	17535.6	.199	17
139.50	1.580	1.580	1.375	17045.2	.204	17
137.00	1.591	1.591	1.367	14680.6	.224	16
135.00	1.602	1.602	1.361	12902.3	.241	15
133.00	1.623	1.623	1.358	11206.0	.265	15
131.00	1.647	1.647	1.357	9602.7	.290	15

THE LOWEST FACTOR OF SAFETY WAS 1.526 AT R = 141.00.

MSW | 3/13/93 | 15 OF 32

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SUMMARY OF RESULTS FOR CRITICAL CIRCLE:
  X = 1200.00.
  Y = 1143.00.
  RADIUS OF CRITICAL CIRCLE = 143.00.

  MINIMUM FS =  1.501 (TENSAR GEOGRIDS AT WORKING STRESS).

  FS (TENSAR GEOGRIDS AT ULTIMATE STRESS) =  1.501.
  FS (UNREINFORCED) =  1.412.
  TOTAL DRIVING MOMENT =  19562.60  KIP-FT/FT
  CRITICAL CIRCLE EXTENDS TO ELEVATION 1000.00.
  ELEVATION OF TENSAR GEOGRIDS INTERSECTED BY CRITICAL CIRCLE :
      1002.00     (PULL OUT STRESS =      1.52 KIPS/FT )
          WITHIN BOTTOM  .05 OF RADIUS. STRENGTH DERATED TO  28.0 PERCENT.
      1006.00     (PULL OUT STRESS =      1.52 KIPS/FT )
          WITHIN BOTTOM  .05 OF RADIUS. STRENGTH DERATED TO  83.9 PERCENT.
      1010.00     (PULL OUT STRESS =      1.52 KIPS/FT )
      1014.00     (PULL OUT STRESS =      1.52 KIPS/FT )
      1018.00     (PULL OUT STRESS =      1.52 KIPS/FT )
      1022.00     (PULL OUT STRESS =      1.52 KIPS/FT )
      1026.00     (PULL OUT STRESS =      1.52 KIPS/FT )
      1030.00     (PULL OUT STRESS =       .79 KIPS/FT )

CALCULATIONS HAVE BEEN COMPLETED.
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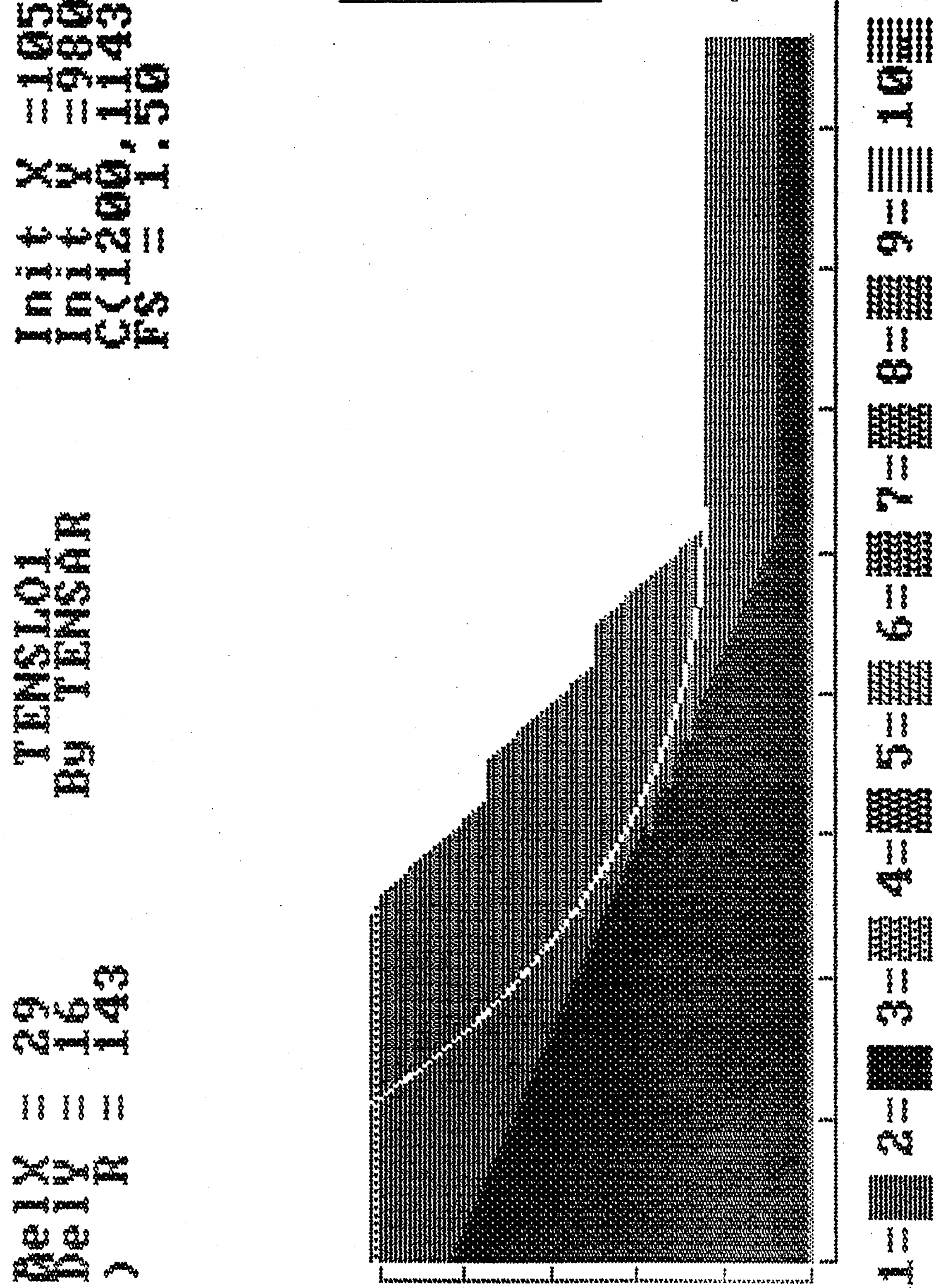
— | 3/ /93 | 32

