DEL MAR BLUFFS STABILIZATION
PROJECT 3 – PRESERVING TRACKBED SUPPORT

GEOTECHNICAL EVALUATION UPDATE

AND

DETERMINATION OF AREAS FOR STABILIZATION

Prepared for:

San Diego Association of Governments
401 B Street, Suite 800
San Diego, California 92101

Project No. 602576-001

April 9, 2010

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

April 9, 2010

Project No. 602576-001

To: SANDAG
401 B Street Suite 800
San Diego, California 92101

Attention: Mr. Christopher Poli, PE

Subject: Del Mar Bluffs Stabilization Project 3 – Preserving Trackbed Support, Geotechnical Evaluation Update and Determination of Areas for Stabilization

In accordance with your request, Leighton in conjunction with David Evans and Associates, is pleased to present the results of our geotechnical evaluation update of the coastal bluff stability between Milepost 244.1 and Milepost 245.7 in the City of Del Mar, California. This report is a follow-up to the previous 2003 Supplemental Geotechnical Evaluation and Determination of Site Specific Conceptual Repair Alternatives. The purpose of this evaluation was update our geological maps and to further analyze the stability of the 50- to 70-foot high coastal bluffs that provide support for the North County Transit District (NCTD) rail alignment, and to develop recommendations for repair measures. The following report provides our conclusions and recommendations with regard to preserving trackbed support of the North County Transit District rail alignment for at least 20 years.

If you have any questions regarding our report, please contact this office. We appreciate this opportunity to be of service

Respectfully submitted,
LEIGHTON CONSULTING, INC.

REGISTERED PROFESSIONAL ENGINEER
WILLIAM D. OLSON
No. 45283
Exp. 9/30/10
CIVIL
STATE OF CALIFORNIA

William D. Olson, RCE 45283
Associate Engineer

REGISTERED GEOLOGIST
MICHAEL R. STEWART
NO. 1349
CERTIFIED ENGINEERING GEOLOGIST
exp 12/31/1
STATE OF CALIFORNIA

Michael R. Stewart, CEG 1349
Principal Geologist/Vice President

PROFESSIONAL GEOLOGIST
ROBERT C. STROH
NO. 2099
CERTIFIED ENGINEERING GEOLOGIST
EXP. 10/31/11
STATE OF CALIFORNIA

Robert C. Stroh, CEG 2099
Senior Project Geologist

Distribution: (6) Addressee
(3) David Evans and Associates, Attention: Ms. Tricia McColl

3934 Murphy Canyon Road, Suite B205 ■ San Diego, CA 92123-4425
858.569.6914 ■ Fax 858.292.0771 ■ www.leightonconsulting.com

TABLE OF CONTENTS

Section Page

Leighton

TABLE OF CONTENTS

Leighton

1.0 INTRODUCTION

1.1 Project Location

The Del Mar Bluffs Stabilization Project 3 - Preserving Trackbed Support (Project 3) is situated along 1.6 miles of North County Transit District (NCTD) railroad right-of-way on the western edge of the City of Del Mar, as shown on Figure 1, Site Location Map. The project area extends from rail Milepost (MP) 244.1 near Coast Boulevard south to MP 245.7 at about Torrey Pines State Beach. Within this reach, the NCTD rail alignment runs atop the 50- to 70-foot high coastal bluffs. Railroad right-of-way varies between approximately 100 feet and 235 feet in width and, in some places, extends onto the beach below.

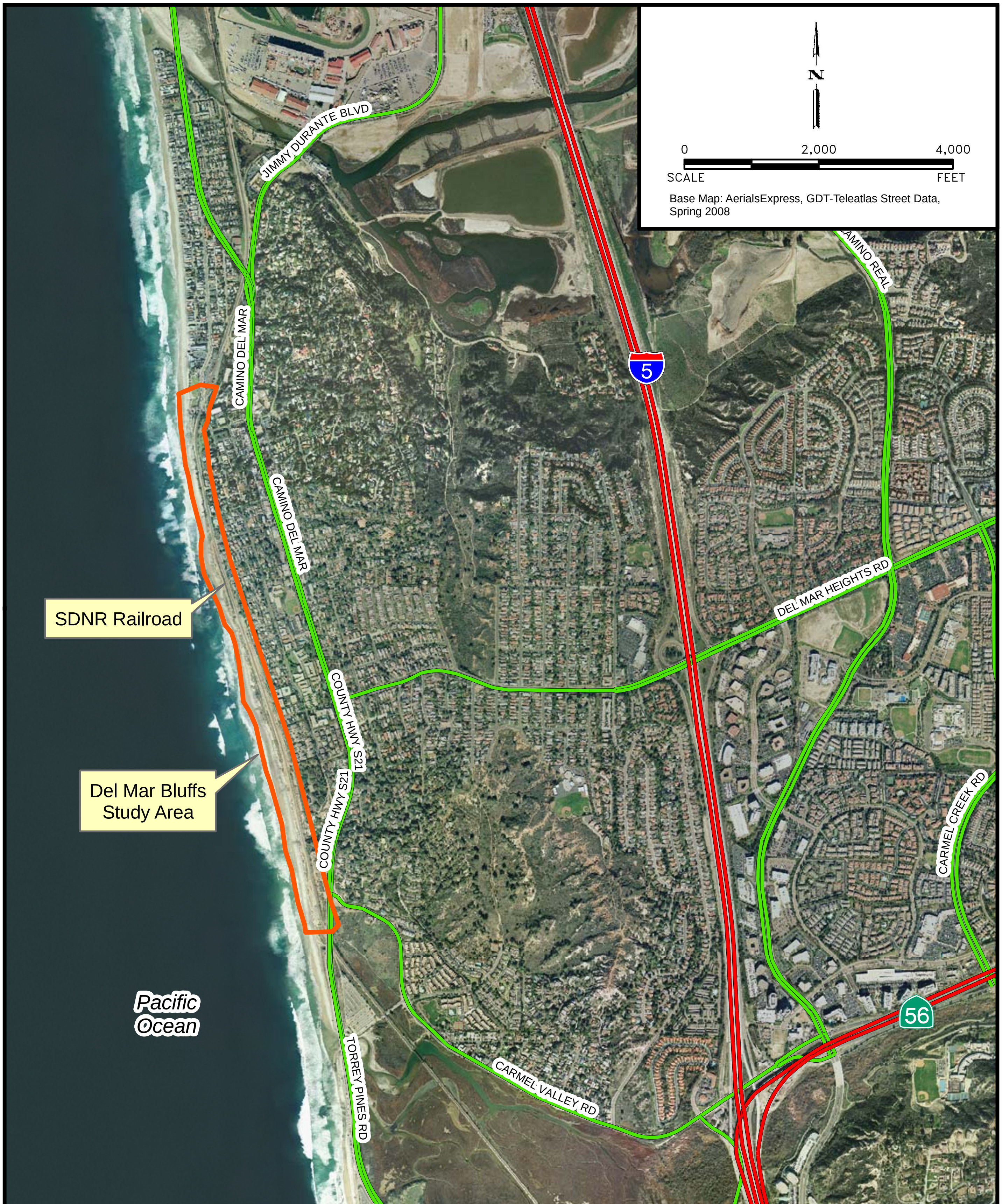
1.2 Project Description

The coastal bluffs supporting the rail alignment in the project area have a history of landslides and surficial failures. In addition, the bluffs are subject to ongoing erosion and failures that could threaten the viability of rail service. Project 3 includes an updated geotechnical evaluation for the design and installation of stabilization measures intended to preserve trackbed support in areas to maintain the viability of rail operations for at least 20 years. It should be noted that this current project is a continuation of Project 2, which consisted of the design and installation of stabilization measures intended to preserve trackbed support in high-priority areas (Leighton, 2003).

1.3 Project Background

The NCTD railroad right-of-way is an integral part of the 128-mile Los Angeles to San Diego (LOSSAN) Rail Corridor. The corridor provides a vital link for passenger and freight movements within San Diego County as well as between San Diego, Los Angeles and points further north. Approximately 46 passenger trains per day traverse the section of track within the project area, including NCTD Coaster commuter rail service and Amtrak Pacific Surfliner inter-city rail service. In terms of ridership, the LOSSAN corridor is Amtrak's second busiest inter-city rail corridor in the nation.

In addition to passenger rail, eight freight trains, operated by the Burlington Northern Santa Fe Railway (BNSF), typically travel this section of track daily carrying several million tons of freight per year.



Del Mar Bluffs Del Mar, California	SITE LOCATION MAP	Project No. **602576-001** Date **April 2010** Figure 1

\\GIS\Drafting\602576\001\GIS\of_09-09-15\Figure1-SiteLocationMap-aerial.mxd

The LOSSAN rail corridor is considered a critical facility due to the dependence on the facility by passengers and by freight movements as mentioned above and due to the fact that it is the only rail line connecting San Diego to points north. Therefore, as part of maintaining its portion of the LOSSAN Rail Corridor, NCTD has adopted a four-phased approach to preserve the track structure, ensure the viability of rail service and protect its investment in the railroad right-of-way located along the bluffs tops. The first phase of this approach included construction of approximately $1.8 million in drainage improvements along the right-of-way which were completed in 1998.

The second phase, which was completed in January 2001, included conducting a geotechnical study of the bluffs. The results of the geotechnical study (herein referenced as the "Geotechnical Study (Leighton, 2001a)"), was entitled the "Del Mar Bluffs Geotechnical Study, Part 1: Geotechnical Evaluation and Part 2: Conceptual Repair Alternatives" and was prepared by Leighton and Associates. The Geotechnical Study (2001a) characterized the nature and causes of bluff erosion, identified and prioritized areas in need of stabilization, and presented a range of conceptual stabilization options. The Geotechnical Study (Leighton, 2001a) concluded that the bluffs are subject to failure due to inadequate lateral support, storm wave action, and significant seismic activity. In addition, groundwater seepage and inadequate surface drainage were identified as factors that contributed to the ongoing degradation of the coastal bluff.

The information provided in the Geotechnical Study (Leighton, 2001a) served as the basis for phase three of the Del Mar Bluffs trackbed preservation program and defined overall project priorities. Phase three (the current phase) originally consisted of two separate Del Mar Bluffs stabilization projects to design and construct stabilization measures within "high-priority" areas.

- The first project entitled "Del Mar Bluffs Stabilization Project 1 – Drainage Improvements and Landslide Warning System" (Project 1) was completed in 2002 and included the installation of surface and subsurface drainage improvements along the NCTD railroad right-of-way and within the defined high-priority areas. Project 1 also included the installation of a landslide warning system within portions of the high-priority areas to provide early warning of slope failure along the railroad right-of-way.

- The second project entitled "Del Mar Bluffs Stabilization Project 2 – Preserving Trackbed Support" (Project 2) was completed in 2008 and involved the design and installation of roughly 1,400 feet of stabilization measures to provide additional lateral support for the railroad right-of-way within high-priority areas that are marginally stable (Leighton, 2003 and SWE, 2008). Project 2 identified areas currently in need of stabilization and prioritized mitigation of those areas based on geotechnical analysis.

Due to funding limitations, not all areas identified in Project 2 as in need of stabilization were stabilized. Consequently, phase three has now been expanded to include a third project.

- The third project entitled "Del Mar Bluffs Stabilization Project 3 – Preserving Trackbed Support" (Project 3) is a continuation of Project 2 and involves the design and installation of stabilization measures to provide additional lateral support for the railroad right-of-way within areas identified in an updated geotechnical evaluation (this report) using new topographic mapping and geological observations.

The fourth phase of the approach (yet to be completed) includes identification of an alternative railroad alignment through the City of Del Mar as a long-term solution to address bluff erosion and slope stability concerns. The California High-Speed Rail Authority (CHSRA), in association with the California Department of Transportation (Caltrans), has evaluated preliminary alignment alternatives as part of their study of conventional rail improvements between Los Angeles and San Diego. The LOSSAN Program EIR/EIS (2007), developed by the agencies, identified two general planning level alignments: however, significant additional study is necessary.

1.4 Purpose and Scope

The Geotechnical Study (2001a) originally identified the areas in need of bluff stabilization and presented a range of stabilization measures; however, the stabilization measures (or repair alternatives) were conceptual in nature and not site specific. In 2003, a follow up geotechnical study report was prepared by Leighton, entitled "Supplemental Geotechnical Evaluation and Determination of Site Specific Conceptual Repair Alternatives" (herein referenced as the "Geotechnical Report (Leighton, 2003)"). The report further evaluated the existing bluff in the previously defined "high-priority areas" and identified site specific repair measures for those areas needing mitigation. Subsequently, construction of the site specific repair measures along some of the high-priority areas were completed in 2008 (SWE, 2008). The purpose of this report is to update the geotechnical evaluation of the bluff in those areas not mitigated by Project 2 construction based on new topographic mapping (DEA, 2009), identify and prioritize those areas currently in need of mitigation and identify the site specific repair measures for those areas.

The first part of this report provides supplemental geotechnical data based on the results of the additional geotechnical evaluation performed since the completion of the Geotechnical Report (Leighton, 2003). As part of this evaluation, geotechnical analysis (including slope stability analysis) of representative geologic cross sections prepared at selected locations along the bluff was performed. The results of the analysis, along with previously completed analyses and topographic and geological features along the right-of-way, are then used to identify areas currently in need of trackbed support, i.e. "stabilization areas." Similar to Project 2, these new stabilization areas are prioritized based on calculated factors of safety and the degree to which additional lateral support/mitigative measures are recommended to mitigate the potential for deep-seated instability of the trackbed foundation materials.

Next, the potential stabilization measures presented in the Geotechnical Report (Leighton, 2003) are revisited to briefly discuss the characteristics of the various conceptual repair alternatives and whether or not they meet the objectives of the project. The project objectives are: preserve trackbed support along the railroad alignment for at least a 20-year period; provide minimum recommended factors of safety; maintain uninterrupted rail operations; and preserve natural bluff areas as much as possible. Those conceptual stabilization measures that do not meet the objectives for stabilization are dismissed from further consideration.

Finally, this report provides recommended conceptual repair alternatives and/or mitigative measures for each of the stabilization areas on a site specific basis using data developed during this and earlier phases of work.

2.0 SITE CONDITIONS, SUBSEQUENT INVESTIGATIONS AND RELATED CONCLUSIONS

2.1 Site Geology

The geologic conditions of the project site were described in the initial Geotechnical Study (Leighton, 2001a). Additional studies recommended in that report have been achieved by: 1) the drilling of additional borings as part of the Project 1 design; 2) the logging of borings drilled as part of the Eighth Street Emergency Repair; 3) additional field mapping; 4) observations of backhoe test pits and trenches, hydro-augers and other exposures related to Project 1 and Project 2 construction; and 5) additional slope stability analyses. As part of this project, a new topographic base map was prepared, and additional geologic mapping performed. Additional cross-sections were prepared and analyzed in preparation of this report.

To summarize the geologic conditions, the site is underlain by sandy permeable materials of the Quaternary-aged Bay Point Formation (i.e. Terrace Deposits) which overlie the generally dense sandstones (Tdss) and relatively impermeable siltstones and claystones (Tdcs) of the Eocene-aged Delmar Formation. This unit also includes localized permeable zones related to sandy lenses and sandy paleo channel infill deposits, and dense resistant layers. The extent and elevations of these dense layers have been better defined by observations during construction activities of Project 1 and the supplemental field mapping activities of this report near the base of the bluff. The Eocene-aged Torrey Sandstone can be observed just east of the track in the southern portion of the site and within Anderson Canyon. This unit is shown on the geologic maps and cross sections but does not underlie the rail alignment.

Within both formations that underlie the right-of-way there are fracture zones that roughly parallel the bluff face. Observations related to Project 1 construction and the logging of the borings drilled since the Geotechnical Study (Leighton, 2001a) also confirmed the presence of near horizontal layers of highly fractured claystone within the Delmar Formation that were identified in some of the earlier borings. Shears within these zones are highly polished and randomly oriented. In addition to these horizontal claystone beds, steeply dipping fractures and joints are also present. Near-vertical fractures and joints are closely spaced near the bluff face, but steeply dipping fractures and joints can also be observed at wider spacing throughout the entire right of way. As an example, closely spaced vertical fractures and joints can be observed at the outlet excavation at 8th Street (Project 1). More steeply dipping fractures and joints were observed in borings LB-2 through LB-6 and also in several of the Project 1 and 2 excavations, which were located near the track. The highly fractured zones near the bluff face can in part be attributed to weathering. East of the bluff face, the formation of these highly fractured claystone beds and the presence of steeply dipping fractures and joints within what is a typically brittle formational unit, are believed to be related to tectonic and/or depositional processes.

These joints and fracture zones consist of breaks in the bedrock and provide weak zones on which failures can occur and also conduits for ground water migration within the bluff.

The approximate areal extent of each of the geologic units and the interpretation of the subsurface geologic conditions are indicated on the provided Geotechnical Maps and Geological Cross-Sections (Plates 1 through 10).

2.2 Field Explorations

In order to evaluate the site's pertinent soil and geologic conditions and develop the site geotechnical maps (Plates 1 through 7) and geologic cross sections (Plates 8 through 10) used for slope stability analysis, several phases of field investigations, geologic mapping and numerous exploratory borings have been performed. Because of the ongoing erosion of the bluff, a new topographic base map was prepared and new geologic mapping performed to evaluate the current site conditions. The result of this work is presented on the Geotechnical Maps and Geological Cross-Sections (Plates 1 through 10). For previous exploration borings, please refer to the Geotechnical Report (Leighton, 2003).

As previously mentioned, additional geologic mapping was performed between May 26 and June 11, 2009. The geologic mapping utilized new topographic base maps which were developed by David Evans and Associates, 2009, using new aerial photographs covering the site area. The contour interval utilized for these maps is 1 foot, thus providing superior accuracy over previous base maps utilized at the site. In addition, the mapped features and geologic contacts were more accurately located with the utilization of a handheld Trimble GeoExplorer 2005 Series GPS. This GPS unit provided lateral sub-meter accuracy. During this phase of recent mapping additional geologic details were provided including mapped sub-units within the Delmar Formation consisting of both the sandstone and claystone units, along with mapping of additional area south of the previously mapped area from the (Leighton, 2003) report to the southern most extent of the bluffs. In addition, this recent phase of mapping included mapping of the slopes eastward of the existing rail road track.

In total, 32 borings have been drilled to depths ranging from 60 to 70 feet below the existing ground surface (bgs). These included 22 small diameter borings drilled by a hollow-stem auger drill rig and 10 large-diameter borings. The large-diameter borings were downhole logged by geologists to better evaluate the subsurface conditions. The borings have been used to characterize the subsurface conditions and develop the geologic cross sections utilized in the slope stability analysis. These cross sections and the geologic maps have been refined with the results of the additional data obtained since the completion of the Geotechnical Study (Leighton, 2001a).

The 32 borings, discussed above, include 4 small-diameter borings drilled by Leighton in 1978, 16 small-diameter borings drilled by others as part of the first phase of the Geotechnical Study (2001a), 6 large-diameter borings drilled by Leighton for the Geotechnical Study (2001a), 3 large-diameter borings and 2 small-diameter borings drilled by Leighton as part of Project 1, and 1 additional large-diameter boring drilled and downhole logged at the Eight Street Emergency Repair (Leighton, 2001b).

In addition to the field investigations and subsurface explorations, geologists have observed the installation of 70 hydro-augers and numerous construction excavations as part of Project 1 and Project 2. The results of this additional work with some refinement have confirmed the findings presented in the Geotechnical Study (Leighton, 2001a), and in the Project 2 Geotechnical Report (Leighton, 2003).

2.3 Laboratory Testing

Laboratory testing was performed on representative soil samples obtained during the various phases of previous site explorations and the results were utilized in the current slope stability analysis. A discussion of the strength parameters utilized in the slope stability analysis is presented in Section 3.3. For test data and details on the laboratory testing, please refer to the Geotechnical Report (Leighton, 2003).

2.4 Bluff Retreat

As discussed in the Geotechnical Study (Leighton, 2001a), average bluff retreat rates in the study area are estimated at a maximum of 0.4 to 0.6 feet per year. This corresponds to a retreat of approximately 10 feet in the project's 20-year design life, assuming that the bluff will retreat at an average rate of 0.5 feet per year for the next 20 years. Bluff retreat is typically episodic with no retreat for some time and then several feet or more occurring in one event.

2.5 Ground Water

As described in the Geotechnical Study (Leighton, 2001a), ground water is a major factor influencing slope stability as it accelerates the degradation of the bluff and bluff face erosion. Based on observations during the various phases of field investigation, hydro-auger installation and construction excavations, the majority of the ground water is located in a perched horizon at the base of the Bay Point Formation with additional localized zones of ground water within near-vertical fractures and joints and sandy channel infills of the Delmar Formation. As discussed previously, geologic observations indicate that the near-vertical fractures and joints within the Delmar Formation are more prevalent near the bluff face, but do extend landward with lesser frequency and typically wider spacing through the entire right-of-way as observed in numerous borings and trenches. These near-vertical fractures and joints create potential pathways for migration of ground water throughout the bluff and the right-of-way.

Ground water can also be observed as numerous localized seeps in the exposed bluff face with additional seepage zones likely masked by dense vegetation or loose surficial soils. Fluctuation in ground water levels within the near-surface soils and weathered and fractured material near the bluff face is also anticipated after periods of heavy rainfall resulting in additional seepage zones and a temporary increase in seepage.

Since construction of the rail alignment in the early 1900's, there have been many efforts to reduce the amount of water in the bluff. Historically these efforts have included construction of a storm drain system, surface drainage improvements and the installation of subdrains. In 1998, NCTD completed construction of $1.8 million of additional surface and subsurface drainage improvements. Additionally, Project 1, which also consists of both surface and subsurface drainage improvements, was completed in 2003.

While these past projects have collected a large amount of subsurface water, not all ground water is intercepted by these improvements as evidenced by lingering seepage in the exposed bluff face in improved areas. In addition, not all areas of the bluff have had drainage improvements installed. In the central "trough" area only limited or no subsurface drainage improvements have been installed primarily because the distance from the track to the bluff face did not result in low factors of safety for this area. Recent site observations indicate an increase of bluff retreat in localized areas that is caused by subsurface flows eroding (piping) the bluff face.

Additional discussion of the ground water levels utilized in the slope stability analysis is presented in Section 3.1.

3.0 SLOPE STABILITY

The Geotechnical Study (Leighton, 2001a) characterized the overall bluff stability, established the high-priority areas and provided conceptual repair alternatives to improve the slope stability. Actual repair recommendations were to be made based on site specific analysis. Since the completion of the Geotechnical Study, additional investigation of the bluff has been accomplished for the Eighth Street Emergency Repair, the Project 1 improvements and the Project 2 improvements. The results of the additional borings, field observations, and laboratory testing are considered in the slope stability analysis of this report. This section presents the results of additional site specific slope stability analysis of the remaining portions of the bluff where stabilization efforts have not yet been performed, i.e. those area not stabilized by Project 2 construction. In addition, this analysis is to be utilized in the selection and further development of stabilization measures.

3.1 Stability Analysis

As background, 25 geological cross sections were previously analyzed for Project 2 (i.e., geological cross sections, Sections 1-1' through 25-25', see Plates 1 through 6). For this study, 21 new geological cross sections were selected at appropriate locations to evaluate selected areas of potential concern. These new cross sections, developed from the new topographic maps, were labeled with letters A through M1 in order to differentiate the new from the old.

The locations of the new geological cross sections were selected based on: 1) the results of the previous slope stability analyses presented in the Geotechnical Report (Leighton, 2003); 2) the site specific geologic conditions; 3) recent field observations that include determining the lateral distance between the track and top edge of the bluff; and 4) the location of stabilization measures constructed as part of Project 2.

The stability analysis performed for this study utilized the computer program Slope/W (Geo-Slope, 2002) with Spencer's and Bishop's methods for block and circular failure modes, respectively. Analyzed scenarios included: 1) static conditions; 2) static conditions with a train surcharge loading; and 3) pseudo-static (seismic) conditions.

While near-vertical fractures and joints can be observed throughout the right of way, they are most concentrated near the bluff face. For the purposes of slope stability analysis, the analyzed static and surcharged scenarios in the northern areas (north of MP 245.21 or Station 1491+20) considered a shallow profile of ground water parallel to the bluff face (a 5-foot hydrostatic head within a 10-foot fractured bluff face zone). This ground water profile is considered to be a valid representation of the current site conditions based on observations during the construction activities of Project 1. For analyses in the southern portions of the project (south of MP 245.21 or Station 1491+20), substantially less ground water seepage is observed on the bluff face. The reduction in the ground water to the south is due to the lack of the permeable terrace deposits on the bluff top, existing drainage improvements that extend through the terrace deposits and the distance from the upslope developments. As a result, the ground water profile model was changed to incorporate a 2-foot hydrostatic head within a 5-foot fracture zone. Figures 2 and 3 present generalized cross sections that illustrate the ground water profiles utilized in the slope stability analysis.

It should be noted that the slope stability analysis contained herein is based on existing conditions and does not include the effects of additional bluff retreat. While additional bluff retreat is likely to occur over the life span of the project, it was not considered in the stability analysis. According to SANDAG, there are limited funds available for stabilization of the bluffs at this time. Therefore, the goal of Project 3 is to identify the areas currently in need of stabilization, prioritize the areas by greatest need, and stabilize the areas in order of priority as funding allows. Additional bluff retreat will, as identified in the Geotechnical Study (Leighton, 2001a), expand the areas in the future and, as additional funds become available, further stabilization will be considered where appropriate.

3.2 Factor of Safety

Consistent with the 2003 Supplemental Geotechnical Evaluation, the calculated factors of safety, generated by the slope stability analysis program for each cross section, were used to assess the stability of the bluff as it exists today. In order to generate these calculated factors of safety, the model required selection of a constant evaluation point at which the potential failure surface intersects the existing ground surface. As there are no specific criteria published to aid in the selection of such a point, the distance was established using engineering judgment that was primarily based on the State of California Department of Transportation (Caltrans) Trenching and Shoring Manual, Section 7 "Railroads".

In the Caltrans manual, shoring requirements are determined based on the relationship of excavations to "railroad reference lines", below which any excavation requires shoring. In view of this, a point 10 feet from the railroad centerline (approximately 6 feet beyond the end of a typical railroad tie) was selected as it is just inside the limits of the aforementioned reference lines and thus would require shoring according to the Caltrans manual. In addition, NCTD has indicated that a failure within 10 feet of the track would be a serious concern and would likely "shut down" the rail line. It should be noted that a greater distance could be chosen if it was required to have maintenance or emergency vehicle access on the west side of the track. Also, as the evaluation point moves toward the edge of the bluff, the calculated factor of safety for all cross sections would decrease and subsequently, the areas with low factors of safety requiring stabilization would increase.

The calculated factors of safety generated, as discussed above, are compared to minimum factors of safety in order to assess the potential for failure within the established 10-foot distance from the railroad centerline. The following minimum factors of safety (FS) were considered reasonable or acceptable parameters:

- Static Analysis: FS = 1.5
- Pseudo-Static (Seismic) Analysis with a seismic coefficient of 0.15: FS = 1.15
- Pseudo-Static (Seismic) Analysis with a seismic coefficient of 0.28: FS = 1.00

The selection of the factor of safety for a static condition is based on various published guidelines, including:

- National Research Council's Transportation Research Board's Special Report 247, "Landslides: Investigation and Mitigation", which states:

  > "The choice of appropriate safety factor for a given slope depends on a number of considerations, such as the quality of the data used in the analysis, which in turn depends on the quality of the subsurface investigations; laboratory and field testing; interpretation of field and laboratory data; quality of construction control; and in some cases, degree of completeness of information about the design problem. The engineer must also consider the probable consequences of failure. In most transportation situations, slope designs generally require safety factors in the range of 1.25 to 1.50. Higher factors may be required if slope movements have the potential for causing loss of human life or great economic loss or if there is considerable uncertainty regarding the pertinent design parameters, construction quality control, potential for seismic activity and so forth. Likewise, lower safety factors may be used if the engineer is confident of the accuracy of the input data and if good construction control may be relied upon. "

- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering, Section 1.2.3.2c (AREMA, 2009), which states:

  > "Generally a factor of safety of 1.5 is considered adequate, although, lower safety factors may be considered acceptable if the engineer performing the stability analysis has sufficient design data available for analysis. Higher safety factors are required when limited test and field data are available for use in the performance of the slope stability analysis."

- Naval Facilities Engineering Command (NAVFAC), Soil Mechanics, Design Manual 7.01, which requires that slopes have a safety factor of no less than 1.5 for reasonable assurance of stability in permanent or sustained loading conditions.

In summary, the aforementioned guidelines recommend selecting a factor of safety between 1.25 and 1.50 or higher depending on various factors. The value determined for this study was primarily influenced by two characteristics of the rail line.

First, as indicated in Section 1.3, the rail line is a critical facility and the only rail line connecting San Diego to points north. Consequently, its loss of use would have a severe impact on NCTD's, Amtrak's, and BNSF's ability to provide service. Second, the rail line can be considered a "lifeline facility". In the event of a natural or manmade disaster, the rail line may be one of the few alternatives to quickly get people and rescue, relief, and/or recovery supplies between Los Angeles and San Diego. These two reasons alone justify a higher factor of safety than a typical project.

Therefore, considering the rail line is a critical, lifeline facility and the associated consequences of failure, a 1.5 factor of safety is established as the appropriate value to evaluate slope stability. A factor of safety higher than 1.5 was considered to be conservative as some of the unknowns which would call for increasing the FS are accounted for in other inputs to the analysis, such as soil strengths.

Discussions supporting the selection of the minimum factors of safety for the pseudo-static (seismic) analyses and the seismic coefficients are presented in Section 3.5.

3.3 Soil Properties

The soil properties used in the analysis consisted of soil strength parameters and unit weights that are based on: 1) laboratory testing from the Geotechnical Study (Leighton, 2001a) and follow up subsurface explorations; 2) field observations during the Eighth Street Emergency Repair, Project 1 and Project 2 construction activities; and 3) engineering judgment. The soil properties used in this analysis are consistent with the Geotechnical Report (Leighton, 2003).

A summary of the assigned soil strength parameters for each geologic unit used in the slope stability analysis is provided in Table 1, below. Based on laboratory test data, the average moist unit weight used in the analyses for the fill soils, beach deposits, Bay Point and Delmar Formations was 125 pounds per cubic foot (pcf), while 110 pcf was used for the landslide materials (Leighton, 2003).

Table 1 Soil Strength Parameters for Slope Stability Analysis			
Material	Unit Weight (pcf)	Friction Angle, (degrees)	Cohesion, (psf)
Fill Soils	125	32	100
Bay Point Formation	125	36	200
Delmar Formation	125	36	300
(within +/- 5° horizontal)	125	25	150
Landslide Materials	110	18	50
Beach Deposits	125	30	0

The overall stability of the slope is significantly affected by the strength of the Delmar Formation. While testing an intact block of the Delmar Formation would yield relatively higher strength parameters, the use of such strength in the slope stability analysis would show no failures occurring on the bluff. As a majority of the bluff has experienced numerous failures, the use of intact strength values is not appropriate. As presented in the 2003 Supplemental Geotechnical Evaluation, a comparison of the average peak and residual strength data indicates that the Delmar Formation experiences significant strength loss once the cementation between the soil grains is broken. Similarly, when joints and fractures develop within the unit from both weathering and tectonic influences, the loss of contact can greatly reduce or eliminate the strength across the break. In addition, the geometry or steepness of the bluff induces a state of tension behind the crest and at times in the middle of the slope face. When the tensile strength of the materials is exceeded, cracks form. These zones of tension tend to expand during earthquakes, leading to additional areas where reduced strengths and higher water pressures are appropriate for use in the analyses. For these reasons, lower bound strength parameters were assigned to the Delmar Formation (i.e., the strength parameters presented on Figure 4, friction angle of 36 degrees and cohesion of 300 pounds per square foot, psf). To account for the presence of sheared siltstone and claystone beds, strength parameters similar to the average residual values of fine-grained Delmar Formation samples (i.e., friction angle of 25 degrees and cohesion of 150 psf) were assigned to this material within 5 degrees of horizontal.

