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GEOTECHNICAL STUDY
PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
E. OCEAN BLVD. BETWEEN LOMA AVE. AND LINDERO AVE.
LONG BEACH, CALIFORNIA

Project No. 107104

Prepared for:

City of Long Beach
Department of Parks, Recreation, and Marine
2760 North Studebaker Road
Long Beach, California

April 30, 2010

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April 30, 2010
Project No. 107104

City of Long Beach
Department of Parks, Recreation, and Marine
2760 North Studebaker Road
Long Beach, California 90802

Attention: Sandra J. Gonzalez, Manager

Subject: Geotechnical Study
Proposed Slope Improvements
Bluff Park
E. Ocean Blvd. between Loma Ave. and Lindero Ave.
Long Beach, California

Dear Ms. Gonzalez:

Kleinfelder West, Inc. (Kleinfelder) is pleased to present this report summarizing our geotechnical study for the proposed slope improvements at Bluff Park in the City of Long Beach, California. This report summarizes the work performed, data acquired, and our findings, conclusions, and geotechnical recommendations for design and construction. The conclusions and recommendations are subject to the limitations presented in Section 6.

We understand that the City of Long Beach is planning landscape improvements for the purpose of beautifying the slope, slowing down the erosion process, and improving surficial stability. In its current condition, the Bluff Park slope is generally very steep and marginally stable in most areas. Over time, the erosion process has over steepened the slope, undermined park hardscapes, and contributed to surficial instability. Artificial fill, slump material, and colluvium that thinly blanket portions of the slope are vulnerable to downslope movements. The implementation of erosion control measures will reduce erosion and decrease the potential for surficial instability of the bluff slope.

Most of the proposed landscape improvements will not incorporate mitigation measures to improve global stability of the slope or reduce the potential for liquefaction and lateral spreading along the toe of the slope. However, global stability of the slope will be improved where the soil-nail facing is installed.

KLEINFELDER 620 West 16th Street, Unit F, Long Beach, CA 90813 p | 562.432.1696 f | 562.432.1796

We appreciate the opportunity to be of service to Planning & Development Bureau Parks, Recreation & Marine, City of Long Beach. If you have any questions regarding this report or require additional information, please do not hesitate to contact our office.

Respectfully submitted,

KLEINFELDER WEST, INC.

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Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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1 INTRODUCTION

This report presents the results of Kleinfelder's geotechnical study for the proposed slope improvements at Bluff Park in Long Beach, California. Bluff Park is located south of East Ocean Boulevard roughly between the south projections of Loma Avenue and Lindero Avenue. A map depicting the location of the site is shown on Plate 1, Site Location Map. The portion of the slope included for this study is roughly 4,300 feet long and extends from the beach stairway southeast of the Museum of Arts (Station 0+00) to the beach stairway roughly in line with Loma Avenue (Station 43+00). The purpose of this geotechnical study was to evaluate subsurface soil conditions at the site in order to provide geotechnical recommendations for the design and construction of the proposed slope improvements.

Our report includes a description of the work performed, a discussion of the geotechnical conditions observed at the site, and recommendations developed from our engineering analysis of field and laboratory data. The recommendations contained within this report are subject to the limitations presented in Section 6. An information sheet prepared by ASFE (the Geoprofessional Business Association) is also included. We recommend that all individuals utilizing this report read the limitations along with the attached ASFE document.

1.1 PROPOSED IMPROVEMENTS

We understand that the City of Long Beach (City) has retained RJM Design Group, Inc. (landscape architect) to provide recommendations for landscaping the Bluff Park slope for the purpose of beautifying the slope, slowing down the erosion process, and improving surficial stability. We understand that landscape improvements will consist of planting new drought tolerant vegetation and ground cover, and installing a new irrigation system. The undermined hardscapes will be repaired by installing posts and boards and backfilling. Geotextile fabrics will be placed on the bluff slope to reduce erosion, and localized grading will be performed. Soil-nail facings will be constructed to improve the surficial stability of steep slope segments where erosion protection measures are not feasible.

1.2 BACKGROUND INFORMATION

A review of historical, stereo-paired aerial photographs from the early 1950's through 1999 indicates that the topography of the park has stayed roughly the same except for some bluff retreat. We understand from park maintenance personnel with knowledge of the park's history that the area was used by the armed forces during the World War II. The water front originally extended to a wall at the toe of slope. It is unknown when the wall was constructed, its height or how it is supported. We also understand that the existing beach, now in front of the wall, was built by importing beach sand, possibly from dredging. The existing improvements on the bluff slopes include approximately 1,120 lineal feet of gabion walls. The gabion walls are approximately 9 to 11 feet in height and appear to be about 4 feet wide. Gabion design and construction data was not available for our review.

In early 2003, a preliminary geotechnical study was performed for the proposed Belmont Shore Bluff Restoration project by Geotechnical Professionals Inc. (GPI) under subcontract to Tetra Tech, Inc. (GPI, 2003). Their study covered bluff areas in the public domain from First Place on the west to Loma Avenue on the east. GPI drilled three borings and advanced three cone penetration tests within the Bluff Park area. They concluded that the existing bluffs have highly variable slope inclinations generally steeper than 1.5:1 (horizontal:vertical). Based on the results of their analyses, GPI concluded that "engineered slopes would either have to be inclined 2:1 or flatter or be reinforced with geogrids, soil nails or piles, in order to have an adequate factor of safety against both massive and surficial failures." At the gabion locations, GPI further concluded that the factors of safety against massive failure could be improved by installing two rows of 35-foot-long soil nails below the gabions.

The Bluff Park slope surface was reported to be uneven and eroded with many areas of shallow slump-type failures. GPI indicated that the slope could be flattened to 2:1 by either cutting the top of the slope or by extending the toe of slope to the south by filling at the toe. GPI also concluded that "if the slope remains unimproved, it will continue to erode and fail, typically in short sections." Based on our site reconnaissance, the recommended improvements by GPI were not implemented.

1.3 SCOPE OF WORK

The scope of our geotechnical study consisted of a literature review, surface geologic mapping, subsurface explorations, geotechnical laboratory testing, engineering evaluation and analysis, and preparation of this report. Studies to assess environmental hazards that may affect the soil and groundwater at the site were beyond our geotechnical scope of work. A description of our scope of services performed for the project is presented below.

Task 1 - Literature Review. Kleinfelder reviewed published and unpublished soils and geologic reports and associated data in our files and as available from selected public agencies. This included a review of fault and geologic maps prepared by the California Geological Survey (formerly known as the California Division of Mines and Geology), United States Geological Survey, and selected Seismic Hazard Zone maps. We also reviewed the reports prepared by GPI (GPI, 2003) and the readily-available historical aerial photos available from Continental Photo, Inc. Aerial photos reviewed for this project are listed in Table 1 below. The remaining documents reviewed as part of our study are presented in Section 7, References.

Table 1
Historical Aerial Photographs Reviewed

Date	Flight No.	Frame Nos.
11/17/1952	AXJ-6K	51, 52
11/19/1953	AXJ-6K	98, 99
04/03/1960	311-5	465-2, 465-3
04/03/1960	311-5	45-1, 45-2
01/31/1970	61-7	176, 177, 178
01/27/1987	F	352, 353
02/24/1999	C-134-36	156, 157

Task 2 - Geologic Mapping. Kleinfelder mapped surface distribution of the geologic units along the slope using a 40-scale map provided by RJM Design Group, Inc. (one-foot contours). The Geologic Map is presented on Plates 2A through 2E.

Task 3 - Field Exploration. The field exploration program consisted of drilling 11 borings. The borings were drilled to depths ranging from about 16.5 to 51.5 feet below the existing ground surface (bgs) using track-mounted hollow-stem auger drilling equipment. The borings were intended to supplement the data collected by GPI (GPI, 2003). The approximate locations of the borings drilled by Kleinfelder and GPI are shown on Plates 2A through 2E, Geologic Map.

Prior to commencement of the fieldwork, various geophysical techniques were used at each boring location in order to identify potential conflicts with subsurface structures. Each location was also cleared for buried utilities through Underground Service Alert (USA). A Kleinfelder engineer supervised the field operations and logged the borings. Selected bulk and drive samples were retrieved, sealed and transported to our laboratory for further evaluation. Drive samples were obtained using either a 3-inch O.D. California-type sampler or a 2-inch O.D. Standard Penetration Test (SPT) sampler. The number of blows needed to drive the samplers a total of 18 inches (in three 6-inch increments) was recorded on the field logs. The blows for the lower 12 inches of each sample were added and presented on the boring logs as the blow count for that sample. Appendix A contains a more detailed description of the field exploration program, including boring logs.

Task 4 - Laboratory Testing. Drive and bulk samples were transported to our Long Beach laboratory for geotechnical testing. Laboratory testing was conducted on selected samples to substantiate field classifications and to provide engineering parameters for geotechnical analysis. Laboratory testing consisted of in-situ moisture content and dry unit weight, Plasticity Index, wash sieve (percent passing the #200 sieve) and direct shear. The results of Kleinfelder's laboratory testing performed on samples collected during the field exploration program are presented either on the boring logs in Appendix A or in Appendix B.

Task 5 - Geotechnical Engineering Analyses. Field and laboratory data were analyzed in conjunction with the proposed improvements to develop geotechnical recommendations for design and construction. The stability of the bluff slope was evaluated, along with constructability considerations related to earthwork. Other potential geologic hazards such as strong ground shaking, fault rupture, the potential for soil liquefaction and associated settlement, and the potential for lateral spreading were also evaluated.

Task 6 - Report Preparation. This report was prepared to present our findings, conclusions and preliminary geotechnical recommendations for design and construction. The results of our field explorations and laboratory testing are presented in this report along with geotechnical recommendations for slope planting and irrigation, soil nailing, installation of post and boards, slope grading, and general earthwork requirements. The report also contains a site location map, geologic map, geologic cross-sections, log of borings, and laboratory test data.

2 SITE CONDITIONS

2.1 SITE DESCRIPTION

Bluff Park is located south of East Ocean Boulevard roughly between the south projection of Loma Avenue and Lindero Avenue. The portion of the slope included for this study is roughly 4,300 feet long and extends from the beach stairway southeast of the Museum of Arts (Station 0+00) to the beach stairway roughly in line with Loma Avenue (Station 43+00). Both the top of bluff and beach area are used for public recreation purposes including walking, running, biking, and other activities. A five- to seven-foot-wide sidewalk with a guardrail extends the entire length of the site along the top of the bluff (See Photo 1 in Appendix D). At each street intersection with East Ocean Boulevard, there are generally one or two concrete paved walkways that connect to the sidewalk along the top of the bluff. Park benches are positioned at various locations along the sidewalks and walkways. The park also contains various historic monuments or memorials. Plates 2A through 2E show the outlines of various park features, ground surface contours, and assigned slope station numbering used to describe the bluff conditions.

The portion of the bluff between East Ocean Boulevard and the top of slope is landscaped with turf and scattered large trees. Underground utilities, such as landscape irrigation, electrical power, sewer, storm drain and water, cross the park at multiple locations. The width of the landscaped area at the top of the bluff is highly variable. West of Lindero Avenue projection, the park width is about 55 to 65 feet between the East Ocean Boulevard curb and the top of bluff sidewalk. Between Lindero Avenue and Molino Avenue, the park is generally 48- to 55-feet wide, except from about Station 5+20 to 6+00, where it is reduced to about 43 feet due to an apparent previous bluff failure. The landscaped width narrows to 23 feet at Station 7+90, where the bluff slope was cut back for construction of a descending beach access stairway. Between Molino Avenue and Paloma Avenue, the landscaped area is generally 45 to 50 feet in width. Between Paloma and Coronado Avenues, the landscaped width gradually increases from 50 to 60 feet going east, except at Coronado Avenue where it is approximately 43 feet wide due to slope flattening for the beach access stairway. East of Coronado Avenue, the bluff park width increases gradually to about 75 feet near Redondo Avenue. Further east at approximate Station 43+00, the landscaped width increases to a maximum of about 200 feet at the east end of the park.

The top of the bluff is essentially flat with the surface gradient generally less than one percent toward the street. There is a gentle gradient from East Ocean Boulevard toward the bluff slope between Stations 0+50 and 3+00.

Ground elevations at the top of the bluff gradually decrease from west to east from Elevations 49 to 43 feet above mean sea level (MSL) between Stations 0+00 and 10+00. Between Stations 10+00 and 20+00, the Elevations generally vary from 43 to 46 feet MSL. Between Stations 20+00 and 30+00 the elevations predominantly range from 46 to 47 feet MSL. The elevations are mostly between 48 and 49 feet MSL from Stations 30+00 to 41+50. At the toe of the slope, on the beach, the elevations are generally between 7.0 and 8.5 feet MSL from Stations 0+00 to 10+00; between 7.4 and 9.8 feet MSL from Station 10+00 to 20+00; between 8.0 and 10.0 feet MSL from Stations 20+00 to 30+00; and between 9.0 and 10.5 feet MSL from Station 30+00 to 43+50. The corresponding slope heights average about 38 feet with a range generally between 36 and 40 feet.

The main features on the beach includes: a partially buried wall at the toe of the slope that extends generally between approximately 1 and 2.5 feet above the beach sand; beach access stairways that connect to a bike trail via concrete paved walkways; and storm drain outlets (See Photo 2 in Appendix D). The stairways are located at approximate Stations 0+00, 8+00, 19+80, 31+50 and 43+00. The storm drain outlets are located at approximate Stations 9+30, 37+10 and 43+20. There are also power poles behind the buried wall that extend from approximate Station 22+10 to the east end of the study area. An electric line is also buried in front of the wall west of Station 22+10.

The existing improvements on the slope include approximately 1,120 lineal feet of gabion walls, the stairways, and two concrete ramps. The gabion walls are located between Stations 5+00 and 10+20, Stations 11+00 and 13+20, Stations 35+70 and

37+00, and Stations 40+50 and 42+70. The walls are approximately 9 to 11 feet high and appear to be about 4 feet wide. No data was available of the design or construction of the gabion walls. However, based on observations made during site reconnaissance, the walls are likely founded on benches carved into native material and built against a steep scarp. Except for localized areas, a small wedge of relatively loose fill may have been placed behind the walls. The walls, which are slightly tilted toward the slope, do not appear to have been designed to retain significant soil mass. The gabion wall at the east end of the site, between Stations 40+50 and 42+70, is located within the upper one third of the slope. The other walls are located within the lower one third of the slope except between Station 7+50 and 8+60 where they turn and rise in elevation adjacent to the beach access stairway.

Two abandoned concrete beach access ramps are located between Stations 27+05 and 35+50. The concrete ramps are about 5 feet wide and rise in elevation longitudinally along the slope face. The ramps appear to have been cut into the slope face. Most of the inner portion of the ramps is filled with eroded material from the adjacent upper slope. Erosion has undermined portions of the outer lower edge of the ramps. A beach access stairway between the two ramps appears to have replaced the ramps.

The bluff slope face has an uneven surface and inclination due to previous failures, accumulations of slump debris, on going erosion, and past grading in localized areas. The slope has composite gradients ranging from about 4½:1 (horizontal:vertical) to near vertical (See Photo 3 in Appendix D). Generally the lower portion of the slope has a gentle gradient as compared to the middle or upper segments. The lower third of the slope generally has a gradient between 1¾:1 and 3½:1. The middle third of the slope generally has gradient ranging between ⅔:1 and 1½:1 with an average of about 1¼:1. The upper third of the slope generally has gradients ranging between ⅔:1 and 1½:1 with an average of about 1.1:1. Portions of the slope have 10 feet of higher segments with ½:1 gradient. The gabion faces are near vertical or slightly tilted toward the slope.

Landscaping on the slope is generally sparse except where gabions walls are present and where grading has occurred. Graded areas have been planted and irrigation sprinklers installed. Landscape planting consists mostly of ice plants, acacia and other low ground cover. Shrubs are present in the previously repaired upper portion of the slope between Stations 5+20 and 6+10. Isolated small- to medium-size palm trees are present, scattered on the slope face as shown in Photo 3.

In addition to the erosion on the slope, evidence of subsurface erosion (piping) was observed at the top of the slope near Station 37+25 (See Photo 4 in Appendix D). The sidewalk is being undermined at several locations, especially in the eastern portion of the bluff between Stations 32+00 and 37+00. The top of slope curb adjacent to the sidewalk has dropped in several areas and in other areas the sidewalk is cantilevered by up to one foot.

2.2 SURFACE DRAINAGE

Surface water drainage at the top of the bluff appears to be mostly via sheet flow towards East Ocean Boulevard. There may be also some localized area drains. There is no brow ditch or berm at the top of the slope or terrace drain on the slope descending to the beach.

3 GEOLOGY AND SEISMICITY

3.1 REGIONAL GEOLOGY

The site is located near the southern margin of the Los Angeles Basin, in the Peninsular Ranges physiographic and tectonic province. The province is characterized by a prevailing northwesterly orientation of geologic features, such as the Whittier, Newport-Inglewood and Palos Verdes faults. The Los Angeles Basin itself is a northwest-trending lowland plain approximately 50 miles long and 20 miles wide. The Basin is bounded by the Santa Monica Mountains to the north; the Puente Hills, Peralta Hills, and San Joaquin Hills to the east and southeast; and the Pacific Ocean to the southwest. The lowland surface of the Los Angeles Basin slopes gently southwestward and westward toward the Pacific Ocean. This sloping plain is interrupted by the Newport-Inglewood structural zone, which extends from Newport Mesa northwesterly towards Beverly Hills. This line of low discontinuous hills and mesas are the surface expression of the Newport-Inglewood fault zone.

The Los Angeles Basin began to form about 7 million years ago, in late Miocene time, as the San Andreas Fault shifted eastward to its present position. For the next 5 million years, the basin subsided along major faults including the Newport-Inglewood fault zone and was rapidly filled with sediments eroded from highlands to the north, northeast, and east. Between about 2 and 5 million years ago, the Newport-Inglewood fault zone began its present right-lateral movement. In the Pleistocene epoch (the last 1.6 million years), the region was warped gently upward forming the present shoreline and topography which includes the area of Bluff Park in Long Beach (Yerkes et al., 1965; Wright, 1991).

3.2 SUBSURFACE CONDITIONS

3.2.1 General

According to a review of available published geologic maps, the area is underlain by Pleistocene Old Paralic Deposits, which is a deposit formed in a shallow marine border sand have dry unit weights ranging between 97 and 125 pcf (average of about 105 pcf).

3.3 GROUNDWATER CONDITIONS

Groundwater levels are near sea level and appear to be influenced by the tides. Groundwater was encountered in all the borings drilled. The approximated depths and elevations of the groundwater levels encountered are presented in Table 2.

Table 2
Groundwater Level Measurements

Boring No.	Station	Approximate Groundwater Depth (feet)	Approximate Groundwater Elevation (feet)	Date Measured
KB-1	4+40	40	5	11/5/09
KB-2	10+70	40	5	11/4/09
KB-3	20+50	40	6	11/4/09
KB-4	25+25	42	4	11/17/09
KB-5	37+55	43	6	11/17/09
KB-6	37+55	6	4	11/17/09
KB-7	34+10	7	3	11/5/09
KB-8	25+25	5	4	11/5/09
KB-9	20+70	4	7	11/5/09
KB-10	10+70	4	4	11/5/09
KB-11	4+40	3	5	11/5/09

The groundwater elevations generally range between +3 and +7 feet above mean sea level. The tide elevations in this area generally range between -1 and +6.5 feet. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture should be anticipated during and following the rainy season or periods of locally intense rainfall.

3.4 GEOLOGIC HAZARDS

3.4.1 General

Due to the steepness of the topography of the bluff slope and its location within an area of high seismicity, the principal geologic hazards for the project consist of the potential for strong groundshaking and landslides induced by earthquakes occurring along one or more of the regional active faults. Other geologic hazards considered for this site include earthquake-induced liquefaction, lateral spreading, and tsunamis. A discussion of each geologic hazard follows.

3.4.2 Faulting

Earthquakes and faulting occurs as the tectonic plates, which comprise the Earth's crust, move relative to one-another. Faults identified by the State as being active are not known to be present at the surface within the project limits. Bluff Park is not located within a State of California-Special Studies Zone, formerly Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). The two known closest faults are the Newport-Inglewood fault zone and the Palos Verdes fault zone located approximately 1.6 miles and 5.7 miles respectively at the closest points to the site. Because of the distance to known active faults, it is our opinion that the risk of surface rupture resulting from faulting can be considered low.

3.4.3 Flooding

Flooding can occur as a result of several factors in developed areas. These factors include rainfall rates that exceed an area's ability to absorb or control the runoff; impounded water retained behind a flood control structure (upstream-inundation); failure of a flood control structure (downstream-inundation); seiche; and tsunami.

The top of the bluff may be subject to water ponding during heavy rain due to the flatness of the grade. Because the site is situated along the ocean shore within an area of known high seismicity, the site is considered at risk from tsunami (CGS, 2009).

3.4.4 Landslides

Landslides and other forms of mass wasting, including mud flows, debris flows, and soil slips occur as soil moves downslope under the influence of gravity. Landslides are frequently triggered by intense rainfall or seismic shaking. The site has a high potential for landslides.

3.4.5 Liquefaction

Liquefaction can occur when saturated loose, coarse-grained or silty soils are subjected to strong shaking resulting from earthquake motions. The coarse-grained or silty soils typically lose a portion or all of their shear strength, and regain strength sometime after the shaking stops when pore pressures dissipate. Soil movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils.

The beach portion of the site is located within a liquefaction potential hazard zone as identified by the State (CDMG, 1998). Based on the results of our field investigation, subsurface conditions at the toe of the slope included loose to medium dense sandy soils below the water table. Therefore, we judge that the risk of liquefaction at the toe of the slope during a design level earthquake is high. In addition, there is a moderate risk of lateral spreading of the toe of the slope towards the beach during a liquefaction event. A discussion regarding the anticipated magnitude of liquefaction-induced settlement and implications to the project is presented in Section 4.2.3.

3.4.6 Seismicity

The site is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. We have performed a computer-aided search of the known active and potentially active faults within a 62-mile (100-kilometer) radius of the site and researched available literature to assess the expected maximum magnitude earthquake to be generated on each fault. Table 3, Significant Faults, summarizes these parameters for three of the plus thirty known active and potentially active faults within the searched radius of the site that, in our opinion, may have the greatest impact upon the site. Table 3 was generated using, in part, the EQFAULT computer program (Blake, 2000) as modified using the fault parameters from CGS Open File Report 96-08 revised June 2003. This table does not identify the probability of reactivation or the onsite effects from earthquakes occurring on these listed faults or any of the other faults in the region.

Table 3
Significant Faults

Fault Name	Approx. Distance from Site mi (km)	Maximum Event (Moment Mag.)
Newport-Inglewood	1.6 (2.6)	7.1
Palos Verdes	5.7 (9.1)	7.3
San Andreas	50 (81)	8.0

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our data review, our field exploration, laboratory testing, and geotechnical engineering analyses, the potential for slope failures can be reduced by decreasing the slope inclination, constructing retaining walls, anchoring/nailing the steep portions of the slope, and protecting the slope from erosion. The slope conditions vary and there is no unique method that may be cost effective for all locations of the approximately 4,300-foot long bluff. Consideration should be given to improve surface drainage and erosion mitigation, reducing slope inclination or reinforcing the slopes. The following sections present our analyses and recommendations to improve the bluff slope conditions.

4.2 SEISMIC DESIGN CONSIDERATIONS

4.2.1 General

The site is located in a seismically active region of southern California, and can therefore be expected to experience moderate to strong seismic shaking. Potential seismic hazards include ground shaking, liquefaction, seismic settlement, and lateral spreading. The following sections discuss these potential seismic hazards with respect to this site.

4.2.2 Liquefaction

To assess the potential for liquefaction of subsurface soils at the site, we used the simplified liquefaction analysis procedure recommended by NCEER (Youd and Idriss, 1997, 2001). For estimating the resulting ground settlements, we used the method proposed by Tokimatsu and Seed (1987). This method utilizes the standard penetration test (SPT) blow count data to estimate the amount of volumetric compaction or settlement during an earthquake.

During the current subsurface exploration, groundwater was encountered in the borings drilled at the toe of slope at depths ranging from about 3 to 7 feet below ground surface. We used the approximate water depth encountered in the borings at the time of drilling to evaluate liquefaction potential at the toe of slope.

According to Section 1802 of the 2007 CBC, the PGA for use in the liquefaction analyses may be obtained by dividing the design response acceleration at short period (S_{DS}) by 2.5. A PGA of 0.47g with an associated Magnitude 7.1 earthquake was used as the design-level seismic event for the liquefaction analyses.

Based on the boring data and our engineering analyses, it is our opinion that the loose to medium dense sandy silt, silty sand, and sand below the groundwater could be subject to liquefaction in the event of a major earthquake occurring on a nearby fault. Based on our analyses with the available data, we estimate that the seismically-induced settlement of saturated granular soils due to strong ground shaking during the design-level seismic event may range from less than ½ inch to approximately 6 inches or more. Table 4 summarizes the estimated liquefaction-induced settlements.

Table 4
Estimated Liquefaction-Induced Settlement

Boring No/ CPT No.	Approximate Station	Assumed Groundwater Depth (ft)	Approximate Liquefaction-Settlement (in)
KB-6	37+55	6.5	0.4
KB-7	134+10	6.5	<0.4
KB-8	25+25	5.0	0.4
KB-9*	20+70	4.0	5.6*
KB-10	10+70	4.0	0.9
KB-11	4+40	3.0	1.4

* Boring drilled on the north side of the toe of slope wall (behind the wall)

4.2.3 Lateral Spreading

Lateral spreading most often occurs on gently sloping ground, but can also occur on flat-lying terrain adjacent to a free face where an underlying layer liquefies in response to earthquake ground motions. During liquefaction, the earth material may spread, or move laterally on continuous liquefied layers of loose, saturated gravel, sand or silt. This type of failure may occur in floodplains, river channel alluvium and in artificial fill on slopes as gentle as 0.3 degree (approximately 0.5%), and can produce displacements ranging from a few inches to tens of feet. Materials may range in composition from clay and silt to fine-grained sand and may include sandy gravel (CGS, 2008). Based on the sampling blow count of the borings drilled, lateral spreading in front of the wall due to liquefaction does not appear to be a major consideration. However, Boring KB-9 drilled behind the buried wall indicated very low blow counts. Because it is unknown when the wall was constructed, its height and how it is supported, we were unable to determine if the buried wall would provide sufficient restraint to prevent lateral spreading during a seismic event. The dimensions shown on the cross sections are for illustration purpose only.

4.3 SLOPE STABILITY ANALYSIS

4.3.1 General

Except where previous grading occurred, the slopes within the subject site are generally underlain by Old Paralic deposits with thin mantles of fill and colluvium in localized areas. The heights range between about 36 and 40 feet. Several geotechnical cross sections, Plates 3A through 3S were prepared, and selected cross sections were used to evaluate the stability of the bluff slope.

4.3.2 Method of Slope Stability Analysis

The stability of selected slopes was analyzed using the computer programs PC Stabl 5 from Purdue University, GStabl 7 from Gregory Geotechnical Software and Slide 5.0 developed by Rocscience. The limit equilibrium analysis programs model a two-dimensional slope and compute a factor of safety for various failure planes. The calculated factor of safety is the ratio of the available resistance to sliding divided by the driving forces. The higher the factor of safety, the more stable the slope. Typically, a slope with a static factor of safety greater than 1.5 indicates a relatively stable slope. Calculated factors of safety near 1.0 to 1.1 indicate that the slope may be on the verge of failure. Calculated factors of safety below 1.0 indicate that the slope analyzed would likely be unstable. For earthquake loads, slope stability methods often yield factors of safety less than 1.0, indicating slope "failure." However, the amount of downward and lateral slope movement during the earthquake will depend on the severity of the earthquake and the static factor of safety of the slope and is beyond the scope of this report.

The computer programs were used to search for critical failure plane surfaces for static and pseudo-static conditions, using the Janbu corrected method, simplified Bishop method, and/or Spencer method. The surface topography of the slopes modeled was obtained from the Geologic Map, Plates 2A through 2E. The shear strength parameters used in the analyses included the cohesion and internal friction angle. The groundwater level or phreatic surface was taken near the depths encountered in the borings. The methodology for selecting these strength parameters is described in Appendix C.

4.3.3 Seismic Slope Stability Screening Analysis

A screening evaluation for the seismic slope stability was performed in accordance with the Section 11.2 of the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California" (SCEC, 2002) and the recently updated SP 117A (CGS, 2008).

Input for the screening procedure include the maximum horizontal acceleration at the site for a soft rock condition (MHA_r) corresponding to a seismic hazard level with a return period of 475 years or 10% of probability of exceedance in 50 years, the mode earthquake magnitude (M) and distance (r) associated with this maximum horizontal acceleration, and the allowable slope displacement. Based on the CGS web site, site acceleration for a return period of 475 years, MHA_r, is estimated to be approximately 0.39g. The associated earthquake magnitude is 7.1 and the distance is less than 10 km. For an allowable slope displacement of about 6 inches, the estimated seismic coefficient is approximately 0.16g, based on the procedures outlined in SP117A.

Using the above seismic coefficient, the pseudo-static safety factor in most cases is less than 1.0. Accordingly, the site does not pass the screening analyses and, therefore, in general is considered unstable under seismic conditions.

4.3.4 Results of Slope Stability Analysis

The slope profiles selected for stability analysis were based on slope height, topographic features and subsurface conditions. The results of the analyses performed are summarized in Table 5. Detailed outputs are presented in Appendix C.

Table 5
Summary of Slope Stability Analysis

Cross Section	Station	Calculated Safety Factor		Depth of Failure Plane on Slope (ft)*	
		Static	Pseudo Static	Top	Middle
1	1+34	1.1	0.9	5	9
2	4+38	1.1	0.9	4	3
5	10+68	1.2	1.0	NA	5 - 9
5 (loss of cementation)	10+68	1.0	<0.9	NA	5 - 9
16	34+11	1.4	1.1	4 - 8	9

*For static analysis with the lowest calculated safety factor

Both Cross Sections 1 and 2 have steep upper slope portions and the potential failure planes pass through the upper portion of the slope. Cross Section 5 includes a gabion wall and the potential failure planes pass below the wall. These three sections have calculated safety factors of about 1.1 to 1.2, which indicate inadequate calculated safety factor under static conditions and during the seismic design event. By geometric comparison, the other sections likely also have low safety factors. Additional analyses were performed on Cross Section 5 to evaluate the effect of loss of cementation of the soil. Based on the analysis, the loss of soil cementation could cause slope instability.

Cross Section 16 has an overall gradient of 1½:1 and is relatively uniform across the entire profile. It contains a concrete ramp that may have helped the stability of the slope by providing a surface drainage path for water. It is the only profile analyzed with calculated safety factors that approaches code requirements.

4.3.5 Surficial Stability and Surface Erosion

The soil profile in several portions of the slope contains clay layers underlain by silty sand or sand layers. The sands are generally fine-grained, uniform and are weakly cemented. The cohesion of the clay and cementation of the sand allow the formation of relatively steep slope segments. However, water can dissolve the cementation of the sand and wash the grains downslope. Water from irrigation, rain, or other sources initiates the process of dissolving the cementation and contributes to erosion. When the slope face becomes excessively steep, slumps occur as mapped at many locations along the slope face.

The erosion process over time steepens the slope and contributes to surficial instability. Artificial fill, landslide deposits, and colluvium that thinly blanket portions of the slope and do not have significant cementation are vulnerable to downslope movements. However, implementation of erosion control measures will reduce erosion and decrease the potential for surficial instability of the bluff slope.

4.4 EROSION CONTROL MEASURES

Surface erosion can be reduced by preventing water access or providing vegetation that will hold the soil particles in place. A polymer is typically sprayed on bare slopes before the rainy season to reduce surface erosion until there is sufficient plant growth. The existing gabions appear to have been successful in reducing surface erosion; however, because of the low slope stability safety factors calculated, they are not recommended.

4.4.1 Slope Planting

Portions of the slope may be planted with deep-rooted drought-resistant vegetation to better control erosion. To reduce the added weight on the slope, shrub heights should be limited and large trees should be avoided. Because of slope stability conditions and softening of the cemented material due to planting and irrigation, we recommend shrub planting only for portions of the slope that are no steeper than 1½:1. For steeper portions of slope, we recommend ground cover (only sparse shrubs may be used in these areas if necessary). Ground cover should be a light-weight vegetation that can shed water (e.g., reduce water penetration below the surface). Ground cover on steep slope segments will need to be held in place with a permanent erosion fabric, as described below. We recommend that the existing ivy ground cover be removed from the areas to be improved. Plants that are heavy due to water retention (e.g., tree aloe) should not be used on slopes steeper than 2½:1 (horizontal:vertical). The planting pattern for all shrubs should be triangular. Table 6 presents the recommended planting spacing derived from modeling soil disturbance due to planting and irrigation and water pressure buildup for various slope inclinations.

Table 6
Recommended Shrub Spacing

Slope Inclination (horizontal:vertical)	Minimum Spacing (feet)	Erosion Fabric
2½:1	4	Jute Mesh
2:1 to 2½:1	6	Jute Mesh
1½:1 to 2:1	8	Reinforcement Mat
> 1½:1	Ground Cover Only	Reinforcement Mat

We understand that the City has a planting setback requirement of 2 feet from sidewalks. In addition, we recommend a setback for shrubs of at least 4½ feet from the top and bottom of soil-nail facing, from the wall at the toe of the slope, and from other structures on the slope, except the gabion walls. For the gabion walls, shrubs should be setback at least 6 feet from the toe and 10 feet at the top of the walls, respectively. Shrubs and ground cover selected should require very low maintenance because of difficulties accessing steeper portions of the slope with trimming equipment. Poorly maintained vegetation could be detrimental to the slope.

Permanent erosion fabrics should be anchored at the top of the slope and stapled to the slope face, as recommended by the manufacturer. The manufacturer recommendations should take into account the specific site and soil conditions. The landscape architect should show on the drawings the proposed anchoring at the top of the slope and fabric stapling pattern on the slope. Staples and anchors to restrain the fabric should be galvanized. Because soil will lose strength as moisture penetrates and cementation dissolves, standard stapling may not be sufficient. Deeper stapling may be required in very steep portions of the slope to hold the fabric in place. Permanent erosion fabrics should not deteriorate significantly with time and should have a tensile strength of at least 1,000 pounds per linear foot and be strong enough to be held from the top of the slope when nails creep down the slope due to ground softening. P550 Permanent Turf Reinforcement Mat by North American Green and Pyramat High Performance Turf Reinforcement Mat (HPTRM) have the tensile strength desired. Other considerations for erosion fabric selection include, but are not limited to, thickness, open volume, porosity, light penetration, and anchoring method. A landscape architect should evaluate various fabrics and determine their suitability based on the proposed vegetation requirements and the capability of the fabric to control erosion under the site conditions. A combination of various fabrics may be used, if necessary.

For the purpose of fabric anchoring at the top of the slope, we recommend installing a curb at the top of the slope located against the sidewalk (some areas already have a curb as shown in Photo 1). The curb should be held in place by concrete posts and should be designed to resist minimum horizontal and vertical loads of 500 pounds per linear foot. Posts may be omitted and the erosion fabric may be anchored to the edge of the sidewalks in addition to "spikes," as recommended by the manufacturer, where there is insufficient space between the top of the soil-nail facing and sidewalk and only ground cover is used. Recommendations for the design of posts are presented in Section 4.5 of this report. Plate 4A, Erosion Control Concept, presents a conceptual design.

Grading will be required to contour (round up) very sharp and uneven surfaces prior to installing the fabric. Fabric needs to fit tightly against the slope face since void space below the fabric will allow erosion and result in failure. Additional fabric staples or longer staples may be required in depression areas. Where the slope is 2:1 (horizontal:vertical) or flatter, depressions may be filled provided the soil is properly compacted. For slopes steeper than 2:1, we do not recommend filling depressions that are deeper than 4 inches due to the difficulty of obtaining compaction. Slope contouring through cutting should be performed instead.. Prior to placement of fabric, representatives from the geotechnical engineer and manufacturer should verify there is no loose soil on the slope face and that the slope has been contoured to obtain a tight fit against the soil. Loose material should be removed from steep segments of the slope, as well as the colluvium that has accumulated over the wall at the toe of the slope, prior to placement of erosion fabrics. Grading removals will vary from locations to locations. For cost estimate purposes, we suggest an average removal depth of one foot (this is in addition to grubbing). At the time of the bid, we recommend that all bidders make their own assessment of removal depths required and amount of soils that will need to be exported off site.

Fill placed within planting areas on the slope should be compacted to at least 92 percent relative compaction as determined by ASTM D1557, except for the upper one foot which may be compacted to at least 85 percent relative compaction. We understand the one gallon shrubs will require an excavation of about one foot diameter and one foot deep. Ground disturbance should be not allowed outside the planting holes. To reduce ground disturbance, erosion fabric should be installed on slopes with an inclination of 2:1 or steeper prior to shrub planting. Excess soil from the shrub excavations should be removed from the slope area. Erosion fabrics should be protected from damage during planting and installation of the irrigation system. The installation of the fabric should be inspected by the manufacturer's representative upon completion of planting and installation of the irrigation system. Any damaged fabric or loose or missing nails should be replaced.

4.4.2 Slope Irrigation

Proper design of the irrigation system is crucial for the slope performance. We understand that the partial irrigation system presently at the site has been turned off in some locations due to pipe breaks caused by soil instability. Excessive irrigation will weaken the underlying soils and exacerbate erosion and slope failure. Irrigation should consist of the minimum amount of water required to maintain plant life and should be uniform so that soil moisture variation is reduced. Irrigation piping should be placed at the ground surface and should be designed and tied in place in anticipation of soil movement. We understand that the main water line for the new system may be buried;. However, it should be located within two foot of the wall at the toe of slope. The system should have automatic shut offs in the event of rain, pipe leak, or sprinkler damage. Irrigation heads should be staggered. The irrigation system for the upper two-thirds of the slope should be located at the top of the slope, and placed on the ground surface where it can be safely accessed for maintenance. The lower portion of the slope may be irrigated from the bottom where irrigation piping can be accessed for maintenance. Irrigation on the steep portions of the slope should consist of a low flow or light mist system to prevent water from flowing and causing erosion on the slope, soaking the flatter and lower portions of the slope, and creating slide prone areas. Our analysis and recommendations in this report assume the irrigation system will be permanently turned off once the plants are established (approximately one year), as indicated by the City.

4.4.3 Sidewalk Undermining

The sidewalk at the top of the slope has been undermined in several locations, as shown in Photo 4. The most prevalently undermined areas are located in the vicinity of Stations 3+80 to 4+00, 5+10 to 5+20, 10+45 to 10+50, 14+60 to 15+20, 18+30 to 18+50, 22+10 to 22+20, 26+65 to 26+85, 29+20 to 29+35, 30+55 to 30+65, 32+15 to 32+25, 34+15 to 34+30, 34+35 to 34+65, 34+75 to 34+95 and 35+25 to 36+10. We recommend that the existing undermined areas be repaired using posts and boards, or other equivalent methods, and further undermining be prevented or the sidewalk be relocated. We recommend that posts in conjunction with timber boards, as described under Section 4.5, be used to repair these areas. Following staking, the stationing for the undermined areas to be repaired should be verified jointly by the Geotechnical Engineer and the Landscape Architect prior to construction.

An alternative to posts and boards would be to remove the sidewalk, overexcavate and level the eroded areas, and backfill using soil reinforced with geogrids. The reinforced soil zones should be used in combination with Verdura wall blocks (or equivalent) as facing to prevent further soil loss from under the sidewalk. Permanent erosion fabrics could be installed prior to recasting the sidewalk, eliminating the need for anchoring with posts or nails. The City should determine where this alternative can be used since it requires removal and replacement of the sidewalks, which temporarily affect the use of the park.

4.4.4 Drainage

The erosion along the edge or under the sidewalk appears to have been caused by water flowing under or over the sidewalk. We recommend that the area be surveyed to verify that drainage at all locations is toward the street. Where there is insufficient gradient to properly drain water to the street, we recommend that the civil engineer or landscape architect design area drains with oulets to the street or to the storm drain.

4.5 POSTS AND TIMBER BOARDS

Posts and timber boards may be used to repair existing undermined areas and prevent further undermining of the sidewalks. Posts should be located below the curb immediately adjacent to the sidewalk and timber boards bolted to the outside of the posts in order to facilitate backfilling below sidewalks. Boards should be at least three inches thick, twelve inches wide, and pressure treated to prevent deterioration. For design purposes, a minimum soil height of two feet behind the boards should be assumed. In localized areas, the soil height will be greater than two feet and the resulting increase in lateral loads should be accounted for in the design. The erosion fabric may be attached to the boards or a beam may be attached to the side of the posts. If boards are used to anchor the erosion fabric, additional lateral and vertical loads should be accounted for by the structural engineer for the design of the posts and the boards. Plate 4B, Post and Board Concept, presents a conceptual design.

Once the post and boards are in place, the undermined sidewalk will need to be backfilled. The lower portion of the backfill should consist of sand to allow drainageThe sand should be compacted per the project specifications. For the upper portion of the backfill that cannot be compacted below the sidewalk, a sand-cement slurry or lean concrete or approved equivalent should be used.

For the purpose of anchoring fabric at the top of the slope, we recommend installing a curb at the top located against the sidewalk (some areas already have a curb, as shown in Photo 1). The curb should be held in place by concrete posts drilled below the curb. Posts should have a diameter of at least 14 inches and an embedment depth of at least 15 feet, except for areas with soil-nail facing. Where soil-nail facing is constructed, the post diameter should be at least 12 inches with an embedment depth of at least 5 feet below the top of the facing. Where there is insufficient space between the top of the soil-nail facing and sidewalk for shrub planting and only ground cover is used, the posts may be omitted and the erosion fabric may be anchored to the edge of the sidewalks in addition to the nailing recommended by the manufacturer. Horizontal spacing of the posts may range from about 10 to 15 feet, depending on the curb and post design. The curb should be designed to resist minimum horizontal and vertical loads of 500 pounds per linear foot. The contractor should select a construction procedure to the build the posts without damaging the sidewalk. Based on past experience, we anticipate that post holes could be constructed without encroaching on the sidewalk using an auger attached to a rubber-track excavator. Plate 4A, Erosion Control Concept, presents a conceptual design. A structural engineer should design the posts and determine their maximum spacing and minimum embedment depth.

Resistance to lateral loads for posts can be derived from passive resistance acting on the faces of the foundation elements oriented perpendicular to the direction of loading, provided these foundation elements are poured against undisturbed material or properly compacted fill. For the posts along the slope face, passive resistance may be assumed to be equal to the pressure developed by a fluid with a density of 200 pcf (this value includes the doubling effect for the California Building Code Pole Formula). For the portion of the post behind and below the top of the soil-nail facing, the equivalent fluid pressure may be increased to 300 pcf. Allowable passive earth pressure values should not exceed 2,000 psf. Due to slope creep and landscaping, we recommend that the first five feet of soil cover be neglected in the passive resistance calculations for the posts.

4.6 SOIL-NAIL SYSTEM

4.6.1 General

Overall slope stability can be improved by buttressing the slope with a geogrid reinforced fill, flattening the slope by cut and fill grading, constructing retaining walls, or constructing a soil-nail facing system. Based on discussions with City, we understand that extensive slope grading would not be considered for this project. Grading is not a viable option because the California Coastal Commission will not allow placement of new fill on the beach. We also understand that the City will not consider cutting the top of the slope back to reduce slope inclination, which will reduce the usable flat space at the top of Bluff Park. Past slumping of the slope and bluff recession have already reduced the usable space in Bluff Park. The use of concrete retaining walls has been considered in the past; however, due to cost and aesthetic reasons, the City prefers not to use retaining walls. We understand that the City has selected a soil-nail system with a shotcrete facing to improve slope stability in areas with slope inclinations near 1:1 or steeper since erosion protection may be extremely difficult or impractical to construct and maintain.

A soil-nail system consists of reinforcing the existing ground by installing closely spaced steel bars to passively hold the soil mass together. Soil nails create a coherent gravity mass, in which stability does not depend only on the facing. A soil-nail system typically includes drilled holes in which threaded bars are placed and grouted under gravity. The bars are connected to bearing plates that distribute the force at the nail end to a shotcrete facing and the ground behind the facing. Vertical geocomposite strip drains are installed between the shotcrete and the excavation face to prevent water pressure build up behind the facing. The drainage system also includes weepholes and a toe drain to convey water away from the shotcrete facing. Permanent facing of a soil-nail system usually consists of shotcrete, but can also consist of cast-in-place concrete or prefabricated concrete panels.

The soil-nail facing can be designed to conform to the slope configuration, if necessary. To improve the aesthetic of the finished surface, the finished shotcrete can be colored with pigmented sealers, thickened, and sculpted to match the surrounding landscape while the concrete is fresh. The facing surface can also be stained for a natural look. Pockets can be sculpted on the facing for planting purpose or a step facing constructed. The landscape architect should provide recommendations for the finished facing configuration.

The design and construction of the soil nail stabilization system should be performed in accordance with the recommendations presented in Report FHWA0-IF-03-017, Geotechnical Engineering Circular No.7 (FHWA, 2003). We recommend that a registered engineer qualified and experienced in the structural design of soil-nail systems be retained to design the facing or that the design be performed by a design-build contractor. The contractor should provide proof of adequate experience with soil nail systems. Prior to bidding the contractor should become familiar with the site and subsurface conditions.

4.6.2 Areas for Soil-Nail Reinforcement

Because of the slope length and height, we understand that funding is not available at this time to improve the entire project slope. We understand that some funding will be allocated for installation of erosion control measures, slope planting, and irrigation. Therefore, we have prioritized areas that have the most pressing need for remediation based on our field observations, review of the existing topography, and soil conditions. The area between Stations 0+40 and 5+00 across from the parking lot and between Stations 14+50 and 18+50 should be remediated first. As shown on Cross Sections 1, 2, 7, and 8 these slopes are generally steeper than 1:1. Portions of the slope between Stations 13+80 and 14+50; Stations 18+50 and 19+15 (Sections 4 and 5); Stations 20+65 and 27+00 (Sections 10, 11, 12, 13, and 14); and Stations 37+20 and 40+30 (Sections 17 and 18) should be considered next for improvement. After improvements are completed in these areas, soil-nail facing to improve the stability of gabion walls should be considered the next priority. The attached Geologic Map, Plates 2A through 2E, show the stationing and cross section locations. The existing and proposed configurations of the slope are shown on the cross sections. With few exceptions, it is proposed to trim most of the areas with slope inclinations of ¾:1 to 1:1. If necessary, the areas can be further divided in shorter segments based on field observation. The slope should be surveyed prior to finalizing the location of the soil-nail facing.

4.6.3 Soil-Nail Stability

Analyses were performed on three cross sections that exhibit slightly different soil profiles and relatively steep slope segments. Cross sections 1, 2, and 7 were selected for analyses. These sections have steep upper slope portions with relatively stiff and slightly cemented materials and relatively flat lower portions with loose colluvium deposits at the surface. For the purpose of our analysis, we have assumed that the surface for Sections 1, 2, and 7 will be trimmed at an inclination of about ¾:1 within the upper portion of the slope and no steeper than an inclination of 2½:1 near the base of the slope.

The global stability of selected cross sections with nails was analyzed using the soil reinforcement computer program SNAILZ developed by the Caltrans Office of Roadway Geotechnical Engineering and Slide 5 developed by Rocscience. The limit equilibrium analysis programs model a two-dimensional slope and compute a factor of safety for various failure planes. SNAILZ considers force equilibrium (not moment equilibrium) and can analyze only bi-linear wedge above the facing toe and tri-linear failure wedges below the toe. Because only passive pressure is considered below the facing toe by the SNAILZ program, Slide 5 was used to check the stability of possible deep toe failures.

In addition to the slope geometry, the parameters used in the analyses included the soil friction, cohesion, unit weights, water level, soil-grout bond ultimate strength, bar size, yield strength of reinforcement, and diameter of grout hole. The soil strength parameters previously presented in Appendix C were used. However, these parameters were factored prior to entering in the SNAILZ program, due to the method of analysis. A No. 9 bar, which consists of steel with a 75 ksi yield strength and a nominal area of one square inch, was assumed in our analyses. Different spacing, length, and number of rows of nails were considered. The calculated safety factors were found to be 1.5 or better for static conditions and 1.1 or better for pseudo-static conditions for the trial failure planes deemed representative. The outputs of selected analyses are included in Appendix C of this report. Table 7 presented in the following section summarizes the preliminary soil nail design parameters.

4.6.4 Soil-Nail Recommendations

The upper portion of the slope should be trimmed at an inclination of about ¾:1 to 1:1 depending upon the location to remove deteriorated materials and to obtain a smooth surface for placement of the shotcrete facing. Because of access constraints on the slope, we anticipate that the contractor will use an excavator with a smooth bucket (toothless) or other appropriate equipment to trim the slope. Since little backfilling is anticipated, most of the excavated soil material may require stockpiling and off-site disposal. The lower portion of the slope, extending from the existing toe wall to the soil nail facing, should be trimmed no steeper than an inclination of 2½:1 to facilitate planting and improve surficial stability. The top and toe of the soil-nail facing should be embedded at least 2 feet into undisturbed Old Paralic Deposits (Qops) to prevent surface water from getting behind the facing.

Our stability analyses were based on the Soil Nail Design Parameters presented in Table 7. The size, embedment depth, and spacing of the soil nails can be adjusted as necessary by the designer. However, we recommend that the soil nails have a minimum embedment depth of 30 feet. Based on the cross sections, we do not anticipate the soil nails to extend within 5 feet of the edge of curb.

Table 7: Soil Nail Design Parameters

Item	Description	Values Adopted
Square Nail Pattern	Vertical, S_V	5 feet
Nail Spacing	Horizontal, S_H	5 feet
Nail Inclination	Uniform	20 degrees
Number of Nails	Per Section	5
Nail Length	Uniform pattern	30 feet
Nail Bar	Type	Threaded No. 9
	Material	Steel Grade 75 ksi

Drill hole	Minimum Diameter	6 inches
Corrosion Protection	Grout Protected Nail Bar	Class I Protection
	Minimum Cover	1.5 inches outside sheathing
	Minimum Encapsulation	1 mm thick corrugated sheathing 10 mm grout between nail & sheeting
	PVC Centralizer	8 feet
Grout	Neat Cement	Minimum f'_C = 3,000 psi at 28 days
Ultimate Bond Strength	Minimum Specified	For Sandy Soils = 1,800 lbs/ft For Silty & Clayey Soils =1,350 lbs/ft

4.6.5 Corrosion Protection

Class I Protection, as defined in Appendix C of the FHWA soil nail design manual, should be provided for the soil nails. Class I Protection includes the use of encapsulated nails. Encapsulation is generally accomplished by grouting the nail tendon inside a corrugated plastic sheath. A neat cement grout containing admixtures to control water bleed from the grout is usually employed to fill the annular space between the plastic sheath and the tendon. For this type of protection, the minimum grout cover between the sheath and the borehole should not be less than 1.5 inches. The corrosion protection will be terminated to expose the bare tendon at the head of the nail in order to allow attachment of the bearing plate and nut. This section of the nail tendon may be susceptible to corrosion due to its exposure to climatic elements. Thus, it is recommended that the bearing plate assembly be embedded in the permanent facing with standard depths of grout cover to control steel corrosion. The nail tendon protection should be extended into the construction shotcrete facing to ensure minimum depth of shotcrete/nail grout cover of 3 inches.

4.6.6 Facing Drainage

The soil nail facing should be drained to reduce the potential for buildup of hydrostatic pressures. The drainage behind the facing should consist of geosynthetic drain strips at least 36 inches wide and should extend the full height of the facing (starting 12 inches below the top of the facing). The drain strips should be located no further than 6 feet on center. Weepholes should be located within one foot of the base of the facing. To avoid soaking and weakening the bottom of the facing soils, we recommend that weepholes not be allowed to drain directly onto the slope below the wall. Weephole drainage should be collected in manifolds and discharged at least 6 feet downslope of the facing. The manifold outlet pipe (minimum of 3 inch diameter) should be located every fourth weephole or 25 feet, whichever is less. Slotted caps should be placed at the ends of the manifold outlet pipes to prevent rodent entrance. The top and toe of the soil-nail facing should be embedded into at least 2 feet into undisturbed Old Paralic Deposits (Qops) to help reduce the risk of surface water entering behind the wall.

4.6.7 Constructability

Some of the soils (relatively clean sand and soils near or below groundwater) may be more prone to caving. The stability of the excavation face and tendency for sloughing, sliding or toppling, and loss of ground occurring during excavation needs to be considered. The shotcrete facing should be constructed immediately upon completing the installation of the soil nails so that sloughing and raveling of the excavation does not occur. Additionally, caving of sandy soils may be encountered when drilling the boreholes for the soil nails. Due to potential soil caving, the contractor should anticipate using hollow-stem-auger drilling equipment to install soil nails throughout the majority of the project alignment. Slow drilling due to cementation should also be anticipated.

If localized sloughing and raveling of the excavation is encountered, consideration should be given to reduce the unsupported height at that installation level and applying a thin coating of shotcrete. Another option is to construct a stabilizing berm against the area of caving and installing the soil nails to improve surface stability. The berm may then be removed when the shotcrete is applied.

4.6.8 Testing Procedures

The bond strength of the nails is affected by the method of drilling and cutting removals. The bond strength should be verified by testing at least two "sacrificial" nails for each soil type encountered. A minimum of four soil nails should be subject to verification tests. Proof testing should be performed on at least 5 percent of the production nails, as recommended by Circular 7. The verification and proof testing should be performed in general accordance with procedures outlined in Appendix E of the FHWA soil nail design manual.

4.7 SLOPE MAINTENANCE

Deterioration of the slope surfaces may be reduced by maintaining the landscaping. Appropriate vegetation, watering and drainage control, along with adequate maintenance will reduce the potential for surficial erosion and surficial instability. Any burrowing rodent/animal activity on the slopes should be controlled. Excessive watering can result in uncontrolled runoff and erosion of the slope face, as well as excess infiltration of water into the subgrade soils. A professional landscape architect should be responsible for the long-term maintenance of the irrigation program.

4.8 SLOPE CREEP AND FILL SLOPE EXTENSION

Slope creep, which is influenced by factors such as slope height, slope geometry, exposure to weather, irrigation, rainfall and landscaping, should be anticipated. The most active zone is usually within the outer 4 to 8 feet of the slope face (perpendicular to the face) but the effect may be noted 10 to 15 feet at the top of the bluff. The exact amount of movement due to slope creep depends largely upon irrigation practice and soil properties, but generally lateral and vertical deflections on the order of a few inches may occur over a few years.

Slope creep is caused by a combination of moisture penetrating the ground, drying of the soil, and force of gravity. Slope creep can not be totally prevented, but its effects can be reduced by proper design and construction practices, drainage, and irrigation. To reduce slope movement due to slope creep, the slope should be maintained. Excessive drying, soaking or saturation of the subgrade soils should be prevented.

4.9 EARTHWORK

4.9.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state or federal specifications. All references to maximum unit weights are established in accordance with ASTM Standard Test Method D1557-07.

Earthwork operations during the wet season may require provisions for drying of soil prior to compaction. If the project necessitates fill placement and compaction in wet conditions, we can provide alternatives for drying the soil during construction. Conversely, additional moisture may be required during drier months. If grading occurs following an extended period of dry weather, we recommend pre-moisture conditioning drier soils to facilitate compaction. One method of pre-moisture conditioning consists of spraying soils repeatedly with water until moisture penetration is achieved, starting a few days before grading. Caution should be used so that the soils are not over-saturated.

4.9.2 Clearing and Removals

Weeds, grass, trees and shrubs (including major root systems), organic topsoil, riprap, construction debris, and other deleterious materials within areas to be graded should be cleared and properly disposed of off-site. Removals of unsuitable topsoil, colluvium, slide debris, undocumented fill, deteriorated engineered fill, and weathered Paralic deposits are required to expose suitable material on which to place engineered fill and support new fill, the soil nail system, or other improvements. The existing colluvium topping the wall at the toe of the slope should be removed to a depth of approximately 4 to 6 inches below the top of the wall.

4.9.3 Processing Following Removals

Following removal, the exposed surfaces within areas to receive fill should be properly scarified, moisture conditioned to at least optimum moisture content, and compacted to at least 90 percent of the maximum dry unit weight as determined by ASTM D-1557.

Scarification and moisture conditioning of the bottom should be uniform to prevent the occurrence of grooves of loose soils following recompaction. Areas temporarily vacated during the grading operation should be similarly scarified, moisture conditioned and reworked to the satisfaction of the Geotechnical Engineer-of-Record prior to placement of additional fill to avoid the occurrence of laminations along the fill interface.

Elevated moisture contents were observed in near-surface soils in localized areas. Moreover, if grading takes place during and following the rainy season, or where soil is irrigated, wet soils may be more predominant. The contractor should be aware that localized excavations may be subject to "pumping" and may have unstable bottoms depending on the location. Some drying of the material may be required to obtain moisture contents necessary for compaction. The contractor may have to use reinforcing fabric and ¾-inch crushed rock to stabilize the removal bottom in localized areas prior to placement of fill soils.

4.9.4 Engineered Fill

We anticipate that most of the on-site soils may be reusable as engineered fill material after all vegetation, construction debris and deleterious material are removed. Engineered fill should have no particles greater than six inches in maximum dimension, be placed in lifts no greater than eight inches thick (loose measurements), and be compacted to a minimum of 92 percent of the soil's maximum dry unit weight (ASTM D1557) except in planting areas where the upper one foot may be compacted to 85 percent relative compaction.

Sandy soils should be moisture conditioned to within 2 percent of the optimum moisture content. Clayey soils should be moisture conditioned between 2 and 4 percent above the optimum moisture content. Each lift of soil should be adequately mixed prior to compaction so that pockets of dissimilar material or moisture content are not readily apparent. If poor mixing is resulting in inconsistent moisture conditioning and/or blending, the Geotechnical Engineer-of-Record may require use of one or more discs to facilitate the mixing process.

4.9.5 Benching

Fill material placed on natural slopes with an inclination steeper than 5:1 (horizontal:vertical) should be benched into competent natural soil to the satisfaction of the Geotechnical Engineer or Engineering Geologist Engineer-of-Record. Benches should be step-like in profile, with each bench not less than four feet in height and established in competent material. Competent material is defined as being essentially free of loose soil and heavy fracturing.

4.9.6 Temporary Cut Slopes

Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept back from the top of the excavation a distance equal to the depth of the excavation, and all surface water should be diverted away from the excavation. Where removals are planned adjacent to existing facilities, grading should not occur within a zone identified by a plane extending down and out at a 1.5:1 inclination from the bottom outside edge of an existing foundation. Properly designed shoring should be used to protect improvements where grading is proposed and slope inclination is not feasible. A representative of the Geotechnical Engineer-of-Record should inspect all excavations during construction to allow any modifications to be made due to variations in the soil conditions.

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. We are providing this information solely as a service to our client. Under no circumstances should the information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

The contractor should be aware that slope height, slope inclination, and/or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations).

4.9.7 Permanent Cut Slope Construction

All the loose material should be removed from the permanent cut slopes. In addition, where space allows, brow ditches or berms should be constructed at the top of the slope to reduce the potential for surface runoff to flow over the face of the slope. The brow ditches should drain to a suitable collection device.

4.9.8 Permanent Fill Slope Construction

Fill slopes should be overfilled a minimum of two feet (measured perpendicular to the slope face), or by a sufficient distance such that when the overbuilt slope is trimmed back, a minimum of 92 percent relative compaction is achieved at the exposed slope face, except for planting areas, where a minimum of 85 percent relative compaction is required within the upper foot. Where space does not allow access to trim the slope, as an alternative to overfilling, the contractor may elect to build the slope to the finish slope face provided compaction is extended to the slope face; backrolling with a sheepsfoot is performed at vertical intervals not exceeding four feet in height; and the final slope is moisture conditioned, shaped, and recompacted with a vibratory roller and finally grid rolled.

5 ADDITIONAL SERVICES

The construction process is an integral design component with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes, and because we sample only a small portion of the soils affecting the performance of the proposed structure, unanticipated or changed conditions can be disclosed during grading. Proper geotechnical observation and testing during construction is imperative to allow the Geotechnical Engineer-of-Record the opportunity to verify assumptions made during the design process. Therefore, we strongly recommend that Kleinfelder be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. Kleinfelder cannot remain Geotechnical Engineer-of-Record for the project if we are not retained to provide these services.

Our services are typically needed at the following stages of construction.

- Site removal operations to confirm that subgrade bottoms soils are suitable for placement of engineered fill;
- Drilling of the posts and the installation of timber boards;
- Subgrade preparation for curbs;
- Anchoring and nailing of the erosion protection fabric;
- Subdrain construction;
- Engineered fill placement to check that moisture and density are per the project specifications; and
- Soil-nail installation.

We further recommend that Kleinfelder perform a general review of the project plans and specifications before they are finalized to verify that our geotechnical recommendations have been properly interpreted and implemented during design. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations.