60' 1H:1V SLOPE
SECTION 7 - STATIC

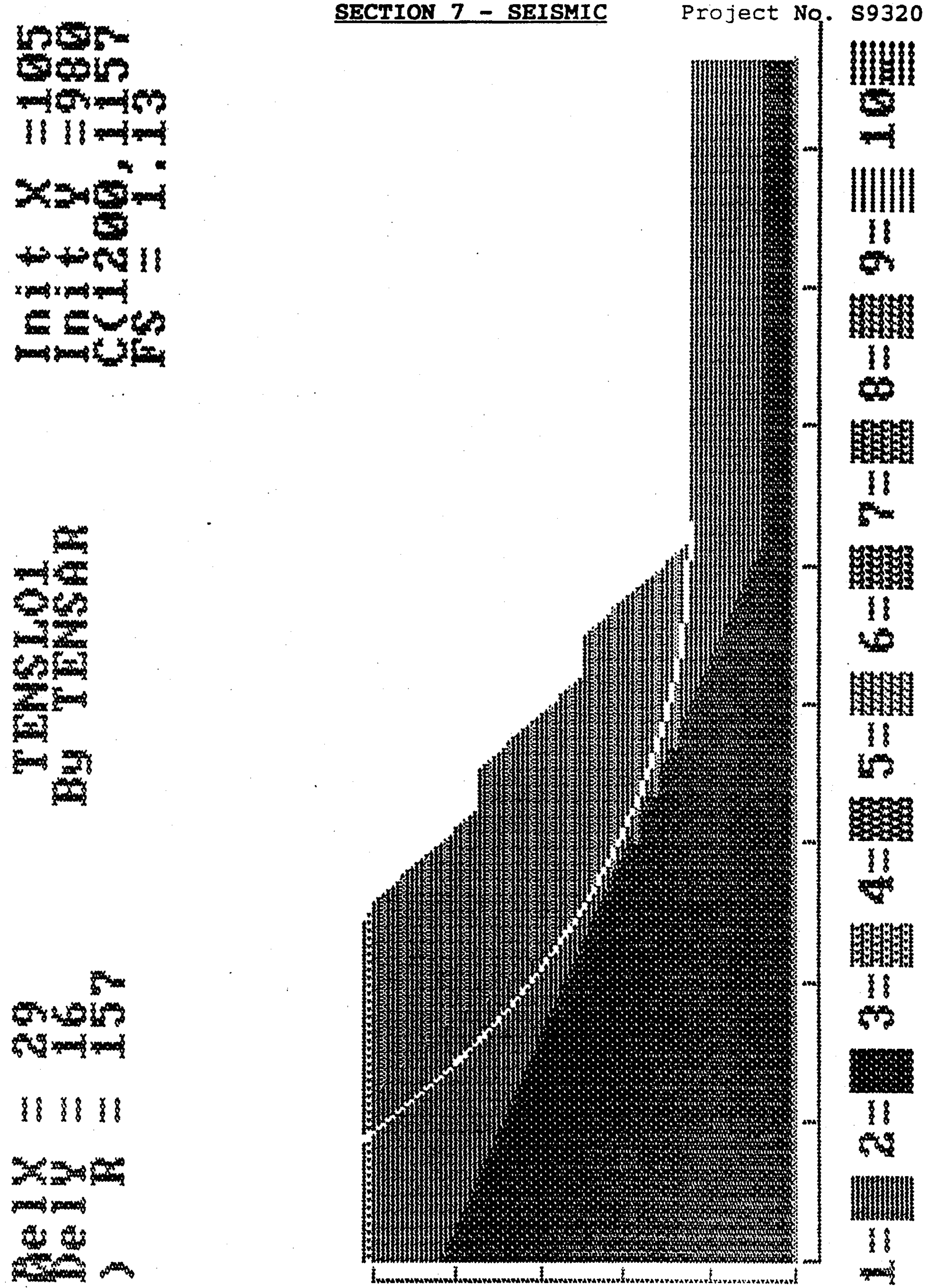
Project No. S93204



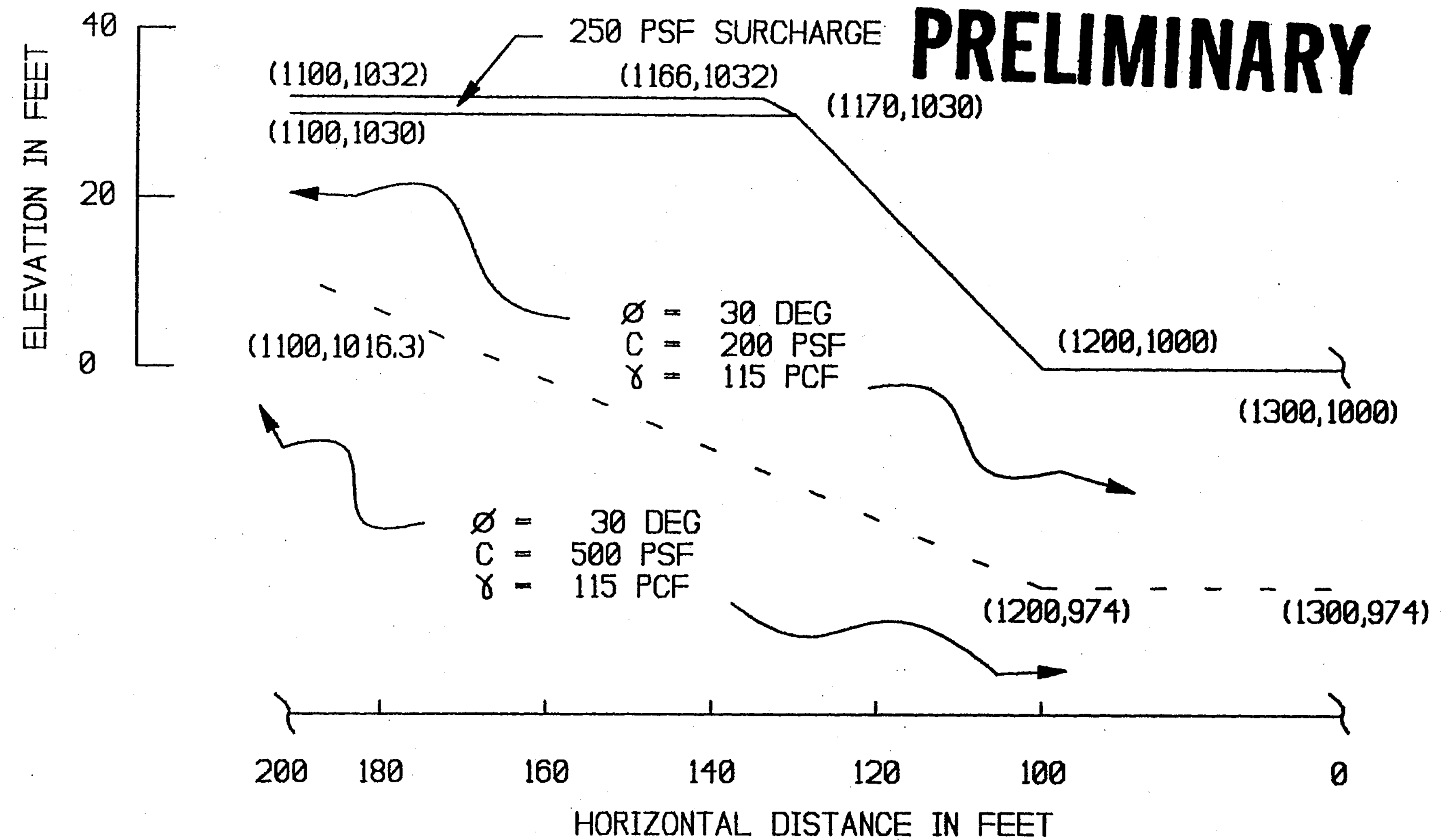
MSW | 3/13/93 | 17 of 32

60' 1H:1V SLOPE
SECTION 7 - SEISMIC

Project No. S93204



MSW | 3/13/93 | 18 of 32



Tensar

TENSAR EARTH TECHNOLOGIES, INC.
3000 CORPORATE CENTER DR. STE. 370
MORROW, GA 30260
TEL: (404) 968-7600

03/15/93 ISSUED FOR REVIEW

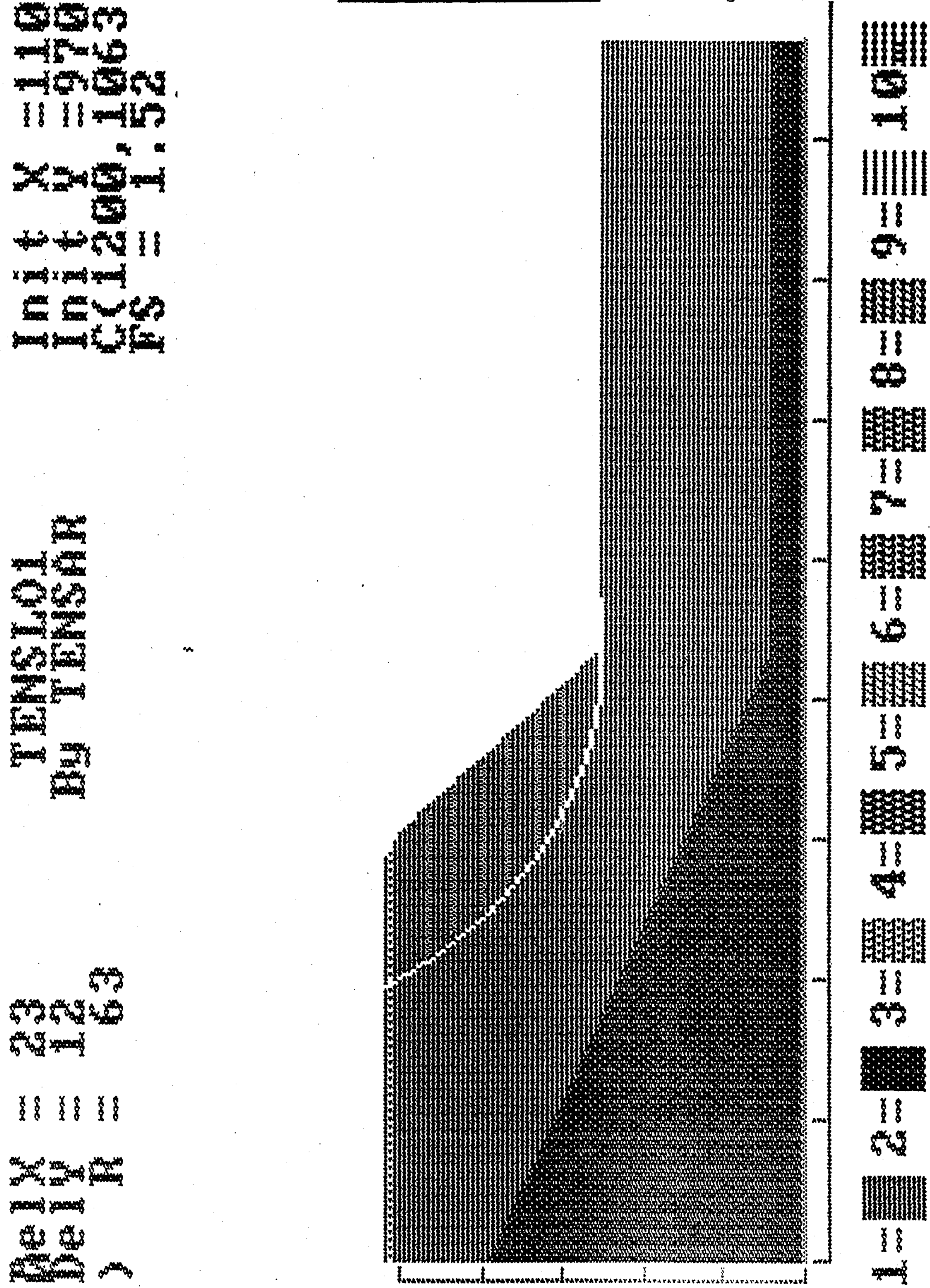
COMPUTER MODEL SECTION
SECTION 7
HIGHLAND TERRACE
CALIFORNIA

PROJECT # S93204 | DRAWING # 2 OF 3

MSW | 3/13/93 | 19 of 32

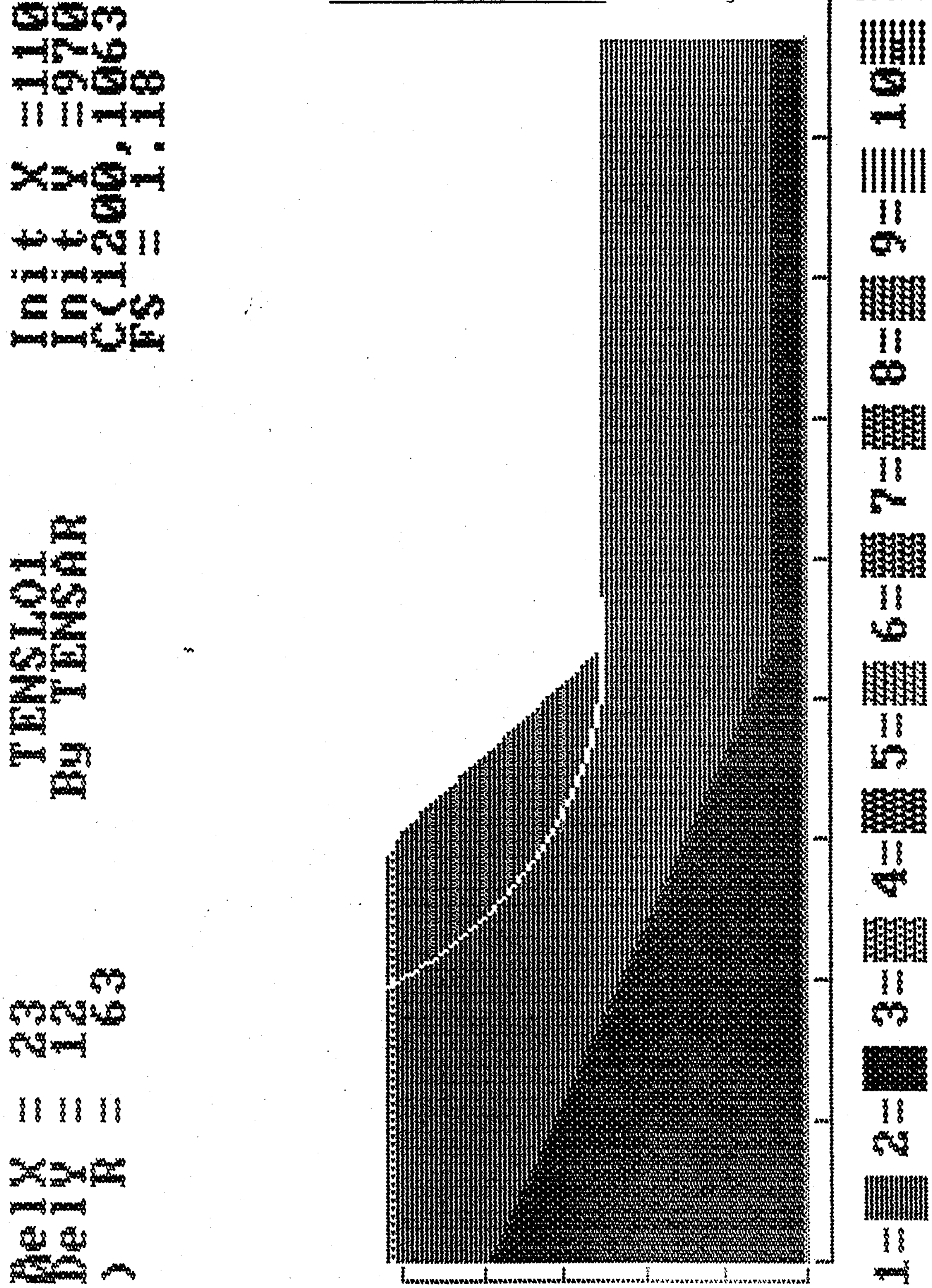
30' 1H:1V SLOPE
SECTION 7 - STATIC

Project No. S93204



MSW 13/13/93 | 20 OF 32

30' 1H:1V SLOPE
SECTION 7 - SEISMIC Project No. S93204

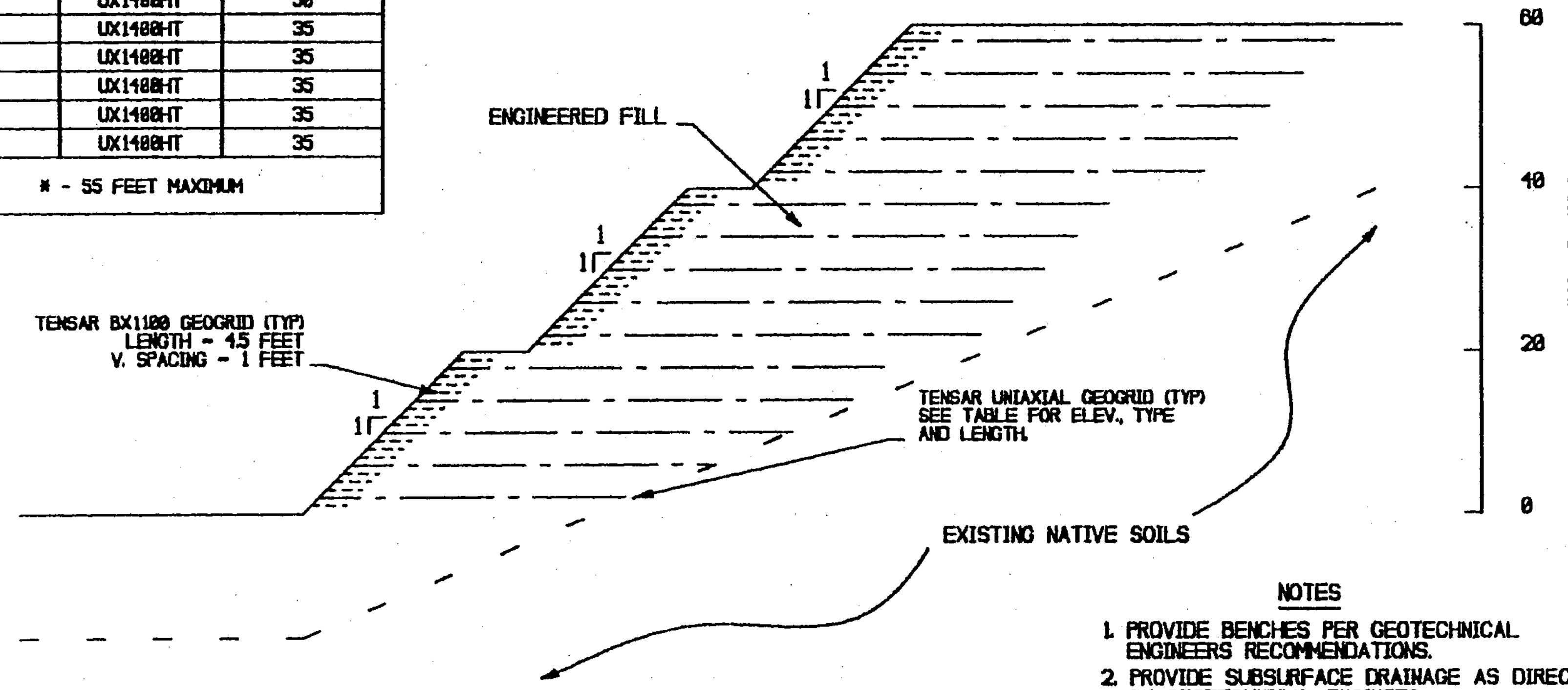


MSW | 3/13/93 | 21 OF 32

PRELIMINARY

ELEV. (FT)	GRID TYPE	LENGTH (FT)
2	UX1400HT	TO BACKCUT*
6	UX1400HT	TO BACKCUT*
10	UX1400HT	TO BACKCUT*
14	UX1400HT	55
18	UX1400HT	55
22	UX1400HT	50
26	UX1400HT	50
30	UX1400HT	50
34	UX1400HT	50
38	UX1400HT	50
42	UX1400HT	35
46	UX1400HT	35
50	UX1400HT	35
54	UX1400HT	35
58	UX1400HT	35

* - 55 FEET MAXIMUM



NOTES

1. PROVIDE BENCHES PER GEOTECHNICAL ENGINEERS RECOMMENDATIONS.
2. PROVIDE SUBSURFACE DRAINAGE AS DIRECTED BY GEOTECHNICAL ENGINEER.