Note that as verification of the soil properties used in the analyses, the 2003 Supplemental Geotechnical Evaluation analyzed two existing landslides on the bluff to back calculate the strength parameters prior to failure (determining what strength parameters generated a factor of safety of approximately 1.0, which corresponds to the moment of failure). The failures analyzed included a block fall at MP 244.47 (Station 1529+60), and a wedge failure at MP 245.27 (Station 1488+85). The results of the back calculation analysis, as presented in the 2003 Supplemental Geotechnical Evaluation, indicate that the selected soil strength parameters for the Delmar Formation appropriately model bluff failures.

3.4 Results of Analysis for Static Conditions

Circular and block failure surfaces were considered in the analysis for static conditions with and without a train surcharge loading. Typically, block or wedge surfaces represent the probable failure for a natural bluff condition while a circular surface represents the probable failure for a fill slope condition. In modeling the train surcharge, a uniform strip load of 3,000 pounds per square foot (psf) was applied across a width of 5 feet. This is considered equivalent to typical stresses under a 50,000 pound train axle load (AREMA, 2003, Practical Guide to Railway Engineering, Section 4.3.3,), which was considered to be an appropriate surcharge loading for this section of the LOSSAN corridor.

Table 2, below, presents a summary of the results for the stability analysis of the existing conditions under both static scenarios (static and static with a train surcharge load). The results of this analysis indicate that 8 of the 21 sections analyzed have a factor of safety less than 1.5 for a static condition with a train surcharge load and a modeled ground water profile as discussed in Section 3.1. The computer program Slope/W calculation plots for the analyses are presented in Appendix B, Slope Stability Analyses.

Table 2 Summary of Results for Static Scenarios				
Section	Station Location	Factor of Safety (FS)		
		Without Surcharge	With Surcharge	Notes
A-A'	1545+00	1.81	1.81	
A1-A1'	1540+05	1.68	1.52	1997 Shear Pins
A2-A2'	1539+21	1.17	1.17	
B-B'	1529+70	1.38	1.33	
C-C'	1529+00	1.36	1.31	
D-D'	1527+45	1.55	1.54	Trough Area
E-E'	1520+95	1.53	1.49	Trough Area
F-F'	1518+72	2.00	2.00	
F1-F1'	1517+06	1.58	1.58	
G-G'	1516+73	1.53	1.52	
G1-G1'	1513+05	1.61	1.61	
G2-G2'	1512+45	1.57	1.57	
G3-G3'	1514+08	1.47	1.47	1978 Shear Pins
H-H'	1512+15	1.52	1.51	
I-I'	1511+56	1.79	1.79	
J-J'	1490+90	1.48	1.48	Landslide area
K-K'	1487+20	1.63	1.59	
K1-K1'	1485+15	1.39	1.39	Landslide area
L-L'	1482+25	1.56	1.56	Anderson Canyon
M-M'	1480+29	1.64	1.64	
M1-M1'	1479+82	1.49	1.49	Landslide area

3.5 Results of Analysis for Pseudo-Static (Seismic) Conditions

Consistent with the 2003 Supplemental Geotechnical Evaluation analysis, an evaluation the bluff stability during the event of a major earthquake on a regional active fault was performed using a seismic slope stability or pseudo-static analysis, as defined by California Division of Mines and Geology in Special Publication 117. For this analysis, two values of the ground motion parameters or seismic coefficients (0.15 and 0.28) were considered.

The seismic coefficient (k_H) of 0.15 was selected based on the range presented by Seed as indicated in California Division of Mines and Geology Special Publication 117 – Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 1997).

According to Seed, a seismic coefficient range of 0.10 to 0.15 corresponds to maximum earthquake magnitudes of M6.5 to M8.25. Although the maximum moment magnitude of the Rose Canyon Fault Zone is considered to be M7.0 by the California Department of Transportation (Caltrans), the upper bound of the seismic coefficient range, $k_H = 0.15$, was elected for the analysis based on the location of the site relative to the Rose Canyon Fault and the standard of practice in Southern California. It should be noted, that for the seismic coefficient of 0.15, the minimum acceptable pseudo-static factor of safety of 1.15, as recommended by Seed (CDMG, 1997), was used to assess bluff stability. This analysis is most appropriate for conditions that may occur during a minor to moderate seismic event.

The higher seismic coefficient, 0.28, is equal to one-half the deterministic peak horizontal ground motion. The peak horizontal ground motion assigned to the site using Caltrans maps is 0.55g (Mualchin, L., 1996). Accordingly, the seismic coefficient was calculated to be 0.28. For the higher seismic coefficient, $k_H = 0.28$, the minimum acceptable pseudo-static factor of safety of 1.0, as specified by Caltrans (Caltrans, 1999), was used to assess bluff stability. It should be noted that use of a higher seismic coefficient, 0.28, is in general agreement with the recently published recommendations for evaluating steep slopes during major seismic events (Ashford and Sitar, 2002).

Table 3 presents a summary of the results for the pseudo-static stability analysis. Results of the slope stability analysis for pseudo static (seismic) conditions indicate that 7 of the 21 cross sections analyzed have a factor of safety less than the minimum acceptable parameter for a seismic coefficient of 0.15 (i.e., FS=1.15), and 15 of the 21 cross sections analyzed were less than the minimum acceptable parameter for a seismic coefficient of 0.28 (i.e., FS=1.00). For the purposes of prioritizing areas to stabilize, the Caltrans (1999) methodology, traditionally used for transportation facilities, using the higher seismic coefficient of 0.28 was selected. This decision was based on the slightly higher conservatism of this method, and the fact that an M7.2 earthquake on the Rose Canyon Fault would be a major seismic event.

Table 3
Summary of Results for Pseudo-Static Analysis

		Factor of Safety	
Section	Station Location	$k_H = 0.15$*	$k_H = 0.28$**
A-A'	1545+00	1.36	1.10
A1-A1'	1540+05	1.30	1.06
A2-A2'	1539+21	1.01	0.93
B-B'	1529+70	1.09	0.92
C-C'	1529+00	1.10	0.91
D-D'	1527+45	1.21	0.97
E-E'	1520+95	1.15	0.93
F-F'	1518+72	1.43	1.12
F1-F1'	1517+06	1.15	0.95
G-G'	1516+73	1.11	0.93
G1-G1'	1513+05	1.22	0.99
G2-G2'	1512+45	1.21	0.98
G3-G3'	1514+08	1.07	0.86
H-H'	1512+15	1.15	0.94
I-I'	1511+56	1.33	1.07
J-J'	1490+90	1.17	0.97
K-K'	1487+20	1.25	1.02
K1-K1'	1485+15	1.10	0.91
L-L'	1482+25	1.17	0.96
M-M'	1480+29	1.27	1.03
M1-M1'	1479+82	1.16	0.96

* Minimum acceptable parameter, factor of safety: 1.15
** Minimum acceptable parameter, factor of safety: 1.0

3.6 Slope Stability Summary

As previously noted, the factor of safety for static slope stability (with surcharge) is considered to be the primary design criteria. The factor of safety for pseudo static (seismic) condition ($k_H = 0.28$) is the secondary design criteria.

Table 4 presents a summary of the cross sections that have one or more factors of safety less than the minimum acceptable parameters defined previously. Therefore, these are the areas that should be considered the first priority for stabilization. As previously noted, the impacts of additional bluff retreat were not included in the slope stability analysis.

Table 4
Summary of Sections with Factors of Safety Below Acceptable Parameters

Section	Station Location	Factor of Safety (FS)	
		Static with Surcharge	Pseudo-Static $k_H = 0.28$
A2-A2'	1539+21	1.17	0.93
B-B'	1529+70	1.33	0.92
C-C'	1529+00	1.31	0.91
*D-D'	1527+45	1.54	0.97
*E-E'	1520+95	1.49	0.93
F1-F1'	1517+06	1.58	0.95
G-G'	1516+73	1.52	0.93
G1-G1'	1513+05	1.61	0.99
G2-G2'	1512+45	1.57	0.98
G3-G3'	1514+08	1.47	0.86
H-H'	1512+15	1.51	0.94
J-J'	1490+90	1.48	0.97
K1-K1'	1485+15	1.39	0.91
L-L'	1482+25	1.56	0.96
M1-M1'	1479+82	1.49	0.96

* Cross-Section D-D' and E-E' are located in the "Trough" area of the project. Additional "typical" failures in this area will actually remove overburden which drives slightly lower factors of safety. As a result, these areas are not included in areas prioritized for stabilization.

4.0 STABILIZATION AREAS

Based on the slope stability analyses presented in Section 3, nine "Stabilization Areas" have been identified. In general, the limits of the stabilization areas were determined based on the slope stability analysis (areas having less than the minimum acceptable parameters or factor of safety) and similar geotechnical and topographic conditions. The Stabilization Areas are numbered from 1 to 9 in order from north to south. In addition, each area is assigned an Implementation Ranking (IR) number that ranges from 1 to 7 to assist in prioritizing the areas for construction (i.e., IR = 1 being the higher priority) based on evaluation of the calculated factor of safety values.

4.1 Stabilization Area 1 (SA-1)

Station 1539+40 to Station 1538+85 (Length: approximately 55 feet)
Area between Project 2 construction SN-7S and SN-3
Low Static and Pseudo-Static FS, based on Cross Section A2-A2'
IR = 1

4.2 Stabilization Area 2 (SA-2)

Station 1530+85 to Station 1528+80 (Length: approximately 205 feet)
Area north of 11th Street and south of an existing soil cement buttress
Low Static and Pseudo-Static FS, based on Cross Sections B-B' and C-C'
IR = 2

4.3 Stabilization Area 3 (SA-3)

Station 1518+55 to Station 1516+57 (Length: approximately 198 feet)
Area north of Project 2 construction SN-6
Low Pseudo-Static FS, based on Cross Sections F1-F1' and G-G'
IR = 6

4.4 Stabilization Area 4 (SA-4)

Station 1514+55 to Station 1513+20 (Length: approximately 135 feet)
Area defined by the shear pins installed in 1978 south of Eighth Street Emergency Repair
Low Static and Pseudo-Static FS, based on Cross Section G3-G3'
IR = 3

4.5 Stabilization Area 5 (SA-5)

Station 1512+65 to Station 1511+75 (Length: approximately 90 feet)
Area north of Sherrie Lane
Low Pseudo-Static FS, based on Cross Section H-H'
IR = 6

4.6 Stabilization Area 6 (North: SA-6N and South: SA-6S)

North: SA-6N; Station 1494+40 to Station 1492+40 (Length: approximately 200 feet)
Low Pseudo-Static FS, based on Cross Section 20-20' from Project 2
IR = 7

South: SA-6S; Station 1491+45 to Station 1487+40 (Length: approximately 405 feet)
Low Static and Pseudo-Static FS, based on Cross Sections J-J', 21-21' and 22-22' from Project 2
IR = 4

4.7 Stabilization Area 7 (SA-7)

Station 1485+80 to Station 1484+80 (Length: approximately 100 feet)
Area is immediately north of Project 2 construction SN-8
Low Static and Pseudo-Static FS, based on Cross Section K1-K1'
IR = 3

4.8 Stabilization Area 8 (SA-8)

Station 1483+55 to Station 1482+20 (Length: approximately 135 feet)
Area west of the mainline track at Anderson Canyon defined by a large gravity sea wall,
Low EQ Pseudo-Static, based on Cross Sections L-L' and 24-24' from Project 2
IR = 5

4.9 Stabilization Area 9 (SA-9)

Station 1481+00 to Station 1479+40 (Length: approximately 160 feet)
Area south of Anderson Canyon
Low Static and Pseudo-Static FS, based on Cross Section M1-M1'
IR = 4

4.10 Priority of Stabilization Areas

Based on the implementation ranking above, Table 5 below presents a summary of the stabilization areas in order of improvement or repair priority.

Table 5 Implementation Ranking of Stabilization Areas		
IR Number	Area	Length of Stabilization (feet)
1	SA-1	55
2	SA-2	205
3	SA-4	135
3	SA-7	100
4	SA-6S	405
4	SA-9	160
5	SA-8	135
6	SA-3	198
6	SA-5	90
7	SA-6N	200
	TOTAL	1,683

5.0 REVIEW AND ANALYSIS OF CONCEPTUAL REPAIR ALTERNATIVES

Consistent with the Geotechnical Report (Leighton, 2003), the first step in determining the site specific stabilization alternatives for the subject areas is a review of the conceptual repair alternatives (i.e. stabilization measures) presented in the Geotechnical Study (Leighton, 2001a).

The conceptual repair alternatives as presented in the Geotechnical Study (Leighton, 2001a) included: 1) maintenance and repair of existing facilities; 2) stabilization at the bluff toe; 3) stabilization of the bluff face; 4) stabilization of the bluff top; 5) drainage improvements; and/or 6) groundwater reduction. Selection of the appropriate mitigation alternative is highly dependent upon the site specific stabilization problem at each of the Stabilization Areas and will likely include a combination of the methods.

Factors that were considered during the evaluation of the conceptual repair alternatives included the need to: 1) preserve track bed support for +/- 20 years; 2) provide for a minimum factor of safety; 3) maintain uninterrupted rail service; and 4) preserve natural bluff areas.

As a result of this review, it has been determined that several of the conceptual repair measures do not adequately meet the needs of the project as they do not provide the minimum recommended factor of safety or they will not be effective over the entire 20-year design life. Therefore, alternatives that do not meet the goals of the project have been dismissed from further consideration.

5.1 Repair of Existing Facilities

The Geotechnical Study (Leighton, 2001a) identified a number of existing facilities at the site that are in need of repair and/or ongoing maintenance, including storm drain outlets and existing sea walls. Repair of some of the drainage facilities was conducted as part of Project 1. Repair of sea walls will be considered where they can be utilized in conjunction with other stabilization methods or where they can be utilized to meet the project goals. Monitoring of all existing improvements will also be performed as part of on-going maintenance. It should be noted that the repair of the existing facilities alone does not improve bluff stability to acceptable levels.

5.2 Stabilization at Bluff Toe

Stabilization at the bluff toe should be considered in areas where slope stability analysis indicates low factors of safety at the base of the bluff and where improvements at the bluff toe will increase the factor of safety. Methods for stabilization include: 1) wooden or concrete sea walls; 2) steel piles and wood lagging walls; 3) soil cement buttress;

4) rock revetments; and 5) beach replenishment. Temporary toe protection, such as beach replenishment, were not taken into consideration as an effective measure that meets the project goals and therefore, were not recommended as part of any stabilization alternative. The other bluff toe stabilization options were considered where appropriate. In general, however, toe protection alone does not meet the goals of the project based on the slope stability analysis, but may help to reduce the expected rate of bluff retreat.

5.3 Stabilization of Bluff Face

Stabilization of the bluff face can be considered where factors of safety indicate adequate lateral support is not present or where additional erosion or failures will move the bluff face landward. Stabilization can be accomplished through slope grading or the use of pipe and board retaining walls.

Slope grading can be used to stabilize the bluff face and re-establish eroded and failed areas. Slope grading would generally consist of the placement of compacted fill soils on the face of the slope to provide additional lateral support and/or flatten localized over-steepened areas. Removal of material from existing slope failures could also be performed as part of slope grading. In areas where a conventional 2:1 horizontal to vertical slope cannot be constructed because of space limitations, the slope grading can incorporate steeper gradients through the use of geogrid reinforcement, or soil cement.

Slope grading alternatives can be designed to meet the project goals. With the goal to minimize disturbance of natural bluffs, the use of slope grading should be limited to areas of existing manufactured slopes. Slope grading would also likely include removals of existing compressible or disturbed material to provide a width of fill soils that is sufficient to achieve the desired factor of safety. These removals may encroach on the trackbed support and require the use of temporary shoring or the disruption of rail service.

Pipe and board retaining walls are generally considered a surficial repair and do not meet the goals of the project to improve overall gross stability of the slope. This option may, however, be applicable for repair of shallow surficial slope failures, repair of localized erosional areas or the retention of plantable soil on a steeper than 2:1 (horizontal to vertical) slope or on a soil cement slope.

5.4 Bluff Top Stabilization

Where the top edge of the bluff within 10 feet of the track has an inadequate factor of safety, additional bluff top stabilization is recommended. In general bluff top stabilization can be accomplished through the installation of a soldier pile wall system within the right-of-way or, in localized areas, it can be accomplished by a system of soil nails installed through the bluff face.

Soldier piles can also incorporate, as needed, tiebacks and grade beams. In addition, if the tops of the soldier piles become exposed over time, lagging can be added to modify the system through the recommended lifetime. Exposed areas can be "rock scaped" as desired to match the surroundings. This option would involve little, if any, disruption of rail operation.

Soil nails can also be considered for stabilization of the bluff top. However, in areas mantled by surficial or disturbed soil deposits, soil nail installation would require disturbance of the natural bluff face and would likely increase erosion and/or require the use of a hard facing to be placed on the bluff face. In these types of areas, the use of soil nails is not being considered because it does not meet the project objectives. However, in localized areas of dense bedrock, the amount of disturbance caused or the extent of hard facing required would be substantially reduced and soil nails may provide an effective solution.

5.5 Drainage Improvements

Drainage improvements were recommended in the Geotechnical Study (Leighton, 2001a) to reduce erosion of the bluff face and infiltration of water into the subsurface soils. Project 1 incorporated both surface and subsurface drainage improvements. Therefore, additional drainage improvements in other areas are not being considered as part of Project 3 except for back drains or surface drainage (i.e., area drains or drainage swales) within future graded areas.

6.0 RECOMMENDED STABILIZATION ALTERNATIVES

The following describes each of the stabilization areas, discusses the existing site specific conditions in each stabilization area, and recommends stabilization alternatives.

6.1 Stabilization Area 1 (SA-1)

Implementation Ranking: 1
Location: Station 1539+40 to Station 1538+85
Total Length: 55 feet
Length Recommended for Stabilization: 55 feet

This area is located between the two previously stabilized areas completed for Project 2 (i.e., Stabilization No. 7S and Stabilization No. 3). The edge of bluff is roughly 38 feet west of the track centerline at Station 1539+21 (Section A2-A2') with an elevation of approximately 46 feet mean sea level (msl). The bluff face is natural and near vertical for the upper portion. The calculated factors of safety for the pseudo-static conditions are below the minimum acceptable criteria and stabilization is recommended.

One option that will provide the necessary stabilization would be the construction of a soldier pile wall system similar to the existing adjacent stabilizations. This alternative could be constructed from the bluff top within the right-of-way with little disruption of rail operations, if constructed and night similar to Project 2. The soldier pile wall system could be buried. Based on the near vertical bluff face, another option could be installation of soil nails on the bluff face. However, the installation of soil nails will likely require work outside of the right-of-way and on the beach. In addition, the soil nails may disturb the natural bluff face and require a permanent concrete surface facing.

6.2 Stabilization Area 2 (SA-2)

Implementation Ranking: 2
Location: Station 1530+85 to Station 1528+80
Total Length: 205 feet
Length Recommended for Stabilization: 205 feet

The area is located north of 11th Street and immediately south of the existing Soil Cement buttress stabilization. The Geotechnical Report (Leighton, 2003) identified this area as a relatively low priority area (Sections 13-13' and 14-14'), and subsequently was not apart of the Project 2 construction. The bluff face in this area is natural and near vertical for the upper portion. The recent slope stability analyses of the bluff from Station

1530+85 to 1528+80 (Sections B-B' and C-C') indicate that the calculated factors of safety for static, train surcharge and pseudo-static analysis condition fall below the acceptable criterion.

Considering the natural topography of the bluff and the dense exposed bluff face, stabilization measures between Stations 1530+85 and 1528+80 could consist of a soldier pile retaining system or an embedded soil nail on the bluff face or soil nails with a facing.

The soldier pile wall system can be constructed within the right-of-way with minimal disturbance to rail operations. The use of soil nails can also be considered; however, it would require work outside the right-of-way and on the beach. In addition, the work is likely to cause additional disturbance of the natural bluff areas. Slope disturbance would probably result in some increased erosion, but this could be reduced by use of a bluff facing in conjunction with soil nail system.

6.3 Stabilization Area 3 (SA-3)

Implementation Ranking: 6
Location: Station 1518+55 to Station 1516+57
Total Length: 198 feet
Length Recommended for Stabilization: 198 feet

This area is located north of a previously stabilized area completed for Project 2 (i.e. SN-6). The slope stability analyses for this portion of the bluff (Sections F1-F1' and G-G') indicate that the calculated factors of safety for the pseudo-static conditions are below the acceptable criterion.

Given the existing steep bluff face, stabilization measures could consist of either a continuation of the soldier pile retaining system (installed as part of Project 2) or embedded soil nails. The soldier pile wall system can be constructed within the right-of-way with minimal disruption to rail operations.

With regard to soil nails, the installation can be done with little impacts to rail operations, but would likely require work outside the right-of-way and on the beach. Soil nail installation will likely result in additional disturbance of the natural bluff area, an increase in slope erosion, and require the use of a facing on the slope.

6.4 Stabilization Area 4 (SA-4)

Implementation Ranking: 3
Location: Station 1514+55 to Station 1513+20
Total Length: 135 feet
Length Recommended for Stabilization: 135 feet

This area is located south of an existing stabilization identified as the Eighth Street Emergency Repair constructed in 2001. This area was previously stabilized in 1978 with 18-inch diameter shear pins reinforced with two 115 pound rails at 5 foot centers with an approximate depth of 32 feet. The slope stability analyses for this portion of the bluff (Section G3-G3') indicate that the calculated factors of safety for the static and pseudo-static conditions are below the acceptable criterion.

It is recommended that the stabilization be accomplished by the installation of a soldier pile wall system or constructing a soil cement buttress. The soldier pile wall system can be constructed within the right-of-way with minimal disruption to rail operations.

The area could also be reconstructed as a soil cement buttress. This option would require removal of a portion of the existing fill soils and necessitate temporary excavations along the track. Temporary excavations would require the use of shoring or a disruption of rail operations. Slope grading of this area will have some impacts to the adjacent areas of natural bluff.

6.5 Stabilization Area 5 (SA-5)

Implementation Ranking: 6
Location: Station 1512+45 to Station 1511+75
Total Length: 90 feet
Length Recommended for Stabilization: 90 feet

This area is located immediately south of proposed SA-4 (discussed above) and north of Sherrie Lane. The edge of bluff is roughly 45 feet west of the track centerline at Station 1512+15 (Section H-H') with an elevation of approximately 60 feet mean sea level (msl). The bluff face is natural and near vertical at the mid to lower portion of the bluff. The slope stability analyses for this portion of the bluff (Sections G2-G2' and H-H') indicate that the calculated factors of safety for the pseudo-static conditions are below the acceptable criterion.

Given the existing steep bluff face, stabilization measures consist of either the soldier pile retaining system or embedded soil nails. The soldier pile wall system can be constructed within the right-of-way with minimal disruption to rail operations.

With regard to soil nails, the installation can be done with little impacts to rail operations, but would likely require work outside the right-of-way and on the beach. Soil nail installation will likely result in additional disturbance of the natural bluff area, an increase in slope erosion, and require the use of a facing on the slope.

6.6 Stabilization Area 6 (North: SA-6N and South: SA-6S)

North: SA-6N
Implementation Ranking: 7
Location: Station 1494+40 to Station 1492+40
Total Length: 200 feet
Length Recommended for Stabilization: 200 feet

South: SA-6S
Implementation Ranking: 4
Location: Station 1491+45 to Station 1487+40
Total Length: 405 feet
Length Recommended for Stabilization: 405 feet

The areas include several new landslides since the 2003 mapping was performed, and incorporates a previously recommended stabilization area, which was identified in the Geotechnical Report (Leighton, 2003). The northern area also includes a fill area and large retaining wall at the toe of the bluff with storm drain outlet located at approximately Station 1493+77. Note that the existing storm drain outlet pipe should be considered in the design. The bluff face is natural, excluding the fill slope are behind the retaining wall, and is near vertical at the upper portion of the bluff. The slope stability analyses for this portion of the bluff (Sections 20-20', J-J', 21-21', and 22-22') indicate that the calculated factors of safety for the static and pseudo-static conditions are below the acceptable criterion.

For both the northern and southern areas, a soldier pile wall system can be constructed within the right-of-way with minimal disturbance to rail operations. The use of soil nails can also be considered for the natural bluff areas; however, it would require work outside the right-of-way and on the beach and the work will likely to cause additional disturbance of the natural bluff areas. Note that a bluff facing in conjunction with soil nail system would be needed.

Another possible alternative for the northern fill area with the large retaining wall could be soil cement buttress. This option would require removal of the existing fill soils and underlying native soil. Temporary excavations may require the use of shoring or a disruption of rail operations. Slope grading of this area for the buttress will have some impacts to the adjacent areas of natural bluff.

6.7 Stabilization Area 7 (SA-7)

Implementation Ranking: 3
Location: Station 1485+80 to Station 1484+80
Total Length: 100 feet
Length Recommended for Stabilization: 100 feet

This area is located north of a previously stabilized area completed for Project 2 (i.e., SN-8). The edge of bluff is roughly 45 feet west of the track centerline at Station 1485+15 (Section K1-K1') with an elevation of approximately 56 feet mean sea level (msl). The bluff face is natural and near vertical at the upper portion of the bluff. The slope stability analyses for this portion of the bluff (Section K1-K1') indicate that the calculated factors of safety for the static and pseudo-static conditions are below the acceptable criterion.

Stabilization measures consist of either a continuation of the soldier pile retaining system (as installed for SN-8) or the installation of embedded soil nails. The soldier pile wall system can be constructed within the right-of-way with minimal disruption to rail operations.

The installation of soil nails can be done with little impact to rail operations, but would likely require work outside the right-of-way and on the beach. Soil nail installation will likely result in additional disturbance of the natural bluff area, an increase in slope erosion, and require the use of a facing on the slope.

6.8 Stabilization Area 8 (SA-8)

Implementation Ranking: 5
Location: Station 1483+55 to Station 1482+20
Total Length: 135 feet
Length Recommended for Stabilization: 135 feet

The area is located at Anderson Canyon west of the track and is underlain by fill with a large retaining wall at the toe of the bluff. Note that this area was a previously recommended stabilization area in the Geotechnical Report (Leighton, 2003), but was not incorporated in Project 2 construction. The slope stability analyses for this area (Sections 24-24' and L-L') indicate that the calculated factors of safety for the pseudo-static conditions are below the acceptable criterion.

Stabilization measures consist of either a continuation of the soldier pile retaining system (as installed for SN-8) or possibly soil cement buttress. The soldier pile wall system can be constructed within the right-of-way with minimal disruption to rail operations. Note that a soil cement buttress alternative would require temporary shoring or the disruption of rail operations during construction, and grading of this area for the buttress will have some impacts to the adjacent areas of natural bluff. Also, the large storm drain culvert crosses the alignment in this area and should be considered in the design of any stabilization.

6.9 Stabilization Area 9 (SA-9)

Implementation Ranking: 4
Location: Station 1481+00 to Station 1479+40
Total Length: 160 feet
Length Recommended for Stabilization: 160 feet

In general, this area has new landslides since the 2003 mapping was performed. The edge of bluff is roughly 35 feet west of the track centerline at Station 1479+82 (Section M1-M1') with an elevation of approximately 52 feet mean sea level (msl). The bluff face is natural with a general slope inclination of 1 to 1 (horizontal to vertical). The slope stability analyses for this portion of the bluff (Section M1-M1') indicate that the calculated factors of safety for the pseudo-static condition is below the acceptable criterion.

Stabilization measures consist of either a soldier pile retaining system or possibly an embedded soil nails. The soldier pile wall system can be constructed within the right-of-way with minimal disruption to rail operations. The installation of soil nails can be done with little impacts to rail operations, but would likely require work outside the right-of-way and on the beach. Soil nail installation will likely result in additional disturbance of the natural bluff area, an increase in slope erosion, and require the use of a facing on the slope.

6.10 Summary of Stabilization Areas

Table 6 provides a summary of the currently recommended lengths for the stabilization areas.

Table 6
Recommended Stabilization Lengths

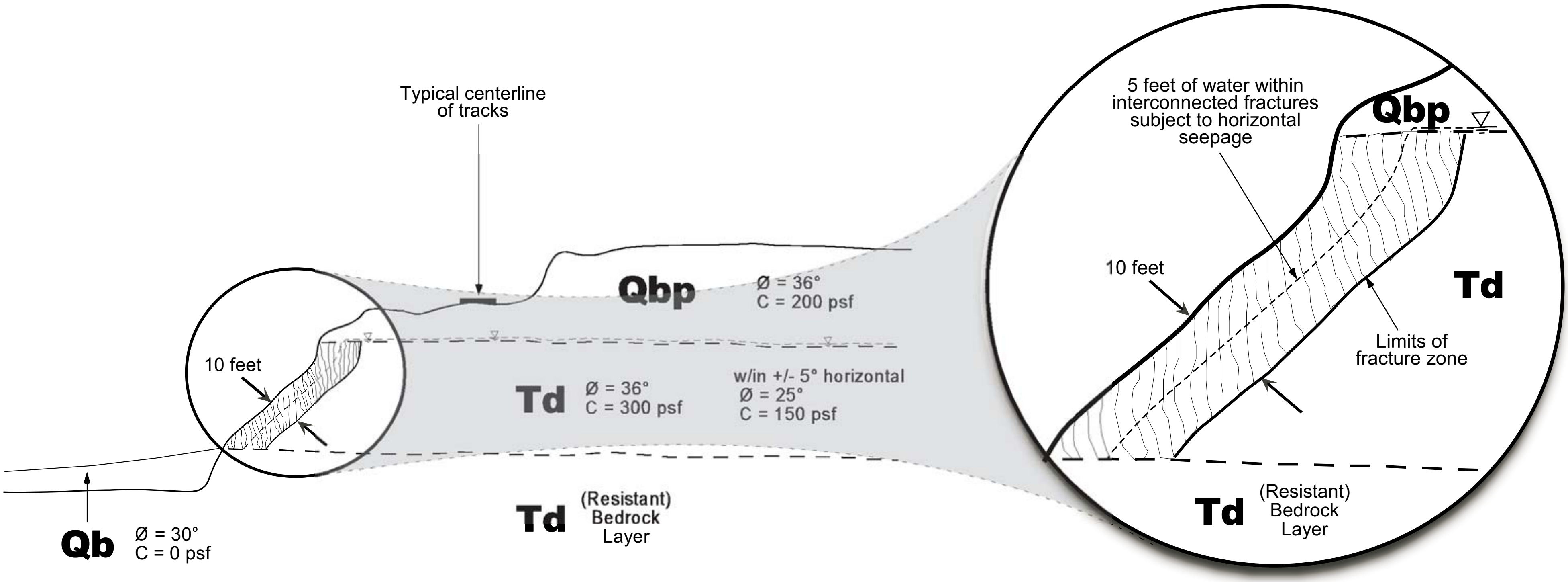
Area	Length of Stabilization (feet)
SA-1	55
SA-2	205
SA-3	198
SA-4	135
SA-5	90
SA-6N	200
SA-6S	405
SA-7	100
SA-8	135
SA-9	160
TOTAL	1,683

As presented in the table above, the current total recommended length of stabilization is 1,683 feet, or roughly 20 percent of the entire bluff length. The total length of the study area is 1.6 miles (8,450 feet).

7.0 CONCLUSIONS

In conclusion, the slope stability analysis included within this report demonstrates which areas of this critical link of the LOSSAN corridor do not currently meet the project criteria for factor of safety and where stabilization is warranted. The conceptual repair alternatives presented in Part 2 of the Geotechnical Study (2001a) have been further evaluated to define which alternatives meet the project needs and objectives. Those alternatives that meet the project needs have been considered in the stabilization alternatives presented in Section 6 of this report.

Through a substantial amount of additional slope stability analysis, areas have now been grouped into Stabilization Areas of like soil and geologic conditions which have then been prioritized with an Implementation Ranking Number based on the calculated factor of safety. This additional analysis has reaffirmed the need for stabilization within the areas defined by the Geotechnical Study (2001a). The analysis also provides for the selection of improvements to the areas with the greatest need at this time. Options for the repair/stabilization of the areas that meet the goals of this project have been reviewed and suggested on a site specific basis.