6 LIMITATIONS

This report has been prepared for the exclusive use of the City of Long Beach's Department of Parks, Recreation & Marine for specific application to the proposed slope improvements at Bluff Park in the City of Long Beach, California. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The scope of services was limited to the field exploration program described in Section 1.3. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our field exploration, laboratory testing programs, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder's on-site representative is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

Our proposed scope of services and fee for the completed subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site. However, Kleinfelder can provide environmental assessments and evaluations if requested by the City.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained to provide construction observation and testing services so that the geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder during site preparation, grading, preparation of foundations, placement of engineered fill, and installation of soil nails. Kleinfelder should be retained to perform periodic observation during installation of the erosion fabric and excavation of the shrub holes. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the Geotechnical Engineer-of-Record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders should visit the site, study the site conditions, perform surveys, if needed, and make their own interpretations. The figures and cross sections are only approximate representations of topographic conditions and may not be relied upon for design or quantity estimates. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's Geotechnical Engineer-of-Record can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

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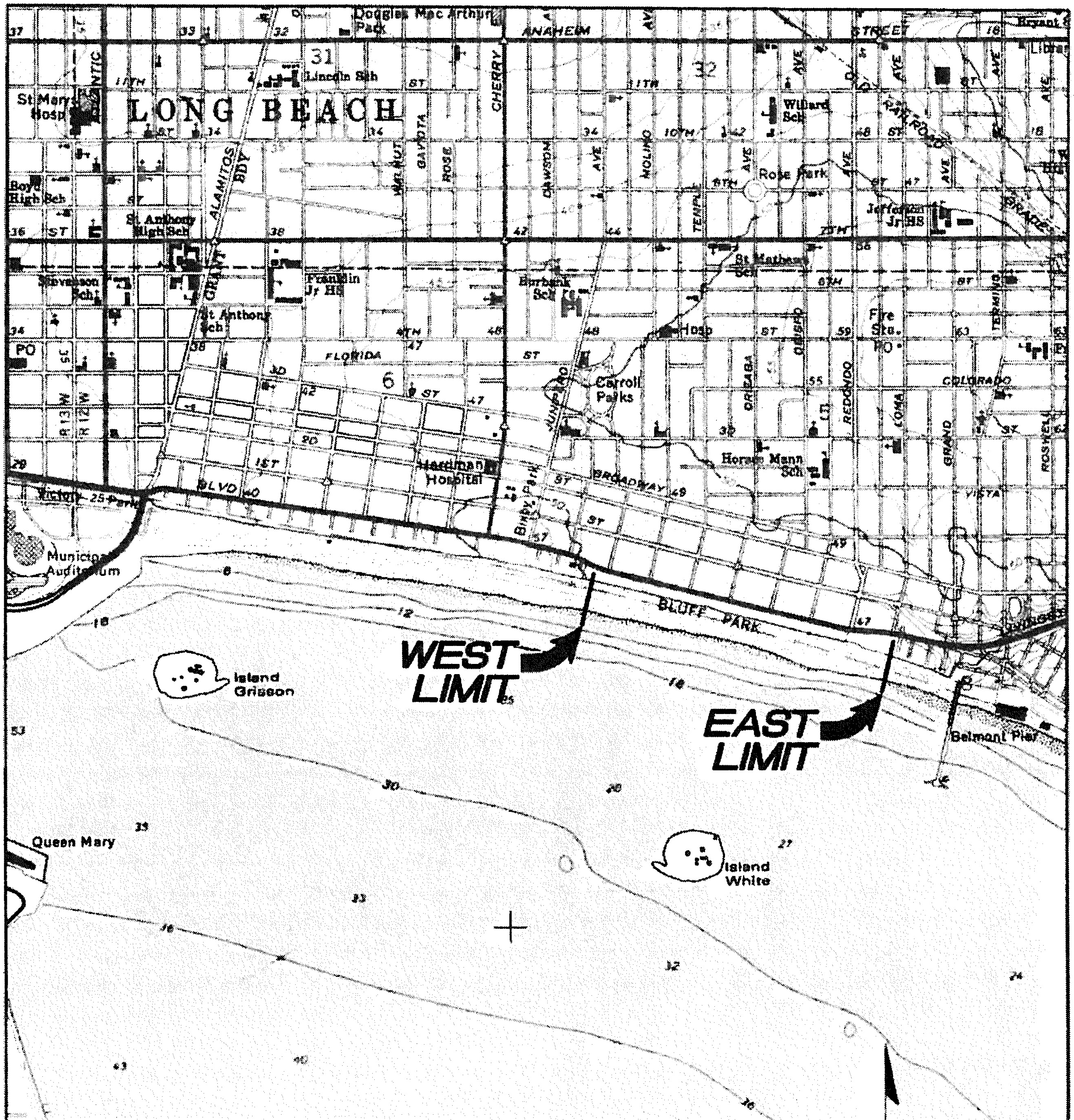
PLATES



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APPROXIMATE SCALE (feet)

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DRAWN BY:	MRG
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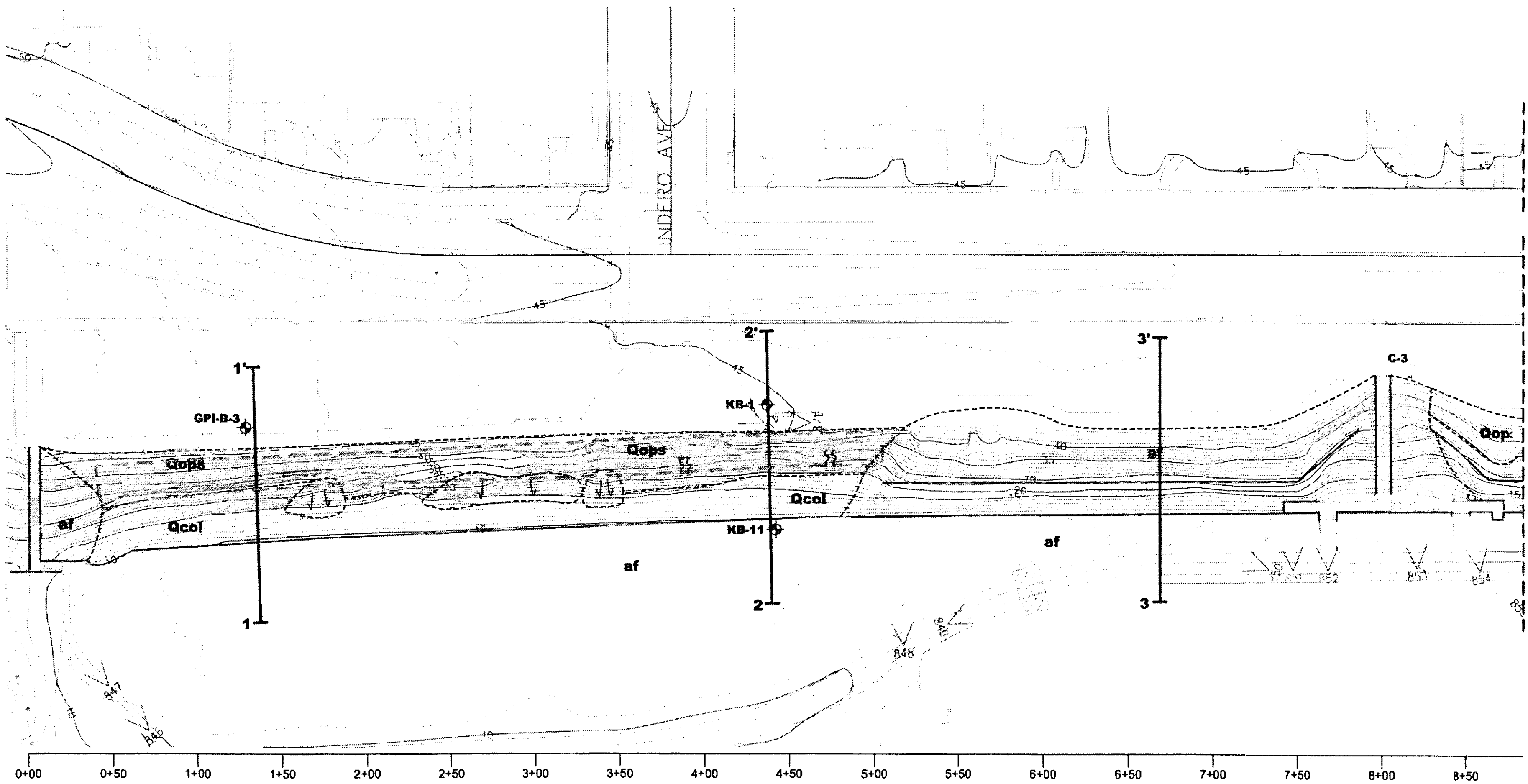
SITE LOCATION MAP

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
1



EXPLANATION

- **af** ARTIFICIAL FILL
- **Qls** LANDSLIDE
- **Qcol** COLLUVIUM
- **Qops** QUATERNARY OLD PARALIC DEPOSITS
- BORING LOCATION BY GPI (GPI-B-3) AND BY KLEINFELDER (KB-11)
- CONE PENETRATION TEST BY GPI (C-5)
- 1 —— 1' CROSS-SECTION LOCATION
- LANDSLIDE DEBRIS/SLUMP MATERIAL
- EROSION AREA
- APPROXIMATE GEOLOGIC CONTACT
- FIRST PRIORITY FOR SOIL NAIL FACING (TO BE FIELD VARIFIED)
- SECOND PRIORITY FOR SOIL NAIL FACING (TO BE FIELD VARIFIED)

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APPROXIMATE SCALE (feet)

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GEOLOGIC MAP
STATION 0+00 TO 8+85

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
2A

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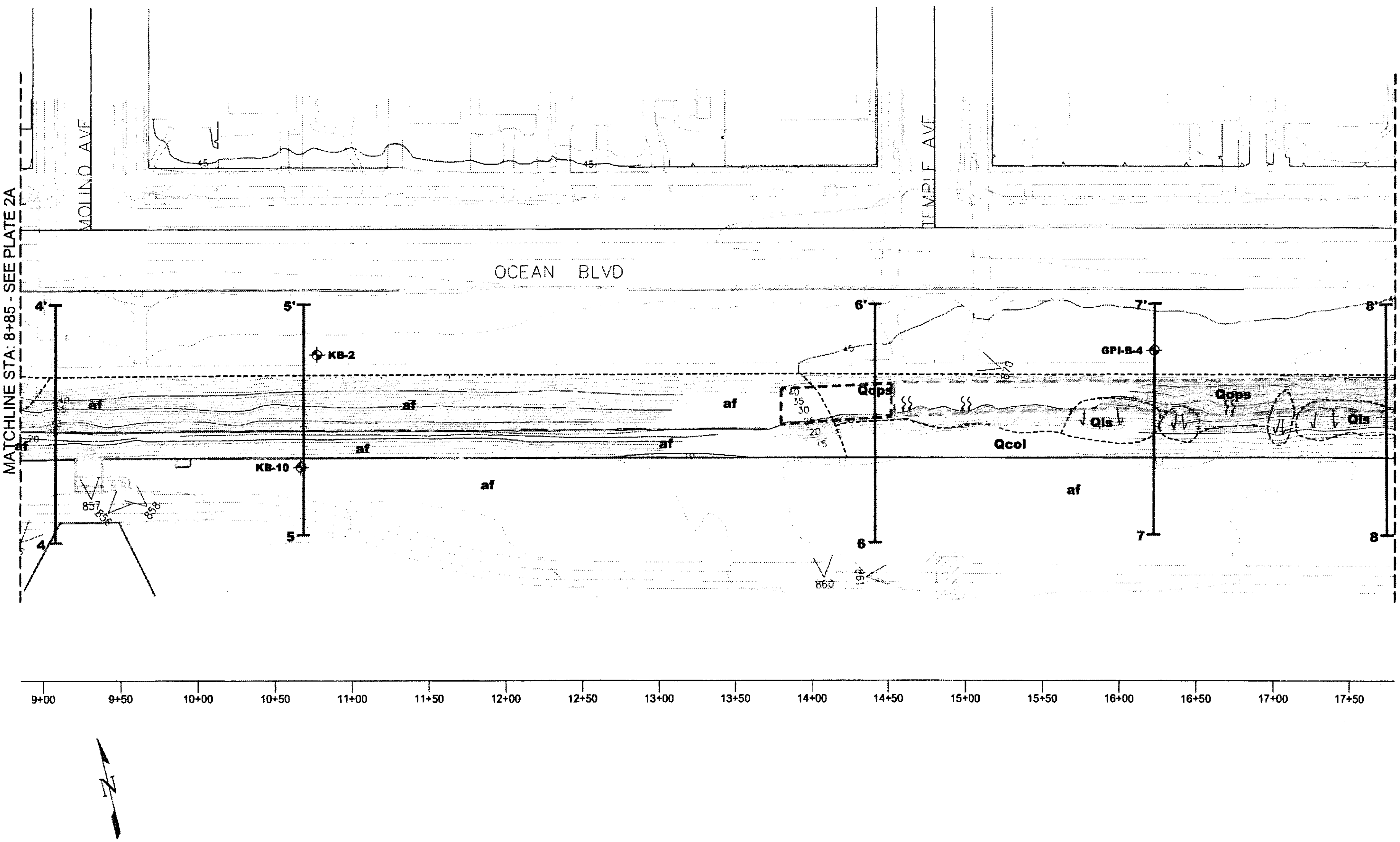

NOTE: SEE PLATE 2A FOR GEOLOGIC EXPLANATION

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GEOLOGIC MAP
STATION 8+85 TO 17+80

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
2B

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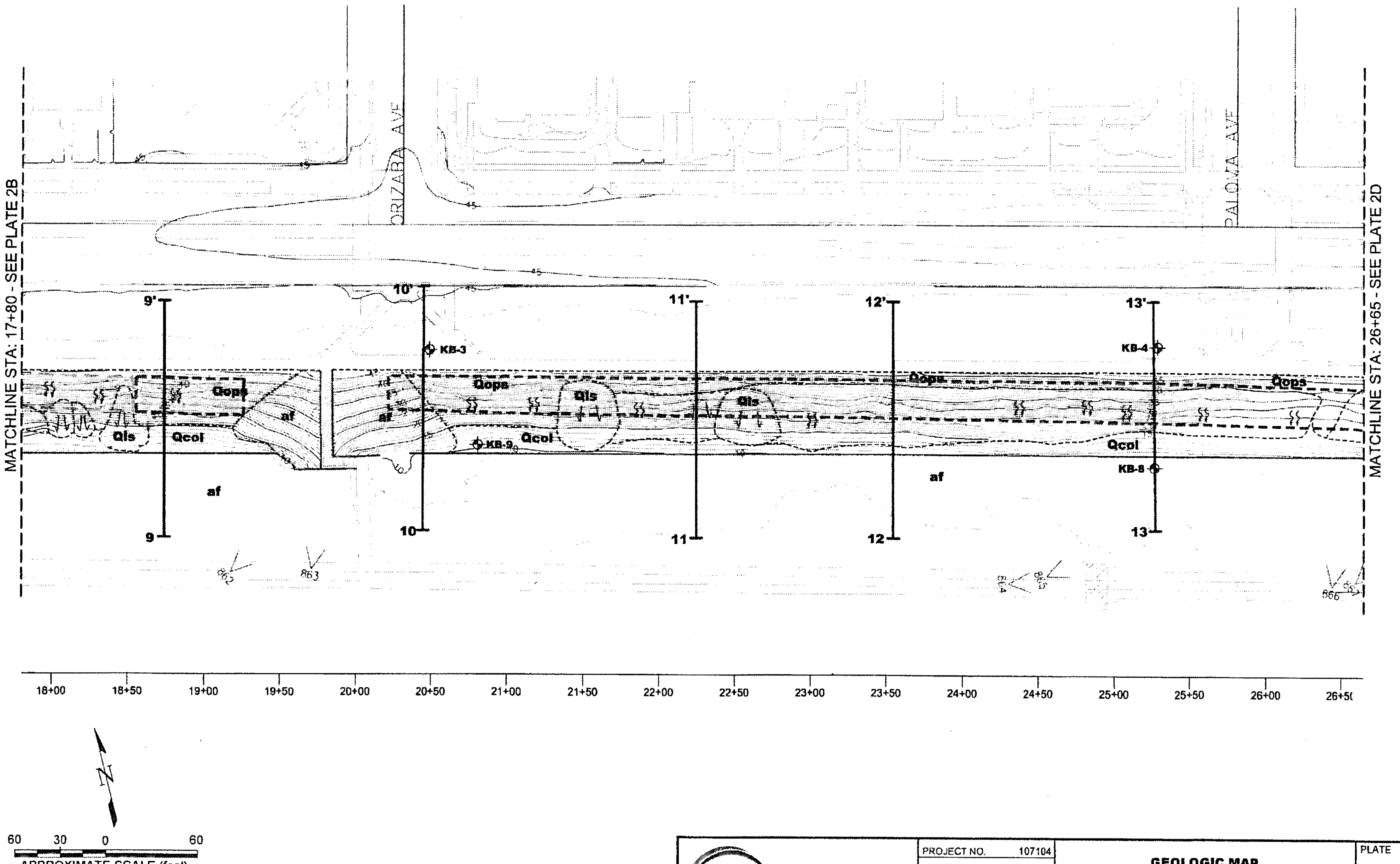

NOTE: SEE PLATE 2A FOR GEOLOGIC EXPLANATION

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GEOLOGIC MAP
STATION 17+80 TO 26+65

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
2C

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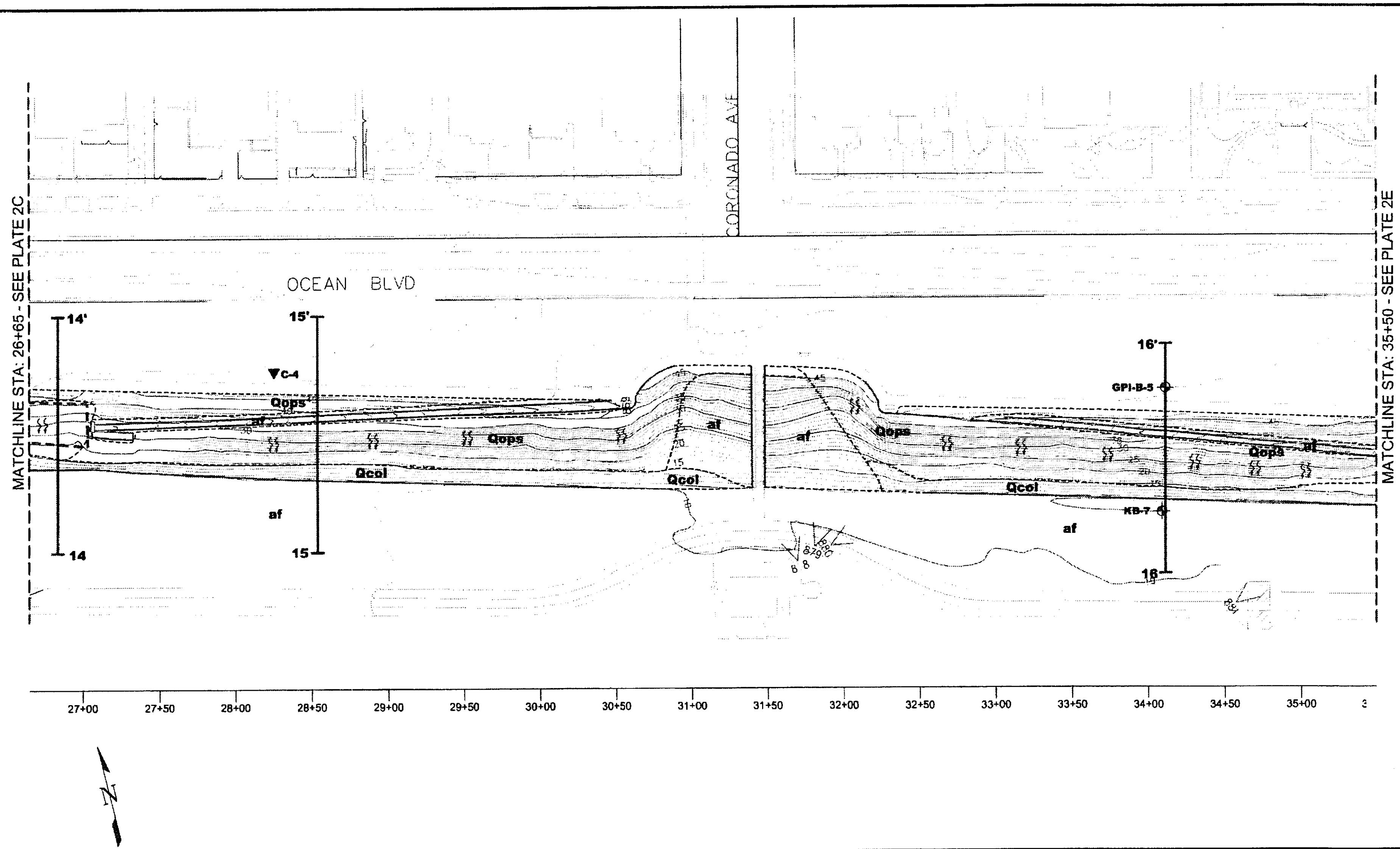

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GEOLOGIC MAP
STATION 26+65 TO 35+50

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
2D

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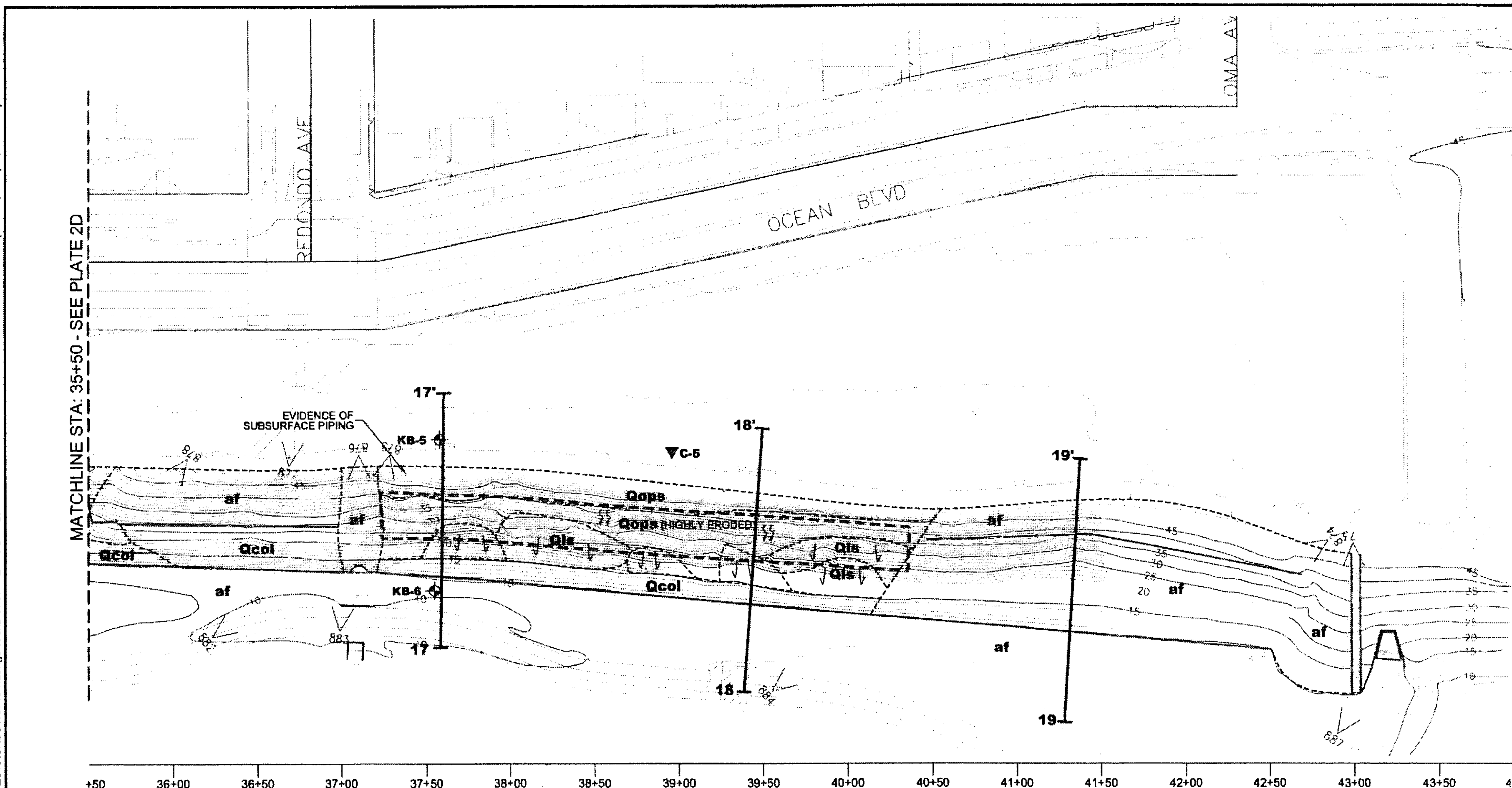

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APPROXIMATE SCALE (feet)

NOTE: SEE PLATE 2A FOR GEOLOGIC EXPLANATION

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GEOLOGIC MAP
STATION 35+50 TO END

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE **2E**

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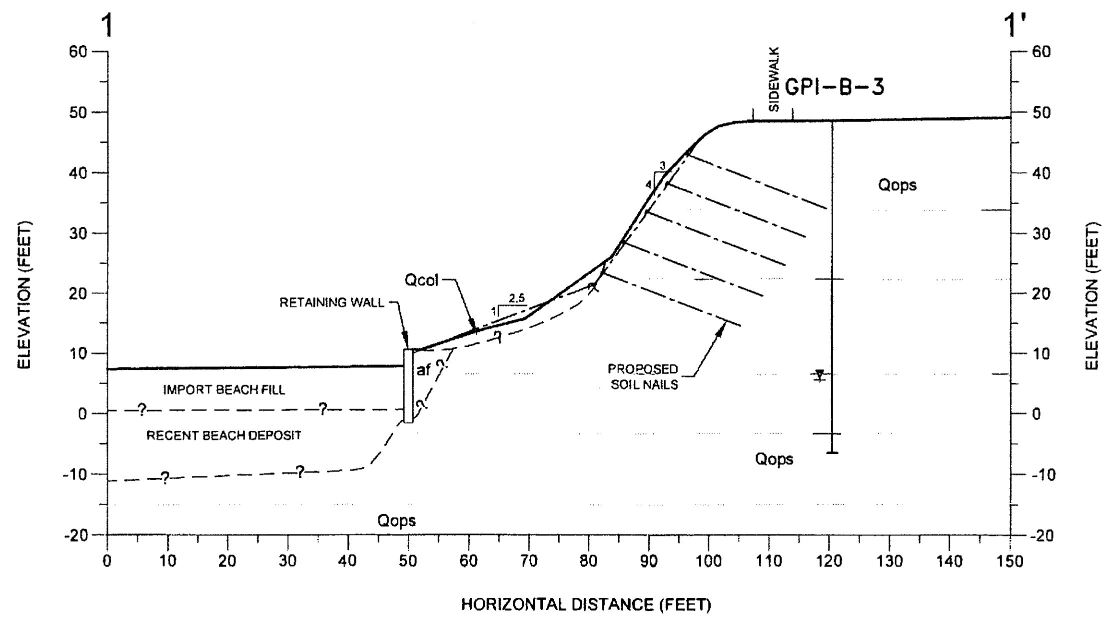

EXPLANATION

- af — ARTIFICIAL FILL
- Qcol — COLLUVIUM
- Qls — LANDSLIDE
- Qops — QUATERNARY OLD PARALIC DEPOSITS
- BORING
- GROUNDWATER LEVEL
- — ? — APPROXIMATE GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)
- GENERALIZED BEDDING

20 10 0 20
APPROXIMATE SCALE (feet)

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PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY	MRG
CHECKED BY:	JR
FILE NAME:	107104p3A to 3S_CS.dwg

GEOLOGIC CROSS-SECTION 1-1'
STATION 1+34.70

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3A

PLOTTED: 27 Apr 2010, 5:05pm, dfahmey
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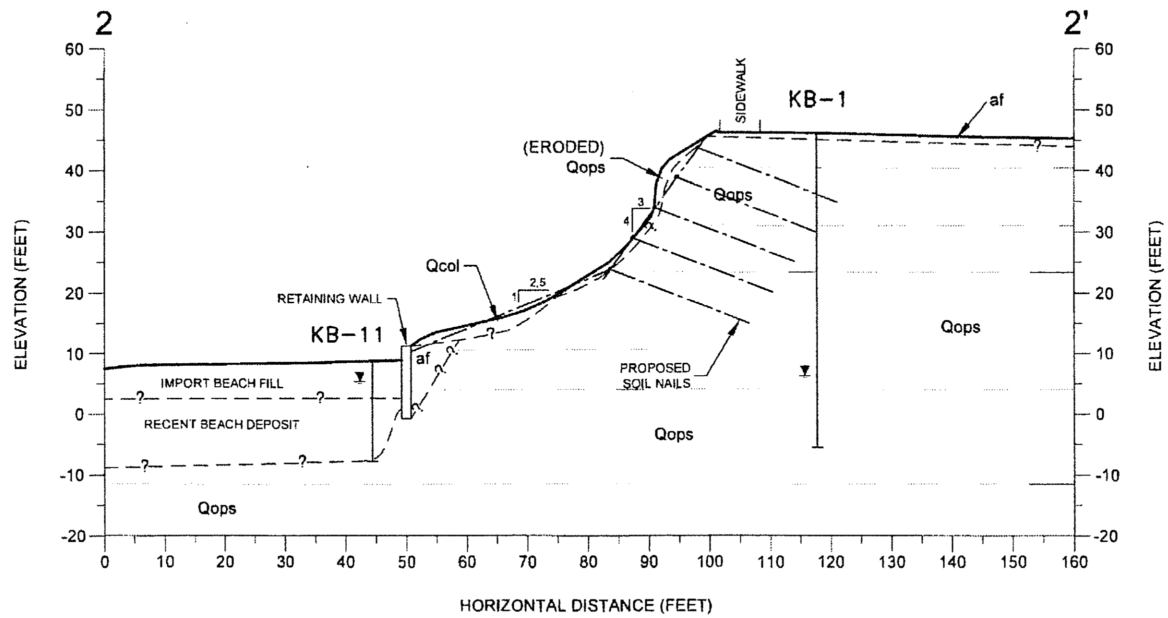

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APPROXIMATE SCALE (feet)

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 107104	GEOLOGIC CROSS-SECTION 2-2' STATION 4+38.24	PLATE
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	DRAWN BY: MRG		
	CHECKED BY: JR	PROPOSED SLOPE IMPROVEMENTS BLUFF PARK LONG BEACH, CALIFORNIA	
	FILE NAME: 107104p3A to 3S_CS.dwg		

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3b PLOTTED: 28 Apr 2010, 12:50pm, dfahrney

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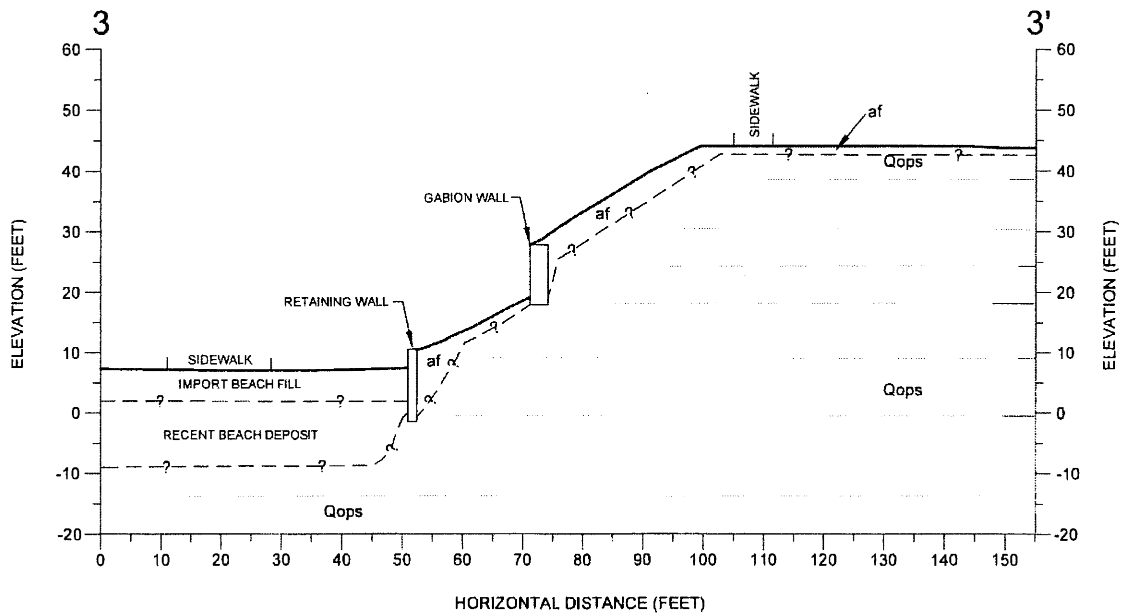

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APPROXIMATE SCALE (feet)

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 107104	GEOLOGIC CROSS-SECTION 3-3' STATION 6+77.33	PLATE
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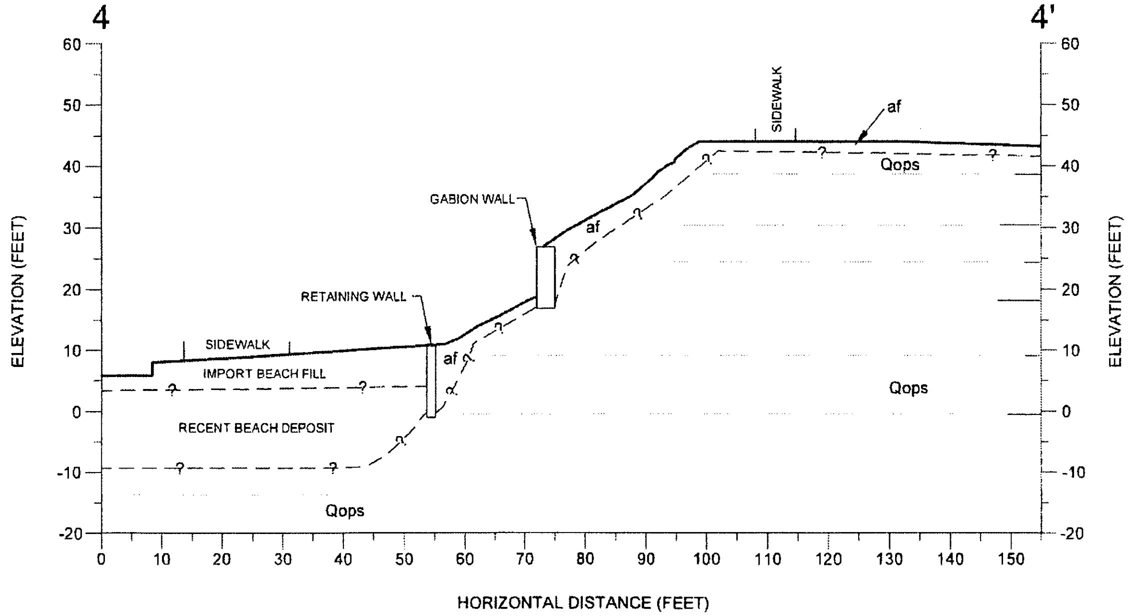
	DRAWN BY: MRG		
	CHECKED BY: JR	PROPOSED SLOPE IMPROVEMENTS BLUFF PARK LONG BEACH, CALIFORNIA	
	FILE NAME: 107104p3A to 3S_CS.dwg		

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3C PLOTTED: 27 Apr 2010, 5:11pm, dfahmey



20 10 0 20
APPROXIMATE SCALE (feet)

PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104p3A to 3S_CS.dwg	

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GEOLOGIC CROSS-SECTION 4-4'
STATION 9+14.43

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3D

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3D PLOTTED: 27 Apr 2010, 5:11pm, dfahmey

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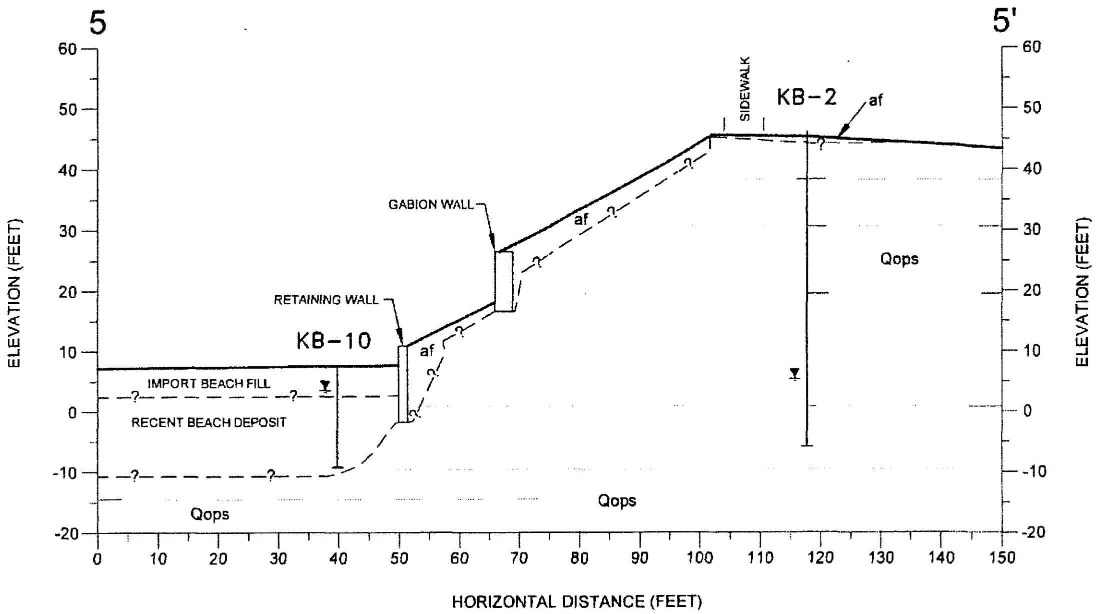

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APPROXIMATE SCALE (feet)

PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107104p3A to 3S_CS.dwg

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GEOLOGIC CROSS-SECTION 5-5'
STATION 10+68.62

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3E

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3E PLOTTED: 27 Apr 2010, 5:12pm, dfahmey

ATTACHED IMAGES:
ATTACHED XREFS:
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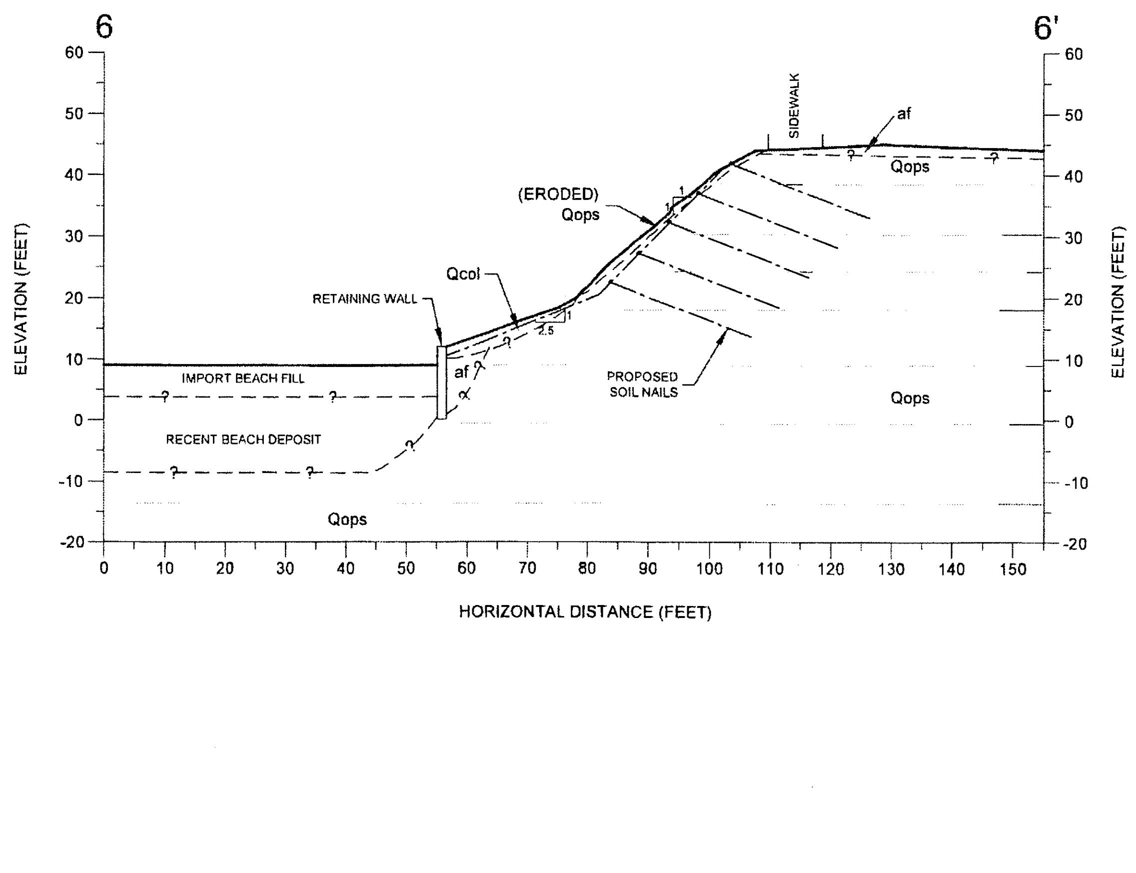

PLOTTED: 27 Apr 2010, 5:14pm, dfahmey

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3F

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

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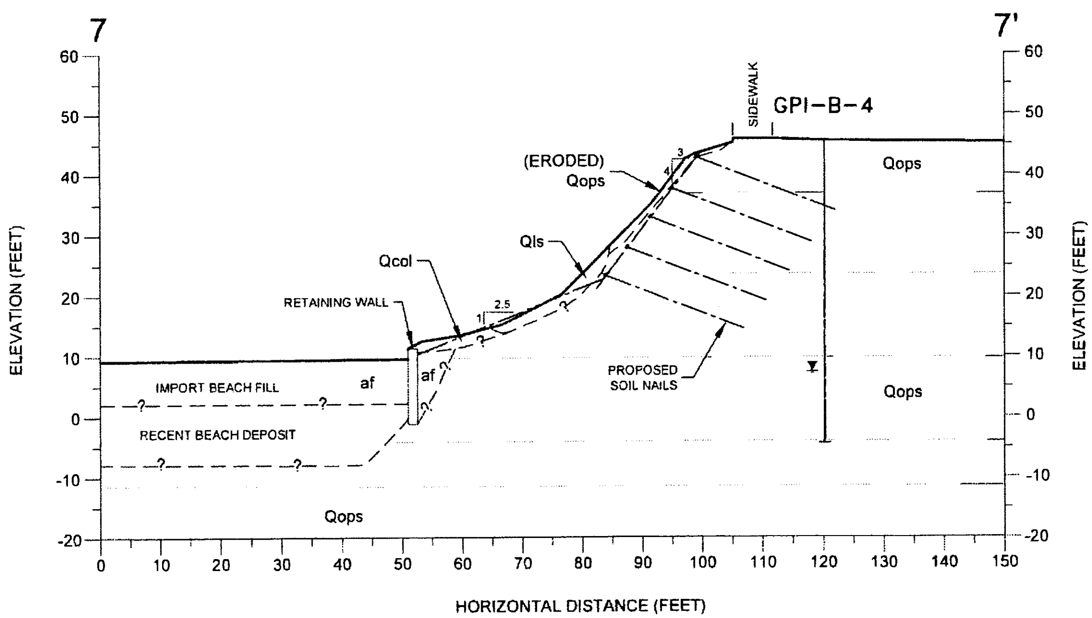
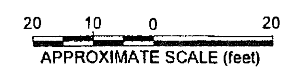
PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107104p3A to 3S_CS.dwg

GEOLOGIC CROSS-SECTION 6-6'
STATION 14+43.31

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3F





KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 107104	GEOLOGIC CROSS-SECTION 7-7' STATION 16+23.16	PLATE
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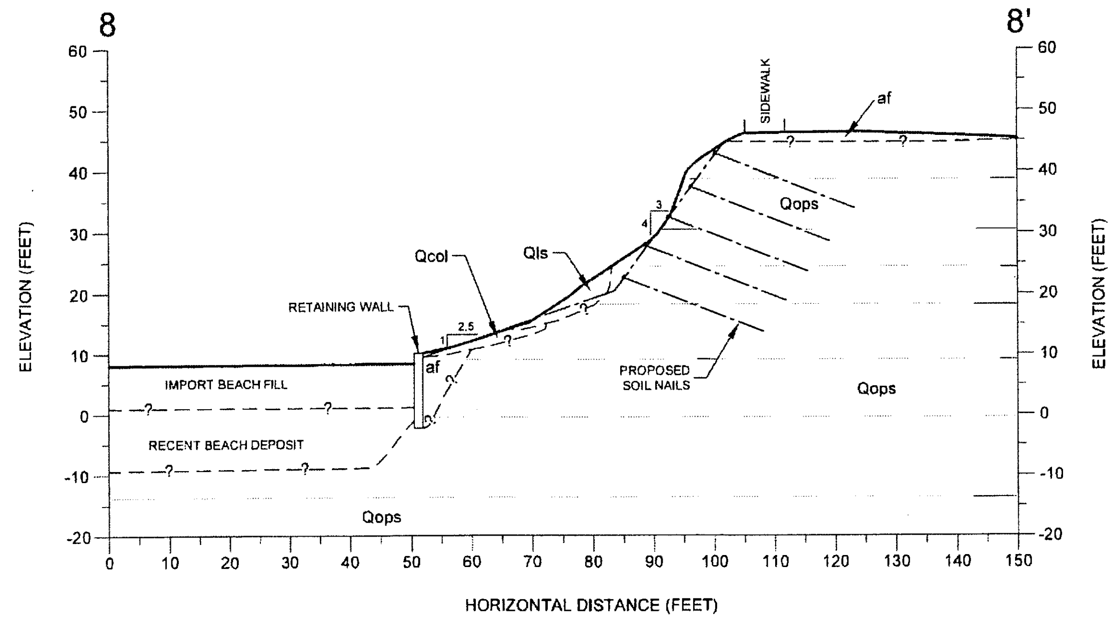
	DRAWN BY: MRG	PROPOSED SLOPE IMPROVEMENTS BLUFF PARK LONG BEACH, CALIFORNIA	
	CHECKED BY: JR		
	FILE NAME: 107104p3A to 3S_CS.dwg		



PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104p3A to 3S_CS.dwg	

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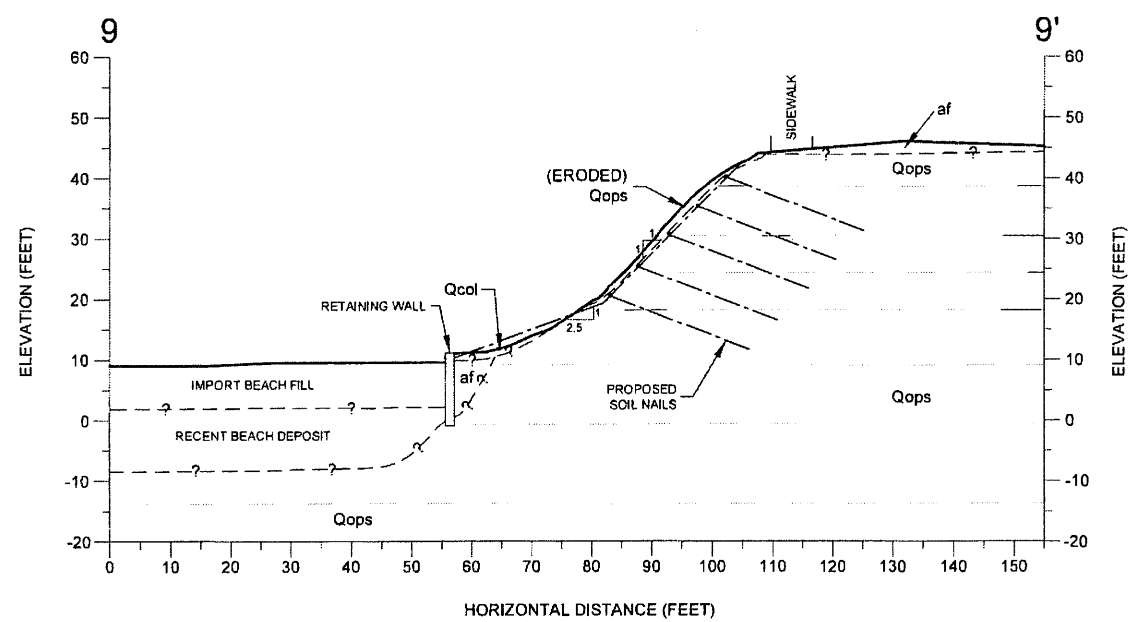
GEOLOGIC CROSS-SECTION 8-8'
STATION 17+74.14

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3H

PLOTTED: 28 Apr 2010, 12:41pm, dfahmey
LAYOUT: 3H
CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA



20 10 0 20

APPROXIMATE SCALE (feet)

PROJECT NO.	107104	
DRAWN:	4/2010	
DRAWN BY:	MRG	
CHECKED BY:	JR	
FILE NAME: 107104p3A to 3S_CS.dwg		

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GEOLOGIC CROSS-SECTION 9-9'
STATION 18+76.70

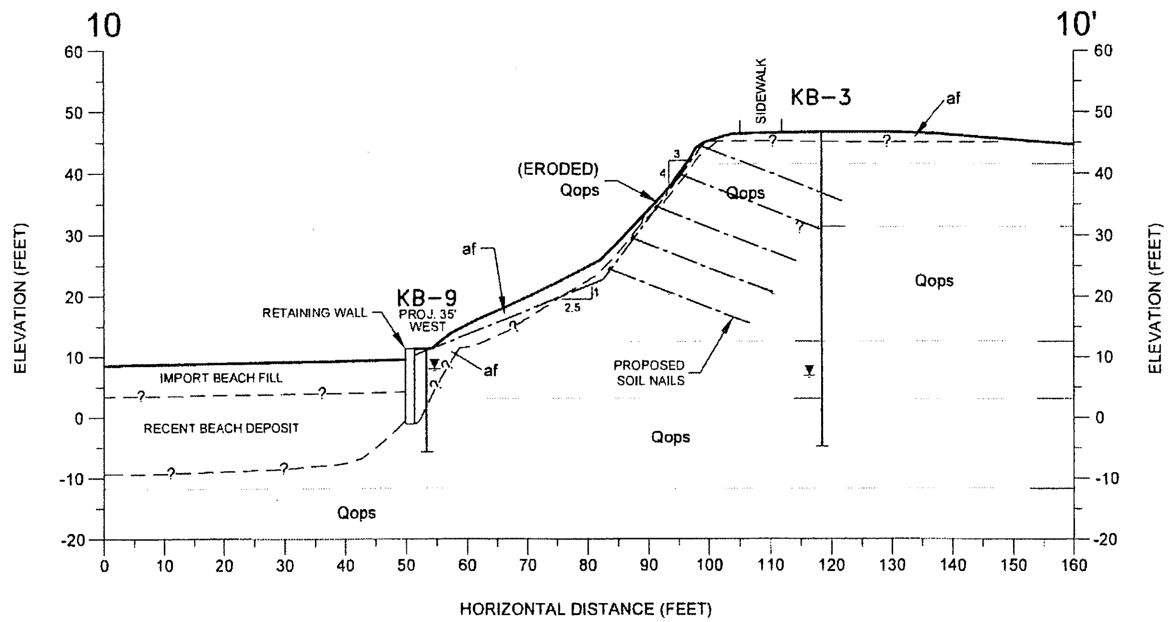
PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3I

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3I

PLOTTED: 27 Apr 2010, 5:19pm, dfahmey



APPROXIMATE SCALE (feet)

PROJECT NO. 107104	GEOLOGIC CROSS-SECTION 10-10' STATION 20+44.88	PLATE
DRAWN: 4/2010		3J
DRAWN BY: MRG	PROPOSED SLOPE IMPROVEMENTS BLUFF PARK LONG BEACH, CALIFORNIA	
CHECKED BY: JR		
FILE NAME: 107104p3A to 3S_CS.dwg		

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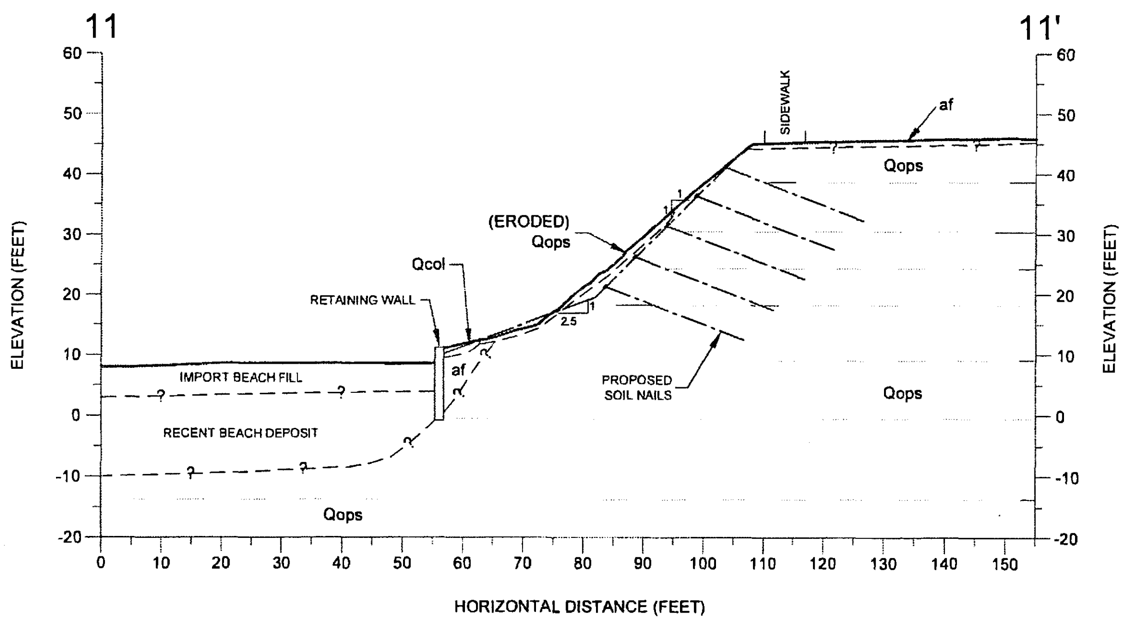
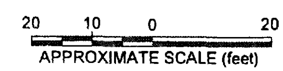
PLOTTED: 28 Apr 2010, 12:43pm, dfahmey
LAYOUT: 3J
ATTACHED IMAGES:
ATTACHED XREFS:
BLUFF PARK PROPOSED SLOPE IMPROVEMENTS ...\2010\CADD\107104\LongBeachBluffs_CS\





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PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104p3A to 3S_CS.dwg	

GEOLOGIC CROSS-SECTION 11-11'
STATION 22+27.47

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3K

PLOTTED: 27 Apr 2010, 5:28pm, dfahmey
CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3K
ATTACHED IMAGES:
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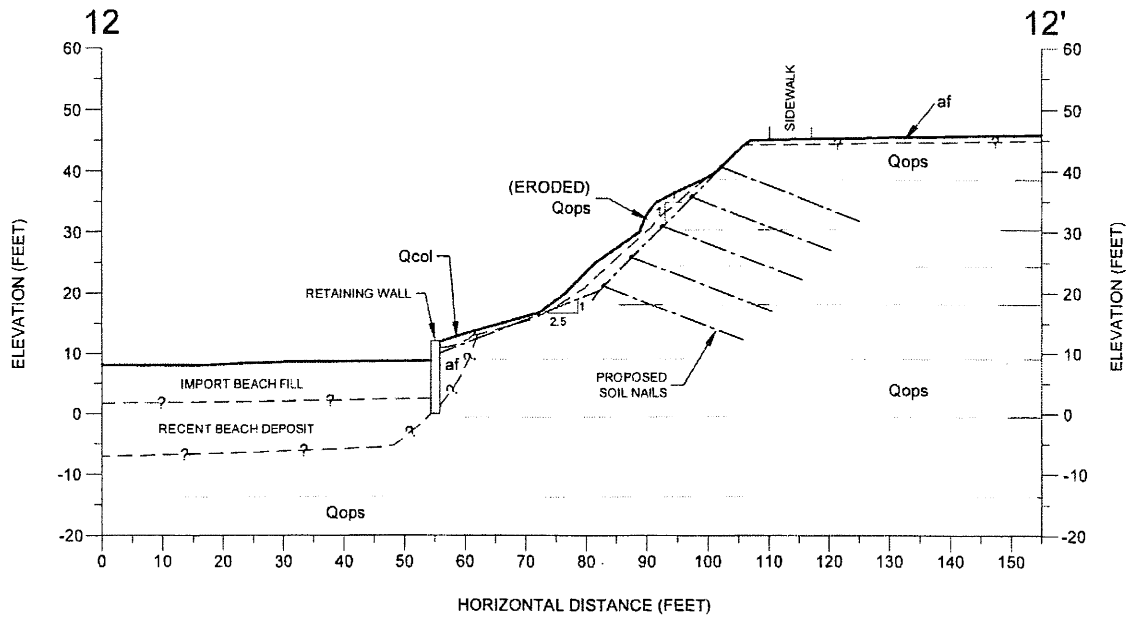
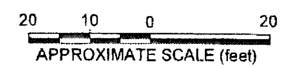
DIAMOND BAR, CA

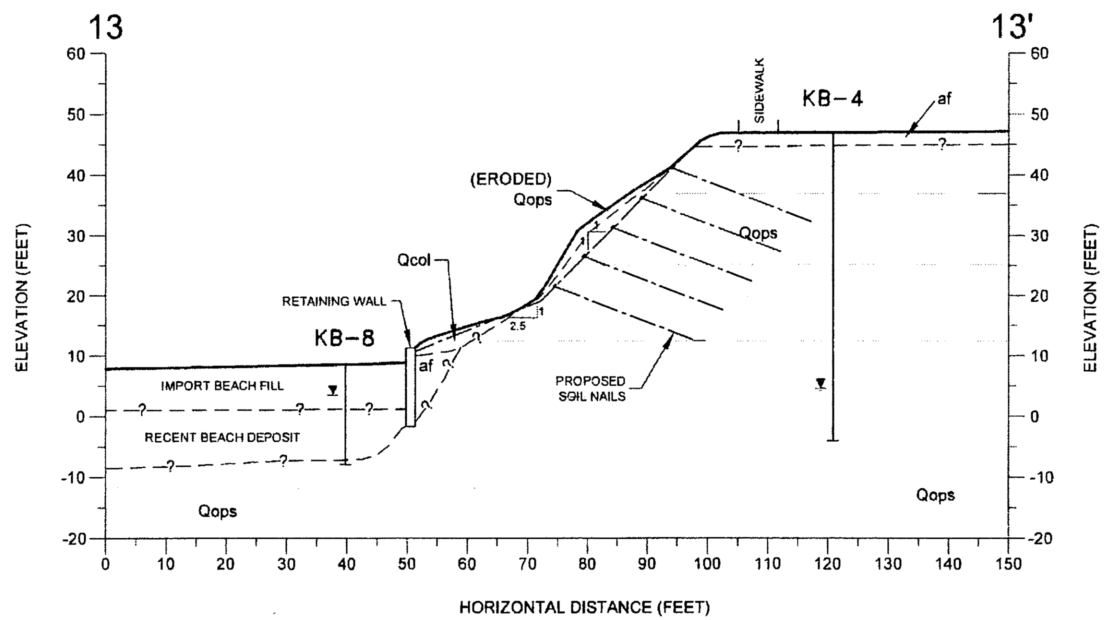


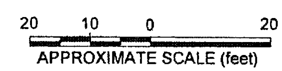


KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 107104 DRAWN: 4/2010 DRAWN BY: MRG CHECKED BY: JR FILE NAME: 107104p3A to 3S_CS.dwg	**GEOLOGIC CROSS-SECTION 12-12'** **STATION 23+62.96** PROPOSED SLOPE IMPROVEMENTS BLUFF PARK LONG BEACH, CALIFORNIA	PLATE **3L**

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3L PLOTTED: 28 Apr 2010, 12:51pm, dfahmey





PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107104p3A to 3S_CS.dwg

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GEOLOGIC CROSS-SECTION 13-13'
STATION 25+26.56

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3M

PLOTTED: 27 Apr 2010, 5:30pm, dfahmey
CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3M
ATTACHED IMAGES:
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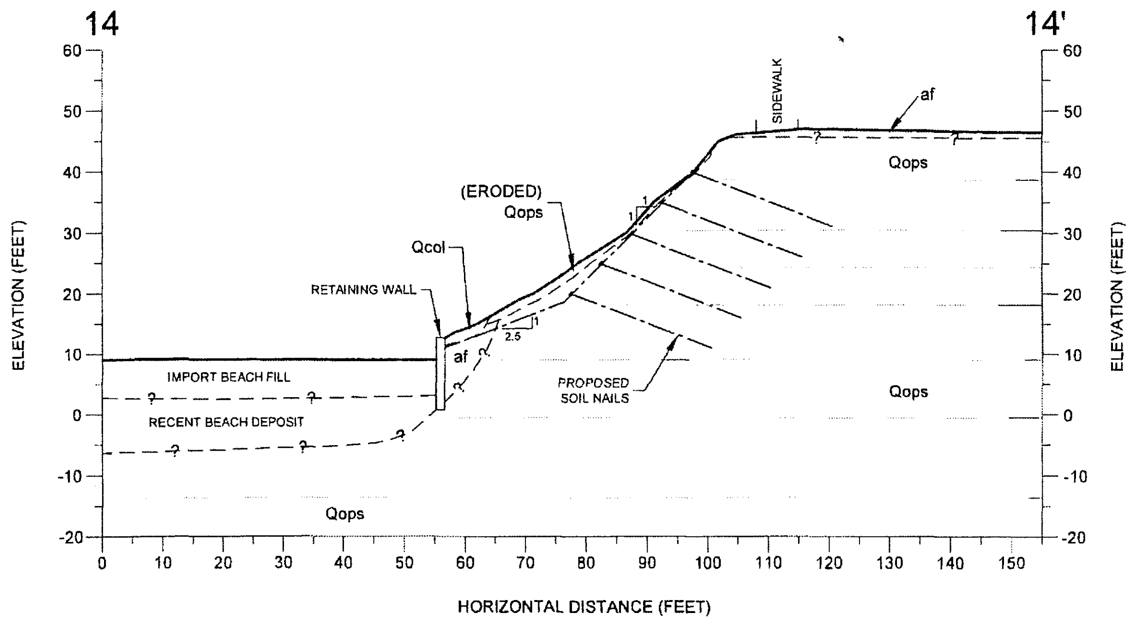
DIAMOND BAR, CA



20 10 0 20

APPROXIMATE SCALE (feet)

PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104p3A to 3S_CS.dwg	

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GEOLOGIC CROSS-SECTION 14-14'
STATION 26+86.10

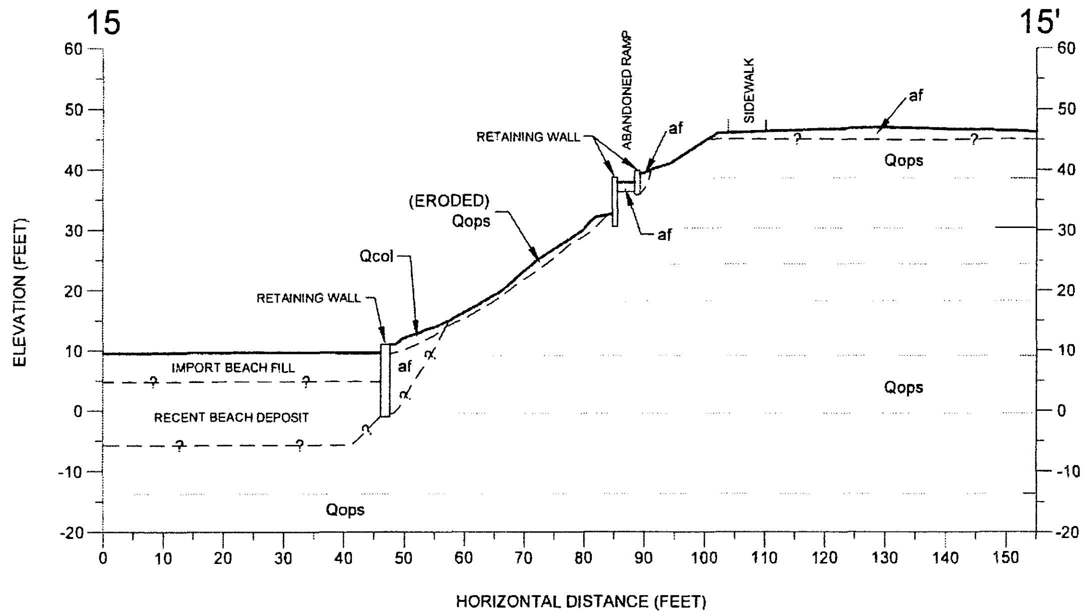
PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3N

PLOTTED: 27 Apr 2010, 5:31pm, dfanmey

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3N

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA



20 10 0 20

APPROXIMATE SCALE (feet)

PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107104p3A to 3S_CS.dwg

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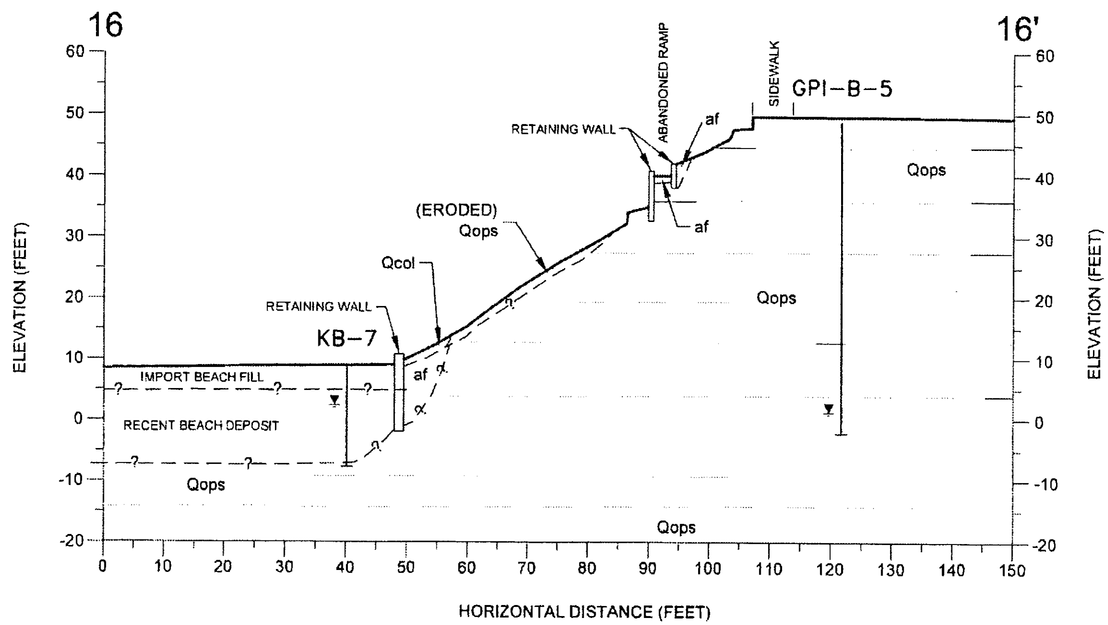
GEOLOGIC CROSS-SECTION 15-15'
STATION 28+62.74

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
30



20 10 0 20
APPROXIMATE SCALE (feet)

PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107104p3A to 3S_CS.dwg

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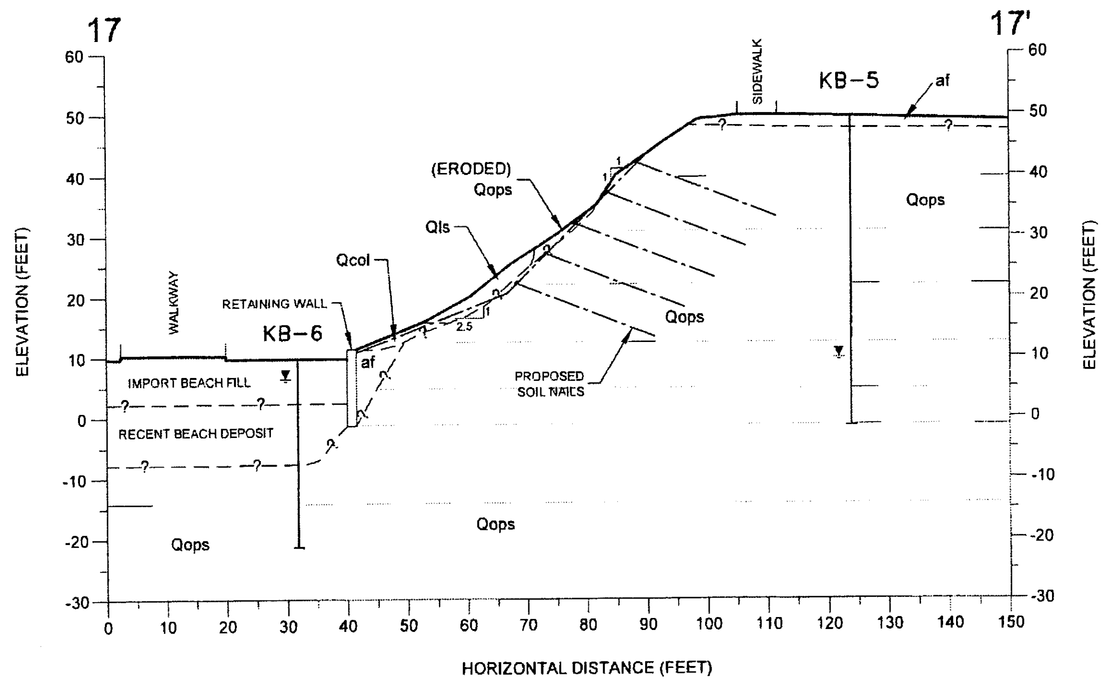
GEOLOGIC CROSS-SECTION 16-16'
STATION 34+11.35

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3P

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3P ·PLOTTED: 27 Apr 2010, 5:38pm, dfahmey

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA



20 10 0 20
APPROXIMATE SCALE (feet)

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PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104p3A to 3S_CS.dwg	

GEOLOGIC CROSS-SECTION 17-17'
STATION 37+59.66

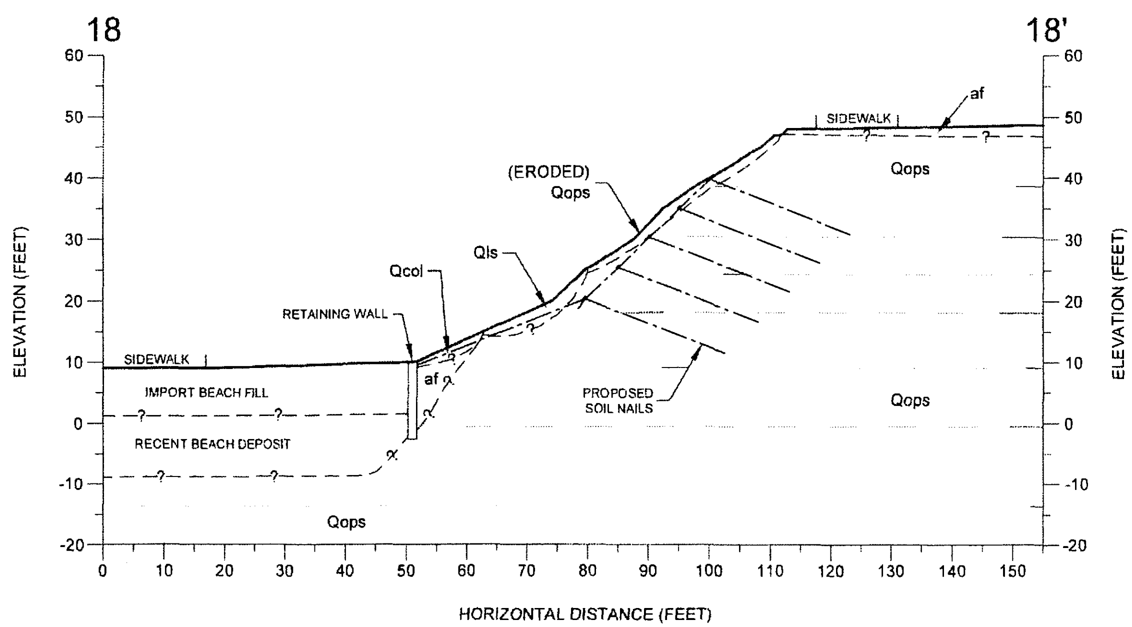
PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

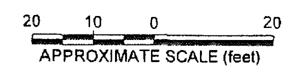
PLATE
3Q

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3Q

PLOTTED: 27 Apr 2010, 5:39pm, dfahmey





PROJECT NO.	107104	**GEOLOGIC CROSS-SECTION 18-18'**
DRAWN:	4/2010	**STATION 39+18.30**
DRAWN BY:	MRG	
CHECKED BY:	JR	PROPOSED SLOPE IMPROVEMENTS
FILE NAME:		BLUFF PARK
107104p3A to 3S_CS.dwg		LONG BEACH, CALIFORNIA

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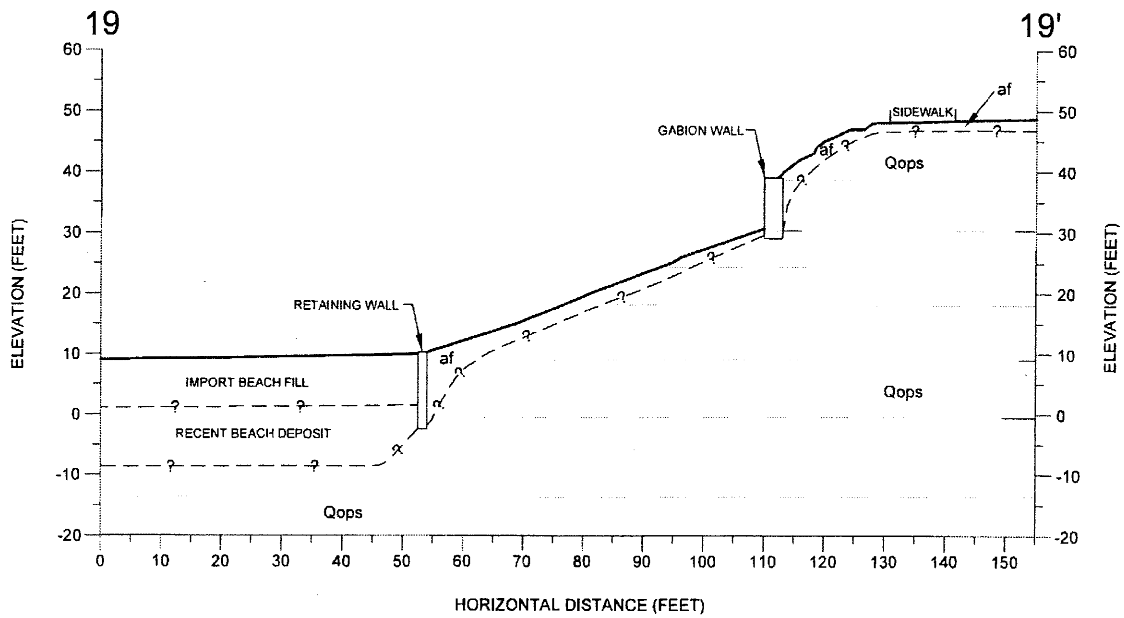
PLATE
3R

PLOTTED: 28 Apr 2010, 12:52pm, dfahmey
LAYOUT: 3R
CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\
ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA



20 10 0 20

APPROXIMATE SCALE (feet)

PROJECT NO.	107104
DRAWN	4/2010
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104p3A to 3S_CS.dwg	

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GEOLOGIC CROSS-SECTION 19-19'
STATION 41+14.68

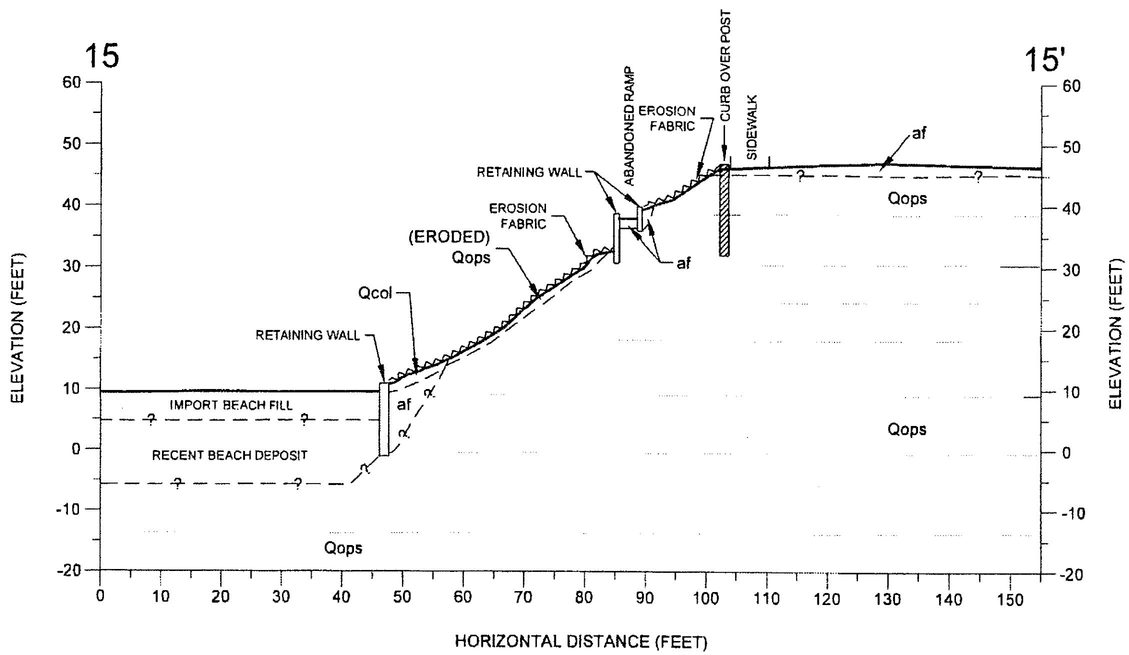
PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
3S

PLOTTED: 27 Apr 2010, 5:44pm, dfahmey

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 3S

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA



EXPLANATION

af	ARTIFICIAL FILL
Qcol	COLLUVIUM
Qops	QUATERNARY OLD PARALIC DEPOSITS
— — ? — —	APPROXIMATE GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)
·······	GENERALIZED BEDDING

NOTE: EITHER THE POST OR THE DEADMAN ANCHOR IS USED, NOT BOTH AT THE SAME LOCATION.

20 10 0 20
APPROXIMATE SCALE (feet)

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PROJECT NO.	107104
DRAWN:	4/2010
DRAWN BY:	DMF
CHECKED BY:	JR
FILE NAME: 107104p4A to 4B_CS.dwg	

EROSION CONTROL CONCEPT

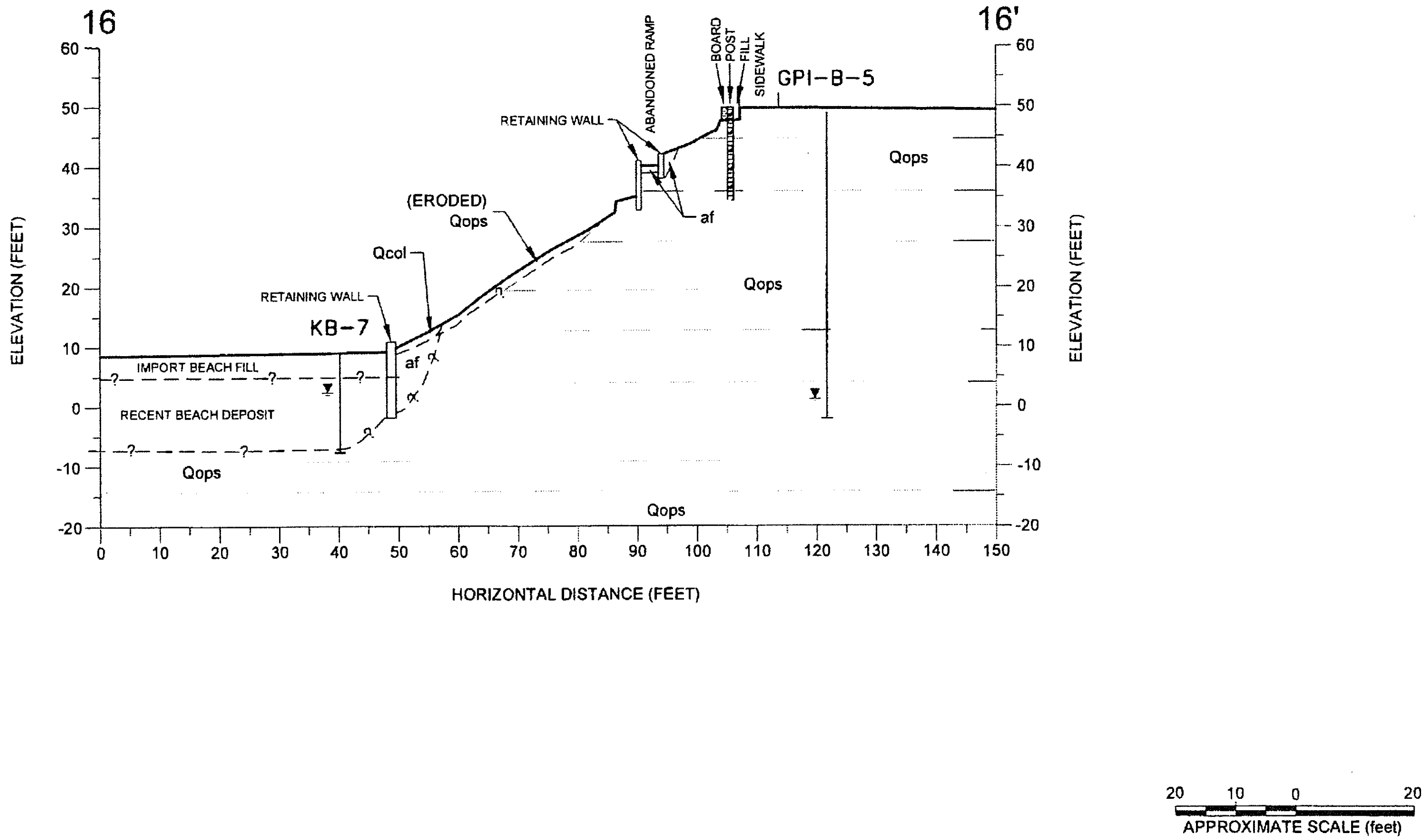
GEOLOGIC CROSS-SECTION 15-15'
STATION 28+62.74

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
4A

PLOTTED: 27 Apr 2010, 7:47pm, dfahmey   CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\   LAYOUT: 4a

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 107104 DRAWN: 4/2010 DRAWN BY: DMF CHECKED BY: JR FILE NAME: 107104p4A to 4B_CS.dwg	**POST AND BOARD CONCEPT** **GEOLOGIC CROSS-SECTION 16-16'** **STATION 34+11.35** PROPOSED SLOPE IMPROVEMENTS BLUFF PARK LONG BEACH, CALIFORNIA	PLATE **4B**

CAD FILE: L:\2010\CADD\107104\LongBeachBluffs_CS\ LAYOUT: 4b PLOTTED: 27 Apr 2010, 7:47pm, dfahmey

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

APPENDIX A

FIELD EXPLORATION

APPENDIX A
FIELD EXPLORATION

GENERAL

The field exploration consisted of several site visits, geological mapping and drilling of borings. Eight borings were drilled on November 4 and 5, 2009 by Cal Pac of Calimesa, California. An additional three borings were drilled on November 17, 2009. The borings were drilled to depths ranging between 16½ to 51½ feet, using limited-access hollow-stem auger drilling equipment. Boring locations were estimated by rough measurement and pacing from fixed references. At the completion of drilling, the boreholes were backfilled with the excavated soils and tamped. In addition to our current field exploration program, three borings were previously drilled and three cone penetration tests were performed within the project site by GPI. The borings and cone penetration test locations are shown on Plates 2A through 2E.

SAMPLING

A California sampler was used to obtain relatively undisturbed samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.4-inch I.D. split barrel shaft that is pushed or driven a total of 18 inches into the soil at the bottom of the boring. The soil was retained in seven 1-inch brass rings for laboratory testing. The sampler was driven using a 140-pound automatic hammer falling 30 inches. The total number of blows required to drive the sampler for the last 12 inches is termed blow count and is recorded on the Logs of Borings.

Samples were also obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18 inches. The sampler was driven using a 140-pound hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the SPT blow count (N) and is recorded on the Logs of Borings. The procedures we employed in the field are generally consistent with those described in ASTM Standard Test Method D1586. Bulk samples of the soils were retrieved directly from the auger blades.

BORING LOGS

The Logs of Borings are presented as Plates A-2 through A-12. An explanation to the logs is presented as Plates A-1a and A-1b. The Logs of Borings describe the earth materials encountered, samples obtained and show the results of laboratory testing. The logs also show the boring number, drilling date and the name of the drilling subcontractor. The boundaries between soil types shown on the logs are approximate because the transitions between different soil layers may be gradual.

Date Drilled:
Drilled By:
Drilling Method:
Logged By:

Water Depth:
Date Measured:
Reference Elevation:
Datum:

Elevation (feet) Depth	Sample	Sample No.	Blow Count (Blows/ft.)	Graphic Log	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		1	6			108	10	DS, SE
		2	12					GS
5								
	(1)	(2)	(3)	(4)	(5)	(6)	(6)	(7)

NOTES ON FIELD INVESTIGATION

1. SAMPLE – Graphical representation of sample type as shown below.
   Split Spoon – Standard Penetration Test Sample (SPT)
   Drive Sample – California Sample (Cal)
   Bulk Sample – Obtained by collecting cuttings in a plastic bag
   Tube Sample – Shelby/Pitcher Tube Sample
2. SAMPLE NO. – Sample Number
3. BLOWS/FT – Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil at the bottom of the hole with a standard (140 lb) hammer dropping a standard 30 inches. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586.

   SCR/RQD – Sample Core Recovery (SCR) in percent (%) and Rock Quality Designation (RQD) in percent (%). RQD is defined as the percentage of core in each run which the spacing between natural fractures is greater than 4 inches. Mechanical breaks of the core are not considered.
4. GRAPHIC LOG – Standard symbols for soil and rock types, as shown on plate A-1b.
5. GEOTECHNICAL DESCRIPTION

   Soil – Soil classifications are based on the United Soil Classification System per ASTM D-2987, and designations include consistency, moisture, color and other modifiers. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.

   Rock – Rock classifications generally include a rock type, color, moisture, mineral constituents, degree of weathering, alteration, and the mechanical properties of the rock. Fabric, lineations, bedding spacing, foliations, and degree of cementation are also presented where appropriate.

   Description of soil origin or rock formation is placed in brackets at the beginning of the description where applicable, for example, Residual Soil.
6. DRY DENSITY, MOISTURE CONTENT: As estimated by laboratory or field testing.
7. ADDITIONAL TESTS – (Indicates sample tested for properties other than the above):

   MAX – Maximum Dry Density
   GS – Grain Size Distribution
   SE – Sand Equivalent
   EI – Expansion Index
   CHEM – Sulfate and Chloride Content, pH, Resistivity
   PM – Permeability
   UU – Unconsolidated Undrained Triaxial
   SG – Specific Gravity
   HA – Hydrometer Analysis
   AL – Atterberg Limits
   RV – R-Value
   CN – Consolidation
   CU – Consolidation Undrained Triaxial
   CD – Consolidated Drained Triaxial
   PP – Pocket Penetrometer
   WA – Wash Analysis
   DS – Direct Shear
   CP – Collapse Potential
   UC – Unconfined Compression
   T – Torvane
8. ATTITUDES – Orientation of rock discontinuity observed in bucket auger boring or rock core, expressed in strike/dip and dip angle, respectively, preceeded by a one-letter symbol denoting nature of discontinuity as shown below.

   B: Bedding Plane   J: Jointing   C: Contact   F: Fault   S: Shear

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EXPLANATION OF LOGS

PLATE
A-1a

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COURSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COURSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COURSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
	HIGHLY ORGANIC SOILS		PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS	
	SILTSTONES		SH	
	CLAYSTONES		CS	
	LIMESTONES		LS	
	SHALE		SL	

CONSISTENCY CRITERIA BASED ON FIELD TESTS

RELATIVE DENSITY - COARSE - GRAIN SOIL

RELATIVE DENSITY	SPT * (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	0 - 15
Loose	4 - 10	15 - 35
Medium Dense	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very Dense	>50	85 - 100

CONSISTENCY- FINE-GRAIN SOIL

		TORVANE	POCKET ** PENETROMETER
CONSISTENCY	SPT (# blows/ft)	UNDRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2 - 4	0.13 - 0.25	0.25 - 0.5
Medium Stiff	4 - 8	0.25 - 0.5	0.5 - 1.0
Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0
Very Stiff	15 - 30	1.0 - 2.0	2.0 - 4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM-1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

Date Drilled:	11/5/09	Water Depth:	40 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/05/09
Drilling Method:	Hollow Stem Auger	Elevation:	45 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				TOPSOIL: Clayey Sand (SC): dark brown, moist			
				FILL: Lean Clay with Sand (CL): brown, moist, very stiff			
		1		**<u>OLD PARALIC DEPOSITS (Qops)</u>** **Lean Clay (CL):** mottled brown and gray, moist, hard, sandy silt lenses and layers, cemented thin layers		13.4	
40	5	2	36		107	19.2	DS WA (92% fines)
		3	44			17.7	
35	10	4	57	**Silty Sand (SM):** reddish and yellowish brown, moist, dense, pin hole porosity, weakly cemented	118	6.9	WA (31% fines)
		5	45	**Sand with Silt (SP-SM):** yellowish brown, moist, dense, fine to medium grained, very weakly cemented		4.7	
30	15	6	46	**Sandy Lean Clay (CL):** brown and yellowish brown, moist, hard, interbeds of fine weakly cemented sand	117	14.1	WA (62% fines)
25	20	7	50	**Silty Sand (SM):** yellowish brown, moist, dense, fine grained, gray sandy silt and sandy clay inclusions, weakly cemented thin layers of sand to 22 feet	103	12.9	
20	25	8	63		102	13.8	
				Sand with Silt (SP-SM): light brown to grayish brown, moist, very dense, fine grained, very weakly cemented			

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-1

PLATE A-2a

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			9	82		**Sand with Silt (SP-SM):** light brown to grayish brown, moist, very dense, fine grained, very weakly cemented *(continued)*	107	8.4	DS WA (10% fines)
10	35		10	86			100	3.6	
5	40		11	25/50-6"			100	20.8	
0	45		12	80			105	21.8	
-5	50		13	23/50-6"			97	26.2	

Total depth 51 feet
Hole backfilled and tamped
Groundwater encountered at 40 feet
Hole drilled about 7 feet from the sidewalk paralell to the top of slope

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-1

PLATE A-2b

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/4/09	Water Depth:	40 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/04/09
Drilling Method:	Hollow Stem Auger	Elevation:	45 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				TOPSOIL: Clayey Sand (SC): dark brown, moist			
				FILL: Lean Clay with Sand (CL): brown, moist, very stiff			
		1		**OLD PARALIC DEPOSITS (Qops)**		15.4	
				Lean Clay with Sand (CL): mottled brown, olive brown and gray, moist, hard, sandy silt lenses and layers			
40	5	2	21	LL=32 PL=18 PI =14		15.0	AL WA (85% fines)
		3	25			22.1	
35	10	4	35		114	14.1	DS
		5	34			19.8	WA (71% fines
30	15	6	33		106	22.1	WA (83% fines)
25	20	7	72	**Poorly Graded Sand (SP):** light grayish brown, moist, dense	106	4.5	
20	25	8	76	**Sand with Silt (SP-SM):** brown to grayish brown, moist, dense, fine grained, very weakly cemented	108	6.4	
15							

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-2

PLATE A-3a

Drafted By: [signature] Reviewed By: [signature]

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			9	65		**Sand with Silt (SP-SM):** brown to grayish brown, moist, dense, fine grained, very weakly cemented *(continued)*	100	7.2	
10	35		10	83			101	4.9	
5	40 ▼		11	28/50-5"		**Poorly Graded Sand with Silt (SP-SM):** brown, wet, very dense, fine grained	97	25.3	WA (6% fines)
0	45		12	20/50-5"			103	20.7	
-5	50		13	14/36 50-3"			107	26.2	
						Total depth 51.3 feet Hole backfilled and tamped Groundwater encountered at 40 feet Hole drilled about 7 feet from the sidewalk paralell to the top of slope			

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-2

PLATE
A-3b

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/4/09	Water Depth:	40 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/04/09
Drilling Method:	Hollow Stem Auger	Elevation:	46 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
45						**TOPSOIL: Clayey Sand (SC):** dark brown, moist			
						FILL **Silty Sand (SM):** reddish brown, moist, medium dense			
			1			**OLD PARALIC DEPOSITS (Qops)**		8.5	
40	5		2	46		**Clayey Sand (SC):** reddish brown, moist, medium dense to dense	125	12.9	WA (46% fines)
			3	5		**Silty Sand (SM):** yellowish to reddish brown, moist, loose, fine grained		7.6	
35	10		4	12		**Poorly Graded Sand with Silt (SP-SM):** yellowish to reddish brown, moist, loose, fine grained, very weakly cemented	102	4.6	WA (5% fines)
			5	20		**Silty Sand (SM):** reddish brown, moist, medium dense, fine grained		11.3	
30	15		6	37		**Poorly Graded Sand (SP):** grayish brown, moist, medium dense, fine grained	104	3.5	
25	20		7	67		**Silty Sand (SM):** brown and yellowish brown, moist, dense, fine grained, weakly cemented	110	12.3	
20	25		8	79		**Poorly Graded Sand (SP):** yellowish brown, moist, dense, fine grained, sandy silt inclusions	101	3.5	

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-3

PLATE A-4a

Drafted By: [signature] Reviewed By: [signature]

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
15			9	66		**Poorly Graded Sand (SP):** yellowish brown, moist, dense, fine grained, sandy silt inclusions *(continued)*	107	3.9	
10	35		10	15/50-6"		**Silty Sand (SM):** yellowish brown, moist, dense, fine grained, layers of sandy lean clay, weakly cemented	112	9.8	
5	40		11	13 18/50-5"		**Poorly Graded Sand with Silt (SP-SM):** light greyish brown, moist, very dense, fine grained, lean clay inclusions	114	14.0	
0	45		12	20/50-6"		**Poorly Graded Sand (SP):** greyish brown, wet, very dense, fine grained	107	17.9	
-5	50		13	26/50-6"		**Sand with Silt (SP-SM):** gray, moist, very dense, fine grained	105	22.9	

Total depth 51 feet
Hole backfilled and tamped
Groundwater encountered at 40 feet
Hole drilled about 5 feet from the sidewalk paralell to the top of slope

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-3

PLATE
A-4b

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/17/09	Water Depth:	42 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/17/09
Drilling Method:	Hollow Stem Auger	Elevation:	46 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				TOPSOIL: Silty Sand (SM): brown, moist			
45		1		**FILL** **Silty Sand (SM):** reddish brown, moist, medium dense		5.6	
				OLD PARALIC DEPOSITS (Qops) **Silty Sand and Sandy Silt (SM/ML):** brown, moist, medium dense, moderately cemented, fine grained			
	5	2	25/50-6"				
40					109	12.6	WA (61% fines)
		3	22				
						17.6	
	10	4	52	**Sandy Lean Clay (CL):** brown, moist, hard, weakly cemented	114	19.2	
35							WA (68% fines)
		5	24	**Silty Sand (SM):** brown to reddish brown, moist, medium dense, fine to medium grained, weakly cemented		12.7	
	15	6	38		103	8.1	
30							WA (20% fines)
	20	7	33	**Sandy Lean Clay (CL):** gray, moist, very stiff	104	23.3	
25				**Silty Sand (SM):** brown and yellowish brown, moist, dense, fine grained, weakly cemented			
	25	8	41		120	9.3	
20				**Sandy Lean Clay (CL):** brown, moist, hard			
				Silty Sand (SM): brown, moist, dense, fine grained, lean clay inclusions			

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-4

PLATE A-5a

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Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
15			9	55		**Silty Sand (SM):** brown, moist, dense, fine grained, lean clay inclusions *(continued)*	105	20.7	
						Poorly Graded Sand (SP): mottled brownish grey, moist, dense, fine grained, weakly cemented			
10	35		10	19/37 50-5"			101	4.6	
5	40			23/50-6"			103	4.9	
						Fat Clay (CH): dark reddish brown, wet, medium stiff to hard, layers of silty sand			
0	45		12	4/23 50-5"			92	47.1	
						Poorly Graded Sand (SP): brownish gray, wet, very dense, fine grained			
	50		13	33/50-4"			104	21.5	
						Total depth 51 feet Hole backfilled and tamped Groundwater encountered at 42 feet Hole drilled about 9 feet from the sidewalk paralell to the top of slope			

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-4

PLATE A-5b

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/17/09	Water Depth:	43 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/17/09
Drilling Method:	Hollow Stem Auger	Elevation:	49 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						TOPSOIL: Silty Sand (SM): brown, moist			
			1			**FILL** **Silty Sand (SM):** brown, moist, medium dense, trace gravel		6.5	
			2			**OLD PARALIC DEPOSITS (Qops)** **Silty Sand (SM):** brown to reddish brown, moist, medium dense, fine to medium grained, weakly cemented		8.4	
45	5		3	44		**Silty Sand and Sandy Silt (SM/ML):** brown, moist, dense, moderately cemented, fine grained	116	13.4	
			4	12		**Silty Sand (SM):** brown, moist, medium dense, weakly cemented		8.9	
40	10		5	15			99	6.8	WA (31% fines)
			6	41				13.1	
35	15		7	66		**Sandy Silt and Silty Sand (ML/SM):** olive to yellowish brown, moist, dense, weakly cemented, fine grained, concretions	110	16.5	DS WA (58% fines)
30	20		8	53		**Poorly Graded Sand (SP):** grayish brown, moist, dense, fine to medium grained, weakly cemented	97	2.4	
25	25		9	68		**Silty Sand (SM):** mottled brown and gray, moist, dense, weakly cemented, layers and inclusions of fine sand with silt	119	8.7	
20									

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

PLATE A-6a

PROJECT NO. 107104

LOG OF BORING KB-5

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			10	76		**Silty Sand (SM):** mottled brown and gray, moist, dense, weakly cemented, layers and inclusions of fine sand with silt *(continued)*	100	9.4	WA (20% fines)
15	35		11	66		**Sand with Silt (SP-SM):** yellowish gray and brown, moist, dense, fine, very weakly cemented	97	4.9	
10	40		12	72			96	9.9	
5	45		13	66		**Poorly Graded Sand (SP):** yellowish brown, wet, dense, fine to medium grained, trace shells	105	18.7	
0	50		14	25/50-6"		-- bluish gray, fine grained	98	27.8	WA (3% fines)

Total depth 51 feet
Hole backfilled and tamped
Groundwater encountered at 43 feet
Hole drilled about 7 feet from the sidewalk paralell to the top of slope

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-5

PLATE
A-6b

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/17/09	Water Depth:	6 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/17/09
Drilling Method:	Hollow Stem Auger	Elevation:	9 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						FILL			
			1			**Poorly Graded Sand (SP):** gray at surface, brown below, moist, loose to medium dense, fine to medium grained		4.1	
5	5		2	30		-- Brownish gray	111	7.1	
						BEACH DEPOSITS			
			3	34		**Poorly Graded Sand (SP):** dark bluish gray, wet, medium dense, fine to medium grained, trace of shells		19.0	
0	10		4	25				23.4	
			5	23				19.3	
-5	15		6	41			108	18.9	
						OLD PARALIC DEPOSITS (Qops)			
-10	20		7	61		**Poorly Graded Sand with Silt (SP-SM):** dark bluish gray, wet, very dense, fine to medium grained		23.4	
-15	25		8	11/34 50-5"				17.4	
-20									

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-6

PLATE A-7a

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		9	14/50-6"				18.9	WA (10% fines)
-25					Total depth 31 feet Hole backfilled Groundwater encountered at 6 feet Hole drilled about 9 feet from the slope toe wall			

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-6

PLATE
A-7b

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/5/09	Water Depth:	7 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/05/09
Drilling Method:	Hollow Stem Auger	Elevation:	10 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						FILL			
			1			**Poorly Graded Sand (SP):** gray at surface, brown below, moist, loose to medium dense, fine to medium grained		3.6	WA (3% fines)
5	5		2	27			95	10.1	
			3	24		**BEACH DEPOSITS**		20.7	
0	10		4	36		**Poorly Graded Sand (SP):** grayish brown to dark bluish gray, wet, medium dense, fine to medium grained, trace of shells	101	21.9	
			5	60				19.3	
-5	15		6	21				18.4	
						OLD PARALIC DEPOSITS (Qops) Sandy Silt (ML): brown			

Total depth 16.5 feet
Hole backfilled
Groundwater encountered at 7 feet
Hole drilled about 10 feet from the slope toe wall

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

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PROJECT NO. 107104

PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-7

PLATE A-8

Drafted By: [signature] Reviewed By: [signature]

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/5/09	Water Depth:	5 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/05/09
Drilling Method:	Hollow Stem Auger	Elevation:	9 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						FILL			
						Poorly Graded Sand (SP): gray at surface, brown below, moist, loose to medium dense, fine to medium grained			
			1					2.2	
5									
	5		2	16			102	21.8	
			3	24		**BEACH DEPOSITS**			
0								17.9	
						Poorly Graded Sand (SP): brownish gray to dark bluish gray, wet, medium dense, fine to medium grained, trace of shells			
	10		4	26			104	22.4	
			5	25					
-5								24.1	
	15		6	29		-- with silt at 15 feet		17.4	WA (12% fines)
						OLD PARALIC DEPOSITS (Qops)			
						Sandy Silt (ML): brown			

Total depth 16.5 feet
Hole backfilled
Groundwater encountered at 5 feet
Hole drilled about 10 feet from the slope toe wall

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-8

PLATE A-9

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/5/09	Water Depth:	4 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/05/09
Drilling Method:	Hollow Stem Auger	Elevation:	11 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
10						**FILL** Silty Sand (SM): dark yellowish brown, moist, loose, fine grained			
			1					10.7	WA (18% fines)
	5		2	2		**Poorly Graded Sand with Silt (SP-SM):** yellowish brown, wet, loose, fine grained			
-5								24.1	
			3	2		**COLLUVIUM** **Silty Sand (SM):** reddish brown, wet, loose, fine grained		21.0	
	10		4	5		**Poorly Graded Sand with Silt (SP-SM):** dark yellowish brown, wet, loose, fine grained		24.3	
-0									
			5	8		**OLD PARALIC DEPOSITS (Qops)** **Silt with Sand (ML):** brown, medium stiff, very moist, interbedded with sand, layers of sandy clay		28.3	
	15		6	9		**Poorly Graded Sand with Silt (SP-SM):** brown, wet, loose, fine grained, trace of shells, lenses of lean clay		25.6	
-5									

Total depth 16.5 feet
Hole backfilled
Groundwater encountered at 4 feet
Hole drilled about 6 feet behind the wall

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-9

PLATE A-10

Drafted By: [signature] Reviewed By: [signature]

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/5/09	Water Depth:	4 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/05/09
Drilling Method:	Hollow Stem Auger	Elevation:	8 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						FILL			
5			1			**Poorly Graded Sand (SP):** gray at surface, grayish brown below, moist, loose to medium dense, fine grained		8.2	
	5		2	32			105	18.9	
0			3	26		**BEACH DEPOSITS**			
						Poorly Graded Sand (SP): gray to dark bluish gray, wet, medium dense, fine to medium grained, trace of shells		20.6	
	10		4	38		**Poorly Graded Sand with Silt (SP-SM):** dark bluish gray, wet, medium dense, fine to medium grained, trace of shells	109	20.8	WA (11% fines)
-5			5	9		**Poorly Graded Sand (SP):** dark bluish gray, wet, medium dense, fine to medium grained, trace shells		23.5	
	15		6	17				19.6	

Total depth 16.5 feet
Hole backfilled
Groundwater encountered at 4 feet
Hole drilled about 10 feet from the slope toe wall

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

PLATE A-11

PROJECT NO. 107104

LOG OF BORING KB-10

GEOTECH DB BLUFF.GPJ KA RDLND.GDT 12/14/09

Drafted By: pm Reviewed By: JRoy

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	11/5/09	Water Depth:	3 feet
Drilled By:	Cal Pac Drilling	Date Measured:	11/05/09
Drilling Method:	Hollow Stem Auger	Elevation:	8 feet (approx.)
Logged By:	JBR	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						FILL			
5			1			**Poorly Graded Sand (SP):** gray at surface, brown below, moist, loose to medium dense, fine grained		3.8	
	5		2	29		-- wet	102	18.5	
0			3	27		**BEACH DEPOSITS** **Poorly Graded Sand (SP):** gray to dark bluish gray, wet, medium dense, fine to medium grained, trace of shells		12.3	
	10		4	24		Cobble or Boulder at 6 feet			
-5			5	12				22.6	
	15		6	16		Thin layer of brown sand		20.9	

Total depth 16.5 feet
Hole backfilled
Groundwater encountered at 3 feet
Hole drilled about 5 feet from the slope toe wall

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

LOG OF BORING KB-11

PLATE A-12

Drafted By: Reviewed By:

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

APPENDIX B

LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed on selected, representative samples as an aid in classifying the soils and to evaluate physical properties of the soils that may affect design and construction procedures. The tests were performed in general conformance with the current ASTM Standards. A description of the laboratory-testing program is presented below.

SOIL CLASSIFICATION

After visual classification of the soil samples in the field, the soil samples were transported to our laboratory. Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) and/or ASTM D 2488. Several classifications were substantiated with No. 200 Sieve Wash testing. Soil classifications are indicated on the logs of borings in Appendix A.

PLASTICITY INDEX

Plasticity Index testing was performed on one sample to evaluate the plasticity characteristics and to aid in the classification of the soil. The test was performed in accordance with ASTM Standard Test Method D 4318, Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The results are presented on the Log of Boring, Plate A-3a.

WASH SIEVE

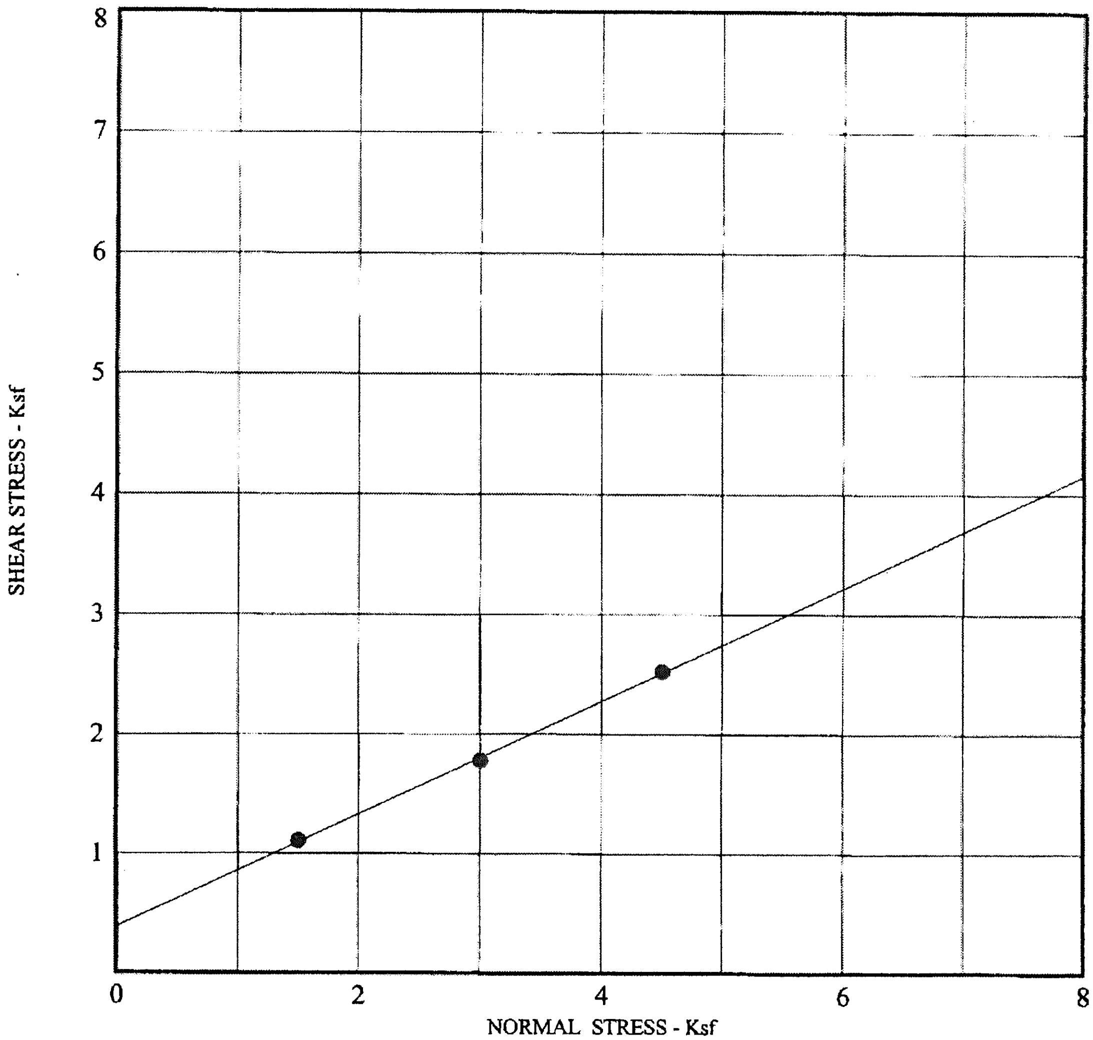
The percentage of material passing the No. 200 sieve of selected soil samples was determined by wash sieving in accordance with ASTM Standard Test Method D1140. The No. 200 sieve test results are presented on the Logs of Boring.

MOISTURE CONTENT AND UNIT WEIGHT

The field moisture content and dry unit weight were determined for relatively undisturbed soil samples obtained during field exploration. The field moisture content was obtained by methods described in ASTM Test Method D2216. In place dry unit weight was computed using the net weight of the entire relatively undisturbed sample. The field moisture content and in place dry unit weight results are presented on the boring logs (Plates A-2 through A-12).

DIRECT SHEAR

The shear strength parameters, friction angle and cohesion of four relatively undisturbed samples were evaluated in general conformance with the procedures outlined in ASTM D3080. This procedure utilizes an apparatus that allows a cylindrical soil sample to be sheared along its mid-height. For each direct shear test, three saturated specimens were sheared, each at different vertical (normal) stresses. For each normal stress, a horizontal "shearing stress" was applied at a constant rate of strain to the sample. Readings of both vertical and horizontal movements were recorded during shearing of the sample. The peak shear stress obtained during the test was plotted against the normal stress for each of the samples tested. The tests performed to determine the shear strength parameters for slope stability analysis were sheared slowly at rates of 0.001 and 0.0007 inch per minute for the sand and clay, respectively. The results of the direct shear tests are plotted graphically on Plates B-1 through B-5, Direct Shear Test.



Boring	KB-1
Depth (ft)	6
Friction Angle - deg	25
Cohesion (Ksf)	0.38
Moisture Content (%)	19.2
Dry Density (pcf)	107
Description	Lean Clay
Classification	CL

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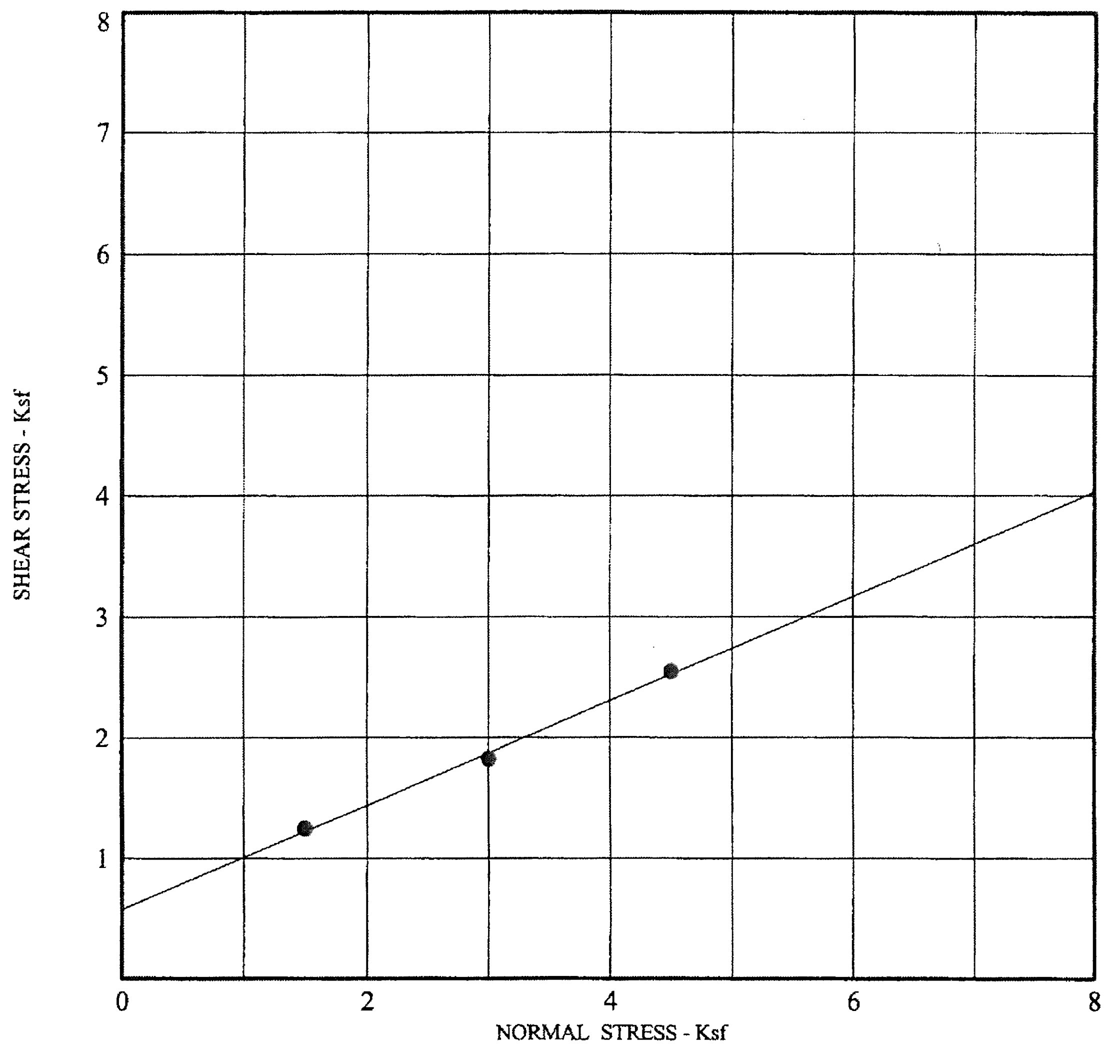
PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

DIRECT SHEAR TEST

PLATE

B-1

DIRECT SHEAR BLUFF.GPJ KA RDLND.GDT 12/14/09



Boring	KB-2
Depth (ft)	11
Friction Angle - deg	23
Cohesion (Ksf)	0.58
Moisture Content (%)	14.1
Dry Density (pcf)	114
Description	Lean Clay with Sand
Classification	CL

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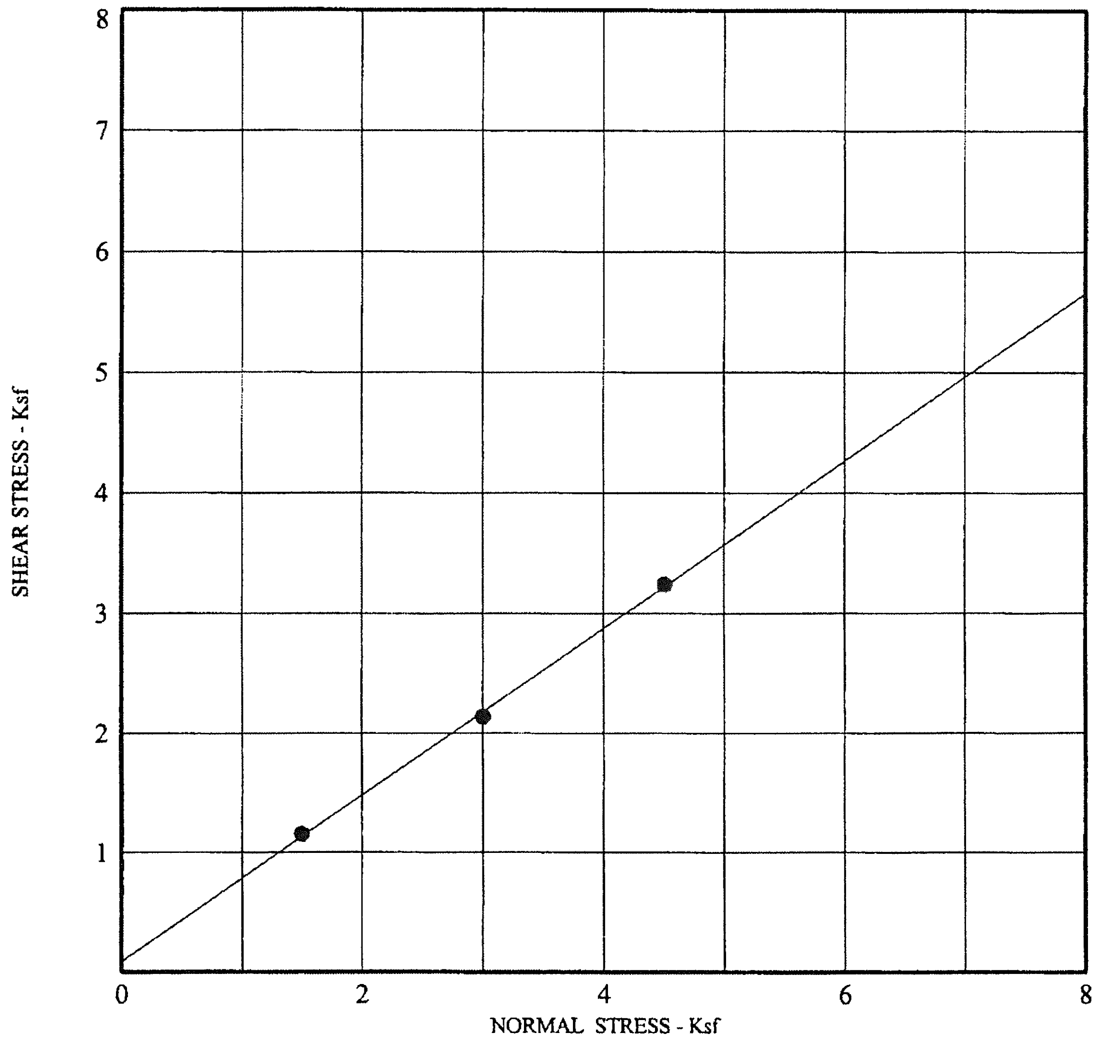
PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

DIRECT SHEAR TEST

PLATE
B-2

DIRECT SHEAR BLUFF.GPJ KA RDLND.GDT 12/14/09



Boring	**KB-3**
Depth (ft)	**31**
Friction Angle - deg	**35**
Cohesion (Ksf)	**0.09**
Moisture Content (%)	**3.9**
Dry Density (pcf)	**107**
Description	**Poorly Graded Sand**
Classification	**SP**

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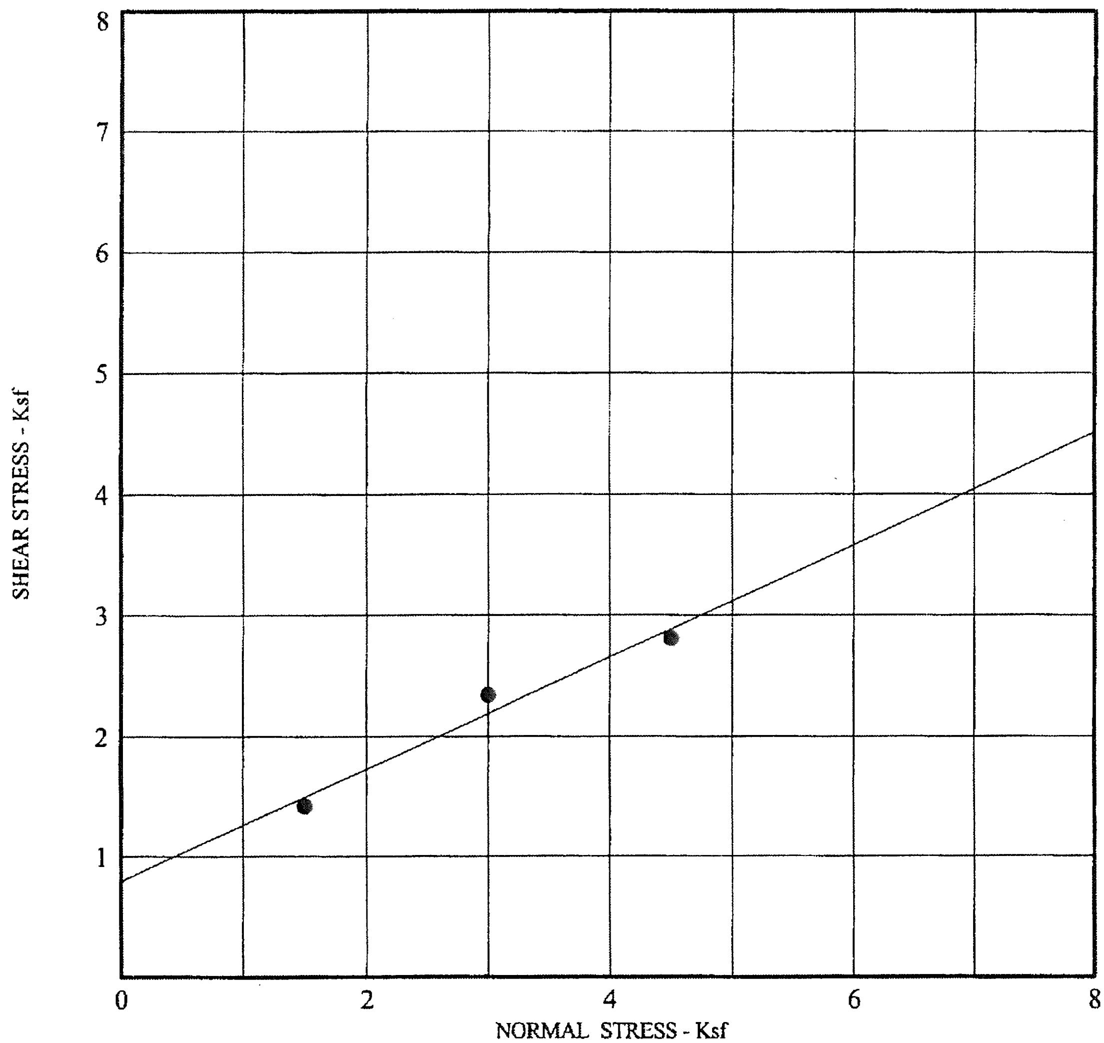
PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

DIRECT SHEAR TEST

PLATE
B-3

DIRECT SHEAR BLUFF.GPJ KA RDLND.GDT 12/14/09



Boring	KB-5
Depth (ft)	16
Friction Angle - deg	25
Cohesion (Ksf)	0.80
Moisture Content (%)	16.5
Dry Density (pcf)	110
Description	Sandy Silt to Silty Sand
Classification	ML/SM

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PROPOSED SLOPE IMPROVEMENTS
Bluff Park
Long Beach, California

DIRECT SHEAR TEST

PLATE

B-4

APPENDIX C

SLOPE STABILITY ANALYSIS

APPENDIX C

SLOPE STABILITY ANALYSIS

The following Appendix C summarizes the method employed for slope stability analysis, the parameters used, and the assumptions made. Selected computer printouts are attached in this Appendix. The results of the analysis on the existing slope are summarized in Sections 4.3. The analysis with soil nails are presented in this Appendix.

METHOD OF ANALYSIS

The selected slope sections were analyzed using the computer programs PC Stabl 5 from Purdue University, GStabl 7 from Gregory Geotechnical Software and Slide 5.0 developed by Rocscience. The limit equilibrium analysis programs model a two-dimensional slope and compute the factor of safety for various failure planes. The programs were used to search for critical failure plane surfaces for static and pseudo-static conditions, using the Janbu corrected method, simplified Bishop method, or Spencer method.

The slope stability of selected cross sections with nails was analyzed using the soil reinforcement computer program SNAILZ developed by the Caltrans Office of Roadway. SNAILZ considers force equilibrium (not moment equilibrium) and can analyze only bi-linear wedge above the facing toe and tri-linear failure wedges below the toe. Because only passive pressure is considered below the facing toe by the SNAILZ program, Slide 5 was used to check the stability of possible deep toe failures.

GEOTECHNICAL/GEOLOGIC CROSS SECTIONS

The cross sections were prepared to represent various slope configurations typical of the site. The surface topography of the profiles modeled was obtained from the site plan provided by RJM Design Group, Inc. The existing ground surface, proposed grade where applicable, and subsurface geologic conditions are shown on the cross sections.

Adjustments were made where discrepancies between our field observation and the topographic map were noted.

SHEAR STRENGTH PARAMETERS

Shear strength parameters for the stability analyses were selected based on laboratory direct shear tests performed on samples obtained from recent subsurface investigations conducted for this study, the previous laboratory tests performed on samples from the site by GPI, published correlations between the sampler blow count and the friction angle, published literature on the geotechnical properties of cemented sand on steep slopes, and professional engineering judgment based on prior experience with similar soils.

Selected Strength Parameters

The shear strengths obtained from the direct shear tests can be considered effective or drained shear strengths. The values used in the stability analyses are summarized in Table C-1.

Table C-1
Summary of Shear Strength Parameters

Deposit	Material Type	Cohesion (psf)*	Angle of Shearing Resistance (degrees)
Slope Fill	Sand and Silty Sand	50 - 125	29
Beach Fill	Sand/Sand with Silt	0	32
Beach Deposit	Sand/Sand with Silt	0	34
Colluvium	Silt, Sand & Silty Sand	50 - 125	27 - 28
Paralic Deposit	Clay	300 -350	25
Paralic Deposit	Silt	200 - 250	27
Paralic Deposit	Silty Sand	100 - 125	35
Paralic Deposit	Sand/Sand with Silt	50 - 100	36
Import Fill	Sand and Silty Sand	0 - 50	32

* No cohesion was used for sand below groundwater

Paralic Deposits

The sand friction angle (ø) generally increases as the material becomes well graded, with an increase in dry unit weight, and an increase in surface roughness. The friction angle may be about 2 degrees less when the soil becomes wet. Cementation tends to increase the cohesion of the sand and not necessarily the friction angle. The correlations between the friction angle and the sampler blow counts were based on the work by Schmertmann (1975) and Hatanaka and Uchida (1996). The following equations were used to evaluate the friction angles based on the blow counts.

$$\phi = \tan^{-1} [N60/(12.3+20.3\ \sigma_{vo}/P_a)]^{0.34} \quad \text{(Schmertmann, 1975)}$$

$$\phi = [\sqrt{15.4(N_1)_{60}}] + 20^\circ \quad \text{(Hatanaka and Uchida, 1996)}$$

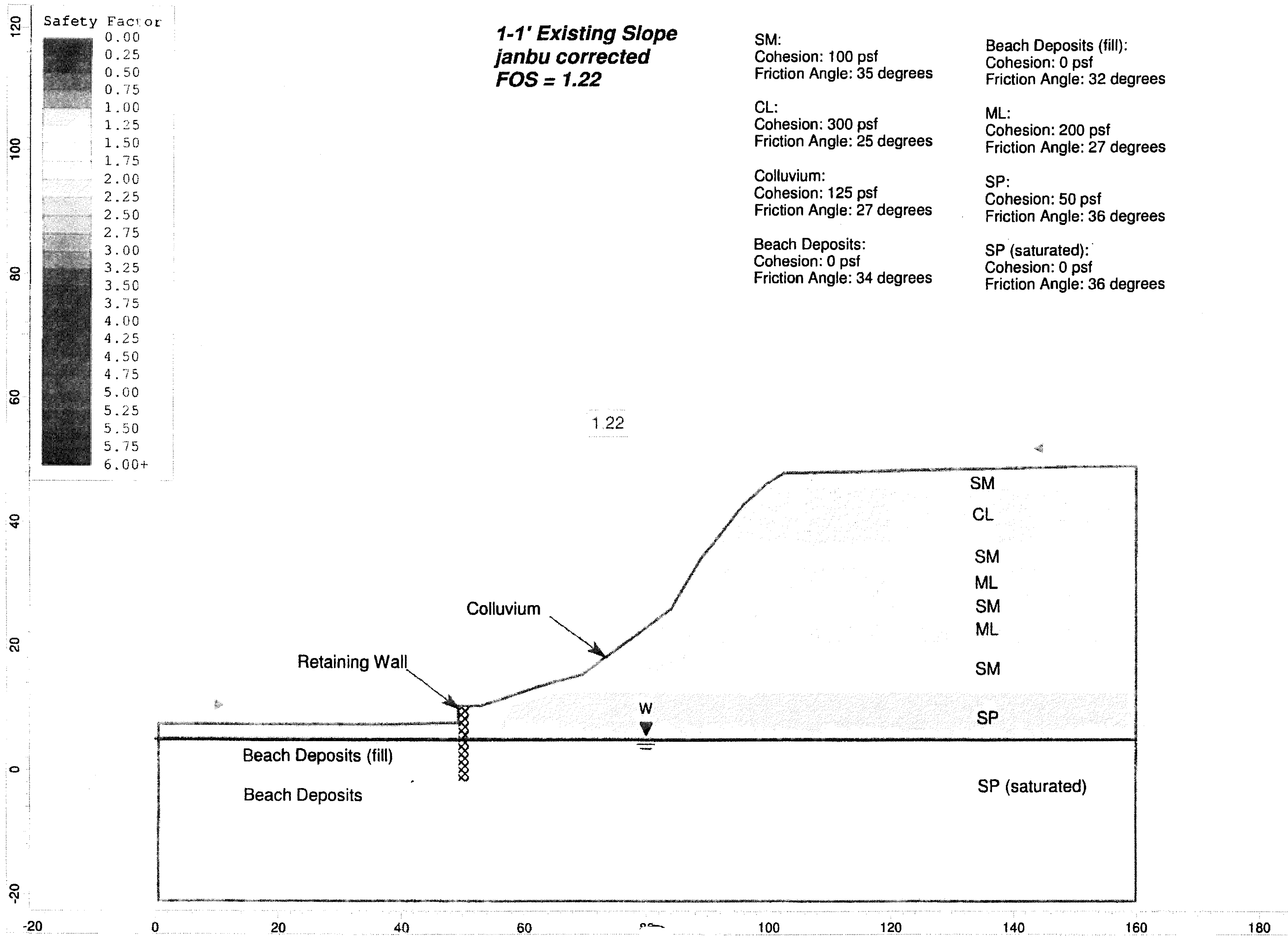
Corrections based on judgment were made for the cementation, water table location, and soil uniformity. We also checked the friction angle based on the dry density using correlations from the U.S. Navy (1971). Furthermore, Collins and Sitar have investigated the geotechnical properties of weakly and moderately cemented sand from eroding cliffs located in central California (Collins and Sitar, 2009). Their data indicated peak friction angle of 39 degrees and cohesion of 125 psf for the weakly cemented sand. GPI obtained peak friction angles between 30 and 37 degrees with corresponding cohesion ranging between 66 and 594 pcf. The average cohesion of these tests was 140 psf. For slope stability analysis, many consultants use ultimate value (i.e. shear stress at a strain beyond the peak value) as opposed to peak strength. The ultimate friction angle values may be several degrees lower than the peak values.