NOT TO SCALE

Tensar

TENSAR EARTH TECHNOLOGIES, INC.
3000 CORPORATE CENTER DR. STE. 370
MORROW, GA 30260
TEL: (404) 968-7600

03/15/93	ISSUED FOR REVIEW

TYPICAL SECTION
SECTION 7; 60' 1H:1V SLOPE
HIGHLAND TERRACE
SAN MATEO COUNTY, CALIFORNIA

PROJECT # S93204 | DRAWING # 3 OF 3

MS w | 3/16/93 | 22 of 32

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*****************************************************************************
                                        *                        p.1 of 2 *
TENSAR EARTH TECHNOLOGIES, INC.         *          TENSWAL                 *
 2121 S. MILL AVENUE, SUITE 212         *  TENSAR GEOGRID REINFORCED       *
     TEMPE, ARIZONA  85282              *  RETAINING WALL ANALYSIS         *
                                        *                                  *
                                        *         Version 3.1              *
  DESIGNER: MSW                         *    Revision Date 01/31/92        *
*****************************************************************************
ject: HIGHLAND TERRACE                 Project #: S93204       Date: 03/12/93
:    SECT. 7; WALL=24.67'; 2H:1V SLOPE=37.5'
      ** Program Analysis for TENSAR Geogrid Reinforcement Only **
IPUT

       REINFORCED WALL FILL DATA

ight (ft)                           =    24.67
igle of Face (deg)                  =     7.13
nsity of fill (lb/ft3)              =     115
i in degrees                        =      30
 Cohesion c (lb/ft2)                =       0

          RETAINED BACKFILL DATA

nsity of fill (lb/ft3)              =     115
i  in degrees                       =      30
 Cohesion c (lb/ft2)                =       0

            FOUNDATION SOIL DATA

i   in degrees                      =      30
hesion  c (lb/ft2)                  =     500
low. bearing press. (lb/ft2)        =    5000

                    LOADING DATA

ight of backfill slope (ft)         =    37.5
ope angle in degrees                =    26.6
rcharge on top of slope (lb/ft2)=         250

              TENSAR GEOGRID DATA

ogrid designation                   =  1600HT
Coverage of TENSAR geogrids         =     100
nimum Geogrid length (ft)           =    12.5
ll soil interaction coeff.          =     0.7
S. for geogrid pullout              =     1.5
n. soil interaction coeff.          =     0.7

                 MISCELLANEOUS DATA

S. for sliding                      =     1.5
S. for overturning                  =       2
S. for uncertainties                =     1.5
sign Methodolgy                     = Tensar Guidelines
nstruction Damage based on          = Sand, Silt, or Clay

NSWAL V3.1 - (c) 1986-1991 by The Tensar Corporation
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MSW | 3/13/93 | 23 OF 32

****

TENSAR EARTH TECHNOLOGIES, INC.
2121 S. MILL AVENUE, SUITE 212
TEMPE, ARIZONA 85282

DESIGNER: MSW

TENSWAL
TENSAR GEOGRID REINFORCED
RETAINING WALL ANALYSIS

Version 3.1
Revision Date 01/31/92

****

ject: HIGHLAND TERRACE   Project #: S93204   Date: 03/12/93
e: SECT. 7; WALL=24.67'; 2H:1V SLOPE=37.5'

** Program Analysis for TENSAR Geogrid Reinforcement Only **

TPUT

of S against sliding = 1.51

of S against overturning = 6.29

aximum Bearing Pressure (Lb/ft2) = 4346

ngle of Inclination, Delta (deg) = 8.5

ccentricity of Pres. Resultant (ft) = -1.10

otal Number of TENSAR Geogrid Layers = 10

levation F TENSAR eogrid(ft)	Allow. Vi (ft)	Length of TENSAR Geogrid(ft)	Working Strength (lb/ft)	Max Force of TENSAR Geogrid(lb/ft)	F.S.	Control Mechanism
22.00	12.14	43.50	1500HT:2571	1465	1.76	W(24.0,30.0)
18.67	5.40	43.50	1500HT:2571	1465	1.76	W(24.0,30.0)
15.34	3.47	43.50	1500HT:2571	1645	1.56	Tension
12.00	2.56	43.50	1500HT:2571	1564	1.64	Tension
10.67	2.96	43.50	1600HT:3286	1872	1.76	W(24.0,30.0)
8.00	2.49	43.50	1600HT:3286	2056	1.60	Tension
6.00	2.22	43.50	1600HT:3286	1975	1.66	Tension
4.00	2.00	43.50	1600HT:3286	1872	1.76	W(24.0,30.0)
2.67	1.88	43.50	1600HT:3286	1940	1.69	Tension
0.67	1.73	43.50	1600HT:3286	2116	1.55	Tension

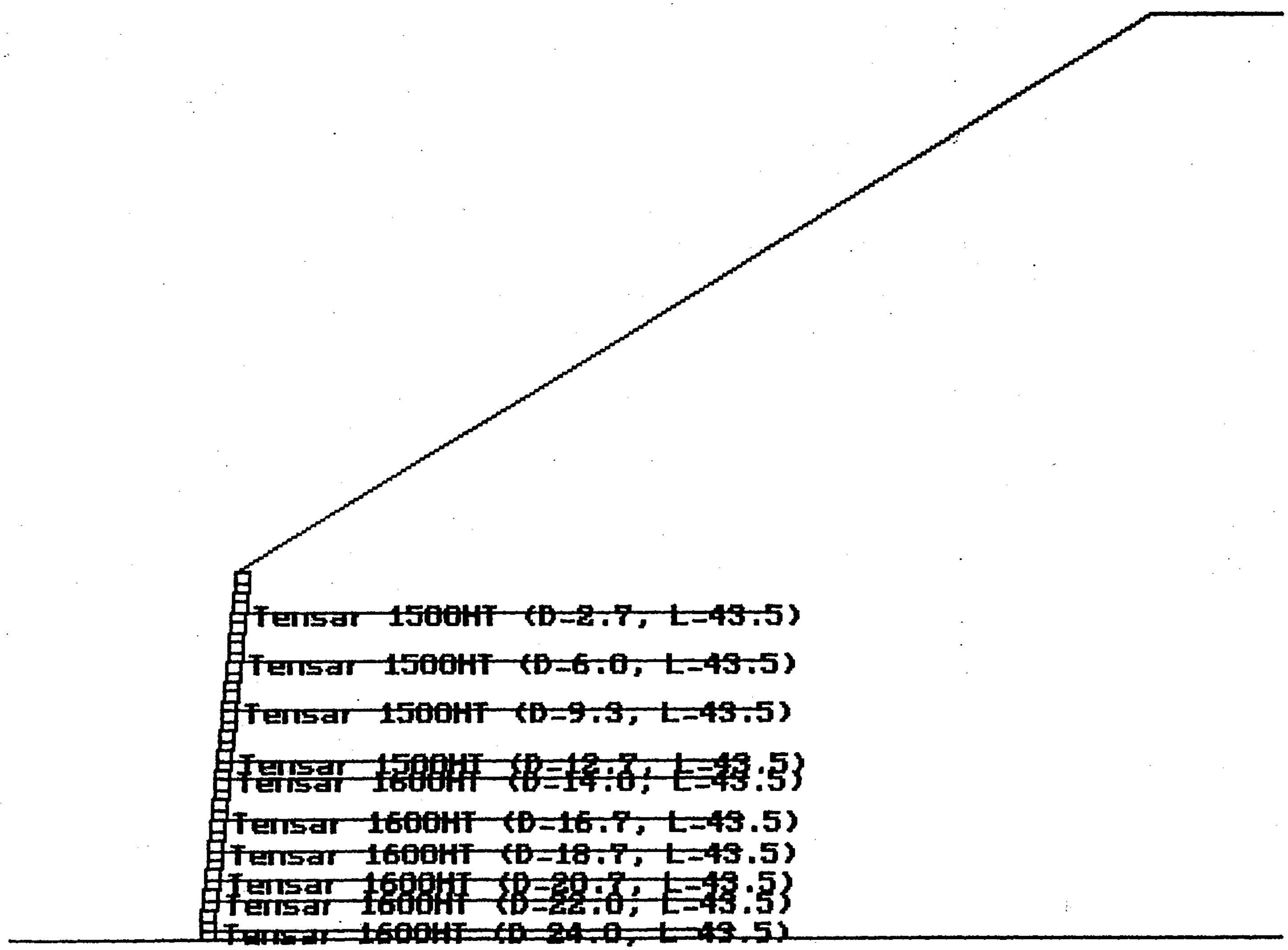
Efficiency of Strength Used vs. Available = 89.85 %

rea of geogrids required (ft2/ft) = 435.00

In accordance with the "Tenswal" licensing Agreement,
ie designer must determine the suitability of program results.

MSW | 3/13/93 | 24 OF 32

TENSAR Retaining Wall | Date: 03/12/93
Project: HIGHLAND TERRACE | Designer: MSW
Number: S93204 | Wall Height: 24.67 feet
Case: SECT. 7; WALL=24.67'; 2H:1V SLOPE=37.5'



MSW | 31.3.93 | 25 OF 32

:NSAR EARTH TECHNOLOGIES, INC.
:121 S. MILL AVENUE, SUITE 212
TEMPE, ARIZONA 85282

TENSWAL
TENSAR GEOGRID REINFORCED
RETAINING WALL ANALYSIS

Version 3.1
Revision Date 01/31/92

DESIGNER: MSW

:ct: HIGHLAND TERRACE   Project #: S93204   Date: 03/12/93
: SECTION 6; HEIGHT = 24 FEET

** Program Analysis for TENSAR Geogrid Reinforcement Only **

'UT

REINFORCED WALL FILL DATA

ight (ft)	=	24
jle of Face (deg)	=	7.3
isity of fill (lb/ft3)	=	115
l in degrees	=	30
:ohesion c (lb/ft2)	=	0

RETAINED BACKFILL DATA

isity of fill (lb/ft3)	=	115
l in degrees	=	30
:ohesion c (lb/ft2)	=	0

FOUNDATION SOIL DATA

l in degrees	=	30
iesion c (lb/ft2)	=	500
low. bearing press. (lb/ft2)	=	5000

LOADING DATA

rcharge (lb/ft2)	=	250

TENSAR GEOGRID DATA

ogrid designation	=	1600HT
:overage of TENSAR geogrids	=	100
iimum Geogrid length (ft)	=	12
ll soil interaction coeff.	=	0.7
S. for geogrid pullout	=	1.5
n. soil interaction coeff.	=	0.7

MISCELLANEOUS DATA

S. for sliding	=	1.5
S. for overturning	=	2
S. for uncertainties	=	1.5
sign Methodolgy	=	Tensar Guidelines
nstruction Damage based on	=	Sand, Silt, or Clay

MSW | 3/13/93 | 26 OF 32

:NSAR EARTH TECHNOLOGIES, INC.
.121 S. MILL AVENUE, SUITE 212
TEMPE, ARIZONA 85282

DESIGNER: MSW

TENSWAL
TENSAR GEOGRID REINFORCED
RETAINING WALL ANALYSIS

Version 3.1
Revision Date 01/31/92

ıct: HIGHLAND   Project #: S93204   Date: 03/12/93
SECTION 6; HEIGHT = 24 FEET
** Program Analysis for TENSAR Geogrid Reinforcement Only **