LEGEND

Qb Beach Deposit

Qbp Bay Point Formation

Td Delmar Formation

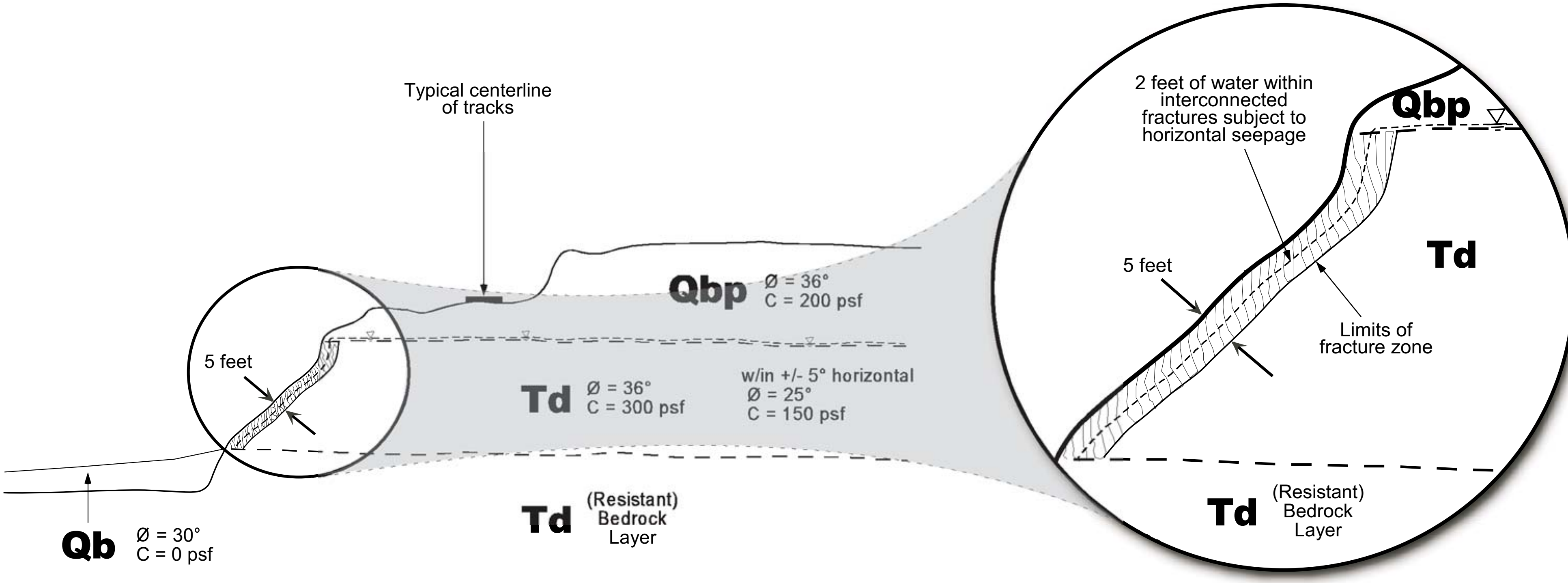
∇ Perched groundwater profile

SIMPLIFIED SLOPE STABILITY MODEL NORTH OF MP 245.21

Del Mar Bluffs Project
Del Mar, California

Project No.	602576-001
Scale	Not to scale
Engr./Geol.	WDO/MRS
Drafted By	KAM
Date	April 2010

Figure No. 2



LEGEND

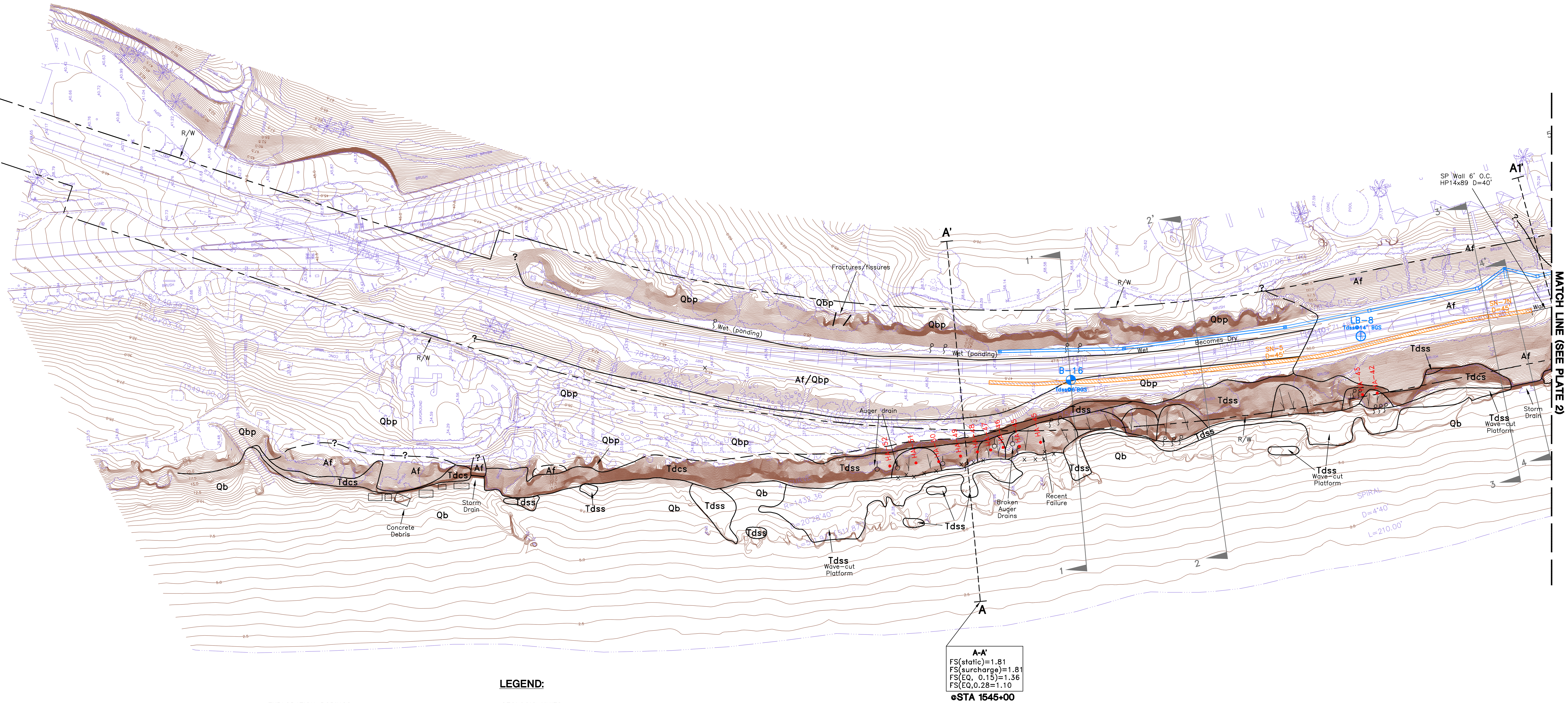
- **Qb** Beach Deposit
- **Qbp** Bay Point Formation
- **Td** Delmar Formation
- ⊻ Perched groundwater profile

SIMPLIFIED SLOPE STABILITY MODEL SOUTH OF MP 245.21

Del Mar Bluffs Project
Del Mar, California

Project No.	602576-001
Scale	Not to scale
Engr./Geol.	WDO/MRS
Drafted By	KAM
Date	April 2010

Figure No. 3



LEGEND:

EXPLORATION BORINGS:

PB-4 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)

B-3 — APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)

LB-5 — APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)

HSA-2 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

Af — UNDIFFERENTIATED FILL
Qb — BEACH DEPOSITS
Qbp — BAY POINT FORMATION
Tdss — DELMAR FORMATION (SANDSTONE FACIES)
Tdcs — DELMAR FORMATION (CLAYSTONE FACIES)

STABILIZATION SYMBOLS:

OLD STABILIZATION AREA

SN-1 to SN-9 — COMPLETED STABILIZATION NUMBER, PROJECT 2 (SWE, 2008)

APPROXIMATE LOCATION OF TRENCH DRAIN-PROJECT 1 BY DMJM (2003)

HA-2 • — APPROXIMATE LOCATION OF HYDRO AUGER, PROJECT 1 BY LEIGHTON & ASSOCIATES (2003)

GENERAL SYMBOLS:

AREA OF BOULDERS
WATER SEEPAGE
NEAR VERTICAL SCARP
FRACTURE
LANDSLIDE DEPOSIT
M1 — M1' GEOLOGIC CROSS-SECTION
21 — 21 GEOLOGIC CROSS-SECTION BY LEIGHTON & ASSOCIATES (2003)
NEW STABILIZATION AREA; SA-1 TO SA-9
R/W RIGHT-OF-WAY

REFERENCE: DAVID EVANS & ASSOCIATES, MAY 2009

N

0 40 80
SCALE FEET

PLATE 1	GEOTECHNICAL MAP DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010
	Drafted By: MAM	Checked By: P:\DRAFTING\602576\001\OF_2010-04-07\PLATES1-7-GEOLOGYMAPS.DWG (04-08-10 2:10:57PM) Plotted by: mmurphy

LEGEND:

EXPLORATION BORINGS:

- PB-4 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
- B-3 — APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)
- LB-5 — APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
- HSA-2 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

- Af — UNDIFFERENTIATED FILL
- Qb — BEACH DEPOSITS
- Qbp — BAY POINT FORMATION
- Tdss — DELMAR FORMATION (SANDSTONE FACIES)
- Tdcs — DELMAR FORMATION (CLAYSTONE FACIES)

STABILIZATION SYMBOLS:

- OLD STABILIZATION AREA
- SN-1 to SN-9 — COMPLETED STABILIZATION NUMBER, PROJECT 2 (SWE, 2008)
- APPROXIMATE LOCATION OF TRENCH DRAIN-PROJECT 1 BY DMJM (2003)
- HA-2 • — APPROXIMATE LOCATION OF HYDRO AUGER, PROJECT 1 BY LEIGHTON & ASSOCIATES (2003)

GENERAL SYMBOLS:

- AREA OF BOULDERS
- WATER SEEPAGE
- NEAR VERTICAL SCARP
- FRACTURE
- LANDSLIDE DEPOSIT
- M1 — M1' GEOLOGIC CROSS-SECTION
- 21 — 21 GEOLOGIC CROSS-SECTION BY LEIGHTON & ASSOCIATES (2003)
- NEW STABILIZATION AREA; SA-1 TO SA-9
- R/W RIGHT-OF-WAY

N

SCALE 0 40 80 FEET

REFERENCE: DAVID EVANS & ASSOCIATES, MAY 2009

PLATE 2	**GEOTECHNICAL MAP** DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010

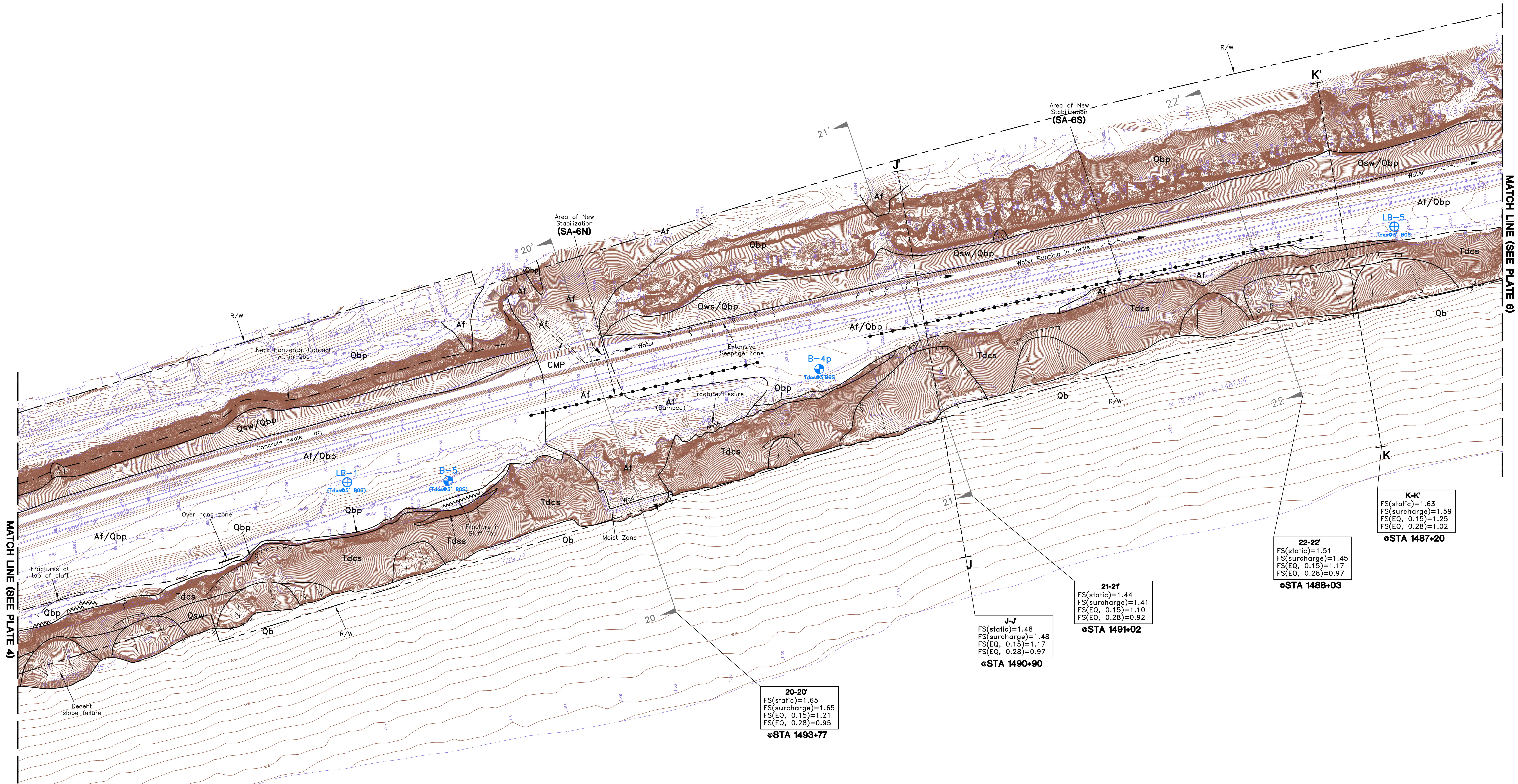
PLATE 3

GEOTECHNICAL MAP
DEL MAR BLUFFS PROJECT 3
DEL MAR, CALIFORNIA

Leighton

Proj: 602576-001	Eng/Geol: WDO/RCS
Scale: 1"=40'	Date: 04/2010


LEGEND:
EXPLORATION BORINGS:
PB-4 APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
B-3 APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)
LB-5 APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
HSA-2 APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)
GEOLOGIC UNITS:
Af UNDIFFERENTIATED FILL
Qb BEACH DEPOSITS
Qbp BAY POINT FORMATION
Tdss DELMAR FORMATION (SANDSTONE FACIES)
Tdcs DELMAR FORMATION (CLAYSTONE FACIES)
STABILIZATION SYMBOLS:
OLD STABILIZATION AREA
SN-1 to SN-9 COMPLETED STABILIZATION NUMBER, PROJECT 2 (SWE, 2008)
APPROXIMATE LOCATION OF TRENCH DRAIN-PROJECT 1 BY DMJM (2003)
HA-2 APPROXIMATE LOCATION OF HYDRO AUGER, PROJECT 1 BY LEIGHTON & ASSOCIATES (2003)
GENERAL SYMBOLS:
AREA OF BOULDERS
WATER SEEPAGE
NEAR VERTICAL SCARP
FRACTURE
LANDSLIDE DEPOSIT
M1 M1' GEOLOGIC CROSS-SECTION
21 21 GEOLOGIC CROSS-SECTION BY LEIGHTON & ASSOCIATES (2003)
NEW STABILIZATION AREA; SA-1 TO SA-9
R/W RIGHT-OF-WAY
MATCH LINE (SEE PLATE 5)
MATCH LINE (SEE PLATE 3)
6TH STREET
SHERRIE LANE
R/W
Area of New Stabilization (SA-5)
H'
I'
H
I
G1
G2
Zone of previous ponding (currently dry)
Af/Qbp
Qbp
Qsw/Qbp
Moist
(concrete swale)
Af (Dumped)
Af
(Storm drain)
Erosion blockage of swale
Abandoned Storm drain pipe
Vertical fissure/gullies
Old slope failure
Old slope failures
Large sandstone boulders
Active water flow from drain
Fractures at Bluff Top
Fracture at Bluff top
Area of erosion
Swale dry
LB-4 (Tdcs@14' BGS)
B-8 Tdcs@6'
B-9p Tdcs@9' BGS
LB-3 Tdss@10' BGS
Tdcs
Tdss
Qsw
Qb
19'
18'
G1-G1' FS(static)=1.61 FS(surcharge)=1.61 FS(EQ, 0.15)=1.22 FS(EQ, 0.28)=0.99 @STA 1513+05
G2-G2' FS(static)=1.57 FS(surcharge)=1.57 FS(EQ, 0.15)=1.21 FS(EQ, 0.28)=0.98 @STA 1512+45
H-H' FS(static)=1.52 FS(surcharge)=1.51 FS(EQ, 0.15)=1.15 FS(EQ, 0.28)=0.94 @STA 1512+15
I-I' FS(static)=1.79 FS(surcharge)=1.76 FS(EQ, 0.15)=1.33 FS(EQ, 0.28)=1.07 @STA 1511+56
N
0 40 80
SCALE FEET
REFERENCE: DAVID EVANS & ASSOCIATES, MAY 2009
PLATE 4
GEOTECHNICAL MAP
DEL MAR BLUFFS PROJECT 3
DEL MAR, CALIFORNIA
Leighton
Proj: 602576-001
Eng/Geol: WDO/RCS
Scale: 1"=40'
Date: 04/2010



LEGEND:

EXPLORATION BORINGS:

- PB-4 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
- B-3 — APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)
- LB-5 — APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
- HSA-2 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

- Af — UNDIFFERENTIATED FILL
- Qb — BEACH DEPOSITS
- Qbp — BAY POINT FORMATION
- Tdss — DELMAR FORMATION (SANDSTONE FACIES)
- Tdcs — DELMAR FORMATION (CLAYSTONE FACIES)

STABILIZATION SYMBOLS:

- OLD STABILIZATION AREA
- SN-1 to SN-9 — COMPLETED STABILIZATION NUMBER, PROJECT 2 (SWE, 2008)
- APPROXIMATE LOCATION OF TRENCH DRAIN-PROJECT 1 BY DMJM (2003)
- HA-2 • — APPROXIMATE LOCATION OF HYDRO AUGER, PROJECT 1 BY LEIGHTON & ASSOCIATES (2003)

GENERAL SYMBOLS:

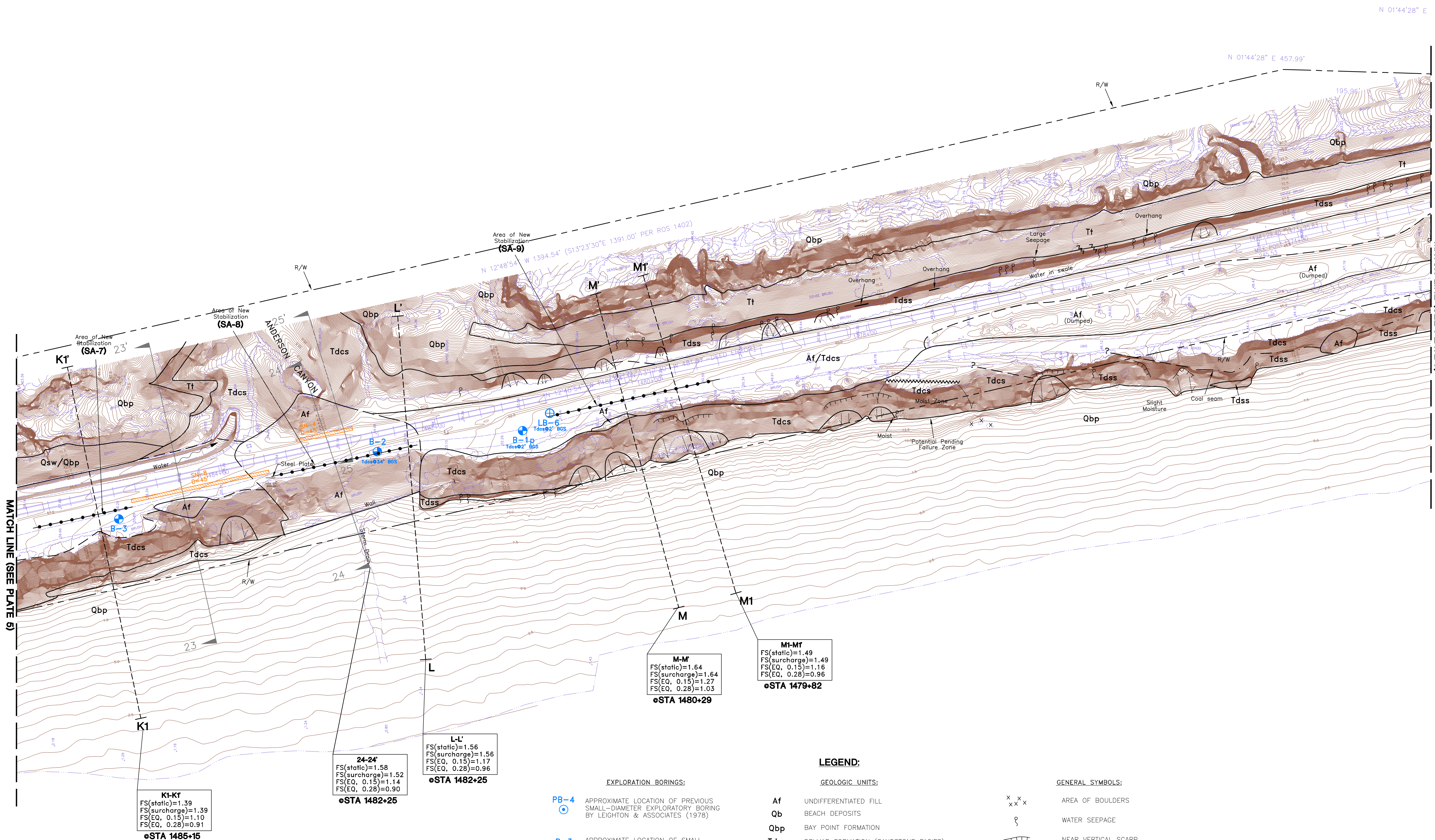
- AREA OF BOULDERS
- WATER SEEPAGE
- NEAR VERTICAL SCARP
- FRACTURE
- LANDSLIDE DEPOSIT
- M1 — M1' — GEOLOGIC CROSS-SECTION
- 21 — 21 — GEOLOGIC CROSS-SECTION BY LEIGHTON & ASSOCIATES (2003)
- NEW STABILIZATION AREA; SA-1 TO SA-9
- R/W RIGHT-OF-WAY

N

SCALE 0 40 80 FEET

REFERENCE: DAVID EVANS & ASSOCIATES, MAY 2009

PLATE 5	GEOTECHNICAL MAP DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010



LEGEND:

EXPLORATION BORINGS:

- PB-4 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
- B-3 — APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)
- LB-5 — APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
- HSA-2 — APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

- Af — UNDIFFERENTIATED FILL
- Qb — BEACH DEPOSITS
- Qbp — BAY POINT FORMATION
- Tdss — DELMAR FORMATION (SANDSTONE FACIES)
- Tdcs — DELMAR FORMATION (CLAYSTONE FACIES)

STABILIZATION SYMBOLS:

- OLD STABILIZATION AREA
- SN-1 to SN-9 — COMPLETED STABILIZATION NUMBER, PROJECT 2 (SWE, 2008)
- APPROXIMATE LOCATION OF TRENCH DRAIN-PROJECT 1 BY DMJM (2003)
- HA-2 — APPROXIMATE LOCATION OF HYDRO AUGER, PROJECT 1 BY LEIGHTON & ASSOCIATES (2003)

GENERAL SYMBOLS:

- AREA OF BOULDERS
- WATER SEEPAGE
- NEAR VERTICAL SCARP
- FRACTURE
- LANDSLIDE DEPOSIT
- M1 — M1' — GEOLOGIC CROSS-SECTION
- 21 — 21 — GEOLOGIC CROSS-SECTION BY LEIGHTON & ASSOCIATES (2003)
- NEW STABILIZATION AREA; SA-1 TO SA-9
- R/W RIGHT-OF-WAY

REFERENCE: DAVID EVANS & ASSOCIATES, MAY 2009

PLATE 6	GEOTECHNICAL MAP DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010

LEGEND:

EXPLORATION BORINGS:

PB–4	APPROXIMATE LOCATION OF PREVIOUS SMALL–DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
B–3	APPROXIMATE LOCATION OF SMALL–DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B–1p)
LB–5	APPROXIMATE LOCATION OF LARGE–DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
HSA–2	APPROXIMATE LOCATION OF PREVIOUS SMALL–DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

Af	UNDIFFERENTIATED FILL
Qb	BEACH DEPOSITS
Qbp	BAY POINT FORMATION
Tdss	DELMAR FORMATION (SANDSTONE FACIES)
Tdcs	DELMAR FORMATION (CLAYSTONE FACIES)

STABILIZATION SYMBOLS:

	OLD STABILIZATION AREA
SN–1 to SN–9	COMPLETED STABILIZATION NUMBER, PROJECT 2 (SWE, 2008)
	APPROXIMATE LOCATION OF TRENCH DRAIN–PROJECT 1 BY DMJM (2003)
HA–2 •	APPROXIMATE LOCATION OF HYDRO AUGER, PROJECT 1 BY LEIGHTON & ASSOCIATES (2003)

GENERAL SYMBOLS:

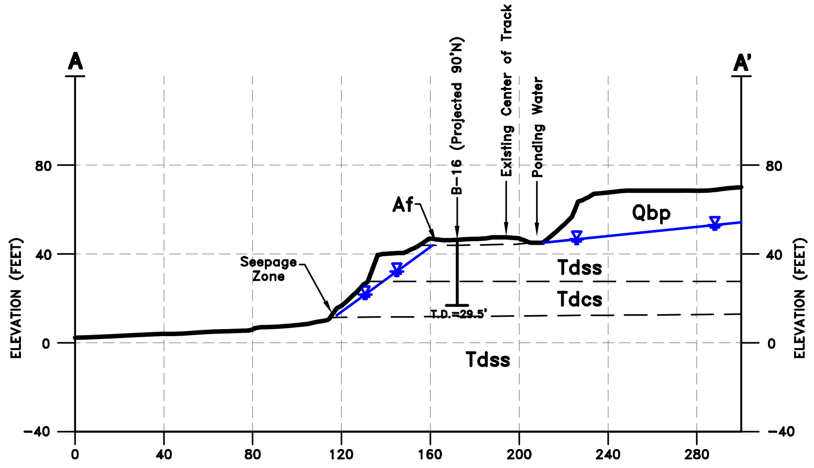
××××	AREA OF BOULDERS
	WATER SEEPAGE
	NEAR VERTICAL SCARP
	FRACTURE
	LANDSLIDE DEPOSIT
M1 – – – M1'	GEOLOGIC CROSS–SECTION
21 21	GEOLOGIC CROSS–SECTION BY LEIGHTON & ASSOCIATES (2003)
• • • • • • •	NEW STABILIZATION AREA; SA–1 TO SA–9
— – —	R/W RIGHT–OF–WAY

N

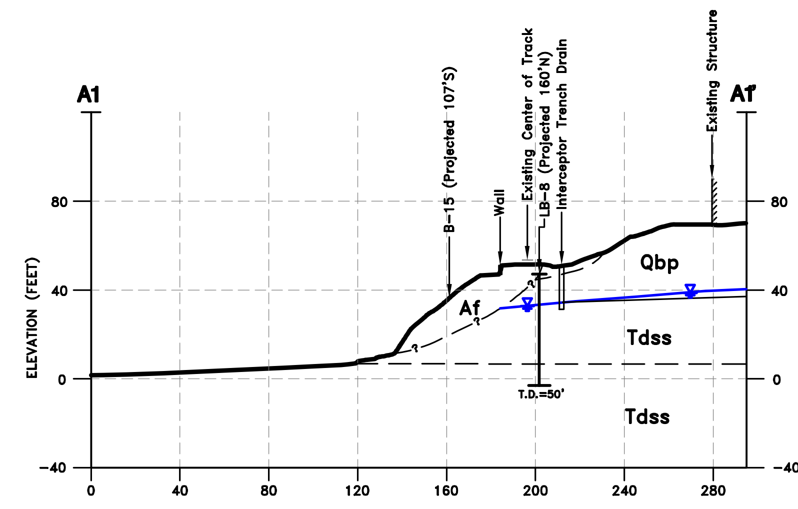
0 40 80
SCALE FEET

REFERENCE: DAVID EVANS & ASSOCIATES, MAY 2009

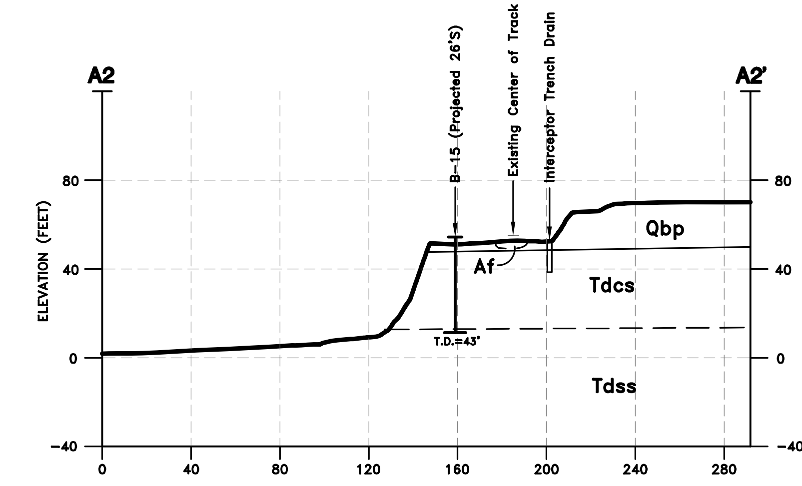
PLATE 7	GEOTECHNICAL MAP DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010



