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PRELIMINARY FOUNDATION INVESTIGATION

Proposed Single-Family Residence

15000 Calle Real

County of Santa Barbara

California

CLIENT
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Kihei, Maui, HI 96753

May 6, 2008
Lab No: 77259-2
File No: 08-10738-2

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INTRODUCTION

This report presents the results of a preliminary foundation investigation performed at 15000 Calle Real in the County of Santa Barbara, California. Presently, the site is an undeveloped hillside, sloping south at approximately 12%. The descending slopes on the east and west sides of the site are as steep as approximately 33%. The ascending slope north of the site is at approximately 25%.

It is proposed to excavate a slope at an angle of 2 horizontal to 1 vertical into the natural slope which ascends north of the proposed house. A cut slope excavation will also trim the uphill edge of the proposed driveway. Soil generated from the excavation of the proposed house pad and from the cut slope excavations will be placed as compacted soil on the east and west slopes. Retaining walls will be used throughout the development area, both in the proposed house and in the surrounding yard, to support abrupt grade changes.

SCOPE OF WORK

It is the purpose of this investigation to classify the soil disclosed by the exploratory borings and excavations by observation and tests on selected samples. In addition, this study includes laboratory tests to evaluate soil strength, the effect of moisture variation on the soil-bearing capacity, compressibility, liquefaction, and expansiveness. Slope stability will be analyzed based on soil mechanics. Based upon this information, we will provide preliminary grading and foundation recommendations for the proposed single-family residence.

The scope of this investigation does not include the analyses of the corrosive potential of the soil, previous site construction, or analyses of geologic structures and their associated features, such as faults, fractures, bedding planes, strike and dip angles, ancient landslides, potential for earth movement in undisturbed or natural soil formations sloped or level, or other sources of potential instability which relate to the geologic conditions, as these items should be addressed by a qualified Engineering Geologist.

This exploration was conducted in accordance with presently accepted geotechnical engineering procedures currently applied in the local community in order to provide the appropriate geotechnical design characteristics of the foundations soils and of the proposed fill soils in order to properly evaluate the proposed structure with respect to differential settlement based upon the anticipated soil characteristics at the time of construction.

LIMITATIONS

This Laboratory's basic assumption is that the soil borings presented herein are representative of the entire footprint of the proposed development, however, no warranty is implied. If, during the course of construction, soil conditions are encountered which vary from those presented herein, please contact this Laboratory immediately so appropriate field modifications may be expeditiously proposed.

It is your responsibility to contact our office, providing at least 48 hours of notice for grading or footing excavation observations and testing. The observation of excavations during the construction phase represents an opportunity by our firm to either confirm soil conditions estimated by the exploratory borings or to discover soil conditions which have not been addressed. When such undisclosed conditions are encountered, opinions and recommendations addressing these conditions will be rendered at that time.

This report is considered preliminary and no person should consider the recommendations or soil conditions described herein as conclusive. The recommendations and conclusions of this report are considered preliminary until all excavations have been observed during the construction phase, after which a final report will be issued stating that the grading and foundation works accomplished and installed are appropriate for the soil conditions encountered.

FIELD INVESTIGATION

The subsurface soil conditions were explored by four truck-mounted auger borings, which were drilled to depths of up to 15 feet, supplemented by one field density test. The locations of the borings were selected as appropriate and representative. Representative relatively "undisturbed" tube soil samples were obtained during the drilling operation by the thin-walled sampling tube method (ASTM D-1587). Laboratory tests and analyses of representative soil samples, obtained during the drilling operation, were performed to estimate the engineering properties and determine the soil classification of earth materials encountered. The locations of the borings are shown on Plate 1. The boring log data is presented in Appendix A, "Field Investigation", while the results of the laboratory tests are provided in Appendix B, "Laboratory Tests".

SOIL CONDITIONS

1. No groundwater was encountered in the exploratory borings which extended to depths of 15 feet. It should be recognized that water table elevations, even seasonal perched water tables, might fluctuate with time, being dependent upon seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

2. The surface soils were found to have a high potential for expansion.

3. The surface soils were found to be compressible and sensitive to both hydroconsolidation and expansion with increased moisture content.

4. The soil profile at this site is judged to be stiff soil corresponding to a Site Class D as defined by Table 1613.5.2 of the California Building Code (CBC). This estimate is based on the 4-foot deep borings which encountered the

geologic formation known as the Rincon, which is widely regarded as a Type D soil profile since the Standard Penetration Resistance typically results in blow counts having a range of between 15 to 50.

5. The potential for liquefaction is considered to be very low due to the absence of sand layers. The site consists of an expansive clay topsoil over the Rincon shale.

SLOPE STABILITY

It is not possible to identify all areas of potential slope instability and, therefore, it is not possible to remove all risk of slope instability. Some risk of slope instability must be accepted by the owner and future owners if the site is to be developed.

In order to determine the margin of safety for the protection of the property and the general public, a Factor of Safety is calculated by dividing the forces resisting a landslide by the forces triggering a landslide. Hillsides having an actual Factor of Safety that will not reduce lower than 1.0 will not slide; however, Factor of Safety calculations are not exact. There are uncertainties related to the measurement of the soil's strength and how representative the test soil sample is compared to the entire potential slide plane. For civil engineering projects involving surcharge loads on natural slopes from manmade improvements or high hazard earth structure, such as an earth dam, the design Factor of Safety is usually set at 1.5. A Factor of Safety of 1.5 may be inappropriate for determining the margin of safety for a natural slope where no high hazard earth works are proposed. Additionally, when a normal investigation is provided for a small to medium potential landslide, Cornforth (2005) has suggested the appropriate Factor of Safety to be between 1.25 and 1.35.

Geotechnical Engineers are soil mechanics specialists who use their education, knowledge, training, and experience to examine earth slope, and attempt to reduce the risk of foundation failure near earth slopes. Clients may choose to accept or disregard the recommendations of the Geotechnical Engineer, or to seek additional advice.

Geotechnical Engineers cannot detect every condition that could possibly lead to a slope failure. Earth slope failures are a natural phenomenon which fail in ways we do not fully understand. Conditions are often hidden and below ground. Geotechnical Engineers cannot guarantee that a slope will be stable or safe under all circumstances, or for a specified period of time. Likewise, remedial treatments, like any medicine, cannot be guaranteed.

Slopes can be managed, but slope failures cannot always be controlled. To live near a slope is to accept some degree of risk.

The strength of the soil making up the slopes at this site was measured on both relatively undisturbed soil samples, as well as remolded samples. The remolded samples were re-compacted to approximately 90% to simulate the proposed compacted earth slopes. The relatively undisturbed soil samples yielded strength values from direct shear tests which were then used in a slope stability analysis to model the proposed cut slope and the foundation soil of the proposed compacted earth slopes. The results of the slope stability analysis indicate a Factor of Safety against failure of 1.5 for the cut slopes of 2:1 north of the proposed house and for the west proposed fill slope. The proposed fill slopes also calculated to a Factor of Safety of 1.5 when bench and key excavations are at least 8 feet below the existing natural grade. The east fill slope, however, must extend to the base of the natural slope. The current plan shows the east fill slope stopping short of the base of the natural slope. The Slope Stability Analysis is documented in Appendix C.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

The expansive clay soil encountered at the site is identified as the most challenging aspect of the proposed development. The clay layer is approximately 15 feet in thickness. We anticipate the clay will transition to a weathered shale below this depth. We recommend a drilled pile and grade beam foundation system to support the residence and/or or any other appurtenant improvement which will not tolerate periodic differential movement. The Rincon Formation is prone to instabilities, such as surficial slope failures, mudflows, creep, expansion, and shrinkage. Perfect performance of appurtenant improvements is an unacceptable level of expectation on the part of the property owner or future owners. Associated features, such as patios, walkways, trellis columns, pool equipment enclosure, and driveways will be subject to movement due to the expansion and shrinkage of the clay surface soils. The appurtenant improvements may also be supported by drilled piles; however, depending on factors, such as economics and tolerance, supporting the appurtenant improvements on piles is typically determined to be economically infeasible. If the decision is made to design the appurtenant improvements without the piles and grade beams, the owner then seeks to provide support economically and must accept the risk.

An economical way of reducing the anticipated movement of the appurtenant improvements, such as walkways and patios, is by moisture control of the supporting soils and by carefully choosing the type of building materials used to construct these associated features. Moisture control can be approached by installing surface storm drain collection systems, controlling surface water, and the proper placement of planting areas around the foundation system. Water entering the subsurface soils can be reduced by the placement of a false bottom below planters or an impervious membrane, such as visqueen beneath decorative rock, patios, or paths. Controlling surface water and directing it away from slopes is critical. However, even with correct drainage the moisture content of the soil will change from summer to winter and, therefore, the soils will shrink and swell, moving any item supported over the soil. Drainage can serve to reduce the rate of movement.

With respect to construction materials for the appurtenant improvements, redwood decks are more flexible than concrete patios and do not reflect as much damage from soil movement. If concrete flatwork is required, there are at least two alternative approaches. One is to maintain exterior flatwork disconnected from the exterior foundation of the house, since the exterior flatwork may experience more movement than the foundation. By leaving the two disconnected, the exterior flatwork is allowed to float. The problem, however, is that the flatwork may tend to float away from the house, creating an uneven gap distance between the foundation and the edge of the concrete slab or creating a trip edge at porch entries.

The other alternative is to dowel the flatwork into the exterior foundation with steel rebar to prevent the differential movement and to prevent the gap and trip edge from occurring. This method, however, holds one end of the flatwork fixed while the other is free to move, thus cracking the concrete. For either method, the owner can expect movement and, therefore, cracks. Individual stone pavers with grass between the joints provide an alternative style of "flatwork" with less cracks and the differential movement is not as noticeable. Where the improvements discussed above are located adjacent to slopes, a lateral and downward creeping occurs which undermines the feature constructed over and supported by the expansive soil. Prevention of the downward movement is only accomplished with a drilled pile and grade beam foundation system.

It is the opinion of this Laboratory the proposed grading and construction are feasible from a soil-engineering perspective provided the recommendations contained in this soil engineering report are incorporated into the design and implemented during construction.

It is the understanding of this Laboratory the proposed single-family residence will be a multi-level wood frame structure with concrete structural slab and/or raised wood floors. Portions of the structure will be below grade. Based upon this understanding, we present the following preliminary recommendations:

GRADING

1. The area to be graded shall be cleared of surface vegetation, including roots and root structures.

2. A key shall be placed at the toe of all fill slopes which are to be constructed on natural slopes which are inclined at an angle of 5 horizontal to 1 vertical or steeper. This key shall be a minimum of 12 feet in width, shall extend a minimum of 24 inches into the undisturbed Rincon shale measured at the toe of the slope, shall extend a minimum of 4 feet beyond the toe of the slope, and shall be inclined slightly into the hill. The stiff Rincon clay layer is anticipated to be approximately 8 feet deep below the existing grade.

3. During fill placement, all contact surfaces between undisturbed original ground and compacted fill material shall be either horizontal or vertical, and shall be located a minimum of 8 feet below the original undisturbed ground surface so as to rest on the stiff Rincon clay.

4. In the area to be prepared to receive compacted soil, the loose topsoil and compressible surface soils shall be removed and observed by a representative of our firm. Upon approval of excavation, the exposed ground surface shall be scarified an additional 6 to 8 inches, moistened to the optimum moisture content, and compacted to 90% of the relative compaction. We anticipate the depth of the surface soil removal to be from 8 feet below the existing grade.

5. The removed surface soils and/or imported approved fill may then be placed in loose lifts of approximately 6 inches, thoroughly mixed, moistened to 5% over the optimum moisture content, and compacted to a minimum of 90% relative compaction.

6. Rocks greater than 6 inches in size shall be removed from the soil being spread for compaction.

7. All fill slopes which are created during the grading operation shall be properly shaped to a maximum slope angle of 2 horizontal to 1 vertical, and compacted by rolling the sheepsfoot roller or similar compaction equipment over the slope face at vertical lift intervals of 30 inches or less.

8. The compaction standard shall be the latest adoption of the ASTM D-1557 method of compaction.

9. Positive surface drainage shall direct water away from all slopes and away from the foundation system of the proposed structure.

FOUNDATIONS

1. The entire foundation shall be supported by piles. These recommendations do not change even when the existing grade is removed by the excavation to create the multi-levels of the proposed graded pad.

2. All footings shall be designed as grade beams able to span between drilled and cast-in-place concrete piles. All floors shall be designed to span between the grade beams.

3. A collapsible cardboard box[1] (CCB) forming material shall be placed below the grade beams and below concrete structural slabs to prevent the uplift swell pressures of the expansive soil from acting on the bottom of the structure.

4. All piles shall be drilled a minimum distance of 10 feet into the stiff shale layer, which was encountered at depths of approximately 15 feet below the present grade. A skin friction value of 1,000 psf may be assumed for that portion of the pile extending into the shale layer. An end bearing value of 3,000 psf at the pile tip may also be assumed. The minimum length of a pile shall be 10 feet below the bottom of the grade beam. The minimum diameter shall be 18 inches.

5. This Laboratory shall be requested to inspect the pile excavations prior to placement of steel and concrete.

6. The foundation shall be designed by a Civil or Structural Engineer.

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[1] Available from Shepler's, 9103 East Almeda, Texas 77054
Telephone: (713) 799-1150, FAX: (713) 799-8431 (Allow at least two weeks for shipping)

RETAINING WALLS

Cantilevered - For cantilevered retaining walls, such as site walls and garden walls, which do not form part of the structure, we recommend the following:

1. The cantilevered retaining wall shall be designed assuming an active soil pressure equivalent to a fluid (E.F.P.) whose weight is 35 pcf for level backfill conditions and 52 pcf for backfill slopes, which are constructed at an angle of up to 27 degrees. These values are based on Coulomb's Equation and the following assumed backfill soil values: internal angle of friction equal to 34 degrees, cohesion equal to 0, and a total unit weight of soil equal to 125 pcf. The E.F.P. value does not include surcharge loads and is based on a free-draining condition. The free-draining condition must be created by placing the backfill specified in this section of the report.

2. Retaining walls may be designed using pseudostatic analyses based on the Mononobe Okabe approach. We have estimated the seismic earth pressures using the Mononobe Okabe method and assuming a horizontal ground acceleration of 0.41g (design basis acceleration from FRISKSP by Blake for 10% probability of exceedence in 50 years) and assuming drained backfill conditions. The seismic earth pressure ($\triangle P_{AE}$) resulting from seismic loads acting on retaining walls may be estimated as $\triangle P_{AE} = 21H^2$, in pounds force per lineal foot of wall, for an inverted triangular pressure distribution with the resultant force acting 0.6H above the base of the wall.

3. The bottom of the retaining wall footing shall extend a minimum distance of 36 inches below the undisturbed natural grade, and shall be designed assuming an allowable soil bearing value of 2,000 psf. For footings placed on slopes, the base of the toe or keyway placed at the toe shall extend to such a depth that there exists 10 horizontal feet between the bottom of the footing and the daylight line of the adjacent slope. It should be noted the key may be placed adjacent to the downhill edge of the retaining wall footing in order to attain the recommended downhill grade footing embedment. This 3-foot-deep spread footing may experience seasonal movement due to the expansive soil. The movement is expected to cause cosmetic cracking, but not be a structural failure problem. The owner must decide if he can tolerate the cosmetic cracking. If not, the wall will need to be supported by piles which conform to the pile recommendations in the FOUNDATIONS section of this document.