ʼUT

ıf S against sliding = 1.54

ıf S against overturning = 3.54

:imum Bearing Pressure (Lb/ft2) = 3365

ʒle of Inclination, Delta (deg) = 0.0

:entricity of Pres. Resultant (ft) = 0.71

:al Number of TENSAR Geogrid Layers = 8

ːvation TENSAR ɔgrid(ft)	Allow. Vi (ft)	Length of TENSAR Geogrid(ft)	Working Strength (lb/ft)	Max Force of TENSAR Geogrid(lb/ft)	F.S.	Control Mechanism
22.68	14.89	13.50	1500HT:2571	1102	2.33	W(14.7,30.0)
19.34	7.61	13.50	1500HT:2571	1381	1.86	W(20.7,30.0)
16.01	5.12	13.50	1500HT:2571	1381	1.86	W(20.7,30.0)
12.67	3.85	13.50	1500HT:2571	1485	1.73	Tension
9.34	3.95	13.50	1600HT:3286	1850	1.78	Tension
6.00	3.30	13.50	1600HT:3286	1995	1.65	Tension
3.34	2.91	13.50	1600HT:3286	1759	1.87	W(22.7,30.0)
1.33	2.68	13.50	1600HT:3286	1910	1.72	Tension

Efficiency of Strength Used vs. Available = 82.35 %

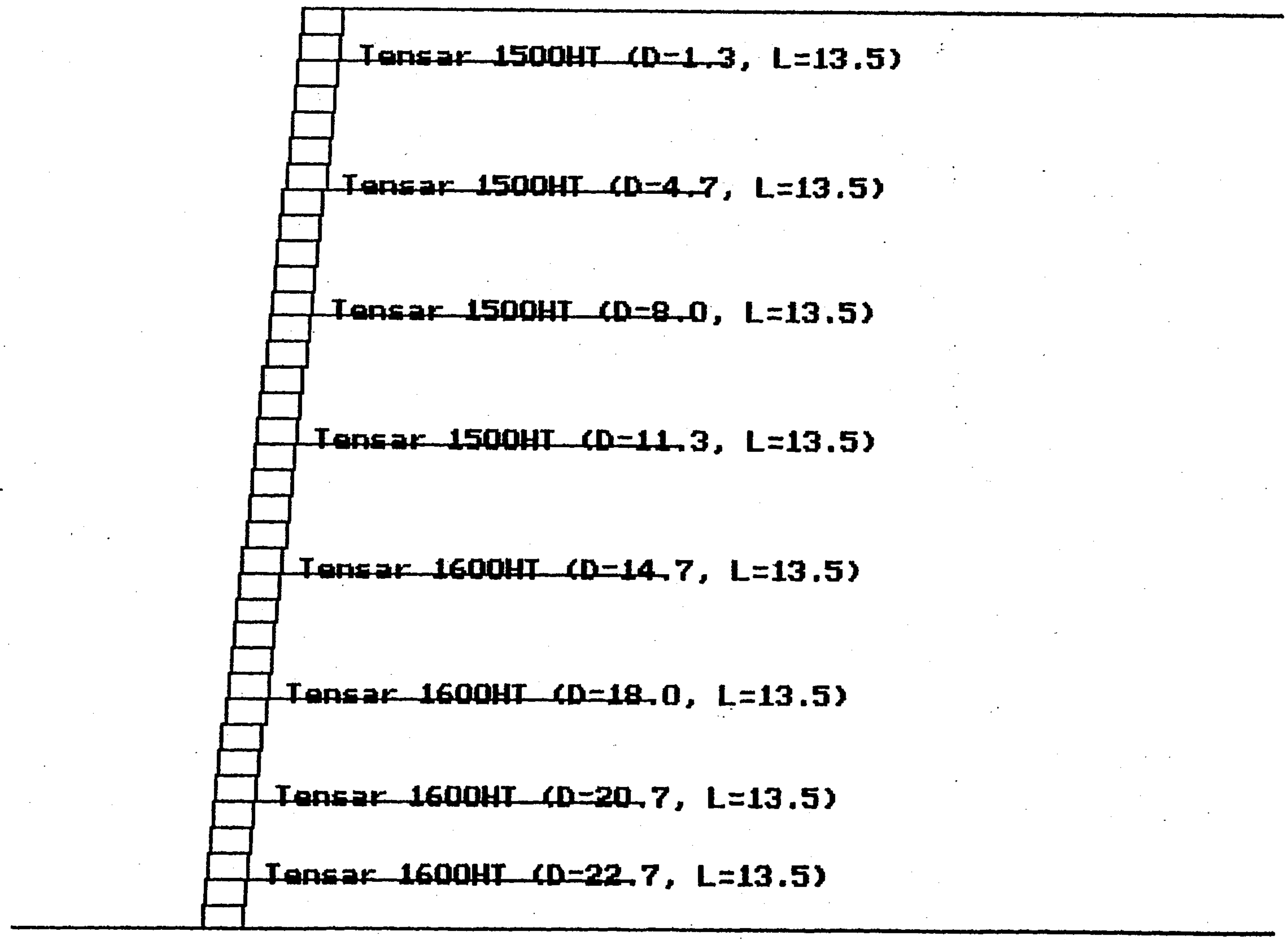
ːa of geogrids required (ft2/ft) = 108.00

In accordance with the "Tenswal" licensing Agreement,
e designer must determine the suitability of program results.

MSW | 3/13/93 | 27 of 32

TENSAR Retaining Wall
Project: HIGHLAND
Number: S93204
Case: SECTION 6; HEIGHT = 24 FEET

Date: 03/12/93
Designer: MSW
Wall Height: 24.00 feet



MSW | 3/13/93 | 28 OF 32

ENSAR EARTH TECHNOLOGIES, INC.
2121 S. MILL AVENUE, SUITE 212
TEMPE, ARIZONA 85282

TENSWAL
TENSAR GEOGRID REINFORCED
RETAINING WALL ANALYSIS

Version 3.1
Revision Date 01/31/92

DESIGNER: MSW

ect: HIGHLAND TERRACE   Project #: S93204   Date: 03/10/93
: SECTION 1; HEIGHT = 27.33 FEET

** Program Analysis for TENSAR Geogrid Reinforcement Only **

PUT

REINFORCED WALL FILL DATA

ight (ft)	=	27.33
gle of Face (deg)	=	7.3
nsity of fill (lb/ft3)	=	115
i in degrees	=	30
Cohesion c (lb/ft2)	=	0

RETAINED BACKFILL DATA

nsity of fill (lb/ft3)	=	115
i in degrees	=	30
Cohesion c (lb/ft2)	=	0

FOUNDATION SOIL DATA

i in degrees	=	30
hesion c (lb/ft2)	=	500
low. bearing press. (lb/ft2)	=	5000

LOADING DATA

rcharge (lb/ft2)	=	250

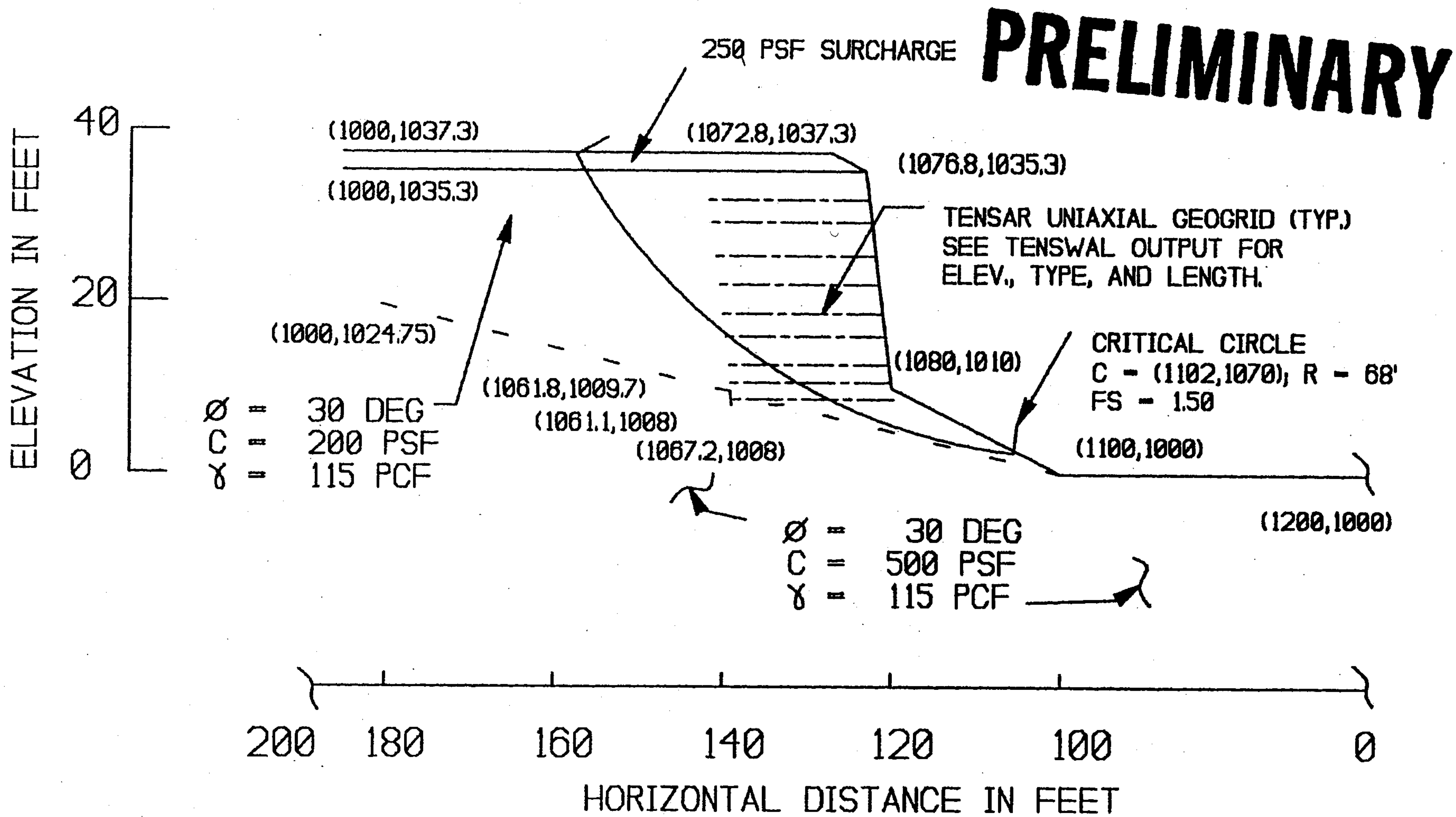
TENSAR GEOGRID DATA

ogrid designation	=	1600HT
Coverage of TENSAR geogrids	=	100
nimum Geogrid length (ft)	=	14
ll soil interaction coeff.	=	0.7
S. for geogrid pullout	=	1.5
n. soil interaction coeff.	=	0.7

MISCELLANEOUS DATA

S. for sliding	=	1.5
S. for overturning	=	2
S. for uncertainties	=	1.5
sign Methodolgy	=	Tensar Guidelines
nstruction Damage based on	=	Sand, Silt, or Clay

MSW | 3/13/93 | 29 of 32



	Tensar	TENSAR EARTH TECHNOLOGIES, INC. 3000 CORPORATE CENTER DR. STE. 370 MORROW, GA 30260 TEL: (404) 968-7600	03/15/93 ISSUED FOR REVIEW	COMPUTER MODEL SECTION SECTION 1 HIGHLAND TERRACE CALIFORNIA PROJECT # S93204 | DRAWING # 1 OF 1

MSW | 3/13/93 | 31 OF 32

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ENSAR EARTH TECHNOLOGIES, INC.	*	p.2 of 2 *
2121 S. MILL AVENUE, SUITE 212	*	TENSWAL *
TEMPE, ARIZONA 85282	*	TENSAR GEOGRID REINFORCED *
	*	RETAINING WALL ANALYSIS *
	*	*
	*	Version 3.1 *
DESIGNER: MSW	*	Revision Date 01/31/92 *