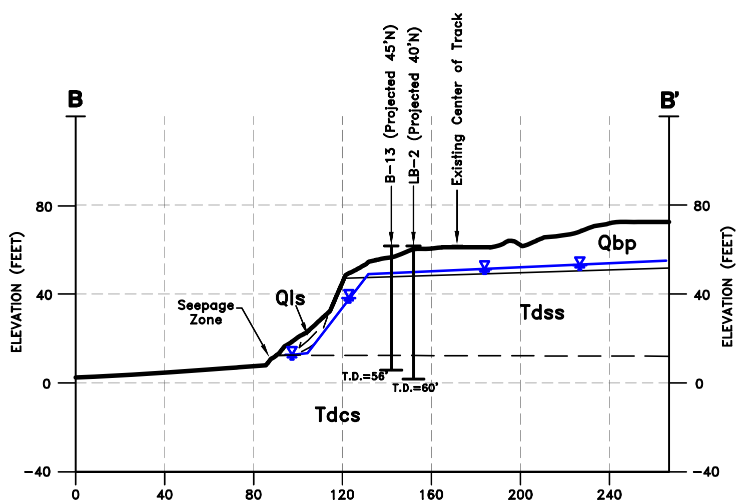
⊙ STATION 1545+00



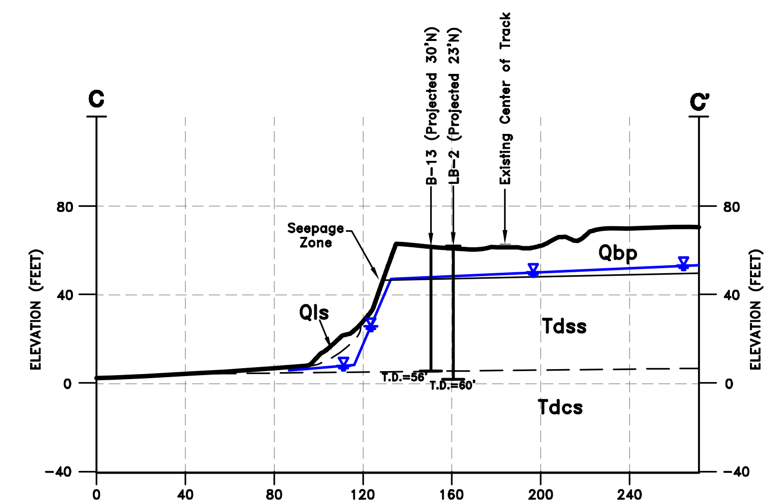
⊙ STATION 1540+05



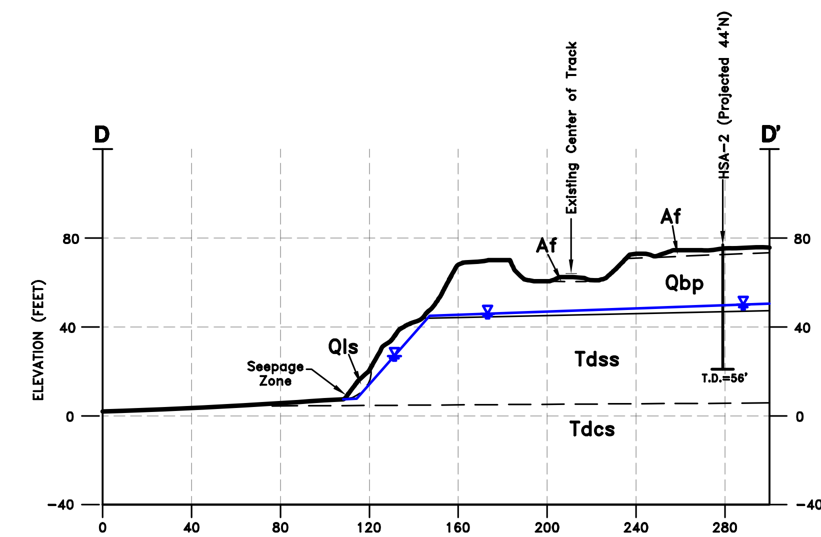
⊙ STATION 1539+21



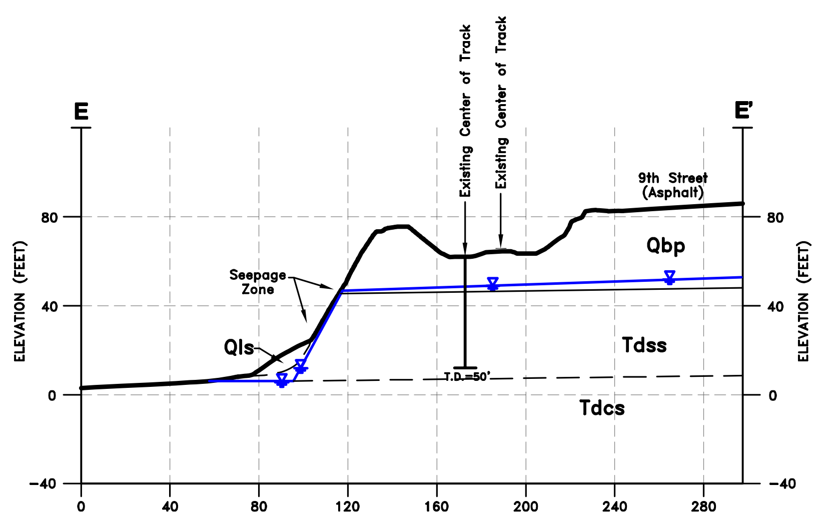
⊙ STATION 1529+70



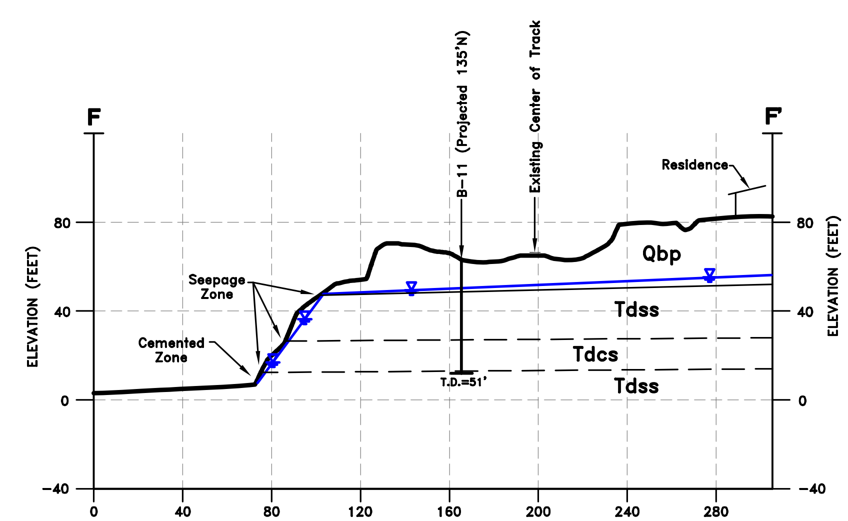
⊙ STATION 1529+00



⊙ STATION 1527+45



⊙ STATION 1520+95



⊙ STATION 1518+72

LEGEND:

EXPLORATION BORINGS:

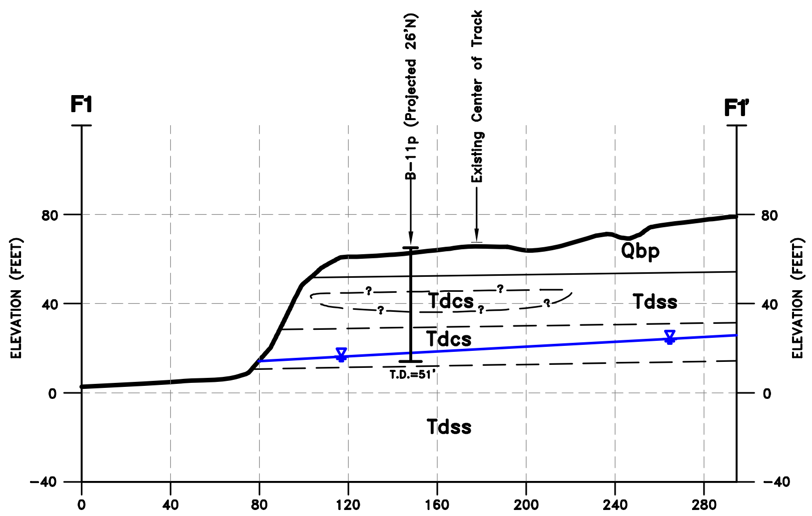
- **PB–4** — APPROXIMATE LOCATION OF PREVIOUS SMALL–DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
- **B–3** — APPROXIMATE LOCATION OF SMALL–DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B–1p)
- **LB–5** — APPROXIMATE LOCATION OF LARGE–DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
- **HSA–2** — APPROXIMATE LOCATION OF PREVIOUS SMALL–DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

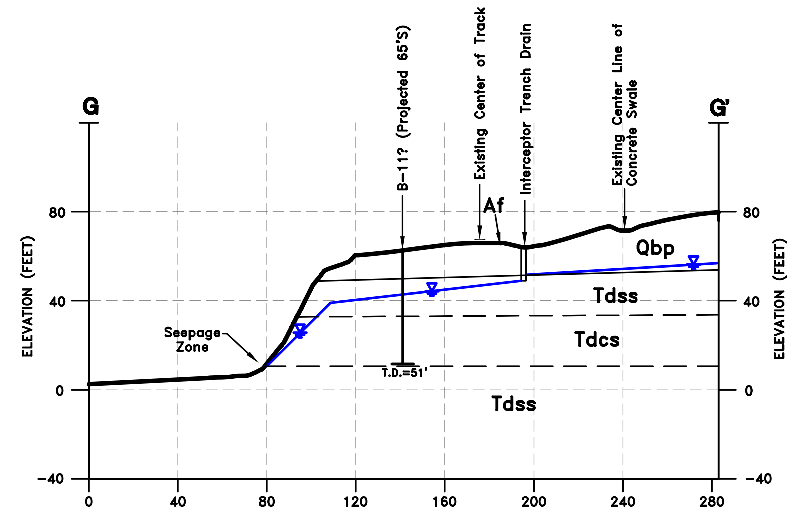
- **Af** — UNDIFFERENTIATED FILL
- **Qb** — BEACH DEPOSITS
- **Qbp** — BAY POINT FORMATION
- **Tdss** — DELMAR FORMATION (CLAYSTONE FACIES)
- **Tdcs** — DELMAR FORMATION (SANDSTONE FACIES)
- ASSUMED GROUND WATER ELEVATION

SCALE 0 40 80 FEET

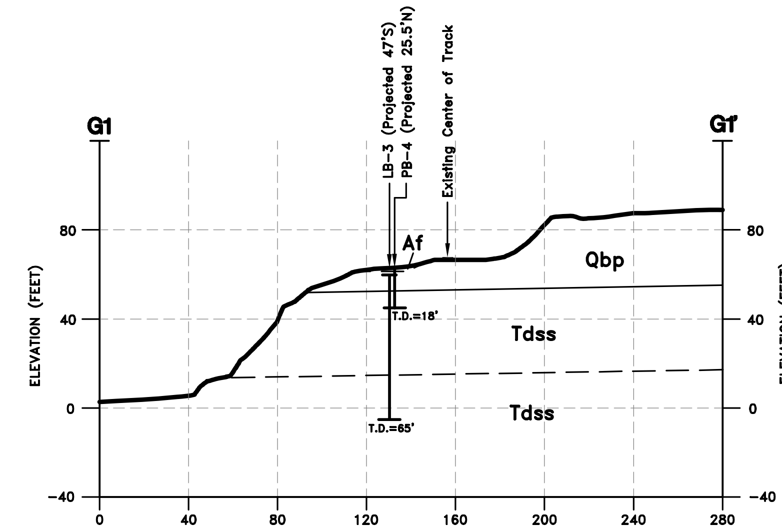
PLATE 8	GEOLOGICAL CROSS-SECTIONS DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010



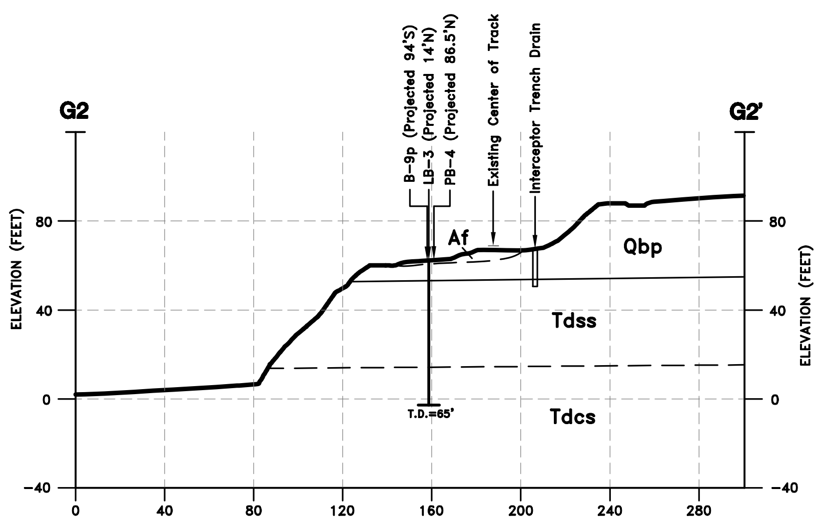
STATION 1517+06



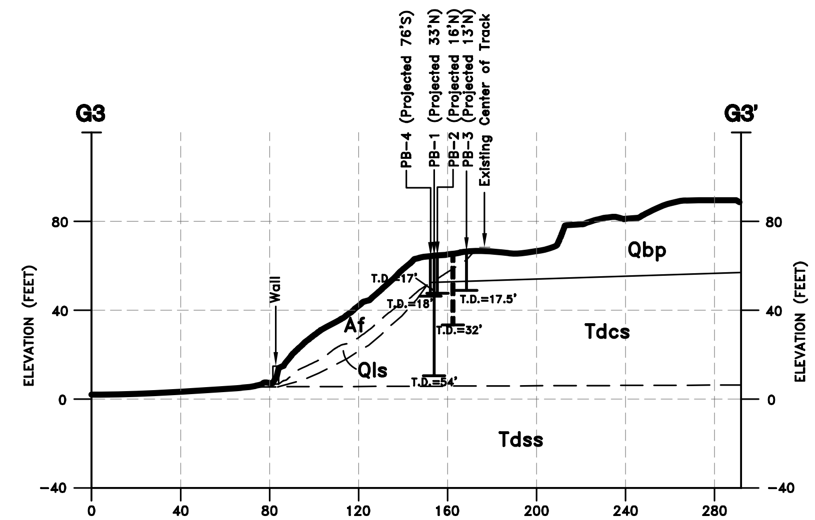
STATION 1516+73



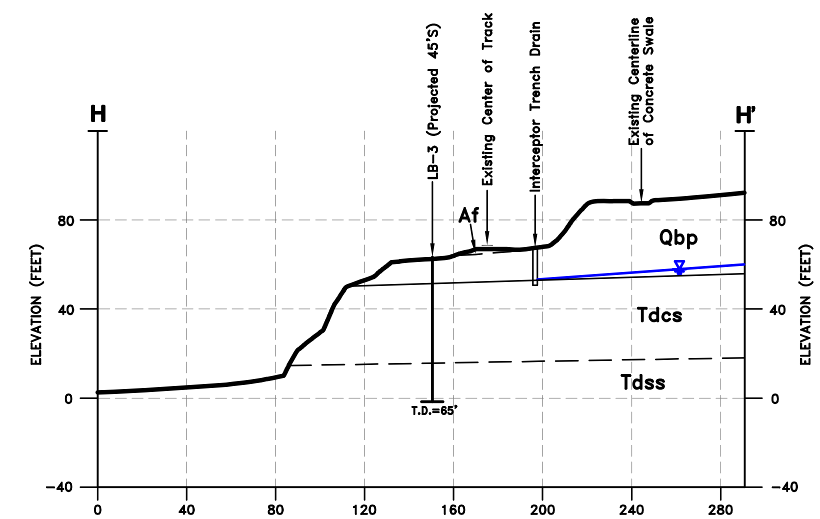
STATION 1513+05



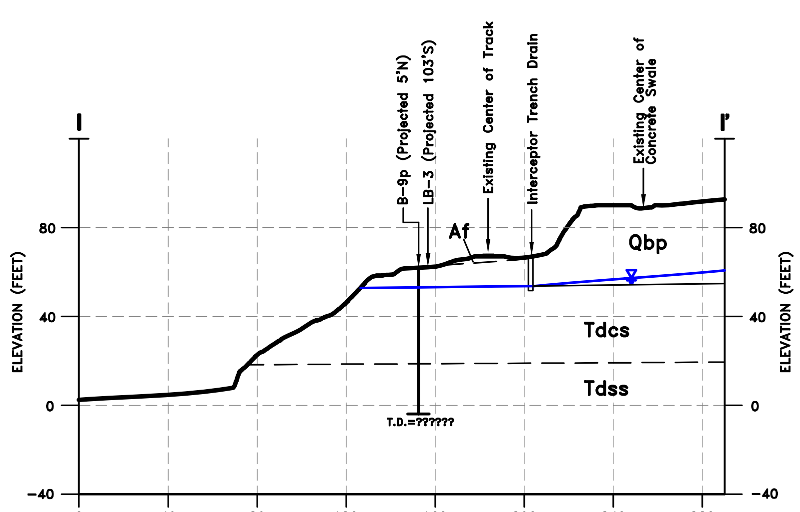
STATION 1512+45



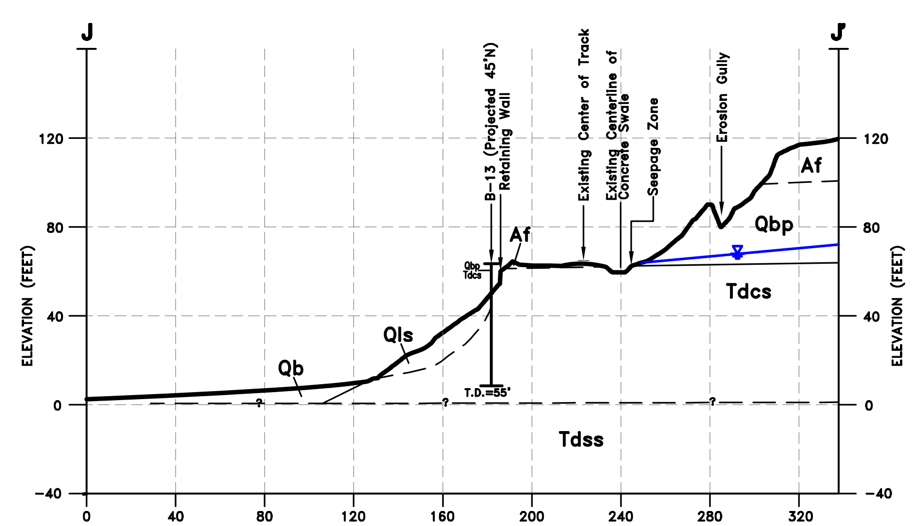
STATION 1514+08



STATION 1512+15



STATION 1511+56



STATION 1490+90

LEGEND:

EXPLORATION BORINGS:

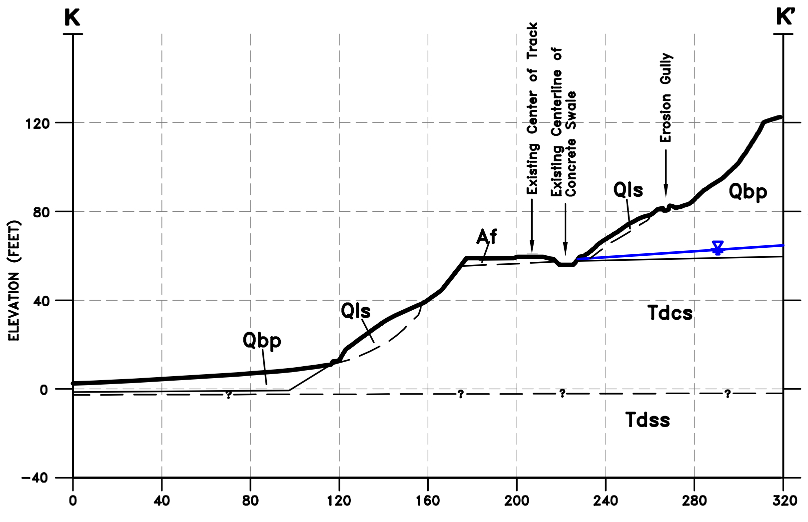
- **PB-4** APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
- **B-3** APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)
- **LB-5** APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
- **HSA-2** APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

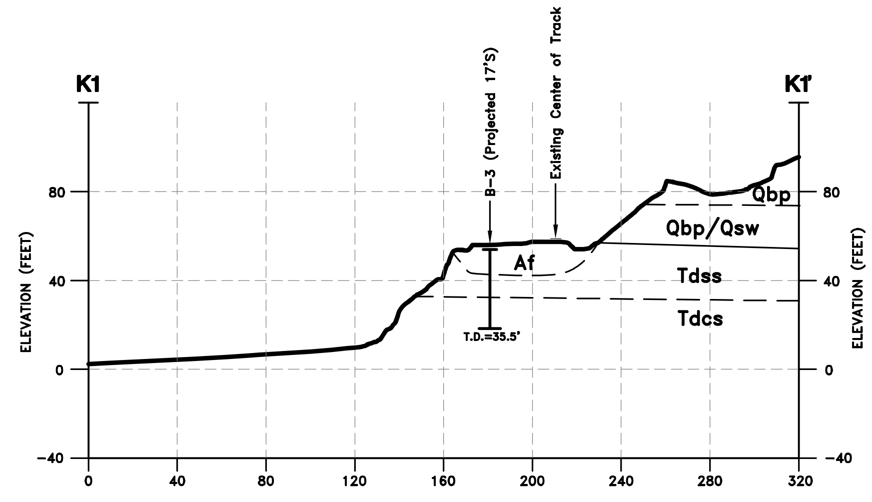
- **Af** UNDIFFERENTIATED FILL
- **Qb** BEACH DEPOSITS
- **Qbp** BAY POINT FORMATION
- **Tdss** DELMAR FORMATION (CLAYSTONE FACIES)
- **Tdcs** DELMAR FORMATION (SANDSTONE FACIES)
- ASSUMED GROUND WATER ELEVATION

SCALE 0 40 80 FEET

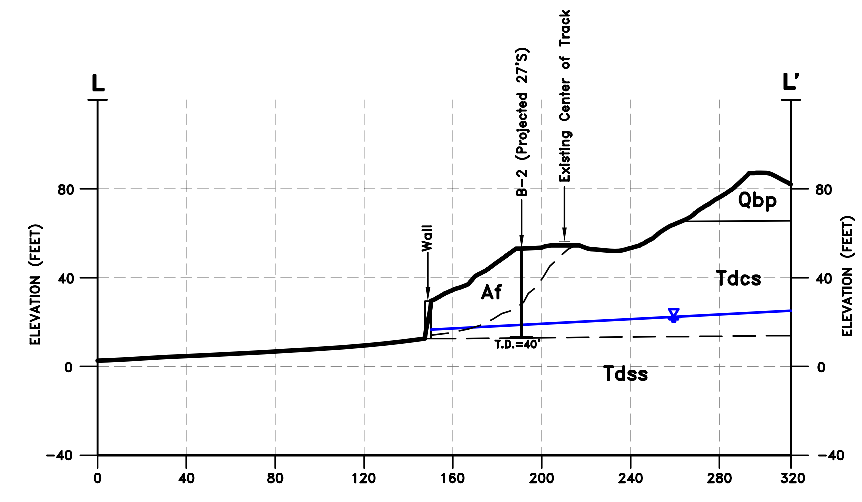
PLATE 9	GEOLOGICAL CROSS-SECTIONS DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010



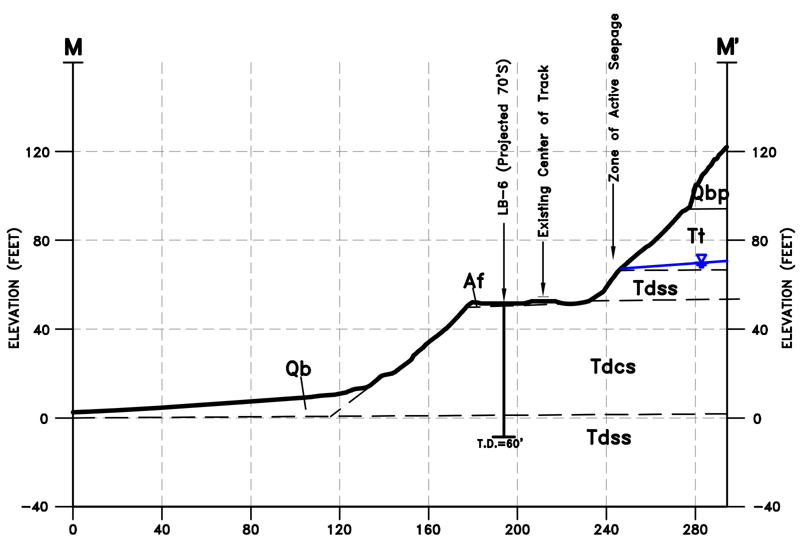
STATION 1487+20



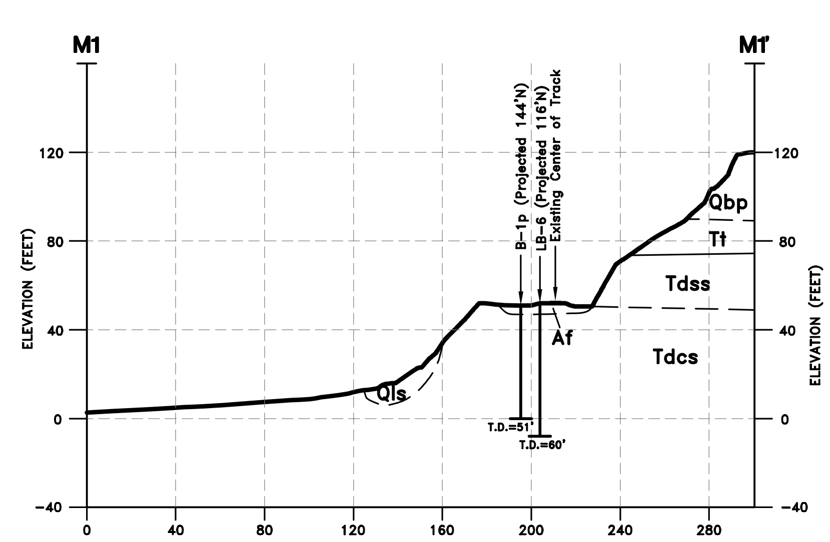
STATION 1485+15



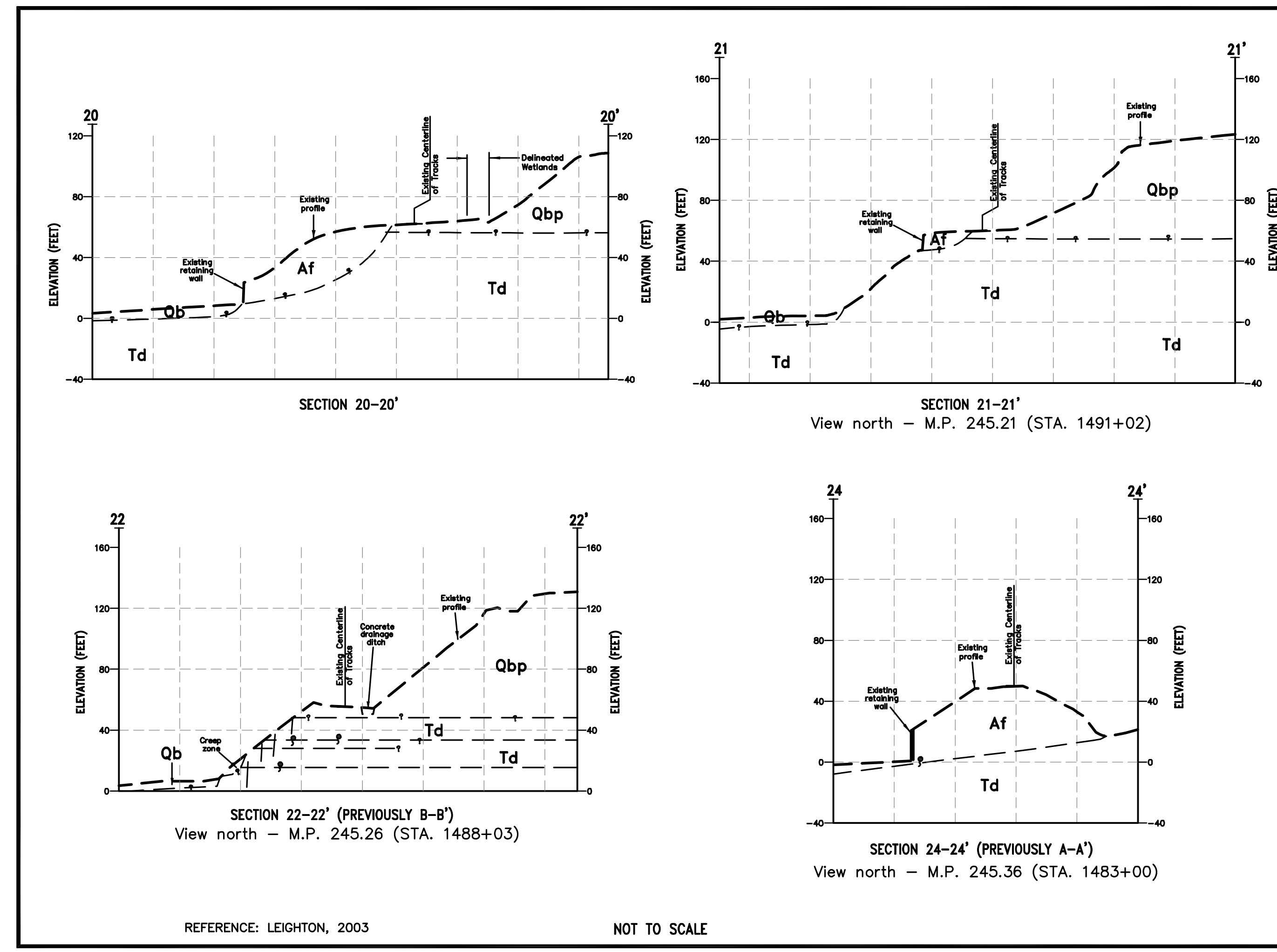
STATION 1482+25



STATION 1480+29



STATION 1479+82



LEGEND:

EXPLORATION BORINGS:

- **PB-4** APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORING BY LEIGHTON & ASSOCIATES (1978)
- **B-3** APPROXIMATE LOCATION OF SMALL-DIAMETER BORINGS BY MAH (1998). BORINGS CONVERTED TO PIEZOMETERS INDICATED BY SUFFIX P (B-1p)
- **LB-5** APPROXIMATE LOCATION OF LARGE-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2000, 2001)
- **HSA-2** APPROXIMATE LOCATION OF PREVIOUS SMALL-DIAMETER EXPLORATORY BORINGS BY LEIGHTON & ASSOCIATES (2002)

GEOLOGIC UNITS:

Af	UNDIFFERENTIATED FILL
Qb	BEACH DEPOSITS
Qbp	BAY POINT FORMATION
Tdss	DELMAR FORMATION (CLAYSTONE FACIES)
Tdcs	DELMAR FORMATION (SANDSTONE FACIES)
	ASSUMED GROUND WATER ELEVATION

PLATE 10	GEOLOGICAL CROSS-SECTIONS DEL MAR BLUFFS PROJECT 3 DEL MAR, CALIFORNIA	
Leighton	Proj: 602576-001	Eng/Geol: WDO/RCS
	Scale: 1"=40'	Date: 04/2010

APPENDIX A

REFERENCES

AREMA, 2003 American Railway Engineering and Maintenance-of-Way Association, Practical Guide to Railway Engineering, 2003

AREMA, 2009 American Railway Engineering and Maintenance-of-Way Association, Manual of Railway Engineering, 2009.

Ashford, Scott A., 1995a, Seismic Coefficients for Steep Slopes, Seventh International Conference on Soil Dynamics and Earthquake Engineering, Soil Dynamics and Earthquake Engineering VII, Computational Mechanics Publications, Southampton, 1995.

——————, 1995b, Topographic Amplification in Steep Slopes, Earthquake Geotechnical Engineering, A.A. Belkema Publishers, Rotterdam, Netherlands, 1995.

Ashford, Scott, A., Roth, L.A. Madsen S.L. and Anderson D.G., 1992, " F.S.=1.5 Is It Appropriate for Enbankment Design: Proceedings, Stabilization and Performance of Slopes and Embankment-11, ASCE Berkeley, California, June 1992.

Ashford, Scott A., and Sitar, Nicholas, 2002, Simplied Method for Evaluating Seismic Stability in Steep Slopes, Journal of Geotechnical and Geoenvironmnetal Engineering, Vol. 128, No. 2, February 1, 2001.

Blake, 2000, EQFAULT, Version 3.0.

California Department of Transportation, (Caltrans), 1996, Trenching and Shoring Manual, dated February 1990, Revised December 1996.

——————, 1999a, Memo to Designers 20-1, Seismic Design Methodology, January 1999.

——————, 1999b, Section 3-6, Guidelines for Foundation Investigations and Reports, Caltran Engineering Service Center, dated May 26, 1999.

California, Division of Mines and Geology (CDMG), 1996, Probabilistic Seismic Hazard Assessment for the State of California, Open File Report 96-08.

——————, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117, March 13, 1997.

David Evans and Associates, 2009, Topographic Map of Del Mar Bluff, Digital File, May 2009.

APPENDIX A (Continued)

Duncan, J.M., and Buchignani, A. L. (1975), An Engineering Manual for Slope Stability Studies, Geotechnical Engineering, Report, Department of Civil Engineering, University of California at Berkeley.

Hart, E.W., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning with Index to Special Study Zones Maps: Department of Conservation, Division of Mines and Geology, Special Publication 42.

International Conference of Building Officials, 1997, Uniform Building Code.

Jennings, C.W., 1994, Fault Map of California: Faults, Volcanoes, Thermal Springs, and Thermal Wells, California Division of Mines and Geology, Geologic Data Map No. 1, Scale 1:750,000.

Kennedy, M.P., 1975, Geology of the San Diego Metropolitan Area, California Section A-Western San Diego Metropolitan Area: California Division of Mines and Geology Bulletin 200, 38p.

Kuhn, G.G. and Shepard, F.P., 1984, Sea Cliffs, Beaches and Coastal Valleys: of San Diego County, 193pp.

Leighton and Associates, 1978, Preliminary Geotechnical Investigation of Landslides at Seventh and Thirteenth Streets, Del Mar Bluffs, Project No. 478008-1, dated January 30, 1978.