4. A passive soil pressure equivalent to a fluid whose weight is 350 pcf and a coefficient of friction against sliding of 0.35 may be assumed for the footing excavation described in the recommendation above.

5. The use of equipment to compact soil within the wedge of backfill defined by a 1:1 line projected up from behind the retaining wall to the surface shall be limited to handheld rammer plate compactors, such as a Wacker BS 45Y. A string line shall be placed along the top of the wall to monitor possible rotation of the wall

due to the compaction surcharge. If the wall begins to bow or lean away from the backfilling operations, the compaction process shall stop and the Geotechnical Engineer shall be notified immediately such that modified compaction recommendations can be given at that time.

6. The finish covering on the face of the wall, such as stucco or paint, may be adversely affected by moisture intrusion from the backfill through the back of the wall. To prevent this, you should consider waterproofing the back of the wall and footing. All waterproofing and application of waterproofing shall be in accordance with the specifications of the product supplier.

7. Retaining wall backfill shall be a clean coarse sand or gravel wrapped in a filter fabric. The gravel shall be separated from adjacent native soil by a filter fabric, such as Mirafi 140N™. The retaining wall shall be serviced by appropriately placed weep holes or a perforated drain. This drainage feature must include at least 2 cubic feet of gravel wrapped in filter fabric. Lower quality native backfill material may be utilized outside the triangular wedge which extends upwards from the inside edge of the retaining wall and is a minimum width of 60% of the wall height at ground surface. The sand between the wall and native soil shall have a Sand Equivalent of 20 or greater and an Expansion Index equal to 0. To avoid excessive amounts of sand and gravel backfill, do not allow the excavation contractor to cut a vertical excavation 2 to 4 feet beyond the back of the retaining wall footing or stem. Cut only to the point needed to install the drainpipe and slope the excavation back as specified.

8. It is assumed that the rough grade excavation behind the retaining wall is to be cut at a temporary slope angle of 1 horizontal to 1 vertical in order to comply with Cal-OSHA safety requirements.

9. All soil backfill shall be compacted to a minimum of 90% relative compaction. It should be noted retaining walls designed assuming active soil conditions are anticipated to deflect seasonally. In addition, surface features which obtain their support from retaining wall backfill materials are anticipated to express differential movement with respect to the retaining wall, as the wall may be resting upon a thinner depth of fill or undisturbed original ground and the surface features may be resting upon a considerable thickness of compacted fill which has settlement characteristics differing from that of original ground. The differential movement between the wall and slab patio may be undesirable. In order to hide or prevent such differential movement, an alternate design may be required, such as but not limited to placing a planter between the wall and slab or connecting the slab to the wall, creating a retaining wall which is pinned at the top, not cantilevered.

Partially Restrained - For restrained or partially restrained retaining walls or cantilevered retaining walls which form a portion of the foundation system of the structure, we recommend the wall be designed as a braced wall utilizing at-rest pressures in accordance with the following recommendations:

1. The retaining wall shall be designed assuming an at-rest soil pressure equivalent to a fluid (E.F.P.) whose weight is 55 pcf for level backfill conditions and 73 pcf for backfill slopes, which are constructed at an angle of up to 27 degrees. These values are based on the same assumed conditions stated in Recommendation No. 1 under the Cantilevered section. The at-rest condition for a level backfill is based on the following equation: E.F.P.=$K_{0\gamma}$ where K_0=1-sin ϕ, γ is the total unit weight of soil, and ϕ is the internal angle of friction.

2. Retaining walls may be designed using pseudostatic analyses based on the Mononobe Okabe approach. We have estimated the seismic earth pressures using the Mononobe Okabe method and assuming a horizontal ground acceleration of 0.41g (design basis acceleration from FRISKSP by Blake for 10% probability of exceedence in 50 years) and assuming drained backfill conditions. The seismic earth pressure ($\triangle P_{AE}$) resulting from seismic loads acting on retaining walls may be estimated as $\triangle P_{AE} = 21H^2$, in pounds force per lineal foot of wall, for an inverted triangular pressure distribution with the resultant force acting 0.6H above the base of the wall.

3. The retaining wall footing shall conform to the FOUNDATIONS recommendations and may be designed assuming an allowable soil bearing value of 2,000 psf. For footings placed on or adjacent to slopes, the base of the toe or keyway placed at the toe shall extend to such a depth that there exists 10 horizontal feet between the bottom of the footing and the daylight line of the adjacent slope.

4. A passive soil pressure equivalent to a fluid whose weight is 350 pcf and a coefficient of friction against sliding of 0.35 may be assumed for the footing excavation described in the recommendation above.

5. The retaining wall shall be serviced by a perforated drain which is located a minimum of 12 inches below top of the adjacent interior concrete slab-on-grade floor.

6. Walls, foundations, and connections between walls and foundations forming interior finished rooms of the structure shall be waterproofed by the proper application of a moisture barrier, such as Mirafi™ M-800, followed by Miradry™. A drainage composite, such as Miradrain™, shall be placed over the Miradry™. All waterproofing products should be applied in strict conformance with the manufacturer's recommendations. The selection of a waterproofing product and the observation of proper installation will not involve Pacific Materials Laboratory. We recognize the need for waterproofing; however, it is not in our realm to know the optimum product for application to the retaining wall or to confirm proper installation.

7. It is assumed that the rough grade excavation behind the retaining wall is to be cut at a temporary slope angle of 1 horizontal to 1 vertical in order to comply with Cal-OSHA safety requirements.

8. Footings located near the retaining wall stem shall extend through any retaining wall backfill and shall be supported on the firm underlying ground surface and behind a 1 horizontal to 1 vertical line projected upward from the base of the wall. As an alternative, this footing can be designed to span across the backfill area and tie into the retaining wall for support.

9. Retaining wall backfill shall include 2 cubic feet per linear foot of wall of 3/8- to 1-inch gravel placed around a 4-inch perforated rigid PVC drainpipe. The perforations of the pipe shall be placed down at the positions of 5 and 7 o'clock. A filter fabric shall separate the gravel from the other backfill soils.

10. Retaining wall backfill above the drainpipe shall be a clean coarse sand or gravel, creating an inverted triangular wedge. Lower quality native backfill material may be utilized outside the triangular wedge which extends upwards from the outside edge of the pipe/gravel at the base of the retaining wall and is a minimum width of 60% of the wall height at ground surface. Coarse clean sand is acceptable when the Sand Equivalent is greater than 20 and the Expansion Index equals 0. To avoid excessive amounts of sand and gravel backfill, do not allow the excavation contractor to cut a vertical excavation 2 to 4 feet beyond the back of the retaining wall footing or stem. Cut only to the point needed to install the drainpipe and slope the excavation back as specified.

11. The use of equipment to compact soil within the wedge of backfill defined by a 1:1 line projected up from behind the retaining wall to the surface shall be limited to handheld rammer plate compactors, such as a Wacker BS 45Y. A string line shall be placed along the top of the wall to monitor possible rotation of the wall due to the compaction surcharge. If the wall begins to bow or lean away from the backfilling operations, the compaction process shall <u>stop</u> and the Geotechnical Engineer shall be notified immediately such that modified compaction recommendations can be given at that time.

12. The engineer designing the retaining wall shall address the following conditions:

    A. When a retaining wall is backfilled without a top restraint, such as a wood floor diaphragm, the stem of the retaining wall acts as a cantilever.

    B. Depending on the rigidity of the top restraint, the wall may act as a beam spanning between the top and bottom points, reversing the tension side of the stem to the front of the wall as opposed to the back as in the case of a cantilever condition.

    C. Structure members deflect when loaded. The users guide to the widely used computer program RetainPro recommends the deflection of the wall be checked because the program does not calculate deflection. Refer to

Section 9 titled "Related Design Considerations" in the manual titled "Basics of Retaining Wall Design", Page 50. As an estimate, the Concrete Reinforcing Steel Institute (CRSI) manual estimates concrete reinforced stems of cantilevered retaining walls will deflect a horizontal distance at the top of the wall equal to the height of the wall divided by 240. We recommend the appropriate deflection equation and values corresponding to load, condition, and material be employed to determine the deflection corresponding to the lateral loads recommended herein such that appropriate connections, tiebacks, bracing, or construction joints can be placed within the structural design to properly account for the deflection. The total deflection may not occur during the backfilling operation, but rather sometime after the frame structure is built over and adjacent to the retaining wall.

PAVEMENT

1. Beneath paved driveway and parking areas, we recommend the top loose surface soils be removed and recompacted to 90% relative compaction, the top 9 inches being recompacted to 95% relative compaction. The subgrade area shall be check rolled in order to detect isolated soft spots. Any areas found to be yielding under the wheel loads of the equipment shall be stabilized by removal and recompaction. The stability referred to here is based on static wheel loads. The soil at this site is expansive and the driveway pavements will be subject to instability as already discussed on Page 4 of this document.

2. The Class 2 aggregate base shall be recompacted to a minimum of 95% relative compaction in accordance with the ASTM D-1557 test method. Asphalt concrete shall be placed only after the Class 2 aggregate base has been demonstrated to be firm and unyielding.

3. If asphalt pavement is selected for the finished pavement surface, we recommend an R-Value of the subgrade soil be performed by this Laboratory in order to provide appropriate thickness of Class 2 aggregate base and asphalt concrete.

4. Maintenance to assist in reducing the potential for rapid deterioration of the asphalt paved areas shall include surface treatment approximately 6 months to 1 year after construction and approximately 3 years from the first treatment. Pavement conditions should be reviewed at least once a year for cracks, puddling of surface water, and overall appearance. If possible, this review should be done in the fall such that cracks may be repaired which may otherwise allow moisture to pass through the pavement and weaken the subgrade.

ADJACENT LOADS

Where footings are placed at varying elevations, the effect of adjacent loads may be calculated using the widely published Formulas for Stresses in Semi-infinite Elastic Foundations or the Boussinesq figures and equations for both vertical and horizontal surcharge loads.

SETTLEMENT

It is the intent of the recommendations contained in this report to achieve angular distortions[2] of approximately 1/480. A total settlement of approximately 1 inch or less is anticipated in the undisturbed, native soil and approximately 1% to 1.5% of the fill height is the anticipated total settlement at areas where compacted fill soil is placed in accordance with the grading recommendations provided in this soil engineering report. Movement from expansive soil has already been discussed in this report under the heading APPURTENANCES. The soil bearing values and estimated settlements contained in this report are preliminary and may need to be modified after the foundation and grading plans are substantially complete.

CONSTRUCTION OBSERVATION

The owner or his agent shall request the Project Geotechnical Engineer to observe all excavations prior to placement of compacted soil, gravel backfill, or rebar and concrete.

PLAN REVIEW

We request the grading and foundation plans be submitted to our office for a general review to verify substantial compliance to the recommendations contained in this report.

CLOSURE

The recommendations contained herein are for the sole use of our client and are based upon this Laboratory's understanding of the project which has been described herein. If the project scope, location, or conceptual design is subsequently altered, this Laboratory shall be requested to modify, as necessary, the recommendations contained herein as is appropriate for the new development concept. If the recommendations of this report are not implemented within one year, we recommend an update and review of the contents of this report be performed by this Laboratory.

[2] Angular distortion is the ratio of the vertical differential settlement divided by the horizontal distance over which the vertical differential is measured.

The recommendations contained herein are based upon the assumption that Pacific Materials Laboratory shall be requested to perform the testing and observation services which will be required during the grading and foundation operations in order to verify that the actual soil conditions encountered and the construction procedures are consistent with the recommendations contained herein. If this service is performed by others, only the technical correctness of the actual analytical soil tests described here is attested to by this Laboratory.

Thank you for the opportunity of providing this service. If you have any questions regarding this matter, please do not hesitate to call.

Respectfully submitted,

PACIFIC MATERIALS LABORATORY, INC.

Ronald J. Pike
Geotechnical Engineer, G. E. 2291

RJP:vlh

cc: Addressee (3)
Hugh Twibell, Architect, FAX (805) 687-9671

SITE PLAN

15000 Calle Real
Santa Barbara County, California

Pacific Materials Laboratory, Inc.

Scale: none	Plate 1 Lab No: 77259-2 File No: 07-10738-2 Date: October 19, 2007

APPENDIX A

FIELD INVESTIGATION

May 6, 2008

Lab No: 77259-2

File No: 08-10738-2

Pacific Materials Laboratory of Santa Barbara, Inc.

Lab No: 77259-2
File No: 07-10738-2

BORING LOG DATA

BORING NO. B-1

Drill Rig Operator: Kump/Kump

Date Drilled: 9/24/07

Dry Density (pcf)	Moisture Content (%)	Depth (ft)	Soil Log	Soil Description
		0		Black sandy CLAY, dry and firm
		1		
79.5	18.4	2		
		3		
78.4	13.4	4		
		5		
		6		
		7		Brown CLAY, dry and firm
78.4	24.9	8		
		9		
		10		
		11		Light brown silty CLAY, moist and firm
113.0	22.8	12		
		13		
		14		
90.0	24.4	15		

LEGEND
■ - Thin-Walled Tube Sample ASTM D-1587

Lab No: 77259-2
File No: 07-10738-2

BORING LOG DATA

BORING NO. B-2

Drill Rig Operator: Kump/Kump Date Drilled: 9/24/07

Dry Density (pcf)	Moisture Content (%)	Depth (ft)	Soil Log	Soil Description
		0		Black sandy CLAY
		1		
	14.1	2		Medium brown sandy CLAY, dry and firm
		3		
85.7	11.1	4		
		5		
		6		
		7		
118.7	9.5	8		Moist and firm
		9		
		10		
		11		
88.2	13.1	12		
		13		
		14		
94.1	14.2	15		

LEGEND
■ - Thin-Walled Tube Sample ASTM D-1587

Lab No: 77259-2
File No: 07-10738-2

BORING LOG DATA

BORING NO. B-3

Drill Rig Operator: Kump/Kump

Date Drilled: 9/24/07

Dry Density (pcf)	Moisture Content (%)	Depth (ft)	Soil Log	Soil Description
		0		Dark brown sandy CLAY, dry and firm
104.8	3.2	2	■	
		3		Brown sandy CLAY, dry and firm
92.8	16.1	4	■	
		6		Gray-brown sandy CLAY, moist and firm
93.5	18.6	8	■	
		10		Brown sandy CLAY, moist and medium firm
100.8	16.7	12	■	
105.3	20.2	15	■	Olive CLAY, moist and firm

LEGEND

■ - Thin-Walled Tube Sample ASTM D-1587

Lab No: 77259-2
File No: 07-10738-2

BORING LOG DATA

BORING NO. B-4

Drill Rig Operator: Kump/Kump Date Drilled: 9/24/07

Dry Density (pcf)	Moisture Content (%)	Depth (ft)	Soil Log	Soil Description
		0		Black sandy CLAY, dry and firm
		1		
80.9	14.2	2		
		3		
66.4	13.9	4		
		5		Medium brown sandy CLAY, dry and firm
		6		
		7		
89.2	13.2	8		
		9		
		10		
		11		
94.7	15.7	12		Moist and firm
		13		
		14		
101.7	8.0	15		

LEGEND
- Thin-Walled Tube Sample ASTM D-1587

APPENDIX B

LABORATORY TESTS

May 6, 2008

Lab No: 77259-2

File No: 08-10738-2

Pacific Materials Laboratory of Santa Barbara, Inc.