Colluvium, Fill and Beach Deposits

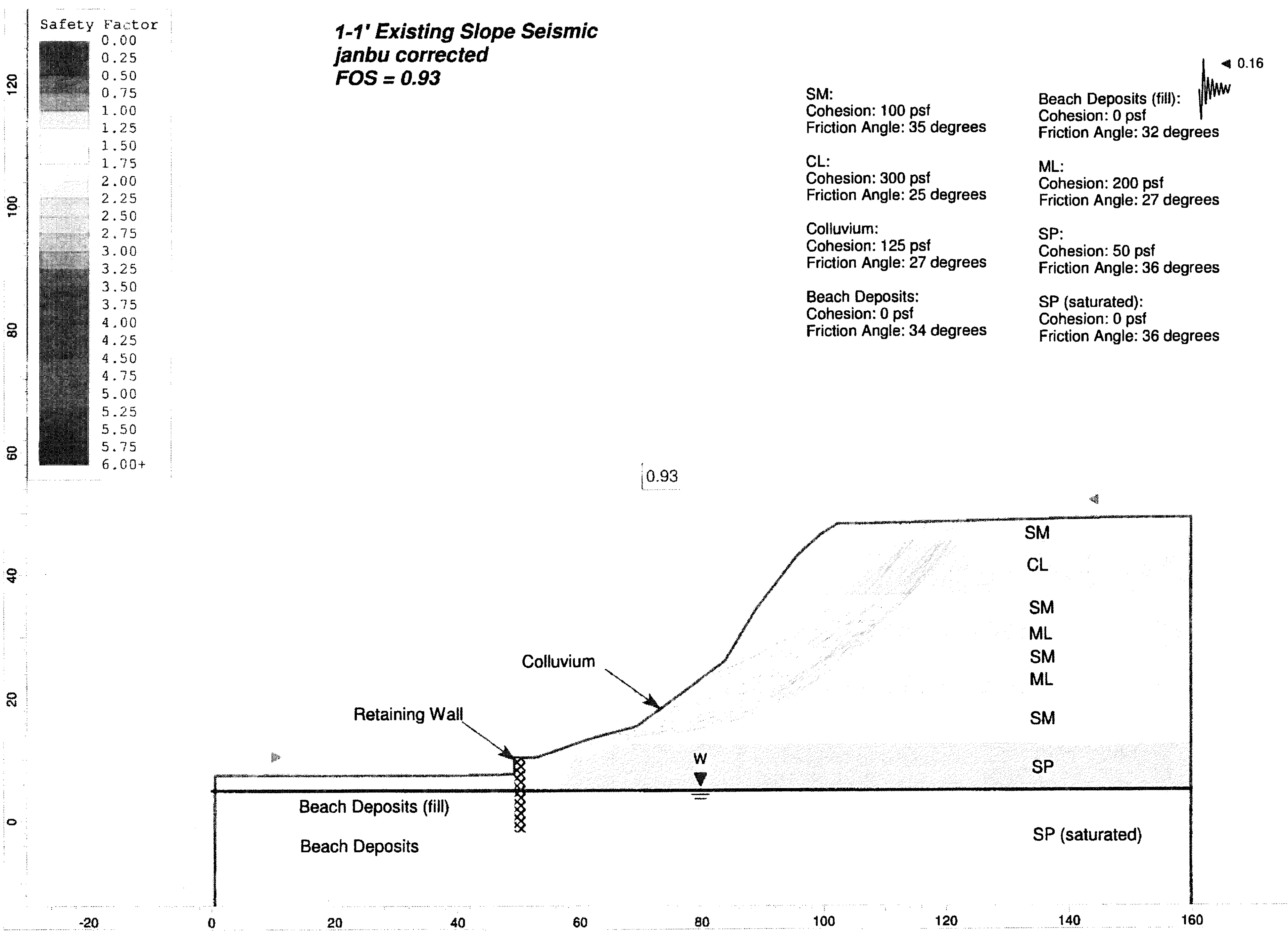
We have estimated the shear strength based on prior experience with similar soils, the blow count during sampling, and the dry unit weight of the soil where applicable.

GROUNDWATER LEVEL

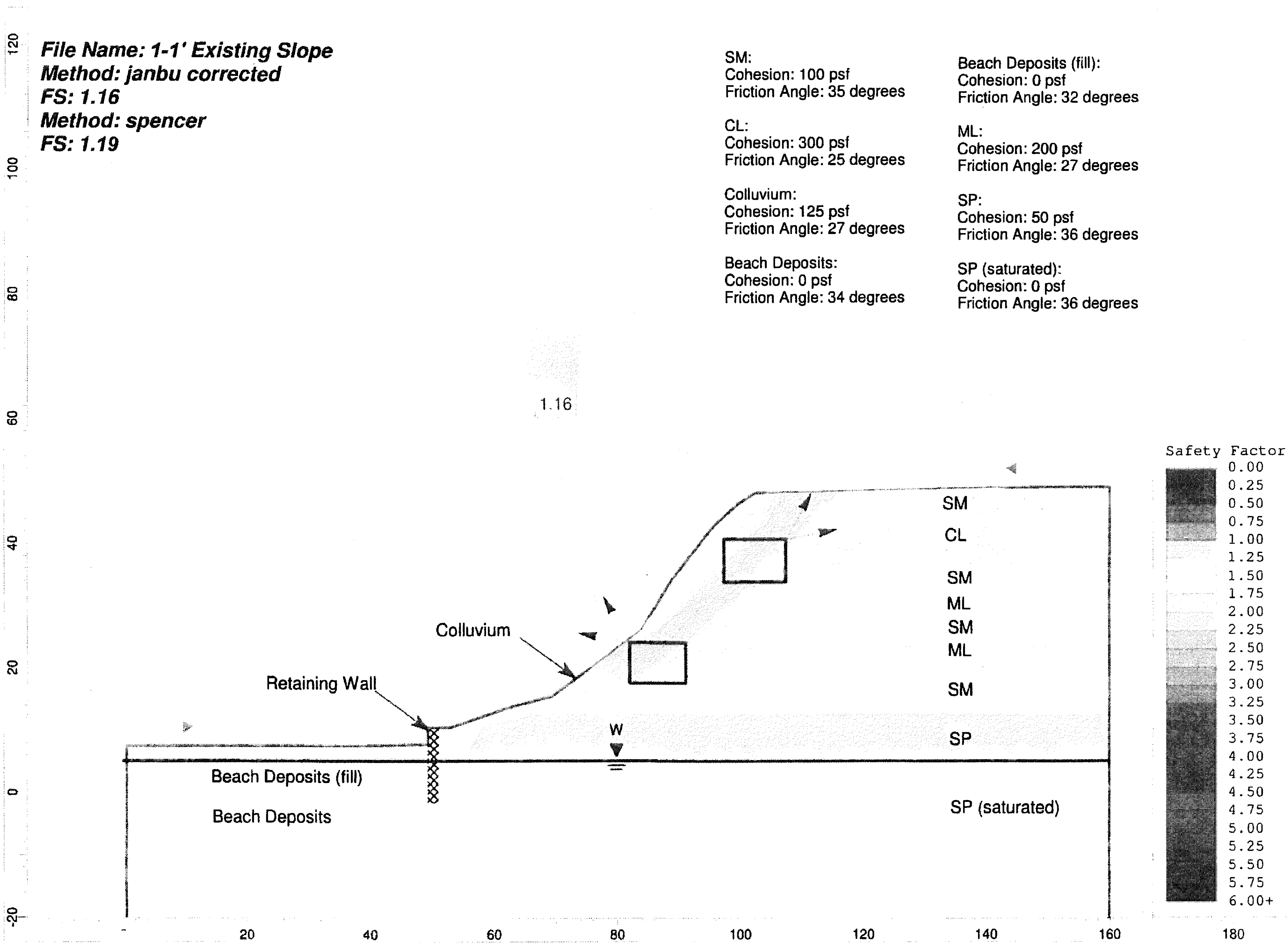
The approximate groundwater levels encountered in the borings at the time of drilling were used in the analyses. High groundwater simulations due to prolonged rains were not performed.

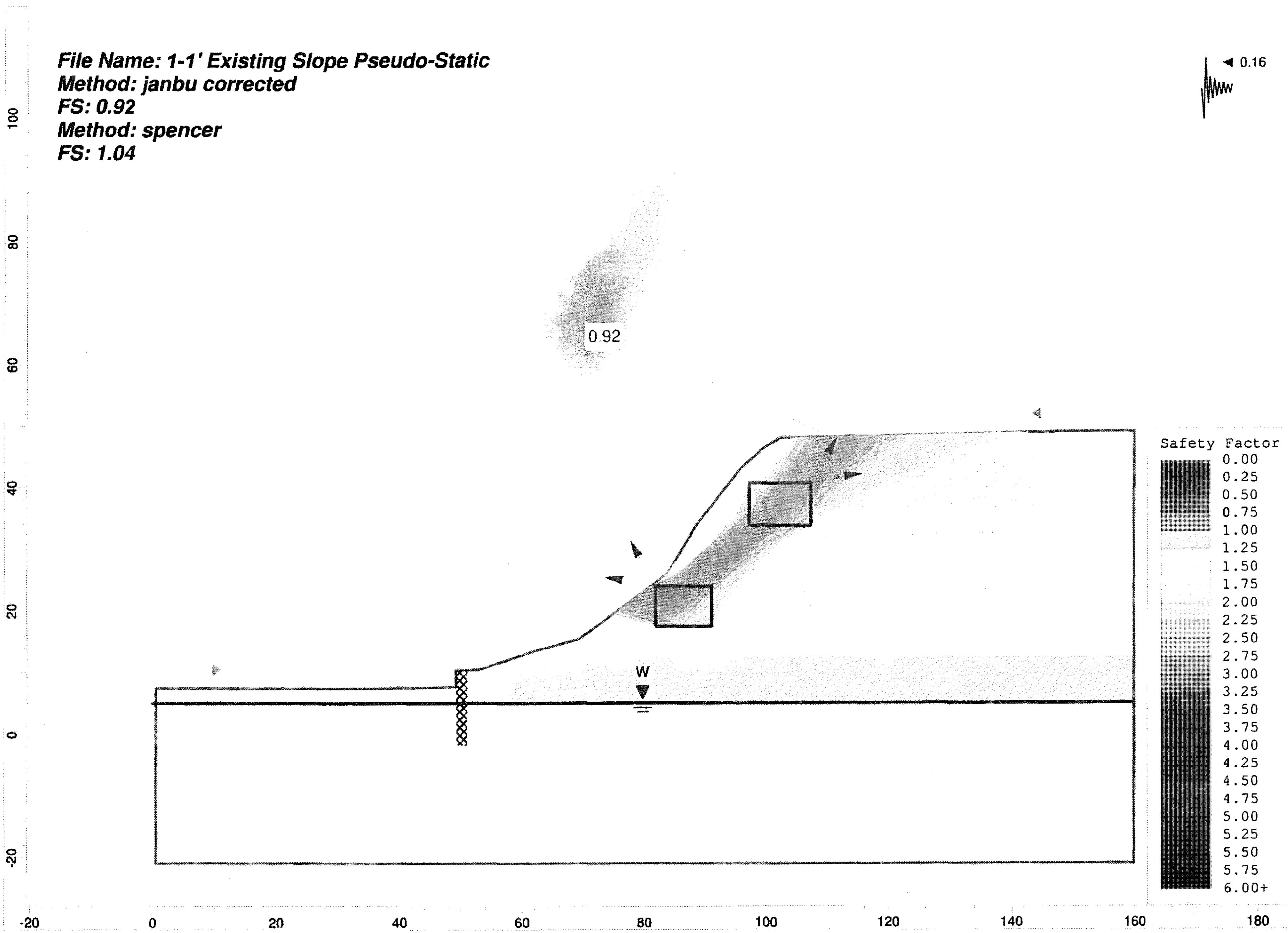


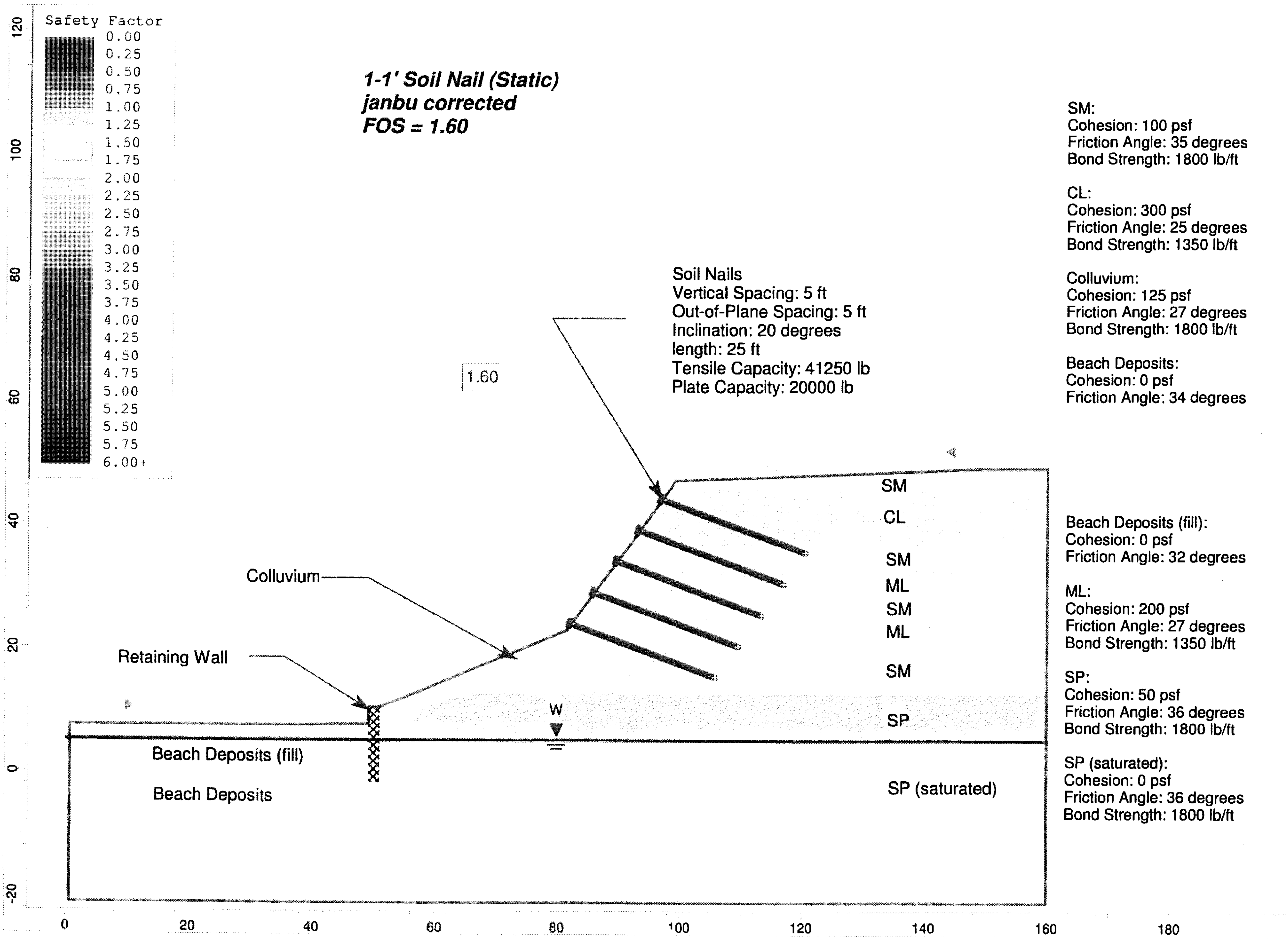
1-1' Existing Slope
janbu corrected
FOS = 1.22
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
SM:
Cohesion: 100 psf
Friction Angle: 35 degrees
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Colluvium:
Cohesion: 125 psf
Friction Angle: 27 degrees
Beach Deposits:
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill):
Cohesion: 0 psf
Friction Angle: 32 degrees
ML:
Cohesion: 200 psf
Friction Angle: 27 degrees
SP:
Cohesion: 50 psf
Friction Angle: 36 degrees
SP (saturated):
Cohesion: 0 psf
Friction Angle: 36 degrees
1.22
Colluvium
Retaining Wall
SM
CL
SM
ML
SM
ML
SM
SP
W
Beach Deposits (fill)
Beach Deposits
SP (saturated)
-20
0
20
40
60
100
120
140
160
180



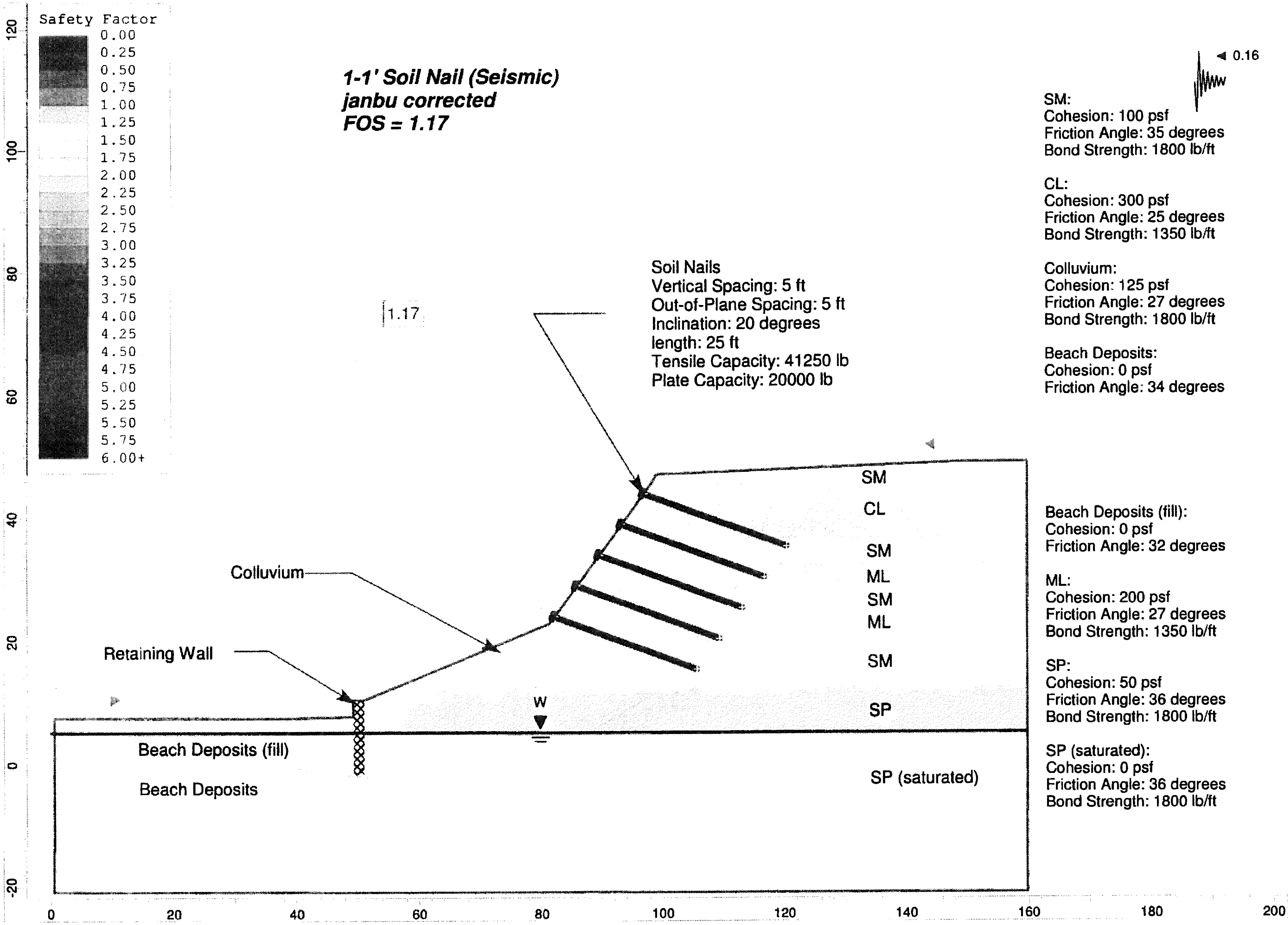
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
1-1' Existing Slope Seismic
janbu corrected
FOS = 0.93
0.16
SM:
Cohesion: 100 psf
Friction Angle: 35 degrees
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Colluvium:
Cohesion: 125 psf
Friction Angle: 27 degrees
Beach Deposits:
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill):
Cohesion: 0 psf
Friction Angle: 32 degrees
ML:
Cohesion: 200 psf
Friction Angle: 27 degrees
SP:
Cohesion: 50 psf
Friction Angle: 36 degrees
SP (saturated):
Cohesion: 0 psf
Friction Angle: 36 degrees
0.93
Colluvium
Retaining Wall
W
SM
CL
SM
ML
SM
ML
SM
SP
SP (saturated)
Beach Deposits (fill)
Beach Deposits
-20
0
20
40
60
80
100
120
140
160

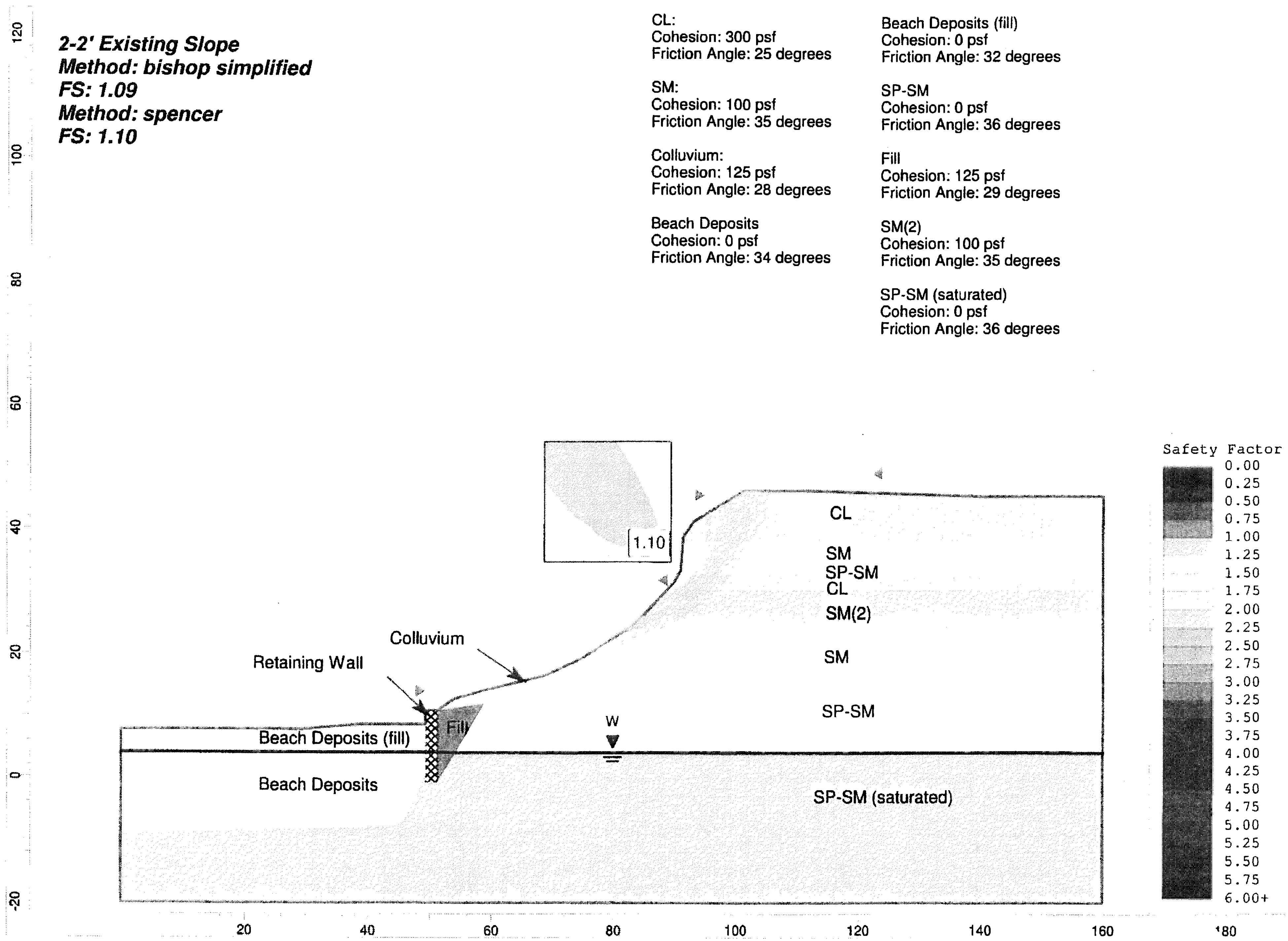

File Name: 1-1' Existing Slope
Method: janbu corrected
FS: 1.16
Method: spencer
FS: 1.19
SM:
Cohesion: 100 psf
Friction Angle: 35 degrees
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Colluvium:
Cohesion: 125 psf
Friction Angle: 27 degrees
Beach Deposits:
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill):
Cohesion: 0 psf
Friction Angle: 32 degrees
ML:
Cohesion: 200 psf
Friction Angle: 27 degrees
SP:
Cohesion: 50 psf
Friction Angle: 36 degrees
SP (saturated):
Cohesion: 0 psf
Friction Angle: 36 degrees
1.16
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
Colluvium
Retaining Wall
W
SM
CL
SM
ML
SM
ML
SM
SP
SP (saturated)
Beach Deposits (fill)
Beach Deposits
120
100
80
60
40
20
0
-20
20
40
60
80
100
120
140
160
180


File Name: 1-1' Existing Slope Pseudo-Static
Method: janbu corrected
FS: 0.92
Method: spencer
FS: 1.04
0.16
0.92
W
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
-20
0
20
40
60
80
100
120
140
160
180

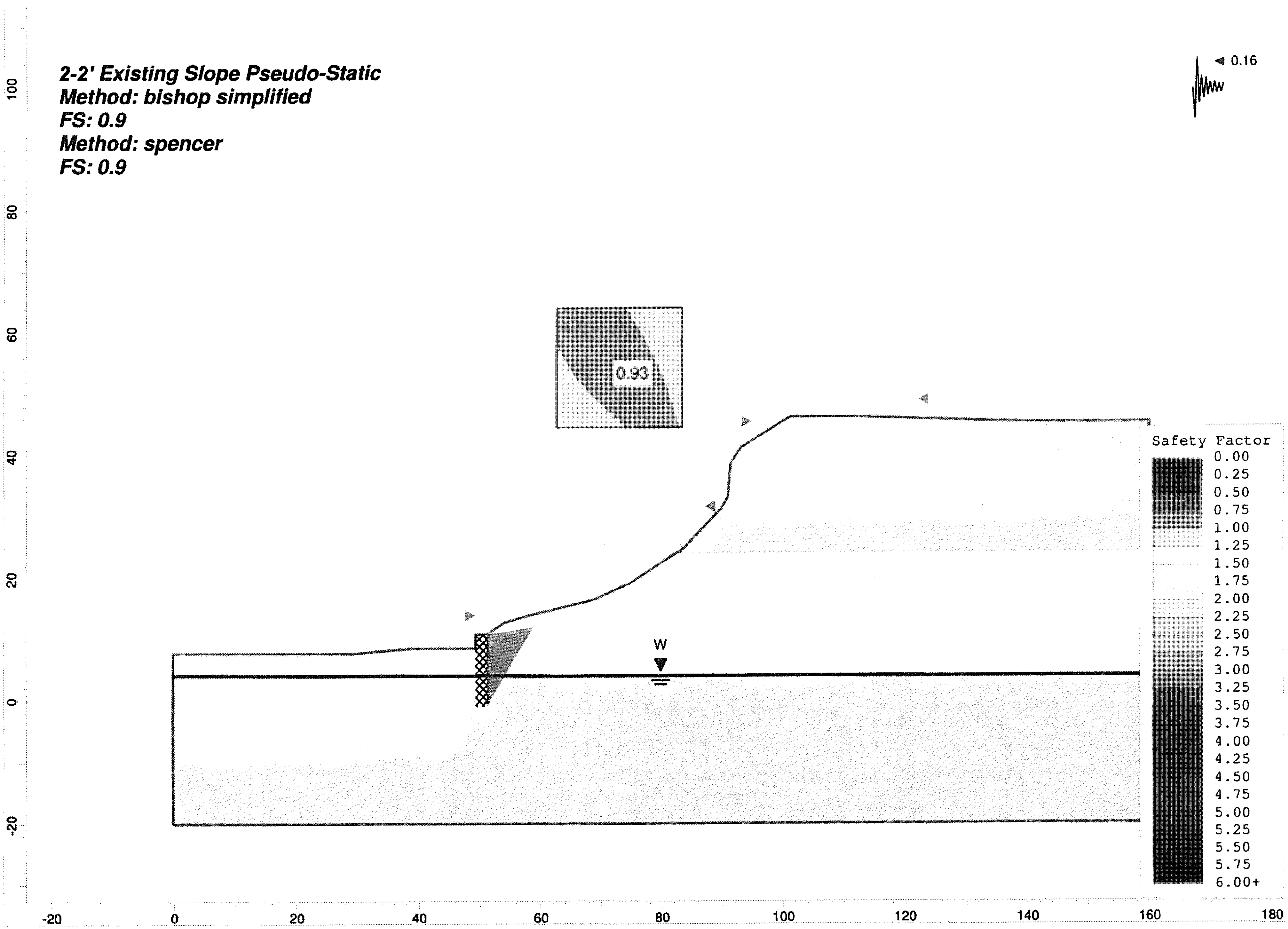


Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
1-1' Soil Nail (Static)
janbu corrected
FOS = 1.60
1.60
Soil Nails
Vertical Spacing: 5 ft
Out-of-Plane Spacing: 5 ft
Inclination: 20 degrees
length: 25 ft
Tensile Capacity: 41250 lb
Plate Capacity: 20000 lb
Colluvium
Retaining Wall
W
Beach Deposits (fill)
Beach Deposits
SM
CL
SM
ML
SM
ML
SM
SP
SP (saturated)
SM:
Cohesion: 100 psf
Friction Angle: 35 degrees
Bond Strength: 1800 lb/ft
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Bond Strength: 1350 lb/ft
Colluvium:
Cohesion: 125 psf
Friction Angle: 27 degrees
Bond Strength: 1800 lb/ft
Beach Deposits:
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill):
Cohesion: 0 psf
Friction Angle: 32 degrees
ML:
Cohesion: 200 psf
Friction Angle: 27 degrees
Bond Strength: 1350 lb/ft
SP:
Cohesion: 50 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft
SP (saturated):
Cohesion: 0 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft
120
100
80
60
40
20
0
-20
0
20
40
60
80
100
120
140
160
180

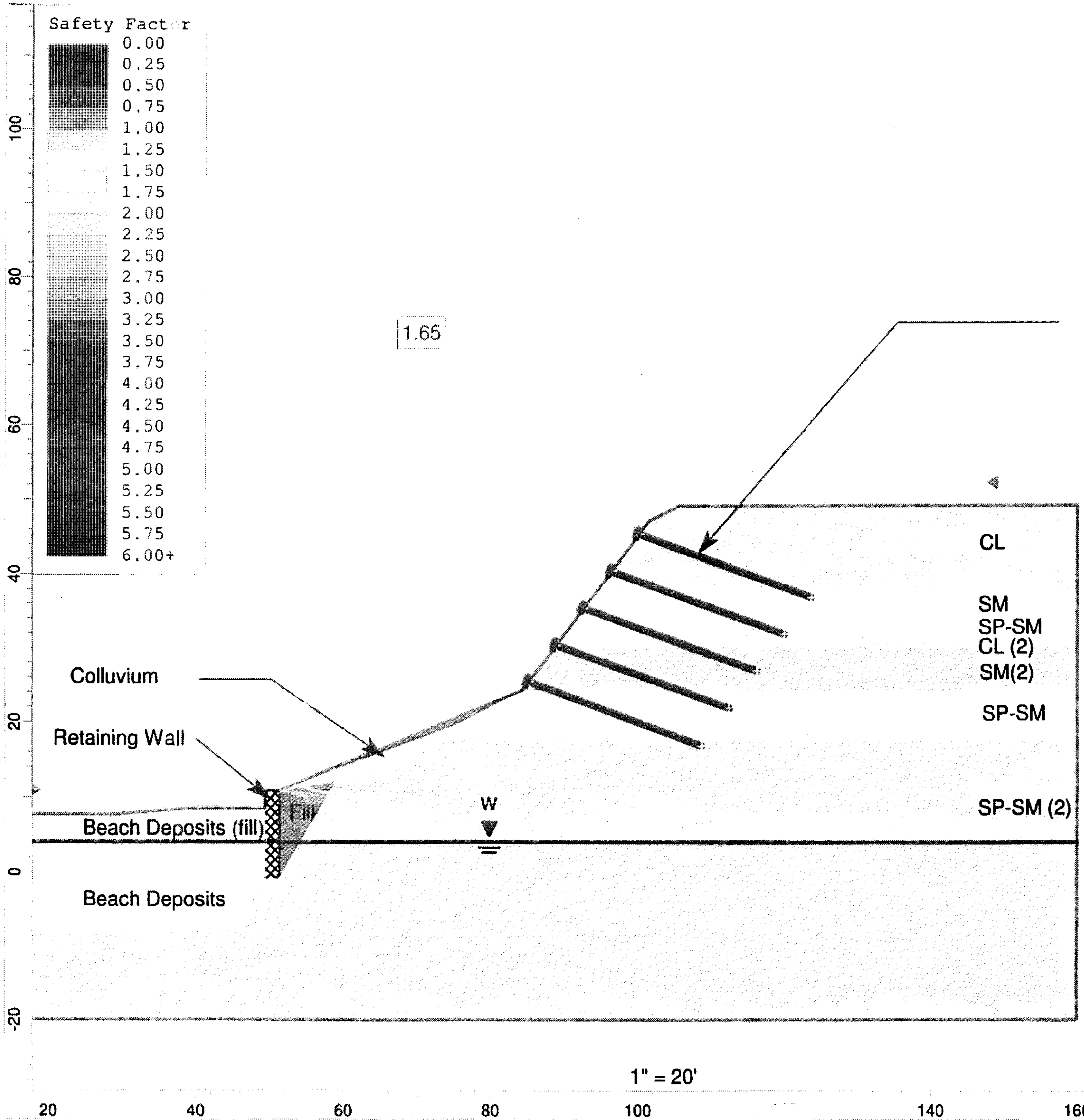

1-1' Soil Nail (Seismic)
janbu corrected
FOS = 1.17
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
1.17
0.16
Soil Nails
Vertical Spacing: 5 ft
Out-of-Plane Spacing: 5 ft
Inclination: 20 degrees
length: 25 ft
Tensile Capacity: 41250 lb
Plate Capacity: 20000 lb
Colluvium
Retaining Wall
Beach Deposits (fill)
Beach Deposits
W
SM
CL
SM
ML
SM
ML
SM
SP
SP (saturated)
SM:
Cohesion: 100 psf
Friction Angle: 35 degrees
Bond Strength: 1800 lb/ft
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Bond Strength: 1350 lb/ft
Colluvium:
Cohesion: 125 psf
Friction Angle: 27 degrees
Bond Strength: 1800 lb/ft
Beach Deposits:
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill):
Cohesion: 0 psf
Friction Angle: 32 degrees
ML:
Cohesion: 200 psf
Friction Angle: 27 degrees
Bond Strength: 1350 lb/ft
SP:
Cohesion: 50 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft
SP (saturated):
Cohesion: 0 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft
0
20
40
60
80
100
120
140
160
180
200
-20
0
20
40
60
80
100
120



2-2' Existing Slope
Method: bishop simplified
FS: 1.09
Method: spencer
FS: 1.10
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
SM:
Cohesion: 100 psf
Friction Angle: 35 degrees
Colluvium:
Cohesion: 125 psf
Friction Angle: 28 degrees
Beach Deposits
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill)
Cohesion: 0 psf
Friction Angle: 32 degrees
SP-SM
Cohesion: 0 psf
Friction Angle: 36 degrees
Fill
Cohesion: 125 psf
Friction Angle: 29 degrees
SM(2)
Cohesion: 100 psf
Friction Angle: 35 degrees
SP-SM (saturated)
Cohesion: 0 psf
Friction Angle: 36 degrees
1.10
Colluvium
Retaining Wall
Beach Deposits (fill)
Fill
Beach Deposits
W
CL
SM
SP-SM
CL
SM(2)
SM
SP-SM
SP-SM (saturated)
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
120
100
80
60
40
20
0
-20
20
40
60
80
100
120
140
160
180


2-2' Existing Slope Pseudo-Static
Method: bishop simplified
FS: 0.9
Method: spencer
FS: 0.9
0.16
0.93
W
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
-20
0
20
40
60
80
100
120
140
160
180

2-2' Soil Nail Min (Static)
janbu corrected
FOS = 1.65



Soil Nails
Vertical Spacing: 5 ft
Out-of-Plane Spacing: 5 ft
Inclination: 20 degrees
length: 25 ft
Tensile Capacity: 41250 lb
Plate Capacity: 20000 lb

CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Bond Strength: 1350 lb/ft

SM:
Cohesion: 100 psf
Friction Angle: 25 degrees
Bond Strength: 1800 lb/ft

SP-SM
Cohesion: 100 psf
Friction Angle: 35 degrees
Bond Strength: 1800 lb/ft

CL(2):
Cohesion: 50 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft

SM(2)
Cohesion: 300 psf
Friction Angle: 25 degrees
Bond Strength: 1800 lb/ft

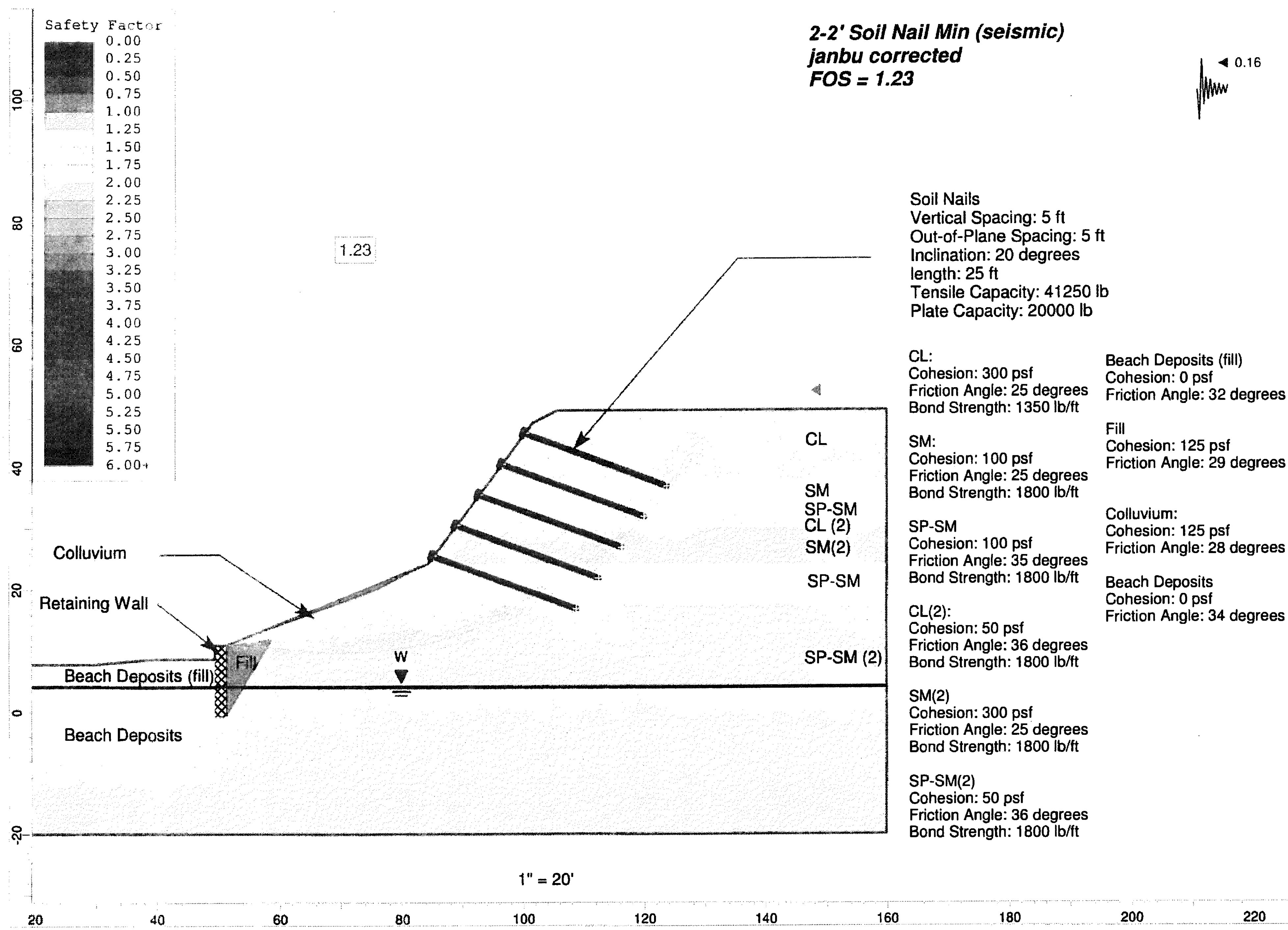
SP-SM(2)
Cohesion: 50 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft

Beach Deposits (fill)
Cohesion: 0 psf
Friction Angle: 32 degree

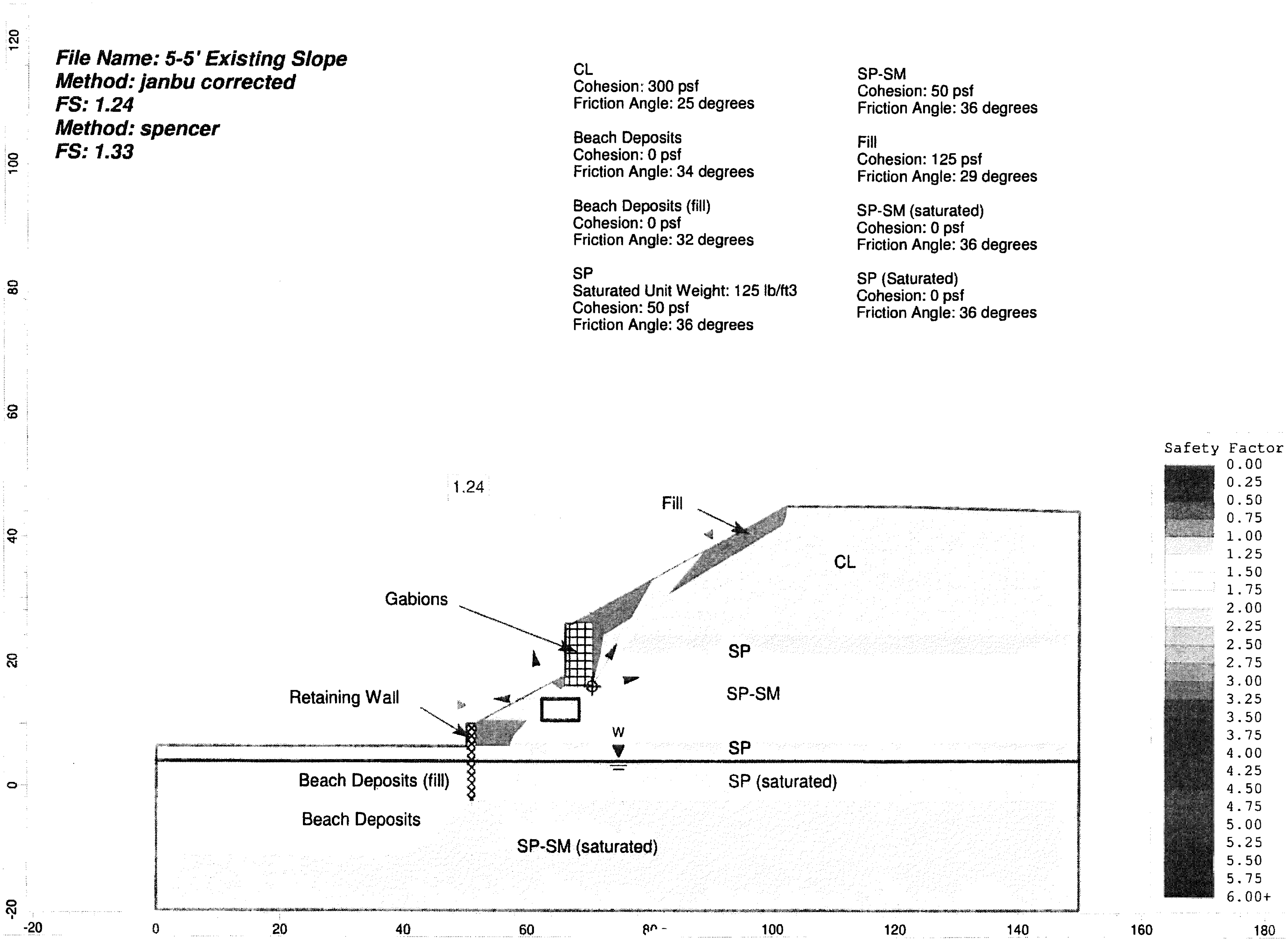
Fill
Cohesion: 125 psf
Friction Angle: 29 degree

Colluvium:
Cohesion: 125 psf
Friction Angle: 28 degree

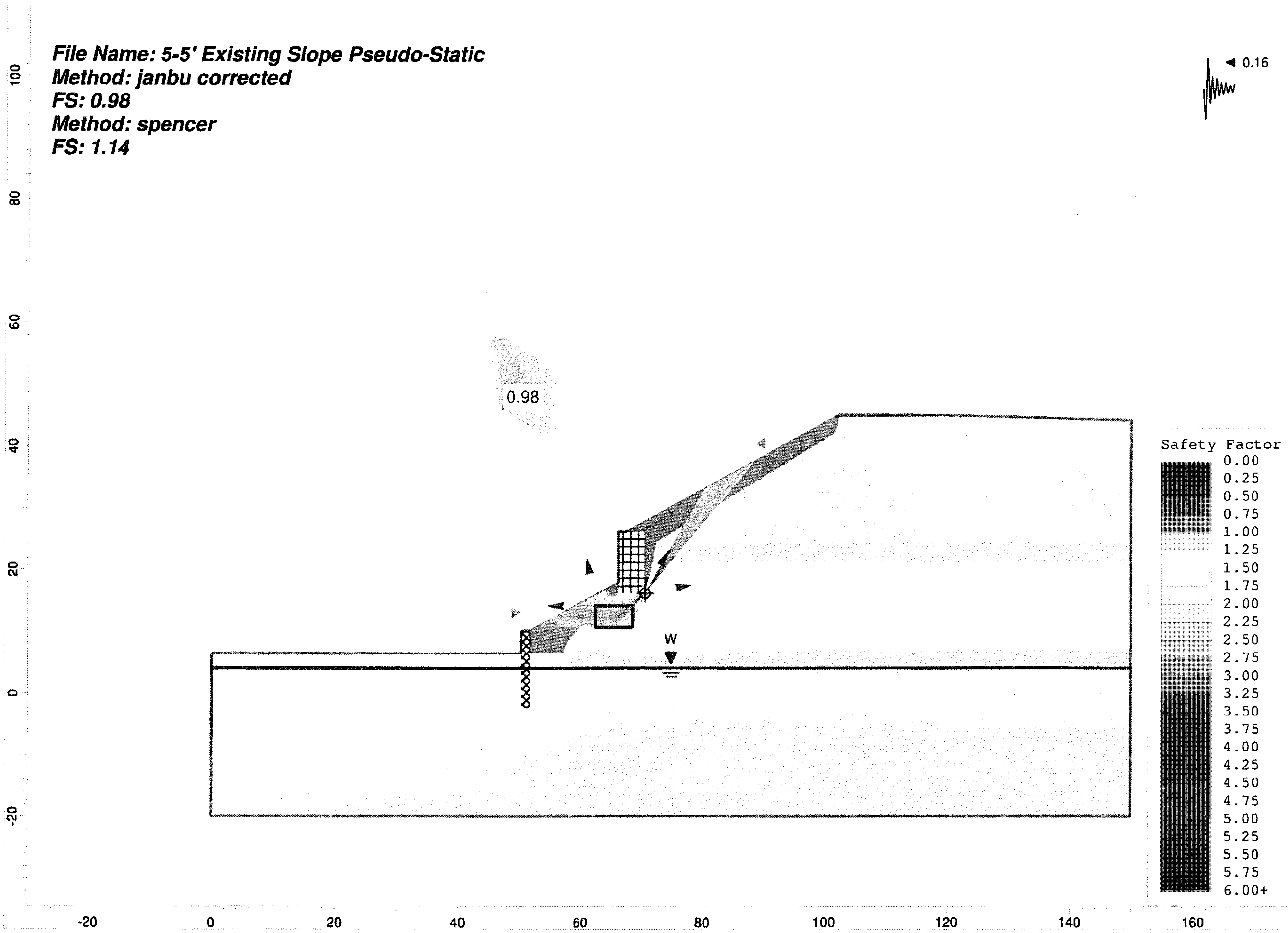
Beach Deposits
Cohesion: 0 psf
Friction Angle: 34 degree



Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
2-2' Soil Nail Min (seismic)
janbu corrected
FOS = 1.23
0.16
1.23
Soil Nails
Vertical Spacing: 5 ft
Out-of-Plane Spacing: 5 ft
Inclination: 20 degrees
length: 25 ft
Tensile Capacity: 41250 lb
Plate Capacity: 20000 lb
CL:
Cohesion: 300 psf
Friction Angle: 25 degrees
Bond Strength: 1350 lb/ft
SM:
Cohesion: 100 psf
Friction Angle: 25 degrees
Bond Strength: 1800 lb/ft
SP-SM
Cohesion: 100 psf
Friction Angle: 35 degrees
Bond Strength: 1800 lb/ft
CL(2):
Cohesion: 50 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft
SM(2)
Cohesion: 300 psf
Friction Angle: 25 degrees
Bond Strength: 1800 lb/ft
SP-SM(2)
Cohesion: 50 psf
Friction Angle: 36 degrees
Bond Strength: 1800 lb/ft
Beach Deposits (fill)
Cohesion: 0 psf
Friction Angle: 32 degrees
Fill
Cohesion: 125 psf
Friction Angle: 29 degrees
Colluvium:
Cohesion: 125 psf
Friction Angle: 28 degrees
Beach Deposits
Cohesion: 0 psf
Friction Angle: 34 degrees
CL
SM
SP-SM
CL (2)
SM(2)
SP-SM
SP-SM (2)
Colluvium
Retaining Wall
Fill
Beach Deposits (fill)
Beach Deposits
W
1" = 20'
100
80
60
40
20
0
-20
20
40
60
80
100
120
140
160
180
200
220


File Name: 5-5' Existing Slope
Method: janbu corrected
FS: 1.24
Method: spencer
FS: 1.33
CL
Cohesion: 300 psf
Friction Angle: 25 degrees
Beach Deposits
Cohesion: 0 psf
Friction Angle: 34 degrees
Beach Deposits (fill)
Cohesion: 0 psf
Friction Angle: 32 degrees
SP
Saturated Unit Weight: 125 lb/ft3
Cohesion: 50 psf
Friction Angle: 36 degrees
SP-SM
Cohesion: 50 psf
Friction Angle: 36 degrees
Fill
Cohesion: 125 psf
Friction Angle: 29 degrees
SP-SM (saturated)
Cohesion: 0 psf
Friction Angle: 36 degrees
SP (Saturated)
Cohesion: 0 psf
Friction Angle: 36 degrees
1.24
Fill
CL
Gabions
SP
SP-SM
Retaining Wall
W
SP
Beach Deposits (fill)
SP (saturated)
Beach Deposits
SP-SM (saturated)
Safety Factor
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00
4.25
4.50
4.75
5.00
5.25
5.50
5.75
6.00+
-20
0
20
40
60
100
120
140
160
180
80
60
40
20
0
-20

File Name: 5-5' Existing Slope Pseudo-Static
Method: janbu corrected
FS: 0.98
Method: spencer
FS: 1.14



File Name: 5-5' Existing Slope (Loss in Cementation)
Method: janbu corrected
FS: 1.01
Method: spencer
FS: 1.15

CL
Cohesion: 300 psf
Friction Angle: 25 degrees

Beach Deposits
Cohesion: 0 psf
Friction Angle: 34 degrees

Beach Deposits (fill)
Cohesion: 0 psf
Friction Angle: 32 degrees

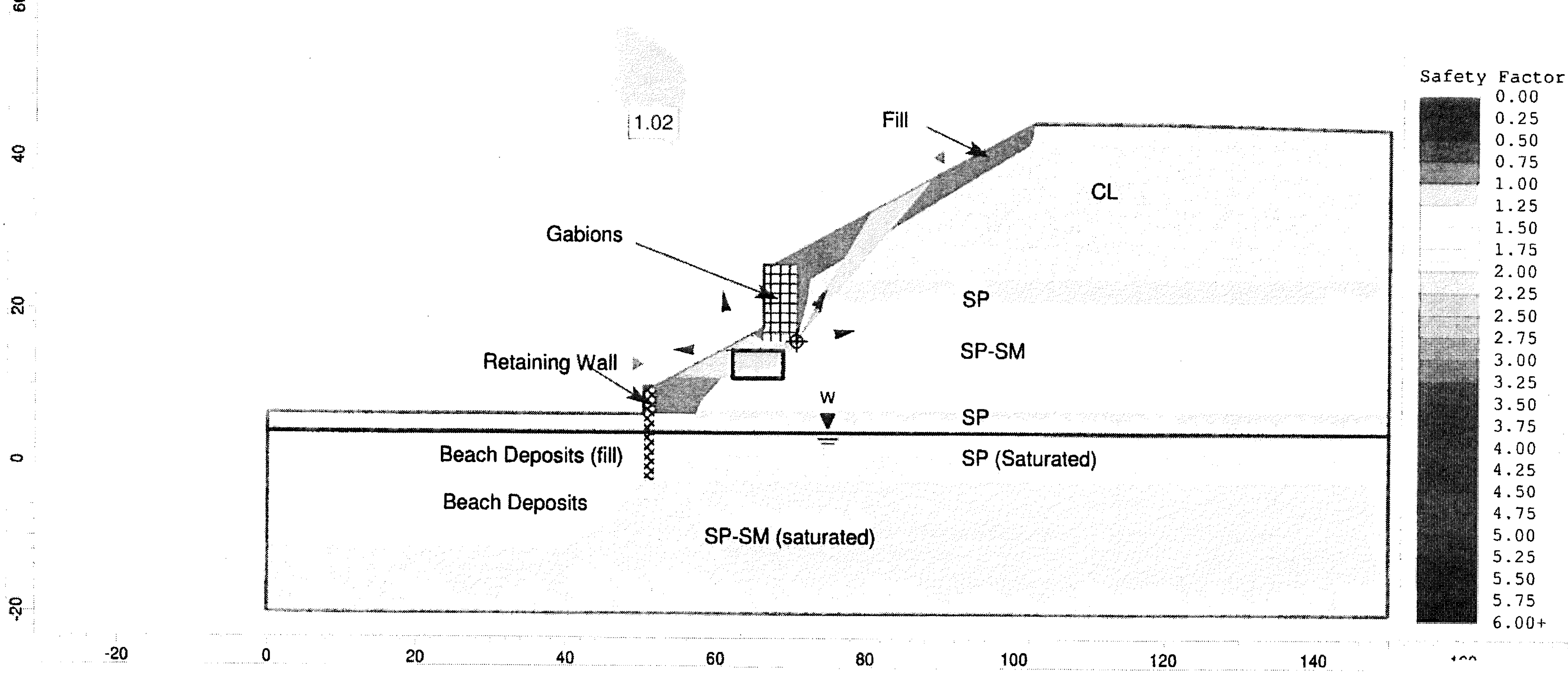
SP
Saturated Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 36 degrees

SP-SM
Cohesion: 0 psf
Friction Angle: 36 degrees

Fill
Cohesion: 0 psf
Friction Angle: 29 degrees

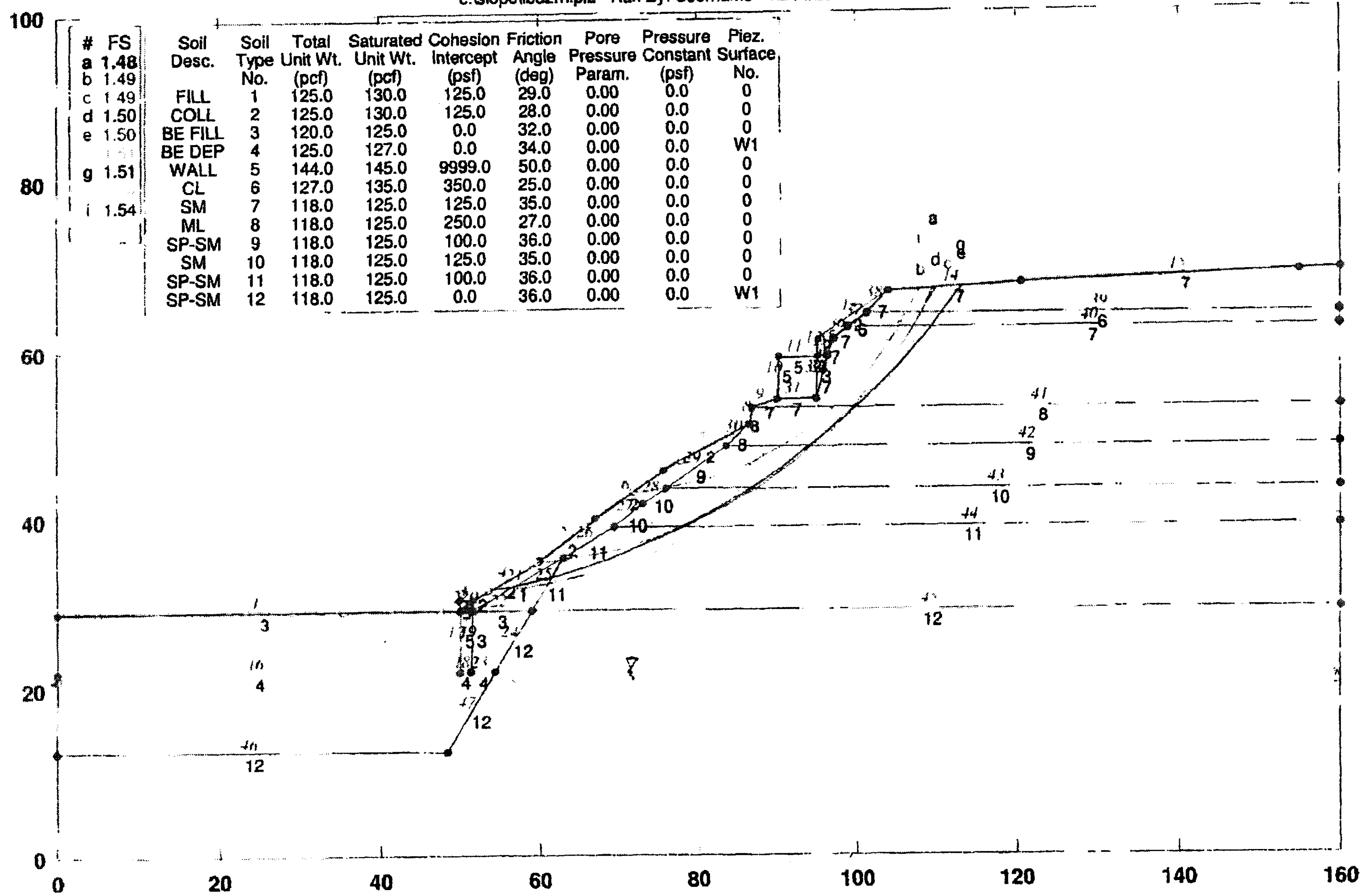
SP-SM (saturated)
Cohesion: 0 psf
Friction Angle: 36 degrees

SP (Saturated)
Cohesion: 0 psf
Friction Angle: 36 degrees



Bluff Section 16-16'

c:\slope\lbd2rh.pl2 Run By: Username 12/14/2009 07:51AM



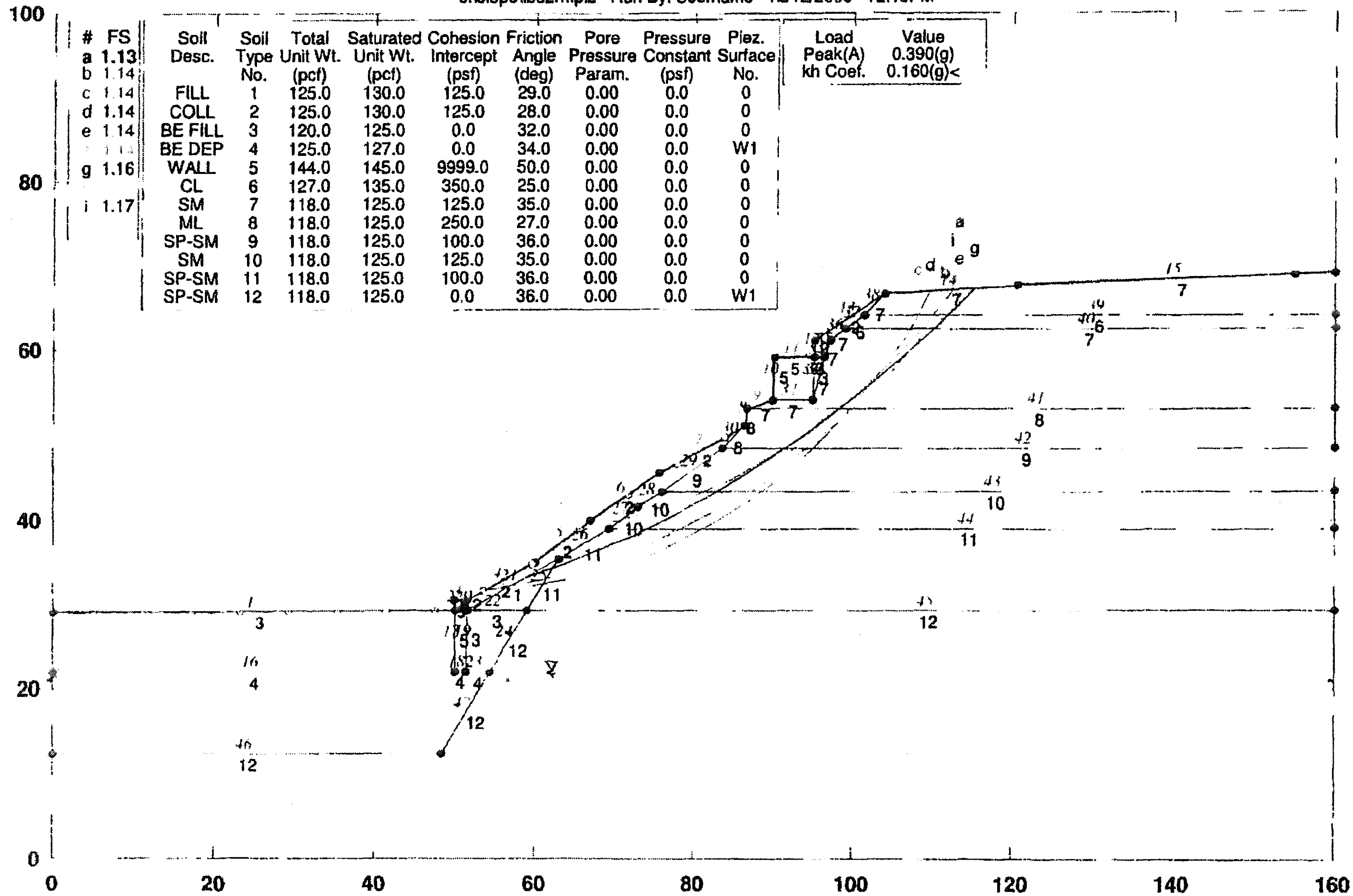
GSTABL7 v.2 FSmin=1.48

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

GSTABL7

Bluff Section 16-16'

c:\slope\lbd2rh.pl2 Run By: Username 12/12/2009 12:18PM



#	FS
a	1.13
b	1.14
c	1.14
d	1.14
e	1.14
[illegible]	[illegible]
g	1.16
i	1.17

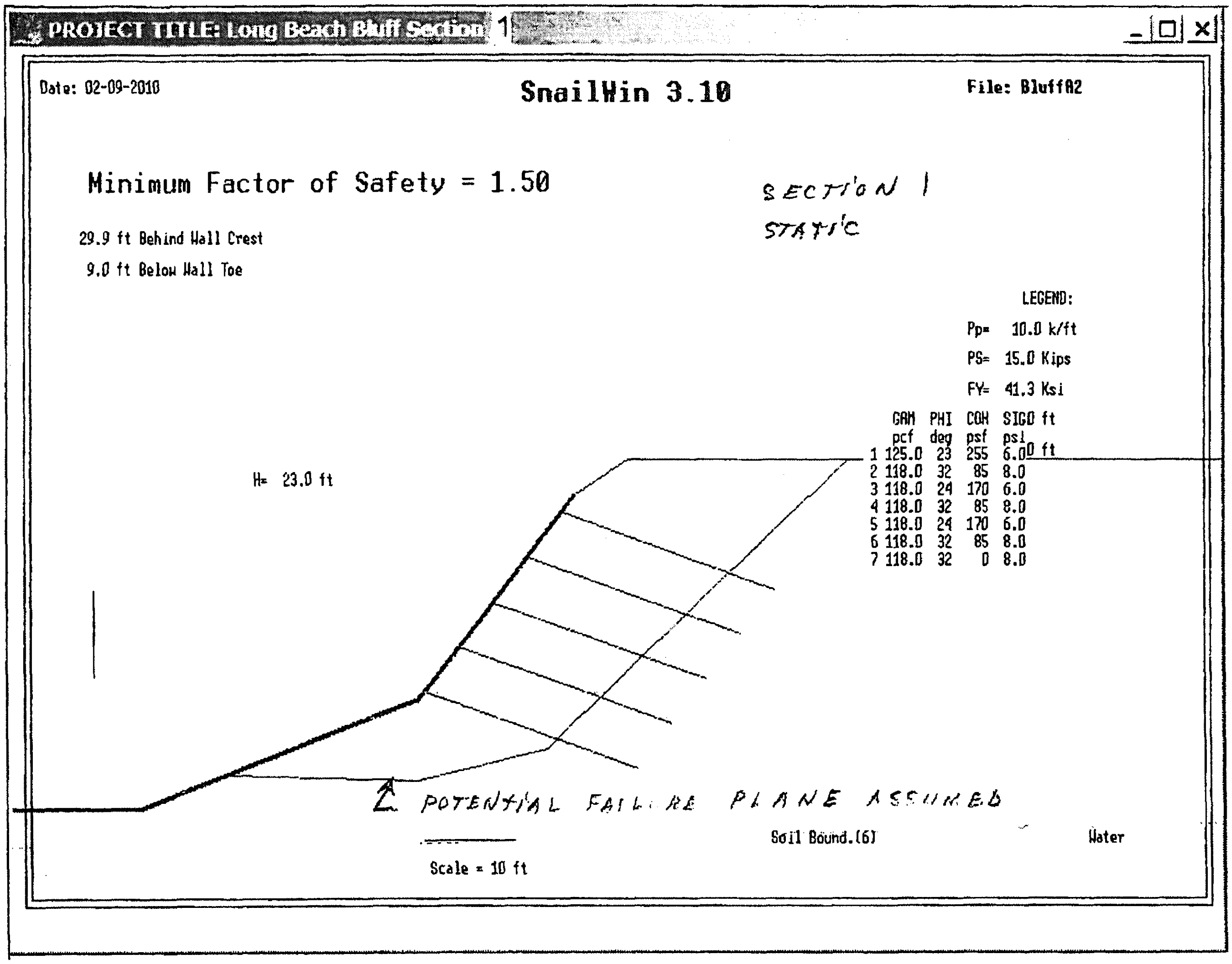
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
FILL	1	125.0	130.0	125.0	29.0	0.00	0.0	0
COLL	2	125.0	130.0	125.0	28.0	0.00	0.0	0
BE FILL	3	120.0	125.0	0.0	32.0	0.00	0.0	0
BE DEP	4	125.0	127.0	0.0	34.0	0.00	0.0	W1
WALL	5	144.0	145.0	9999.0	50.0	0.00	0.0	0
CL	6	127.0	135.0	350.0	25.0	0.00	0.0	0
SM	7	118.0	125.0	125.0	35.0	0.00	0.0	0
ML	8	118.0	125.0	250.0	27.0	0.00	0.0	0
SP-SM	9	118.0	125.0	100.0	36.0	0.00	0.0	0
SM	10	118.0	125.0	125.0	35.0	0.00	0.0	0
SP-SM	11	118.0	125.0	100.0	36.0	0.00	0.0	0
SP-SM	12	118.0	125.0	0.0	36.0	0.00	0.0	W1