****

ect: HIGHLAND TERRACE   Project #: S93204   Date: 03/10/93

: SECTION 1; HEIGHT = 27.33 FEET

** Program Analysis for TENSAR Geogrid Reinforcement Only **

PUT

of S against sliding = 1.87

of S against overturning = 5.28

ximum Bearing Pressure (Lb/ft2) = 3494

gle of Inclination, Delta (deg) = 0.0

centricity of Pres. Resultant (ft) = 0.27

tal Number of TENSAR Geogrid Layers = 9

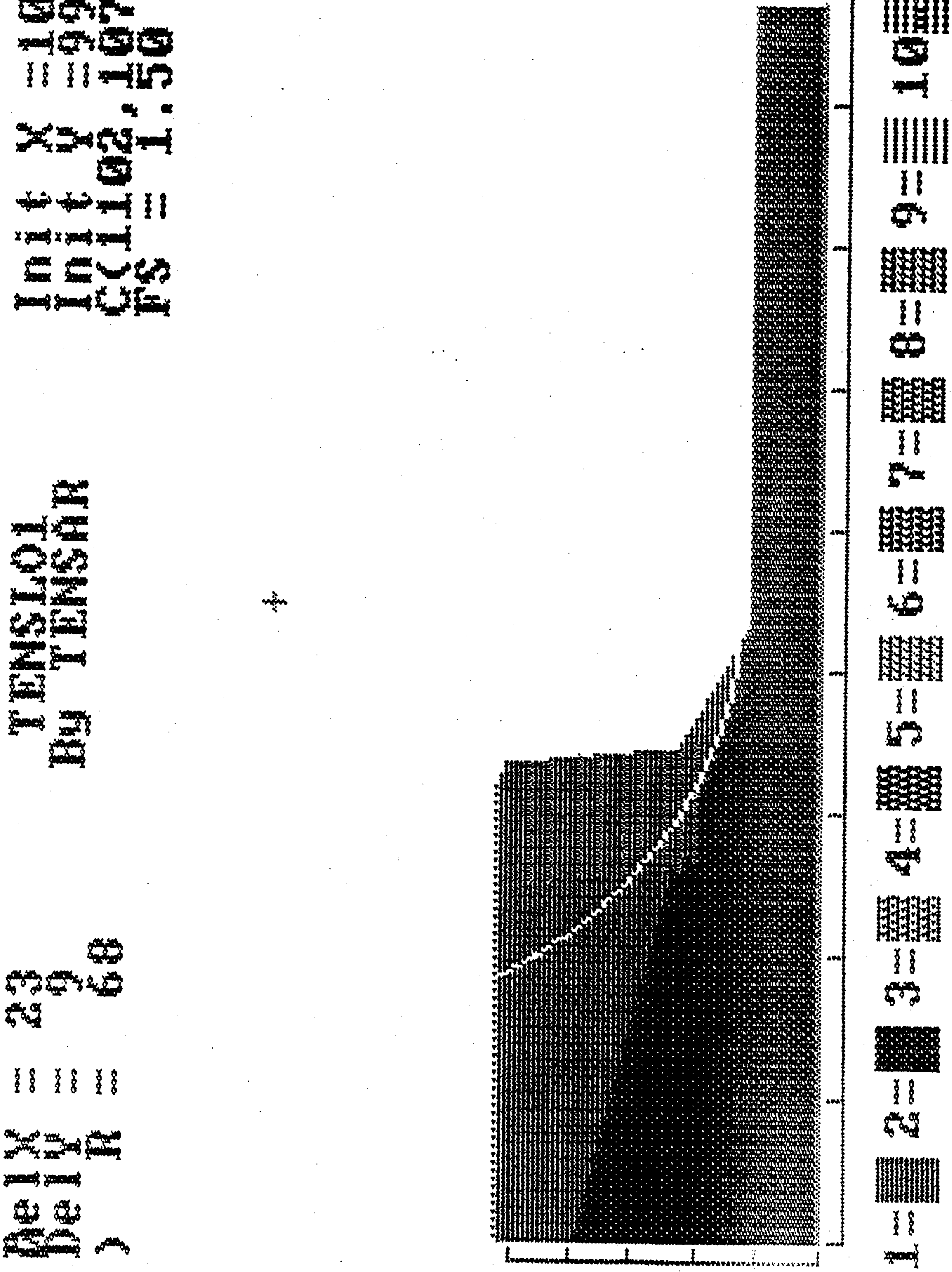
evation TENSAR ograid(ft)	Allow. Vi (ft)	Length of TENSAR Geogrid(ft)	Working Strength (lb/ft)	Max Force of TENSAR Geogrid(lb/ft)	F.S.	Control Mechanism
24.00	9.45	19.00	1500HT:2571	1537	1.67	W(22.7,30.0)
21.34	6.37	19.00	1500HT:2571	1537	1.67	W(22.7,30.0)
17.34	4.28	19.00	1500HT:2571	1537	1.67	W(22.7,30.0)
14.00	3.35	19.00	1500HT:2571	1704	1.51	Tension
10.67	3.53	19.00	1600HT:3286	1965	1.67	W(22.7,30.0)
8.00	3.09	19.00	1600HT:3286	2126	1.55	Tension
4.67	2.68	19.00	1600HT:3286	2182	1.51	Tension
2.67	2.48	19.00	1600HT:3286	1910	1.72	W(26.7,30.0)
0.67	2.31	19.00	1600HT:3286	1755	1.87	W(27.3,30.0)

Efficiency of Strength Used vs. Available = 91.26 %

ea of geogrids required (ft2/ft) = 171.00

in accordance with the "Tenswal" licensing Agreement,
e designer must determine the suitability of program results.

MSW | 3/13/93 | 30 of 32



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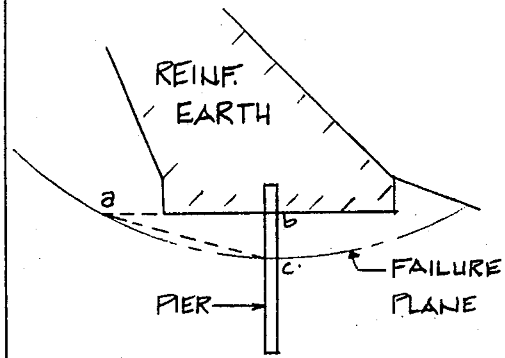
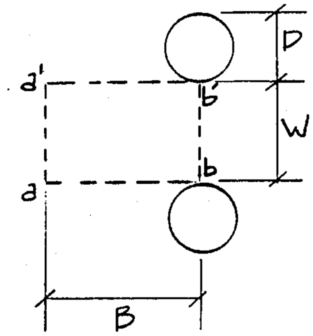
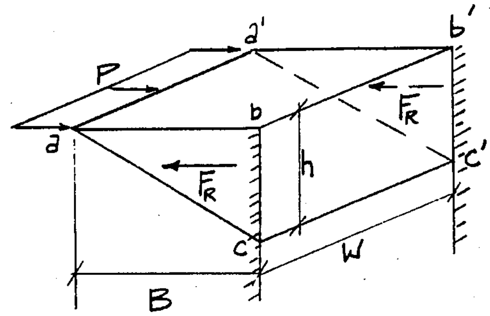
PIER ANALYSIS FOR SLOPE STABILIZATION

A] LIMITING EDGE-TO-EDGE SPACING – "GS-7b"







• NO RELATIVE DISPLACEMENT AT □ aa'bb'

$F_R = \left[C + K_o \gamma Z \tan\phi\right]\frac{Bh}{2}$ $\qquad \Sigma F_R = \left[C + K_o \gamma Z \tan\phi\right] Bh = PW$

Z = DEPTH BELOW SLOPE (OR R.E.) SURFACE

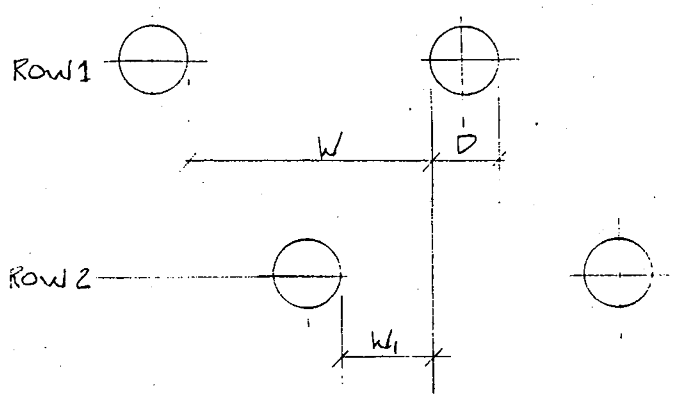
FOR GS-7b: $P = 210$ kip/ft, $Z = 40$ ft, $B \doteq 50$ ft, $h = 5$ ft
$K_o \doteq 0.7$, $\phi = 25°$, $C = 1.0$ ksf, $\gamma \doteq 0.13$ kcf

$\Sigma F_R = \left[1.0 + 0.7 \times 0.13 \times 40 \times \tan 25°\right] \times 5 \times 50 = 675$ kips

ALLOW 1/3 INCREASE FOR SEISMIC LOADING,

$\Sigma F_R \doteq 900$ kips

FOR STAGGERED PIER LAYOUT:



$P \times W_1 = \Sigma F_R$

$W_1 = 900/210 = 4.3'$

$W = 2 \times W_1 + D$

PIER DIA.

D, FT.	0.5	1.0	1.5	2.0
W, FT.	18.2D	9.6D	6.8D	5.3D

USE $\underline{\underline{W = 5D \text{ MAX}}}$

PIER ANALYSIS FOR SLOPE STABILIZATION

B] COEFFICIENTS FOR USE IN ELASTIC SOLUTIONS OF LATERAL PIERS

1) COEFFICIENT OF HORIZONTAL REACTION IN BOWLE'S SOLUTION:
(FOUNDATION ANALYSIS & DESIGN, BOWLES, McGraw Hill, 4TH EDITION)

P. 408 : $k_s = A_s + B_s z$ —— Eq. 9-10

$A_s = K[cN_c + \frac{1}{2}\gamma d N_\gamma]$, $B_s = K[\gamma N_q]$

(Note: $K \equiv C$ in Bowles).

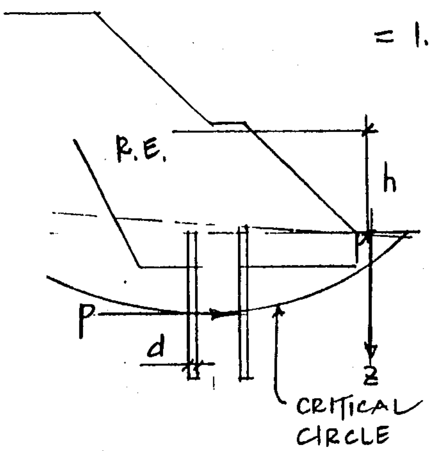
P. 774 : $k_s = S_1 A_s + S_2 B_s z$; S_1 & S_2 : SHAPE FACTORS

$S_1 = 1.3 \sim 1.7$ FOR ROUND PILE — Use $\begin{cases} S_1 = 1.5 \\ S_2 = 3.2 \end{cases}$

$S_2 = 2.0 \sim 4.4$ FOR " " —

$k_s = 1.5 A_s + 3.2 B_s z$

$= 1.5 K[cN_c + \frac{1}{2}\gamma d N_\gamma] + 3.2 K[\gamma z + q_s]$



P. 772: $K = 24$

$k_s = [36 c N_c + 18 \gamma d N_\gamma] + 76.8[\gamma z + q_s]$

$c = 1.0$ KSF, $\phi = 25°$, $q_s = \gamma h$, $\gamma = 0.13$ KSF

N_c, N_γ, N_q : Bearing Capacity Factors

$N_c = 22$, $N_\gamma = 7$, $N_q = 11.5$

$k_s = 792 + 16d + 115z + 883 q_s$

Say, $\boxed{k_s = 790 + 115z + 880 q_s}$ — [Eq. 6]

2) P-Y CURVES FOR REESE SOLUTION:

BOWLE'S P. 773 : $k_s = \frac{1.6 E_s}{d}$; $E_s = \frac{P}{y} = \underline{\underline{\frac{k_s d}{1.6}}}$

k_s : value from Eq. 6.