———, 2001a, Del Mar Bluffs Geotechnical Study (2001a), Part 1 - Geotechnical Evaluation (Volumes 1 and 2), Part 2 – Conceptual Repair Alternatives, Project No. 040151-001, dated January 31, 2001.

———, 2001b, Summary of Observations, Preliminary Assessment of Recent Failure, 7th – 8th Street, Del Mar, California, Project No. 040151-002, dated April 19, 2001

———, 2001c, Preliminary Recommendations for Emergency Slope Strengthening, Del Mar Bluffs, Between 7th and 8th Streets, Del Mar, California, Project No. 040151-004, dated April 27, 2001.

———, 2001d, Lateral Pier Analysis, Emergency Slope Strengthening, Del Mar Bluffs, Between 7th and 8th Streets, Del Mar, California, Project No. 040151-004, dated May 9, 2001.

———, 2002, Summary of Geotechnical Observations and Testing, Emergency Slope Stabilization at the 8th Street Bluff, Del Mar, California, March 12, 2002.

APPENDIX A (Continued)

———, 2003, Del Mar Bluffs Stabilization Project 2 – Preserving Trackbed Support, Supplemental Geotechnical Evaluation and Determination of Site Specific Conceptual Repair Alternatives, November 5, 2003.

Mualchin, L., 1996, A Technical Report to Accompany the Caltrans California Seismic Hazard Map, July 1996.

National Research Council, Transportation Research Board, 1996, Landslides: Investigation and Mitigation Special Report 247.

Naval Facilities Engineering Command (NAVFAC), 1986, Soil Mechanics, Design Manual 7.01, September, 1986.

Simon Wong Engineering (SWE), 2008, As-Built Project Plans, Del Mar Bluffs Stabilization, Project 2, June 10, 2008.

Southern California Chapter, American Public Works Association and Southern California Districts, Associated General Contractors, 1997, "Green Book" Standard Specifications for Public Works Construction with 1999 Supplement.

U.S. Army Corps of Engineers (USCOE), 1984, Shore Protection Manual, Volumes I-II, dated 1984.

———, 1995a, Engineering and Design, Coastal Geology, Engineer Manual 1110-2-1010 dated January 31, 1995.

———, 1995b, Engineering and Design, Design of Coastal Revetments Seawalls and Bulkheads, Engineer Manual 1110-2-16H, dated June 30, 1995.

———, 1996, Reconnaissance Report, Encinitas Shoreline, San Diego County, California, dated March 1996.

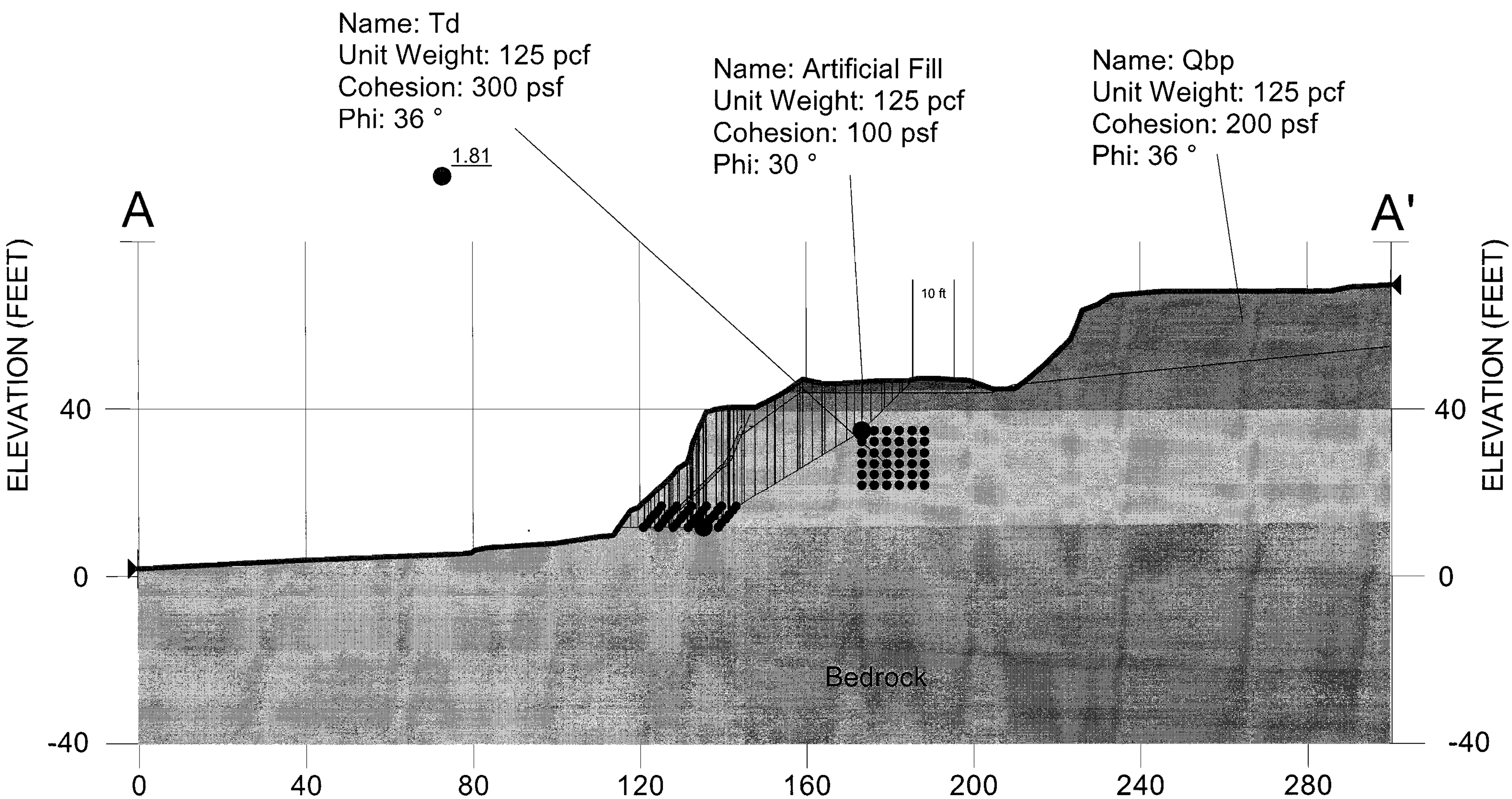
<u>Aerial Photographs</u>

Agency	Date	Flight No.	Photo Nos.
USDA	1953	AXN-8M	82, 83 and 84
GTI	November 26, 1969	16	37a BU, 37b BU
GTI	July 29, 1990		41-44, 49-51, 54-60 (oblique photos)
GTI	January 28, 1999		86-95 (oblique photos)

CROSS SECTION A-A'

Del Mar Bluffs Cross Section A-A'
Slope Stability Analysis
File Name: Section A-A' Static B.gsz
Analysis Method: Spencer

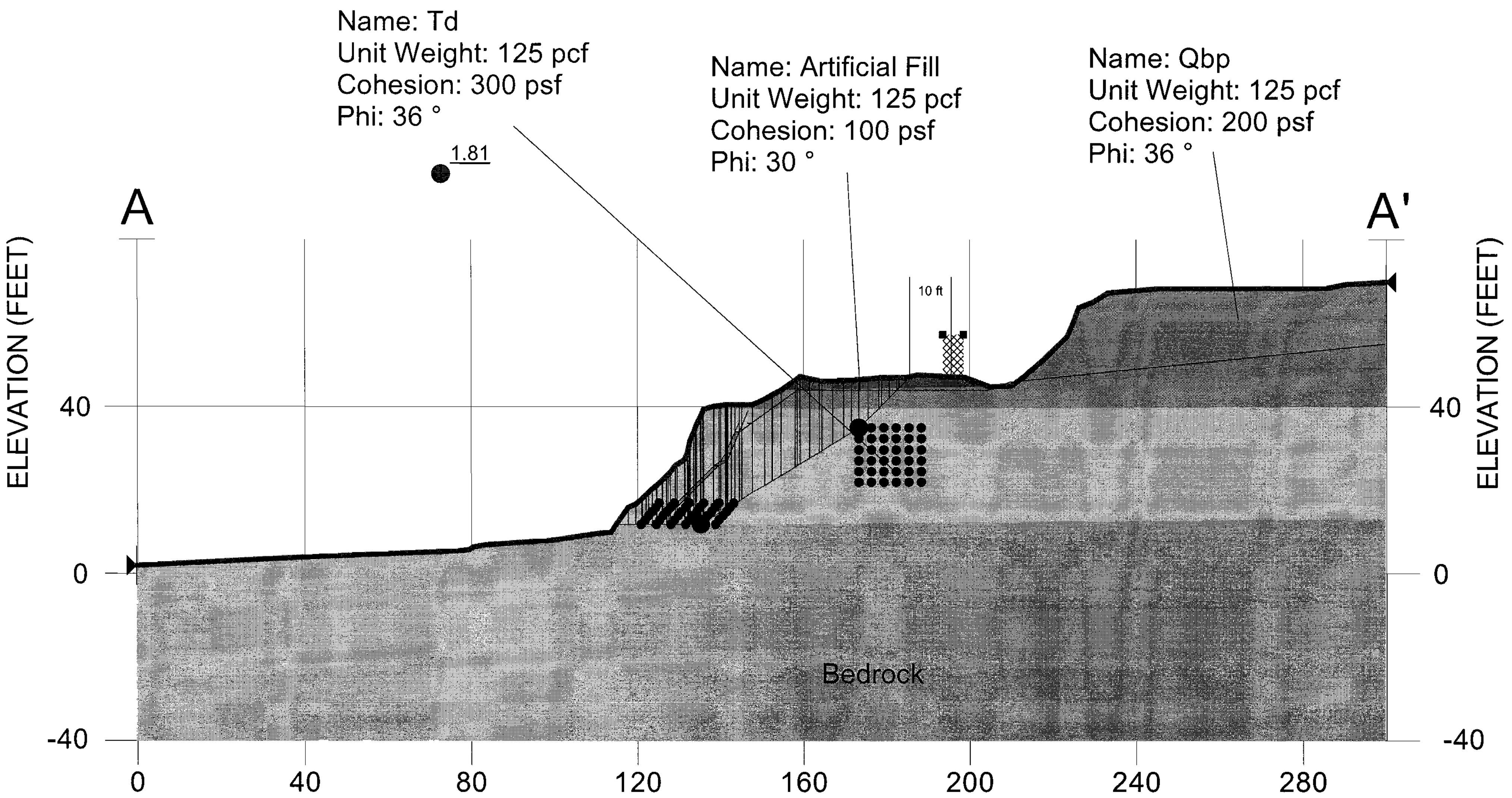
Factor of Safety: 1.81



Del Mar Bluffs Cross Section A-A'
Slope Stability Analysis
File Name: Section A-A' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.81

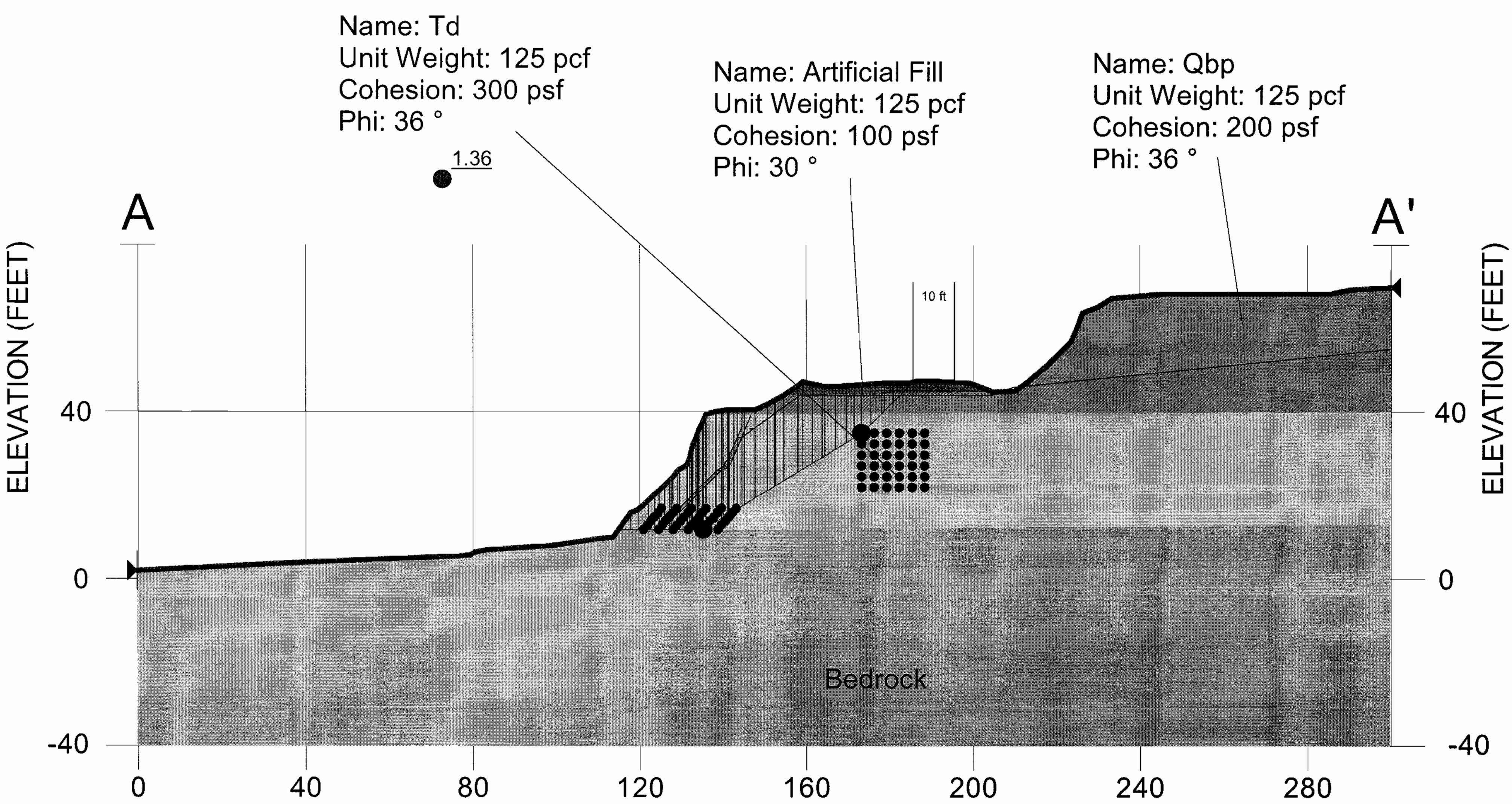
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section A-A'
Slope Stability Analysis
File Name: Section A-A' Pseudostatic B Kh= 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.36

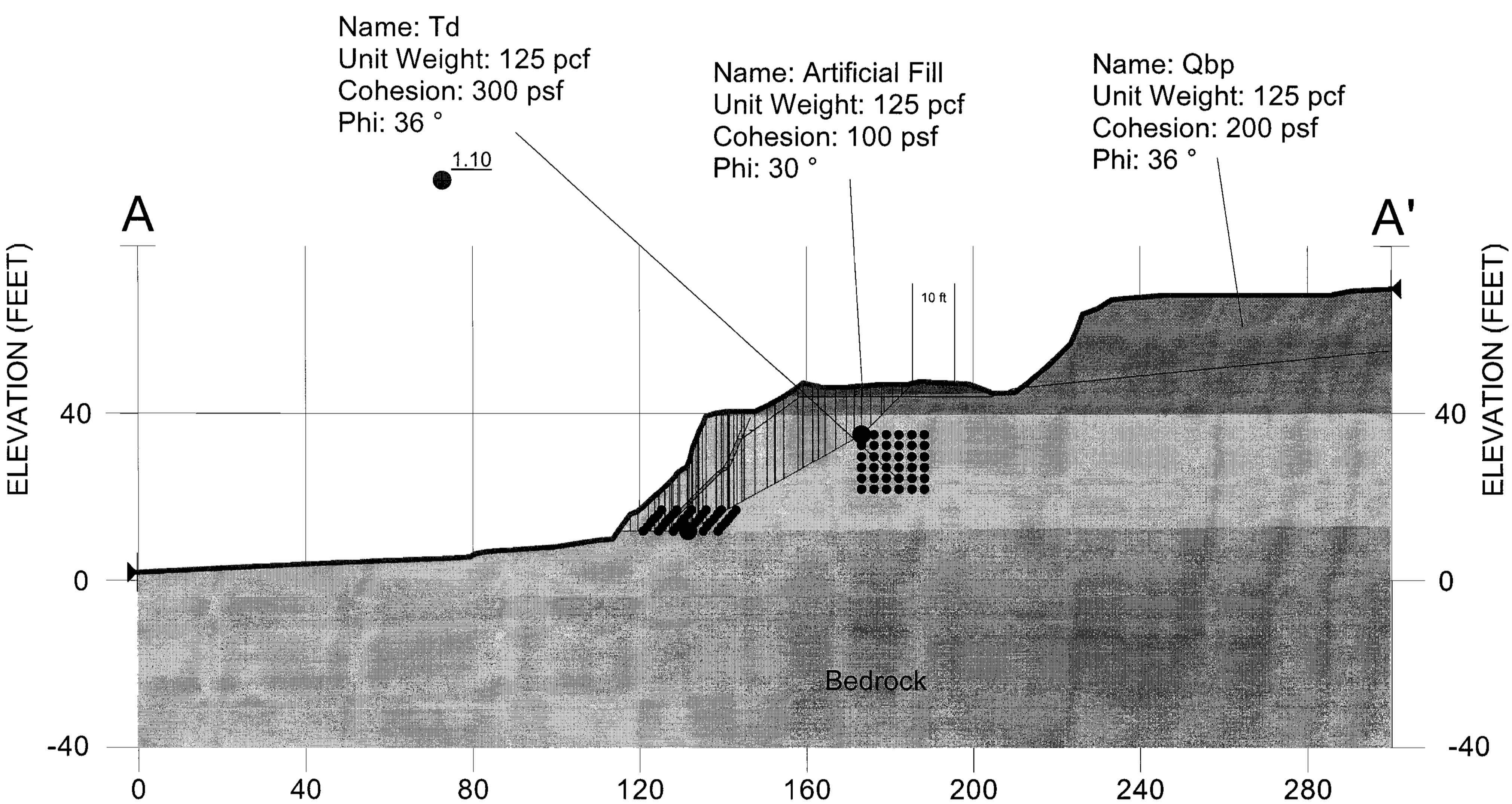
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section A-A'
Slope Stability Analysis
File Name: Section A-A' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 1.10

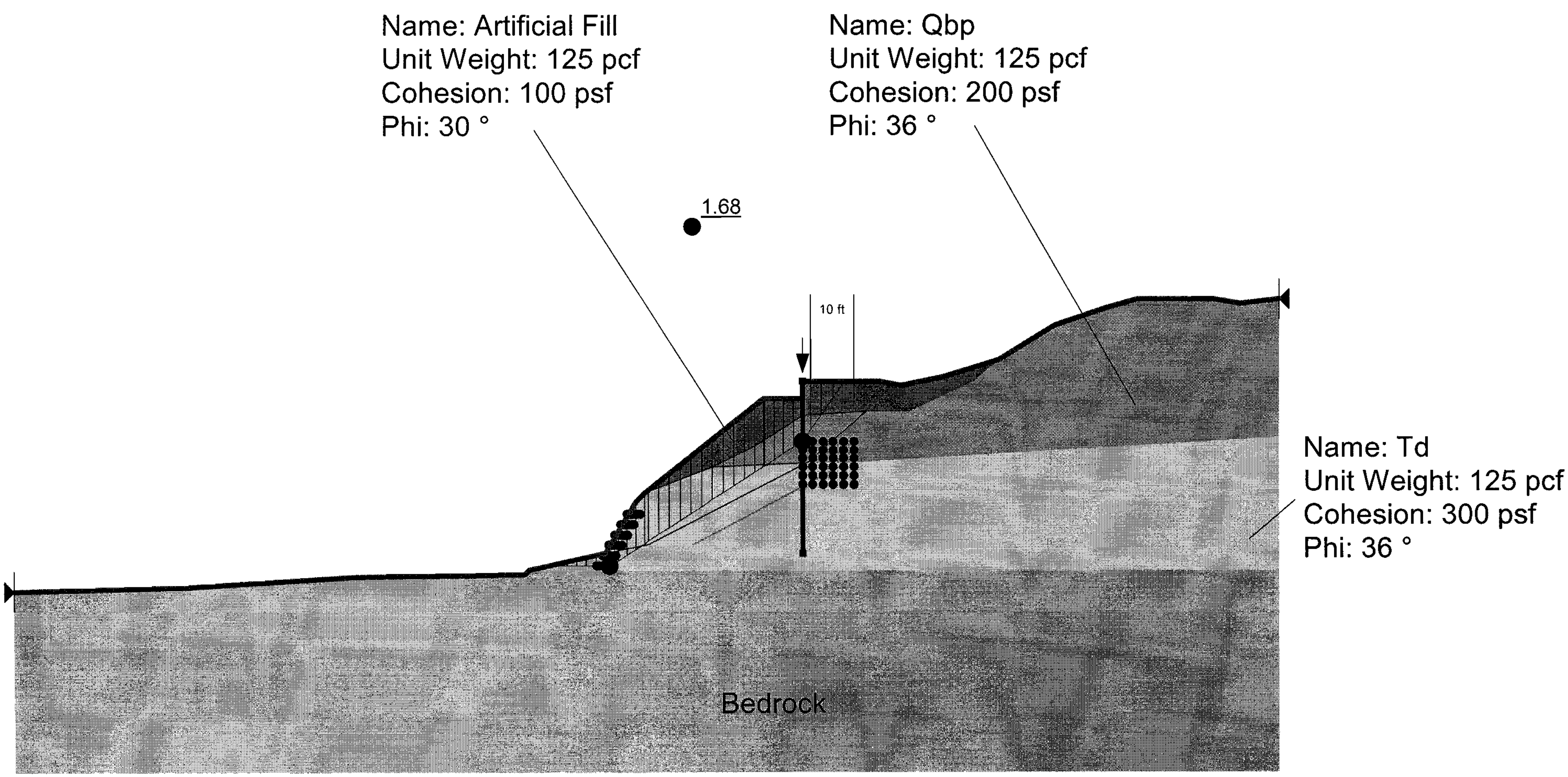
Seismic Coefficient = 0.28



CROSS SECTION A1-A1'

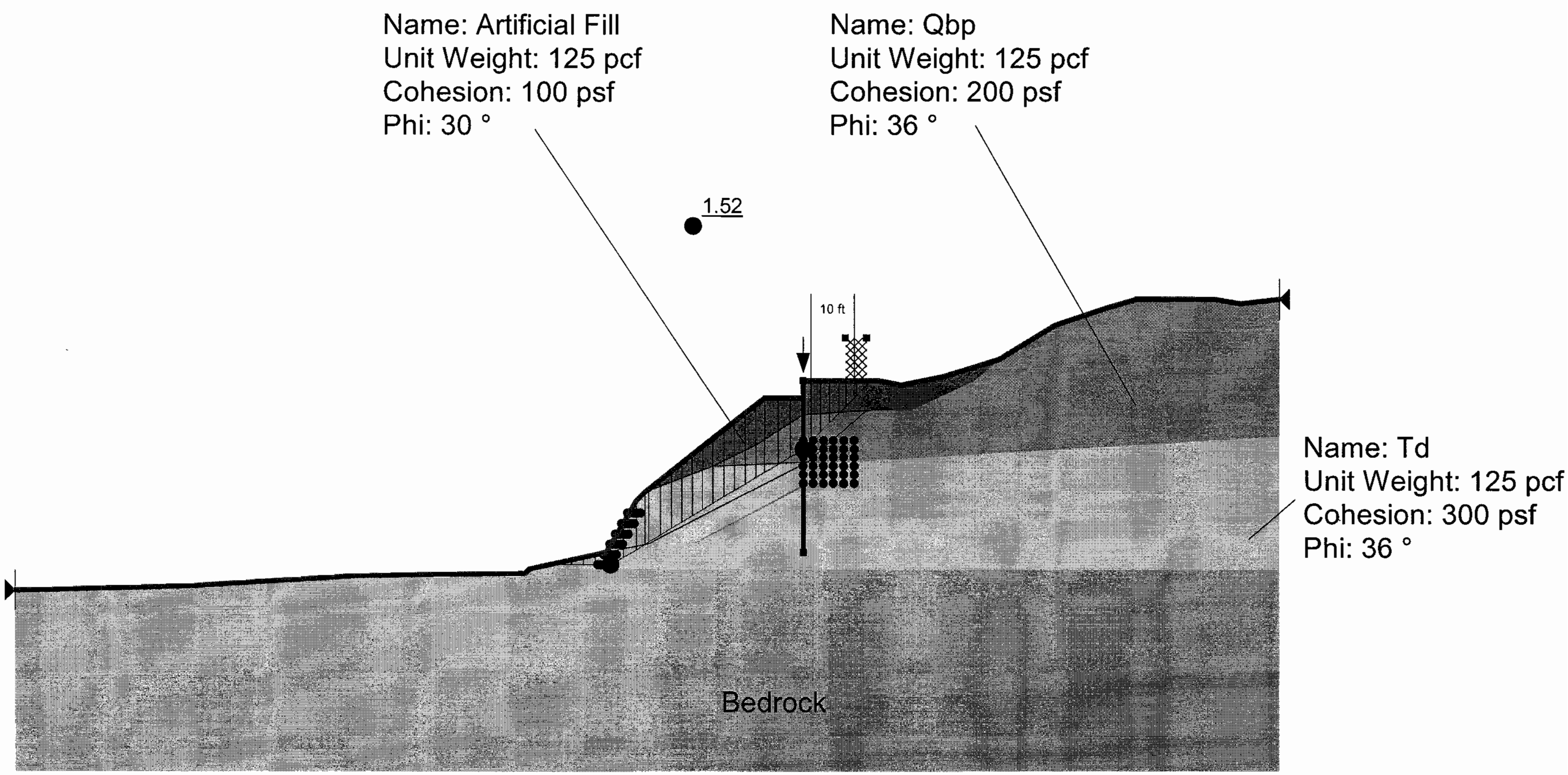
Del Mar Bluffs Cross Section A1-A1'
Slope Stability Analysis
File Name: Section A1-A1' Static 1.gsz
Analysis Method: Spencer

Factor of Safety: 1.68



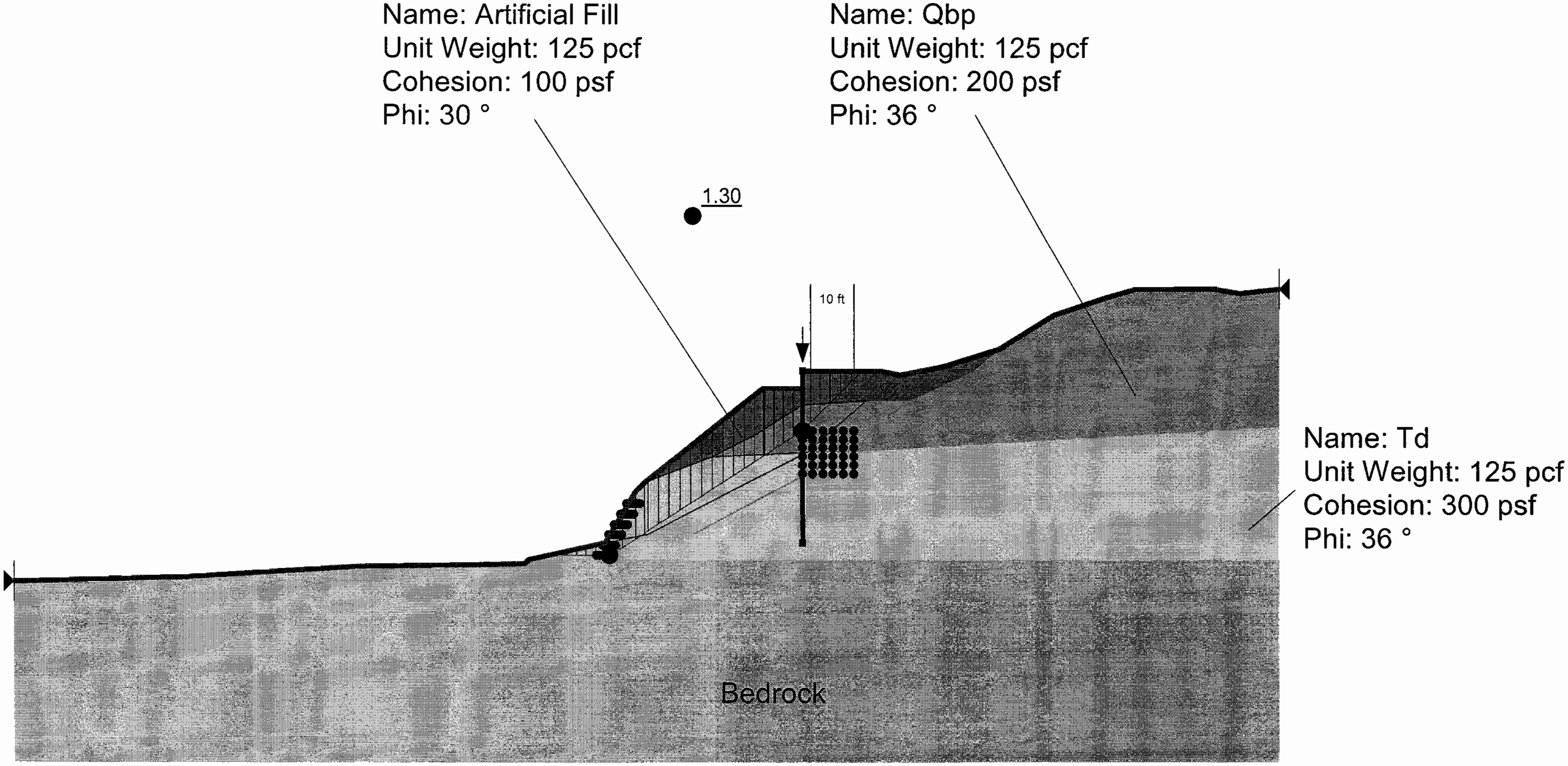
Del Mar Bluffs Cross Section A1-A1'
Slope Stability Analysis
File Name: Section A1-A1' Static 1 + Surcharge(wdo).gsz
Analysis Method: Spencer