MOISTURE DENSITY DETERMINATIONS (ASTM D-1557)

Maximum Density-Optimum Moisture data were determined in the laboratory from soil samples using the ASTM D-1557 Method of Compaction. The results of the Maximum Density-Optimum Moisture tests are tabulated below:

SOIL TYPE	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
I	Brown CLAY with organics	96.1	12.1
Curve Points:	(93.9 @ 12.5) (96.0 @ 17.5) (95.8 @ 20.0)		
II	Brown silty CLAY	121.1	11.5
Curve Points:	(119.1 @ 10.3) (119.1 @ 12.8) (121.0 @ 11.6)		
III	Black CLAY	112.8	15.4
Curve Points:	(110.3 @ 14.0) (111.5 @ 16.5) (112.7 @ 15.3)		

FIELD DENSITY SUMMARY (Sand Cone Method ASTM D-1556)

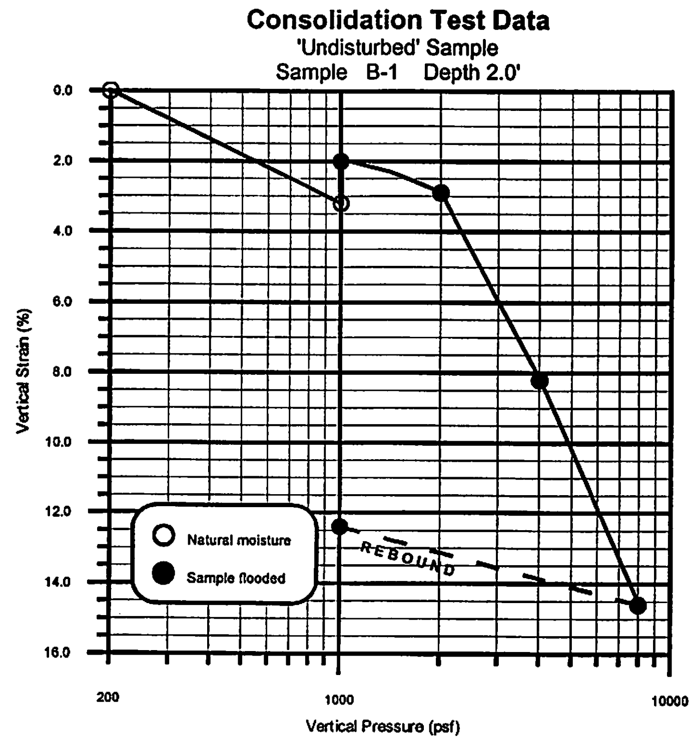
SAMPLE LOCATION	DEPTH (in.)	SOIL TYPE	FIELD MOIST. CONTENT (%)	DRY DENSITY (pcf)	% OF MAX. DRY DENSITY
D-1	12	I	8.6	73.1	76.1

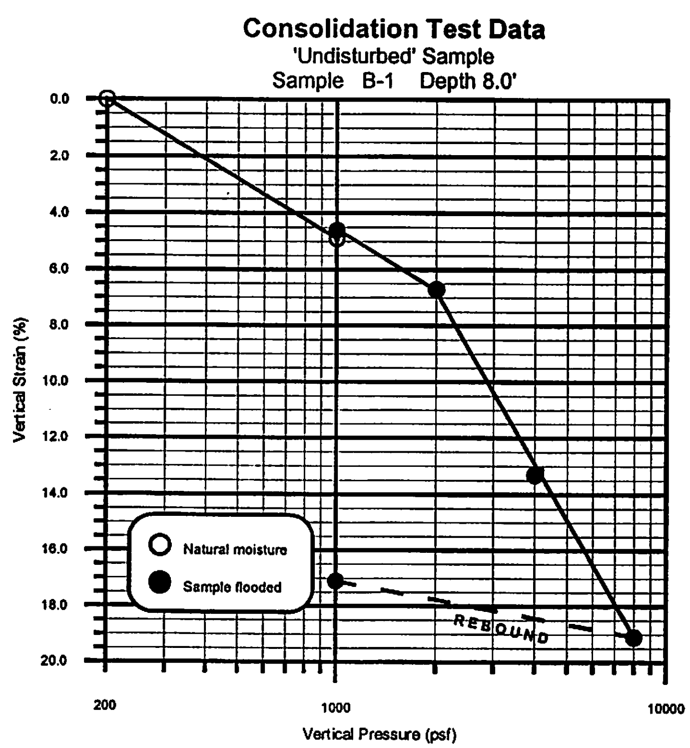
MECHANICAL ANALYSES (Values in Percent Passing ASTM D-422)

SIEVE SIZE	B-1 @ 15'	B-3 @ 15'	B-4 @ 2'	B-1, B-3, B-4 @ 2' and 4'	B-2, B-4 @ 4', 8', 12'
1/2 Inch	98.8	100.0	100.0	100.0	100.0
3/8 Inch	98.8	100.0	100.0	100.0	100.0
No. 4	98.2	99.6	99.9	99.9	99.8
No. 8	96.6	98.2	99.5	99.0	96.8
No. 16	94.2	96.2	98.2	97.6	93.8
No. 30	91.7	92.0	95.5	93.9	87.6
No. 50	89.4	85.3	91.9	88.5	80.2
No. 100	87.6	75.6	84.4	80.0	66.5
No. 200	86.0	68.7	77.5	71.9	54.7

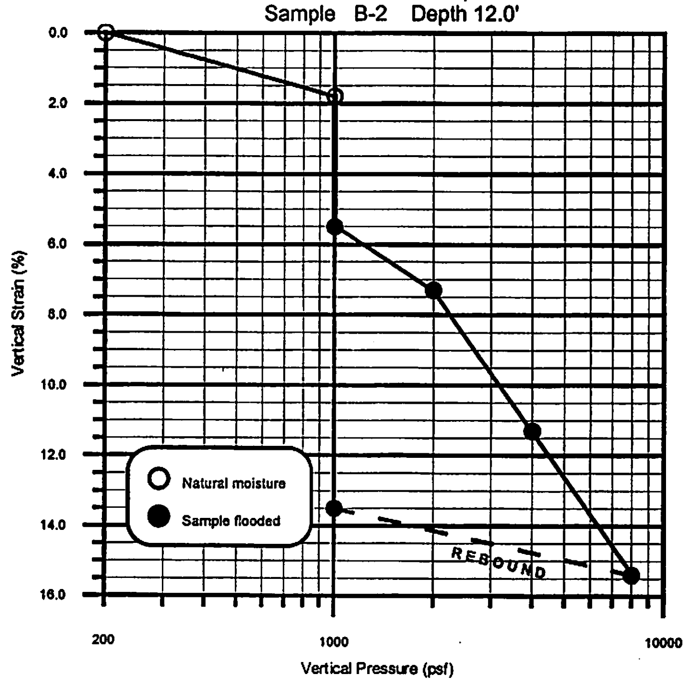
CONSOLIDATION TESTS (ASTM D-2435)

Four consolidation tests were performed on representative in-place tube soil samples in both the natural field and at increased moisture contents. The results of the consolidation tests are presented graphically below.

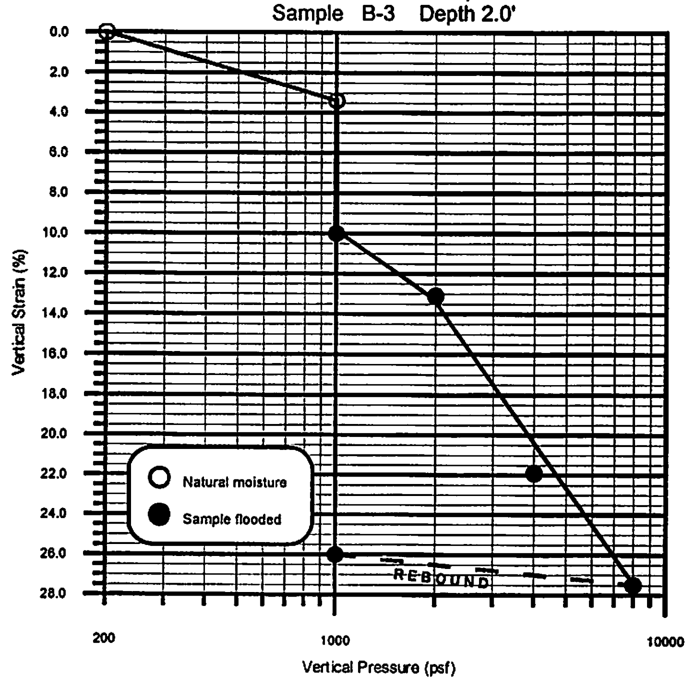




Consolidation Test Data
'Undisturbed' Sample
Sample B-2 Depth 12.0'



Consolidation Test Data
'Undisturbed' Sample
Sample B-3 Depth 2.0'



SAND-SILT-CLAY (By Hydrometer ASTM D 422)

SAMPLE LOCATION	DEPTH (ft.)	SAND %	SILT %	CLAY %	SOIL DESCRIPTION
B-1	15	2	12	86	CLAY
B-3	15	66	8	26	Clayey SAND
B-4	2	20	14	66	CLAY
B-1, B-3, B-4	2, 4	32	16	52	CLAY
B-2, B-4	4, 8, 12	44	16	40	Sandy silty CLAY

EXPANSION TESTS (UBC 18-2)

The Expansive Soil Index was determined by the present UBC 18-2 Expansion Determination Procedure. The results are tabulated below:

SAMPLE LOCATION	DEPTH (ft.)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	EXPANSION INDEX	POTENTIAL FOR EXPANSION
B-1	8	94.0	17.0	128	High
B-2	2	103.3	11.5	72	Medium
B-2	8	114.8	8.7	11	Very low
B-4	2	103.6	11.9	84	Medium

ATTERBERG LIMITS (ASTM D-4318)

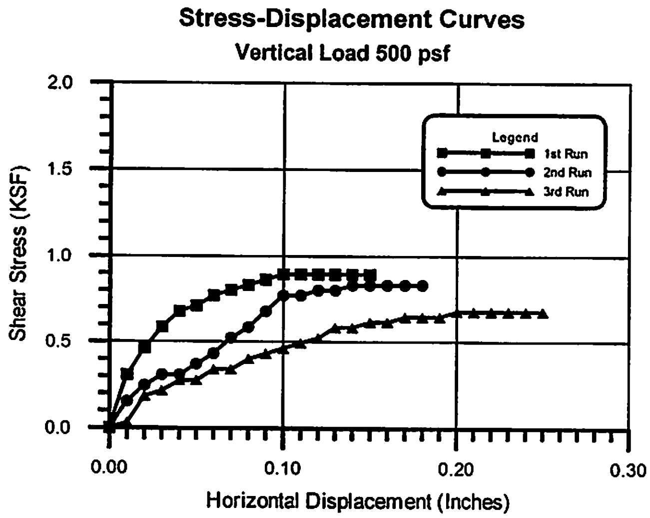
SAMPLE LOCATION	DEPTH (ft.)	SOIL TYPE	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	DEGREE OF EXPANSION	INT. ANGLE OF FRICTION (°) STARK
B-1	15	CH	53	28	25	High	20
B-3	15	CL	38	20	19	Medium	20
B-4	2	CL	39	20	19	Medium	20
B-1, B-3, B-4	2, 4	--	40	28	12	Medium	20
B-2, B-4	1, 4, 12	--	39	18	21	Medium	20

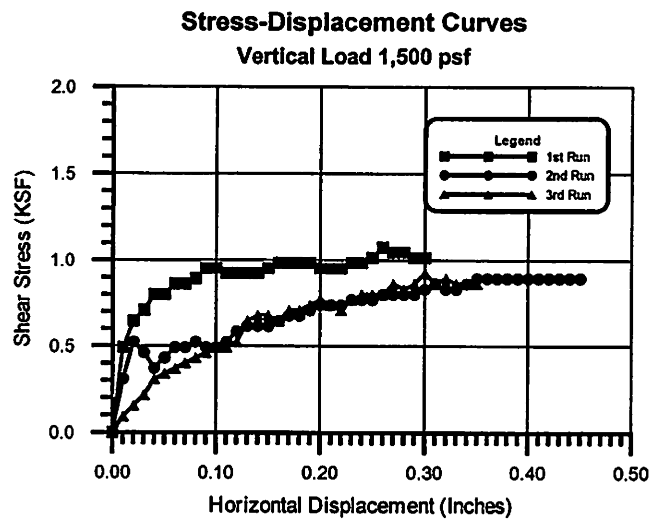
DIRECT SHEAR TESTS (Undisturbed - ASTM D-3080)

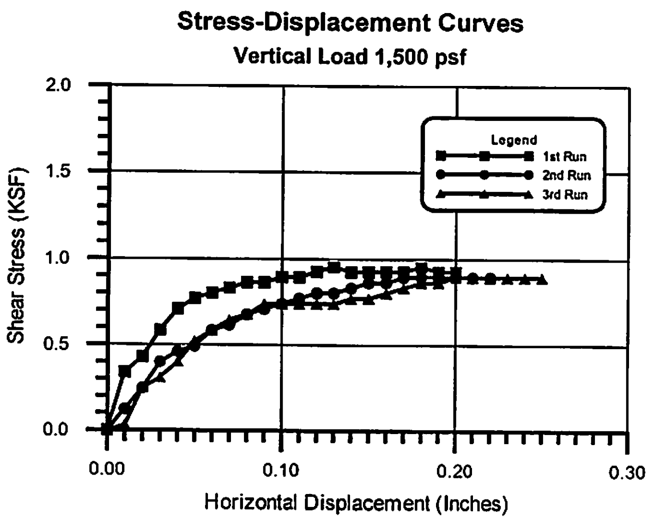
Three direct shear tests were performed on representative "undisturbed" soil samples which were 2.365 inches in diameter and 1 inch thick. The tests were performed under flooded conditions. The results are tabulated below:

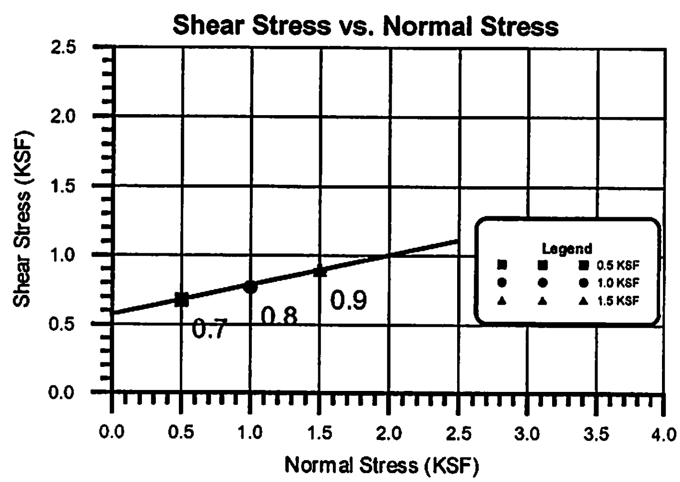
SAMPLE LOCATION	DEPTH (ft.)	INTERNAL ANGLE OF FRICTION (degrees)	COHESION (psf)
B-1	15	11	600
B-3	15	28	420
B-4	2	33	50