Load	Value
Peak(A)	0.390(g)
kh Coef.	0.160(g)<

GSTABL7 v.2 FSmin=1.13

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7


PROJECT TITLE: Long Beach Bluff Section 1
Date: 02-09-2010
SnailWin 3.10
File: BluffA2
Minimum Factor of Safety = 1.50
29.9 ft Behind Wall Crest
9.0 ft Below Wall Toe
SECTION 1
STATIC
LEGEND:
Pp= 10.0 k/ft
PS= 15.0 Kips
FY= 41.3 Ksi
GAM PHI COH SIGO ft
pcf deg psf psi
1 125.0 23 255 6.0 0 ft
2 118.0 32 85 8.0
3 118.0 24 170 6.0
4 118.0 32 85 8.0
5 118.0 24 170 6.0
6 118.0 32 85 8.0
7 118.0 32 0 8.0
H= 23.0 ft
POTENTIAL FAILURE PLANE ASSUMED
Soil Bound.[6]
Water
Scale = 10 ft

PROJECT TITLE: Long Beach Bluff Section 1

Date: 02-09-2010

SnailWin 3.10

File: BluffA2

Minimum Factor of Safety = 1.10

SECTION 1
SEISMIC

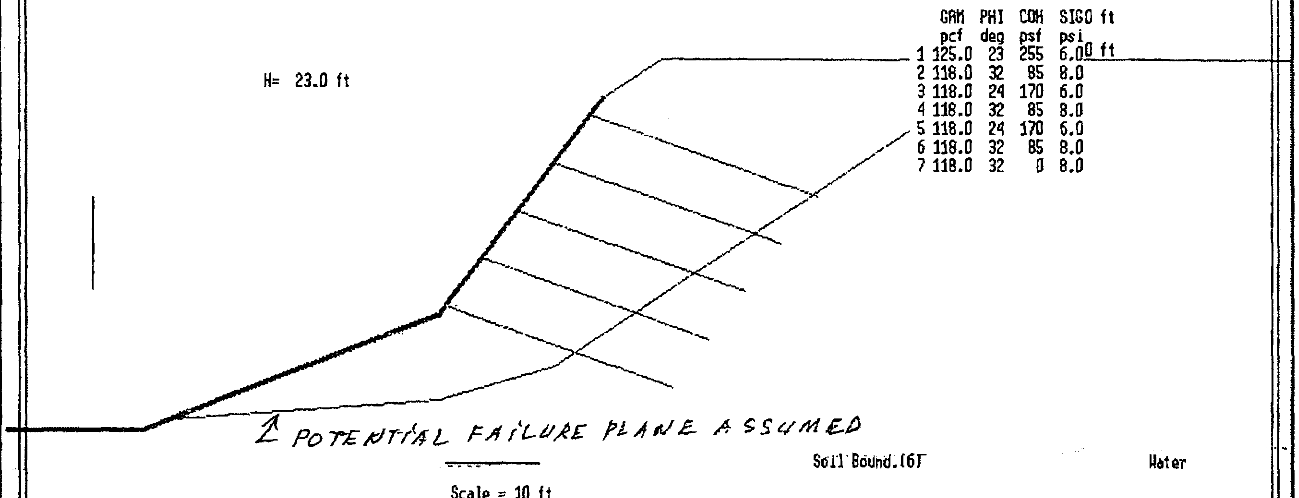
42.7 ft Behind Wall Crest
9.0 ft Below Wall Toe

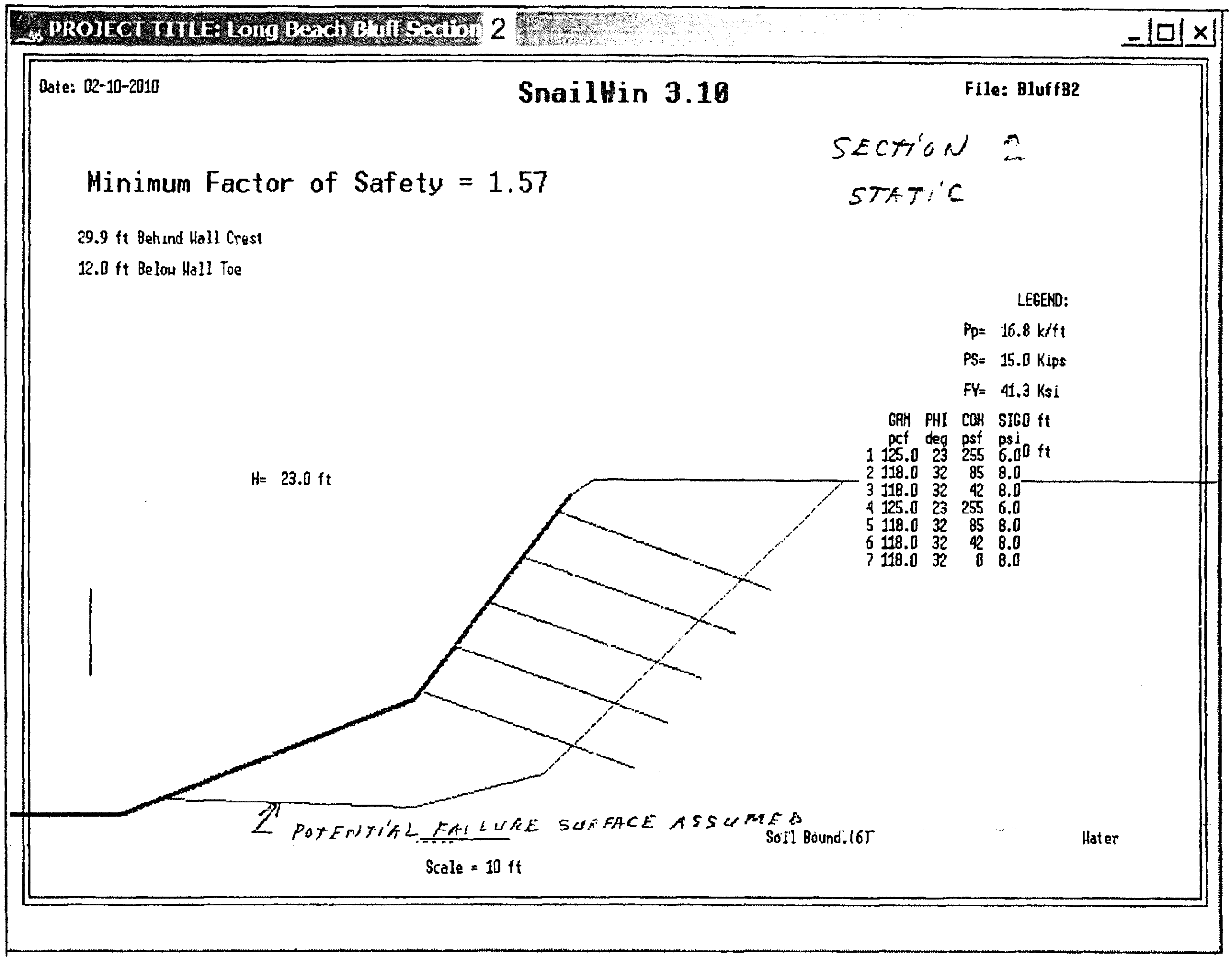
Hoz. KH= 0.16g
Vrt.PKH= 0.00g

LEGEND:

Pp= 7.9 k/ft
PS= 15.0 Kips
FY= 41.3 Ksi

	GAM pcf	PHI deg	COH psf	SIGO psi
1	125.0	23	255	6.0
2	118.0	32	85	8.0
3	118.0	24	170	6.0
4	118.0	32	85	8.0
5	118.0	24	170	6.0
6	118.0	32	85	8.0
7	118.0	32	0	8.0




PROJECT TITLE: Long Beach Bluff Section 2
Date: 02-10-2010
SnailWin 3.10
File: BluffB2
SECTION 2
STATIC
Minimum Factor of Safety = 1.57
29.9 ft Behind Wall Crest
12.0 ft Below Wall Toe
LEGEND:
Pp= 16.8 k/ft
PS= 15.0 Kips
FY= 41.3 Ksi
GRM PHI COH SIGO ft
pcf deg psf psi
0 ft
1 125.0 23 255 6.0
2 118.0 32 85 8.0
3 118.0 32 42 8.0
4 125.0 23 255 6.0
5 118.0 32 85 8.0
6 118.0 32 42 8.0
7 118.0 32 0 8.0
H= 23.0 ft
POTENTIAL FAILURE SURFACE ASSUMED
Soil Bound.(6)
Water
Scale = 10 ft

PROJECT TITLE: Long Beach Bluff Section 2

Date: 02-10-2010

SnailWin 3.10

File: BluffB2

SECTION 2
SEISMIC

Minimum Factor of Safety = 1.12

38.4 ft Behind Wall Crest

12.0 ft Below Wall Toe

Hoz. KH= 0.16g

Vrt.PKH= 0.00g

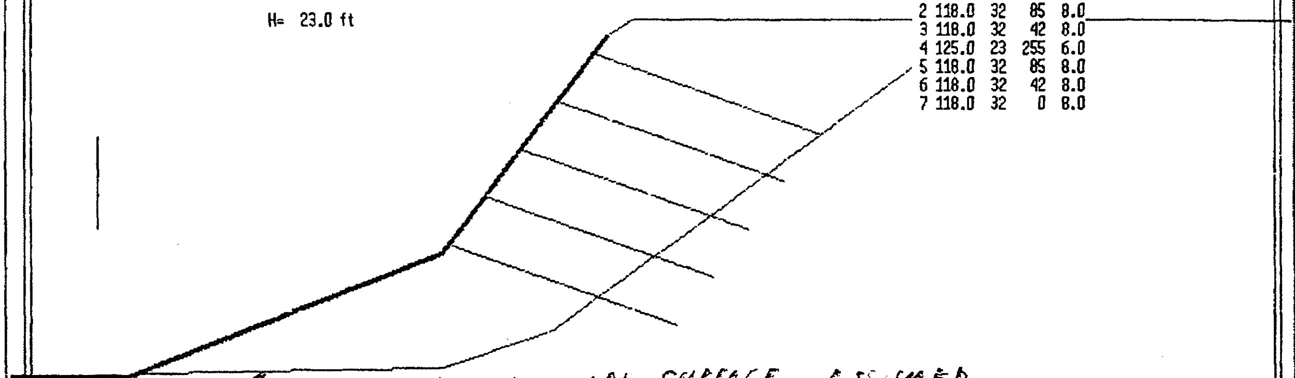
LEGEND:

Pp= 13.1 k/ft

PS= 15.0 Kips

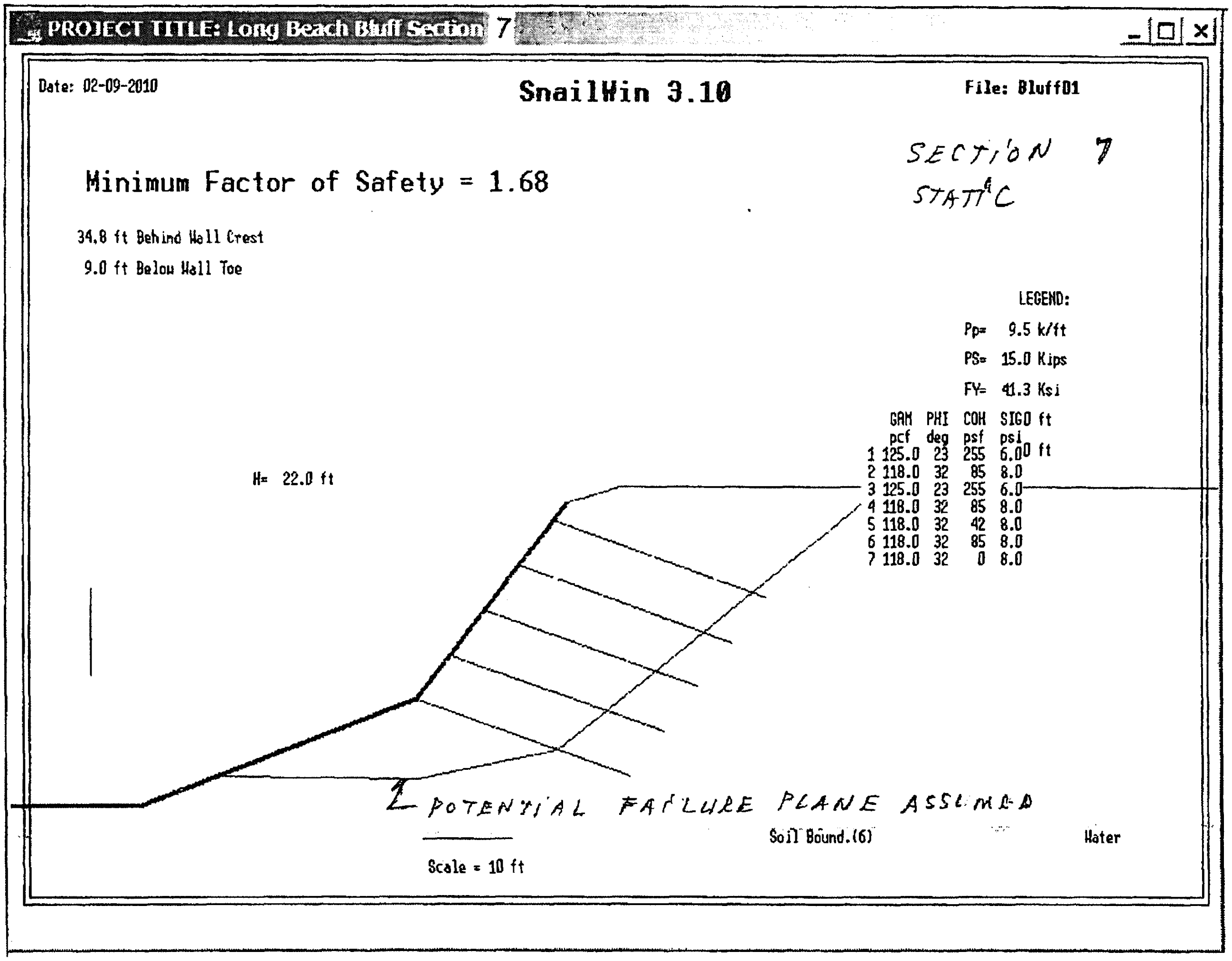
FY= 41.3 Ksi

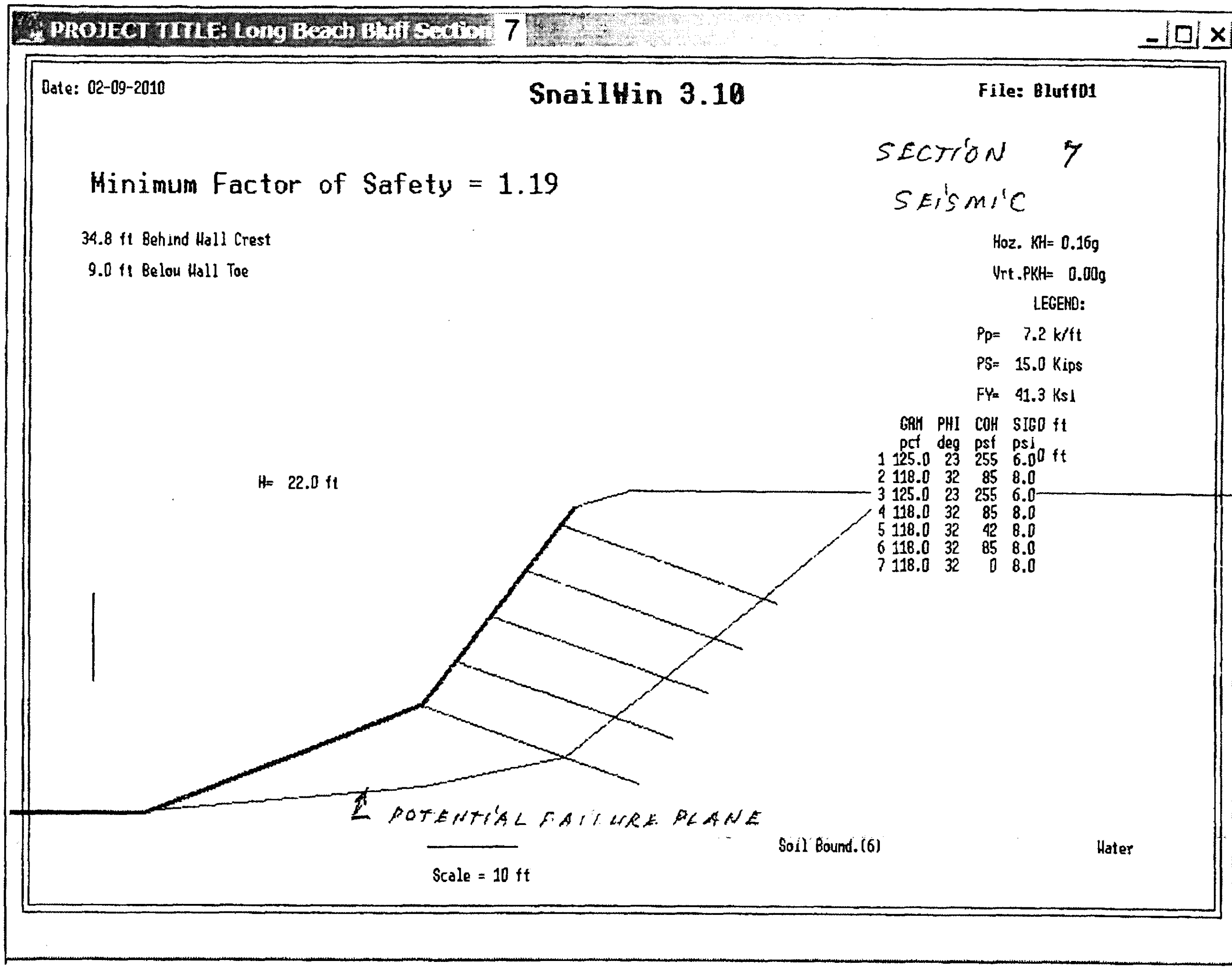
	GAM pcf	PHI deg	COH psf	SIGO psi
1	125.0	23	255	6.0
2	118.0	32	85	8.0
3	118.0	32	42	8.0
4	125.0	23	255	6.0
5	118.0	32	85	8.0
6	118.0	32	42	8.0
7	118.0	32	0	8.0



2 POTENTIAL FAILURE SURFACE, ASSUMED

Scale = 10 ft


PROJECT TITLE: Long Beach Bluff Section 7
Date: 02-09-2010
SnailWin 3.10
File: Bluff01
SECTION 7
STATIC
Minimum Factor of Safety = 1.68
34.8 ft Behind Wall Crest
9.0 ft Below Wall Toe
LEGEND:
Pp= 9.5 k/ft
PS= 15.0 Kips
FY= 41.3 Ksi
GAM PHI COH SIGO ft
pcf deg psf psi
0 ft
1 125.0 23 255 6.0
2 118.0 32 85 8.0
3 125.0 23 255 6.0
4 118.0 32 85 8.0
5 118.0 32 42 8.0
6 118.0 32 85 8.0
7 118.0 32 0 8.0
H= 22.0 ft
POTENTIAL FAILURE PLANE ASSUMED
Soil Bound.(6)
Water
Scale = 10 ft


PROJECT TITLE: Long Beach Bluff Section 7
Date: 02-09-2010
SnailWin 3.10
File: Bluff01
SECTION 7
SEISMIC
Minimum Factor of Safety = 1.19
34.8 ft Behind Wall Crest
9.0 ft Below Wall Toe
Hoz. KH= 0.16g
Vrt.PKH= 0.00g
LEGEND:
Pp= 7.2 k/ft
PS= 15.0 Kips
FY= 41.3 Ksi
GAM PHI COH SIGO ft
pcf deg psf psi
1 125.0 23 255 6.0 0 ft
2 118.0 32 85 8.0
3 125.0 23 255 6.0
4 118.0 32 85 8.0
5 118.0 32 42 8.0
6 118.0 32 85 8.0
7 118.0 32 0 8.0
H= 22.0 ft
POTENTIAL FAILURE PLANE
Soil Bound.(6)
Water
Scale = 10 ft

PROJECT TITLE: Long Beach Bluff Section 7

Date: 02-09-2010

SnailWin 3.10

File: Bluff01

SECTION 7
STATIC

Minimum Factor of Safety = 2.55

34.8 ft Behind Wall Crest

0.0 ft Below Wall Toe

LEGEND:

PS= 15.0 Kips

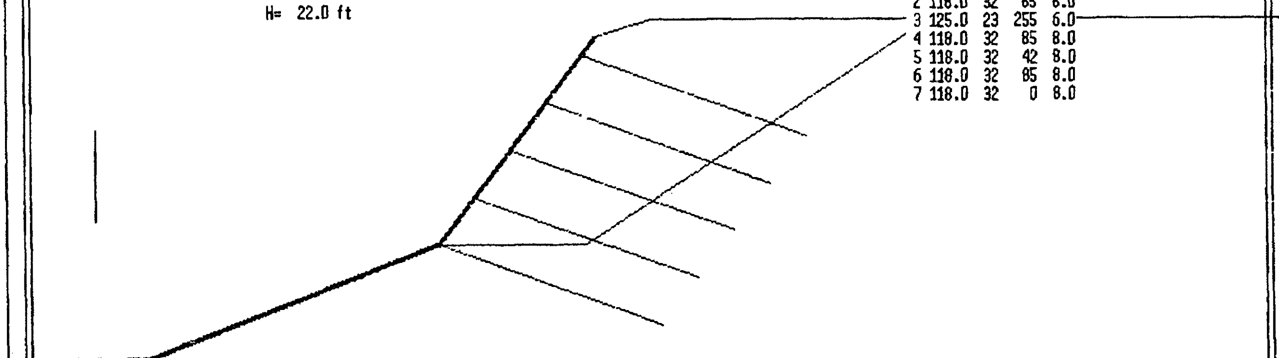
FV= 41.3 Ksi

	GAM pcf	PHI deg	COH psf	SIGO psi
1	125.0	23	255	6.0
2	118.0	32	85	8.0
3	125.0	23	255	6.0
4	118.0	32	85	8.0
5	118.0	32	42	8.0
6	118.0	32	85	8.0
7	118.0	32	0	8.0

ft

0 ft

H= 22.0 ft



Soil Bound.(6)

Water

Scale = 10 ft

PROJECT TITLE: Long Beach Bluff Section 7

Date: 02-09-2010

SnailWin 3.10

File: Bluff01

SECTION 7
SEISMIC

Minimum Factor of Safety = 1.70

34.8 ft Behind Wall Crest

0.0 ft Below Wall Toe

LEGEND:

Crit.Ac= 0.36g

Hoz. KH= 0.16g

Vrt.PKH= 0.00g

PS= 15.0 Kips

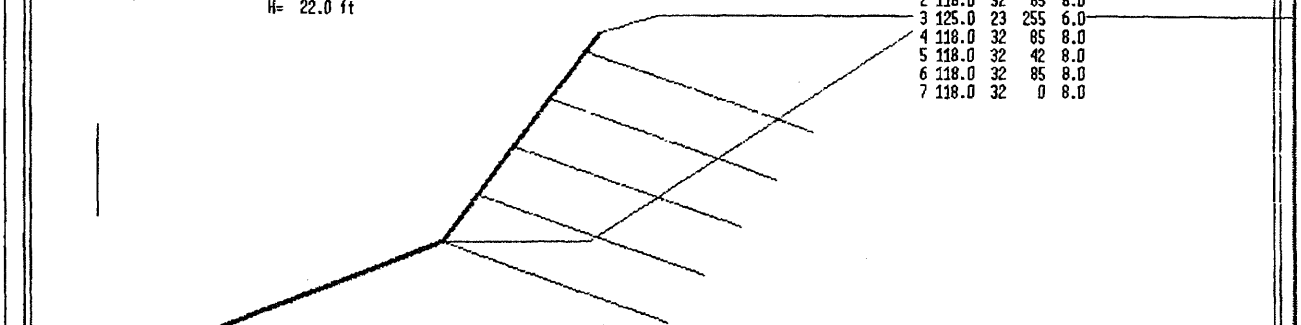
FY= 41.3 Ksi

	GAM pcf	PHI deg	COH psf	SIGO psi
1	125.0	23	255	6.0
2	118.0	32	85	8.0
3	125.0	23	255	6.0
4	118.0	32	85	8.0
5	118.0	32	42	8.0
6	118.0	32	85	8.0
7	118.0	32	0	8.0

ft

0 ft

H= 22.0 ft



Scale = 10 ft

Soil Bound.(6)

Water

APPENDIX D

SITE PHOTOGRAPHS

ATTACHED IMAGES: Images: DSC_0017.JPG Images: DSC_0053.JPG Images: DSC_0058.JPG Images: DSC_0084.JPG Images: DSC_0097.JPG
ATTACHED XREFS:
DIAMOND BAR, CA CAD FILE: L:\2009\CADD\107104\LongBeachBluffs\ LAYOUT: D1 PLOTTED: 09 Dec 2009, 4:16pm, MGriffin

OVERVIEW IN MID PORTION OF THE PARK LOOKING EAST.

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO.	107104
DRAWN:	12/2009
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	
107	1 to D5.dwg

SITE PHOTOGRAPH

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
D1

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ILLUSTRATE TOE WALL, GABION WALL PARTIALLY COVERED WITH SHRUB AND STORM DRAIN OUTLET.

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PROJECT NO.	107104
DRAWN:	12/2009
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107104pD1 to D5.dwg

SITE PHOTOGRAPH

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
D2

ATTACHED IMAGES: Images: DSC_0017.JPG Images: DSC_0053.JPG Images: DSC_0058.JPG Images: DSC_0084.JPG Images: DSC_0097.JPG
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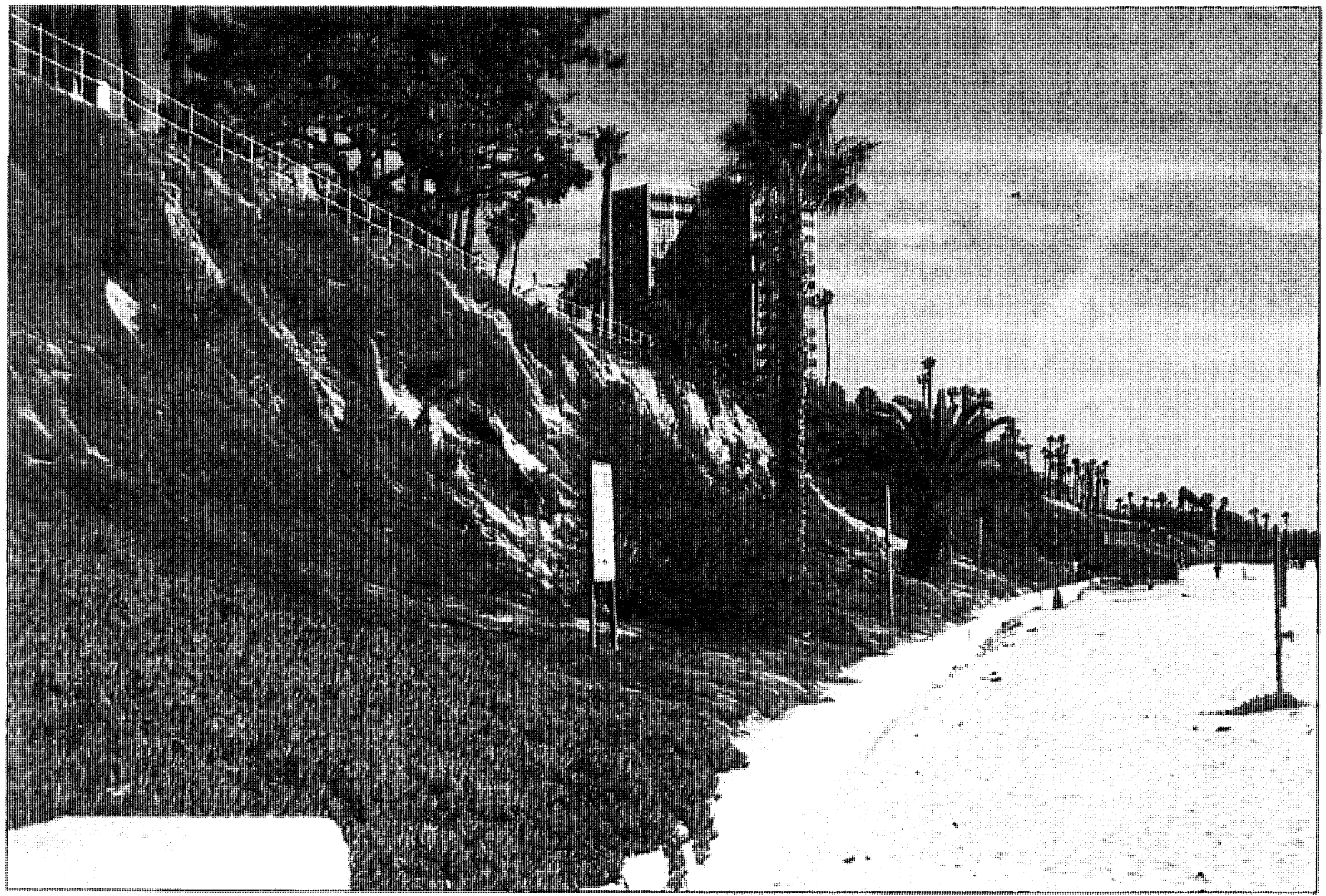

UNEVEN SLOPE CONDITION, NEAR VERTICAL SEGMENT IN UPPER PORTION AND ABOUT 4.5:1 IN LOWER PORTION.

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PROJECT NO.	107104
DRAWN:	12/2009
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107 1 to D5.dwg

SITE PHOTOGRAPH

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
D3

ATTACHED IMAGES: Images: DSC_0017.JPG Images: DSC_0053.JPG Images: DSC_0058.JPG Images: DSC_0084.JPG Images: DSC_0097.JPG
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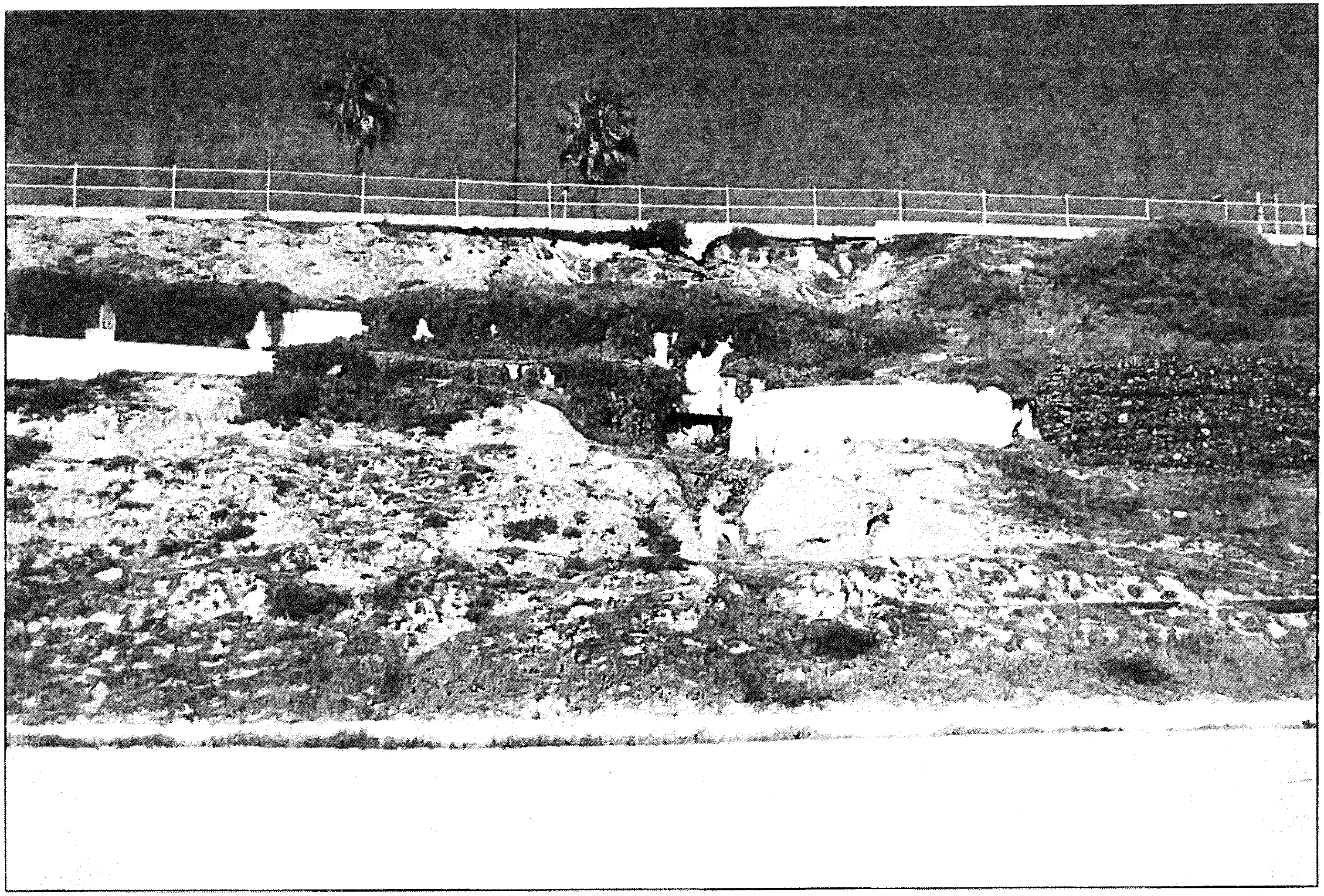

SURFACE SLOPE EROSION AND SIDEWALK UNDERMINING.

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PROJECT NO.	107104
DRAWN:	12/2009
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME: 107104pD1 to D5.dwg	

SITE PHOTOGRAPH

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
D4

ATTACHED IMAGES: Images: DSC_0017.JPG Images: DSC_0053.JPG Images: DSC_0058.JPG Images: DSC_0084.JPG Images: DSC_0097.JPG
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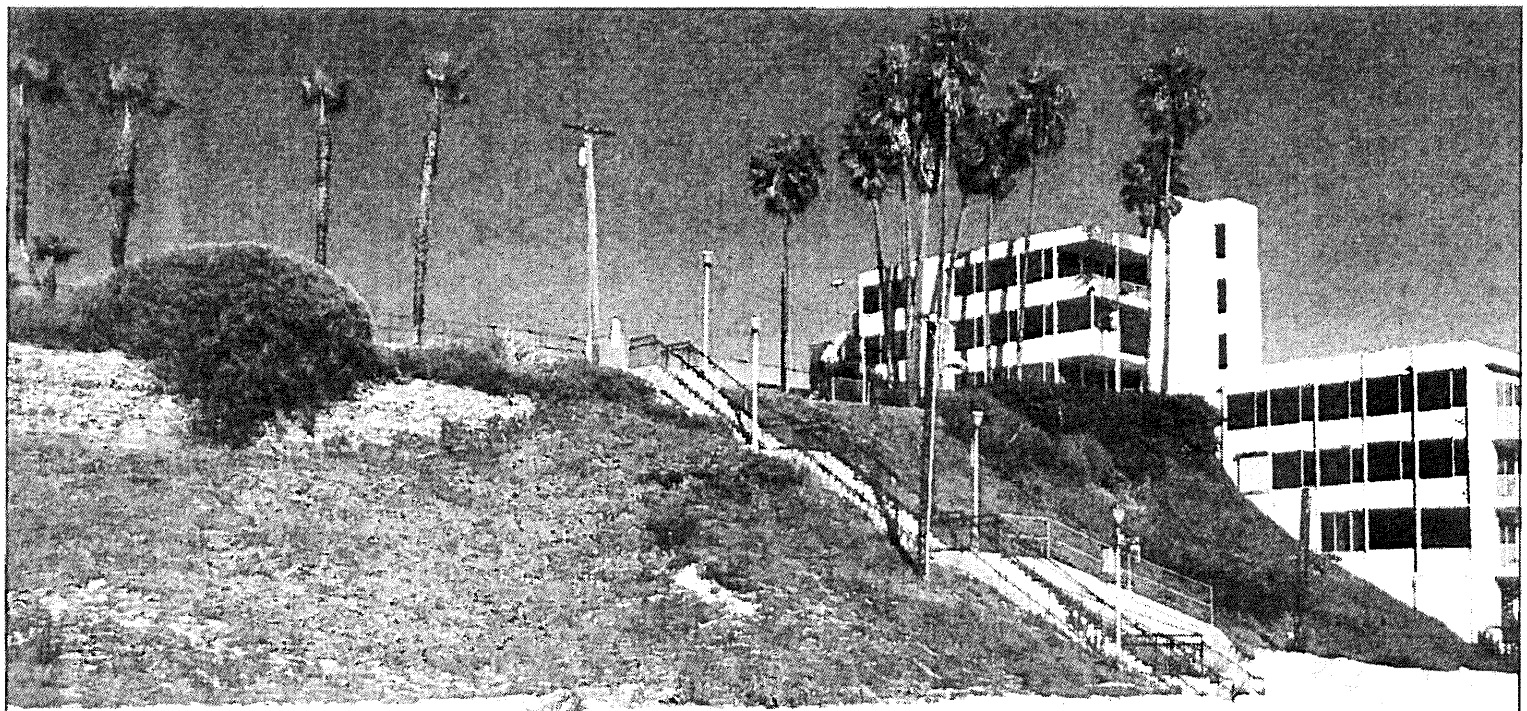

SLOPE EXTENDED BEYOND THE TOE WALL TO CONSTRUCT THE STAIRWAY.

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PROJECT NO.	107104
DRAWN:	12/2009
DRAWN BY:	MRG
CHECKED BY:	JR
FILE NAME:	107 1 to D5.dwg

SITE PHOTOGRAPH

PROPOSED SLOPE IMPROVEMENTS
BLUFF PARK
LONG BEACH, CALIFORNIA

PLATE
D5

PRELIMINARY GEOTECHNICAL INVESTIGATION
Revised September 3, 2003

PRELIMINARY
GEOTECHNICAL INVESTIGATION
PROPOSED BELMONT SHORE
BLUFF RESTORATION
LONG BEACH, CALIFORNIA

Prepared for:
Tetra Tech, Inc.
401 E. Ocean Boulevard, Suite 810
Long Beach, California 90802

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

Project No. 1652.I

Initially Issued May 20, 2000
Revised September 3, 2003

PRELIMINARY GEOTECHNICAL INVESTIGATION PROPOSED BELMONT SHORE BLUFF RESTORATION LONG BEACH, CALIFORNIA

Prepared for:
Tetra Tech, Inc.
401 E. Ocean Boulevard, Suite 810
Long Beach, California 90802

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Initially Issued May 20, 2000
Revised September 3, 2003

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1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for restoration of public property bluffs between Downtown Long Beach and Belmont Shore. This project covers only bluffs in the public domain within approximately a two-mile stretch from First Place on the west to Loma Avenue on the east. Within the western half of the project site, public property bluffs are typically limited to the southern edges of public streets. Within the eastern half of the project site (east of Cherry Avenue), the bluffs covered by this report, encompass the southern edge of two public parks flanking the Long Beach Museum of Art (Bixby Park to the west and Bluff Park to the east). The project site location is presented in Figure 1. The topographic conditions along the bluff are presented in strip maps (Figures 2.1 through 2.6).

The primary objective of the overall project is to restore the stability of the bluff areas that exhibit excessive erosion and slumping failures. Secondary objectives include improving the appearance of function of the bluff areas. The design process will involve public input and will evolve over a period of several months.

Consistent with the objectives of the overall bluff restoration program, the main objectives of this geotechnical investigation were to assess the geotechnical stability of the existing slope conditions and to evaluate the feasibility of a range of conceptual designs aimed at improving the stability to acceptable levels. The conceptual designs will be finalized based on aesthetic and functional considerations. Therefore, the recommendations presented in this report will need to be reviewed and possibly adjusted once the design concepts are finalized.

2.0 SCOPE OF WORK

The scope of our geotechnical investigation was as outlined in our proposal, dated February 16, 2000 and included field investigation, laboratory testing, geotechnical analysis, and preparation of this report.

Our field investigation consisted of a geotechnical site reconnaissance, five cone penetration tests (CPT's), five hollow-stem auger borings and one hand-auger boring. The exploration locations are shown on the Site Plans (Figure 2.1 through 2.6). The CPT soundings extended to depths ranging from 40 to 62 feet and were used primarily to evaluate the variation of sol layering and strength across the site. CPT field procedures and logs are presented in Appendix A. The hollow-stem auger borings extended to depths ranging between 41 and 55 feet and were used to obtain soil samples for laboratory testing and to check groundwater levels. The hand auger boring was used to investigation fill soil conditions at the eroded slope surface of the Junipero Avenue ramp. Drilling procedures and logs of borings are presented in Appendix B.

The laboratory testing program focused primarily on moisture-density and shear strength, but also included grain size analyses. Laboratory test procedures and results are presented in Appendix C.

Our geotechnical analyses focused primarily on slope stability. A summary and computer output for our stability analyses are presented in Appendix D.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site encompasses approximately two miles of bluffs within the public right-of-way, between First Place on the west end and south of Loma Avenue at the east end. West of Cherry Avenue, the public portion of the bluffs is typically limited to the south ends of streets, separated by private property. East of Cherry Avenue, the entire bluff area is within the public right-of-way. The bluff conditions vary significantly from location to location and are described below.

3.1.1 First Place

As a result of apparent slope failure(s), the bluff at the time of the field investigation consisted of a near-vertical scarp, undermining the pavement/sidewalk at the south end of the street, and a slope with a variable inclination, ranging from 1½:1 to 2:1 (horizontal:vertical). Vegetative cover consisted mostly of wild grasses but also included two palm trees. The overall bluff height is on the order of 25 feet.

Subsequent to our initial investigation, the slope at First Place was re-graded, as part of the new condominium development on the east side of First Pace.

3.1.2 Second Place

The Second Place slope appears to have been graded to a relatively uniform slope with an average inclination close to 2:1. Stairs have been constructed in the middle of the slope. Vegetation consists mostly of ice plant. In general, the slope appears to be in stable condition. Total slope height is on the order of 26 feet.

3.1.3 Third Place

The slope conditions are very similar to those at Second Place, with the exception that the overall slope height is on the order of 30 feet.

3.1.4 Fourth Place

The bluff at Fourth Place has a variable slope ranging from 1⅜:1, in the upper/western part of the slope, to 1¾:1, along the eastern edge (against an existing building). The slope within the private property to the west is steeper and exhibits slumping failures in the upper part of the slope. The overall bluff height is on the order of 28 feet. Vegetative cover consists mostly of ice plant and some shrubs. In general, the slope within the public right-of-way appears to be grossly stable.

3.1.5 Fifth Place

The bluff at this location has been partially improved. The slope has been flattened to approximately 2:1 within the western half of the slope, while the upper part of the eastern half of the slope has an inclination on the order of 1½:1 with localized areas approaching 1:1. The overall slope height is on the order of 30 feet. Stairs have been constructed in the western half of the slope. Ice plant covers most of the slope, while heavy growth of Oleander covers the eastern half of the slope.

3.1.6 Sixth Place

The slope at Sixth Place has a variable inclination, ranging from flatter than 2:1 in the middle to nearly 1:1 in the upper/western parts of the slope. The slope is heavily vegetated, with large trees and shrubs covering the west half of the slope. Grasses and shrubs cover the east half of the slope. A retaining wall with a sloping top retains the west side of the slope. The overall slope height is on the order of 32 feet. The slope appears to be grossly stable.

3.1.7 Seventh Place

The bluff at Seventh Place has a gradually variable slope, ranging from about 1¼:1 near the top to about 2:1 over the lower half. Based on information provided by a local resident, we understand that the slope was re-graded to its existing condition and re-planted after a shallow landslide. Currently, vegetative cover over the majority of the slope consists of ice plant. Heavy growth of Bougainvillea covers the western third of the slope. A heavy retaining wall with a sloping top retains most of the western edge of the slope. Shallow, terraced modular block walls have been placed by the residents along the lower eastern parts of the slope. Natural slope drainage appears to be to the two sides, as well as to the south.

3.1.8 Eighth Place

The bluff at the south end of Eighth Place has an overall height of 32 feet and highly variable slope conditions. Stairs have been constructed along the southerly projection of the centerline of the street. The average slope (top to bottom) across the stairs is about 1¾:1, which is consistent with the relatively uniform slope to the east (between Eighth Place and Ninth Place). The slope to the west has been terraced with timber cribbing and has an overall inclination on the order of 1½:1. Vegetation consists of shrubs and ice plant.

3.1.9 Ninth Place

The slope at the south end of Ninth Place has been re-graded to a 2:1 slope within the eastern half, transitioning to a 1¾ slope to the west. Stairs have been constructed along the southerly projection of the street centerline. The overall slope height is about 32 feet. Vegetative cover includes ice plant, flowers, shrubs, and palm trees. The slope appears to be grossly stable. However, localized erosion and minor slumping was noted on the east side of the stairs along the alignment of a storm drain, which is buried in the slope. At this time, the storm drain outlet is located approximately 30 feet south of the toe of slope.

3.1.10 Tenth Place

The slope at the south end of tenth place has been graded to an inclination of 2:1 and stairs have been constructed along the middle of the slope. To the east of the stairs, the slope transitions to a steeper inclination matching the slope of the adjacent private property (about 1¼:1). Vegetation consists of flowers and palm trees on the west side, and wild grasses and shrubs on the east side. The slope within the public right-of-way appears to be grossly stable.

3.1.11 Eleventh Place

Variable slope conditions prevail at the south end of Eleventh Place. The middle of the slope has been flattened to an inclination of 2:1 and stairs have been constructed. The two sides have steeper slopes. On the west side, due to physical constraints imposed by the adjacent private property, the overall slope inclination is about 1½:1 with localized steeper sections. Similarly, the slope on the east side also has an average inclination of 1½:1 although there are no physical constraints for a flatter slope. The slope, which has an overall height on the order of about 32 feet, appears to be grossly stable. However, slope creep was noted in localized, over-steepened areas of the slope, such as the upper parts, west of the stairs. Additionally, the contouring of the slope results in drainage being directed to the two sides of the slope. Vegetative cover consists mostly of ice plant with one tree on the west side of the slope. A concrete retaining structure supporting up to about 15 to 20 feet of earth above the slope surface is located on the east side of the Eleventh Street slope. The filled area supports auto parking stalls.

3.1.12 Twelfth Place

The bluff at the south edge of Twelfth Place is very steep, exhibits evidence of surficial failures/slumps, and has been shored by a variety of means. The overall slope height is on the order of 30 feet. The slope is terraced (between rows of shoring) with an overall (top to bottom) inclination of 1:1. A masonry retaining wall is visible along the toe of the slope. The wall appears to wrap around the eastern edge of the public right-of-way and supports grade differentials of 5 to 8 feet. Two rows of 'make-shift" shoring with timber lagging support the upper parts of the slope. The upper part of the slope in the western part of the adjacent property is slumped with near vertical scarps. A driveway (Bluff Way) providing access to the beach is located along the toe of slope.

3.1.13 Thirteenth Place

The majority of the slope at Thirteenth Place is uniformly sloped about 1½:1. The eastern parts of the slope are somewhat steeper, due to physical constraints imposed by the adjacent property. Overall slope height is about 30 feet. The slope appears to be grossly stable although the slope surface is uneven. Vegetation consists of ice plant, covering most of the slope, large bougainvillea shrubs in the eastern half of the slope, and two palm trees.

3.1.14 Fourteenth Place

At the south end of Fourteenth Place stairs have been constructed in the mid section of the slope, which has an overall height of about 30 feet. Along the stairs, the slope has an average inclination of about 2½:1. However, at the two edges, the slope is steeper (up to about 1½:1 on the west side). The slope surface is sparsely vegetated with ice plant. The slope appears to be grossly stable.

3.1.15 Fifteenth Place

The slope at the south end of Fifteenth Place has an overall height of about 35 feet. The upper 20 feet of the slope has an average inclination of 1½:1. There is a timber retaining structure in the middle of the slope. The slope becomes progressively flatter below the retaining structure, gradually transitioning to the relatively flat beach beyond the toe. Vegetative cover consists mostly of ice plant. The slope does not exhibit any apparent signs of instability.

3.1.16 Bixby Park (16th Place to Junipero Avenue)

The slope along the south edge of Bixby Park has an average inclination of about 2½:1 and an overall height of about 44 feet. A tunnel crosses the slope near the west end. Vegetative cover consists of light growth of wild grasses and, locally, ice plant. In general, the slope is stable and exhibits only localized areas of erosion, apparently due to concentrated surface water flow. The most significant erosion was noted in the western parts of the slope near the tunnel.

3.1.17 Junipero Avenue Ramp

The slope on the south and southwesterly side of the ramp embankment has an average inclination of 2:1 and overall height ranging up to 35 feet (at the west end). The slope appears to be grossly stable. However, extensive erosion gullies have formed over the slope surface, apparently as a result of concentrated flow of water. The depth of these gullies is typically on the order of two feet. The soils exposed at the surface of the slope appear to be compacted. Vegetation on the slope surface is sparse, consisting of light growth of wild grasses and a few palm trees.

Subsequent to our initial field investigation (May 2000), parts of the Junipero Avenue ramp were regraded and replanted as part of a plant demonstration project.

3.1.18 Bluff Park - Museum to East of Lindero Avenue

This segment of bluff spans approximately 500 feet from the east edge of the Long Beach Museum of Arts to the west end of the gabion-stabilized slope, east of Lindero Avenue. This section of bluff, which has an overall height of about 40 feet is essentially unimproved, with variable slope inclination, slumped areas, and sparse vegetation. A buried wall is located at the toe of slope. The average slope inclination is on the order of 1½:1, with near-vertical portions near the middle of the slope and generally much flatter portion near the bottom of the slope. The variable slope conditions can be attributed to subtle variations in soil layering, with the steeper portions corresponding to cohesive soil layers and flatter portions to sandy layers. The steep areas have very uneven profile, exhibiting deep erosion and slumping.

3.1.19 Gabion-Stabilized Area West of Temple Avenue

This 900-foot long segment of bluff-extending up to Temple Avenue, has been stabilized using gabions (blocks retained by wire mesh). The gabions are located in the middle section of the slope and have a near vertical exposed face. The smooth-graded slopes above and below the gabions have average inclinations of about 2:1. A buried wall is located at the toe of slope. The slope has an overall height in the range of 36 to 38 feet and an average inclination of 1⅝:1. Near the middle of this bluff segment, the slope has been flattened to 2:1 and stairs have been constructed. Vegetation above the gabions consists of shrubs. Vegetation in the slope segment below the gabions is sparse. No evidence of gross slope instability was noted in this segment of bluff.

3.1.20 Bluff Park - East of Temple Avenue

This ½-mile segment of bluff is mostly unimproved with a few localized exceptions. In general, the condition of the bluff is very similar to the segment east of the museum, with uneven slope surface, sparse vegetation, erosion gullies and slumped areas, and generally steeper slope inclinations in the mid-portions of the slope. The average height of slope is about 37 feet.

Remnants of two ramps are located east and west of the stairs at Coronado Avenue. The ramps are approximately 400 feet long and do not extend to the toe of slope. Severe erosion has undermined parts of the ramps, as well as parts of the sidewalk at the top of slope.

Two relatively short sections of gabions are located near the east end of the bluff, east and west of Redondo Avenue. These sections are approximately 160 feet and 200 feet long, and have slope geometries similar to those described previously (Section 3.1.19).

Stairs have been constructed across the slope at Orizaba Avenue, Coronado Avenue, and Loma Avenue. At each of these locations, the slope has been flattened relative to the adjacent slope areas, either by moving the top of slope to the north (Coronado Avenue) or moving the toe to the south (Orizaba and Loma Avenues). The average slope inclination across the stairs is on the order of 2:1.

3.2 SUBSURFACE SOILS CONDITIONS

The natural soils across the entire bluff area consist predominantly of dense, fine-grained sands and silty sands, interbedded with very stiff to hard clays and silts. In general, the soil profile becomes progressively sandier and denser with depth. The distribution of clay and silt layers varied from location to location. However, in general, these layers were found more frequently in the upper half of slope areas. The natural soils are marine terrace deposits and have engineering properties comparable to those of well-compacted fill and approaching those of friable/weathered sedimentary bedrock.

Embankment fills, consisting of reworked native soils were encountered in the ramp at Junipero Avenue. These fills appeared to be compacted. Locally, scattered fills are also present across other parts of the bluff. These fills have typically been placed as part of slope repairs and include both engineered fills (mostly near stairs built in the last decade) and random, undocumented fills.

The moisture content of soils encountered in our borings was variable. Most of the sandy soils had low moisture contents while silts and clays had much higher moisture. The soils exposed at the face of the bluffs can be expected to have lower moisture content due to evaporation.

3.3 GROUNDWATER CONDITIONS

Groundwater was encountered in the lower parts of our borings at elevations ranging between +4 and +9 feet (MLLW datum). The groundwater levels can be expected to be impacted by both slope irrigation and tidal fluctuations.

3.4 SEISMIC HAZARDS

The main seismic hazard affecting the site area is the potential for strong ground motion, resulting from earthquakes in the Los Angeles Basin. The closest two known active earthquake faults are the Newport-Inglewood Fault, located approximately 1½ mile northeasterly of the project site, and the Palos Verdes Fault, located approximately 5½ miles southwesterly of the site. We performed probabilistic seismic hazard evaluations, using the computer program FRISKSP with the California Division of Mines and Geology Fault data base. Our analysis, which combines the effects of earthquakes on faults located within a 60-mile radius from the site indicates that peak ground acceleration of 0.55g has a 10 percent probability of being exceeded during a project design life of 50 years.

There are no known active faults crossing the site area. Therefore, the potential for ground rupture due to faulting is negligible.

The sandy soils encountered below the groundwater level are dense to very dense. Therefore, soil liquefaction is unlikely to occur at the bluffs, even under very strong earthquake shaking.

The main hazard related to earthquakes is the potential for slope instability, which is addressed later in this report.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

The results of our geotechnical investigation indicate that while the soil conditions exhibit only subtle variations across the project site, the bluff conditions are highly variable. The primary factors affecting the condition of the bluffs are the steepness of the slopes, drainage conditions, and slope vegetation. Each of these factors are discussed below. Location-specific discussions and recommendations follow this overview.

4.1.1 Slope Inclination

Current standard of practice for engineered slopes is to have a minimum factor of safety of 1.5 against both surficial and deep slope failures under static (gravity) loads and a minimum factor of safety of 1.1 when slopes are evaluated using a pseudo-static lateral force coefficient of 0.15g (to simulate earthquake loading). These standards have been developed empirically, based on actual performance of slopes.

Our slope stability analyses for bluffs with uniform slope inclinations and a total height of 41 feet indicate the following results:

UNIFORM SLOPE INCLINATION (horizontal:vertical)	FACTOR OF SAFETY	
	Static	Pseudo-Static (0.15g)
2:1	1.54	1.09
1½:1	1.20	0.91

From the above results, it can be seen that 2;1 slopes would have adequate factor of safety by current standards while steeper slopes would not.

The existing bluffs have highly variable slope inclinations, but generally are steeper than 1½:1. Exceptions include the bluff on the south side of Bixby Park (2½:1), the embankment slopes for the Junipero Avenue ramp (2:1), and the localized areas adjacent to relatively recently-constructed stairs (2:1). Parts of the bluff have very steep inclinations (1:1 or steeper) and in several areas steeper parts have already failed, in the form of shallow landslides. Slope erosion is also accelerated in areas where the slope is steeper than 2:1.

Based on the results of our analyses, engineered slopes would either have to be inclined 2:1 or flatter or be reinforced with geogrids, soil nails or piles, in order to have an adequate factor of safety against both massive and surficial failures.

4.1.2 Drainage

The predominant soil type at the project site is a fine silty sand, which is highly susceptible to erosion when subjected to concentrated flow of surface water. Surface water from the top of slope should not be channeled to drain over the slope, unless erosion resistant surfaces are built on the slope. Typically, concrete V-channels are constructed to intercept and discharge slope drainage in engineered slopes. If such channels are objectionable from an aesthetic point of view, it is imperative that all surface water from above the slope be intercepted before it reaches the slope and more aggressive protective measures (see Section 4.1.3) be taken, to minimize the erodability of the slope materials.

In several areas of the bluff, including the embankment slopes of the Junipero Avenue ramp, there is evidence that at least part of the erosion was caused by malfunctioning irrigation systems. For example, severe gullying was noted in the vicinity of sprinkler heads. It is imperative that irrigation systems be monitored systematically and adjustments made if any evidence of erosion is noted in the vicinity of sprinkler heads, valves, etc.

Finally, in some areas of Bluff Park, evidence of subsurface erosion was noted near the top of the bluff. The erosion appears to extend under the sidewalk and could have been caused either by malfunctioning irrigation systems or ponding of water on depressed areas of the lawn. Such areas should be re-graded to drain surface water away from the slope. Subsurface drains consisting of drainage pipe in rock filled trenches could also be used to intercept ponded water.

4.1.3 Vegetation

No slope surface, no matter how well drained, would be immune to erosion unless it is also covered with vegetation. We recommend the use of deep-rooting, drought-resistant vegetation. Roots extending to depths on the order of 3 to 5 feet can provide significant slope reinforcement against surficial slope instability. In contrast, shallow-rooted plants requiring heavy irrigation could induce surficial instability.

4.2 LOCATION-SPECIFIC RECOMMENDATIONS

The existing slope conditions as well as physical constraints related to both bluff-top conditions and adjacent properties, vary significantly across the project site. Therefore, preliminary geotechnical recommendations are provided for the various localized site conditions. Where appropriate, recommendations for site areas with similar conditions have been grouped to avoid repetition. Once design concepts are finalized (based on other, non-geotechnical considerations), we should review and, possibly, update our recommendations.

4.2.1 First Place

As noted previously, the upper, relatively steep parts of the bluff at this location have failed, leaving a near-vertical scarp, which undermined the sidewalk at the southern end of the street. Based on the physical constraints imposed by the adjacent private properties, it would not be practically feasible to regrade the slope to a 2:1 inclination, as would be needed to provide an adequate factor of safety. The recommended mitigation concept for this area is to provide a geogrid reinforced modular block wall for he upper parts of the slope, and regrade the remaining parts of the slope to a 2:1 inclination (close to the existing condition).

The design concept is depicted in Figure 3. The details of the design will need to be finalized by a design-build contractor. However, our preliminary evaluations indicate that the geogrids will need to be approximately 15 feet long and will require a temporary back cut extending (at the top) to about 25 feet north of the face of the wall. Thus, the impact of the proposed construction on existing street improvements will need to be evaluated. If there are significant utility conflicts that make this concept prohibitive, a drilled pile wall (as recommended for Twelfth Place) can be substituted for the modular block wall.

If stairs are to be constructed at this location, we recommend that the upper 10 feet of the staircase be oriented in the east-west direction along the face of the wall and then continue in the north-south direction along the 2:1 slope.

Subsequent to the issuance of our preliminary recommendations, presented above, the slope surface was regraded in conjunction with the condominium development on the east side of First Street. The project involved demolition of previously existing structures (which imposed physical constraints) and, thus, facilitated construction of a new slope. This work was done under geotechnical observation and testing by others.

4.2.2 Second and Third Place

The bluff slopes at these two locations appear to be grossly stable and do not appear to require geotechnical mitigation.

4.2.3 Fourth Place

This slope appears to be grossly stable despite the fact that the upper parts of the slope are relatively steep and would not have an adequate calculated factor of safety (i.e., less than 1.5). The adjacent property to the west has apparent slope failures, with near vertical scarps near the top of the slope. The potential of similar shallow failures extending into the public right-of-way cannot be precluded. However, any mitigation measures will need to be coordinated with slope conditions to the west. If the slope of the adjacent property were to be improved by filling the slumped areas, the stability of the slope at the south end of Fourth Place could also be improved by filling the slope areas, as needed, to provide a uniform 2:1 slope. Such a slope could also be constructed by placing a retaining structure, such as geogrid-reinforced modular block wall, along the west edge of the Fourth Place slope. Other design alternatives include concepts recommended for First Place or Twelfth Place.

4.2.4 Fifth Place

The bluff at this location has been partially improved by constructing stairs over a regraded portion of the slope. However, due to apparent space constraints, the eastern parts of the slope are steeper and drain to the southeast. Heavy growth of Oleander covers the steeper parts of the slope and probably provides some reinforcement. The slope appears to be grossly stable at this time, even through the steeper east half of the slope would not have adequate factor of safety against slumping in the southeasterly direction. Due to space constraints imposed by the private property to the east, improving the stability of this slope would require construction of a retaining structure along the east edge of the slope and filling to flatten the slope. Other slope reinforcement alternatives include drilled piles or soil nailing. Detailed geotechnical recommendations will be presented in a site-specific supplement report.