Factor of Safety: 1.52



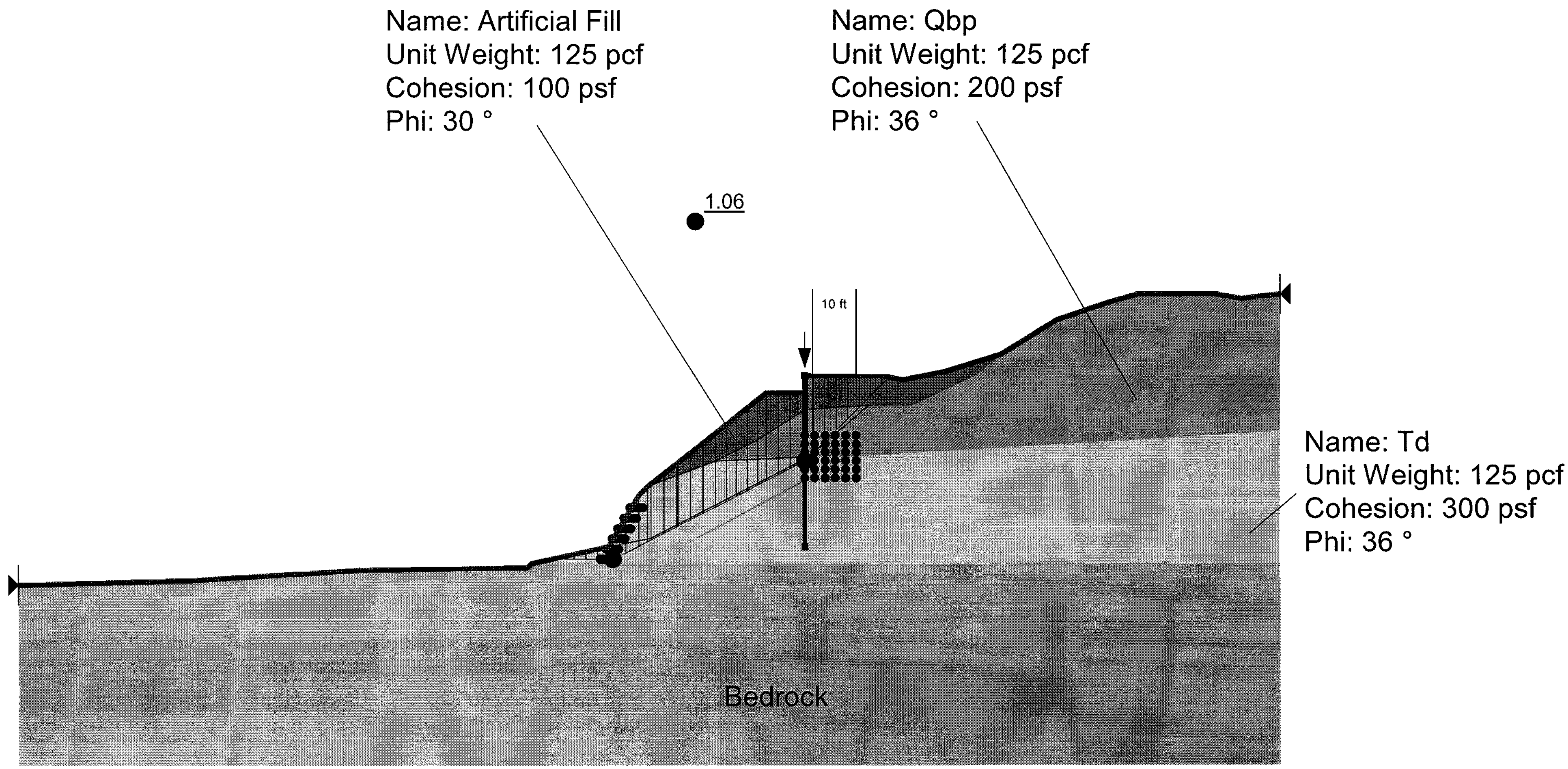
Del Mar Bluffs Cross Section A1-A1'
Slope Stability Analysis
File Name: Section A1-A1' Pseudostatic Kh = 0.15(wdo).gsz
Analysis Method: Spencer

Factor of Safety: 1.30



Del Mar Bluffs Cross Section A1-A1'
Slope Stability Analysis
File Name: Section A1-A1' Pseudostatic Kh = 0.28(wdo).gsz
Analysis Method: Spencer

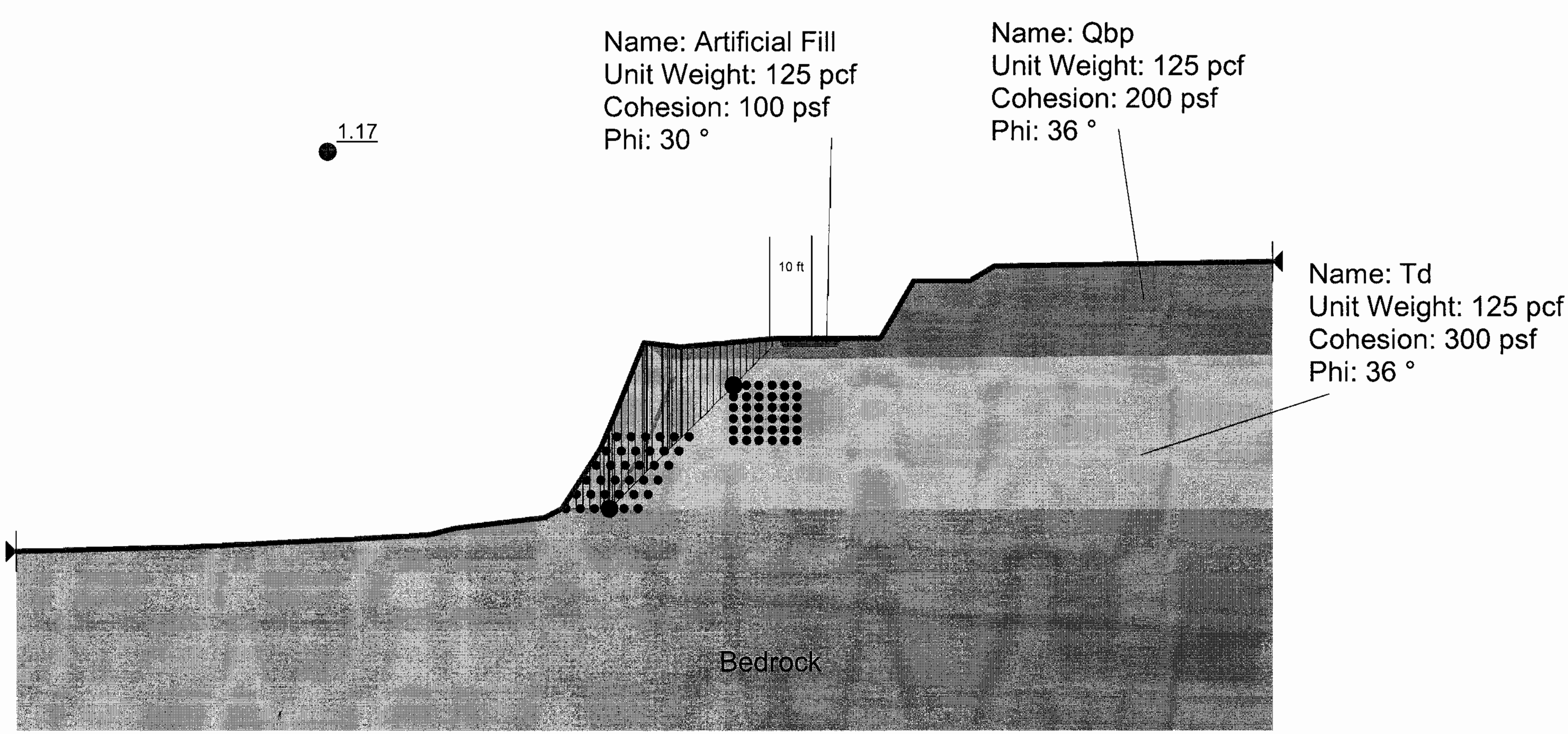
Factor of Safety: 1.06



CROSS SECTION A2-A2’

Del Mar Bluffs Cross Section A2-A2'
Slope Stability Analysis
File Name: Section A2-A2' Static B.gsz
Analysis Method: Spencer

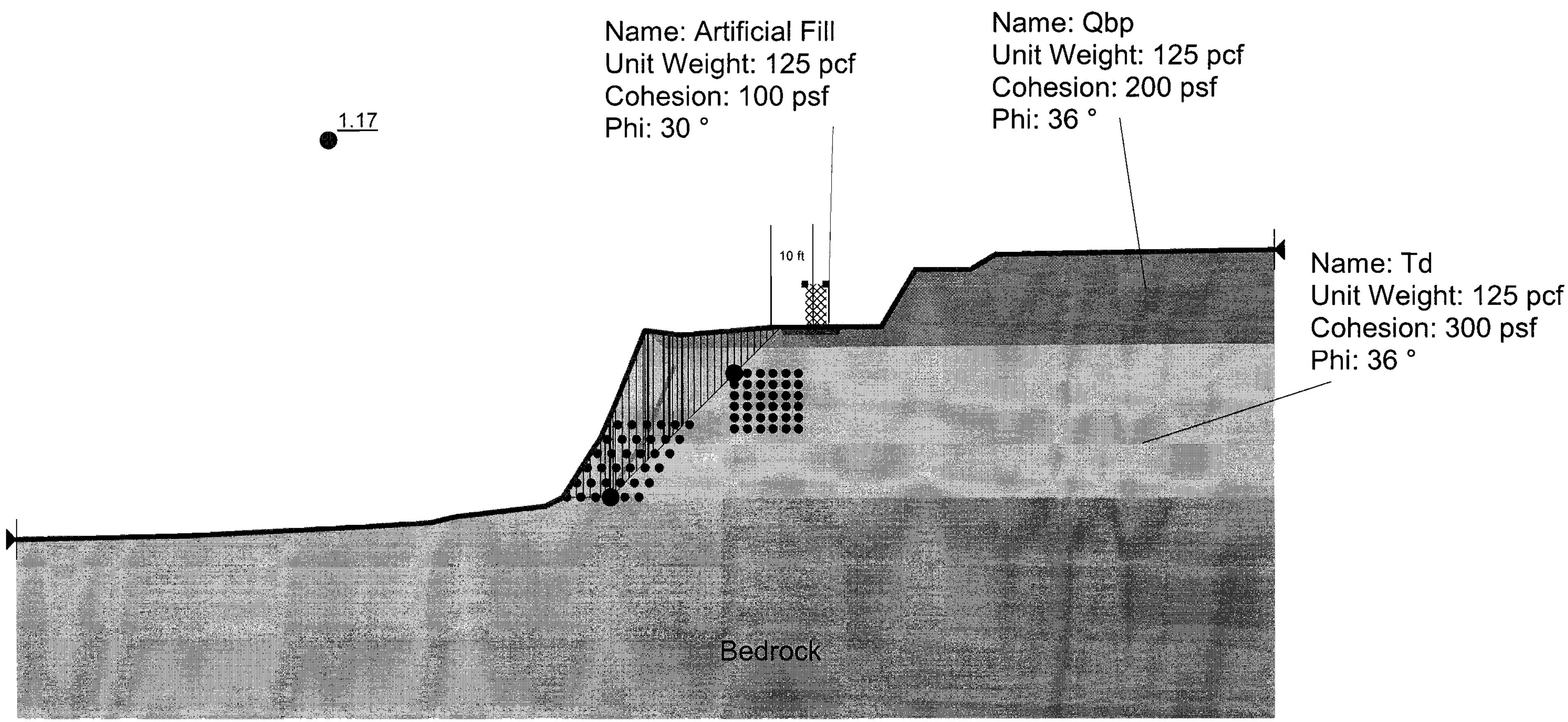
Factor of Safety: 1.17



Del Mar Bluffs Cross Section A2-A2'
Slope Stability Analysis
File Name: Section A2-A2' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.17

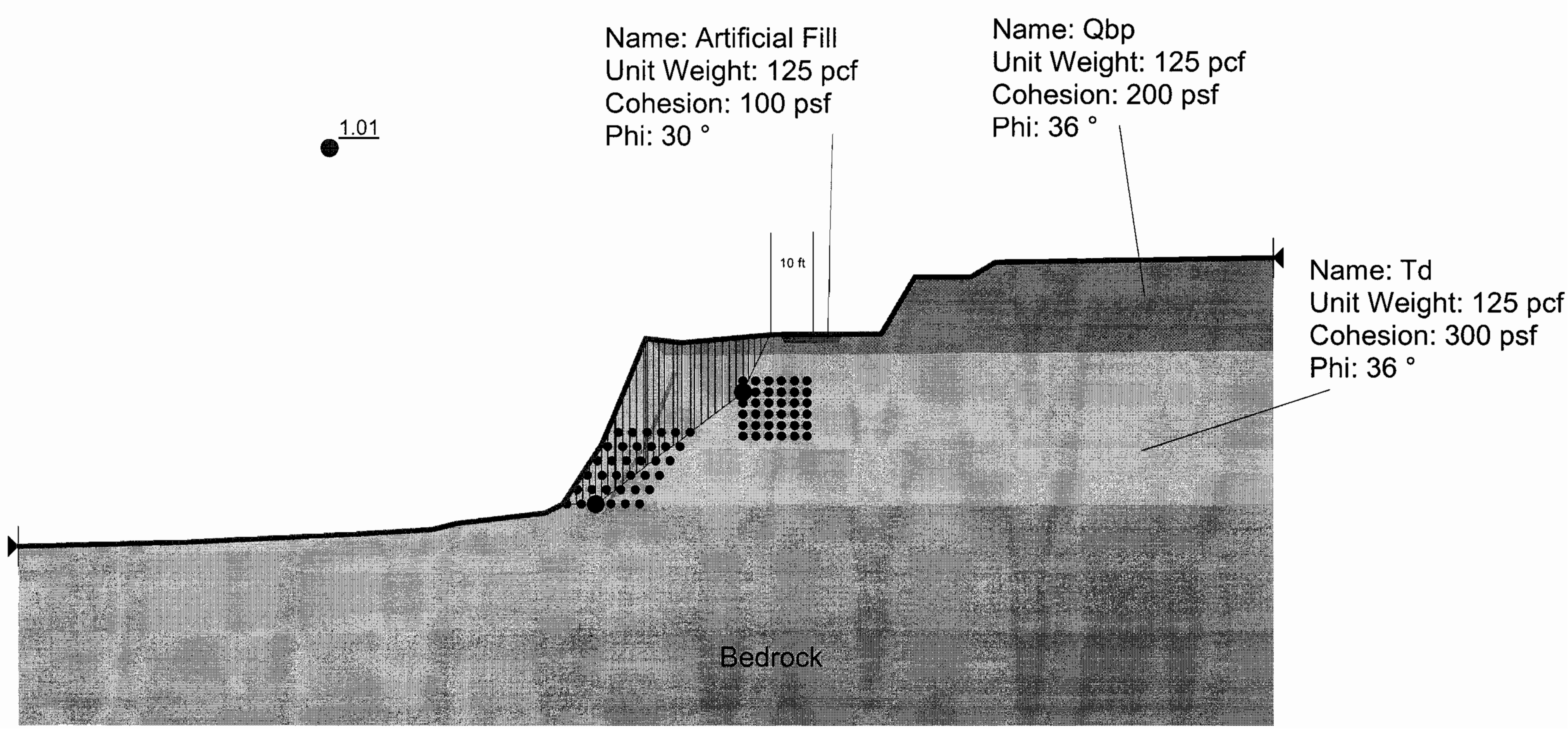
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section A2-A2'
Slope Stability Analysis
File Name: Section A2-A2' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.01

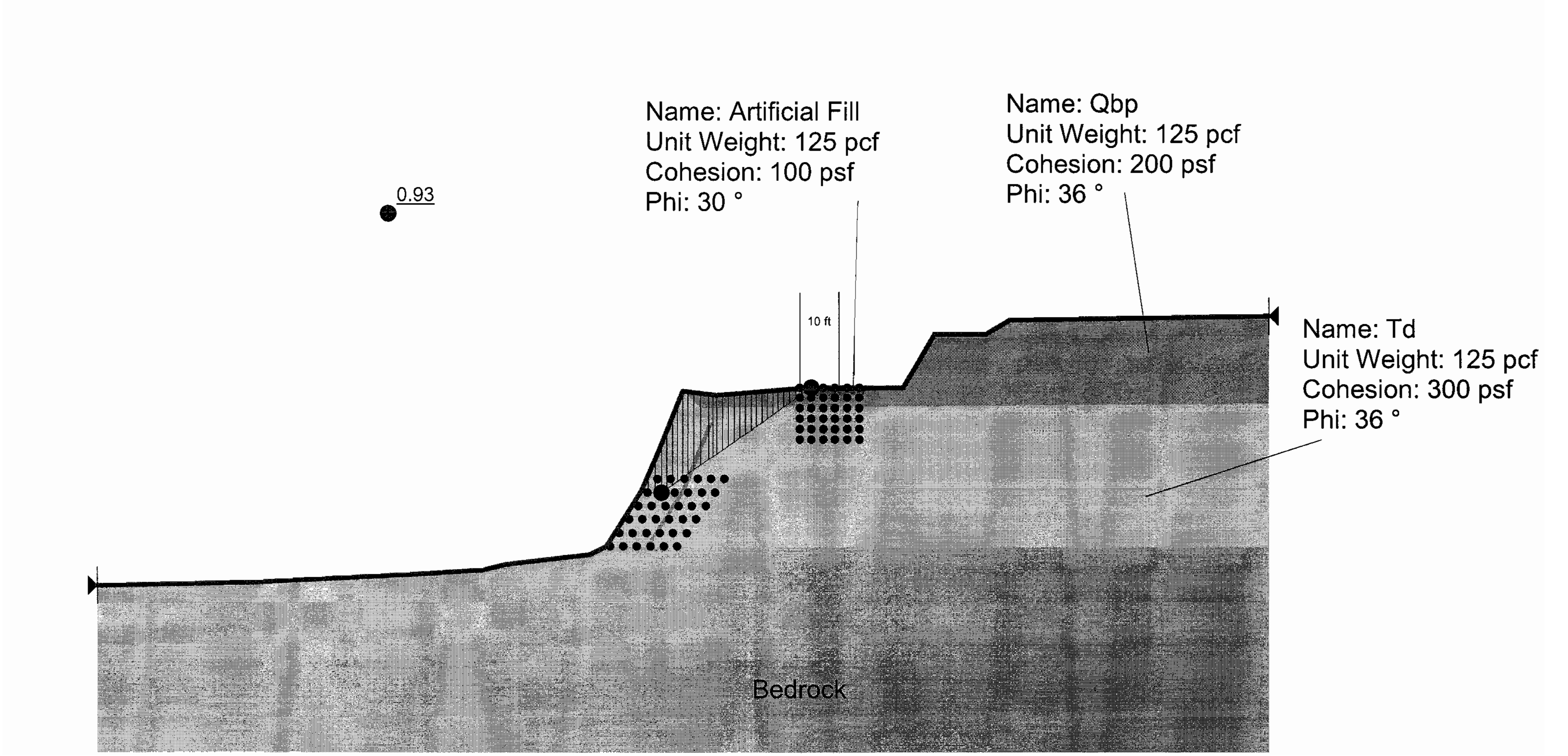
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section A2-A2'
Slope Stability Analysis
File Name: Section A2-A2' Pseudostatic Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.93

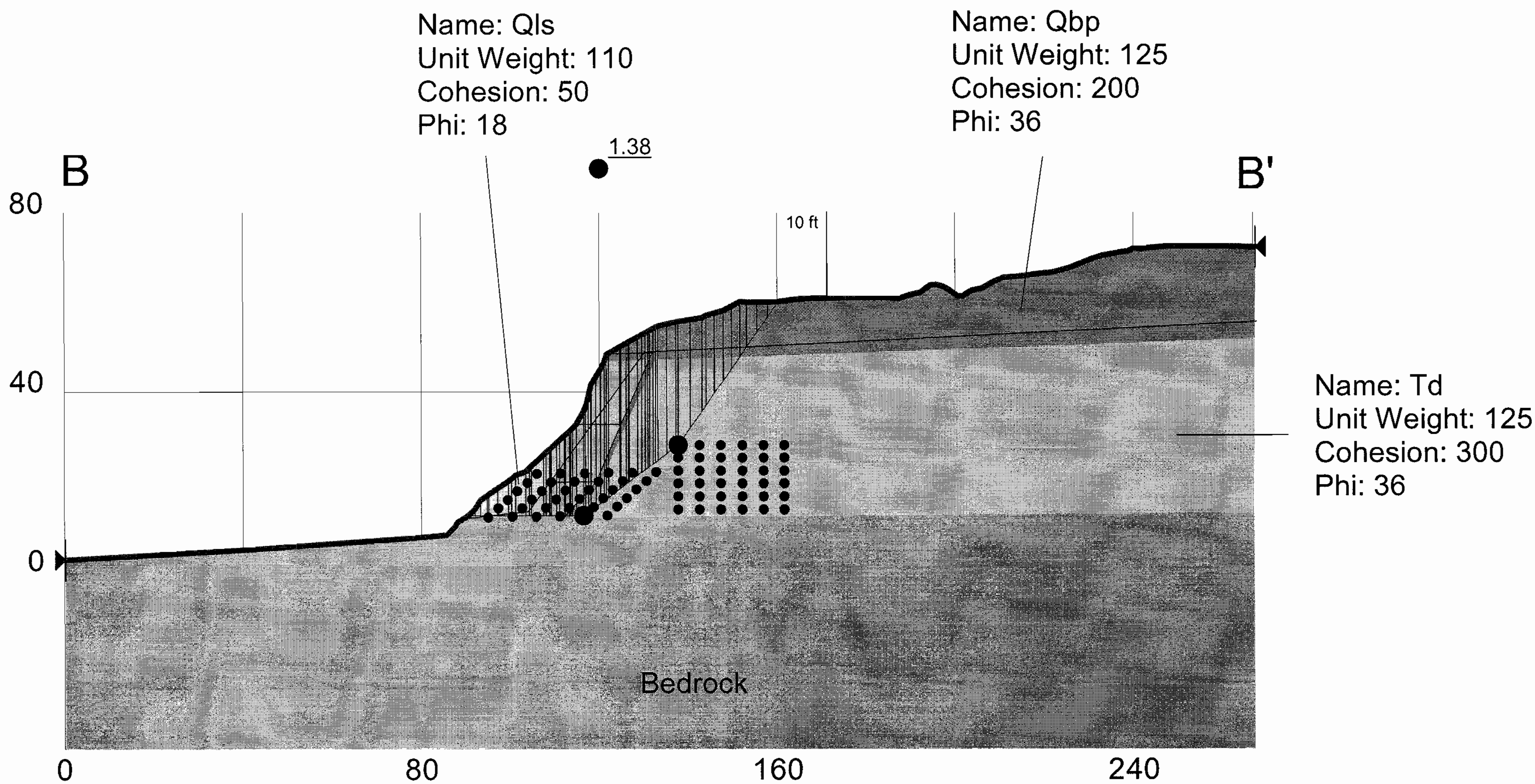
Seismic Coefficient = 0.28



CROSS SECTION B-B’

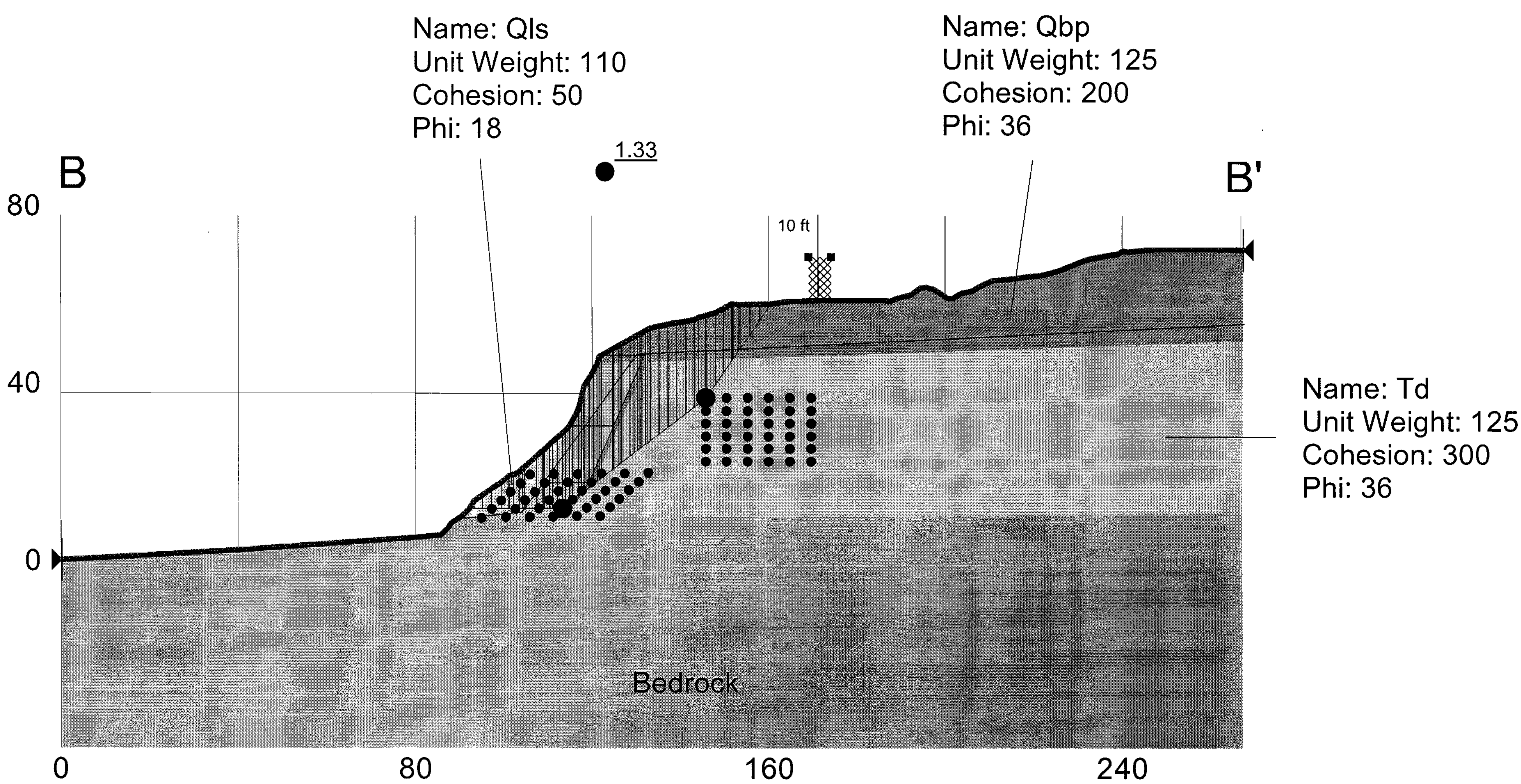
Del Mar Bluffs Cross Section B-B'
Slope Stability Analysis
File Name: Section B-B' Static B.gsz
Analysis Method: Spencer

Factor of Safety: 1.38



Del Mar Bluffs Cross Section B-B'
Slope Stability Analysis
File Name: Section B-B' Static B + Surcharge.gsz
Analysis Method: Spencer

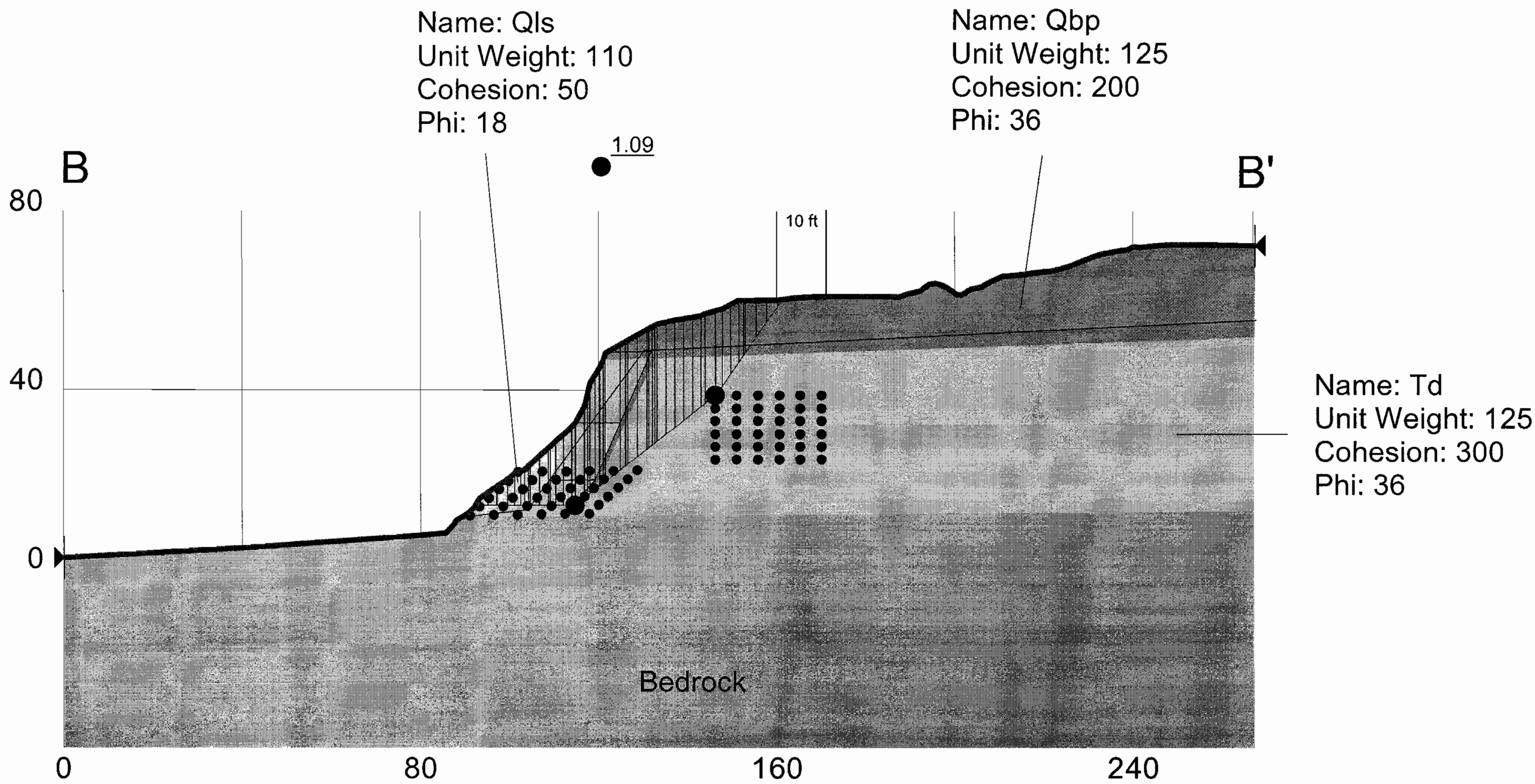
Factor of Safety: 1.33



Del Mar Bluffs Cross Section B-B'
Slope Stability Analysis
File Name: Section B-B' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.09

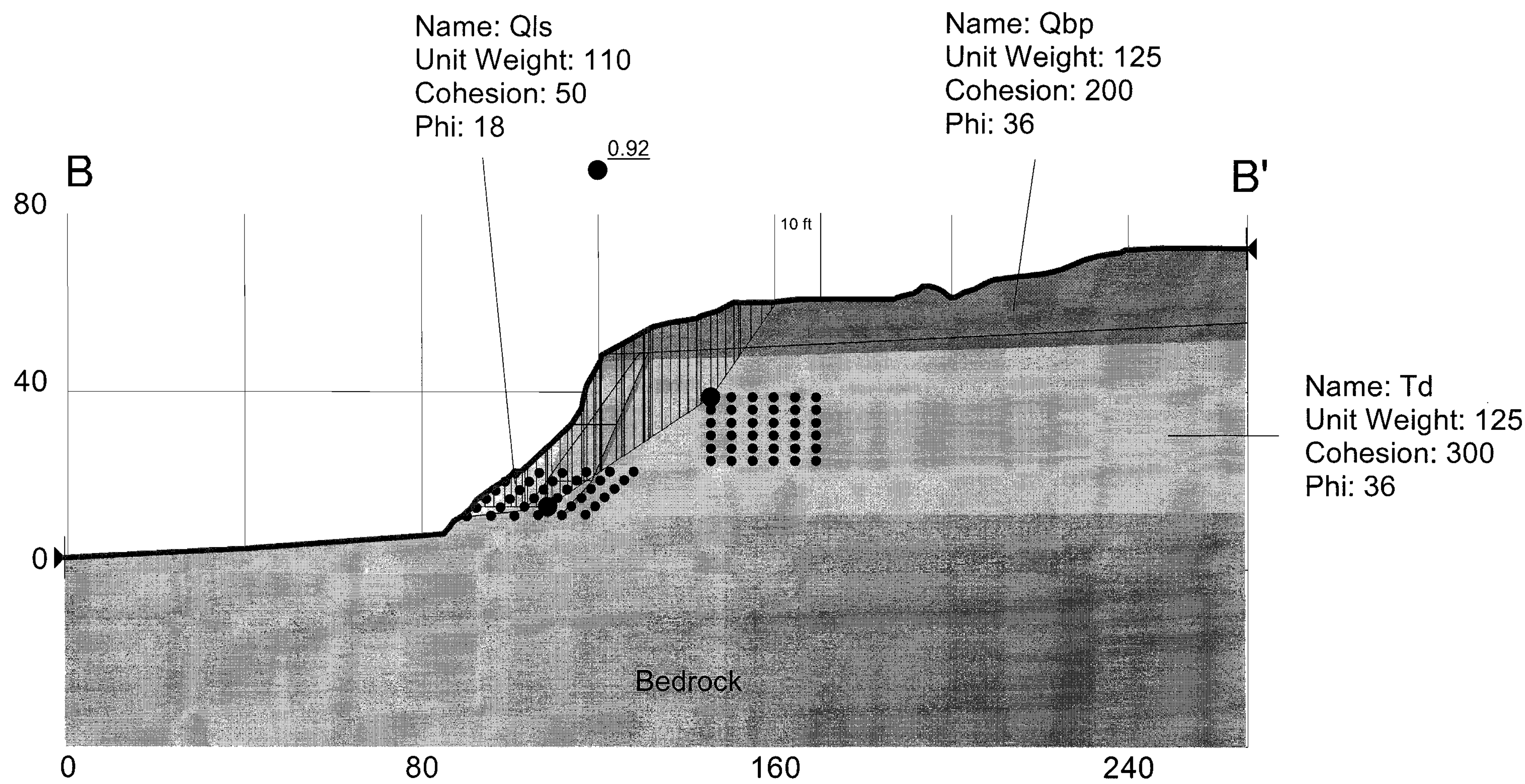
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section B-B'
Slope Stability Analysis
File Name: Section B-B' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.92