Sample B-1 @ 15'

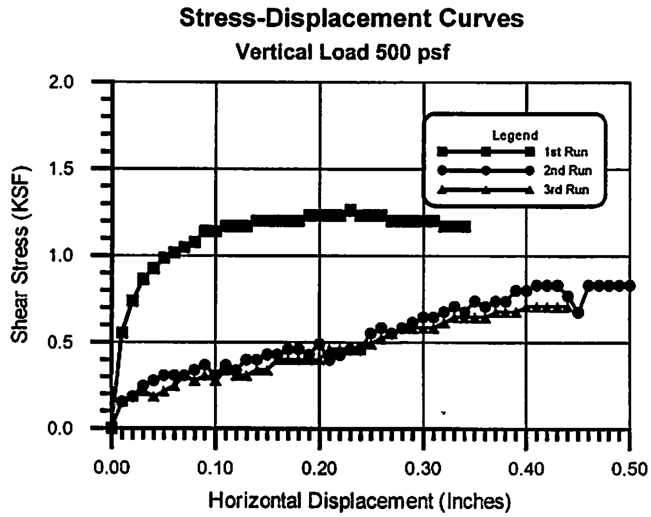


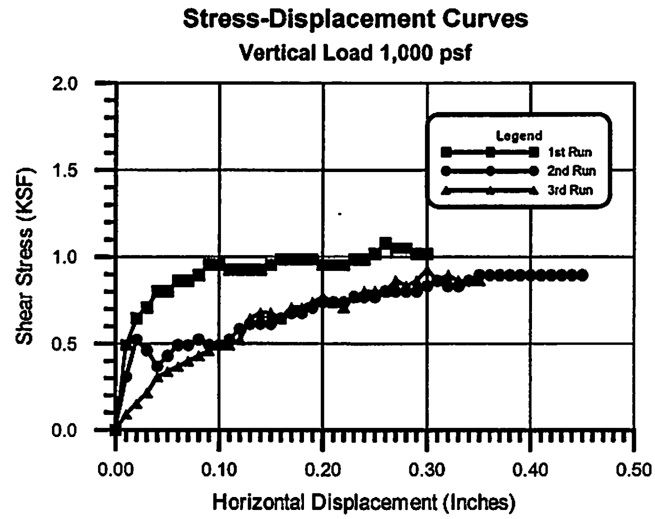


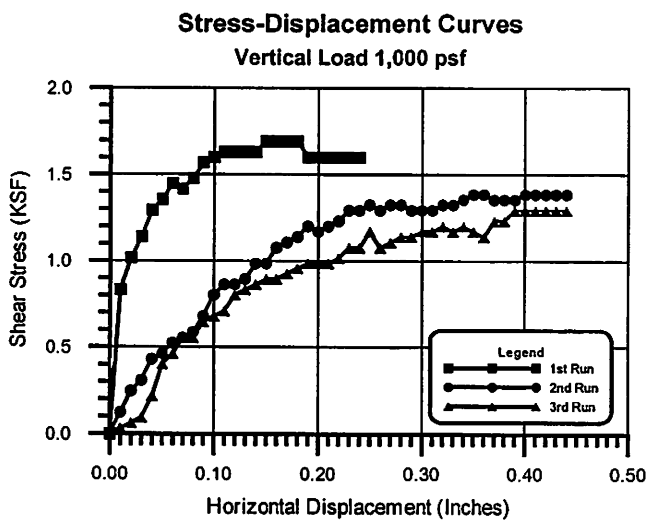


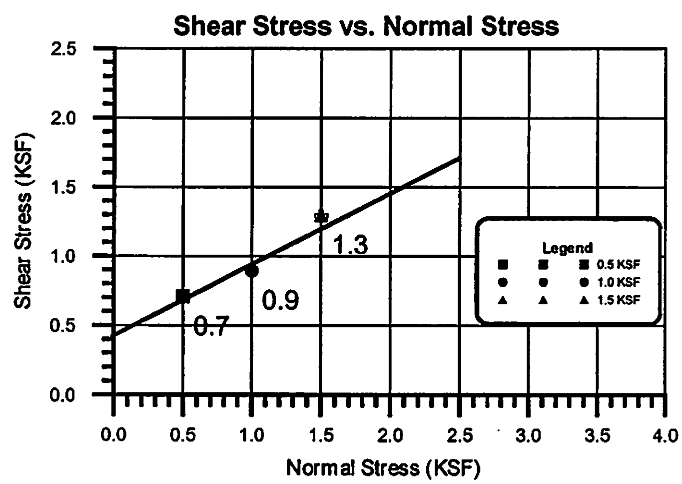


Sample B-3 @ 15'


Stress-Displacement Curves
Vertical Load 500 psf
Shear Stress (KSF)
Horizontal Displacement (Inches)
Legend
1st Run
2nd Run
3rd Run
2.0
1.5
1.0
0.5
0.0
0.00
0.10
0.20
0.30
0.40
0.50

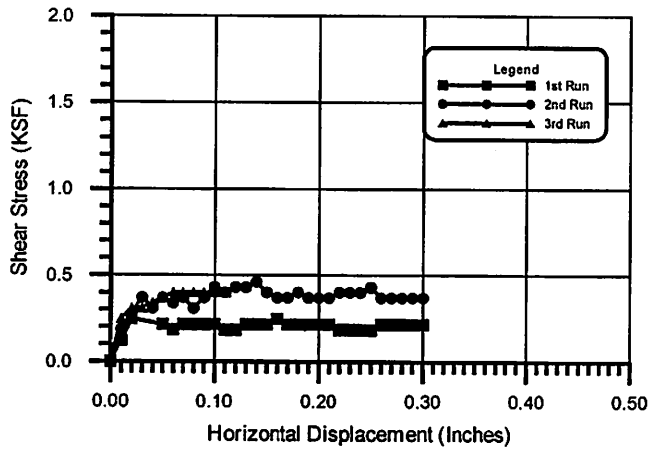

Stress-Displacement Curves
Vertical Load 1,000 psf
Shear Stress (KSF)
Horizontal Displacement (Inches)
Legend
1st Run
2nd Run
3rd Run
2.0
1.5
1.0
0.5
0.0
0.00
0.10
0.20
0.30
0.40
0.50


Stress-Displacement Curves
Vertical Load 1,000 psf
Shear Stress (KSF)
Horizontal Displacement (Inches)
Legend
1st Run
2nd Run
3rd Run
2.0
1.5
1.0
0.5
0.0
0.00
0.10
0.20
0.30
0.40
0.50

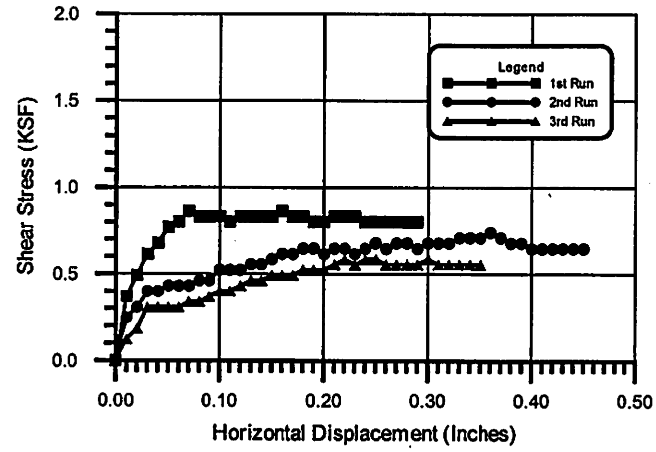

Shear Stress vs. Normal Stress
Shear Stress (KSF)
Normal Stress (KSF)
Legend
0.5 KSF
1.0 KSF
1.5 KSF
1.3
0.9
0.7
2.5
2.0
1.5
1.0
0.5
0.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

Sample B-4 @ 2'

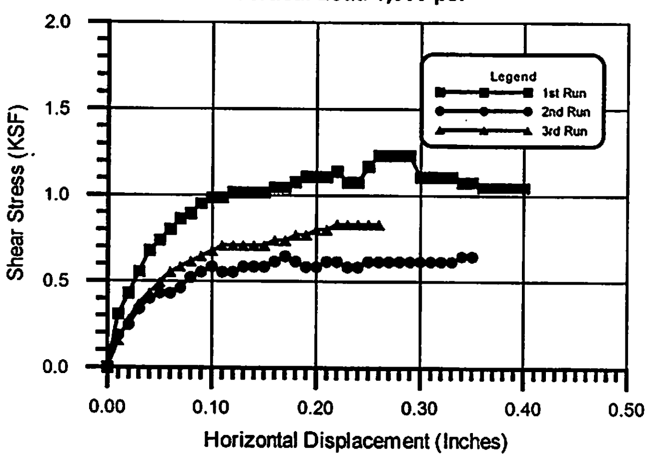
Stress-Displacement Curves
Vertical Load 500 psf



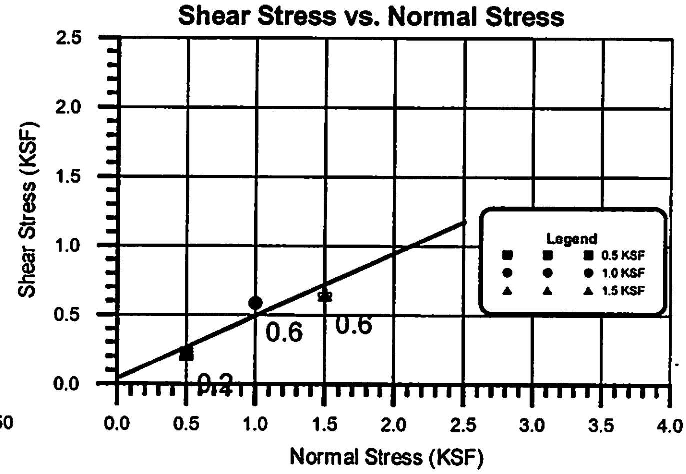
Stress-Displacement Curves
Vertical Load 1,000 psf



Stress-Displacement Curves
Vertical Load 1,500 psf



Shear Stress vs. Normal Stress

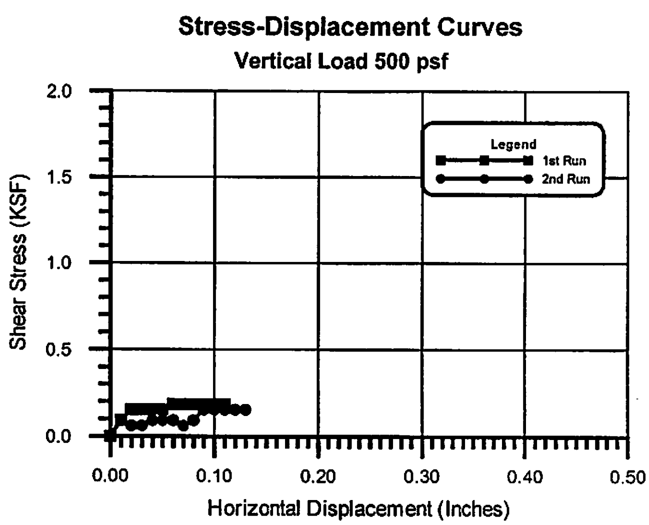


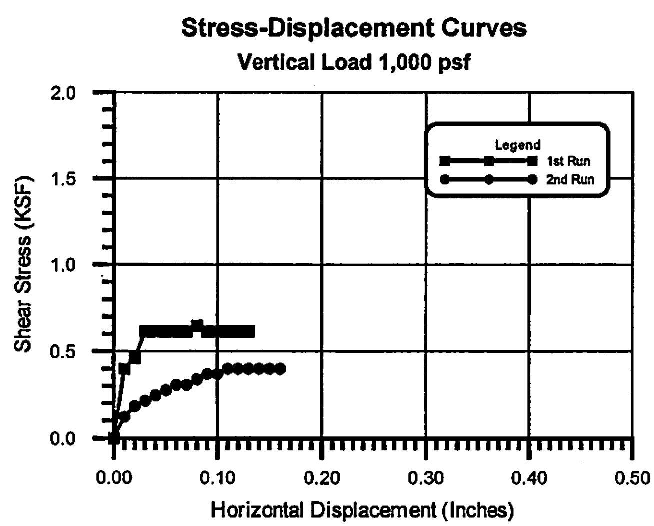
DIRECT SHEAR TESTS (Remoulded - ASTM D-3080)

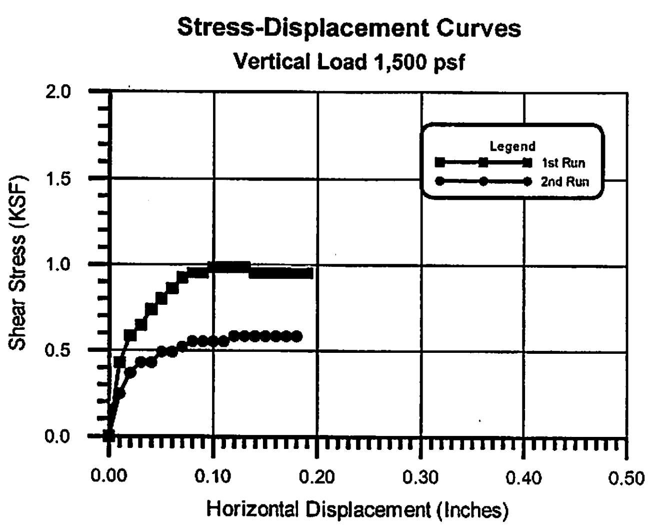
Three direct shear tests were performed on representative "remolded" soil samples which were 2.365 inches in diameter and 1 inch thick. The tests were performed under flooded conditions. The results are tabulated below:

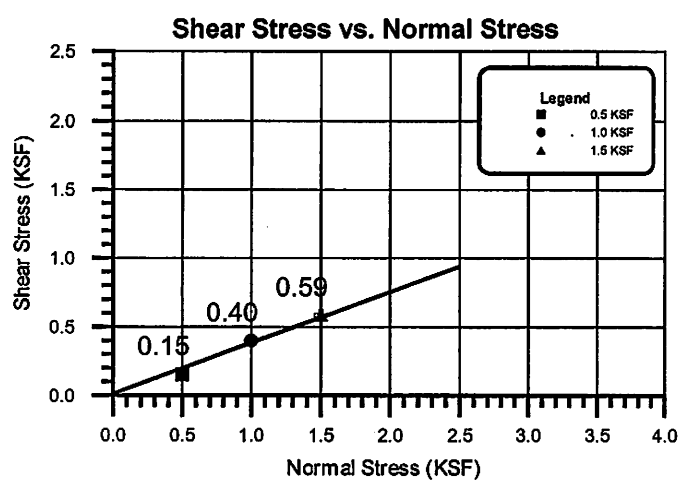
SAMPLE LOCATION	DEPTH (ft.)	INTERNAL ANGLE OF FRICTION (degrees)	COHESION (psf)
B-1, B-3, B-4	2, 4	20	0
B-2, B-4	4, 8	24	0
B-2, B-4	4, 8, 12	22	0