4.2.5 Sixth Place

This slope appears to be grossly stable, despite the fact that the northwestern parts of the slope have steep inclinations. The roots of large trees and shrubs provide significant reinforcement for the mid portions of the slope. The stability of the upper portions of the slope could be improved by constructing relatively shallow modular block walls with geogrid-reinforced backfill. However, based on the current, apparently grossly stable, condition of the slope, this part of the bluff could be placed on a lower priority for improvement.

4.2.6 Seventh Place

The slope at this location has experienced shallow landslides with a resulting over-steepened upper portion. The stability of the slope could be improved in the same manner as First Place, with the additional option of constructing a sloping modular block wall in the upper parts of the slope. The existing private property poses significant physical constraints (such as locations of windows) that would need to be considered in the detailed design of the finished slope. Furthermore, as part of the regrading of the slope, we recommend that the existing terraced modular walls, constructed in the southeastern parts of the slope, be removed and replaced with a compacted fill. Detailed geotechnical recommendations will be presented in a site-specific supplemental report.

4.2.7 Eighth Place

We recommend that the existing "make shift" terraced timber walls be replaced with geogrid-reinforced modular block walls. The details of the design will need to be determined by the design-build contractor, based on aesthetic/landscaping criteria determined by the design-build contractor, based on aesthetic/landscaping criteria provided by the landscape architect. However, the geogrids will need to be long enough (probably longer than 10 feet), in order to adequately reinforce the overall slope.

4.2.8 Ninth Place

The slope has been recently graded to a 2:1 inclination and appears to be stable. However, localized gullying has been observed along the alignment of a sloped storm drain, which we understand underlies the slope surface. We suspect that the observed subsidence and erosion could be due to settlement of the storm drain backfill. We recommend that the backfill be investigated further and if found to be loose, as we suspect, be replaced with compacted fill.

4.2.9 Tenth Place

The western half of the slope, including the location of the stairs has an adequate factor of safety against failure. The slope under the adjacent property to the east does not have an adequate factor of safety against slope failure, and the eastern half of the Tenth Place slope provides a gradual transition between the flatter slope to the west and the over-steepened slope to the east. If the slope on the adjacent property were to fail (due to excessive steepness), it would also impact the slope within the eastern parts of the public right-of-way. However, any improvements within the eastern half of the slope would need to be coordinated with improvements in the adjacent private property. At this point, the slope within the public right-of-way does not exhibit obvious signs of instability, and improvement of this slope, beyond planting deep-rooted vegetation, does not appear to be a high priority. The stability of this slope could be improved by constructing a drilled pile wall along the eastern property line, and then, excavating the upper parts of the slope to a uniform inclination of 2:1.

4.2.10 Eleventh Place

Along the center, this slope has been flattened to 2:1 and stairs have been constructed. The stability of the eastern parts of the slope (which is steeper) could be improved by raising the grades along the eastern parts of the slope and at the toe of the adjacent retaining structure. Due to physical constraints imposed by the adjacent private property, improvement of slope conditions along the western edge of the public right-of-way could be prohibitively expensive at this time. The potential for massive, deep-seated slope failures at this location is remote. However, there is a significant potential for surficial slope creep and shallow slope failures in the over-steepened portions of the slope. These conditions could be partially improved by planting deep-rooted, drought resistant vegetation. Full mitigation, to improve the factor of safety against surficial failures to over 1.5 (under gravity loads) will involve more elaborate measures similar to those shown for Twelfth Place.

4.2.11 Twelfth Place

This slope has apparently been repaired on several occasions using a combination of steel piles with timber lagging and a stone retaining wall. The resulting composite slope does not have an adequate factor of safety against either deep-seated massive failure (under the retaining wall) or surficial failure. In fact, the western parts of the slope have slumped and eroded. We understand that the grades at the toe of the slope, may be raised to improve access to the beach from Eleventh Place, which should help the overall stability and facilitate the concepts recommended herein. We recommend two alternative concepts, depicted in Figure 4, for adequately improving the stability of this slope.

The first concept involves construction of a drilled pile wall to support the upper 10 feet(±) of the slope and a 2:1 slope below the wall. Our preliminary evaluations indicate that the piles will need to be approximately 35 feet long, have a diameter of 30 inches and be spaced at 6-foot intervals. Computer analyses of lateral deflections, bending moments, and shear failure for the embedded portion of the piles (below the surface of the 2:1 slope) are presented graphically in Figure 5. A shotcrete wall face should be constructed to cover the space between the piles. The shotcrete section would need to be structurally tied to the piles. The structural design of the wall should be performed by a structural engineer, based on the geotechnical parameters presented in this report. The wall should be drained by using a combination of pre-fabricated drainage composites, such a Miradrain, behind the shotcrete wall and weep holes through the wall. The main advantages of this concept are that it involves no excavation in the street and can be easily adjusted to match the geometry of the support conditions on the adjacent property to the west.

The second concept involves construction of a geogrid-reinforced modular block wall, in the upper part of the slope with a 2:1 descending slope below. This concept will require excavations into the street area, in order to accommodate placement of geogrid reinforcement. The design of such a wall should be performed by a turn-key design-build contractor. However, our preliminary evaluations indicate that the geogrids will need to extend about 15 feet behind the face of the wall. Allowing for a 1:1 back cut, excavations approximately 20 feet beyond the top of the wall into the street will be required.

Detailed geotechnical recommendations for the final design concept will be presented in a site-specific supplemental report.

4.2.12 Thirteenth, Fourteenth and Fifteenth Place

The existing slopes at these locations appear to be grossly stable even though parts of the slopes are over-steepened and do not have an adequate calculated factor of safety of 1.5. In general, due to physical constraints imposed by adjacent properties, flattening of the slopes to 2:1 cannot be done economically. Partial improvement in surficial stability could be achieved by planting deep-rooted drought-resistant vegetation at this time. The slopes should be monitored for any signs of slope creep and additional improvement considered, if signs of instabilty are detected.

4.2.13 Bixby Park and Junipero Avenue Ramp

The slopes at this location are adequately flat to have a minimum factor of safety of 1.5 under static conditions and 1.1 under pseudo-static/seismic conditions. However, the western parts of the Bixby Park slope and most of the Junipero Avenue ramp slope have severe erosion gullies. The concentrated slope erosion is apparently due to inadequate drainage from up slope areas and malfunctioning of irrigation systems. We recommend a combination of four measures for improving the condition of these slopes, as discussed below.

a. The eroded areas should be filled with compacted fill after removing any loose soil, muck, debris, and vegetation from the gullies. Due to poor access to construction equipment, this work will need to be done mostly with manual labor.

b. The slope should be planted with deep-rooting drought-resistant vegetation. Existing ground cover is sparse and consists mostly of wild grades and low shrubs.

c. The drainage conditions above the slope should be evaluated by a civil engineer and surface drainage be improved to prevent concentrated flow over the slope.

d. The irrigation system should be monitored to prevent directing any concentrated source of water (such as from a broken sprinkler head) onto the slope.

After our preliminary geotechnical investigation, a portion of the Junipero Avenue Ramp slope was improved as part of a planting demonstration project.

4.2.14 Bluff Park - Gabion Stabilized Slope Areas

None of the Gabion-stabilized portions of the bluffs show evidence of instability (surficial or deep seated). The computed factors of safety against massive slope failure under the Gabions range from 1.02 to 1.33 under static conditions and from 0.79 to 1.00 under pseudo static loading conditions. Considering that these parts of the bluff do not show signs of instability, we recommend that improvement of these bluffs be placed on a lower priority than other sections. On the other hand, the factors of safety against massive slope failure could be improved to adequate values by reinforcing the bluff by soil nailing placed below the base of the Gabions. Soil nails consist of steel rods, encased in grout, placed in drilled near-horizontal holes. The design of soil nailing is performed by turn-key design-build contractors. However, based on similar past projects, for preliminary planning purposes, we anticipate the need for two rows of soil nails placed at approximate spacings of 5 feet. The length of the nails would be on the order of 35 feet. The nails would be inclined approximately 15 degrees from horizontal to facilitate placement of the grout.

4.2.15 Bluff Park - Unimproved Slope Areas

The majority of the bluff, along Bluff Park, is in its natural unimproved condition. The slope surface is uneven, eroded, with many areas of shallow slump-type failures. If these slopes remain unimproved, they will continue to erode and fail, typically in short sections. The factor of safety against both surficial and deep failures can be improved to adequate values by flattening these slopes to an inclination of 2:1 (horizontal to vertical). This can be accomplished either by cutting into the top of the slope by moving the toe of the slope to the south and placing imported fill.

Along portions of the bluff, where 2:1 slopes cannot be accommodate due to space constraints or not desired, two types of geogrid reinforced embankments can be used. One such area is the "Gliders Slope" where, we understand, the mid section of the slope would need to be 1:1 or steeper. In Figure 6, we present two alternative concepts for such areas. For mid-slope inclinations steeper than ¾:1, segmental block walls with geogrid-reinforced backfill could be used. For mid-slope inclinations of 1:1 or flatter, geogrid reinforced embankments could be used. Some types of segmental block walls have pockets for plants. The design of geogrid reinforced walls and embankments will need to be performed by a design-build contractor. However, the conceptual designs shown in Figure 6 were evaluated for gross stability and could be used for preliminary planning purposes.

4.3 EARTHWORK

4.3.1 Overview

The scope of earthwork activities anticipated for this project will vary significantly from location to location. In some areas earthwork will be limited to minor slope repairs such as filling shallow erosion gullies. In other areas where the gross stability of the slope will need to be improved, much more extensive earthwork will be required. Recommendations presented below cover the range of earthwork activities that are anticipated. In general, these activities include clearing and grubbing, excavation, subgrade preparation, as well as placement and compaction of fill.

4.3.2 Clearing and Grubbing

Prior to grading, all areas to be filled should be stripped of all vegetation, debris, and loose soils. Gullies should be widened, as needed, to fit compaction equipment.

4.3.3 Excavations

Minor excavations, to depths on the order of 4 feet or less will need to be made to remove loose soils and debris in gullies and for various irrigation and drainage trenches. Such excavations could be made with near vertical sides. In eroded areas to be filled, the sloping bottom of the excavation should be benched/stepped in order to facilitate placement and compaction of fill. Such steps should be at least 2 feet wide.

More significant excavations will be required in areas where the overall stability of the bluff slopes will need to be improved by placing compacted fill buttresses or geogrid-reinforced embankments. The sides of temporary excavations deeper than 4 feet will need to be sloped or shored. In general, temporary cuts into the undisturbed native formation materials can be expected to be grossly stable with inclinations on the order of 1:1. However, temporary cut stability will need to be confirmed based on soil conditions exposed in cuts during grading. For example, in some areas where relatively clean sands are exposed in the lower parts of the bluffs, the lower parts of the cut may need to be flattened to 1½:1 in order to prevent excessive sloughing and raveling. On the other hand, in areas where clayey soils are exposed in the cut (usually the upper parts of Bluff Park slopes), steeper cuts may be stable over parts of the slope.

All excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards.

Excavations into the on-site materials can be expected to be feasible using conventional excavation equipment such as backhoes, excavators, loaders and scrapers.

4.3.4 Subgrade Preparation

Subgrade soils in areas to receive fill should be scarified, moisture-conditioned, and compacted to densities equal to at least 90 percent of maximum density (ASTM D1557).

In areas to be paved, the finish subgrade soils should be scarified to a depth of 6 inches, moisture-conditioned, and compacted to at least 95 percent of maximum dry density in accordance with ASTM D 1557.

4.3.5 Material for Fill

Material available from on-site excavations may be used for fill, subject to limitations discussed in this report. The majority of materials available from on-site excavations will break down to silty and gravelly sands, which will be suitable for use as non-expansive fill. However, clayey soil layers are also present at the site, particularly in the upper portions of the bluffs along Bluff Park. Clayey soils should not be used directly in geogrid reinforced embankment fills or in wall backfills. The clayey soils could be used in such fills, if they are thoroughly blended with sands to form a uniform mixture that has less than 50 percent fines and low expansion potential. In general, each unit volume of clayey soil would need to be blended with two volumes of sandy soil to form a satisfactory mixture.

Imported fill material should be predominately granular, have relatively low expansion potential (EI ≤40) and contain no more than 50 percent fines (portion passing No. 200 sieve). The Geotechnical Engineer should be notified at least 72 hours in advance, of the location of any soils proposed for import. Each proposed import source should be sampled, tested, and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by the Geotechnical Engineer may be rejected if not suitable for use as compacted fill.

Crushed, inert demolition debris, such as crushed concrete may be used in deeper parts of fills (at least 8 feet below the slope surface) with the following processing requirements:

- If the inert debris is crushed to a well graded mixture with maximum particle size of 1½ inches, the crushed material may be used directly in the fill without further blending.

- Inert debris up to a maximum size of 6 inches may also be used in fills provided it is thoroughly blended with moisture-conditioned on-site soil to form a well graded mixture. In general, at least three volumes of soil will need to be blended with each volume of debris.

All materials used for fill should be free of organics and pieces larger than 6 inches.

4.3.6 Placement and Compaction of Fills

All fills should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to dry densities equal to the following percentages of their respective maximum densities (ASTM D1557):

Upper 8 inches of pavement subgrade:	95 percent
Slope fill deeper than 3 feet of slope surface:	93 percent
Slope fill in outer 3 feet of slope surface:	90 percent
All other backfills:	90 percent

Compaction should be verified by in-place density tests performed to within 6 inches of the finish slope surface.

The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors (wackers)	4-6 inches
Small vibratory or sheepsfoot rollers (5-ton±)	6-8 inches
Loaders and heavy vibratory rollers	8-12 inches

The maximum lift thickness should never be greater than 12 inches.

In general, small equipment such as plate compactors, walk behind vibratory, or small sheepsfoot rollers will need to be used to achieve the required compaction in the outer 3 feet of the slope surface. Alternatively, the slope may be overfilled and trimmed back to well compacted soil.

In general, the on-site soils should be placed and compacted at moisture contents at or slightly above optimum. The moisture content of the on-site soils is variable, ranging from well below optimum to well above optimum. Moisture conditioning (both wetting and drying) will be needed in order to facilitate compaction.

During backfilling of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Bulking is the gain of soil volume caused by compaction of fills to a lower density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. Subsidence in the underlying materials should be nominal (less than 0.1 foot). These values are estimates only. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Fill Settlement and Slope Creep

The deeper portions of fills will compress under the surcharge imposed by the overlying materials. The potential for long-term, post-construction settlement can be minimized by placing and compacting the fills at a moisture content wet of optimum, as recommended herein. Post-construction settlement at the surface of fills, placed and compacted as recommended in this report, are expected to be on the order of one inch or less.

4.3.8 Observation and Testing

A representative of GPI should observe all excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of any additional fill.

4.4 SHALLOW FOUNDATIONS

The proposed improvements will not include buildings or major structures. The recommendations presented below are for minor structures, such as small retaining walls (less than 5 tall) or equipment pads supported on undisturbed native ground or engineered/compacted fill.

Maximum allowable static bearing pressure:	2,400 psf
Maximum allowable bearing pressure (wind or seismic):	3,200 psf
Minimum footing width:	18 inches
Minimum footing depth:	18 inches
Lateral passive resistance - flat ground:	300 psf/ft
Lateral passive resistance - 2:1 descending slope:	150 psf/ft
Minimum horizontal distance from slope face to footing edge:	3 feet
Friction coefficient for base of footing:	0.4

4.5 DRILLED PILES

Drilled piles are primarily considered as a means of reinforcing overly steep slope areas at 12th Place, and possibly 5th Place and 7th Place. The design of these piles will be governed by lateral load considerations. The preliminary concepts evaluated for this report were based on the lateral load analyses summarized in Figure 5. More detailed recommendations for design and construction of drilled piles will be presented in site-specific supplemental reports, once the preliminary concepts are finalized.

4.6 LATERAL EARTH PRESSURES

Lateral earth pressures for the design of retaining structures will depend on local soil conditions as well as the final slope geometry. Detailed recommendations will be provided on a site-specific basis in supplemental reports. The following lateral earth pressure parameters were considered for the preliminary concepts presented in this report.

Active soil pressures can be used for retaining systems that can be allowed to yield sufficiently (on the order of one inch or more at the top) to mobilize the shear strength of the soil. Active earth pressures used for our preliminary evaluations are represented by a triangular pressure distribution, equivalent to the hydrostatic pressure of a fluid having the following unit weights:

Horizontal ground above wall:	36 pcf
2:1 Sloping ground above wall:	54 pcf

5.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by Tetra Tech, Inc. and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided during grading, excavation, and foundation construction by GPI. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If construction phase services are performed by others they must accept full responsibility for all geotechnical aspects of the project including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

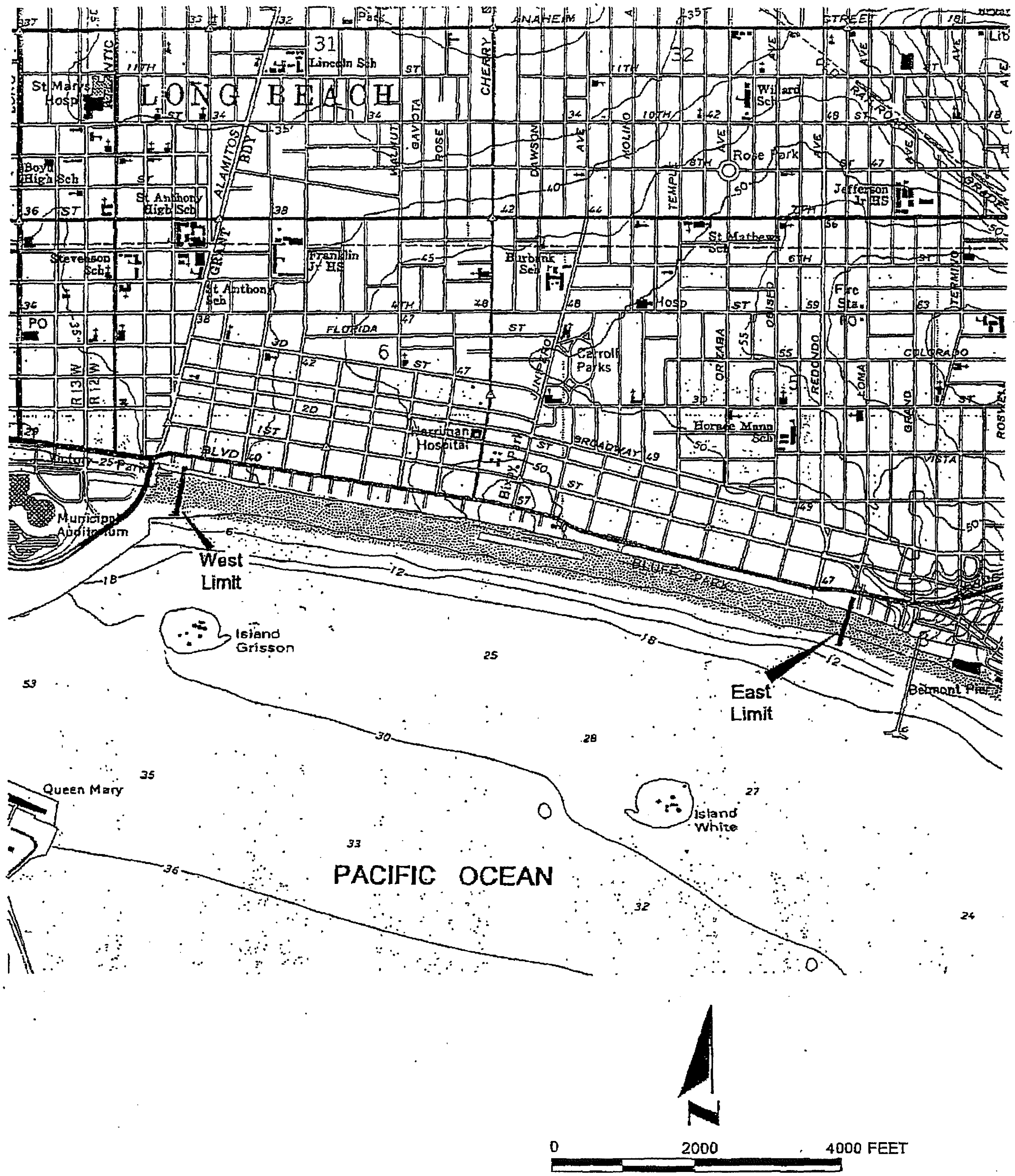
Respectfully submitted,
Geotechnical Professionals Inc.

Byron Konstantinidis, G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
BYRON KONSTANTINIDIS
NO. GE482
Exp. 12-31-2005
GEOTECHNICAL
STATE OF CALIFORNIA

SEP - 3 2003

BK:sph



SITE LOCATION MAP REPRODUCED FROM U.S.G.S. 7.5 MINUTE QUADRANGLE MAP

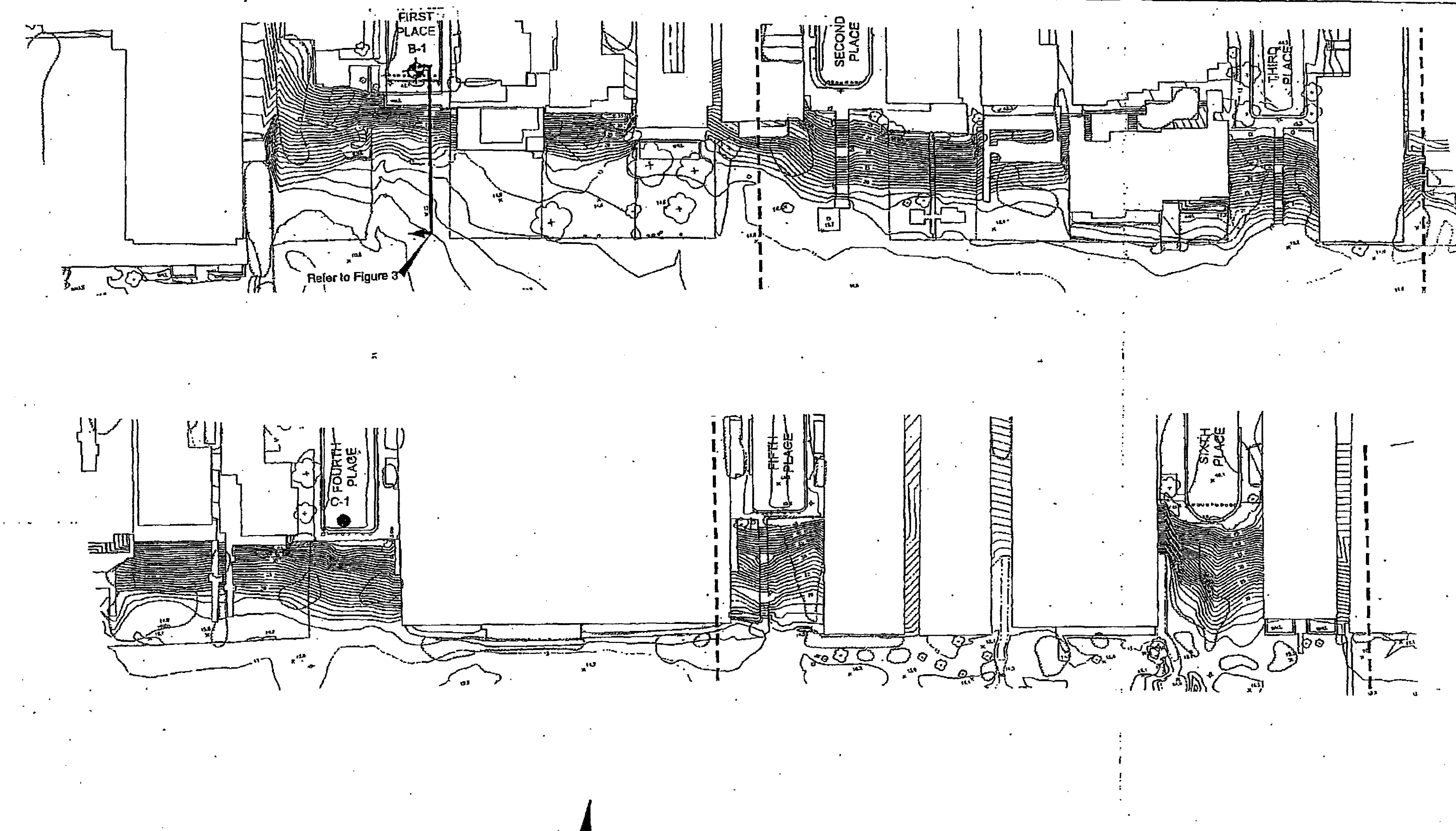
GEOTECHNICAL PROFESSIONALS, INC.

LONG BEACH/BELMONT SHORE - BLUFF RESTORATION

GPI PROJECT NO.:1652.I	SCALE: 1"=2000'

SITE LOCATION MAP

FIGURE 1

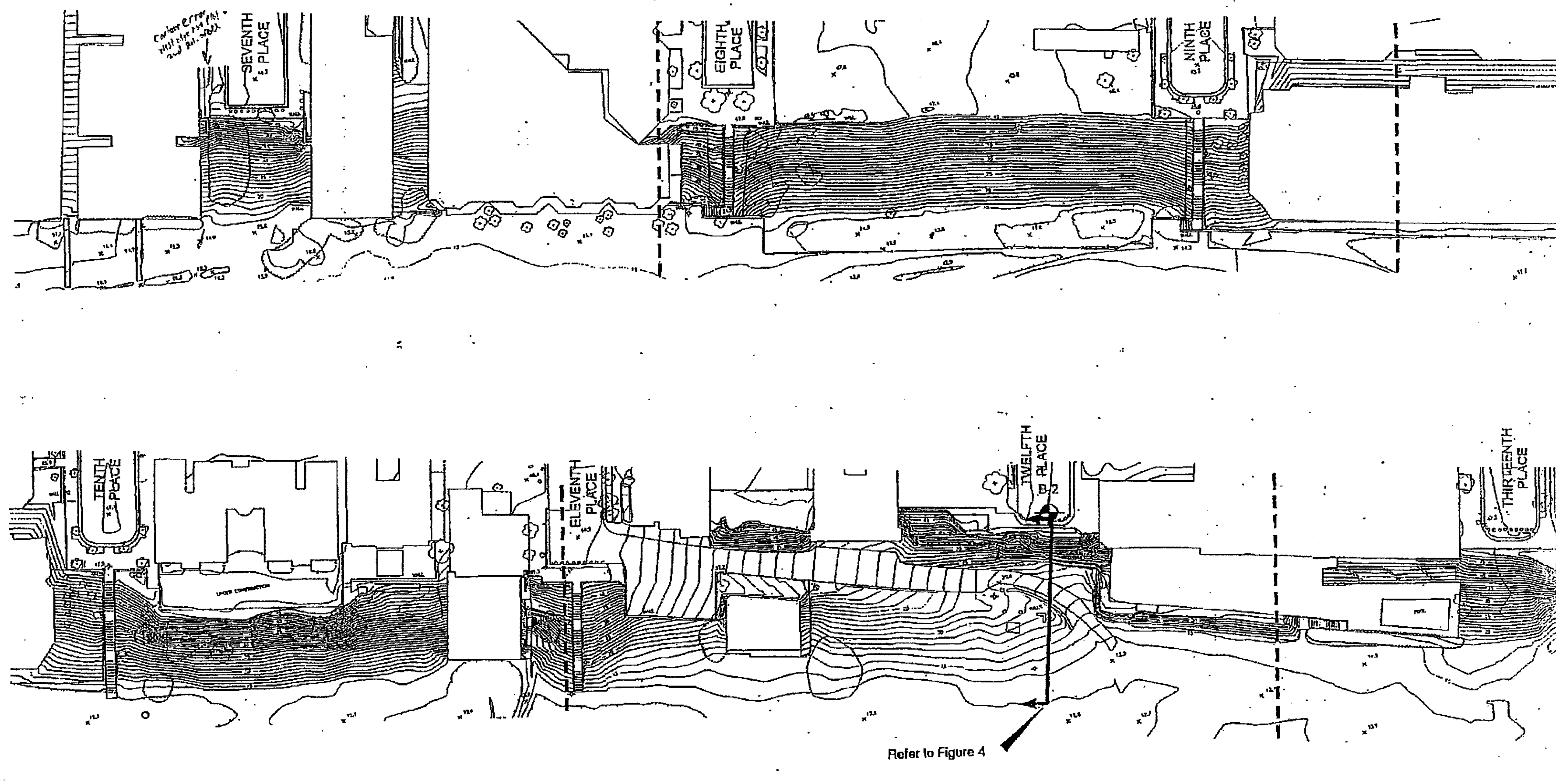


EXPLANATION

- ● APPROXIMATE LOCATION OF CONE PENETROMETER TEST
- ⌖ APPROXIMATE LOCATION OF EXPLORATORY BORINGS

SITE PLANS REPRODUCED FROM TOPOGRAPHIC PLANS PROVIDED BY TETRA TECH, INC.

GPI GEOTECHNICAL PROFESSIONALS, INC.	SITE PLAN
BLUFF RESTORATION	
GPI PROJECT NO.: 1652.1 — SCALE: 1"=60'	FIGURE 2.1

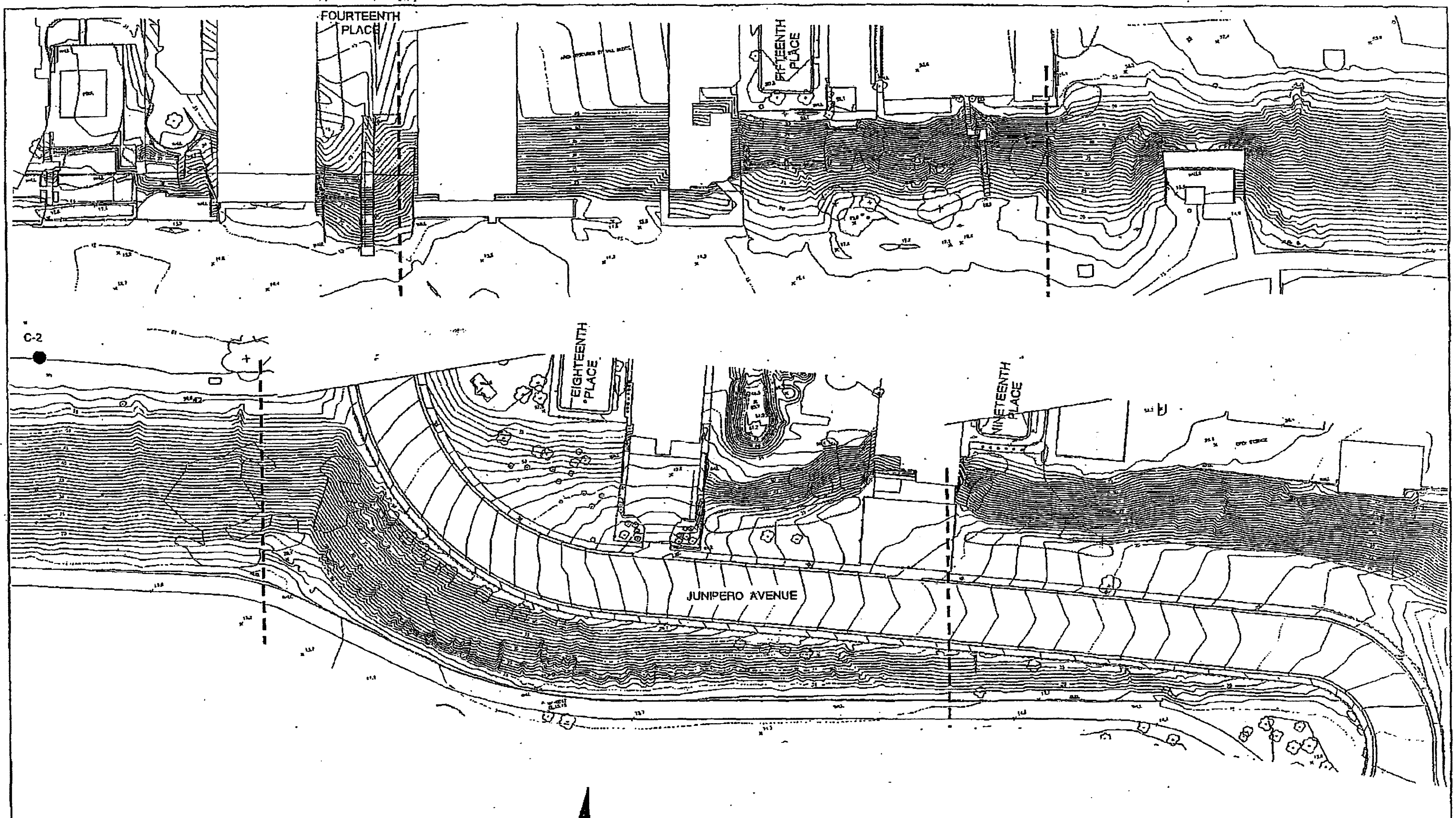


EXPLANATION

- APPROXIMATE LOCATION OF CONE PENETROMETER TEST
- APPROXIMATE LOCATION OF EXPLORATORY BORINGS

SITE PLANS REPRODUCED FROM TOPOGRAPHIC PLANS PROVIDED BY TETRA TECH, INC.

GPI GEOTECHNICAL PROFESSIONALS, INC.	SITE PLAN
BLUFF RESTORATION	
GPI PROJECT NO.: 16521 SCALE: 1"=60'	FIGURE 2.2



SITE PLANS REPRODUCED FROM TOPOGRAPHIC PLANS PROVIDED BY TETRA TECH, INC.

EXPLANATION

- ● APPROXIMATE LOCATION OF CONE PENETROMETER TEST
- ⯐ APPROXIMATE LOCATION OF EXPLORATORY BORINGS

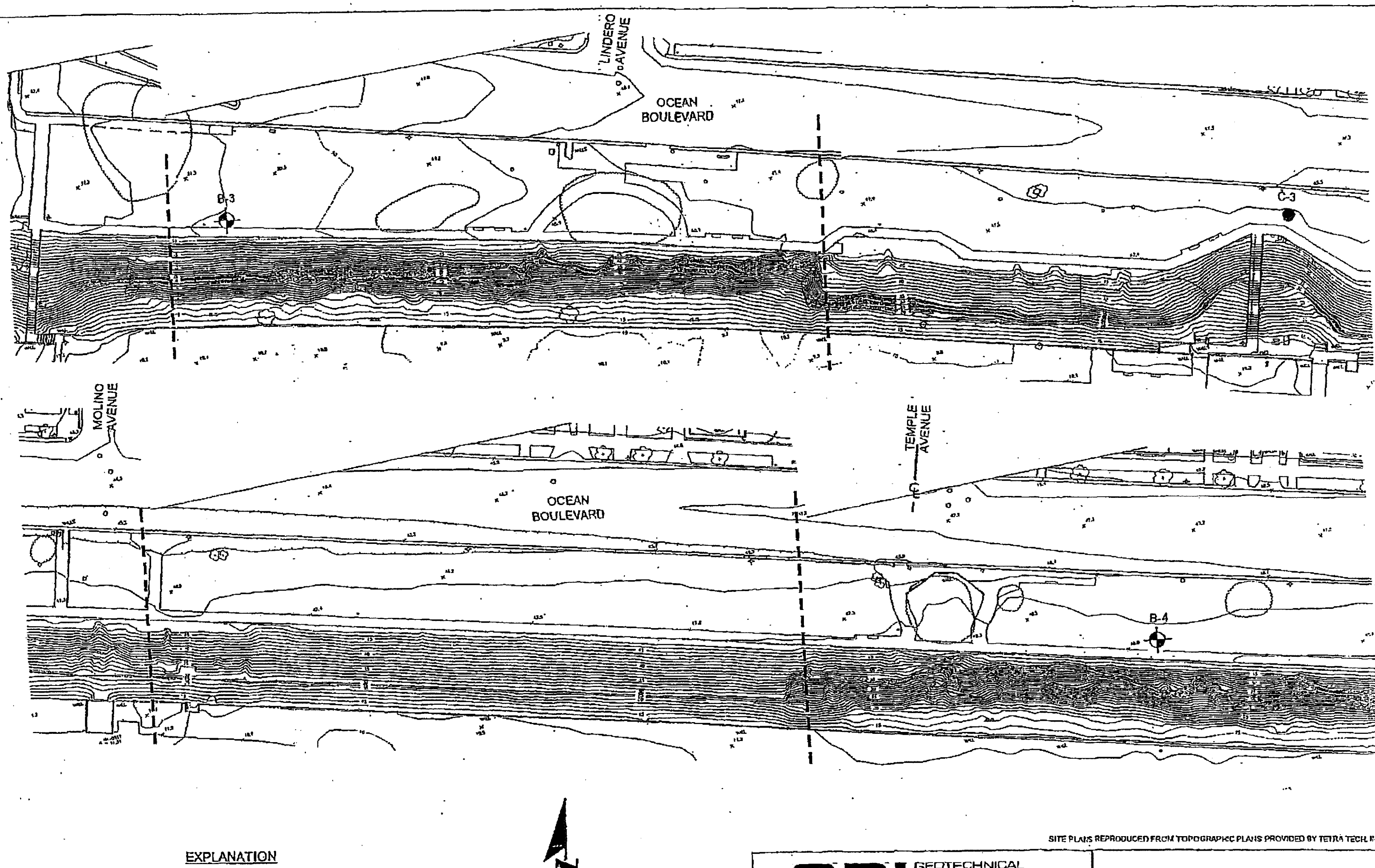
GPI GEOTECHNICAL PROFESSIONALS, INC.

BLUFF RESTORATION

GPI PROJECT NO.: 1652.I | SCALE: 1"=60'

SITE PLAN

FIGURE 2.3



SITE PLANS REPRODUCED FROM TOPOGRAPHIC PLANS PROVIDED BY TETRA TECH, INC.

EXPLANATION

- APPROXIMATE LOCATION OF CONE PENETROMETER TEST
- APPROXIMATE LOCATION OF EXPLORATORY BORINGS

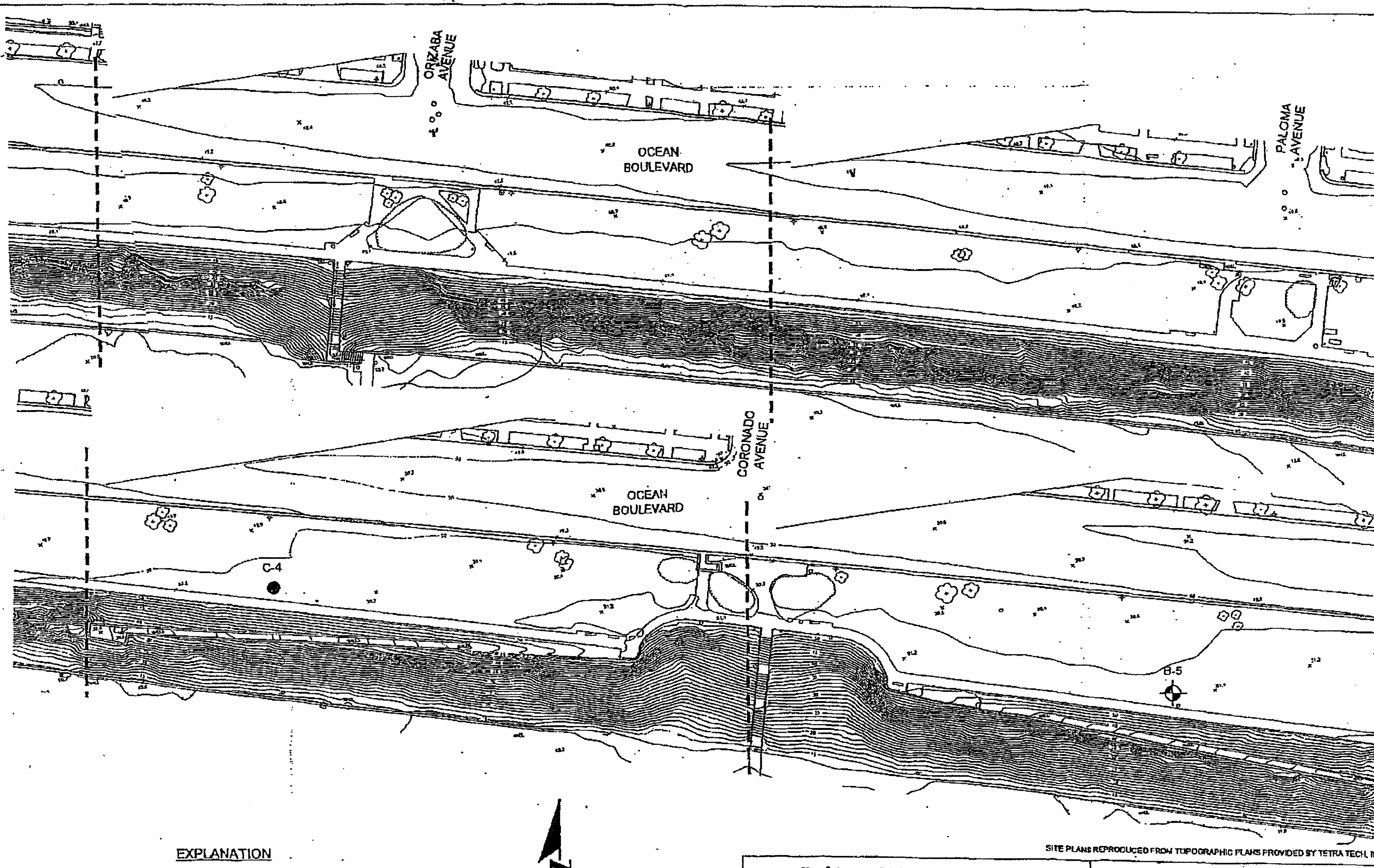
GPI GEOTECHNICAL PROFESSIONALS, INC.

BLUFF RESTORATION

GPI PROJECT NO.: 1652.I | SCALE: 1"=60'

SITE PLAN

FIGURE 2.4

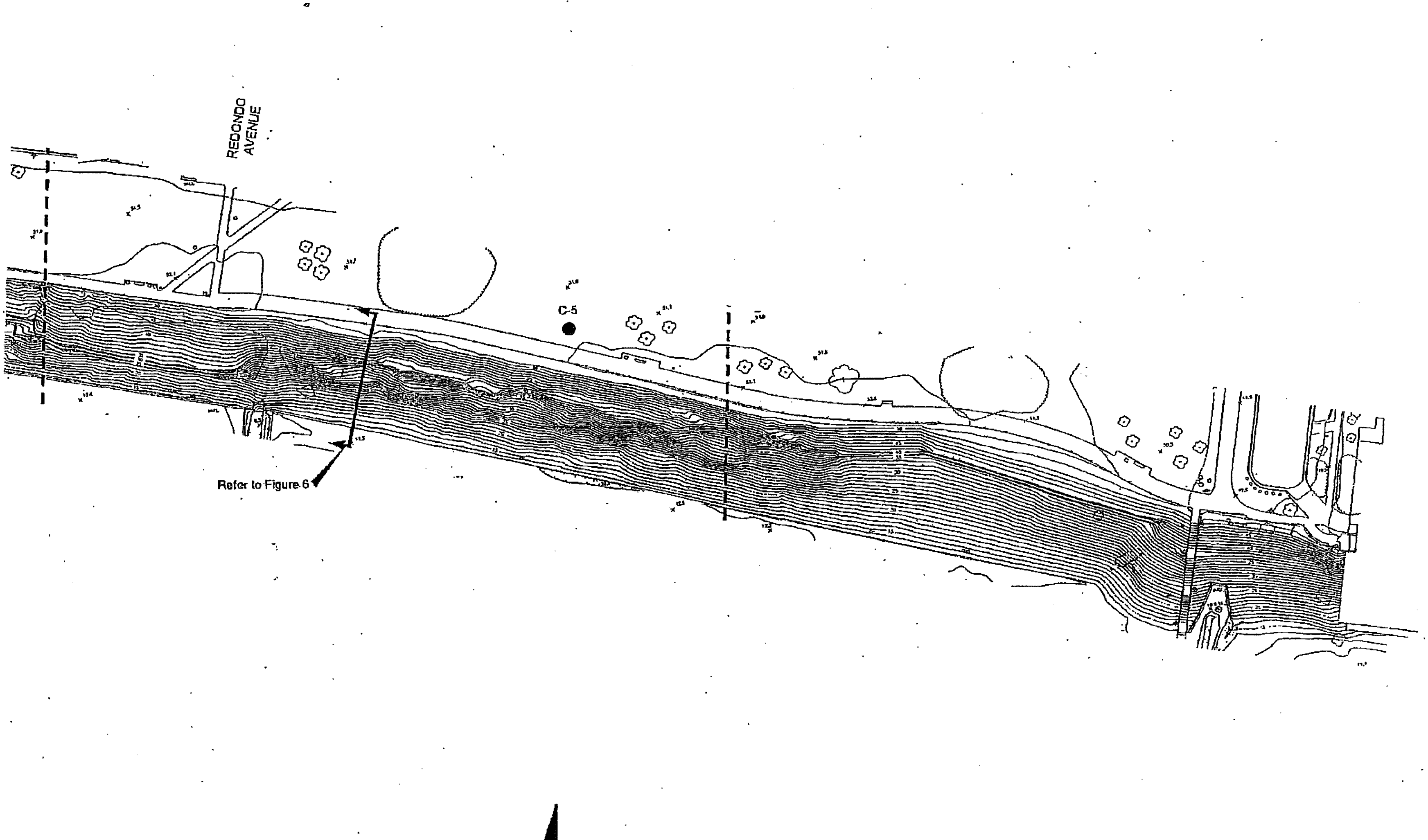


EXPLANATION

- APPROXIMATE LOCATION OF CONE PENETROMETER TEST
- APPROXIMATE LOCATION OF EXPLORATORY BORINGS

SITE PLANS REPRODUCED FROM TOPOGRAPHIC PLANS PROVIDED BY TETRA TECH, INC.

GPI GEOTECHNICAL PROFESSIONALS, INC.		SITE PLAN
BLUFF RESTORATION		
GPI PROJECT NO.: 1652.I	SCALE: 1"=60'	FIGURE 2.5

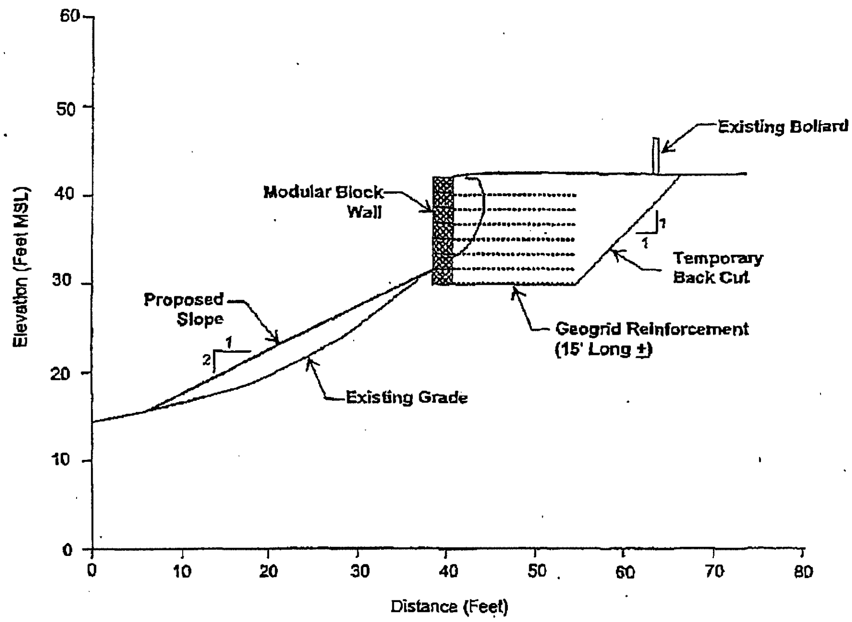


EXPLANATION

- ● APPROXIMATE LOCATION OF CONE PENETROMETER TEST
- ⌖ APPROXIMATE LOCATION OF EXPLORATORY BORINGS

SITE PLANS REPRODUCED FROM TOPOGRAPHIC PLANS PROVIDED BY TETRA TECH, INC.

GPI GEOTECHNICAL PROFESSIONALS, INC.		SITE PLAN
BLUFF RESTORATION		
GPI PROJECT NO.: 1652.I	SCALE: 1"=60'	FIGURE 2.6



GEOTECHNICAL PROFESSIONALS, INC.

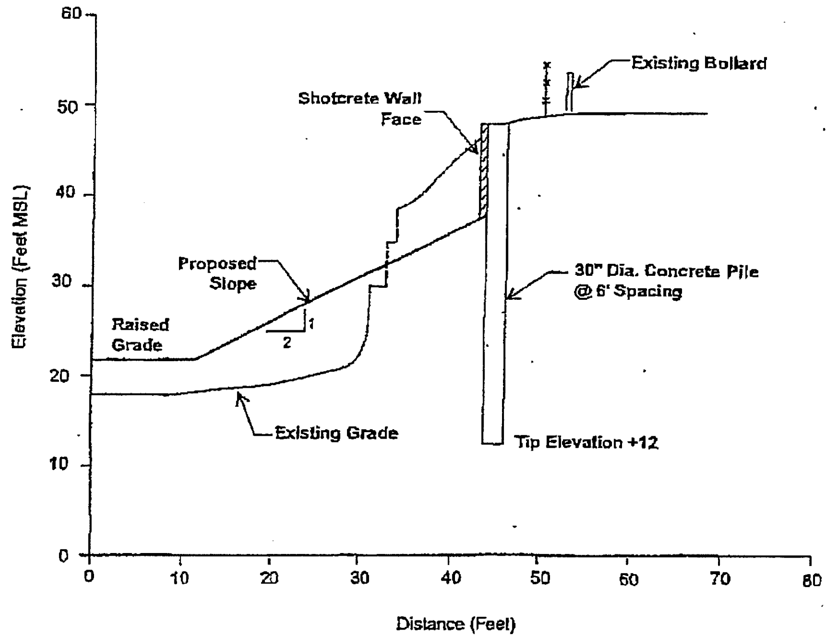
BLUFF RESTORATION

GPI PROJECT NO.: 1652.I

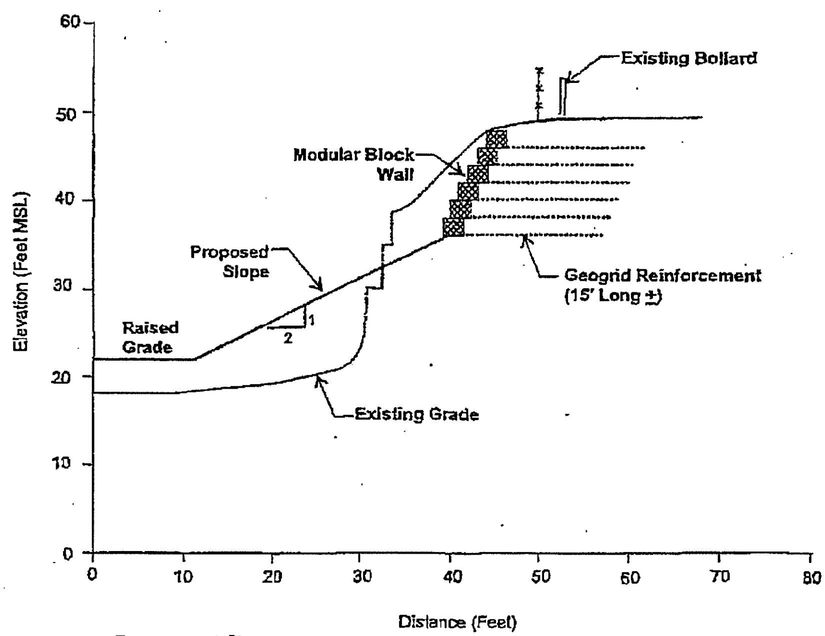
MAY 2000

FIRST PLACE CONCEPTUAL DESIGN

FIGURE 3



Concept A



Concept B

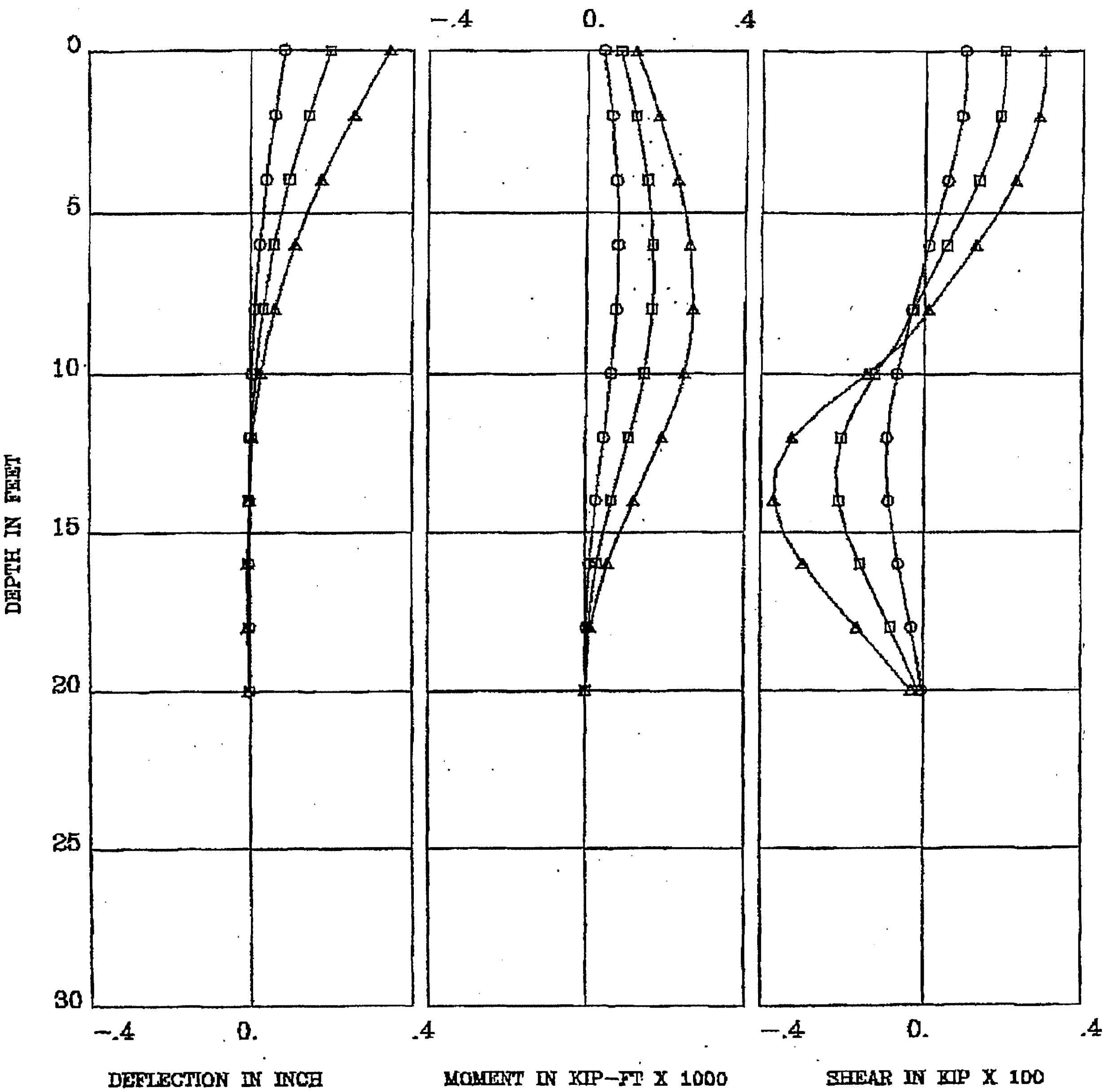
GEOTECHNICAL PROFESSIONALS, INC.

BLUFF RESTORATION

GPI PROJECT NO.: 1652.I | MAY 2000

TWELFTH PLACE
CONCEPTUAL DESIGNS A & B

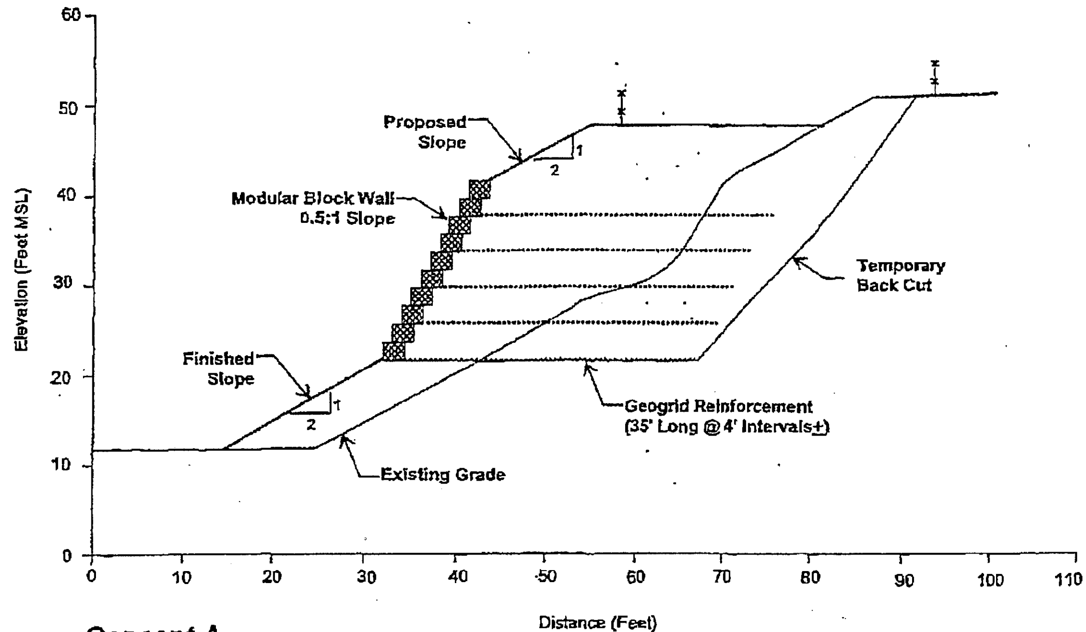
FIGURE 4



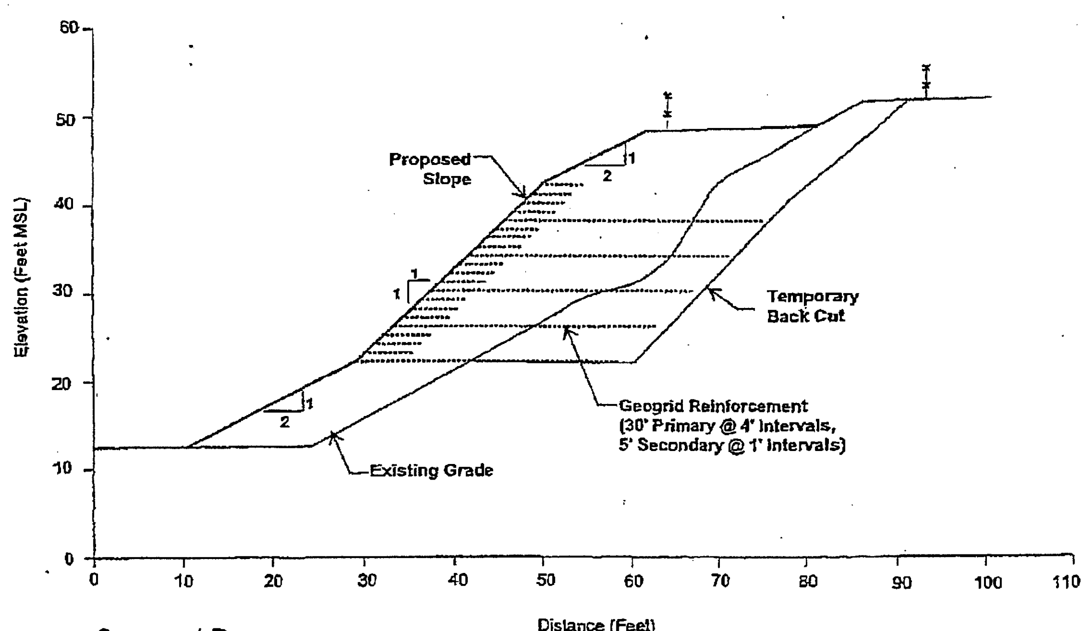
CASE	SYMBOL	DESCRIPTION
1	○	12TH STREET, 30 IN. DIA. CONC. DRILLED PILE, 10 KIP LATERAL, FREE HEAD
2	□	12TH STREET, 30 IN. DIA. CONC. DRILLED PILE, 20 KIP LATERAL, FREE HEAD
3	△	12TH STREET, 30 IN. DIA. CONC. DRILLED PILE, 30 KIP LATERAL, FREE HEAD

1652.I	BLUFF RESTORATION
G P I	DEFLECTION, MOMENT, AND SHEAR VS DEPTH

FIGURE 5



Concept A



Concept B

GPI GEOTECHNICAL PROFESSIONALS, INC.

BLUFF RESTORATION

GPI PROJECT NO.: 1652.I | MAY 2000

GLIDERS SLOPE
CONCEPTUAL DESIGNS A & B

FIGURE 6

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

Five Cone Penetration Tests (CPT's) were performed at the site during the investigation. The locations of the CPT's are shown on the Site Plans, Figures 2.1 through 2.6. These soundings were advanced to depths of 40 to 62 feet below existing grades.

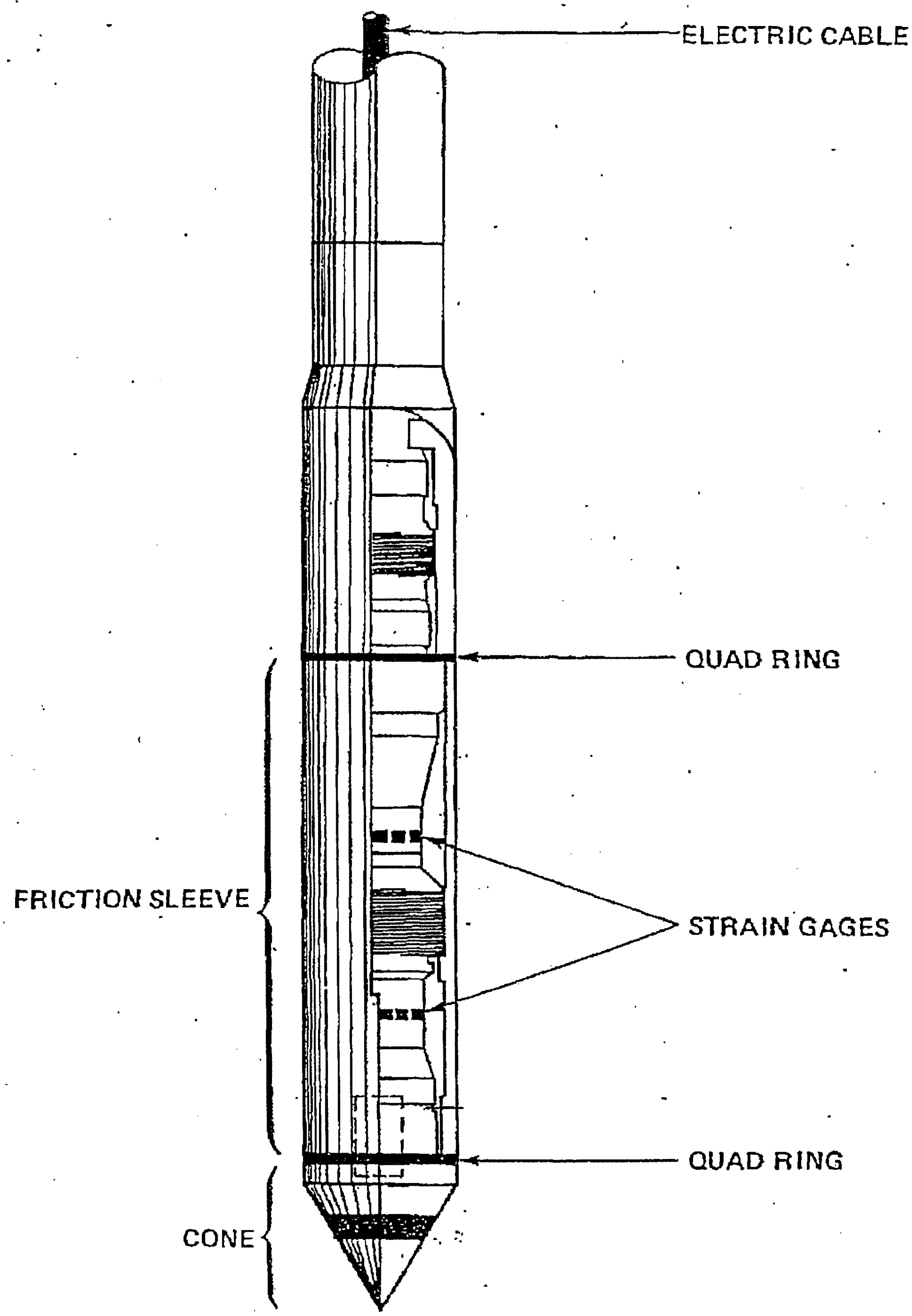
The CPT consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (see Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM specifications (ASTM D3441) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed all-wheel drive truck is used to transport and house the test equipment and to provide a 23-ton reaction to the thrust of the hydraulic rams.