Reese & Welch (1975) : $P = P_{ult}$ @ $y = 16 y_{50} = 16 \times [2.5 d\, \varepsilon_{50}]$

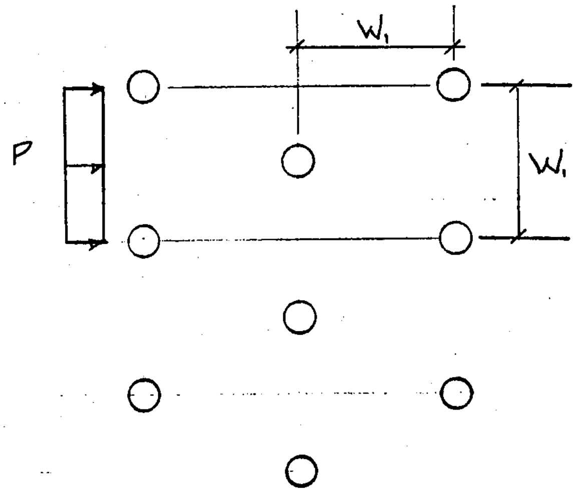
$\varepsilon_{50} = 0.002 \sim 0.006$ (PER Report Data)

Use $\varepsilon_{50} = 0.005$

$y = 16 \times 2.5 d \times 0.005 = \underline{0.2d}$

PIER ANALYSIS FOR SLOPE STABILIZATION

c] FORCES ON PIERS



n ROWS OF PIERS WITH EDGE-TO-EDGE SPACING OF 5D, ALTERNATE ROWS STAGGERED

$W_1 = 6D$ D = DIAMETER

P = 210 KIP/FT

FORCE ON EACH PIER, $P_p = PW_1/n$

D(IN)	n	P_p (KIP/PIER)
6	5	126
	10	63
	15	42
8	5	168
	10	84
	15	56
12	5	252
	6	210
	7	180
	8	158
	10	126
18	4	473
	6	315
	8	236
	10	189

PIER ANALYSIS FOR SLOPE STABILIZATION

D] DETERMINATION OF MOMENT & SHEAR

MOMENT & SHEAR ON PIERS FROM F.E.M. PROGRAM BY BOWLES — SEE ATTACHED COMPUTER PRINTOUTS

E] DETERMINATION OF REQUIRED STEEL

- $A_s^V = \frac{V - V_c}{f_s}$ = STEEL REQ'D. FOR STABILIZING FORCE (P)
- $A_s^M = M/ad$ = STEEL REQ'D. FOR MOMENT (COMPUTER RUN)

  $a = f_s j / 12000$, j FROM WORKING STRESS TABLES
- $f_y = 60000$ psi ; $f'_c = 3000$ psi

  $f_s = 0.4 f_y = 24000$ psi , $f_c = 0.45 f'_c = 1350$ psi
- WORKING STRESS INCREASED 1/3 FOR SEISMIC LOADING

  $\therefore$ $f_s = 32000$ psi & $f_c = 1800$ psi $\Rightarrow$ $j = 0.886$
- $V_c = (A_g - A_s) \times 2\sqrt{f'_c}$

DIMENSIONS	# OF ROWS	M (KIP·FT)	V (KIP)	V_c (KIP)	$V_s = V - V_c$ (KIP)	A_s^M (IN²)	A_s^V (IN²)	A_s^V / A_s^M
6" Φ PIER, b ≈ 6", d ≈ 3"	5	24	126	3	123	3.39	3.84	1.13
	10	12	63	3	60	1.69	1.88	1.11
	15	8	42	3	39	1.13	1.22	1.08
8" Φ PIER, b ≈ 8", d ≈ 5"	5	49	168	5	163	4.15	5.09	1.23
	10	25	84	5	79	2.12	2.45	1.16
	15	16	56	5	51	1.35	1.59	1.18
12" Φ PIER, b ≈ 12", d ≈ 8"	5	117	252	12	240	6.19	7.50	1.21
	6	97	210	12	198	5.13	6.19	1.21
	7	83	180	12	168	4.39	5.25	1.20
	8	73	158	12	146	3.86	4.56	1.18
	10	58	126	12	114	3.07	3.56	1.16
18" Φ PIER, b ≈ 18", d ≈ 12"	4	309	473	27	446	10.90	13.94	1.28
	6	206	315	27	288	7.27	9.00	1.24
	8	154	236	27	209	5.43	6.53	1.20
	10	123	189	27	162	4.34	5.06	1.17

Highland 6"   06-06-1994
Solution for laterally loaded pile

ANALYSIS OF LATERALLY LOADED PIERS

No. of NP= 32
No. of elements= 15
No. of non-zero P entries= 1
No. of load cases= 3
Corrected node springs= 0
Node soil begins= 1
No. of boundary conditions= 0

Mod. of elasticity= 586000 kips/ft^2
Unit weight = 0
Max. soil displacement= .1 ft
Modulus of Subgrade Reaction= 5000 + 0 *Z kips/ft^3
Ground line reduction factor= 1

seg.	Length (ft)	Width (ft)	Moment of Inertia (ft^4)
1	2	.5	3.067871E-03
2	2	.5	3.067871E-03
3	2	.5	3.067871E-03
4	2	.5	3.067871E-03
5	2	.5	3.067871E-03
6	2	.5	3.067871E-03
7	2	.5	3.067871E-03
8	2	.5	3.067871E-03
9	2	.5	3.067871E-03
10	2	.5	3.067871E-03
11	3	.5	3.067871E-03
12	4	.5	3.067871E-03
13	5	.5	3.067871E-03
14	6	.5	3.067871E-03
15	7	.5	3.067871E-03

Node #	Soil modulus kip/ft^3	Node spring kip/ft
1	5000	2500
2	5000	5000
3	5000	5000
4	5000	5000
5	5000	5000
6	5000	5000
7	5000	5000
8	5000	5000
9	5000	5000
10	5000	5000
11	5000	6250
12	5000	8750
13	5000	11250
14	5000	13750
15	5000	16250
16	5000	8750

** ** ** ** ** ** ** ** ** ** **

5 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	-0.0000	-0.0668
2	0.0668	-0.5233
3	0.5233	1.3057
4	-1.3057	11.2628
5	-11.2628	-24.0021
6	24.0021	11.2629
7	-11.2629	1.3059
8	-1.3059	-0.5246
9	0.5246	-0.0711
10	0.0711	0.0255
11	-0.0255	-0.0027
12	0.0027	0.0004
13	-0.0004	-0.0001
14	0.0001	0.0000
15	-0.0000	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	0.0334	0.00003	0.00001	0.0668	0.0000	0.0000
2	0.1948	-0.00001	0.00004	0.1948	0.0000	0.0000
3	-1.1427	-0.00034	-0.00023	-1.1427	0.0000	0.0000
4	-4.0641	0.00010	-0.00081	-4.0641	0.0000	0.0000
5	22.6110	0.00709	0.00452	22.6110	0.0000	0.0000
6	90.7350	-0.00000	0.01815	90.7350	0.0000	126.0000
7	22.6110	-0.00709	0.00452	22.6110	0.0000	0.0000
8	-4.0633	-0.00009	-0.00081	-4.0633	0.0000	0.0000
9	-1.1419	0.00034	-0.00023	-1.1419	0.0000	0.0000
10	0.1784	0.00001	0.00004	0.1784	0.0000	0.0000
11	0.0577	-0.00002	0.00001	0.0462	0.0000	0.0000
12	-0.0102	0.00000	-0.00000	-0.0058	0.0000	0.0000
13	0.0009	-0.00000	0.00000	0.0004	0.0000	0.0000
14	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000
15	0.0000	-0.00000	0.00000	0.0000	0.0000	0.0000
16	-0.0000	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 126.0000   VERSES THE SUM OF THE APPLIED FORCES= 126

** ** ** ** ** ** ** ** ** ** **

10 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	-0.0000	-0.0334
2	0.0334	-0.2616
3	0.2616	0.6528
4	-0.6528	5.6314
5	-5.6314	-12.0011
6	12.0011	5.6314
7	-5.6314	0.6529
8	-0.6529	-0.2623
9	0.2623	-0.0356
10	0.0356	0.0128
11	-0.0128	-0.0013
12	0.0013	0.0002
13	-0.0002	-0.0000
14	0.0000	0.0000
15	-0.0000	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	0.0167	0.00001	0.00001	0.0334	0.0000	0.0000
2	0.0974	-0.00001	0.00002	0.0974	0.0000	0.0000
3	-0.5714	-0.00017	-0.00011	-0.5714	0.0000	0.0000
4	-2.0320	0.00005	-0.00041	-2.0320	0.0000	0.0000
5	11.3055	0.00354	0.00226	11.3055	0.0000	0.0000
6	45.3675	-0.00000	0.00907	45.3675	0.0000	63.0000
7	11.3055	-0.00354	0.00226	11.3055	0.0000	0.0000
8	-2.0316	-0.00005	-0.00041	-2.0316	0.0000	0.0000
9	-0.5710	0.00017	-0.00011	-0.5710	0.0000	0.0000
10	0.0892	0.00000	0.00002	0.0892	0.0000	0.0000
11	0.0289	-0.00001	0.00000	0.0231	0.0000	0.0000
12	-0.0051	0.00000	-0.00000	-0.0029	0.0000	0.0000
13	0.0004	-0.00000	0.00000	0.0002	0.0000	0.0000
14	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000
15	0.0000	-0.00000	0.00000	0.0000	0.0000	0.0000
16	-0.0000	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 63.0000 VERSES THE SUM OF THE APPLIED FORCES= 63

** ** ** ** ** ** ** ** ** ** **

** ** ** ** ** ** ** ** ** ** **

15 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	-0.0223
2	0.0223	-0.1744
3	0.1744	0.4352
4	-0.4352	3.7543
5	-3.7543	-8.0007
6	8.0007	3.7543
7	-3.7543	0.4353
8	-0.4353	-0.1749
9	0.1749	-0.0237
10	0.0237	0.0085
11	-0.0085	-0.0009
12	0.0009	0.0001
13	-0.0001	-0.0000
14	0.0000	0.0000
15	-0.0000	-0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	0.0111	0.00001	0.00000	0.0223	0.0000	0.0000
2	0.0649	-0.00000	0.00001	0.0649	0.0000	0.0000
3	-0.3809	-0.00011	-0.00008	-0.3809	0.0000	0.0000
4	-1.3547	0.00003	-0.00027	-1.3547	0.0000	0.0000
5	7.5370	0.00236	0.00151	7.5370	0.0000	0.0000
6	30.2450	-0.00000	0.00605	30.2450	0.0000	42.0000
7	7.5370	-0.00236	0.00151	7.5370	0.0000	0.0000
8	-1.3544	-0.00003	-0.00027	-1.3544	0.0000	0.0000
9	-0.3806	0.00011	-0.00008	-0.3806	0.0000	0.0000
10	0.0595	0.00000	0.00001	0.0595	0.0000	0.0000
11	0.0192	-0.00001	0.00000	0.0154	0.0000	0.0000
12	-0.0034	0.00000	-0.00000	-0.0019	0.0000	0.0000
13	0.0003	-0.00000	0.00000	0.0001	0.0000	0.0000
14	-0.0000	0.00000	-0.00000	-0.0000	0.0000	0.0000
15	0.0000	-0.00000	0.00000	0.0000	0.0000	0.0000
16	-0.0000	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 42.0000 VERSES THE SUM OF THE APPLIED FORCES= 42

** ** ** ** ** ** ** ** ** ** **

Highland 8"   06-06-1994
Solution for laterally loaded pile

No. of NP= 32
No. of elements= 15
No. of non-zero P entries= 1
No. of load cases= 3
Corrected node springs= 0
Node soil begins= 1
No. of boundary conditions= 0

Mod. of elasticity= 586000 kips/ft^2
Unit weight = 0
Max. soil displacement= .1 ft
Modulus of Subgrade Reaction= 5000 + 0 *Z kips/ft^3
Ground line reduction factor= 1

seg.	Length (ft)	Width (ft)	Moment of Inertia (ft^4)
1	2	.6667	9.697927E-03
2	2	.6667	9.697927E-03
3	2	.6667	9.697927E-03
4	2	.6667	9.697927E-03
5	2	.6667	9.697927E-03
6	2	.6667	9.697927E-03
7	2	.6667	9.697927E-03
8	2	.6667	9.697927E-03
9	2	.6667	9.697927E-03
10	2	.6667	9.697927E-03
11	3	.6667	9.697927E-03
12	4	.6667	9.697927E-03
13	5	.6667	9.697927E-03
14	6	.6667	9.697927E-03
15	7	.6667	9.697927E-03

Node #	Soil modulus kip/ft^3	Node spring kip/ft
1	5000	3333.5
2	5000	6667
3	5000	6667
4	5000	6667
5	5000	6667
6	5000	6667
7	5000	6667
8	5000	6667
9	5000	6667
10	5000	6667
11	5000	8333.75
12	5000	11667.25
13	5000	15000.75
14	5000	18334.25
15	5000	21667.75
16	5000	11667.25