Sample B-1 @ 4', B-3 @ 2', B-4 @ 4'

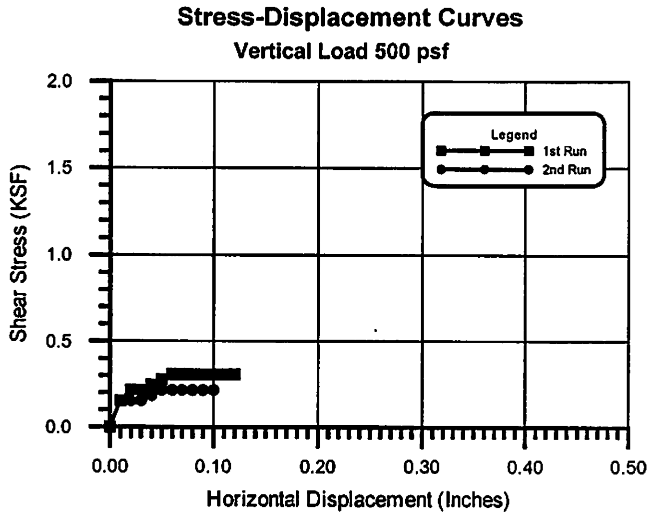


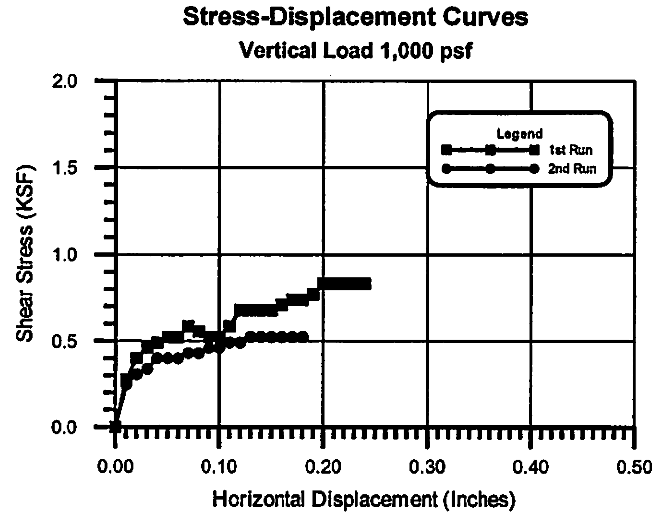


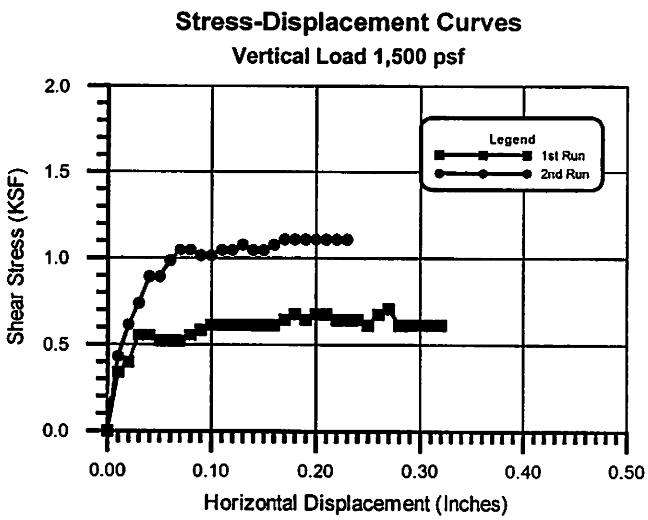


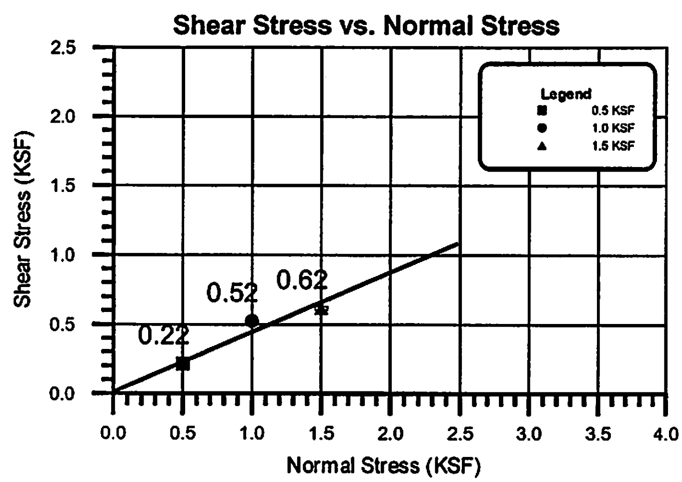


Sample B-2 @ 4', B-4 @ 8'



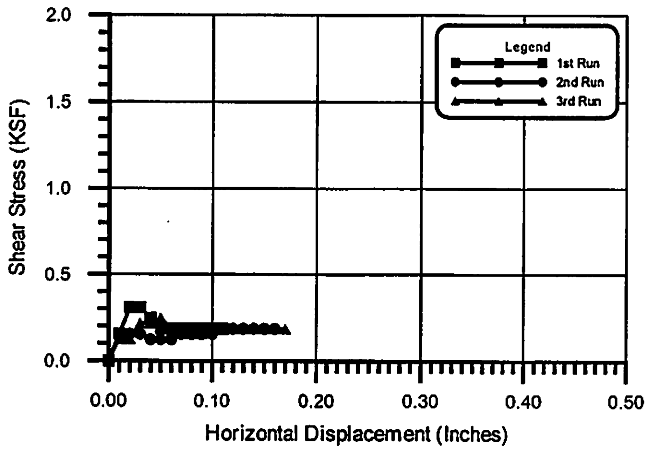




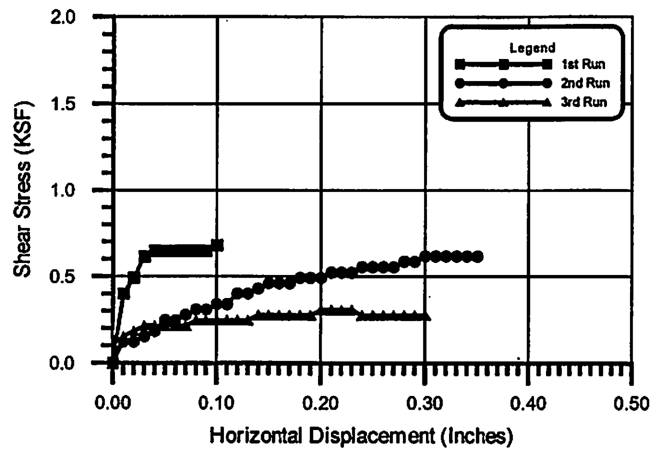


Sample B-2 @ 4', B-4 @ 8', B-4@12'

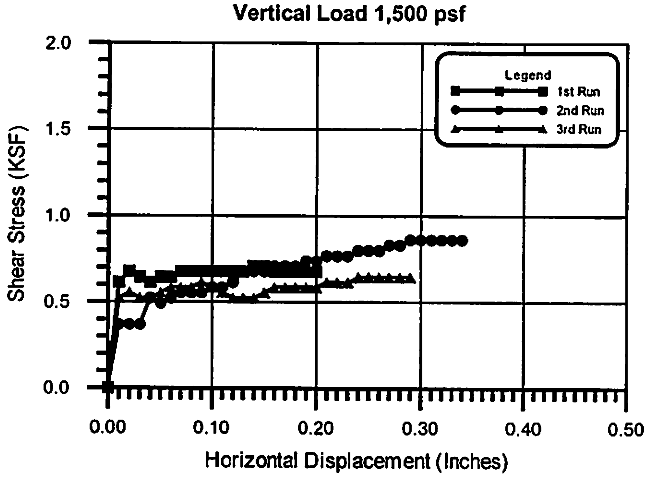
Stress-Displacement Curves
Vertical Load 500 psf



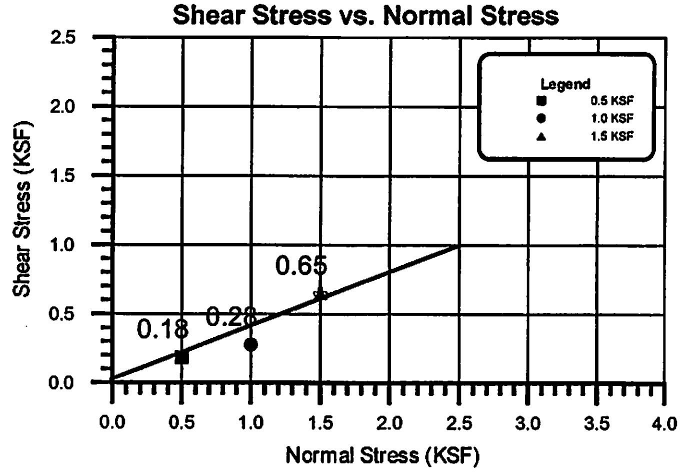
Stress-Displacement Curves
Vertical Load 1,000 psf



Stress-Displacement Curves
Vertical Load 1,500 psf



Shear Stress vs. Normal Stress



APPENDIX C

SLOPE STABILITY ANALYSIS

May 6, 2008

Lab No: 77259-2

File No: 08-10738-2

Pacific Materials Laboratory of Santa Barbara, Inc.