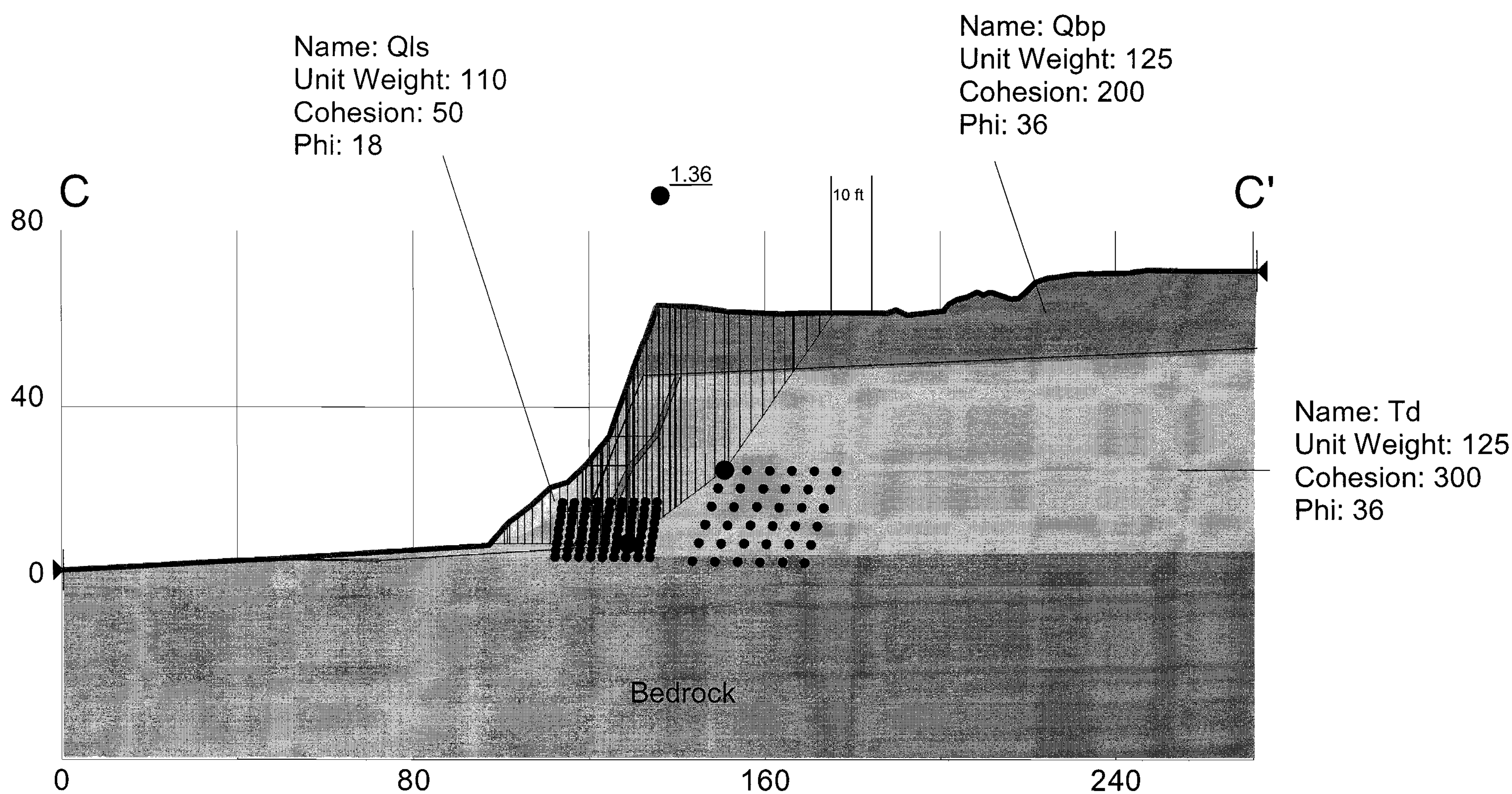
Seismic Coefficient = 0.28



CROSS SECTION C-C'

Del Mar Bluffs Cross Section C-C'
Slope Stability Analysis
File Name: Section C-C' Static 1B.gsz
Analysis Method: Spencer

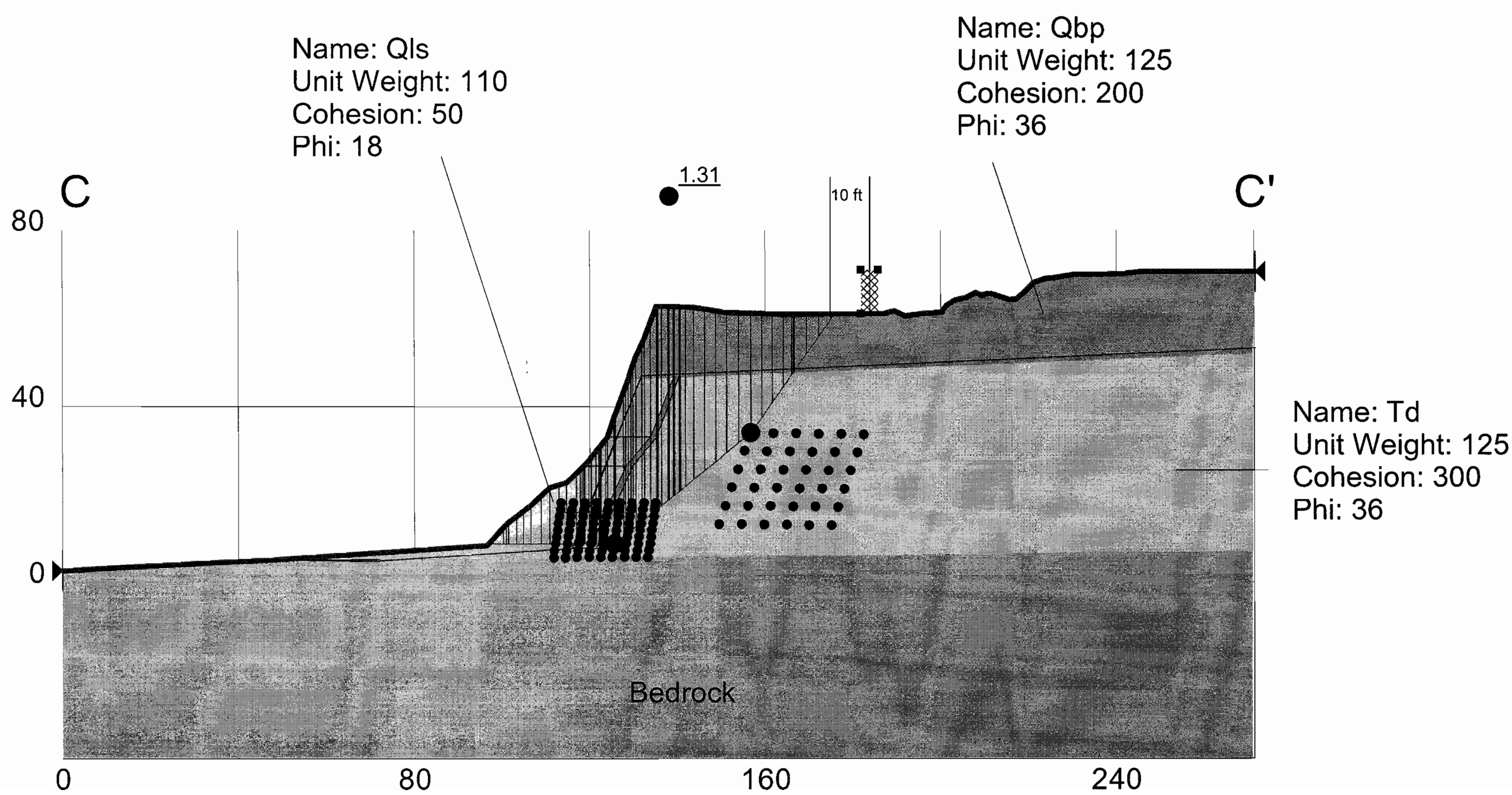
Factor of Safety: 1.36



Del Mar Bluffs Cross Section C-C'
Slope Stability Analysis
File Name: Section C-C' Static 1B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.31

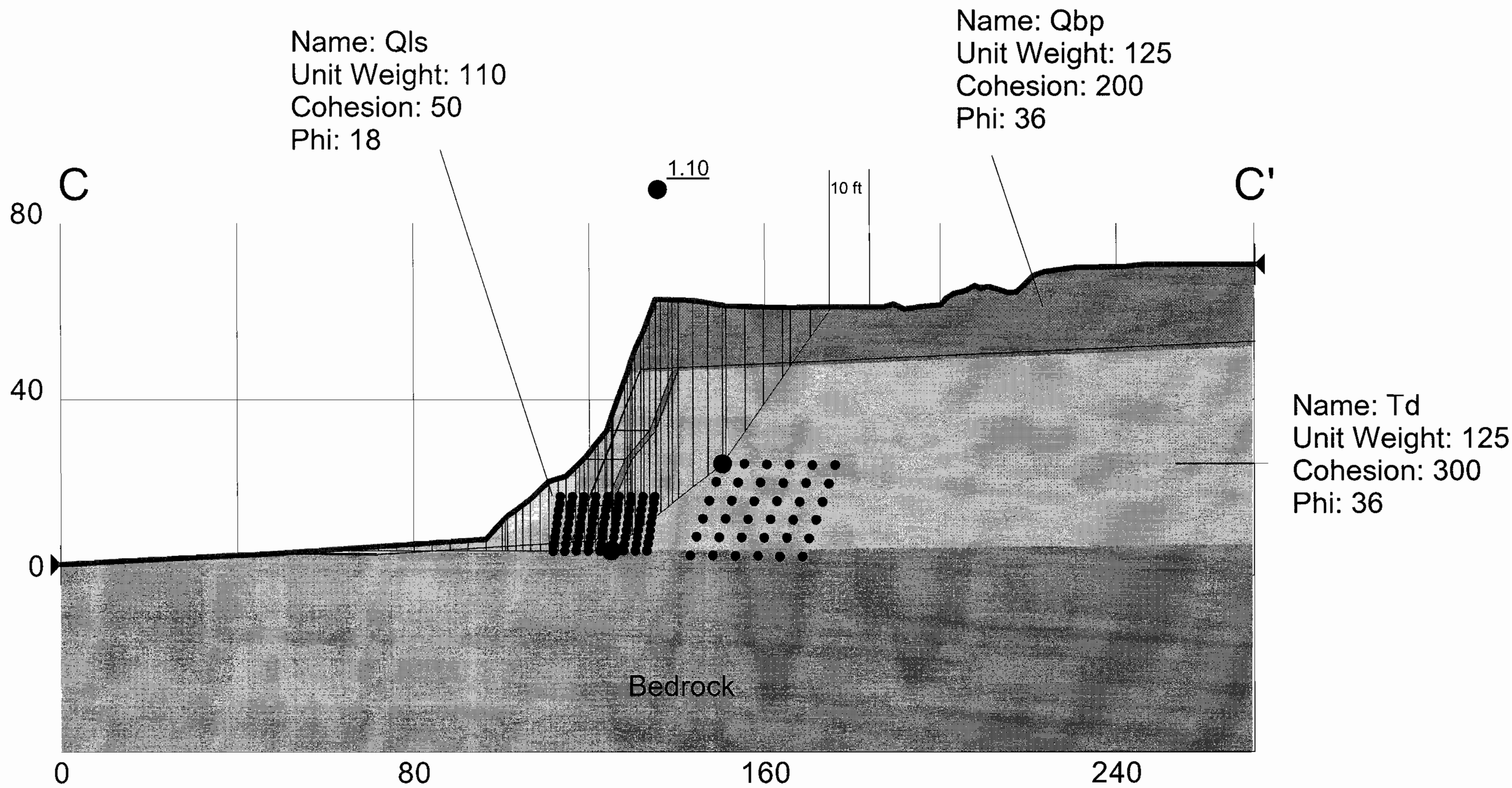
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section C-C'
Slope Stability Analysis
File Name: Section C-C' Pseudostatic B kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.10

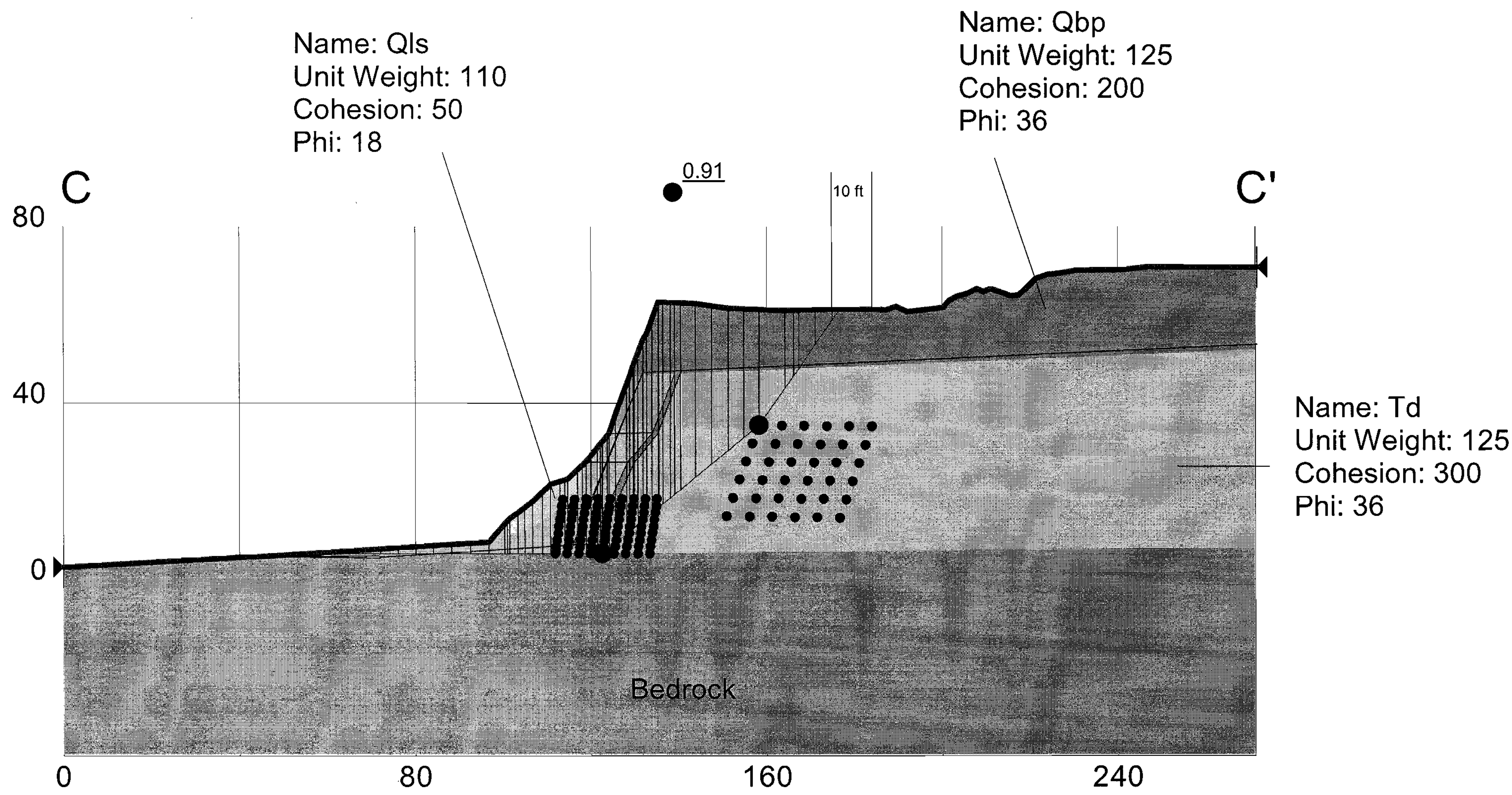
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section C-C'
Slope Stability Analysis
File Name: Section C-C' Pseudostatic B kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.91

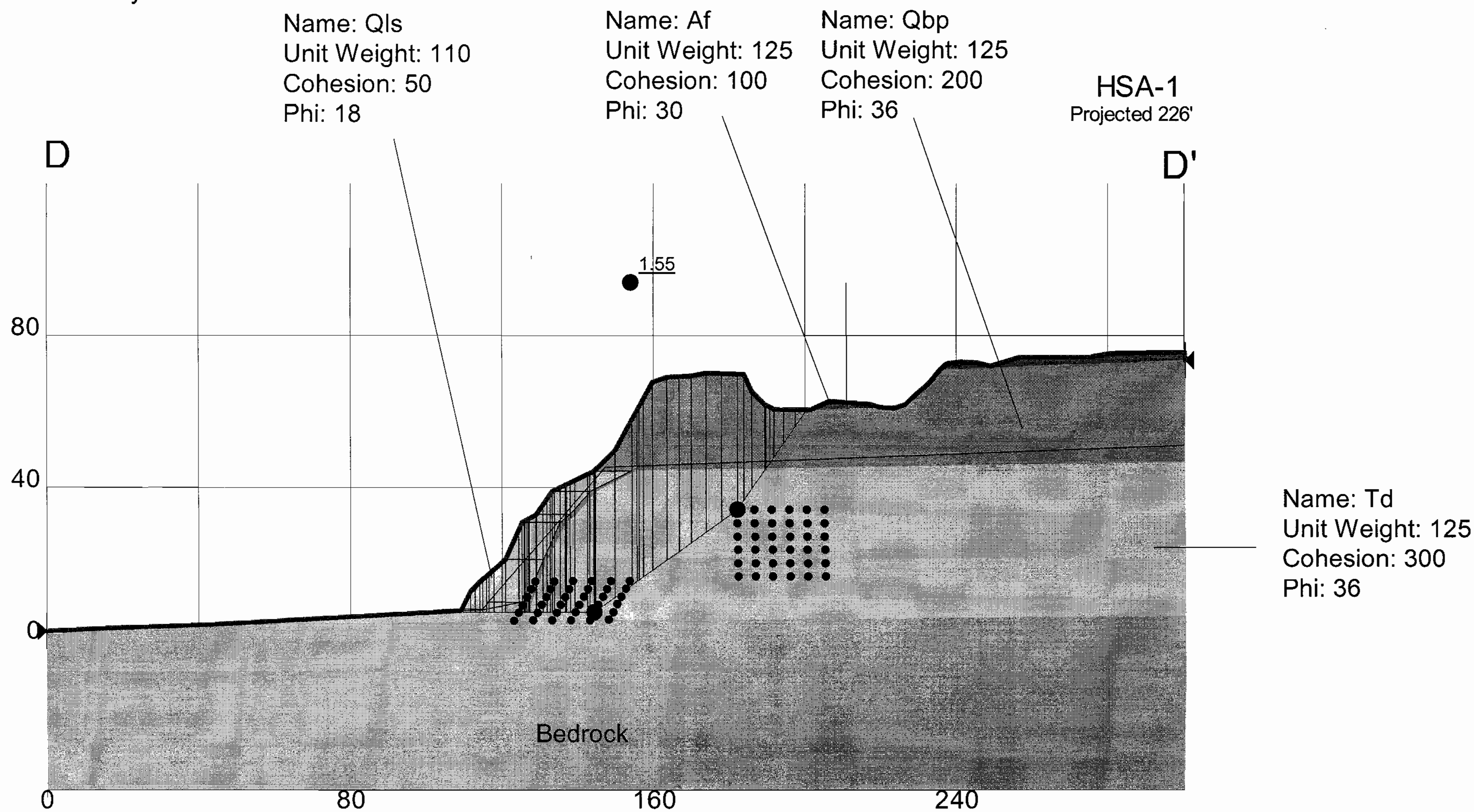
Seismic Coefficient = 0.28



CROSS SECTION D-D'

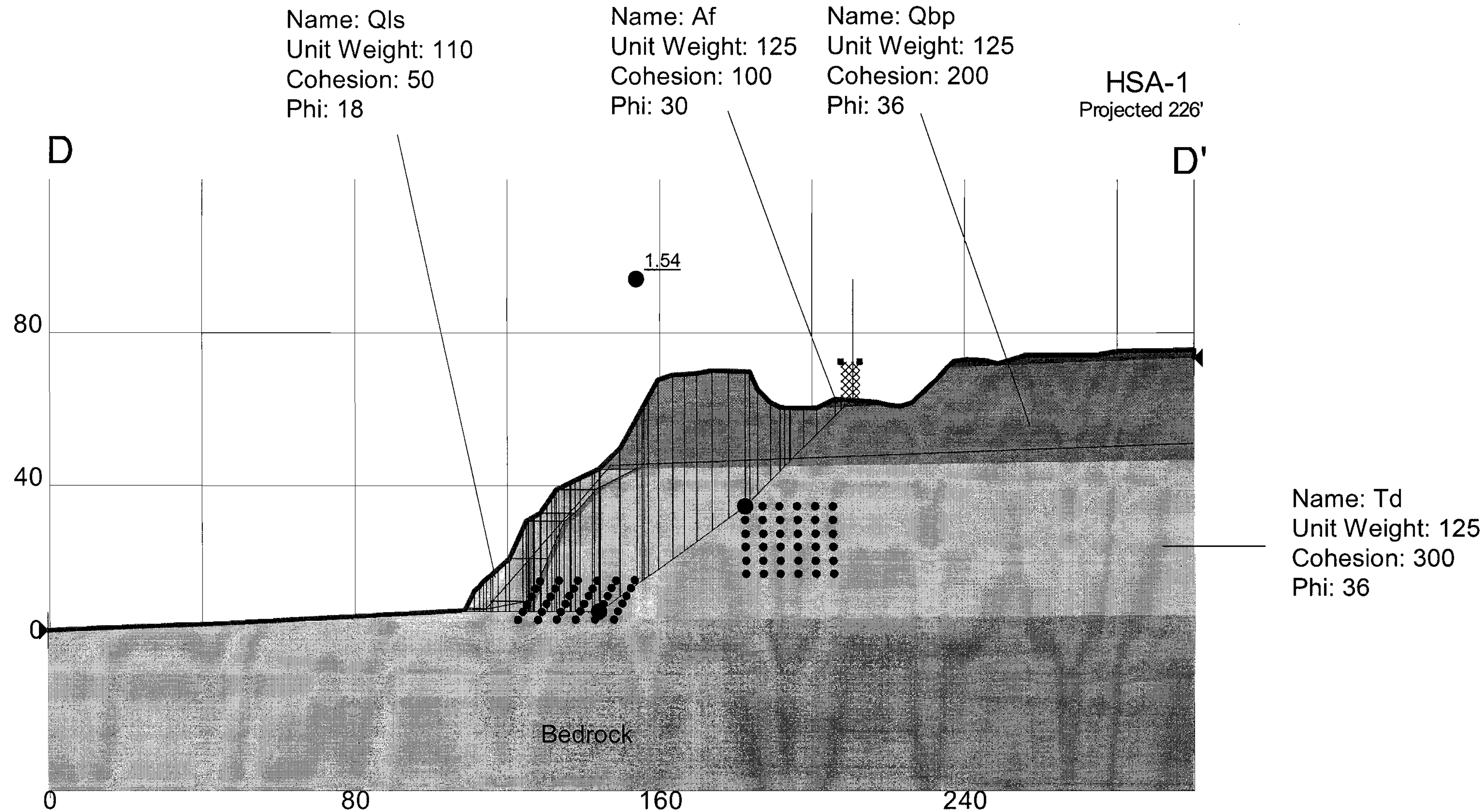
Del Mar Bluffs Cross Section D-D'
Slope Stability Analysis
File Name: Section D-D' Static 1B.gsz
Analysis Method: Spencer

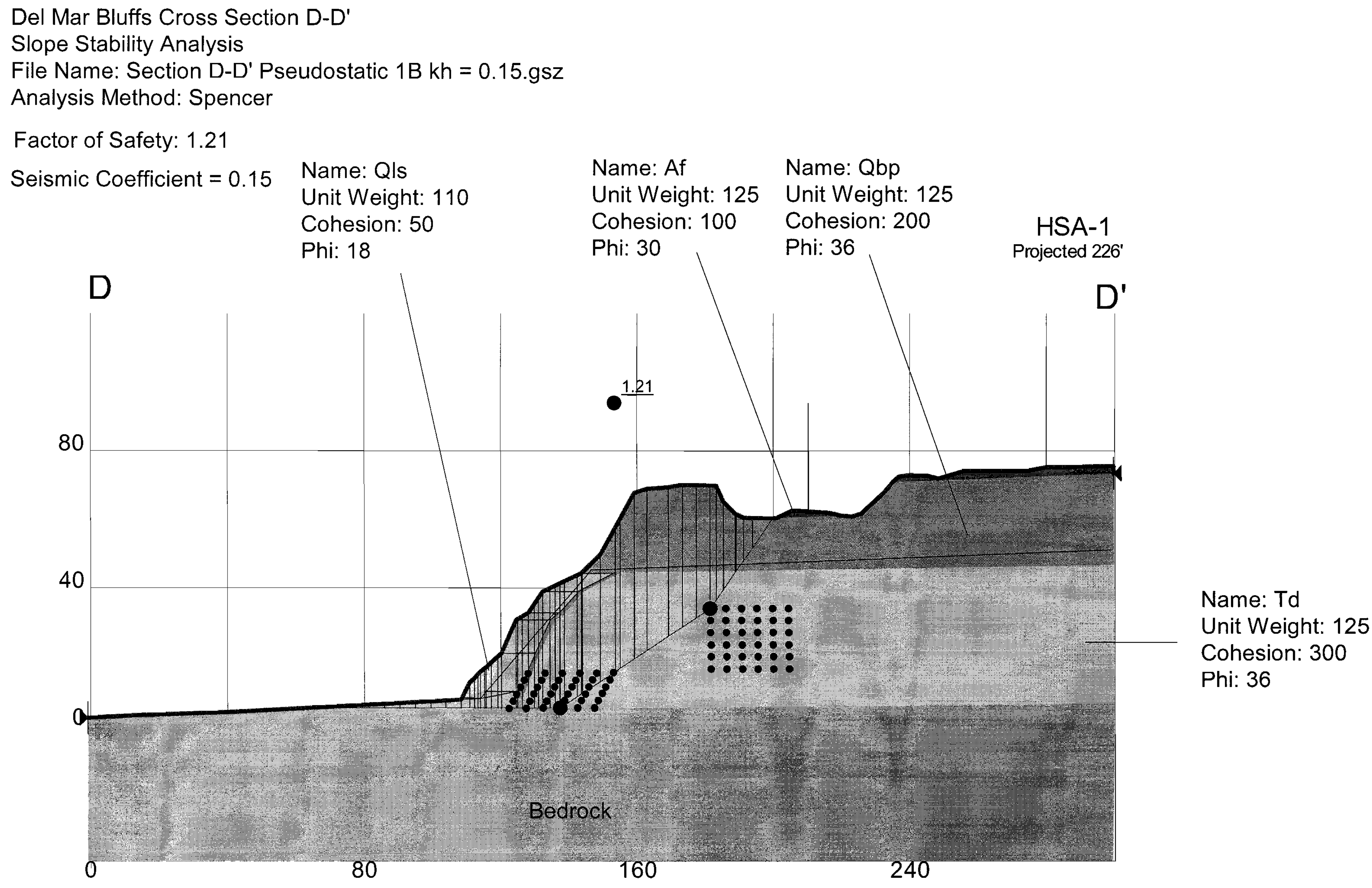
Factor of Safety: 1.55



Del Mar Bluffs Cross Section D-D'
Slope Stability Analysis
File Name: Section D-D' Static 1B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.54

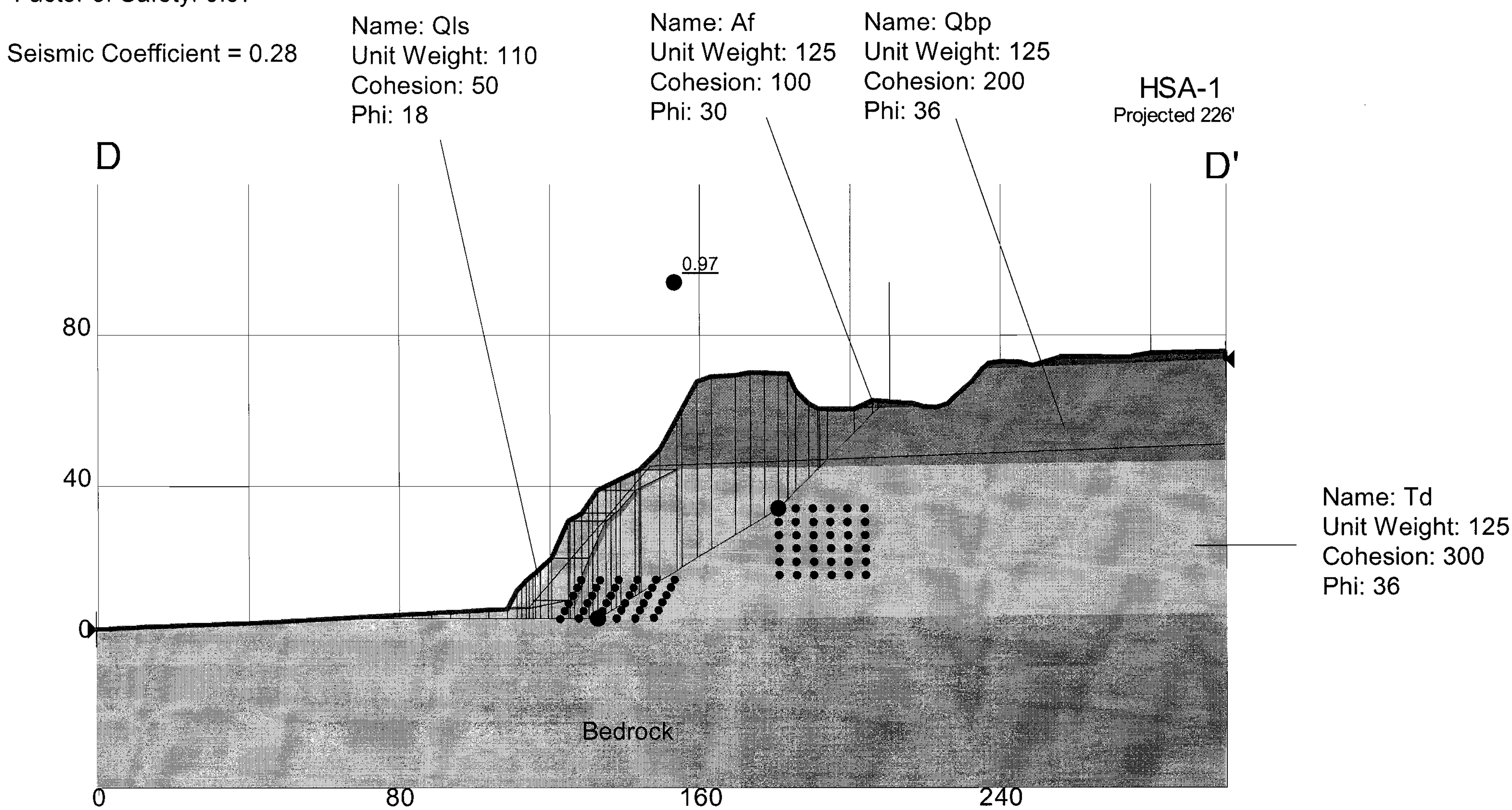



Del Mar Bluffs Cross Section D-D'
Slope Stability Analysis
File Name: Section D-D' Pseudostatic 1B kh = 0.15.gsz
Analysis Method: Spencer
Factor of Safety: 1.21
Seismic Coefficient = 0.15
Name: Qls
Unit Weight: 110
Cohesion: 50
Phi: 18
Name: Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
HSA-1
Projected 226'
D
D'
1.21
80
40
0
Name: Td
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock
0
80
160
240