Data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations, which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 through A-6 of this appendix. The field testing and digitizing of the data was performed by Fugro Geoscience, Inc., under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

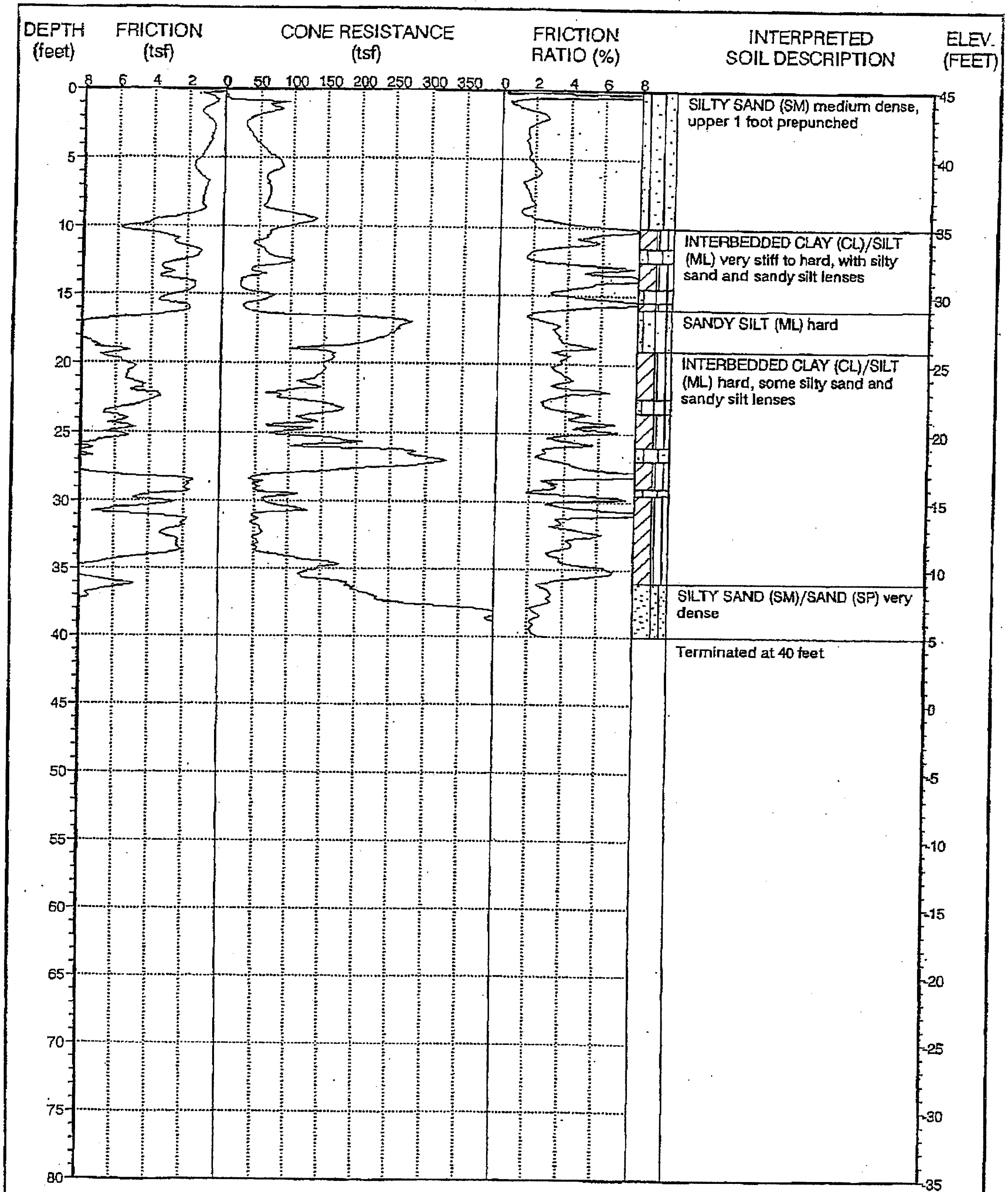
The ground surface elevations shown on the CPT logs were estimated from topographic plans by Tetra Tech, Inc. We understand that the elevations shown on these plans are relative to the Mean Lower Low Water (MLLW) datum.



GPI GEOTECHNICAL PROFESSIONALS INC.

CONE PENETROMETER

FIGURE A-1



Date performed:3-23-00

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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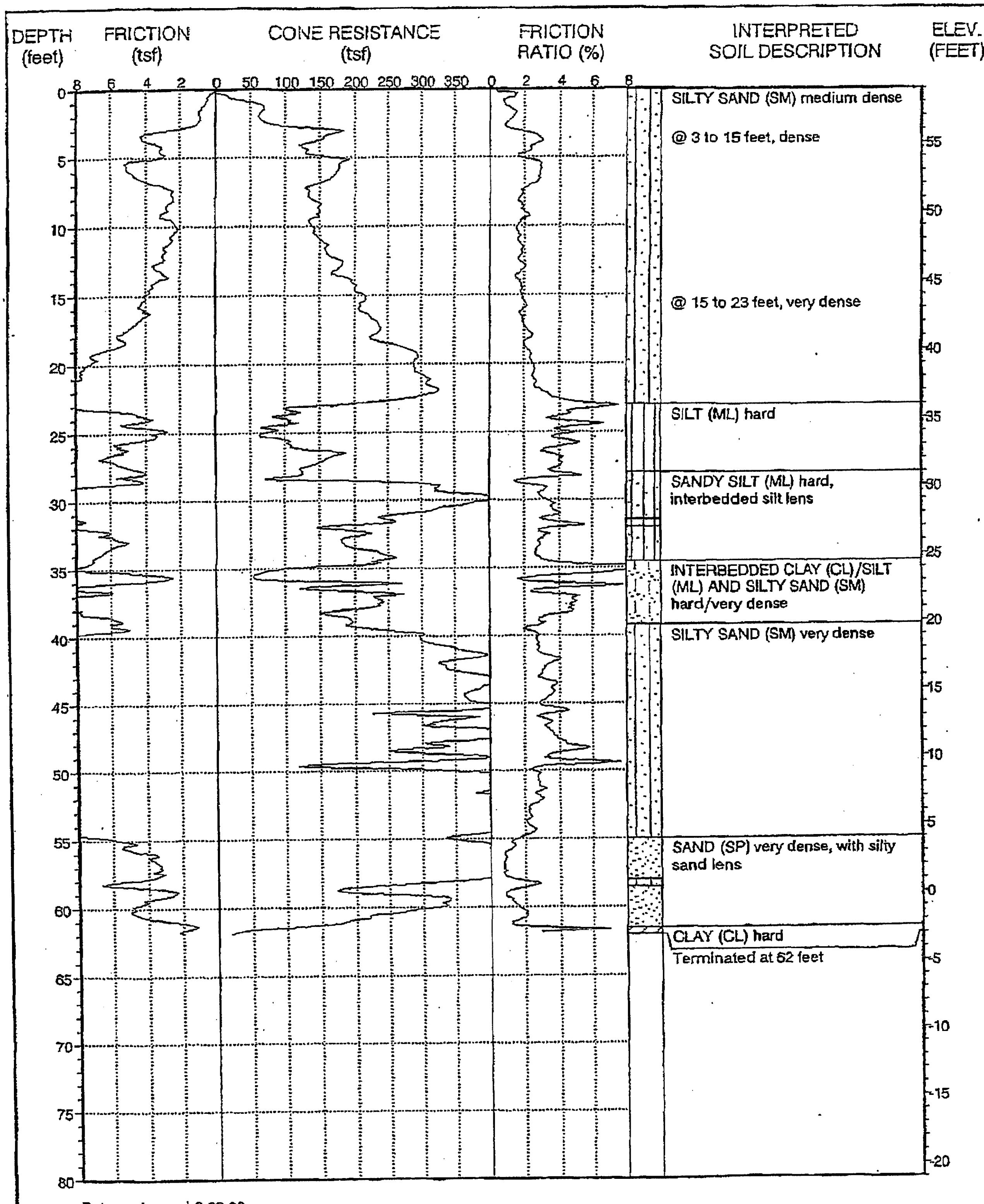
LOG OF CPT NO. C- 1

FIGURE A-2



Date performed:3-23-00

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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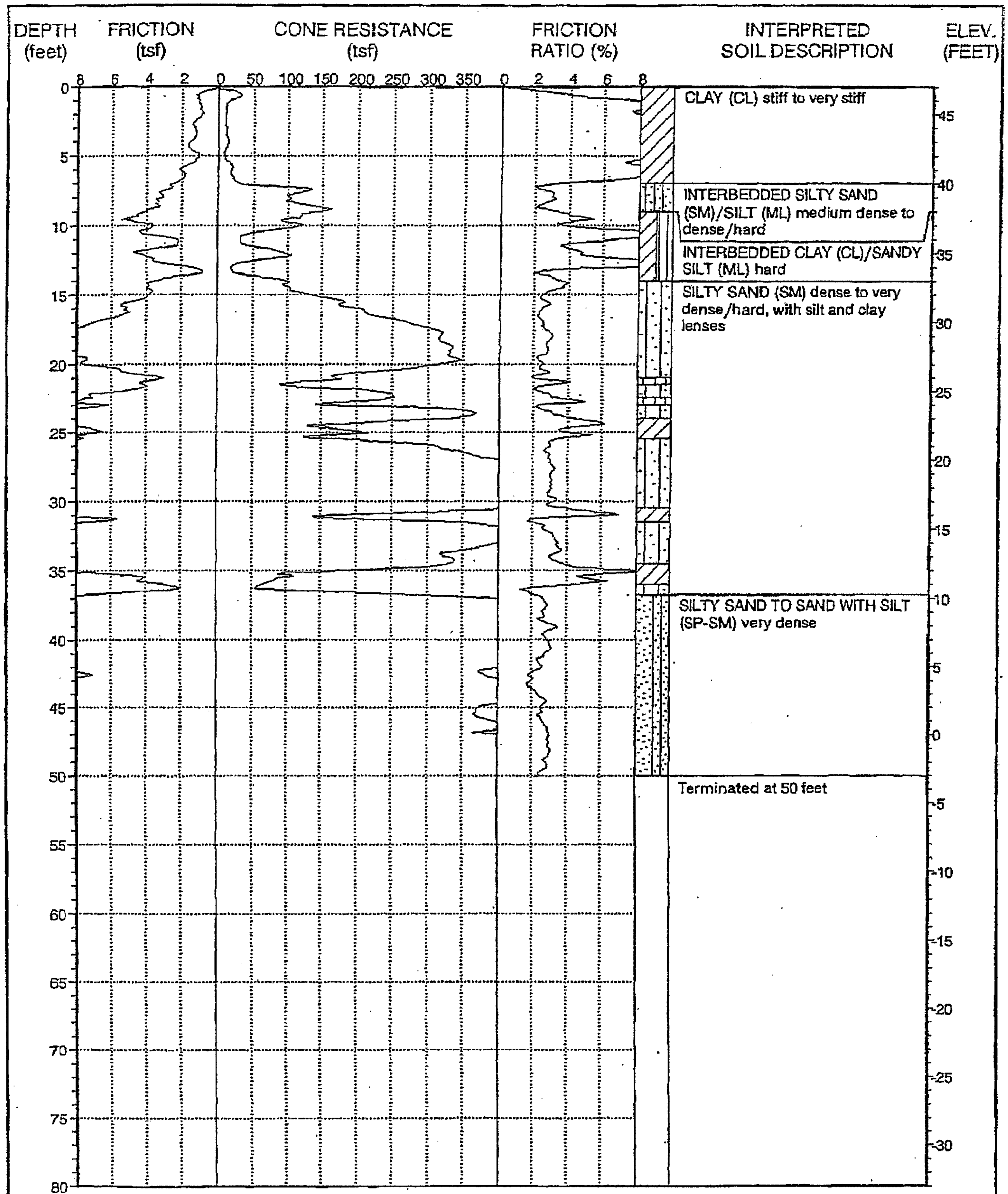
LOG OF CPT NO. C-2

FIGURE A-3



Date performed:3-23-00

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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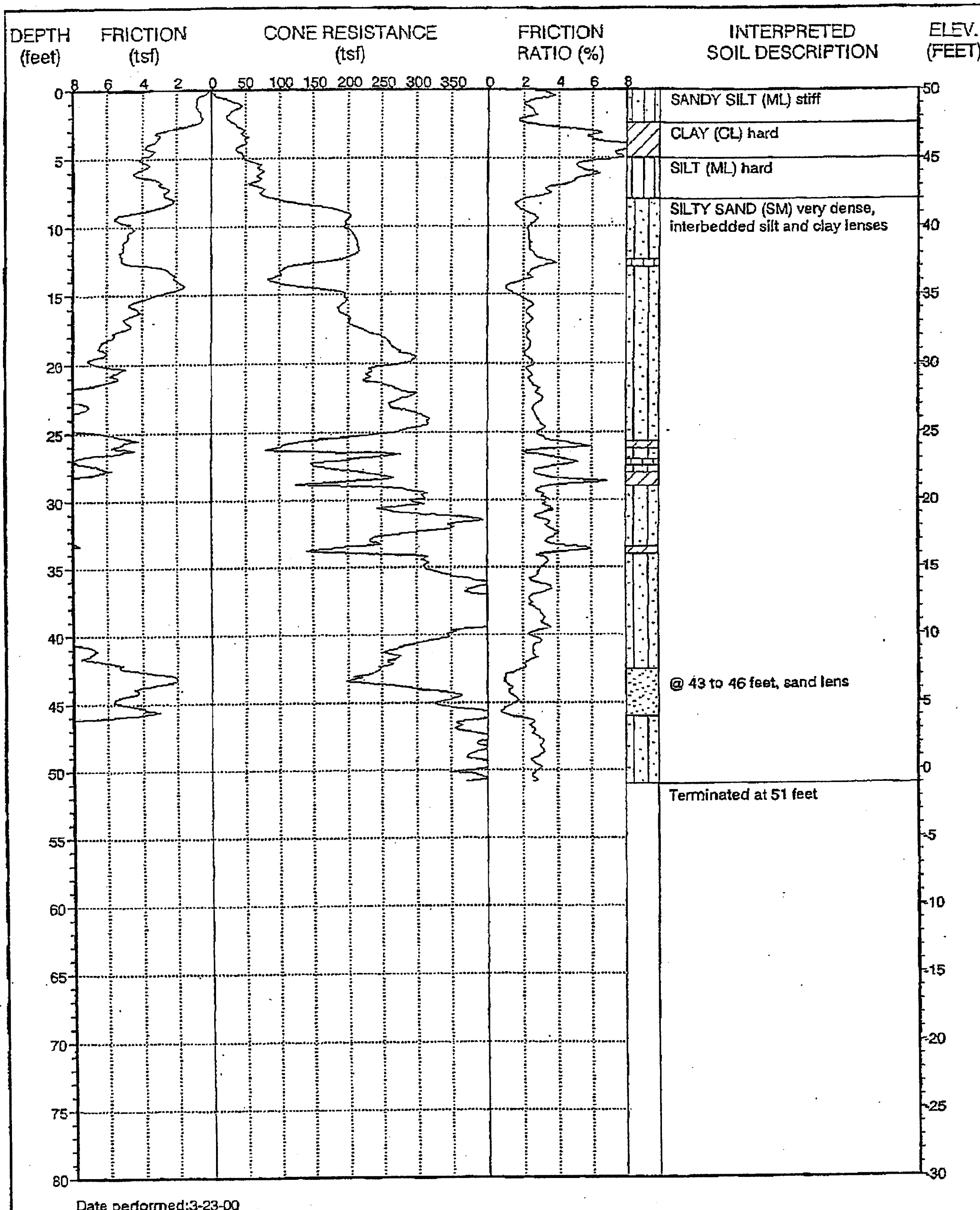
BLUFF RESTORATION

LOG OF CPT NO. C-3

FIGURE A-4



This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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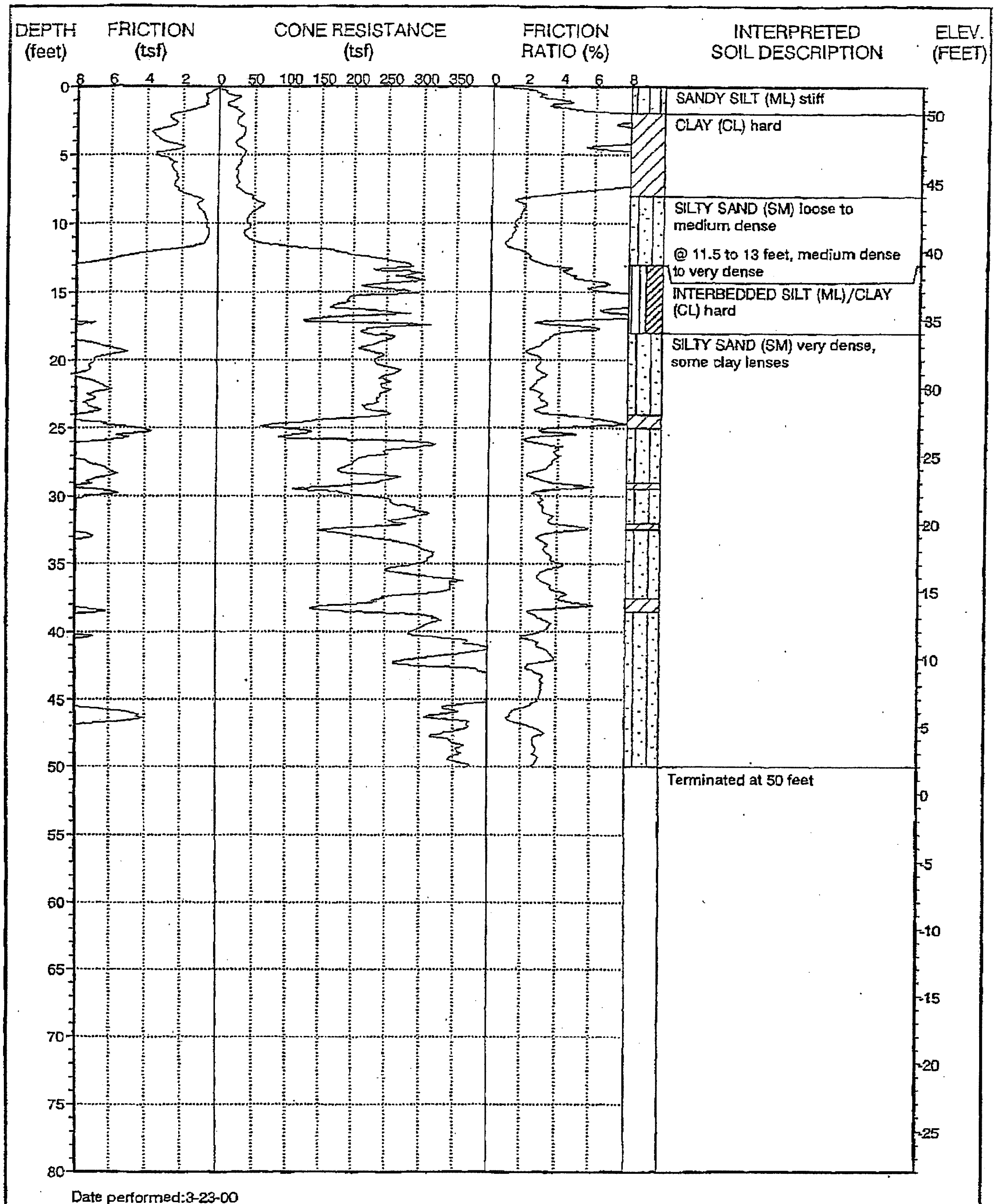
LOG OF CPT NO. C-4

FIGURE A-5



This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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LOG OF CPT NO. C- 5

FIGURE A-6

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling five hollow-stem auger borings. The boring locations are shown on the Site Plan, Figure 2. The borings were advanced to depths ranging from 4 to 55 feet.

The borings were drilled using a truck-mounted hollow-stem auger equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler, with an inside diameter of 2.42 inches, driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive into the soil sampler was recorded as the penetration resistance. Due to the use of a free-fall hammer (rather than a hammer attached to rope), the blow-counts recorded with the drive (D) sampler are approximately equal to the Standard Penetration Test blow-count (N_{60}).

The field explorations were performed under the continuous technical supervision of GPI's senior technician, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures B-1 to B-5 in this appendix.

Upon completion, the boreholes were backfilled with the excavated soil cuttings. In paved areas, the tops of the boreholes were sealed with asphalt cold patch.

The ground surface elevations shown on the boring logs were obtained from topographic plans provided by Tetra Tech, Inc. We understand that the elevations shown on these plans are relative to the Mean Lower Low Water (MLLW) datum.

Moisture (%)	Dry Density (PCF)	Penetration Resistance (Blows/Foot)	Sample Type	Depth (Feet)	Description of Subsurface Materials	Elevation (Feet)
				0	1 inch AC over 2 inches AB	
			B		**SILTY SAND (SM)** brown and dark brown, moist	40
					@ 2 to 9 feet, yellow-red, moist, dense	
7.8	117	49	D			
11.0	113	35	D	5	@ 5 feet, increased fines, trace clay	35
14.1	119	71	D	10	**SILTY SAND (SM)/SANDY SILT (ML)** olive-brown, moist, very dense/hard, partially cemented	30
29.2	99	70	D	15	**SILT WITH SAND (ML)** brown and olive-brown, wet, hard	
					SILTY SAND (SM) brown, moist, very dense	25
3.5	94	36	D	20	**SAND WITH SILT (SP/SW-SM)** light brown, slightly moist, dense	20
28.1	90	57	D	25	**SANDY SILT (ML)** brown, wet, hard	15
20.6	106	35	D		**SILTY SAND (SM)** grey, very moist to wet, medium dense to dense, with clay	
				30	**SANDY CLAY (CL)** grey with yellow-red, very moist, wet, hard, with sand lenses	
24.5	101	30	D		**SANDY SILT (ML)** olive, wet, hard, with frequent sand lenses	10
19.6	108	41	D	35	**SILTY SAND (SM)** brown and yellow-red, very moist, dense, with shells and shell fragments	
					SAND WITH SILT (SP/SW-SM) light brown, wet, very dense	5

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-30-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 33 feet

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PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-1

FIGURE B-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
21.9	100	100/9"	D	40		
					Refusal at 41 feet	0

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-30-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL:
Water at 33 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-1

FIGURE B-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	6 inches AC	
					SILTY SAND (SM) dark brown, moist, dense	
9.4	119	32	D		@ 2 to 11 feet, red-brown	
						45
9.3	114	40	D	5		
12.2	115	31	D		@ 8 feet, trace clay	40
				10		
13.2	103	17	D		**SILT WITH SAND (ML)** brown and olive-brown, moist, very stiff	
			B			
						35
28.2	90	66	D	15	@ 15 to 20 feet, hard	
						30
10.2	99	57	D	20	**SILTY SAND (SM)** brown, moist, very dense	
6.5	97	52	D			
				25	@ 23 to 29 feet, brown and light brown, with decreased fines	25
						20
					SAND (SP) light brown, dry, very dense	
2.0	98	90/11"	D	30		
						15
					INTERBEDDED SAND (SP/SW)/SILT (ML) light brown to brown, moist, very dense/hard, with shell fragments, trace fine gravel	
10.5	117	60	D	35		
					SILTY SAND (SM) yellow brown, moist, very dense, with shell fragments	
9.2	102	82	D			10

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-30-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 45 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-2

FIGURE B-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					40		
							5
	22.4	102	70	D	45	@ 45 to 50 feet, wet	
	25.8	99	95	D			0
					50	Total Depth 50 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-30-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL:
Water at 45 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-2

FIGURE B-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	**SILTY SAND (SM)/SANDY SILT (ML)** dark brown to black, moist, loose to medium dense/stiff	50
					SILTY CLAY (CL) dark brown and black, very moist, hard	
18.2	113	23	D			
			B			
				5		45
24.4	94	16	D		@ 6 to 10 1/2 feet, brown	
			B			
				10		
22.8	107	46	D		@ 10 1/2 to 12 feet, brown with yellow-red, with sand	40
12.9	109	40	D		**SILTY SAND (SM)** brown with yellow-red, moist, dense	
			B			
				15		35
15.8	110	36	D		**SILTY SAND (SM)/SANDY SILT (ML)** brown, very moist, dense/hard	
				20	**SILTY SAND (SM)** brown and light brown, slightly moist, very dense, low fines content	30
3.2	102	57	D			
29.8	92	31	D			
				25	**SILT WITH SAND (ML)** brown and yellow-red, wet, very stiff	25
					SILTY SAND (SM) brown and yellow-brown, moist, very dense	
				30		20
13.3	100	71	D			
				35		
7.9	96	80	D		**SAND (SP/SW)** light brown, moist, very dense	15

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-30-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 43 feet

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PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-3

FIGURE B-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					40		
	19.9	105	83/11"	D		**SILTY SAND (SM)** brown and yellow-red, wet, very dense, with occasional silt lenses.	10
			98/11"	D	45	**SAND WITH SILT (SP/SW-SM)** light brown, wet, very dense	5
			80/9"	D	50		0
			68/10"	D	55	Total Depth 55 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-30-00

EQUIPMENT USED: 8" Hollow Stem Auger

GROUNDWATER LEVEL: Water at 43 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-3

FIGURE B-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	**SILTY SAND (SM)/SANDY SILT (ML)** dark brown, moist, medium dense/stiff	
					SILTY CLAY (CL) dark brown, moist, hard	45
16.2	104	30	D			
				5	@ 5 to 8 feet, olive-brown, very moist, trace roots	
19.2	99	48	D			
					SILTY SAND (SM)/SANDY SILT (ML) brown, moist, medium dense to dense/stiff to very stiff	40
12.1	110	33	D	10	**SILTY CLAY (CL)** brown, moist, hard, with silt nodules	
						35
15.7	108	53	D	15	**SILTY SAND (SM)/SANDY SILT (ML)** brown, very moist, dense/hard	
					SILTY SAND (SM) brown, moist, dense	30
2.5	96	64	D	20	**SAND WITH SILT (SP-SM)** light brown, dry, very dense	
						25
				25		
3.7	96	81	D			20
3.2	97	83	D	30		
						15
16.6	100	59	D	35	**SILTY SAND (SM)** light brown and brown, very moist to wet, very dense	
						10

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-31-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 40 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-4

FIGURE B-4

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	26.9	97	58	D	40		
						SAND WITH SILT (SP/SW-SM) light brown, wet, very dense, with shell fragments	5
	24.4	99	69	D	45		
							0
	23.8	101	78/11"	D	50		
						Total Depth 50 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-31-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL:
Water at 40 feet

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PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-4

FIGURE B-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	**SILTY SAND (SM)** red-brown, moist, medium dense	50
13.5	119	22	D B		**CLAYEY SAND (SC)/SANDY CLAY (CL)** red-brown, moist, medium dense/stiff	
				5	**SILTY SAND (SM)** red-brown, moist, medium dense to dense	45
12.5	108	25	D			
9.2	99	33	D	10		40
					SILT (ML) brown, very moist, hard	
22.9	94	63	D	15		35
					SAND WITH SILT (SP/SW-SM) light brown and yellow brown, slightly moist to moist, dense	
5.6	99	48	D	20		30
					SILTY SAND (SM) light brown and yellow brown, very moist, dense	
16.4	89	31	D	25		25
					SAND WITH SILT (SP/SW-SM) light brown and yellow brown, slightly moist to moist, very dense	
5.6	98	75	D	30		20
4.5	100	85	D			
				35		15
5.0	101	53	D			

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-31-00

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Water at 47 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-5

FIGURE B-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					40		10
	10.7	94	80	D	45		5
						@ 47 to 50 feet, grey, wet	
	28.8	93	87	D	50		
						Total Depth 50 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 3-31-00

EQUIPMENT USED: 8" Hollow Stem Auger

GROUNDWATER LEVEL: Water at 47 feet

GPI

PROJECT NO.: 1652.I
BLUFF RESTORATION

LOG OF BORING NO. B-5

FIGURE B-5

APPENDIX C

APPENDIX C

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM 2216-71. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs, in Appendix B.

PASSING NO. 200 SIEVE

Two soil samples were dried, weighed and soaked in water until individual soil particles were separated and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The results are tabulated below:

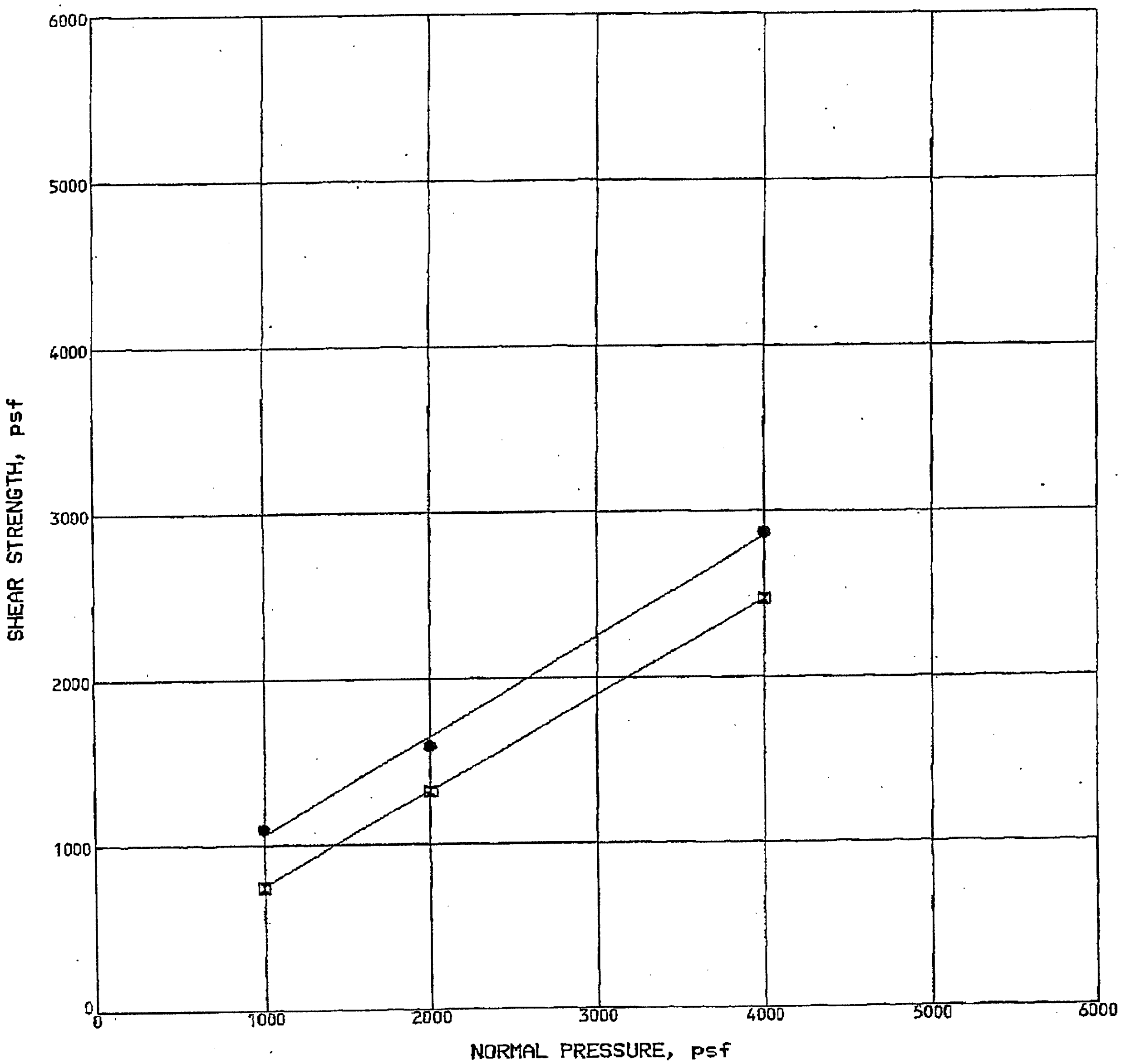
BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	% FINER THAN NO. 200
B-3	21	Silty Sand (SM)	16
B-5	15	Silt (ML)	91

DIRECT SHEAR

Direct shear tests were performed on selected undisturbed samples in accordance with ASTM D 3080. The samples were placed in the shear machine, and pre-selected normal loads were applied. Each sample was submerged, allowed to consolidate, and then was sheared to failure. The procedure was repeated on additional test specimens from the same soil layer under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-1 to C-5.

GRAIN SIZE DISTRIBUTION

Two representative samples of sand were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The portion of the material retained on the No. 200 sieve was oven-dried and then run through a standard set of sieves in accordance with ASTM D 422. The grain size distribution data are shown in Figure C-6.



PEAK STRENGTH
Friction Angle = 31 degrees
Cohesion = 456 psf

RESIDUAL STRENGTH
Friction Angle = 30 degrees
Cohesion = 168 psf

Note:

	Sample Location		Classification		DD,pcf	MC,%
●	B-1	28.0	SANDY CLAY (CL)	Peak	106	21
⊠	B-1	28.0	SANDY CLAY (CL)	Residual	106	21

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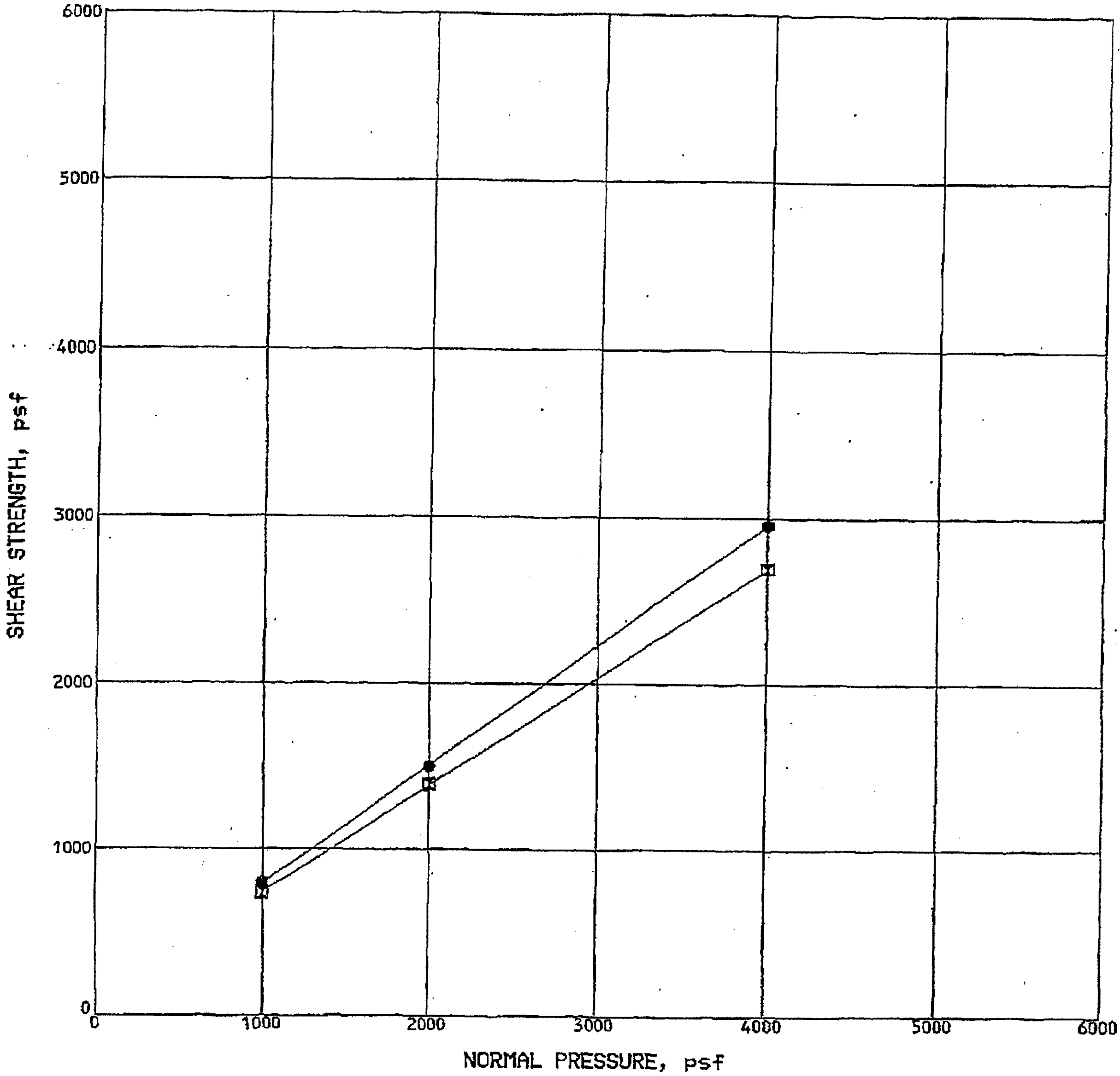
DIRECT SHEAR TEST RESULTS

FIGURE C-1



PEAK STRENGTH
Friction Angle= 36 degrees
Cohesion= 66 psf

RESIDUAL STRENGTH
Friction Angle= 33 degrees
Cohesion= 84 psf

Note:

	Sample Location		Classification		DD,pcf	MC,%
●	B-2	38.0	SILTY SAND (SM)	Peak	102	9
⊠	B-2	38.0	SILTY SAND (SM)	Residual	102	9

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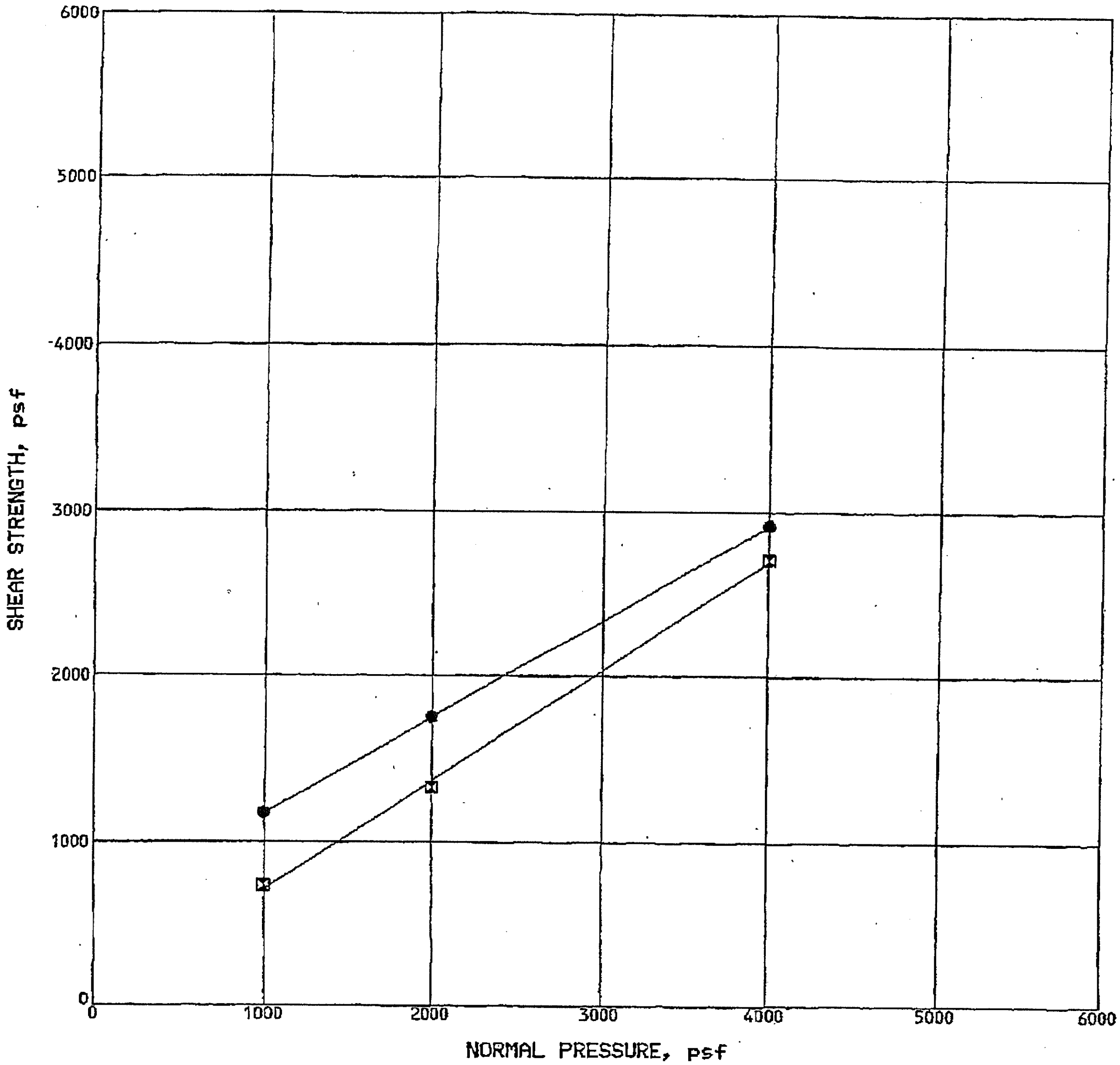
DIRECT SHEAR TEST RESULTS

FIGURE C-2



PEAK STRENGTH
Friction Angle = 30 degrees
Cohesion = 594 psf

RESIDUAL STRENGTH
Friction Angle = 34 degrees
Cohesion = 42 psf

Note:

	Sample Location		Classification		DD,pcf	MC,%
●	B-3	31.0	SILTY SAND (SM)	Peak	100	13
⊠	B-3	31.0	SILTY SAND (SM)	Residual	100	13

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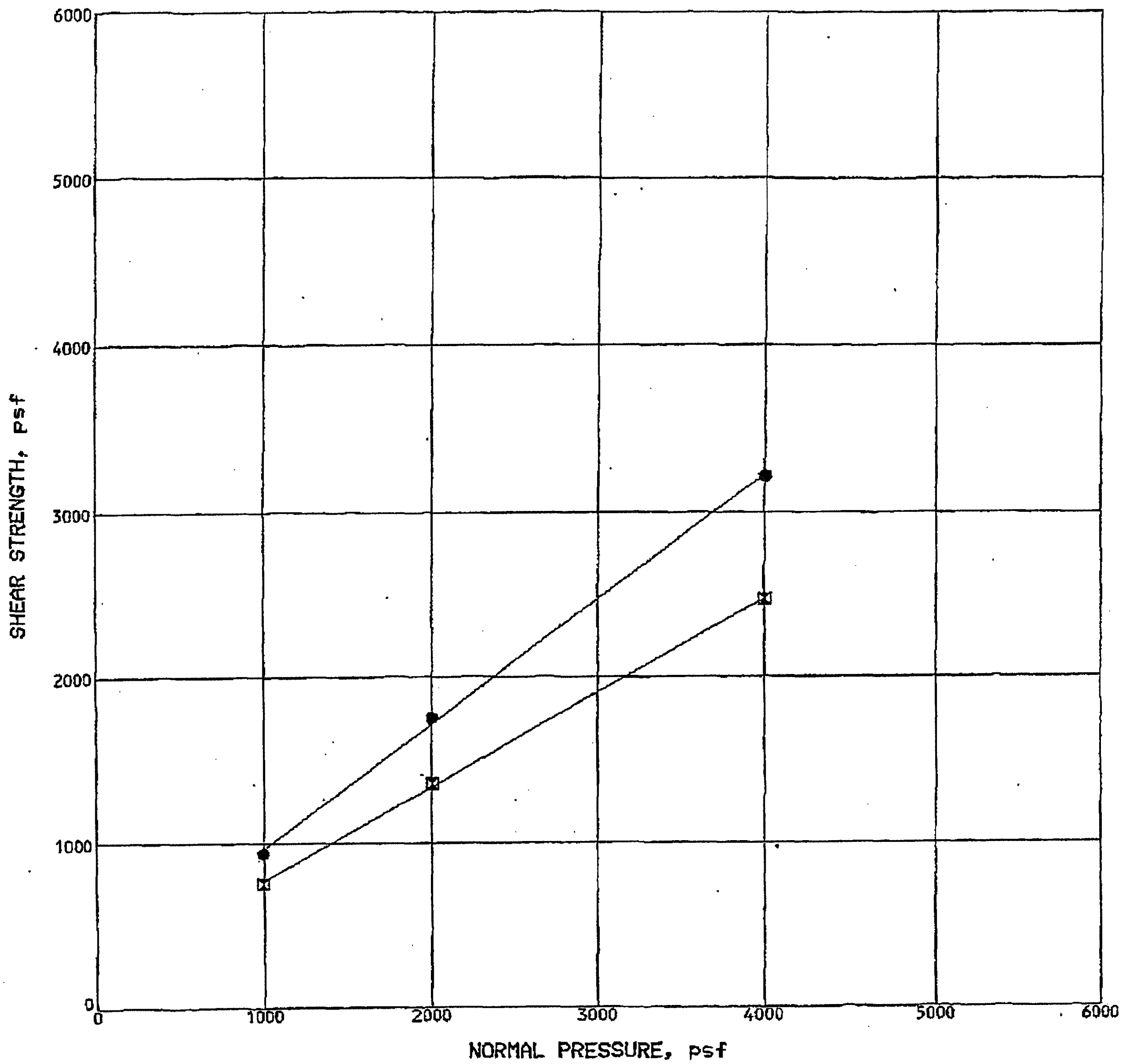
DIRECT SHEAR TEST RESULTS

FIGURE C-3



PEAK STRENGTH
Friction Angle= 37 degrees
Cohesion= 204 psf

RESIDUAL STRENGTH
Friction Angle= 30 degrees
Cohesion= 198 psf

Note:

	Sample Location		Classification		DD,pcf	MC,%
●	B-4	20.0	SAND WITH SILT (SP-SM)	Peak	96	3
⊠	B-4	20.0	SAND WITH SILT (SP-SM)	Residual	96	3

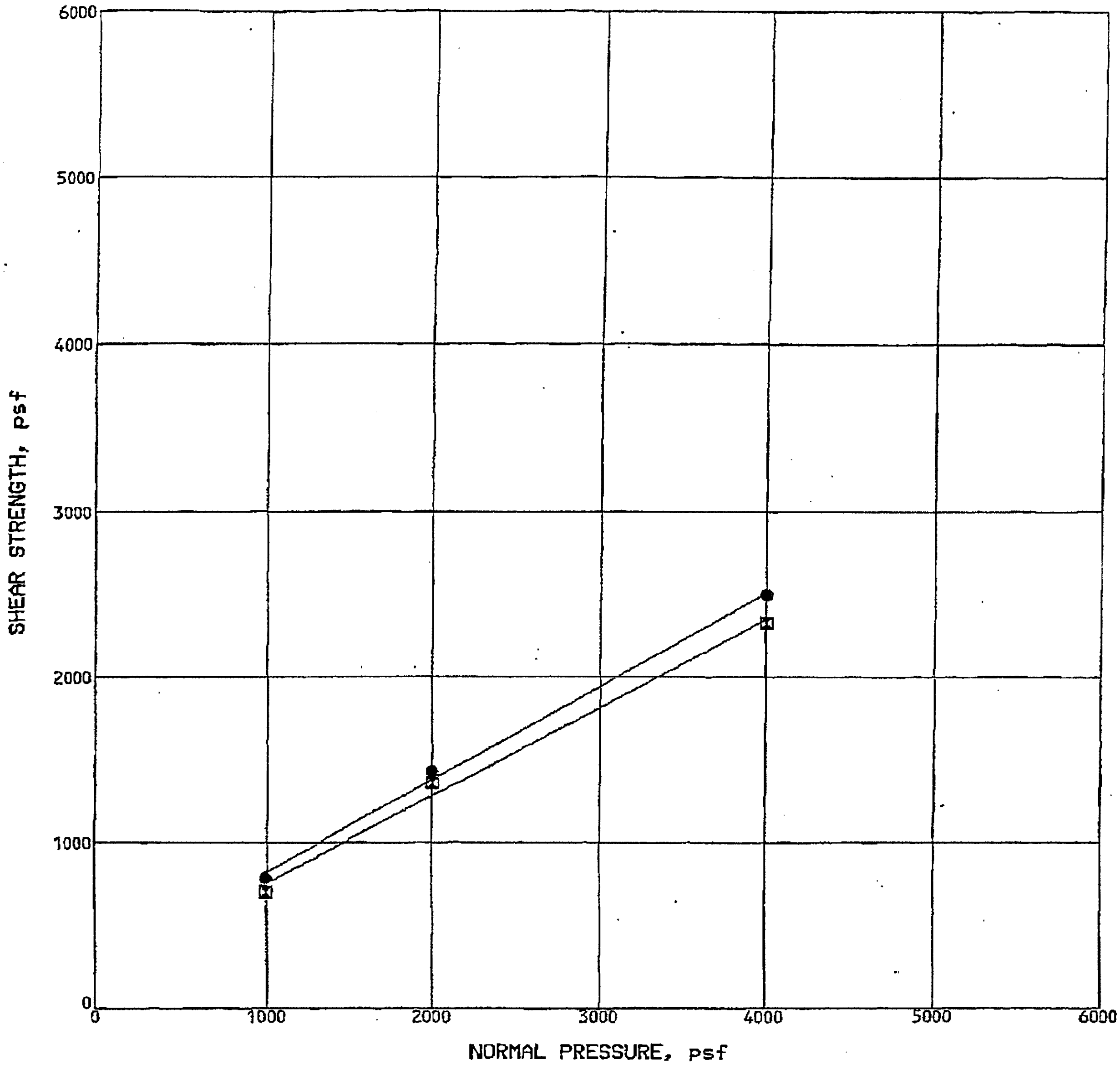
PROJECT: BLUFF RESTORATION

PROJECT NO.: 1652.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-4



PEAK STRENGTH
Friction Angle= 30 degrees
Cohesion= 246 psf

RESIDUAL STRENGTH
Friction Angle= 28 degrees
Cohesion= 210 psf

Note:

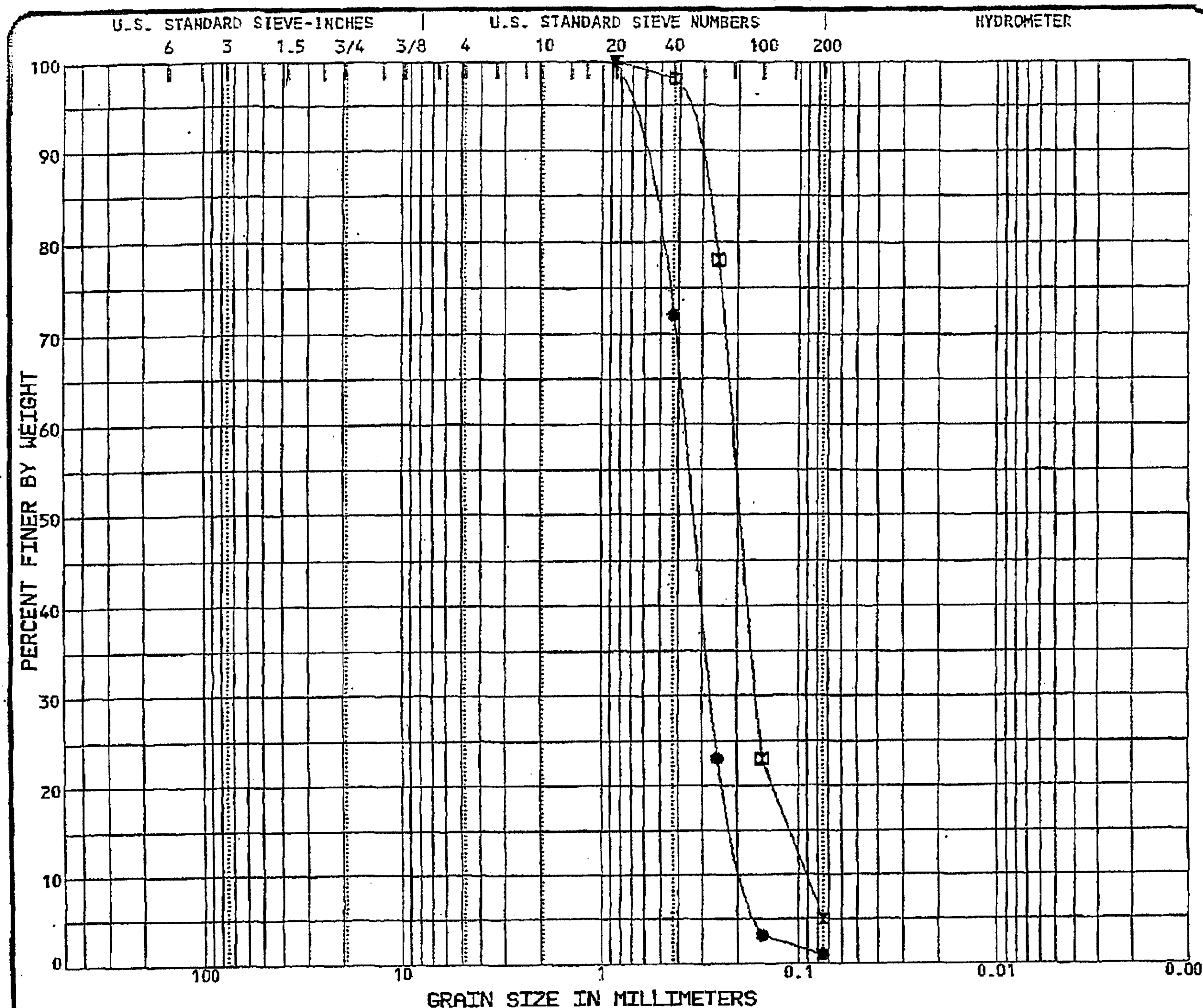
	Sample Location		Classification		DD,pcf	MC,%
●	B-5	24.0	SILTY SAND (SM)	Peak	89	16
⊠	B-5	24.0	SILTY SAND (SM)	Residual	89	16

PROJECT: BLUFF RESTORATION PROJECT NO.: 1652.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-5



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-2	30.0	SAND (SP)	2				1.09	2.1
⌧	B-4	20.0	SAND WITH SILT (SP-SM)	2				1.33	2.3

	Sample Location		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2	30.0	0.85	0.37	0.270	0.1788	0.0	99.0	1.0	
⌧	B-4	20.0	0.85	0.21	0.160	0.0909	0.0	95.0	5.0	

PROJECT: BLUFF RESTORATION PROJECT NO. 1652.I

GPI

GRAIN SIZE DISTRIBUTION

FIGURE C-6

APPENDIX D

SLOPE STABILITY ANALYSIS

We performed stability analyses for various slope configurations. We evaluated the gross stability of the slope configurations using Bishop's Method of analyses and the computer program STABL5M, and STABL6H, developed by Purdue University. The analyses were performed for the following cases:

- First Place
- Twelfth Place
- Highest Slope
- Gabion Slopes
- Gliders Slope

In some cases, we evaluated both existing and proposed slope configurations at the same location. Several analyses also include a geogrid reinforced slope or a keystone wall.

The soil strengths used for our analyses were determined by performing direct shear testing on undisturbed samples. Based on the results of the tests, we determined that the soils, along the bluff, exhibit fairly consistent residual shear strength characteristics. The following densities and residual strengths were used in our analyses:

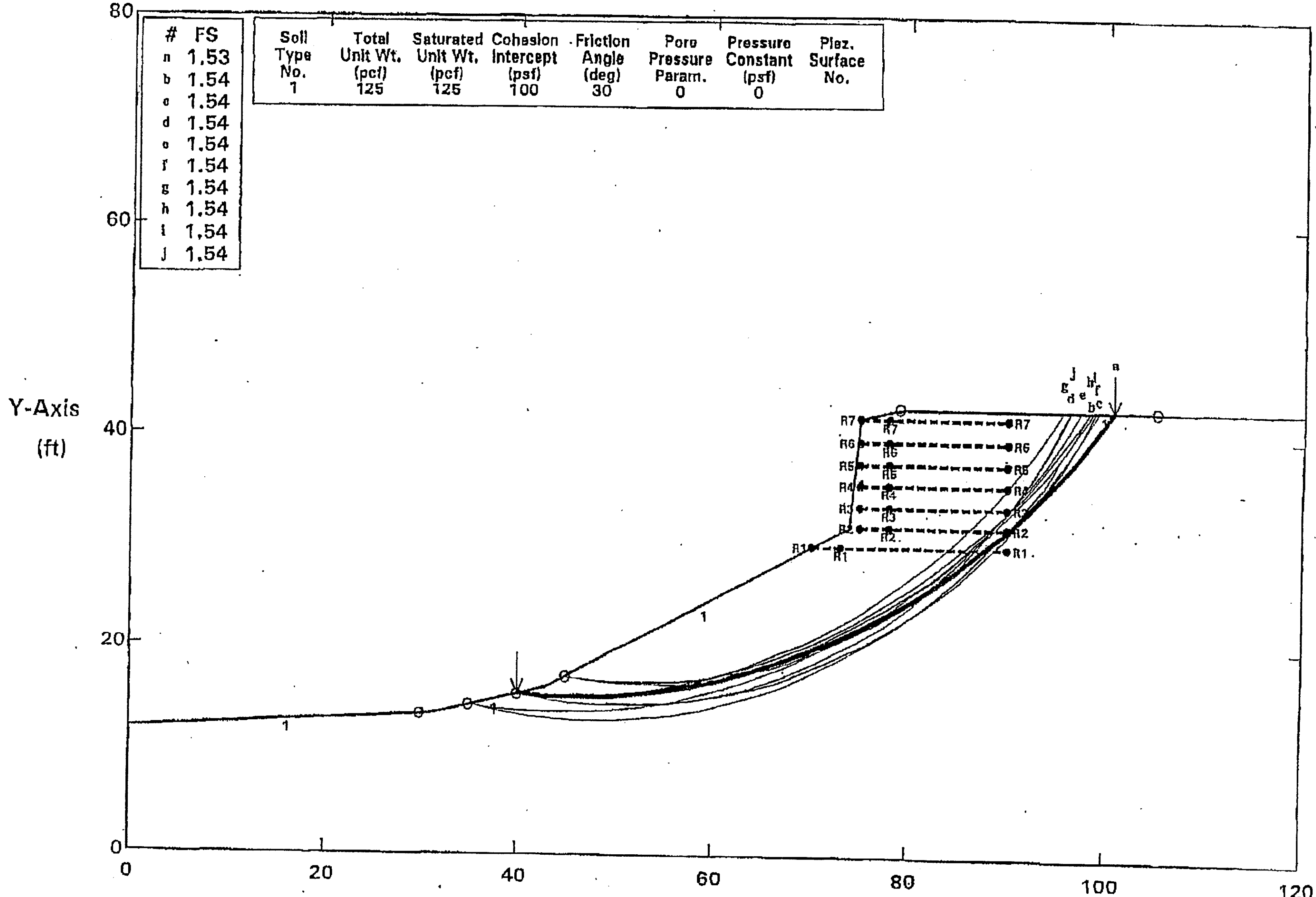
MATERIAL TYPE	DENSITY (pcf)	COHESION (psf)	FRICTION ANGLE (°)
Soil	120	100	30
Gabion	130	500	42

Recommendations for various slope configurations are included within the text of this report. The results of our analyses are summarized below. Details of the analyses, including plots of the critical failure surfaces/slope configurations, are presented in this appendix.