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                         ***  GSTABL7  ***
                  ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*********************************************************************************
                     SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*********************************************************************************
Analysis Run Date:        04/25/08
Time of Run:              04:15PM
Run By:                   Pacific Materials Laboratory
Input Data Filename:      C:\Program Files\G72SW\15kef extended Surface #1.in
Output Filename:          C:\Program Files\G72SW\15kef extended Surface #1.OUT
Unit System:              English
Plotted Output Filename:  C:\Program Files\G72SW\15kef extended Surface #1.PLT
PROBLEM DESCRIPTION:  15,000 Calle Real
                      East Fill Slope to Creek
BOUNDARY COORDINATES
   14 Top   Boundaries
   54 Total Boundaries
Boundary    X-Left     Y-Left    X-Right    Y-Right    Soil Type
   No.       (ft)       (ft)       (ft)       (ft)     Below Bnd
```

1	0.00	137.00	63.00	155.00	1
2	63.00	155.00	80.70	160.00	3
3	80.70	160.00	98.50	165.00	3
4	98.50	165.00	117.00	169.70	3
5	117.00	169.70	119.00	169.70	3
6	119.00	169.70	123.00	169.10	3
7	123.00	169.10	135.00	169.00	3
8	135.00	169.00	138.00	168.95	3
9	138.00	168.95	138.01	173.90	3
10	138.01	173.90	156.00	175.00	3
11	156.00	175.00	161.00	175.20	3
12	161.00	175.20	161.01	170.00	3
13	161.01	170.00	201.00	170.00	3
14	201.00	170.00	226.00	170.00	2
15	0.00	126.50	0.01	124.50	2
16	0.01	124.50	10.00	124.50	2
17	10.00	124.50	10.01	129.00	2
18	10.01	129.00	20.00	129.00	2
19	20.00	129.00	20.01	133.00	2
20	20.01	133.00	30.00	133.00	2
21	30.00	133.00	30.01	138.00	2
22	30.01	138.00	40.00	138.00	2
23	40.00	138.00	40.01	141.00	2
24	40.01	141.00	50.00	141.00	2
25	50.00	141.00	50.50	143.50	2
26	50.50	143.50	60.50	143.50	2
27	60.50	143.50	60.51	146.00	2
28	60.51	146.00	70.00	146.00	2
29	70.00	146.00	70.01	148.00	2
30	70.01	148.00	80.00	148.00	2
31	80.00	148.00	80.01	150.00	2
32	80.01	150.00	90.00	150.00	2
33	90.00	150.00	90.01	153.00	2
34	90.01	153.00	100.00	153.00	2
35	100.00	153.00	100.01	154.00	2
36	100.01	154.00	110.00	154.00	2
37	110.00	154.00	110.01	156.00	2
38	110.01	156.00	120.00	156.00	2
39	120.00	156.00	120.01	158.00	2
40	120.01	158.00	130.00	158.00	2
41	130.00	158.00	130.01	159.00	2
42	130.01	159.00	140.00	159.00	2
43	140.00	159.00	140.01	161.00	2
44	140.01	161.00	150.00	161.00	2
45	150.00	161.00	150.01	163.00	2
46	150.01	163.00	160.00	163.00	2
47	160.00	163.00	160.01	164.00	2
48	160.01	164.00	170.00	164.00	2
49	170.00	164.00	170.01	166.00	2
50	170.01	166.00	180.00	166.00	2
51	180.00	166.00	180.01	168.00	2
52	180.01	168.00	190.00	168.00	2
53	190.00	168.00	190.01	169.00	2
54	190.01	169.00	226.00	169.00	2

User Specified Y-Origin = 80.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	50.0	20.0	0.00	0.0	0
2	120.0	120.0	50.0	20.0	0.00	0.0	0
3	120.0	120.0	0.0	26.0	0.00	0.0	0

Trial Failure Surface Specified By 26 Coordinate Points

Point X-Surf Y-Surf

No.	(ft)	(ft)
1	2.121	137.606
2	7.098	137.129
3	12.088	136.803
4	17.085	136.627
5	22.085	136.603
6	27.083	136.731
7	32.075	137.009
8	37.057	137.439
9	42.023	138.019
10	46.969	138.749
11	51.891	139.629
12	56.785	140.657
13	61.644	141.833
14	66.466	143.155
15	71.246	144.622
16	75.979	146.234
17	80.662	147.988
18	85.289	149.883
19	89.856	151.917
20	94.360	154.088
21	98.796	156.395
22	103.161	158.835
23	107.449	161.405
24	111.658	164.105
25	115.783	166.930
26	119.477	169.628

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	1.74	5.00
2	1.72	5.00
3	1.74	5.00
4	1.74	5.00
5	1.72	5.00
6	1.75	5.00
7	1.73	5.00
8	1.73	5.00
9	1.74	5.00
10	1.73	5.00
11	1.74	5.00
12	1.73	5.00
13	1.73	5.00
14	1.75	5.00
15	1.73	5.00
16	1.74	5.00
17	1.73	5.00
18	1.73	5.00
19	1.74	5.00
20	1.73	5.00
21	1.73	5.00
22	1.74	5.00
23	1.73	5.00

Circle Center At X = 20.375(ft) ; Y = 301.763(ft); and Radius = 165.169(ft)

* * Factor Of Safety Is Calculated By The Modified Bishop Method * *

Factor Of Safety For The Preceding Specified Surface = 1.619