** ** ** ** ** ** ** ** ** ** **

5 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	-0.5812
2	0.5812	-0.0602
3	0.0602	7.5583
4	-7.5583	17.7332
5	-17.7332	-49.0118
6	49.0118	17.7349
7	-17.7349	7.5637
8	-7.5637	-0.0662
9	0.0662	-0.6571
10	0.6571	-0.1004
11	0.1004	0.0383
12	-0.0383	-0.0057
13	0.0057	0.0011
14	-0.0011	-0.0002
15	0.0002	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	0.2906	-0.00005	0.00009	0.4359	0.0000	0.0000
2	-0.5511	-0.00015	-0.00008	-0.4133	0.0000	0.0000
3	-3.5488	-0.00027	-0.00053	-2.6615	0.0000	0.0000
4	-1.2782	0.00105	-0.00019	-0.9586	0.0000	0.0000
5	38.4600	0.00550	0.00577	28.8436	0.0000	0.0000
6	101.2541	-0.00000	0.01519	75.9368	0.0000	168.0000
7	38.4590	-0.00550	0.00577	28.8428	0.0000	0.0000
8	-1.2707	-0.00105	-0.00019	-0.9530	0.0000	0.0000
9	-3.5195	0.00027	-0.00053	-2.6395	0.0000	0.0000
10	-0.5738	0.00014	-0.00009	-0.4304	0.0000	0.0000
11	0.2322	0.00001	0.00003	0.1393	0.0000	0.0000
12	0.0572	-0.00001	0.00000	0.0245	0.0000	0.0000
13	-0.0124	0.00000	-0.00000	-0.0041	0.0000	0.0000
14	0.0016	-0.00000	0.00000	0.0004	0.0000	0.0000
15	-0.0002	0.00000	-0.00000	-0.0001	0.0000	0.0000
16	0.0000	-0.00000	0.00000	0.0000	0.0000	0.0000

SUM SPRNG= 168.0000   VERSES THE SUM OF THE APPLIED FORCES= 168

** ** ** ** ** ** ** ** ** ** **

10 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	-0.2906
2	0.2906	-0.0301
3	0.0301	3.7792
4	-3.7792	8.8666
5	-8.8666	-24.5059
6	24.5059	8.8675
7	-8.8675	3.7818
8	-3.7818	-0.0331
9	0.0331	-0.3286
10	0.3286	-0.0502
11	0.0502	0.0191
12	-0.0191	-0.0029
13	0.0029	0.0005
14	-0.0005	-0.0001
15	0.0001	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	0.1453	-0.00003	0.00004	0.2179	0.0000	0.0000
2	-0.2755	-0.00008	-0.00004	-0.2066	0.0000	0.0000
3	-1.7744	-0.00013	-0.00027	-1.3307	0.0000	0.0000
4	-0.6391	0.00053	-0.00010	-0.4793	0.0000	0.0000
5	19.2300	0.00275	0.00288	14.4218	0.0000	0.0000
6	50.6270	-0.00000	0.00759	37.9684	0.0000	84.0000
7	19.2295	-0.00275	0.00288	14.4214	0.0000	0.0000
8	-0.6353	-0.00053	-0.00010	-0.4765	0.0000	0.0000
9	-1.7597	0.00013	-0.00026	-1.3197	0.0000	0.0000
10	-0.2869	0.00007	-0.00004	-0.2152	0.0000	0.0000
11	0.1161	0.00000	0.00001	0.0696	0.0000	0.0000
12	0.0286	-0.00000	0.00000	0.0123	0.0000	0.0000
13	-0.0062	0.00000	-0.00000	-0.0021	0.0000	0.0000
14	0.0008	-0.00000	0.00000	0.0002	0.0000	0.0000
15	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000
16	0.0000	-0.00000	0.00000	0.0000	0.0000	0.0000

SUM SPRNG= 84.0000 VERSES THE SUM OF THE APPLIED FORCES= 84

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15 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	-0.0000	-0.1937
2	0.1937	-0.0201
3	0.0201	2.5194
4	-2.5194	5.9111
5	-5.9111	-16.3373
6	16.3373	5.9116
7	-5.9116	2.5212
8	-2.5212	-0.0221
9	0.0221	-0.2190
10	0.2190	-0.0335
11	0.0335	0.0128
12	-0.0128	-0.0019
13	0.0019	0.0004
14	-0.0004	-0.0001
15	0.0001	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	0.0969	-0.00002	0.00003	0.1453	0.0000	0.0000
2	-0.1837	-0.00005	-0.00003	-0.1378	0.0000	0.0000
3	-1.1829	-0.00009	-0.00018	-0.8872	0.0000	0.0000
4	-0.4261	0.00035	-0.00006	-0.3195	0.0000	0.0000
5	12.8200	0.00183	0.00192	9.6145	0.0000	0.0000
6	33.7514	-0.00000	0.00506	25.3123	0.0000	56.0000
7	12.8197	-0.00183	0.00192	9.6143	0.0000	0.0000
8	-0.4236	-0.00035	-0.00006	-0.3177	0.0000	0.0000
9	-1.1732	0.00009	-0.00018	-0.8798	0.0000	0.0000
10	-0.1913	0.00005	-0.00003	-0.1435	0.0000	0.0000
11	0.0774	0.00000	0.00001	0.0464	0.0000	0.0000
12	0.0191	-0.00000	0.00000	0.0082	0.0000	0.0000
13	-0.0041	0.00000	-0.00000	-0.0014	0.0000	0.0000
14	0.0005	-0.00000	0.00000	0.0001	0.0000	0.0000
15	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000
16	0.0000	-0.00000	0.00000	0.0000	0.0000	0.0000

SUM SPRNG= 56.0000 VERSES THE SUM OF THE APPLIED FORCES= 56

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Highland 12" 06-06-1994
Solution for laterally loaded pile

No. of NP= 32
No. of elements= 15
No. of non-zero P entries= 1
No. of load cases= 5
Corrected node springs= 0
Node soil begins= 1
No. of boundary conditions= 0

Mod. of elasticity= 576000 kips/ft^2
Unit weight = 0
Max. soil displacement= .1 ft
Modulus of Subgrade Reaction= 5000 + 0 *Z kips/ft^3
Ground line reduction factor= 1

seg.	Length (ft)	Width (ft)	Moment of Inertia (ft^4)
1	2	1	4.908594E-02
2	2	1	4.908594E-02
3	2	1	4.908594E-02
4	2	1	4.908594E-02
5	2	1	4.908594E-02
6	2	1	4.908594E-02
7	2	1	4.908594E-02
8	2	1	4.908594E-02
9	2	1	4.908594E-02
10	2	1	4.908594E-02
11	3	1	4.908594E-02
12	4	1	4.908594E-02
13	5	1	4.908594E-02
14	6	1	4.908594E-02
15	7	1	4.908594E-02

Node #	Soil modulus kip/ft^3	Node spring kip/ft
1	5000	5000
2	5000	10000
3	5000	10000
4	5000	10000
5	5000	10000
6	5000	10000
7	5000	10000
8	5000	10000
9	5000	10000
10	5000	10000
11	5000	12500
12	5000	17500
13	5000	22500
14	5000	27500
15	5000	32500
16	5000	17500

** ** ** ** ** ** ** ** ** ** **

5 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	1.0087
2	-1.0087	10.6542
3	-10.6542	28.6288
4	-28.6288	20.5505
5	-20.5506	-116.6817
6	116.6817	20.5901
7	-20.5901	28.7236
8	-28.7236	10.7161
9	-10.7161	0.6191
10	-0.6191	-1.5678
11	1.5678	-0.2964
12	0.2964	0.0894
13	-0.0894	-0.0104
14	0.0104	0.0014
15	-0.0014	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-0.5044	-0.00018	-0.00010	-0.5044	0.0000	0.0000
2	-4.3184	-0.00014	-0.00043	-2.1592	0.0000	0.0000
3	-4.1645	0.00027	-0.00042	-2.0823	0.0000	0.0000
4	13.0264	0.00166	0.00130	6.5132	0.0000	0.0000
5	64.5770	0.00340	0.00646	32.2885	0.0000	0.0000
6	114.7480	-0.00000	0.01147	57.3740	0.0000	252.0000
7	64.5692	-0.00340	0.00646	32.2846	0.0000	0.0000
8	13.0705	-0.00165	0.00131	6.5352	0.0000	0.0000
9	-3.9552	-0.00026	-0.00040	-1.9776	0.0000	0.0000
10	-3.9550	0.00014	-0.00040	-1.9775	0.0000	0.0000
11	-1.5173	0.00011	-0.00012	-0.6069	0.0000	0.0000
12	0.3274	0.00001	0.00002	0.0935	0.0000	0.0000
13	0.1164	-0.00001	0.00001	0.0259	0.0000	0.0000
14	-0.0219	0.00000	-0.00000	-0.0040	0.0000	0.0000
15	0.0022	-0.00000	0.00000	0.0003	0.0000	0.0000
16	-0.0002	0.00000	-0.00000	-0.0001	0.0000	0.0000

SUM SPRNG= 252.0000 VERSES THE SUM OF THE APPLIED FORCES= 252

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** ** ** ** ** ** ** ** ** ** **

6 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	0.8406
2	-0.8406	8.8785
3	-8.8785	23.8574
4	-23.8574	17.1254
5	-17.1255	-97.2348
6	97.2347	17.1584
7	-17.1584	23.9364
8	-23.9363	8.9301
9	-8.9301	0.5159
10	-0.5159	-1.3065
11	1.3065	-0.2470
12	0.2470	0.0745
13	-0.0745	-0.0086
14	0.0086	0.0012
15	-0.0012	-0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-0.4203	-0.00015	-0.00008	-0.4203	0.0000	0.0000
2	-3.5987	-0.00012	-0.00036	-1.7993	0.0000	0.0000
3	-3.4704	0.00023	-0.00035	-1.7352	0.0000	0.0000
4	10.8553	0.00138	0.00109	5.4277	0.0000	0.0000
5	53.8141	0.00283	0.00538	26.9071	0.0000	0.0000
6	95.6233	-0.00000	0.00956	47.8117	0.0000	210.0000
7	53.8076	-0.00283	0.00538	26.9038	0.0000	0.0000
8	10.8921	-0.00138	0.00109	5.4460	0.0000	0.0000
9	-3.2960	-0.00022	-0.00033	-1.6480	0.0000	0.0000
10	-3.2959	0.00012	-0.00033	-1.6479	0.0000	0.0000
11	-1.2644	0.00009	-0.00010	-0.5058	0.0000	0.0000
12	0.2728	0.00001	0.00002	0.0779	0.0000	0.0000
13	0.0970	-0.00001	0.00000	0.0216	0.0000	0.0000
14	-0.0183	0.00000	-0.00000	-0.0033	0.0000	0.0000
15	0.0018	-0.00000	0.00000	0.0003	0.0000	0.0000
16	-0.0002	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 210.0000   VERSES THE SUM OF THE APPLIED FORCES= 210

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7 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	0.7205
2	-0.7205	7.6102
3	-7.6102	20.4492
4	-20.4492	14.6789
5	-14.6790	-83.3441
6	83.3441	14.7072
7	-14.7072	20.5169
8	-20.5169	7.6544
9	-7.6544	0.4422
10	-0.4422	-1.1199
11	1.1199	-0.2117
12	0.2117	0.0638
13	-0.0638	-0.0074
14	0.0074	0.0010
15	-0.0010	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-0.3603	-0.00013	-0.00007	-0.3603	0.0000	0.0000
2	-3.0846	-0.00010	-0.00031	-1.5423	0.0000	0.0000
3	-2.9747	0.00019	-0.00030	-1.4873	0.0000	0.0000
4	9.3046	0.00119	0.00093	4.6523	0.0000	0.0000
5	46.1264	0.00243	0.00461	23.0632	0.0000	0.0000
6	81.9629	-0.00000	0.00820	40.9814	0.0000	180.0000
7	46.1208	-0.00243	0.00461	23.0604	0.0000	0.0000
8	9.3361	-0.00118	0.00093	4.6680	0.0000	0.0000
9	-2.8252	-0.00019	-0.00028	-1.4126	0.0000	0.0000
10	-2.8250	0.00010	-0.00028	-1.4125	0.0000	0.0000
11	-1.0838	0.00008	-0.00009	-0.4335	0.0000	0.0000
12	0.2338	0.00001	0.00001	0.0668	0.0000	0.0000
13	0.0831	-0.00000	0.00000	0.0185	0.0000	0.0000
14	-0.0157	0.00000	-0.00000	-0.0028	0.0000	0.0000
15	0.0015	-0.00000	0.00000	0.0002	0.0000	0.0000
16	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 180.0000   VERSES THE SUM OF THE APPLIED FORCES= 180