CASE NAME/LOCATION	MINIMUM COMPUTED FACTOR OF SAFETY		COMMENTS
	STATIC	PSEUDO-STATIC	
B1GEOS./1st PLACE and BIGEOP.PLT	1.53	1.13	Geogrid/keystone wall configuration
B1BKCUTS:PLT/1st PLACE	1.44	NA	Temporary back cut
B-221PLT & B-221PLP.PLT/12th PLACE	1.54	1.19	Drilled pile option with 2:1 slope in front
B3-15S.PLT & B3-15-.PLT	1.20	091	1.5:1 slope in area of highest slope
B3-21S.PLT & B3-21P.PLT	1.54	1.09	2:1 slope in area of highest slope
C3EXISTS.PLT & C3EXISTP.PLT	1.02	0.79	Gabion section near CPT C-3
GABIONS.PLT & GABIONP.PLT	1.33	1.00	Gabion section
C5-11S.PLT	0.92	NA	Gliders slope, 1:1 section
C5HALF1S.PLT & C5HALF1P.PLT	1.51	1.11	Gliders slope ½:1 section with geogrid
C511GEOS.PLT & C511GEOP.PLT	1.47	1.07	Gliders, 1:1 section with geogrid

B-1, 1ST.PLACE, 2:1, GEOGRID/KEYSTONE STATIC

Ten Most Critical. C:B1GEOS.PLT By: CM 5/26/2000 5:01pm

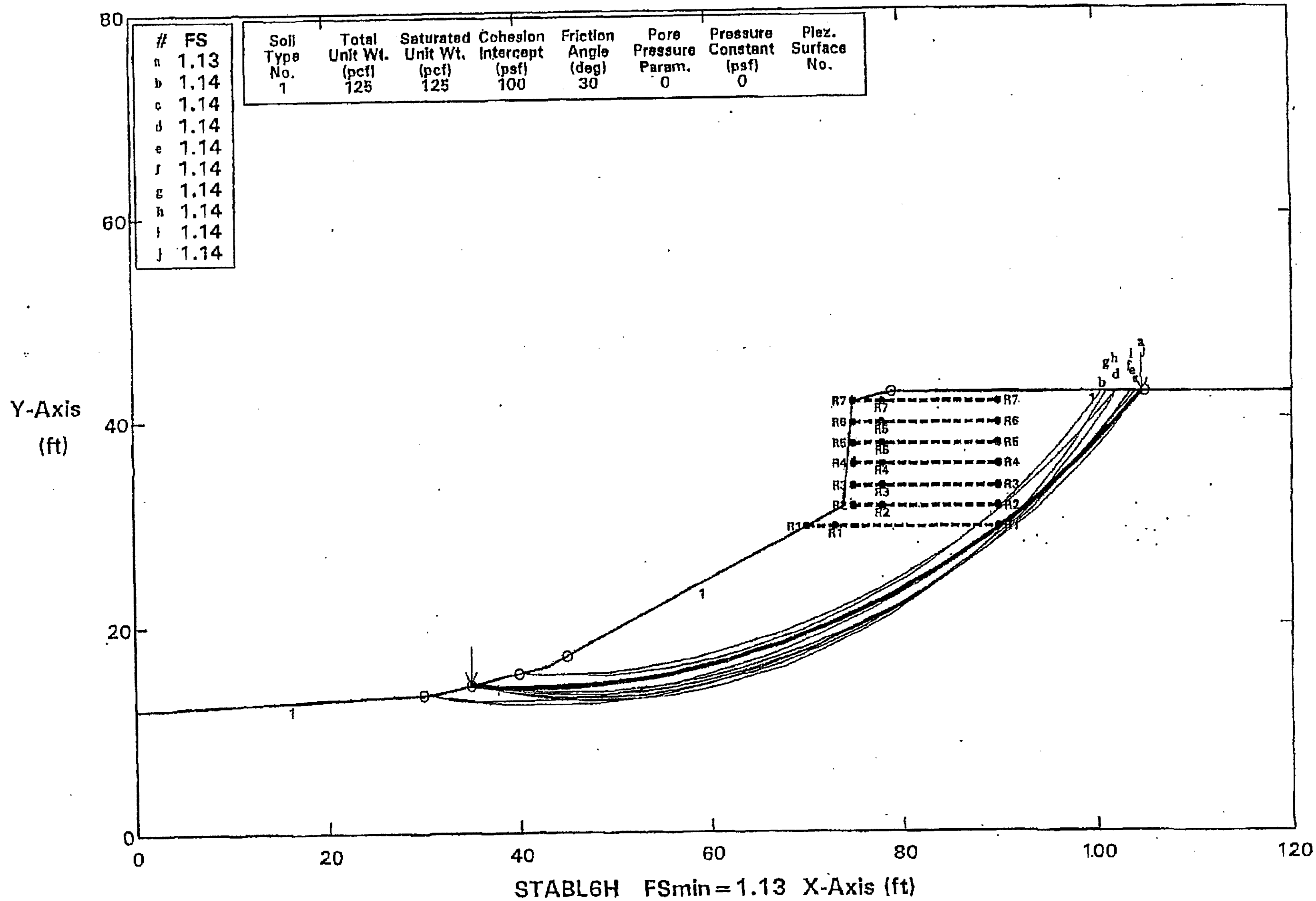


STABL6H FSmin = 1.53 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

B-1, 1ST.PLACE, 2:1, GEOGRID/KEYSTONE PSEUDOSTATIC

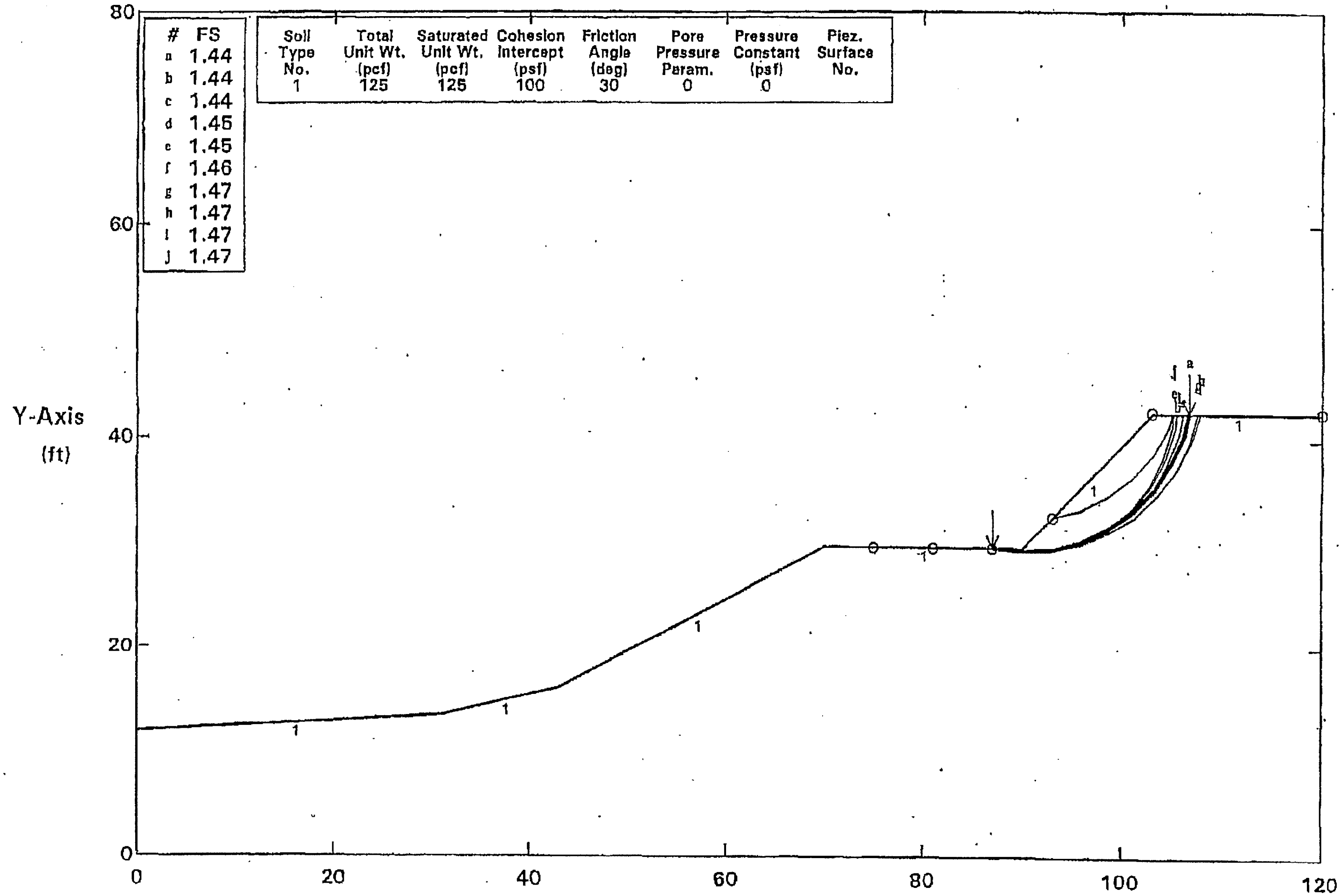
Ten Most Critical. C:B1GEOP.PLT By: CM 5/26/2000 5:04pm



Factors Of Safety Calculated By The Modified Bishop Method

B-1, 1ST.PLACE, TEMP. BACKCUT, GEOGRID/KEYSTONE, STATIC

Ten Most Critical. C:B1BKCUTS.PLT By: CM 5/23/2000 8:52am

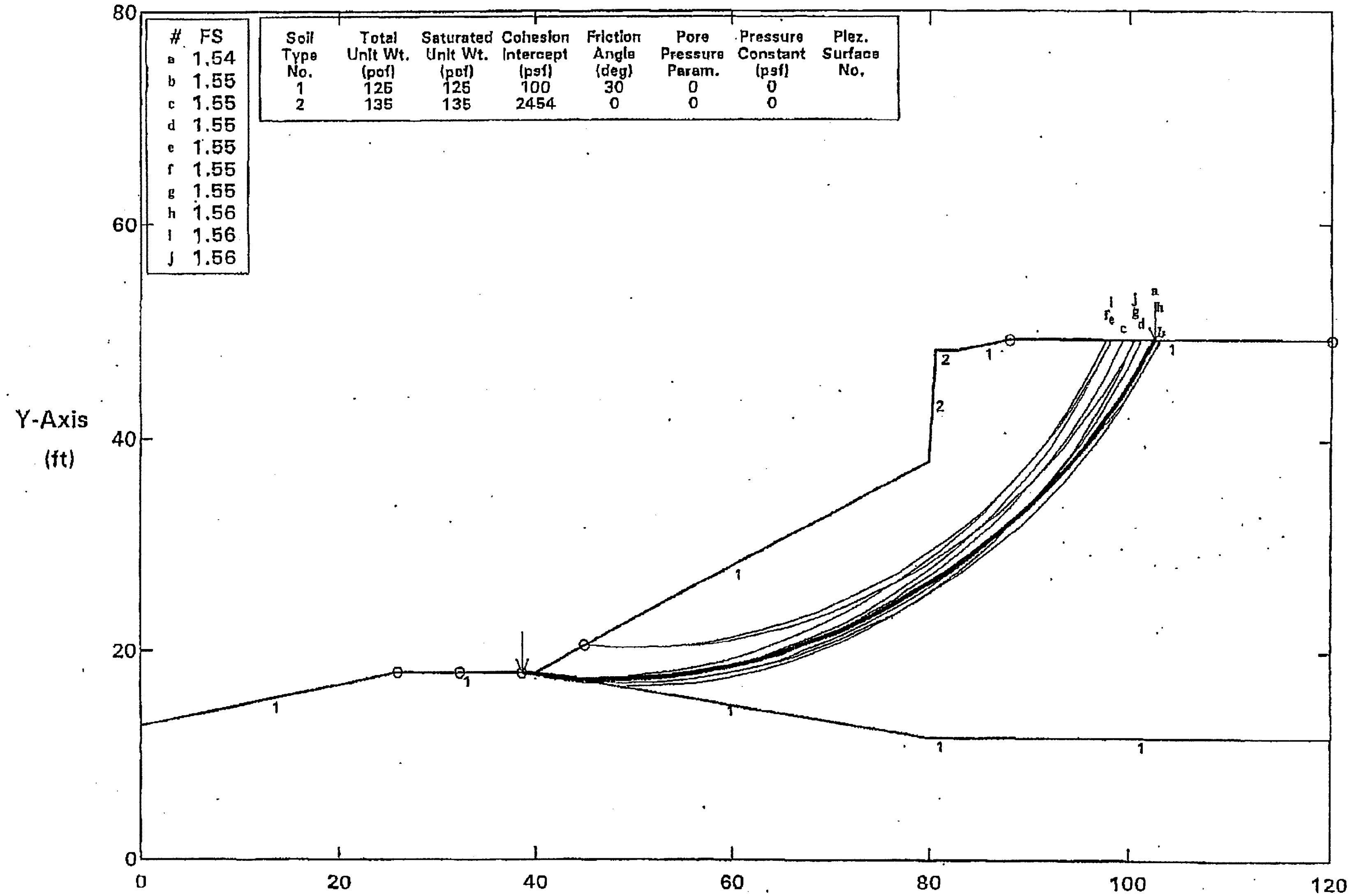


PCSTABL5M/SI FSmin = 1.44 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

B-2, 30' HIGH, 2:1 SLOPE WITH PILE,STATIC

Ten Most Critical. C:B2-21PIL.PLT By: CM 5/11/2000 2:18pm



PCSTABL5M/SI FSmin = 1.54 X-Axis (ft)

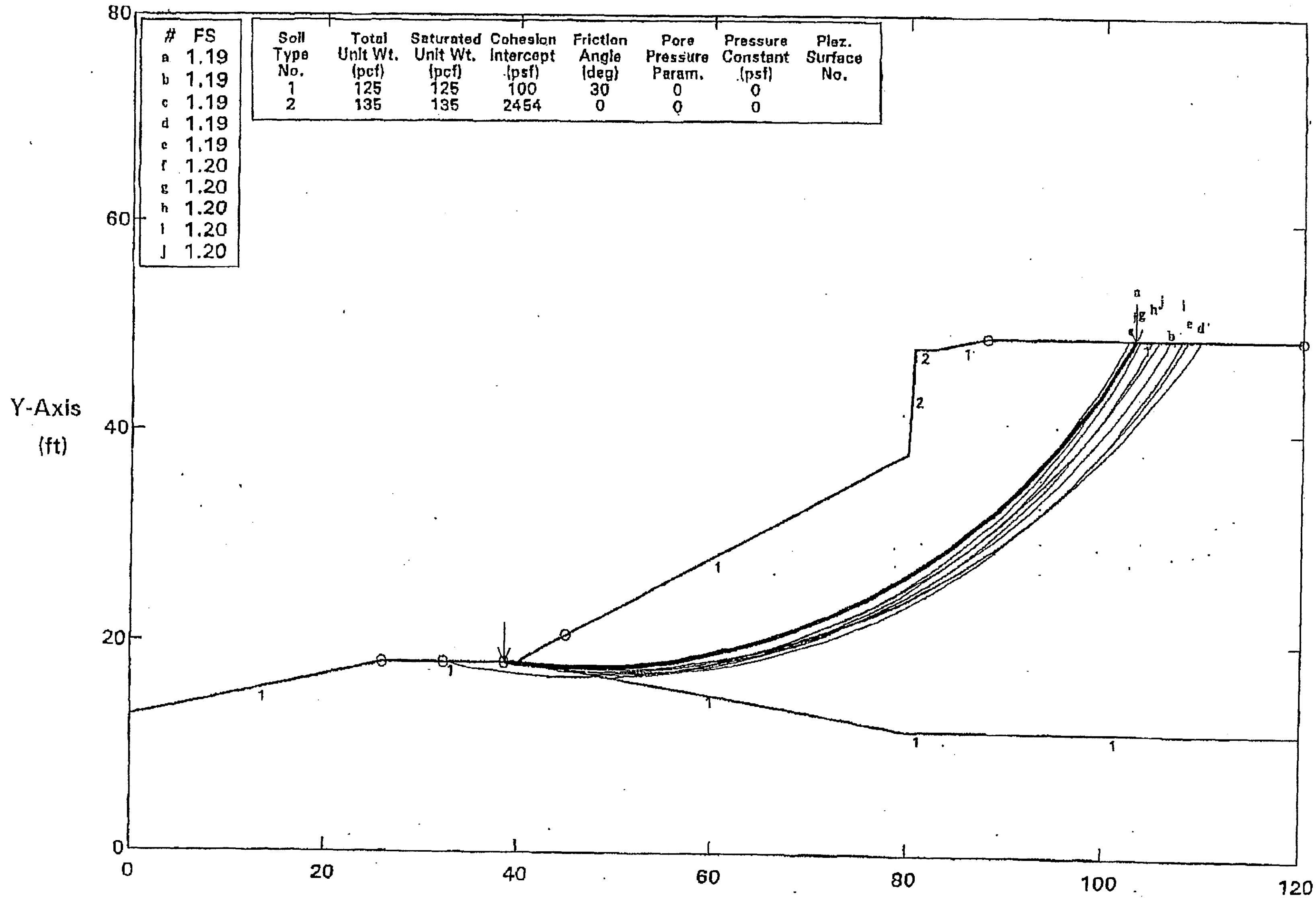
Factors Of Safety Calculated By The Modified Bishop Method

B-2, 30' HIGH, 2:1 SLOPE WITH PILE, PSEUDOSTATIC

Ten Most Critical. C:B2-21PLP.PLT By: CM 5/12/2000 8:13am

#	FS
a	1.19
b	1.19
c	1.19
d	1.19
e	1.19
f	1.20
g	1.20
h	1.20
i	1.20
j	1.20

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	125	125	100	30	0	0	
2	135	135	2454	0	0	0	

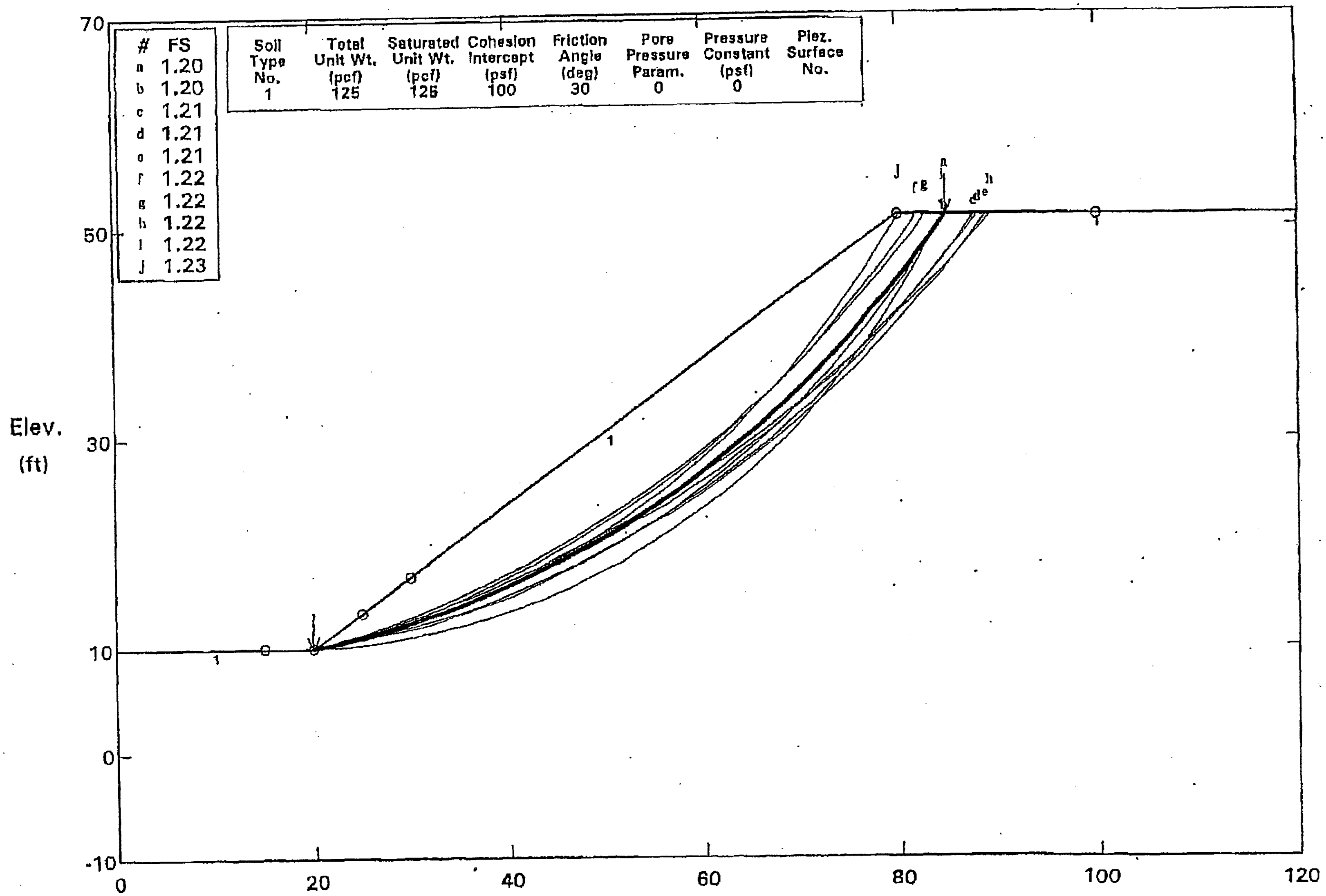


PCSTABL5M/SI FSmin=1.19 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

B-3, 41' HIGH, 1.5:1, STATIC

Ten Most Critical. C:B3-15S.PLT By: CM 5/09/2000 8:31am

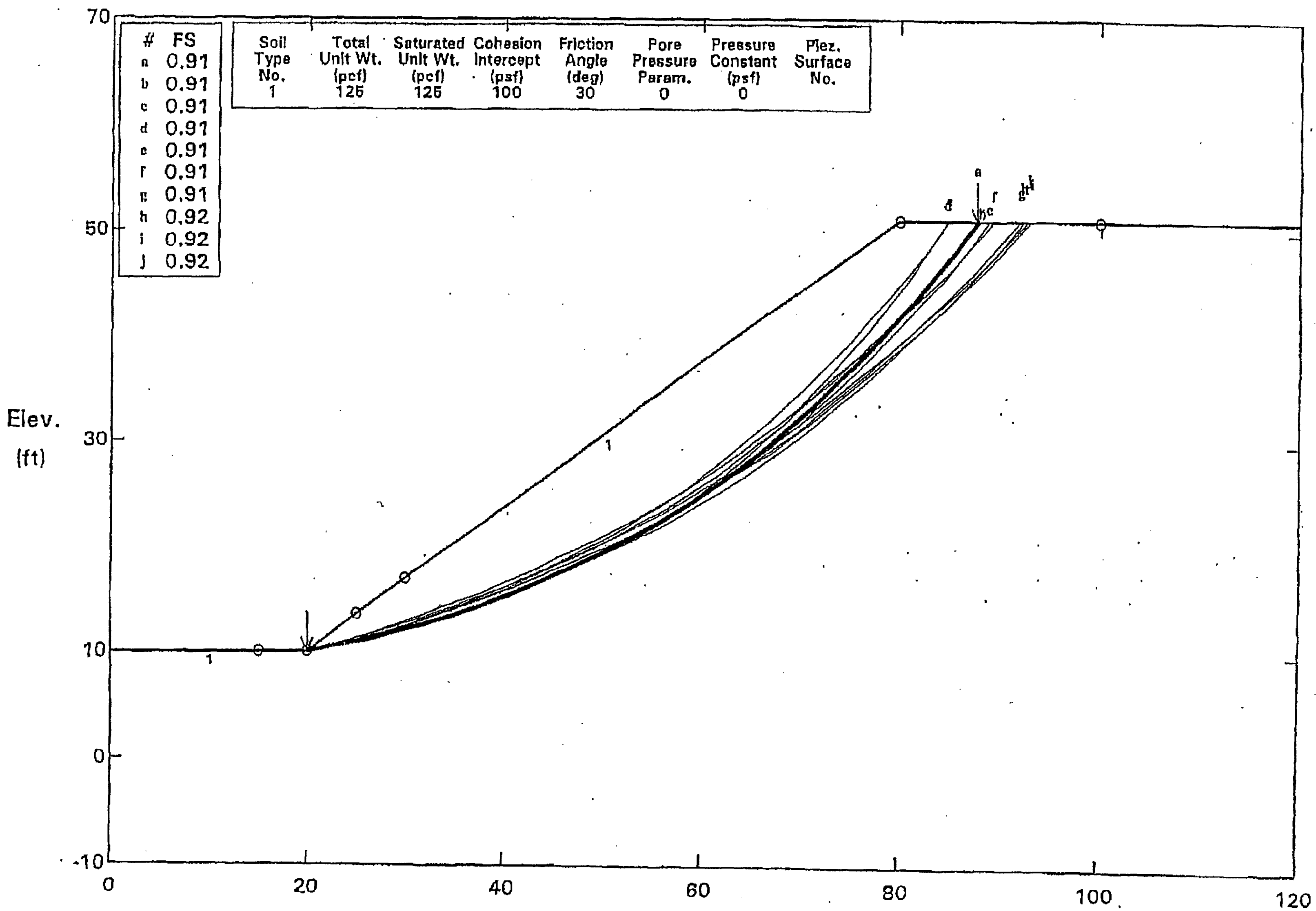


PCSTABL5M/SI FSmin = 1.20 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

B-3, 41' HIGH, 1.5:1, PSEUDOSTATIC

Ten Most Critical. C:B3-15P.PLT By: CM 5/09/2000 8:34am

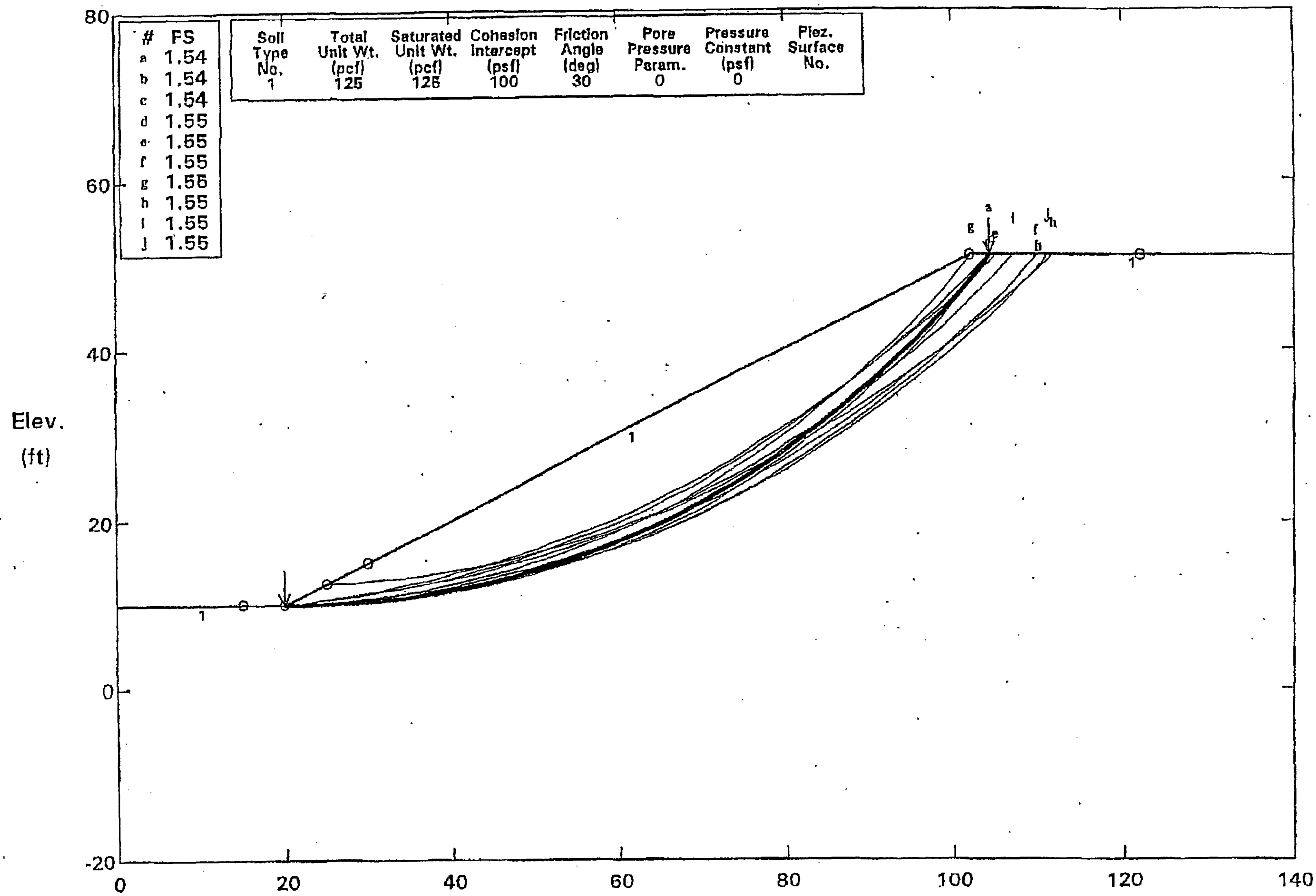


PCSTABL5M/SI FSmin=0.91 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

B-3, 41' HIGH, 2:1, STATIC

Ten Most Critical. C:B3-21S.PLT By: CM 5/09/2000 7:56am

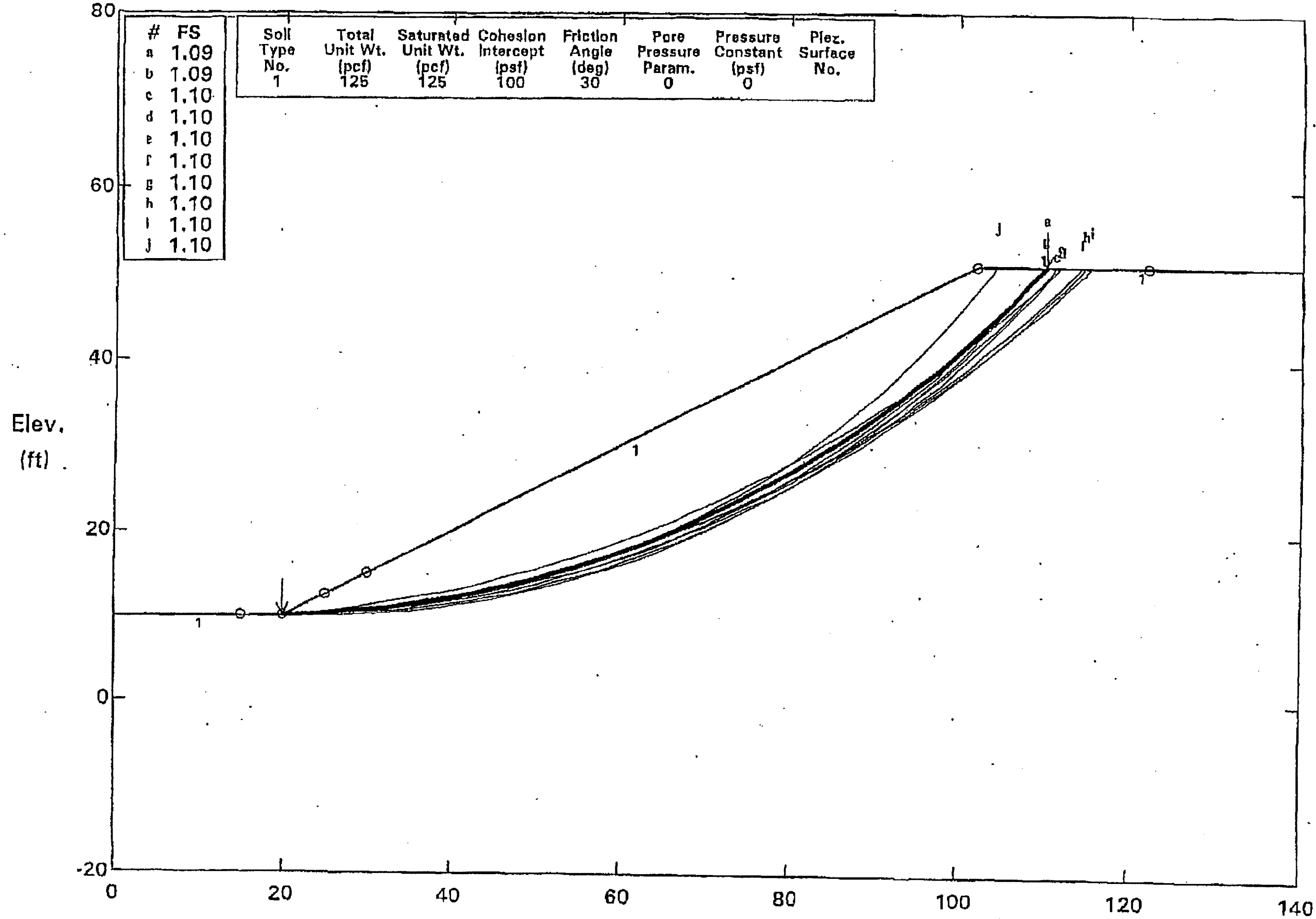


PCSTABL5M/SI FSmin = 1.54 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

B-3, 41' HIGH, 2:1, PSEUDOSTATIC

Ten Most Critical. C:B3-21P.PLT By: CM 5/09/2000 7:55am

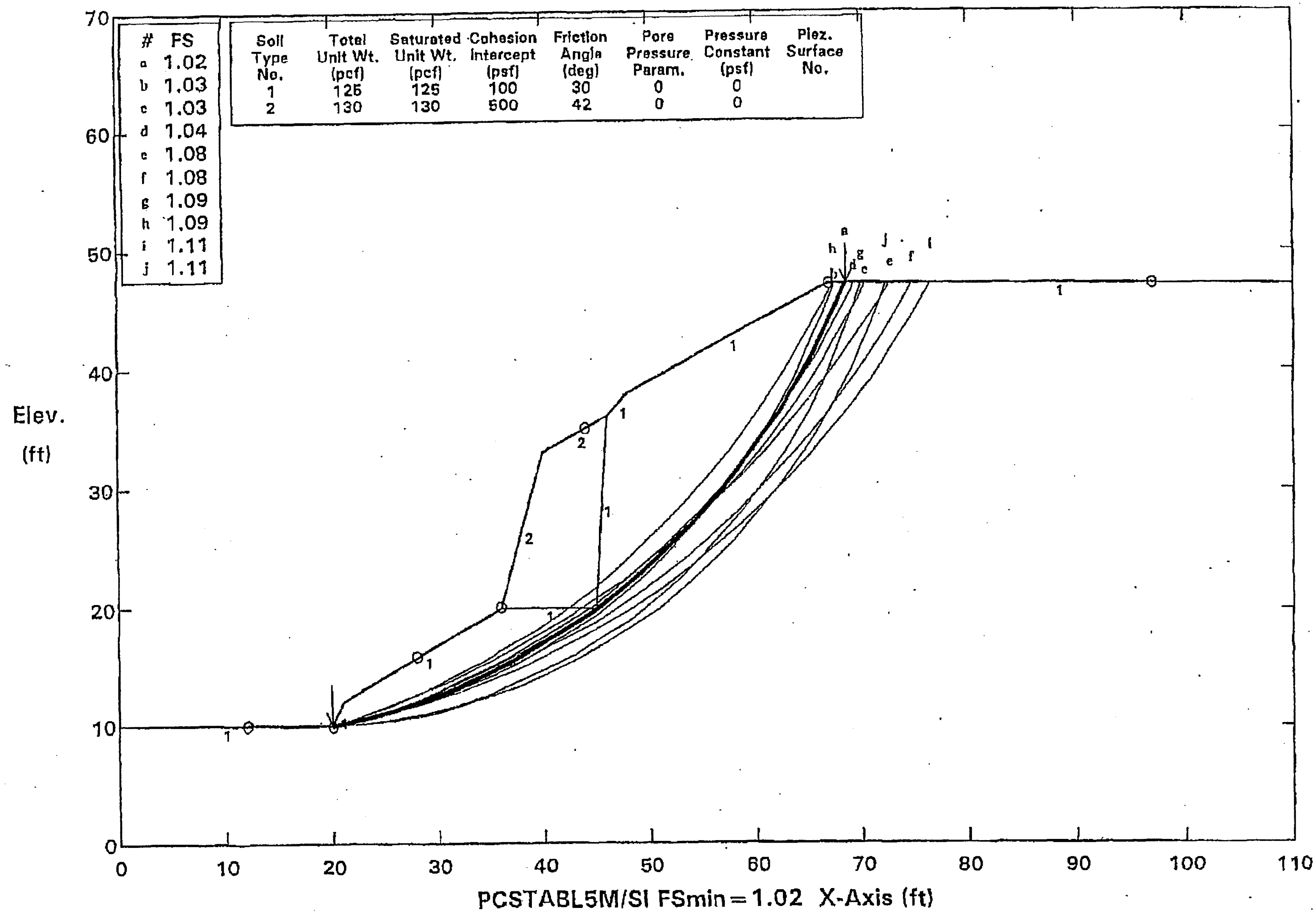


PCSTABL5M/SI FSmin = 1.09 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

C-3, 37' HIGH, GABIONS, EXISTING, STATIC

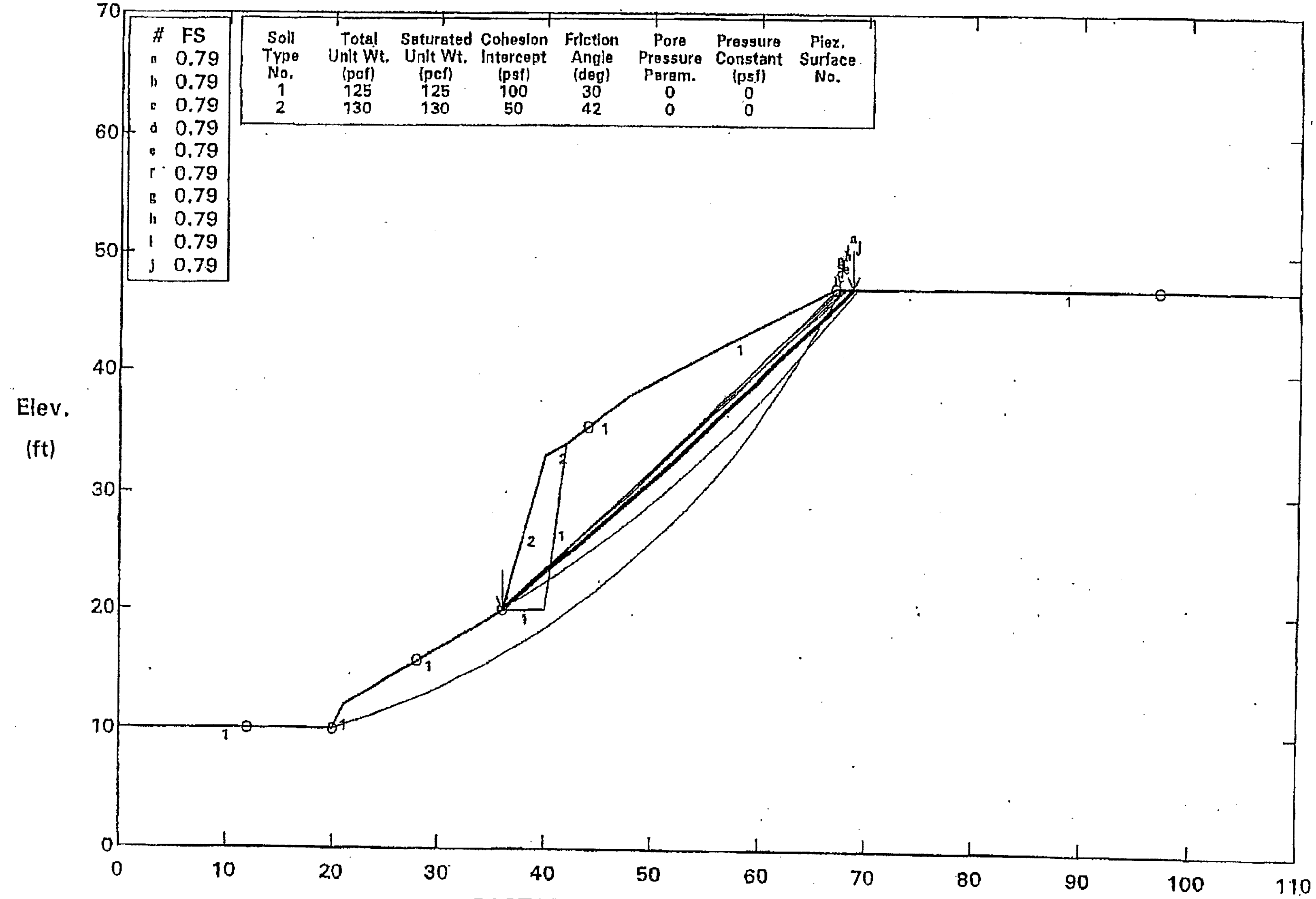
Ten Most Critical. C:C3EXISTS.PLT By: CM 5/09/2000 10:37am



Factors Of Safety Calculated By The Modified Bishop Method

C-3, 37' HIGH, GABIONS, EXISTING, PSEUDOSTATIC

Ten Most Critical. C:C3EXISTP.PLT By: CM 5/09/2000 10:36am

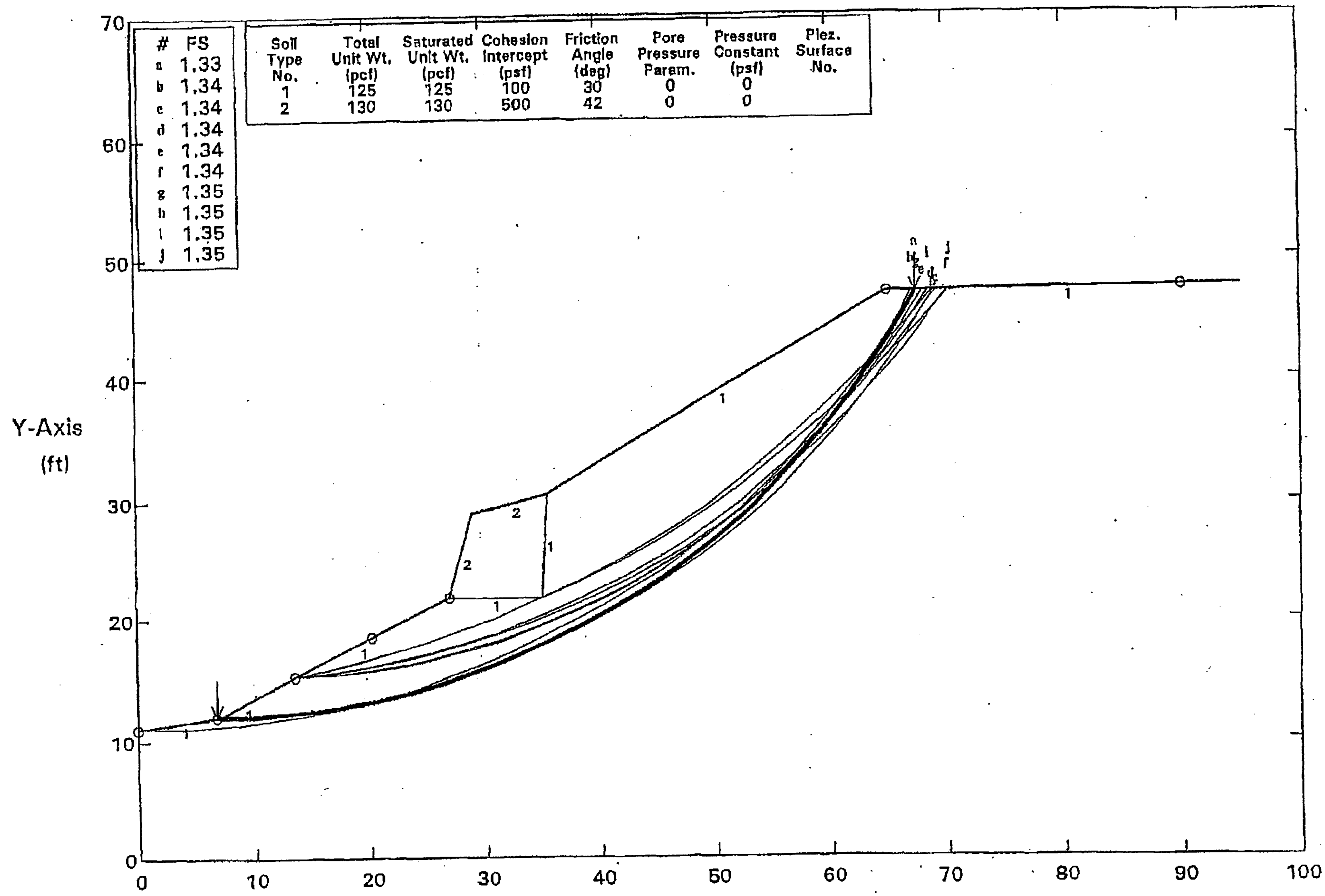


PCSTABL5M/SI FSmin = 0.79 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

36' HIGH, GABIONS, EXISTING, STATIC

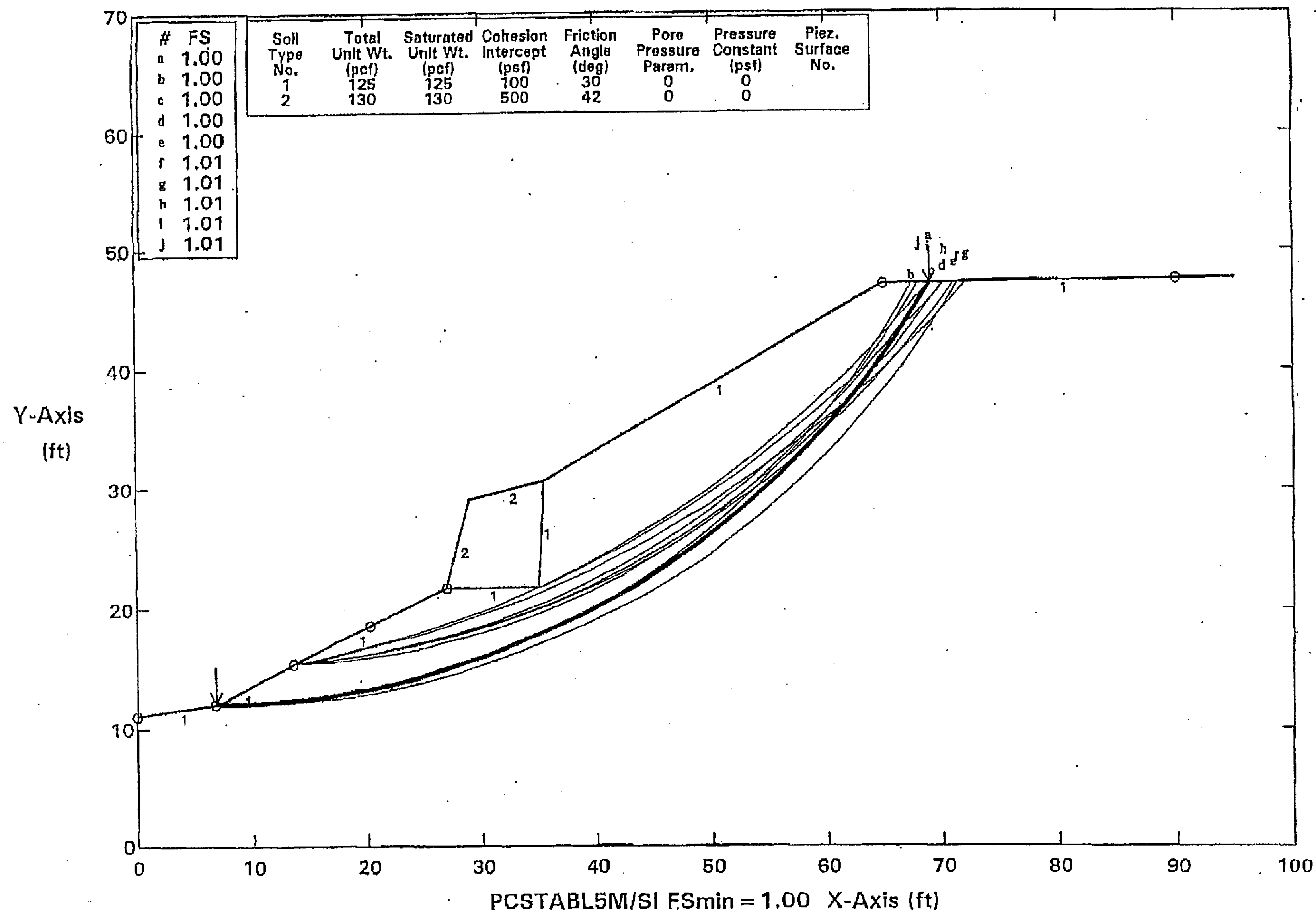
Ten Most Critical. C:GABIONS.PLT By: CM 5/18/2000 4:08pm



Factors Of Safety Calculated By The Modified Bishop Method

36' HIGH, GABIONS, EXISTING, PSEUDOSTATIC

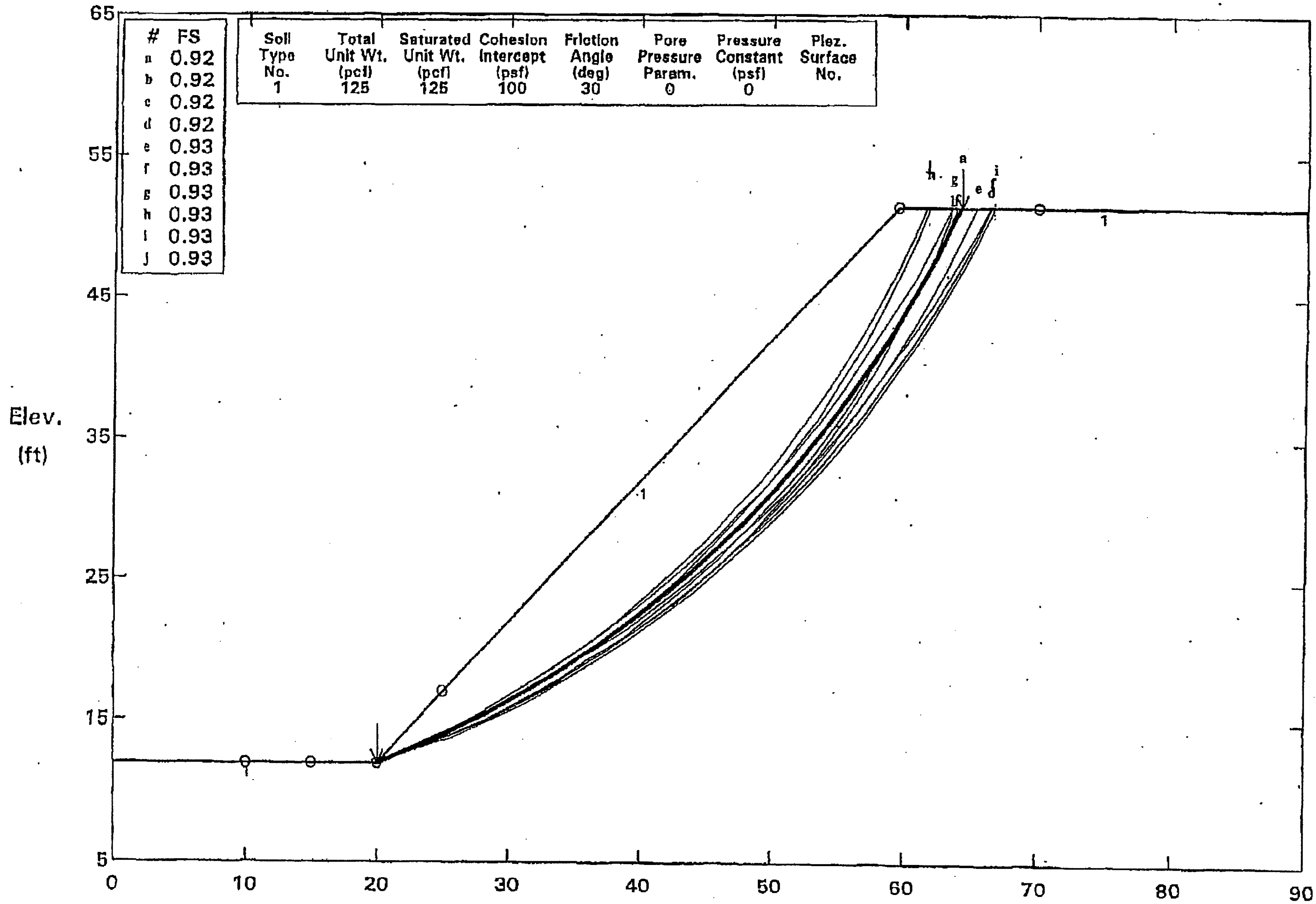
Ten Most Critical. C:GABIONP.PLT By: CM 5/22/2000 4:53pm



Factors Of Safety Calculated By The Modified Bishop Method

C-5, 39.5' HIGH, 1:1, STATIC, GLIDERS SLOPE

Ten Most Critical. C:C5-11S.PLT By: CM 5/09/2000 11:53am

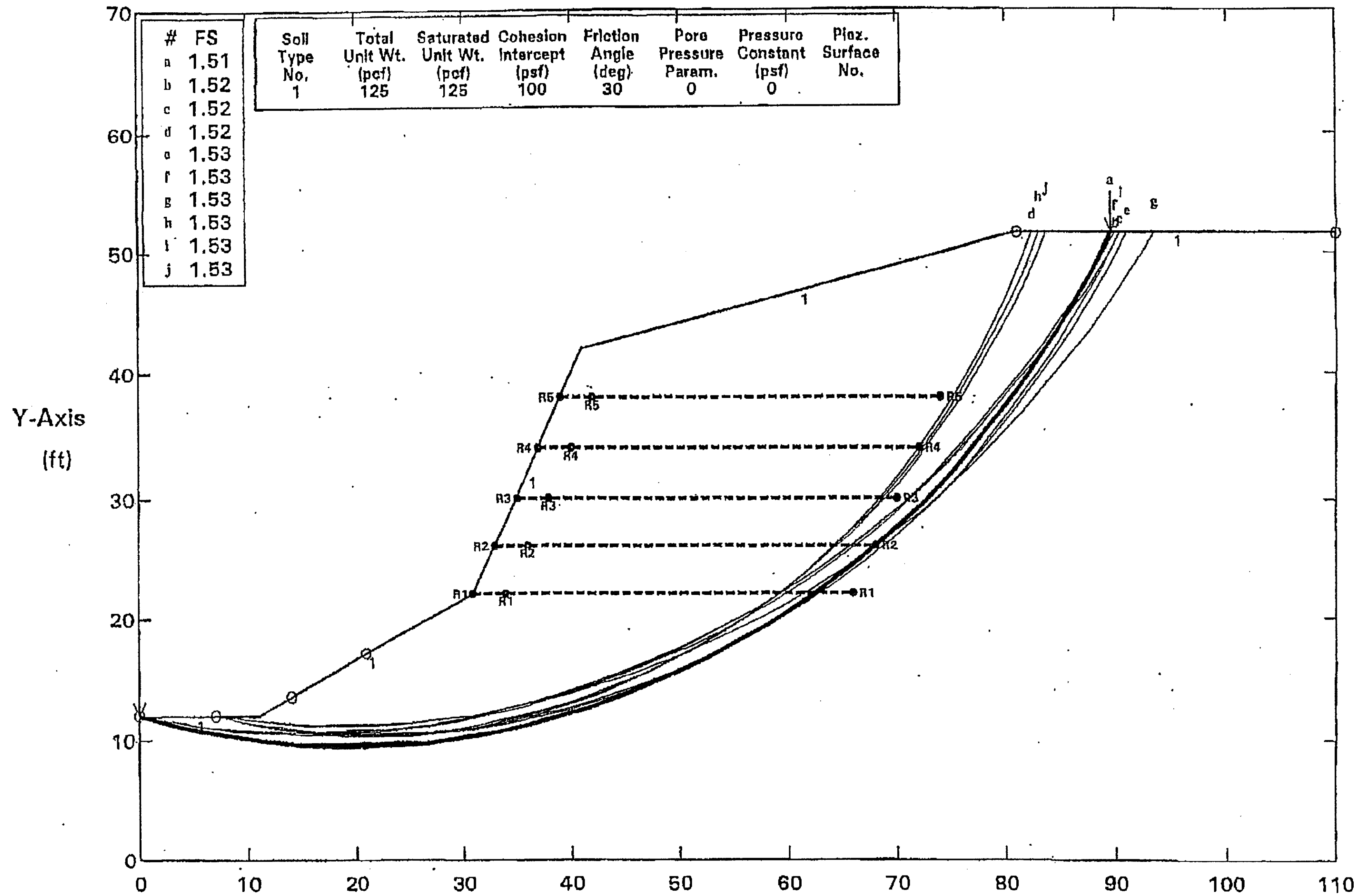


PCSTABL6M/SI FSmin = 0.92 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

C-5, 39.5' HIGH, 1/2:1, WITH GEOGRID, STATIC, GLIDERS SLOPE

Ten Most Critical. C:C5HALF1S.PLT By: CM 5/26/2000 5:35pm

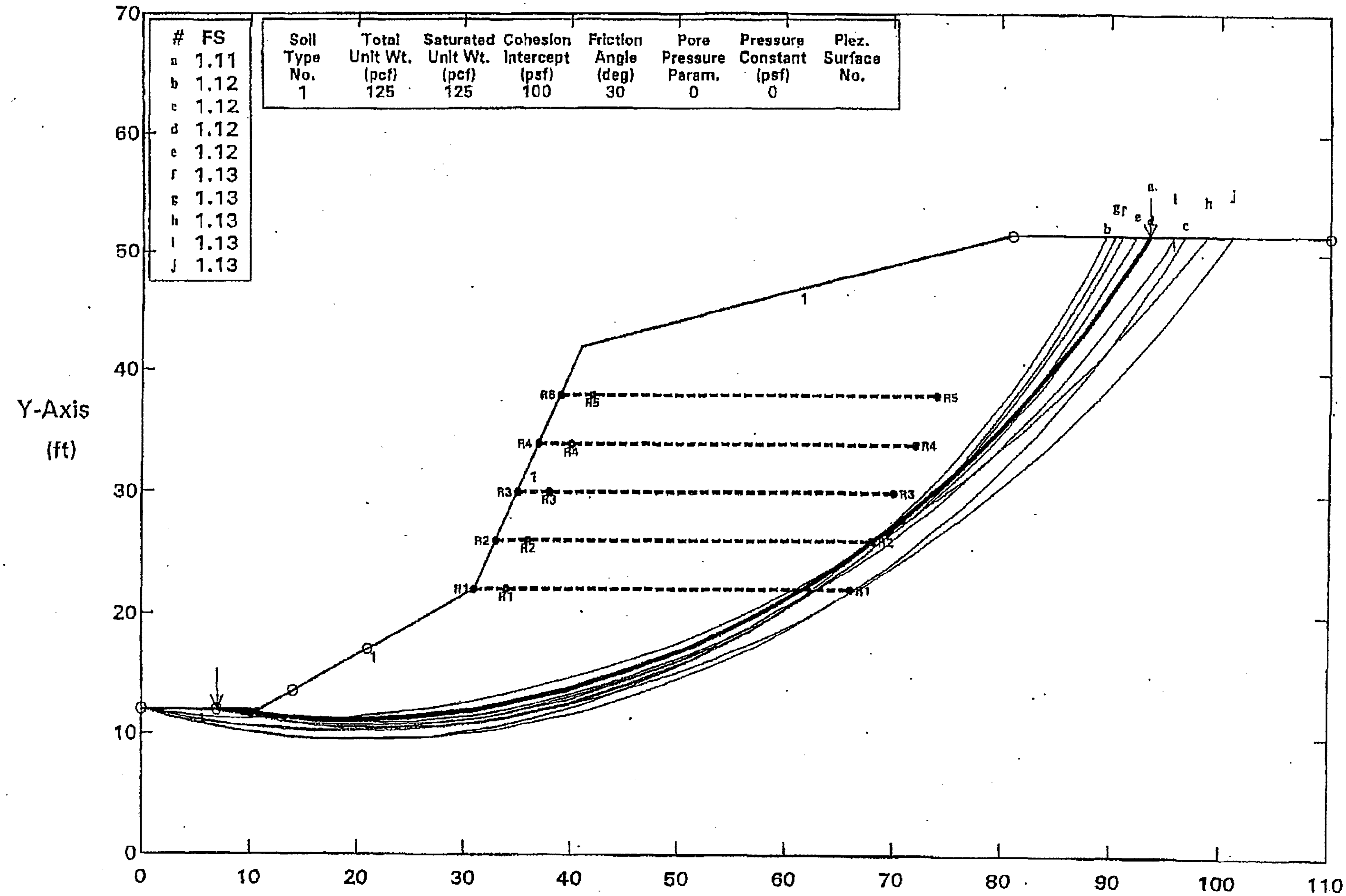


STABL6H FSmin = 1.51 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

C-5, 39.5' HIGH, 1/2:1, WITH GEOGRID, PSEUDOSTATIC, GLIDERS SLOPE

Ten Most Critical. C:C5HALF1S.PLT By: CM 5/26/2000 5:41pm

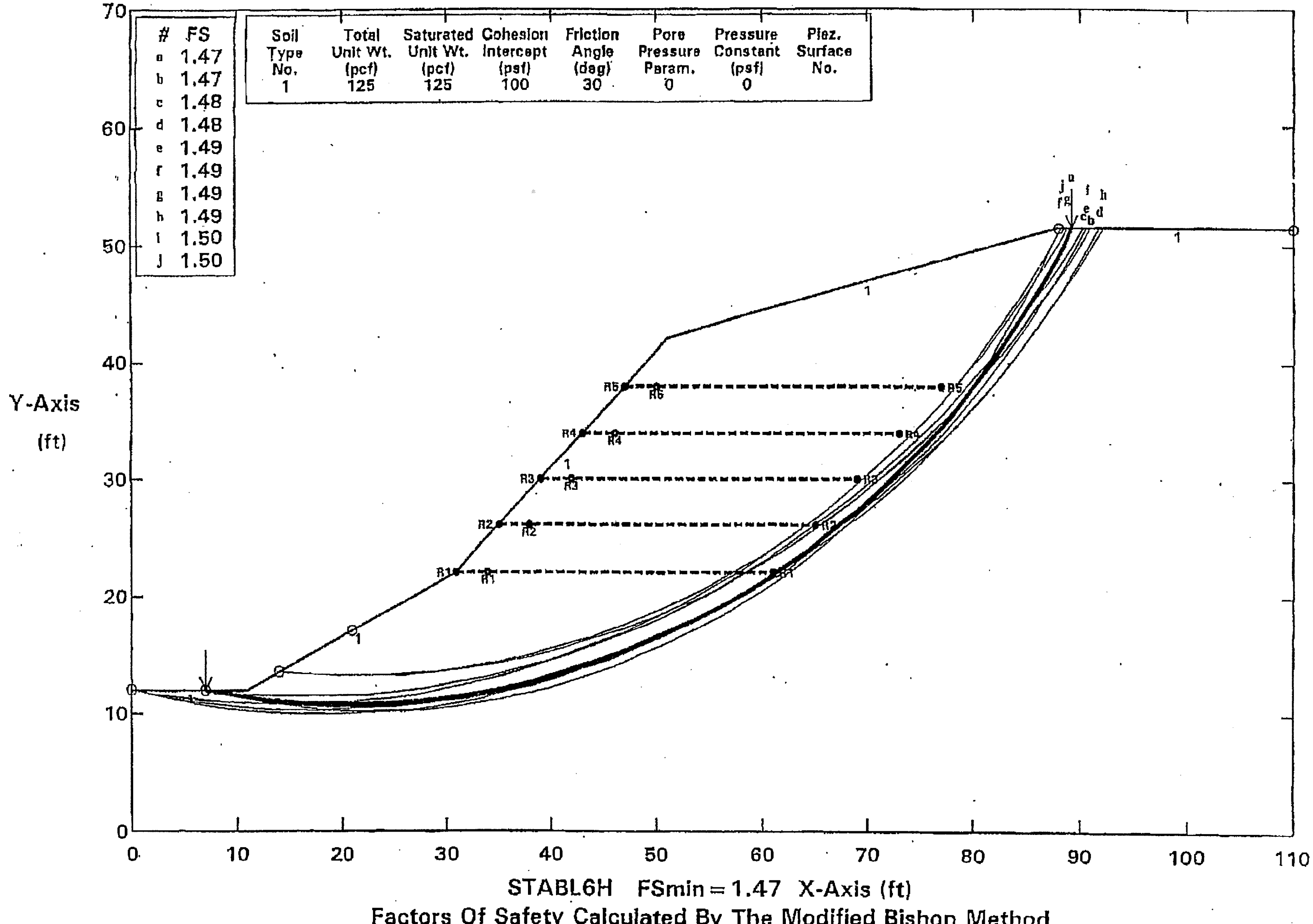


STABL6H FSmin = 1.11 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method

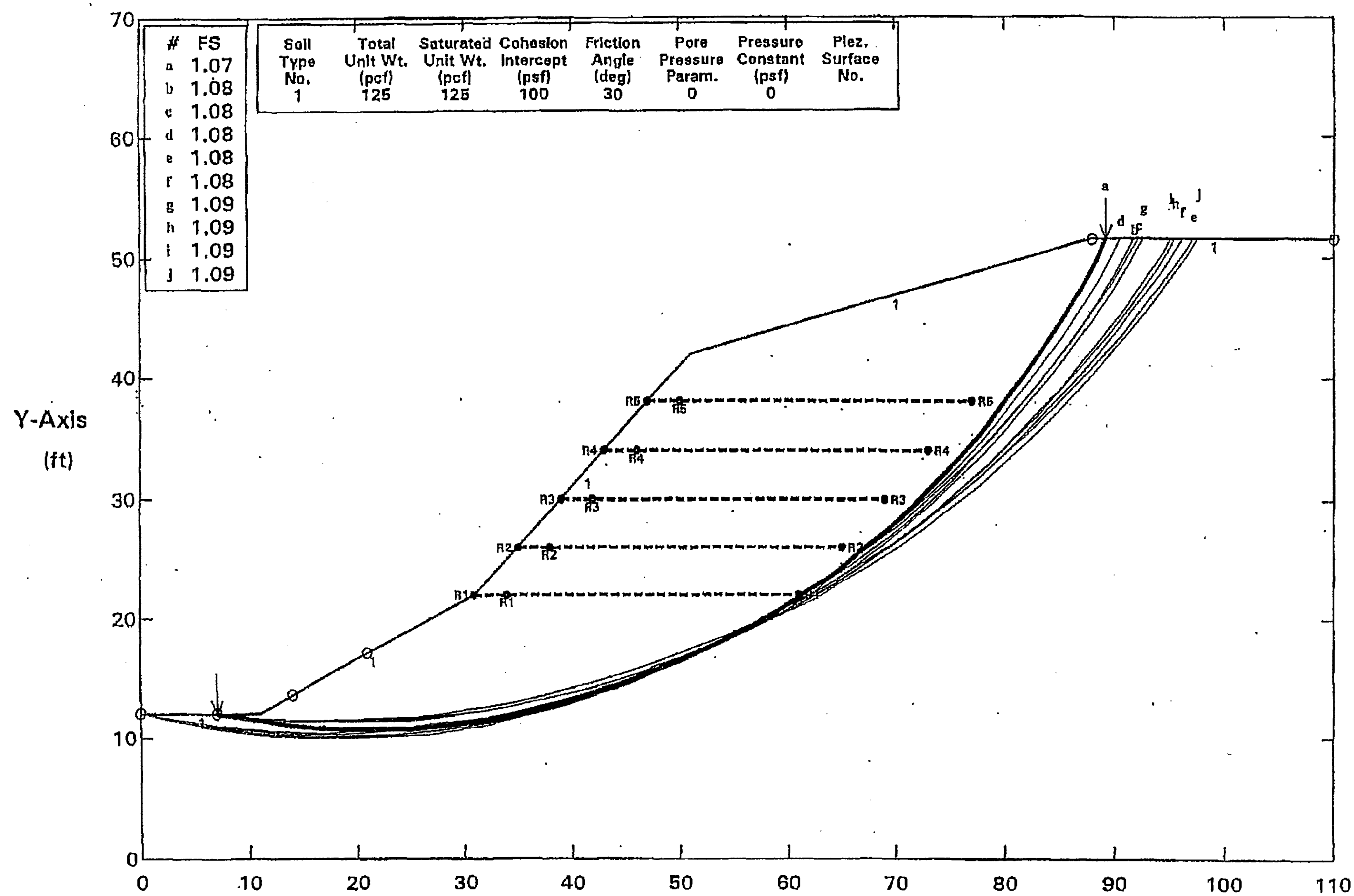
C-5, 39.5' HIGH, 1:1 WITH GEOGRID, STATIC, GLIDERS SLOPE

Ten Most Critical. C:C511GEOS.PLT By: CM 5/26/2000 4:56pm



C-5, 39.5' HIGH, 1:1 WITH GEOGRID, PSEUDOSTATIC, GLIDERS SLOPE

Ten Most Critical. C:C511GEOP.PLT By: CM 5/26/2000 4:57pm



STABL6H FSmin = 1.07 X-Axis (ft)

Factors Of Safety Calculated By The Modified Bishop Method