Table 1 - Individual Data on the 46 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	567.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	5.0	1661.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	5.0	2669.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	5.0	3588.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	5.0	4412.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	2.9	2928.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	0.0	2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0

8	2.1	2209.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	5.0	5770.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	2.9	3671.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.0	12.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	2.0	2613.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	4.9	6724.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	3.0	4303.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.5	721.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	1.4	2023.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	4.9	7271.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	3.7	5638.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.0	15.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	1.1	1735.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	1.4	2080.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	3.5	5327.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	3.5	5420.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	0.0	15.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	1.2	1887.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
26	4.7	7139.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	4.0	5909.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.0	14.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.7	941.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.0	54.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	4.6	6443.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
32	0.3	358.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
33	4.3	5686.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
34	0.2	192.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
35	0.0	4.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
36	2.1	2615.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	2.3	2702.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
38	4.1	4597.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	0.3	309.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
40	4.4	4198.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
41	4.3	3400.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
42	4.2	2552.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
43	4.1	1657.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
44	1.2	317.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
45	2.0	276.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
46	0.5	12.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table 2 - Base Stress Data on the 46 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	-5.47	4.61	5.00	93.49	-10.82
2	-3.74	9.59	5.00	173.75	-21.66
3	-2.02	14.59	5.00	246.42	-18.80
4	-0.28	19.58	5.00	311.55	-3.45
5	1.47	24.58	5.00	369.20	22.59
6	3.19	28.55	2.93	409.28	55.58
7	3.19	30.01	0.00	423.81	57.83
8	3.19	31.04	2.07	434.06	59.41
9	4.93	34.57	5.00	462.59	99.23
10	6.66	38.53	2.96	491.22	143.76
11	6.66	40.00	0.01	501.83	147.21
12	6.66	41.02	2.03	509.10	149.57
13	8.40	44.50	5.00	527.32	196.37
14	10.14	48.48	3.08	544.91	246.01
15	10.14	50.25	0.51	552.83	249.93
16	10.14	51.20	1.41	557.08	252.04
17	11.86	54.34	5.00	564.16	298.92
18	13.61	58.64	3.82	571.35	347.02
19	13.61	60.50	0.01	574.72	349.25
20	13.61	61.08	1.17	575.75	349.94
21	15.33	62.32	1.41	573.17	391.29
22	15.33	64.73	3.59	574.09	391.97
23	17.06	68.23	3.70	568.96	430.18
24	17.06	70.01	0.01	567.20	428.73

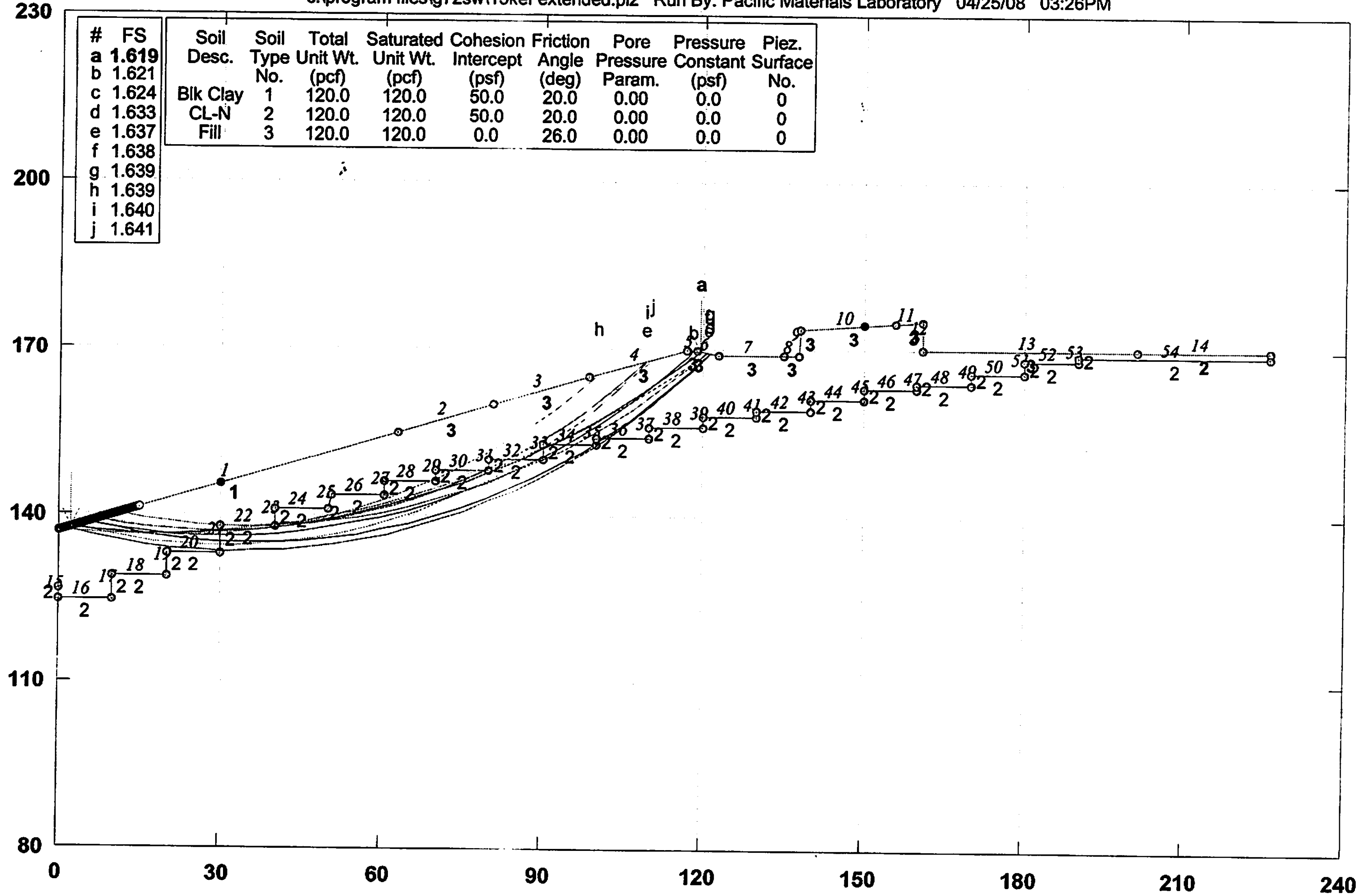
25	17.06	70.63	1.29	566.57	428.21
26	18.81	73.61	5.00	556.40	460.33
27	20.53	77.99	4.29	539.48	482.74
28	20.53	80.01	0.01	532.01	475.43
29	20.53	80.34	0.70	530.78	474.23
30	22.27	80.68	0.04	525.66	505.00
31	22.27	82.99	4.96	513.76	492.47
32	24.01	85.42	0.29	497.42	507.72
33	24.01	87.70	4.71	568.09	490.97
34	25.73	89.93	0.17	543.75	499.40
35	25.73	90.01	0.00	463.77	498.67
36	25.73	91.06	2.32	455.46	488.78
37	25.73	93.23	2.51	509.83	468.24
38	27.48	96.43	4.67	468.24	454.57
39	27.48	98.65	0.33	441.20	428.32
40	29.20	100.98	5.00	401.46	409.62
41	30.94	105.31	5.00	327.63	349.70
42	32.68	109.55	5.00	247.85	275.58
43	34.41	113.72	5.00	162.50	187.37
44	36.15	116.39	1.51	104.14	124.09