** ** ** ** ** ** ** ** ** ** **

8 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	0.6324
2	-0.6324	6.6800
3	-6.6800	17.9498
4	-17.9498	12.8849
5	-12.8849	-73.1576
6	73.1576	12.9097
7	-12.9097	18.0093
8	-18.0092	6.7188
9	-6.7189	0.3882
10	-0.3882	-0.9830
11	0.9830	-0.1858
12	0.1858	0.0560
13	-0.0560	-0.0065
14	0.0065	0.0009
15	-0.0009	-0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-0.3162	-0.00011	-0.00006	-0.3162	0.0000	0.0000
2	-2.7076	-0.00009	-0.00027	-1.3538	0.0000	0.0000
3	-2.6111	0.00017	-0.00026	-1.3055	0.0000	0.0000
4	8.1673	0.00104	0.00082	4.0837	0.0000	0.0000
5	40.4887	0.00213	0.00405	20.2444	0.0000	0.0000
6	71.9452	-0.00000	0.00719	35.9726	0.0000	158.0000
7	40.4838	-0.00213	0.00405	20.2419	0.0000	0.0000
8	8.1950	-0.00104	0.00082	4.0975	0.0000	0.0000
9	-2.4799	-0.00016	-0.00025	-1.2399	0.0000	0.0000
10	-2.4797	0.00009	-0.00025	-1.2399	0.0000	0.0000
11	-0.9513	0.00007	-0.00008	-0.3805	0.0000	0.0000
12	0.2053	0.00001	0.00001	0.0586	0.0000	0.0000
13	0.0730	-0.00000	0.00000	0.0162	0.0000	0.0000
14	-0.0137	0.00000	-0.00000	-0.0025	0.0000	0.0000
15	0.0014	-0.00000	0.00000	0.0002	0.0000	0.0000
16	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 158.0000 VERSES THE SUM OF THE APPLIED FORCES= 158

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** ** ** ** ** ** ** ** ** ** **

10 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	0.5044
2	-0.5044	5.3271
3	-5.3271	14.3144
4	-14.3144	10.2753
5	-10.2753	-58.3409
6	58.3409	10.2951
7	-10.2951	14.3618
8	-14.3618	5.3581
9	-5.3581	0.3096
10	-0.3096	-0.7839
11	0.7839	-0.1482
12	0.1482	0.0447
13	-0.0447	-0.0052
14	0.0052	0.0007
15	-0.0007	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-0.2522	-0.00009	-0.00005	-0.2522	0.0000	0.0000
2	-2.1592	-0.00007	-0.00022	-1.0796	0.0000	0.0000
3	-2.0823	0.00014	-0.00021	-1.0411	0.0000	0.0000
4	6.5132	0.00083	0.00065	3.2566	0.0000	0.0000
5	32.2885	0.00170	0.00323	16.1442	0.0000	0.0000
6	57.3740	-0.00000	0.00574	28.6870	0.0000	126.0000
7	32.2846	-0.00170	0.00323	16.1423	0.0000	0.0000
8	6.5352	-0.00083	0.00065	3.2676	0.0000	0.0000
9	-1.9776	-0.00013	-0.00020	-0.9888	0.0000	0.0000
10	-1.9775	0.00007	-0.00020	-0.9888	0.0000	0.0000
11	-0.7586	0.00005	-0.00006	-0.3035	0.0000	0.0000
12	0.1637	0.00000	0.00001	0.0468	0.0000	0.0000
13	0.0582	-0.00000	0.00000	0.0129	0.0000	0.0000
14	-0.0110	0.00000	-0.00000	-0.0020	0.0000	0.0000
15	0.0011	-0.00000	0.00000	0.0002	0.0000	0.0000
16	-0.0001	0.00000	-0.00000	-0.0000	0.0000	0.0000

SUM SPRNG= 126.0000 VERSES THE SUM OF THE APPLIED FORCES= 126

** ** ** ** ** ** ** ** ** ** **

Highland 18" 06-06-1994
Solution for laterally loaded pile

No. of NP= 32
No. of elements= 15
No. of non-zero P entries= 1
No. of load cases= 4
Corrected node springs= 0
Node soil begins= 1
No. of boundary conditions= 0

Mod. of elasticity= 504000 kips/ft^2
Unit weight = 0
Max. soil displacement= .1 ft
Modulus of Subgrade Reaction= 5000 + 0 *Z kips/ft^3
Ground line reduction factor= 1

seg.	Length (ft)	Width (ft)	Moment of Inertia (ft^4)
1	2	1.5	.2484976
2	2	1.5	.2484976
3	2	1.5	.2484976
4	2	1.5	.2484976
5	2	1.5	.2484976
6	2	1.5	.2484976
7	2	1.5	.2484976
8	2	1.5	.2484976
9	2	1.5	.2484976
10	2	1.5	.2484976
11	3	1.5	.2484976
12	4	1.5	.2484976
13	5	1.5	.2484976
14	6	1.5	.2484976
15	7	1.5	.2484976

Node #	Soil modulus kip/ft^3	Node spring kip/ft
1	5000	7500
2	5000	15000
3	5000	15000
4	5000	15000
5	5000	15000
6	5000	15000
7	5000	15000
8	5000	15000
9	5000	15000
10	5000	15000
11	5000	18750
12	5000	26250
13	5000	33750
14	5000	41250
15	5000	48750
16	5000	26250

** ** ** ** ** ** ** ** ** ** **

4 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	16.7220
2	-16.7220	52.4133
3	-52.4132	73.5478
4	-73.5479	-1.2950
5	1.2947	-308.6788
6	308.6785	-0.7913
7	0.7912	75.6840
8	-75.6842	57.7764
9	-57.7765	25.0706
10	-25.0706	4.1568
11	-4.1568	-3.8180
12	3.8180	-0.8193
13	0.8193	0.2200
14	-0.2200	-0.0141
15	0.0141	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External Loads (kip-ft)	(kips)
1	-8.3610	0.00020	-0.00111	-5.5740	0.0000	0.0000
2	-9.4846	0.00033	-0.00063	-3.1615	0.0000	0.0000
3	7.2784	0.00088	0.00049	2.4261	0.0000	0.0000
4	47.9887	0.00189	0.00320	15.9962	0.0000	0.0000
5	116.2703	0.00246	0.00775	38.7568	0.0000	0.0000
6	165.3642	-0.00001	0.01102	55.1214	0.0000	473.0000
7	115.7060	-0.00248	0.00771	38.5687	0.0000	0.0000
8	47.1915	-0.00188	0.00315	15.7305	0.0000	0.0000
9	7.3991	-0.00082	0.00049	2.4664	0.0000	0.0000
10	-5.8960	-0.00016	-0.00039	-1.9653	0.0000	0.0000
11	-7.7987	0.00008	-0.00042	-2.0796	0.0000	0.0000
12	-3.4079	0.00008	-0.00013	-0.6491	0.0000	0.0000
13	0.5418	0.00001	0.00002	0.0803	0.0000	0.0000
14	0.2469	-0.00000	0.00001	0.0299	0.0000	0.0000
15	-0.0410	0.00000	-0.00000	-0.0042	0.0000	0.0000
16	0.0020	-0.00000	0.00000	0.0004	0.0000	0.0000

SUM SPRNG= 472.9997 VERSES THE SUM OF THE APPLIED FORCES= 473

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6 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	-0.0000	11.1362
2	-11.1362	34.9053
3	-34.9052	48.9800
4	-48.9800	-0.8624
5	0.8624	-205.5681
6	205.5681	-0.5271
7	0.5269	50.4027
8	-50.4028	38.4769
9	-38.4769	16.6961
10	-16.6961	2.7683
11	-2.7683	-2.5426
12	2.5426	-0.5456
13	0.5456	0.1465
14	-0.1465	-0.0094
15	0.0094	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-5.5681	0.00013	-0.00074	-3.7121	0.0000	0.0000
2	-6.3164	0.00022	-0.00042	-2.1055	0.0000	0.0000
3	4.8471	0.00059	0.00032	1.6157	0.0000	0.0000
4	31.9587	0.00126	0.00213	10.6529	0.0000	0.0000
5	77.4316	0.00164	0.00516	25.8105	0.0000	0.0000
6	110.1263	-0.00001	0.00734	36.7088	0.0000	315.0000
7	77.0558	-0.00165	0.00514	25.6853	0.0000	0.0000
8	31.4278	-0.00125	0.00210	10.4759	0.0000	0.0000
9	4.9275	-0.00054	0.00033	1.6425	0.0000	0.0000
10	-3.9265	-0.00010	-0.00026	-1.3088	0.0000	0.0000
11	-5.1936	0.00005	-0.00028	-1.3850	0.0000	0.0000
12	-2.2696	0.00005	-0.00009	-0.4323	0.0000	0.0000
13	0.3608	0.00000	0.00001	0.0535	0.0000	0.0000
14	0.1644	-0.00000	0.00000	0.0199	0.0000	0.0000
15	-0.0273	0.00000	-0.00000	-0.0028	0.0000	0.0000
16	0.0013	-0.00000	0.00000	0.0003	0.0000	0.0000

SUM SPRNG= 314.9998 VERSES THE SUM OF THE APPLIED FORCES= 315

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8 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	8.3433
2	-8.3433	26.1512
3	-26.1512	36.6961
4	-36.6962	-0.6461
5	0.6460	-154.0131
6	154.0129	-0.3948
7	0.3948	37.7621
8	-37.7621	28.8272
9	-28.8272	12.5088
10	-12.5088	2.0740
11	-2.0740	-1.9050
12	1.9050	-0.4088
13	0.4088	0.1098
14	-0.1098	-0.0070
15	0.0070	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External (kip-ft)	Loads (kips)
1	-4.1717	0.00010	-0.00056	-2.7811	0.0000	0.0000
2	-4.7323	0.00016	-0.00032	-1.5774	0.0000	0.0000
3	3.6315	0.00044	0.00024	1.2105	0.0000	0.0000
4	23.9436	0.00094	0.00160	7.9812	0.0000	0.0000
5	58.0122	0.00123	0.00387	19.3374	0.0000	0.0000
6	82.5073	-0.00001	0.00550	27.5024	0.0000	236.0000
7	57.7307	-0.00124	0.00385	19.2436	0.0000	0.0000
8	23.5459	-0.00094	0.00157	7.8486	0.0000	0.0000
9	3.6917	-0.00041	0.00025	1.2306	0.0000	0.0000
10	-2.9418	-0.00008	-0.00020	-0.9806	0.0000	0.0000
11	-3.8911	0.00004	-0.00021	-1.0376	0.0000	0.0000
12	-1.7004	0.00004	-0.00006	-0.3239	0.0000	0.0000
13	0.2703	0.00000	0.00001	0.0401	0.0000	0.0000
14	0.1232	-0.00000	0.00000	0.0149	0.0000	0.0000
15	-0.0205	0.00000	-0.00000	-0.0021	0.0000	0.0000
16	0.0010	-0.00000	0.00000	0.0002	0.0000	0.0000

SUM SPRNG= 235.9998 VERSES THE SUM OF THE APPLIED FORCES= 236

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10 rows of piers

Member No.	Moments (kip-ft) near end	far end
1	0.0000	6.6817
2	-6.6817	20.9431
3	-20.9431	29.3880
4	-29.3881	-0.5175
5	0.5174	-123.3409
6	123.3408	-0.3162
7	0.3162	30.2417
8	-30.2417	23.0861
9	-23.0862	10.0177
10	-10.0177	1.6609
11	-1.6610	-1.5256
12	1.5256	-0.3274
13	0.3274	0.0879
14	-0.0879	-0.0056
15	0.0056	0.0000

Node No.	Spring Force (kips)	Rotation (rads)	Deflect. (ft.)	Soil Pressure (k/ft^2)	External Loads (kip-ft)	(kips)
1	-3.3409	0.00008	-0.00045	-2.2272	0.0000	0.0000
2	-3.7898	0.00013	-0.00025	-1.2633	0.0000	0.0000
3	2.9083	0.00035	0.00019	0.9694	0.0000	0.0000
4	19.1752	0.00075	0.00128	6.3917	0.0000	0.0000
5	46.4589	0.00098	0.00310	15.4863	0.0000	0.0000
6	66.0758	-0.00000	0.00441	22.0253	0.0000	189.0000
7	46.2335	-0.00099	0.00308	15.4112	0.0000	0.0000
8	18.8567	-0.00075	0.00126	6.2856	0.0000	0.0000
9	2.9565	-0.00033	0.00020	0.9855	0.0000	0.0000
10	-2.3559	-0.00006	-0.00016	-0.7853	0.0000	0.0000
11	-3.1162	0.00003	-0.00017	-0.8310	0.0000	0.0000
12	-1.3617	0.00003	-0.00005	-0.2594	0.0000	0.0000
13	0.2165	0.00000	0.00001	0.0321	0.0000	0.0000
14	0.0986	-0.00000	0.00000	0.0120	0.0000	0.0000
15	-0.0164	0.00000	-0.00000	-0.0017	0.0000	0.0000
16	0.0008	-0.00000	0.00000	0.0002	0.0000	0.0000

SUM SPRNG= 188.9999 VERSES THE SUM OF THE APPLIED FORCES= 189

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