45	36.15	118.00	2.48	55.19	65.76
46	36.15	119.24	0.59	10.07	12.00

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing Soil Nail, and Applied Forces if applicable) = 51269.94 (lbs)
Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing, Soil Nail, and Applied Forces if applicable) = 411.56(psf)
Sum of the Driving Forces = 31673.40 (lbs)
Average Mobilized Shear Stress = 254.25(psf)
Total length of the failure surface = 124.57(ft)
CAUTION - Factor Of Safety Is Calculated By The Modified Bishop Method. This Method Is Valid Only If The Failure Surface Approximates A Circular Arc.

**** END OF GSTABL7 OUTPUT ****

15,000 Calle Real East Fill Slope to Creek

c:\program files\g72sw\15kef extended.pl2 Run By: Pacific Materials Laboratory 04/25/08 03:26PM



#	FS
a	**1.619**
b	1.621
c	1.624
d	1.633
e	1.637
f	1.638
g	1.639
h	1.639
i	1.640
j	1.641

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Blk Clay	1	120.0	120.0	50.0	20.0	0.00	0.0	0
CL-N	2	120.0	120.0	50.0	20.0	0.00	0.0	0
Fill	3	120.0	120.0	0.0	26.0	0.00	0.0	0

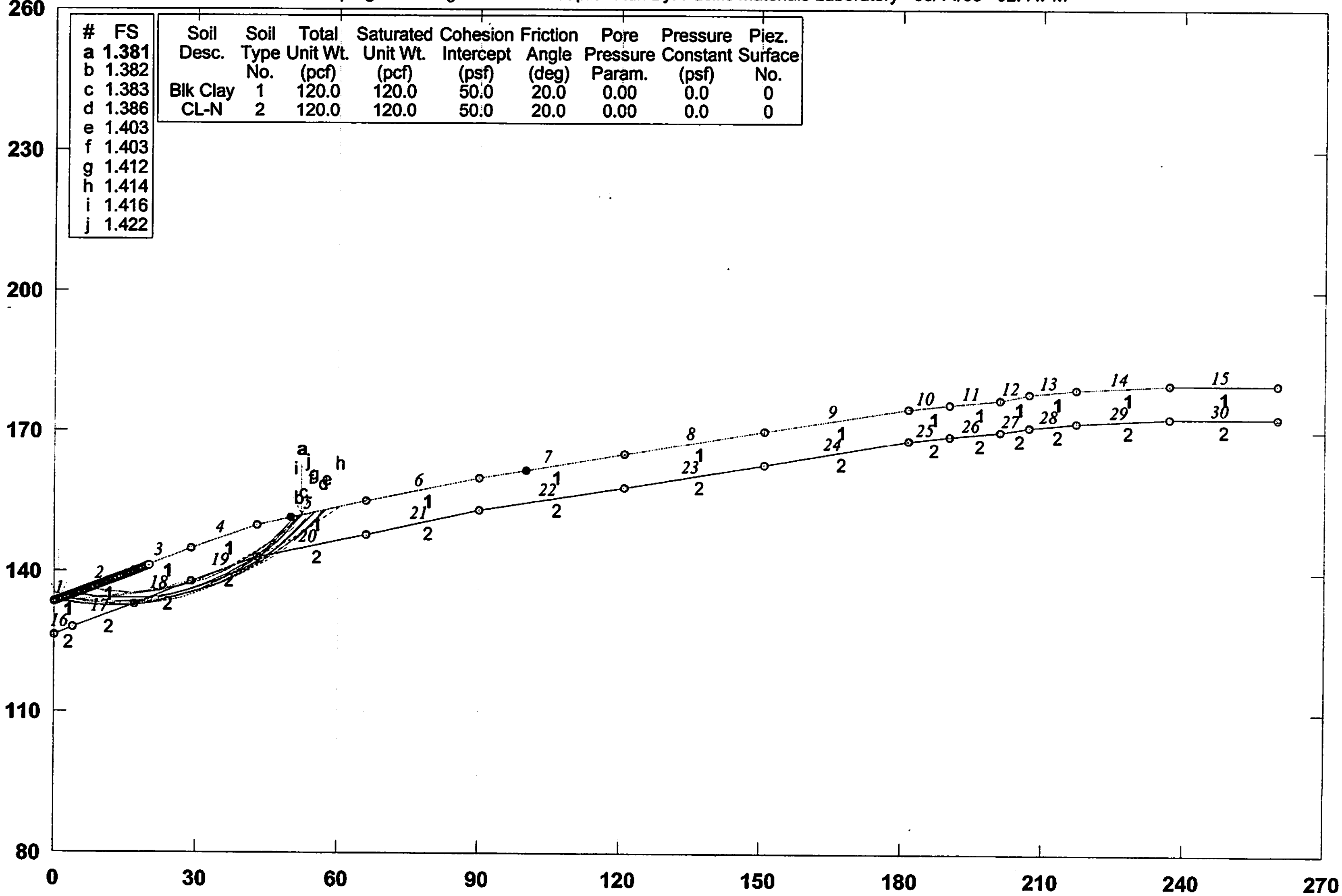
GSTABL7 v.2 FSmin=1.619

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

15,000 Calle Real East Natural Slope

c:\program files\g72sw\15ke-nat.pl2 Run By: Pacific Materials Laboratory 03/14/08 02:44PM



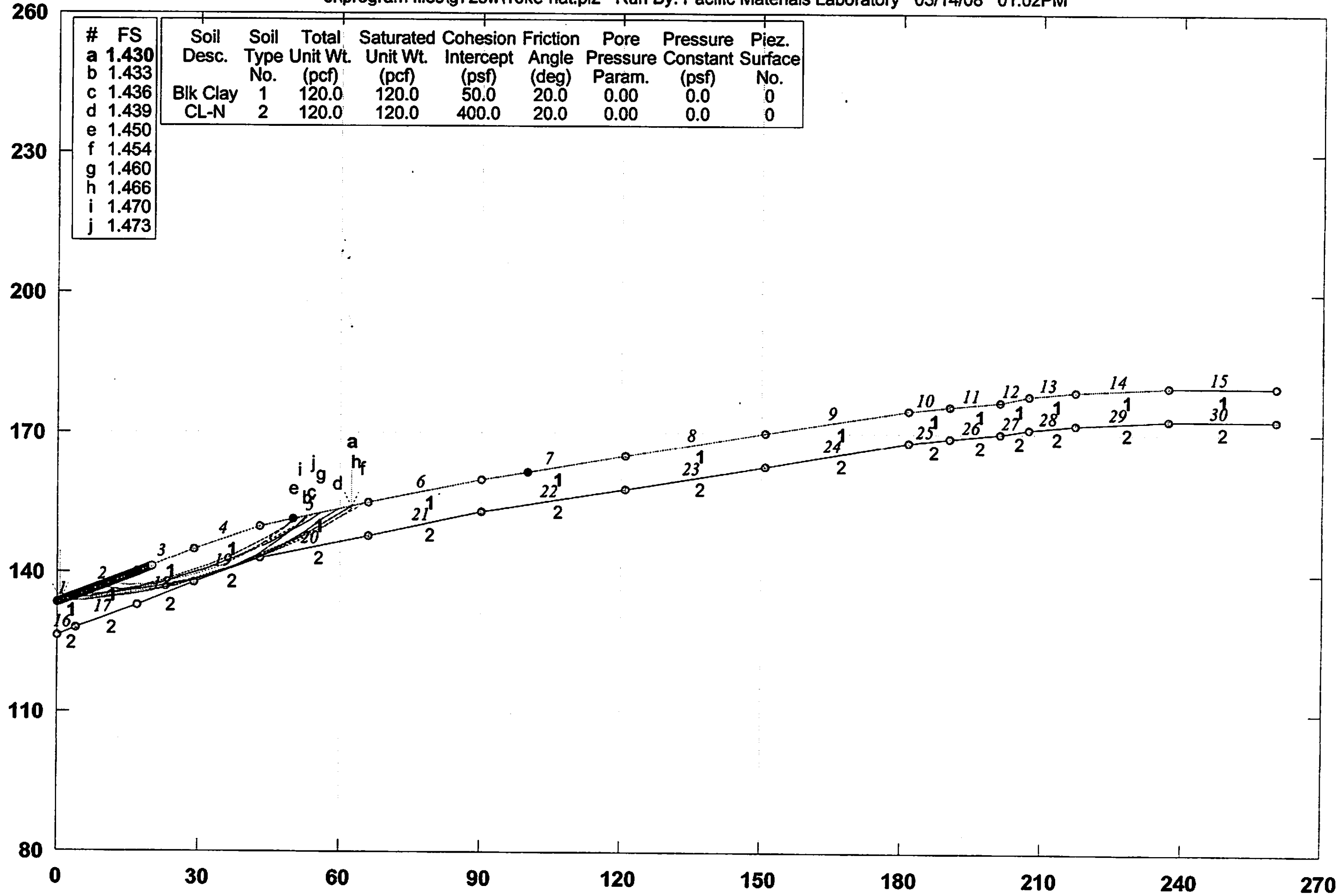
GSTABL7 v.2 FSmin=1.381

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

15,000 Calle Real East Natural Slope

c:\program files\g72sw\15ke-nat.pl2 Run By: Pacific Materials Laboratory 03/14/08 01:02PM



GSTABL7 v.2 FSmin=1.430

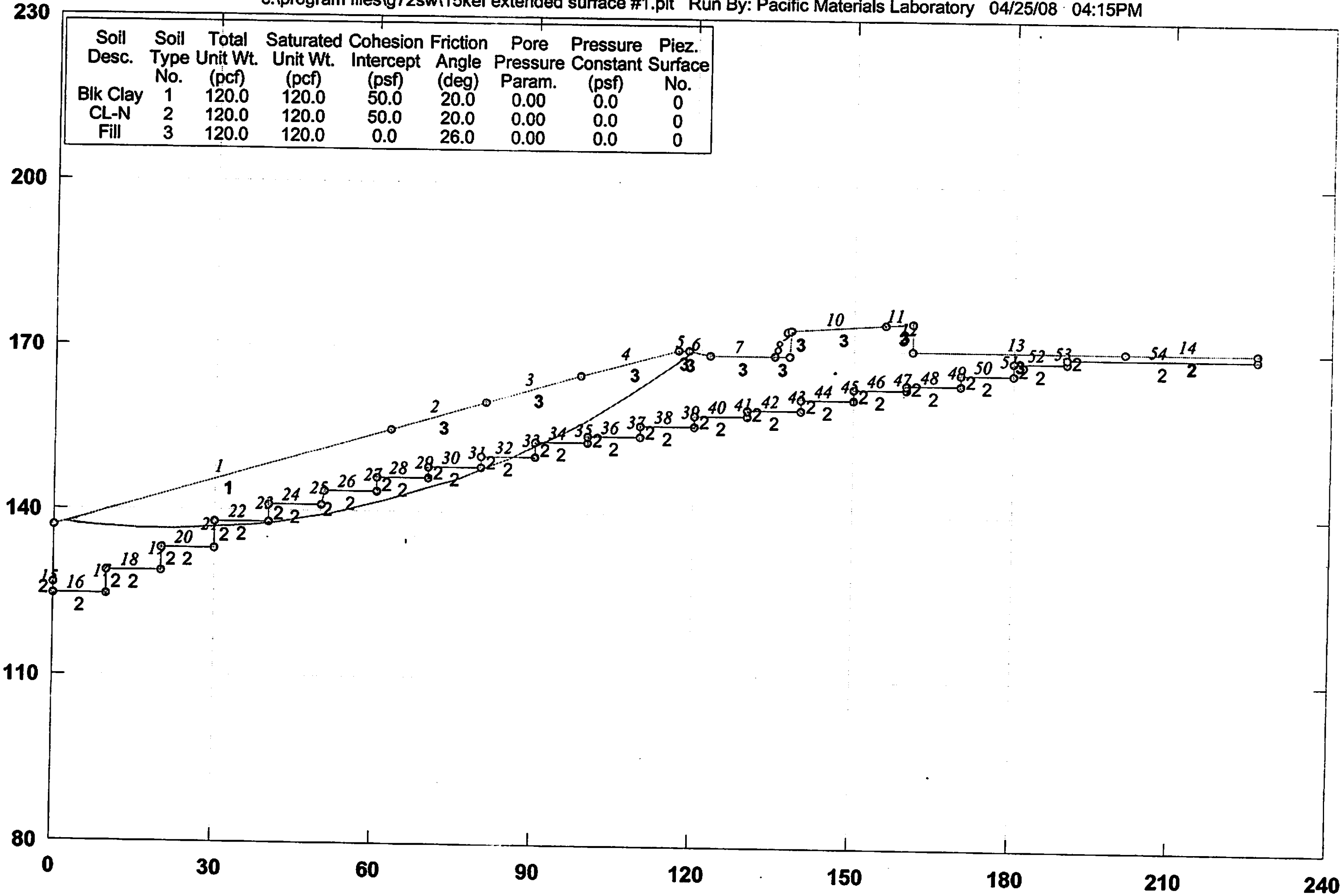
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

15,000 Calle Real East Fill Slope to Creek

c:\program files\g72sw\15kef extended surface #1.plt Run By: Pacific Materials Laboratory 04/25/08 04:15PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Blk Clay	1	120.0	120.0	50.0	20.0	0.00	0.0	0
CL-N	2	120.0	120.0	50.0	20.0	0.00	0.0	0
Fill	3	120.0	120.0	0.0	26.0	0.00	0.0	0



GSTABL7 v.2 FSmin=1.619

Factor Of Safety Is Calculated By The Modified Bishop Method

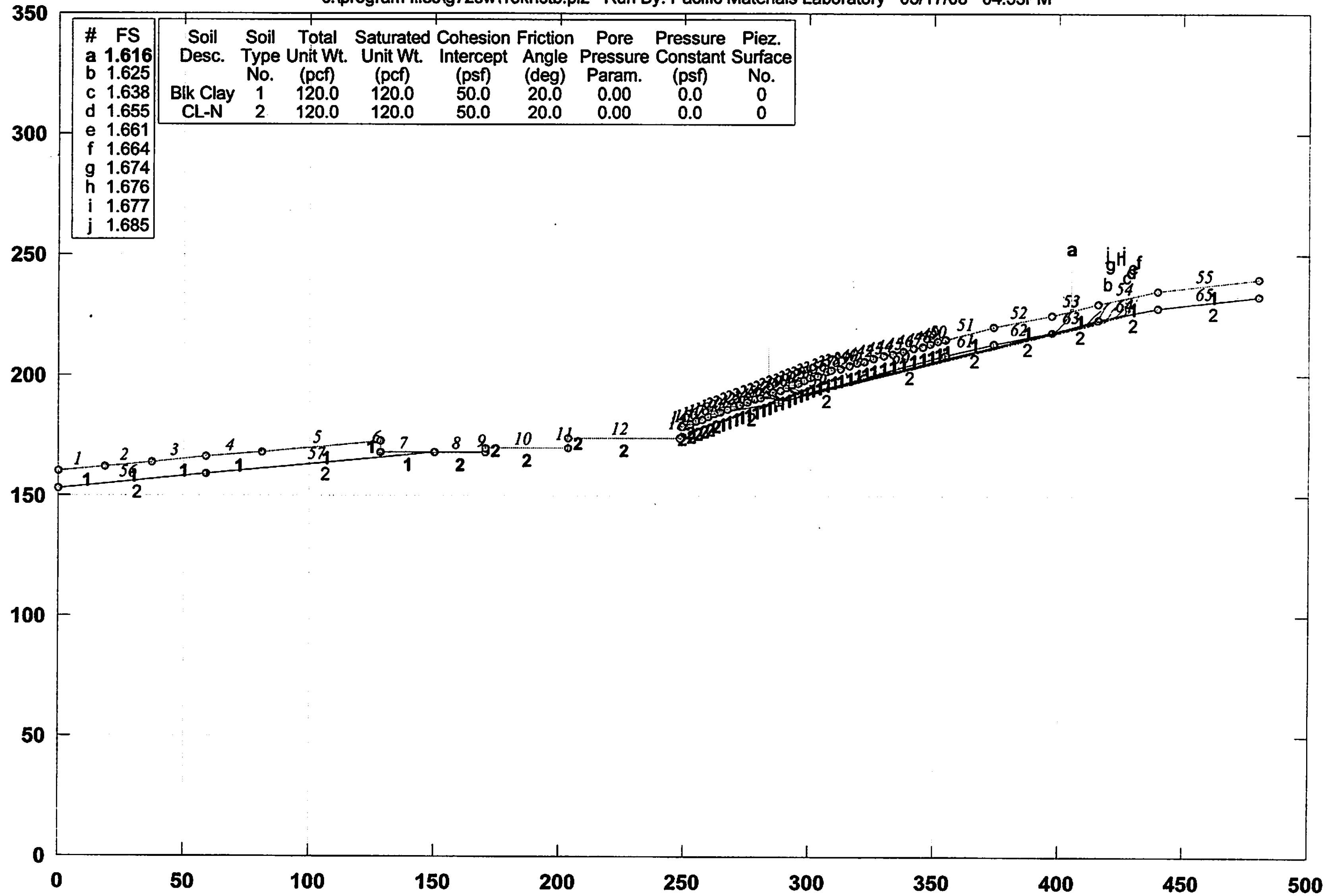
GSTABL7

15,000 Calle Real North Cut Slope Block Failure Surfaces

c:\program files\g72sw\15knctb.pl2 Run By: Pacific Materials Laboratory 03/17/08 04:53PM

#	FS
a	**1.616**
b	1.625
c	1.638
d	1.655
e	1.661
f	1.664
g	1.674
h	1.676
i	1.677
j	1.685

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Blk Clay	1	120.0	120.0	50.0	20.0	0.00	0.0	0
CL-N	2	120.0	120.0	50.0	20.0	0.00	0.0	0



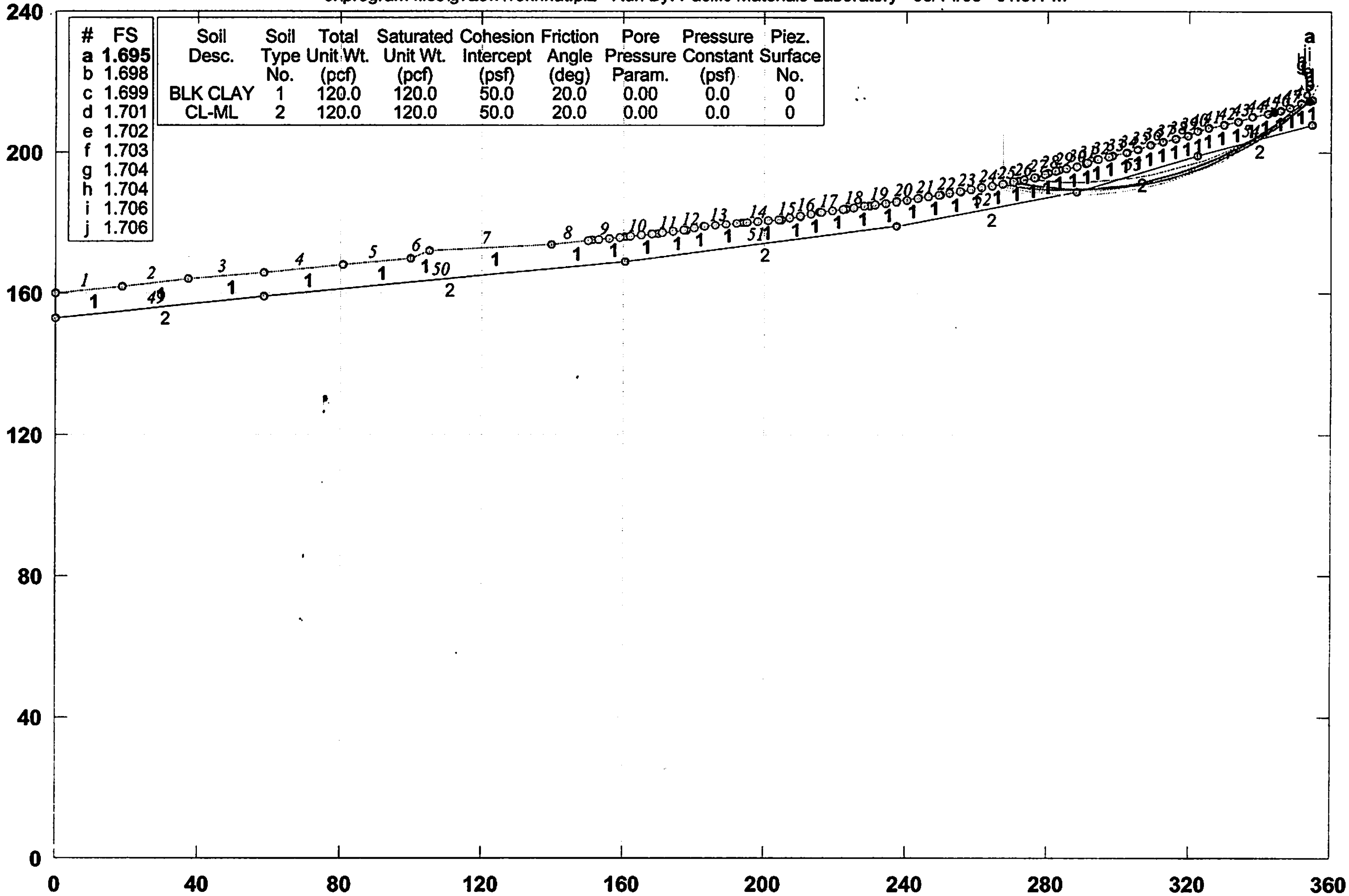
GSTABL7 v.2 FSmin=1.616

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

15,000 Calle Real North Natural Slope

c:\program files\g72sw\15knnat.pl2 Run By: Pacific Materials Laboratory 03/14/08 01:37PM

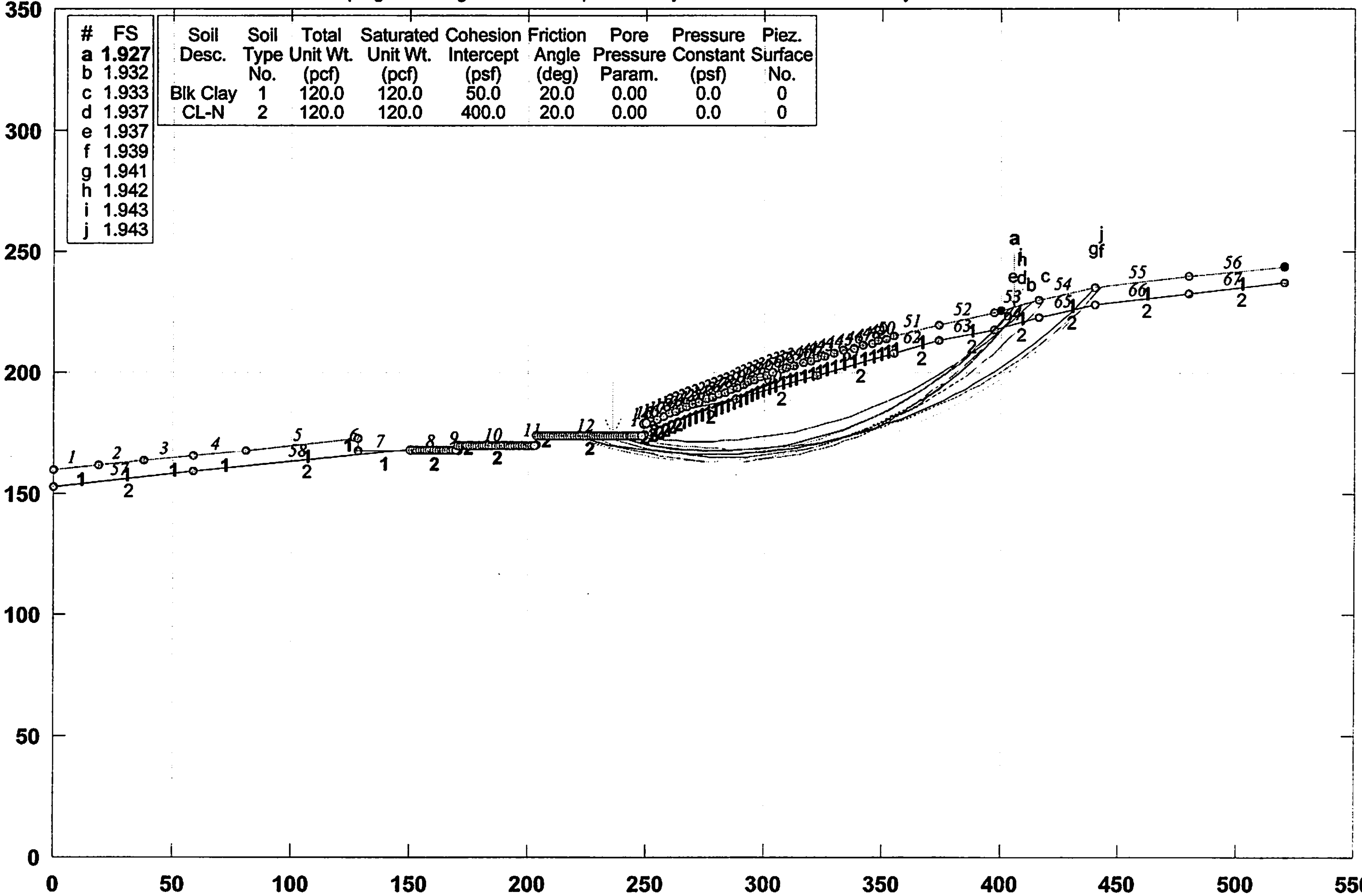


GSTABL7 v.2 FSmin=1.695
Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

15,000 Calle Real North Cut Slope

c:\program files\g72sw\15kncut.pl2 Run By: Pacific Materials Laboratory 03/13/08 04:54PM



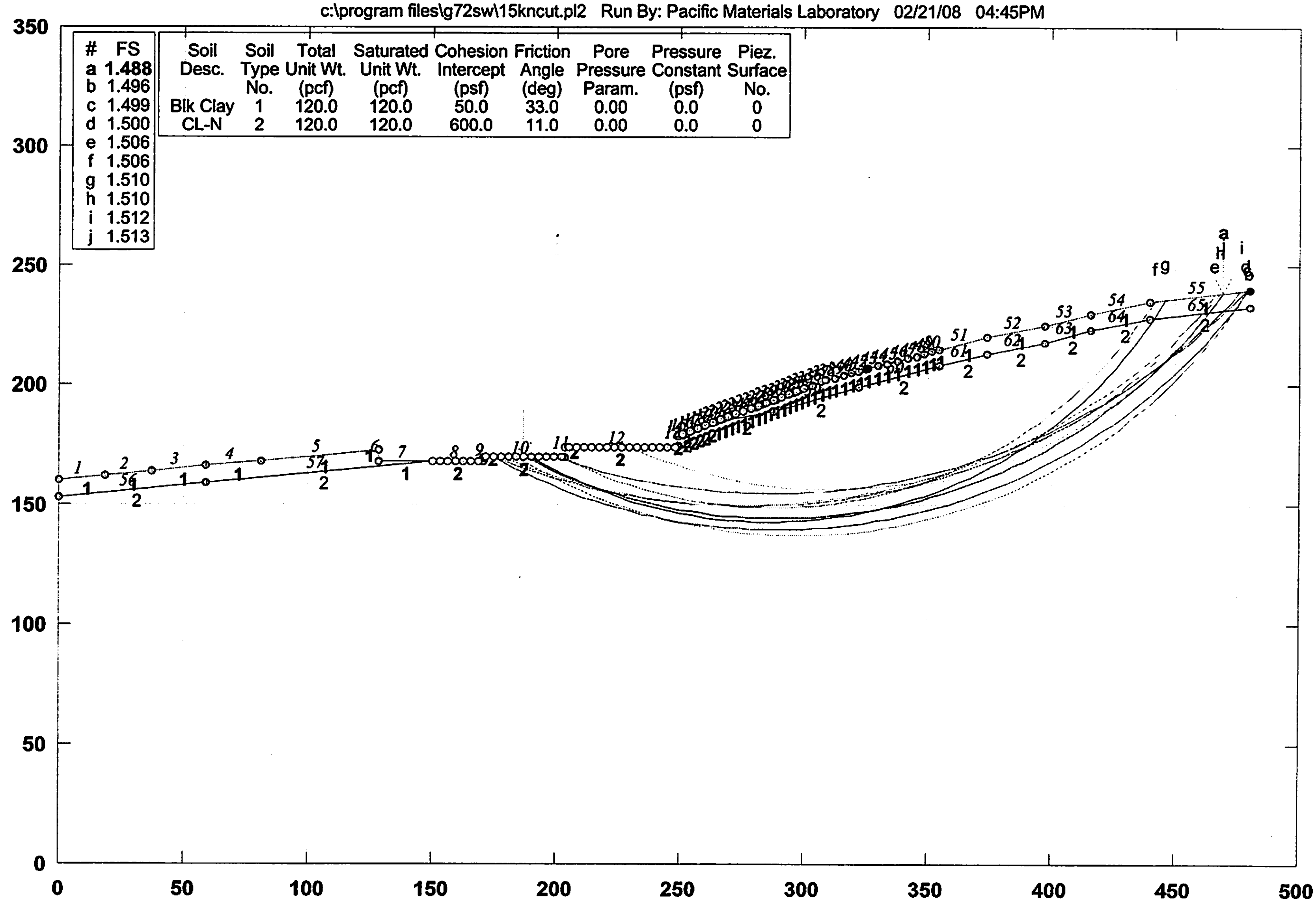
GSTABL7 v.2 FSmin=1.927

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

15,000 Calle Real North Cut Slope

c:\program files\g72sw\15kncut.pl2 Run By: Pacific Materials Laboratory 02/21/08 04:45PM



#	FS
a	**1.488**
b	1.496
c	1.499
d	1.500
e	1.506
f	1.506
g	1.510
h	1.510
i	1.512
j	1.513

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Blk Clay	1	120.0	120.0	50.0	33.0	0.00	0.0	0
CL-N	2	120.0	120.0	600.0	11.0	0.00	0.0	0

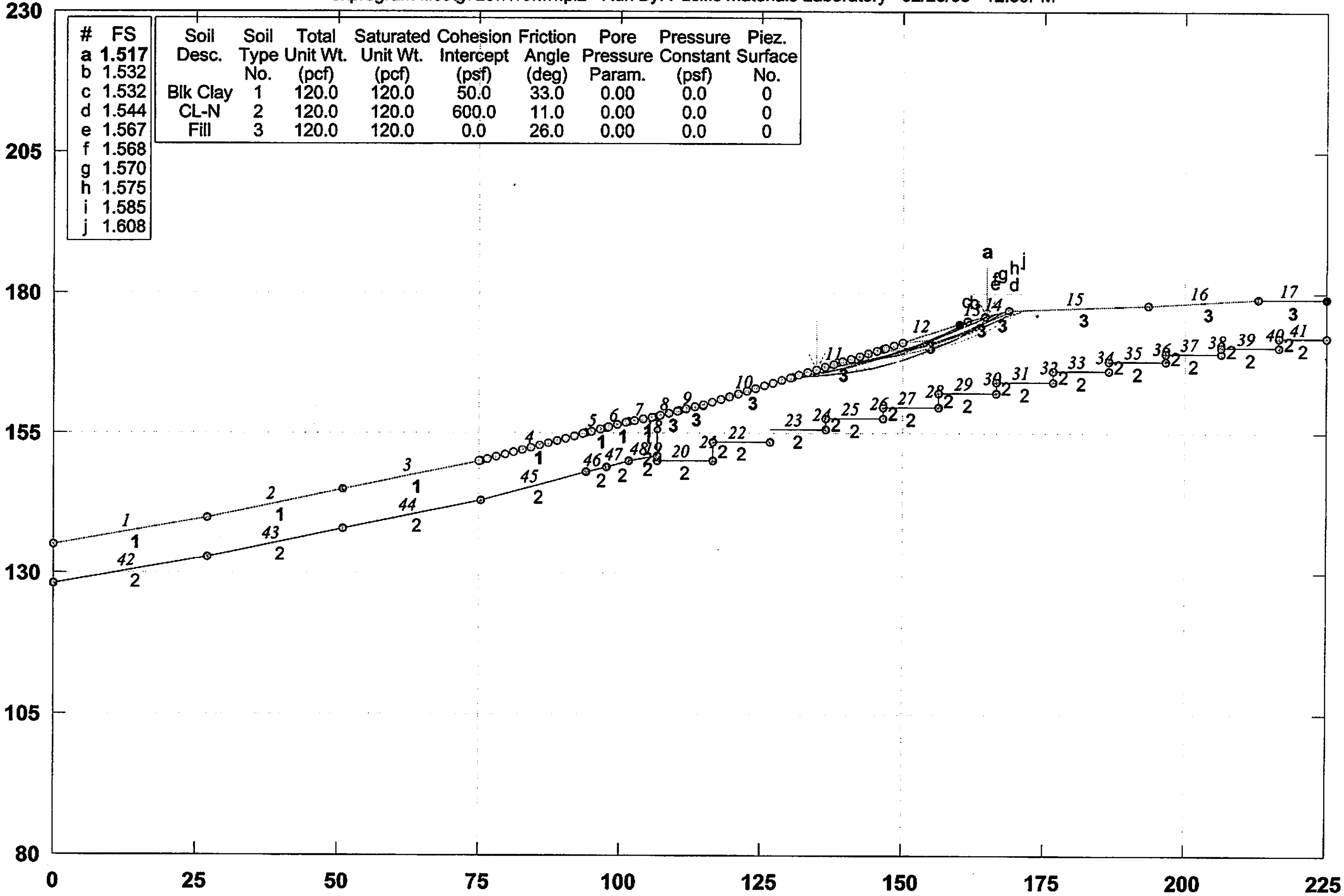
GSTABL7 v.2 FSmin=1.488

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

15,000 Calle Real West Fill Slope

c:\program files\g72sw\15kwf.pl2 Run By: Pacific Materials Laboratory 02/26/08 12:58PM



#	FS
a	**1.517**
b	1.532
c	1.532
d	1.544
e	1.567
f	1.568
g	1.570
h	1.575
i	1.585
j	1.608

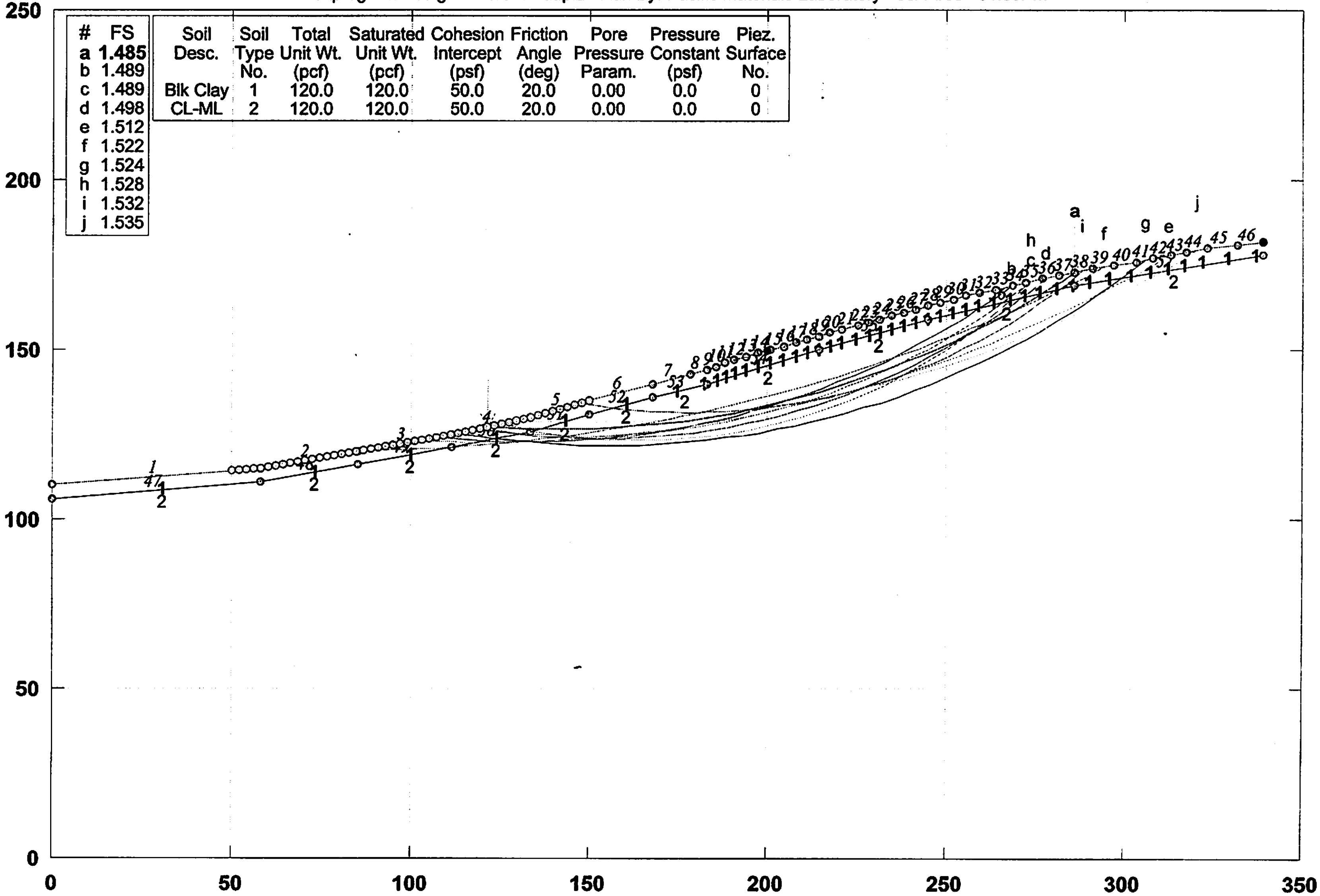
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Blk Clay	1	120.0	120.0	50.0	33.0	0.00	0.0	0
CL-N	2	120.0	120.0	600.0	11.0	0.00	0.0	0
Fill	3	120.0	120.0	0.0	26.0	0.00	0.0	0

GSTABL7 v.2 FSmin=1.517

Safety Factors Are Calculated By The Modified Bishop Method

15,000 Calle Real West Natural Slope

c:\program files\g72sw\15kwnat.pl2 Run By: Pacific Materials Laboratory 03/14/08 01:50PM



GSTABL7 v.2 FSmin=1.485

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7