Del Mar Bluffs Cross Section D-D'
Slope Stability Analysis
File Name: Section D-D' Pseudostatic 1B kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.97

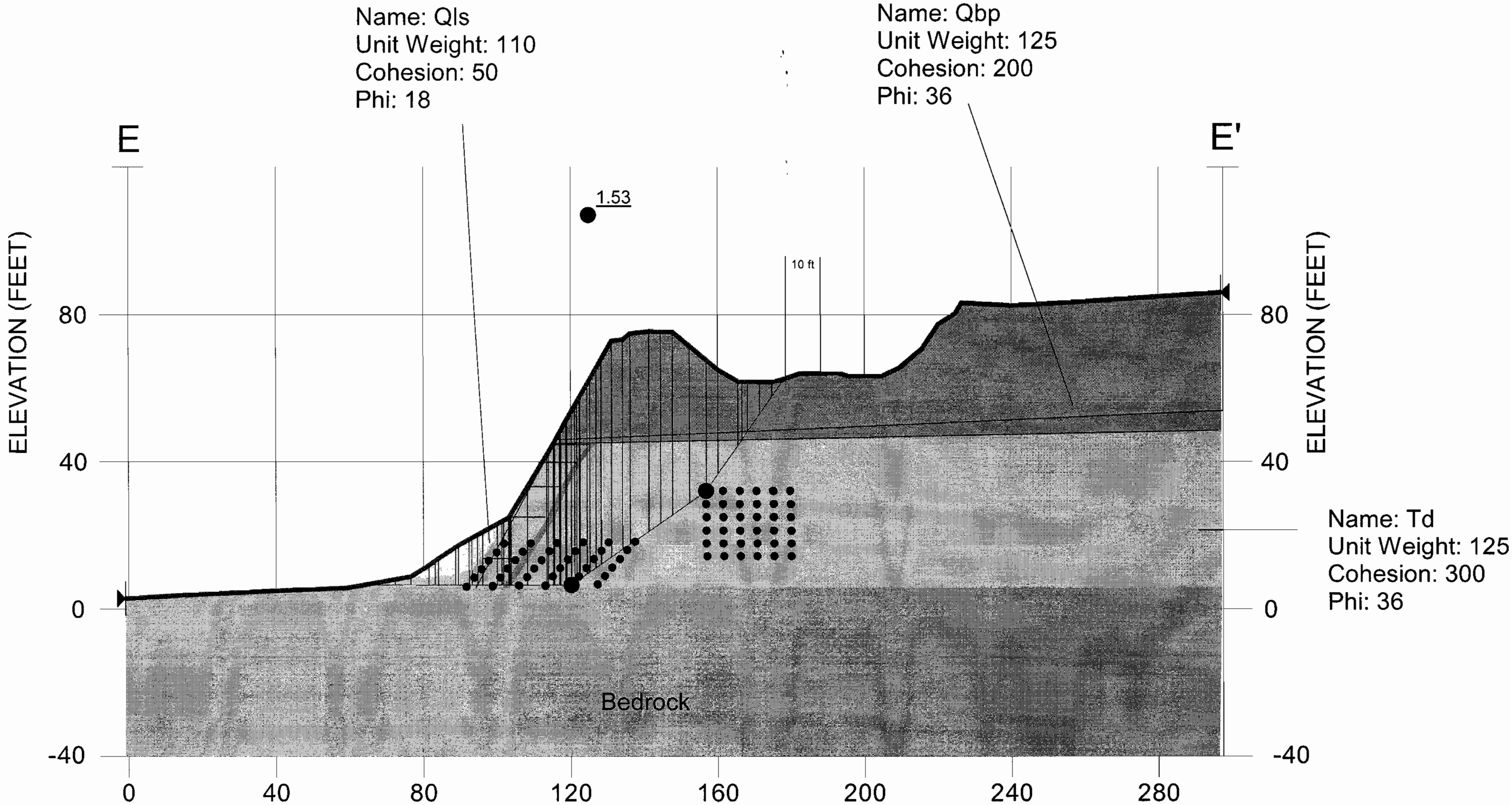
Seismic Coefficient = 0.28

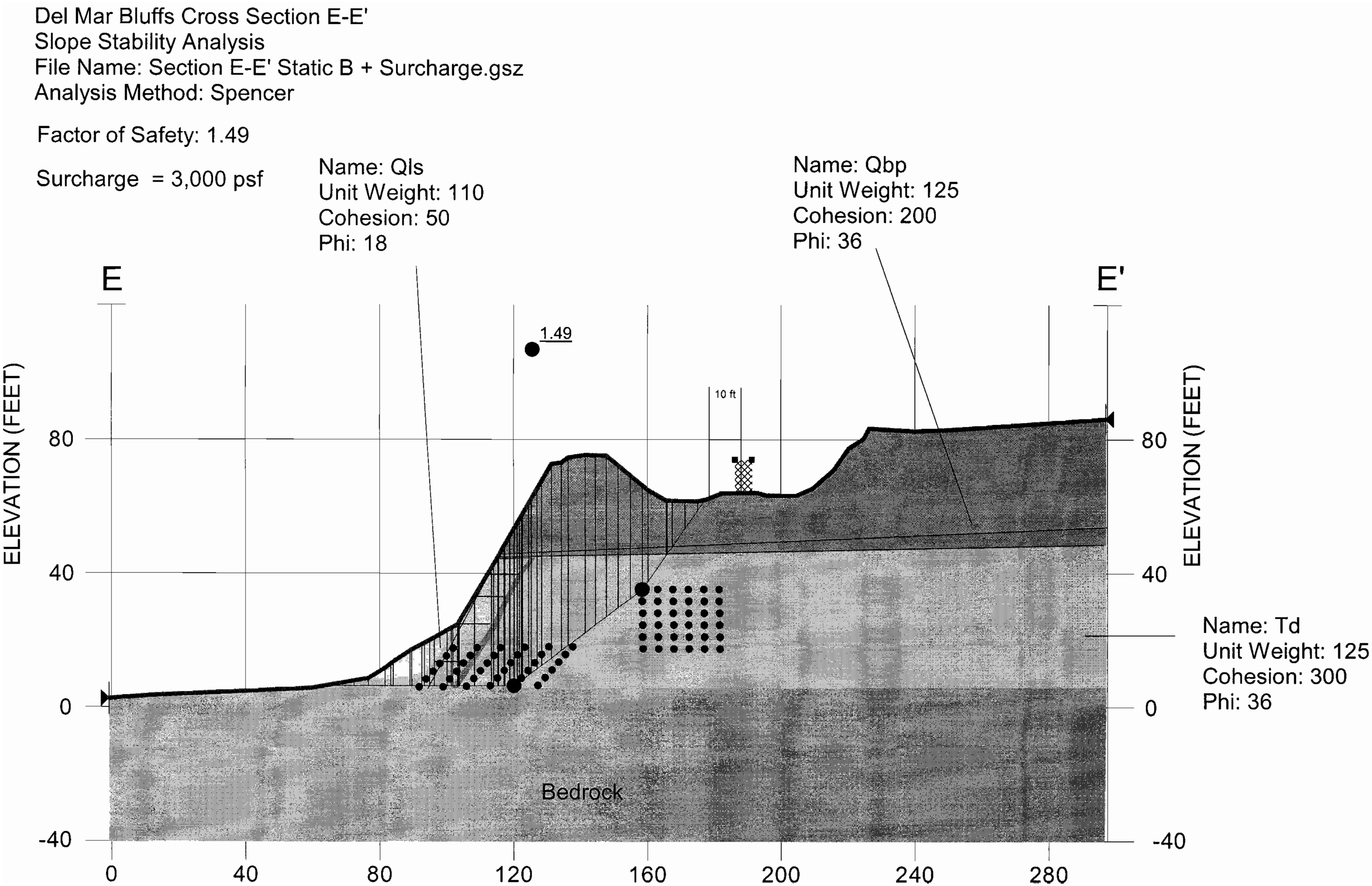


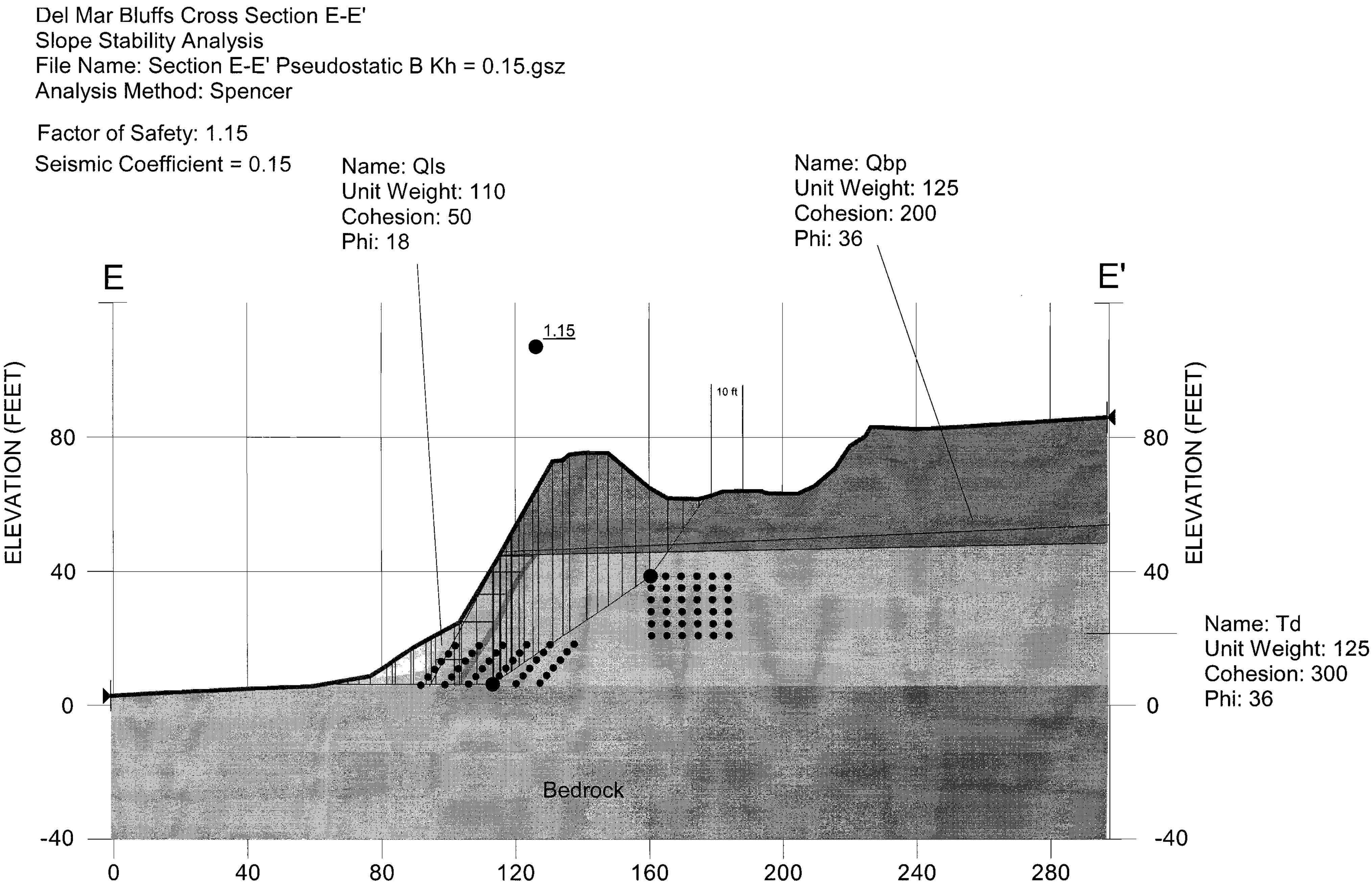
CROSS SECTION E-E'

Del Mar Bluffs Cross Section E-E'
Slope Stability Analysis
File Name: Section E-E' Static B.gsz
Analysis Method: Spencer

Factor of Safety: 1.53



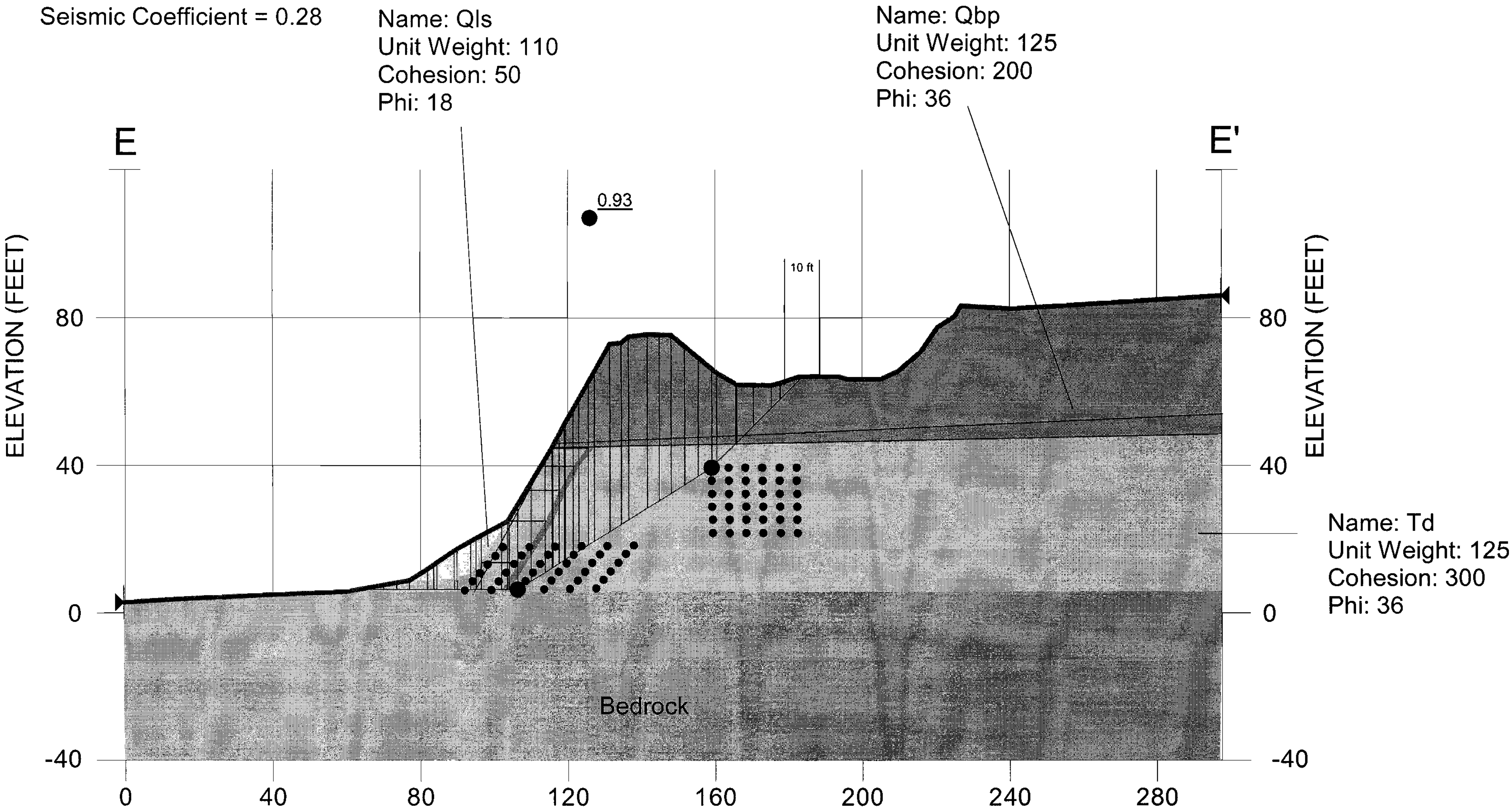

Del Mar Bluffs Cross Section E-E'
Slope Stability Analysis
File Name: Section E-E' Static B + Surcharge.gsz
Analysis Method: Spencer
Factor of Safety: 1.49
Surcharge = 3,000 psf
Name: Qls
Unit Weight: 110
Cohesion: 50
Phi: 18
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Name: Td
Unit Weight: 125
Cohesion: 300
Phi: 36
E
E'
1.49
10 ft
Bedrock
ELEVATION (FEET)
ELEVATION (FEET)
80
40
0
-40
0
40
80
120
160
200
240
280


Del Mar Bluffs Cross Section E-E'
Slope Stability Analysis
File Name: Section E-E' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer
Factor of Safety: 1.15
Seismic Coefficient = 0.15
Name: Qls
Unit Weight: 110
Cohesion: 50
Phi: 18
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Name: Td
Unit Weight: 125
Cohesion: 300
Phi: 36
E
E'
1.15
10 ft
Bedrock
ELEVATION (FEET)
80
40
0
-40
0
40
80
120
160
200
240
280

Del Mar Bluffs Cross Section E-E'
Slope Stability Analysis
File Name: Section E-E' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.93

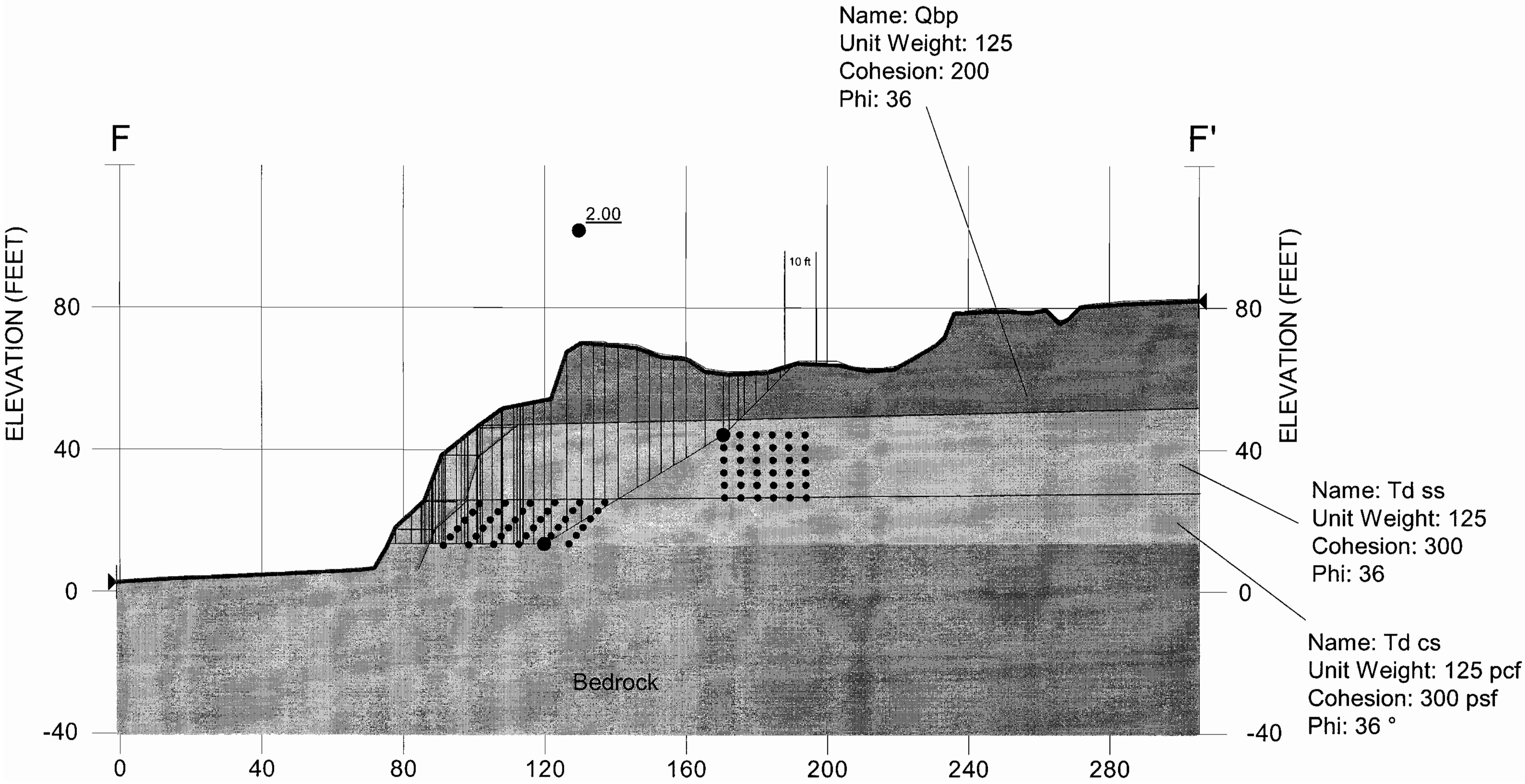
Seismic Coefficient = 0.28



CROSS SECTION F-F’

Del Mar Bluffs Cross Section F-F'
Slope Stability Analysis
File Name: Section F-F' Static B.gsz
Analysis Method: Spencer

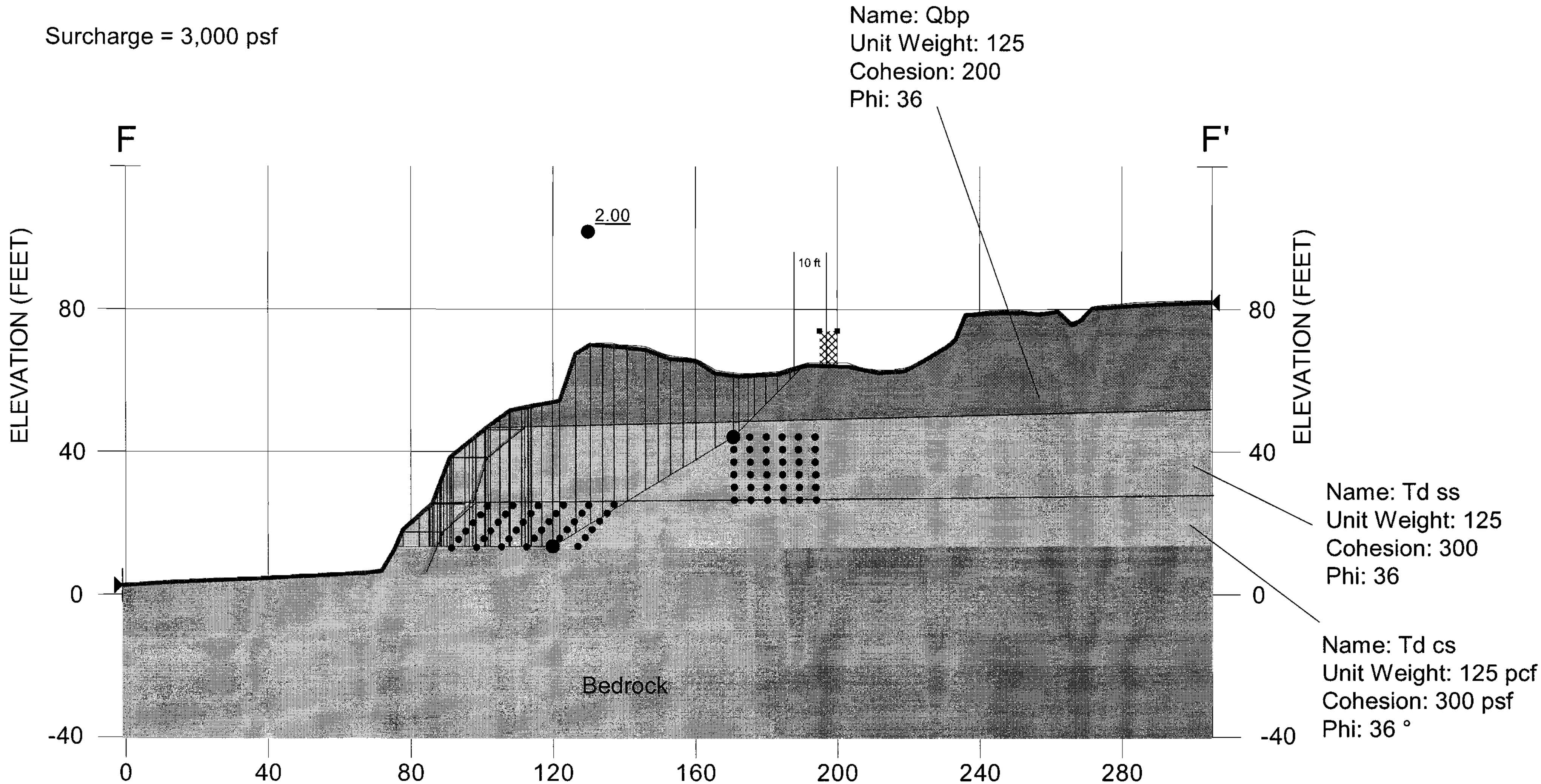
Factor of Safety: 2.00



Del Mar Bluffs Cross Section F-F'
Slope Stability Analysis
File Name: Section F-F' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 2.00

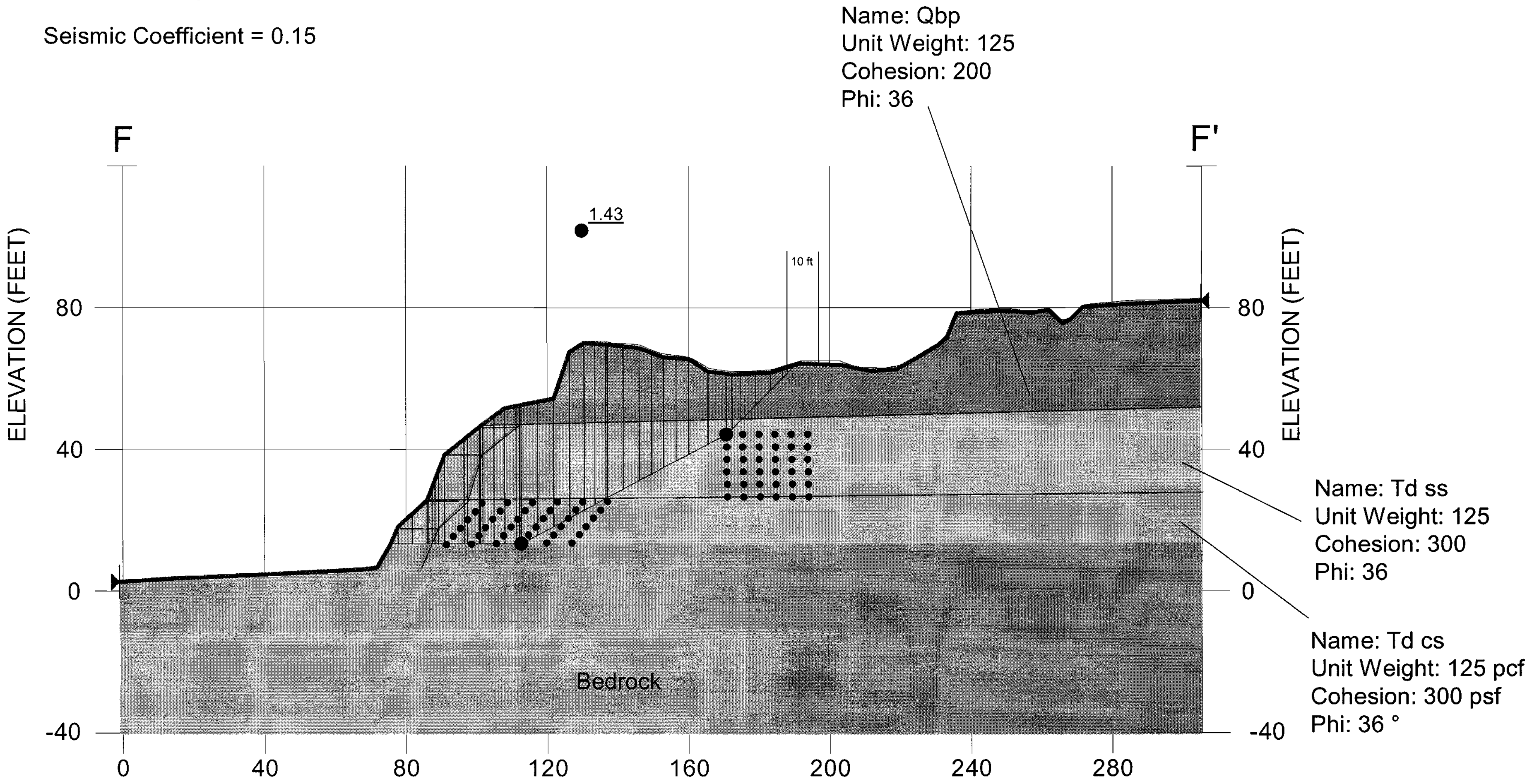
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section F-F'
Slope Stability Analysis
File Name: Section F-F' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.43

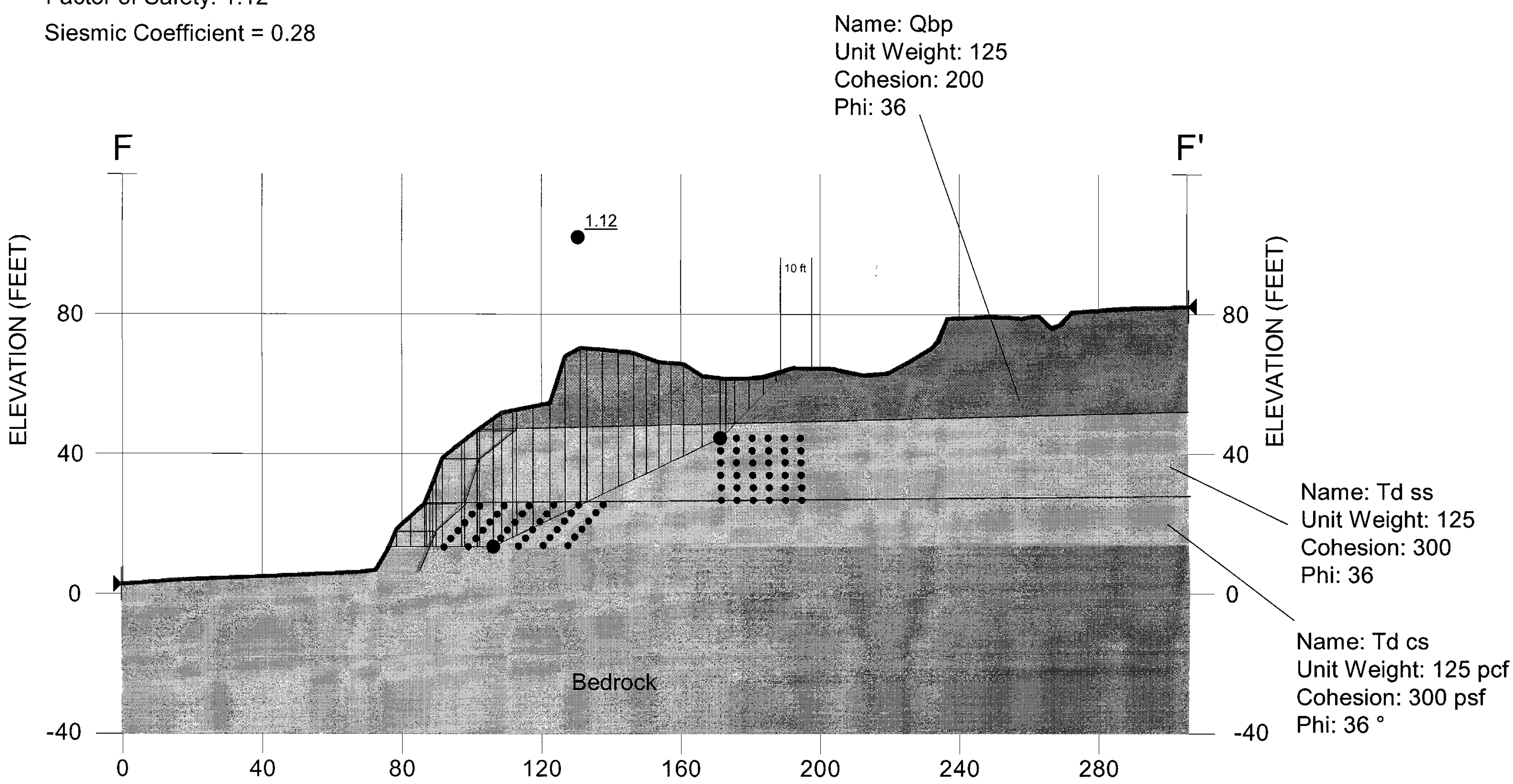
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section F-F'
Slope Stability Analysis
File Name: Section F-F' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 1.12

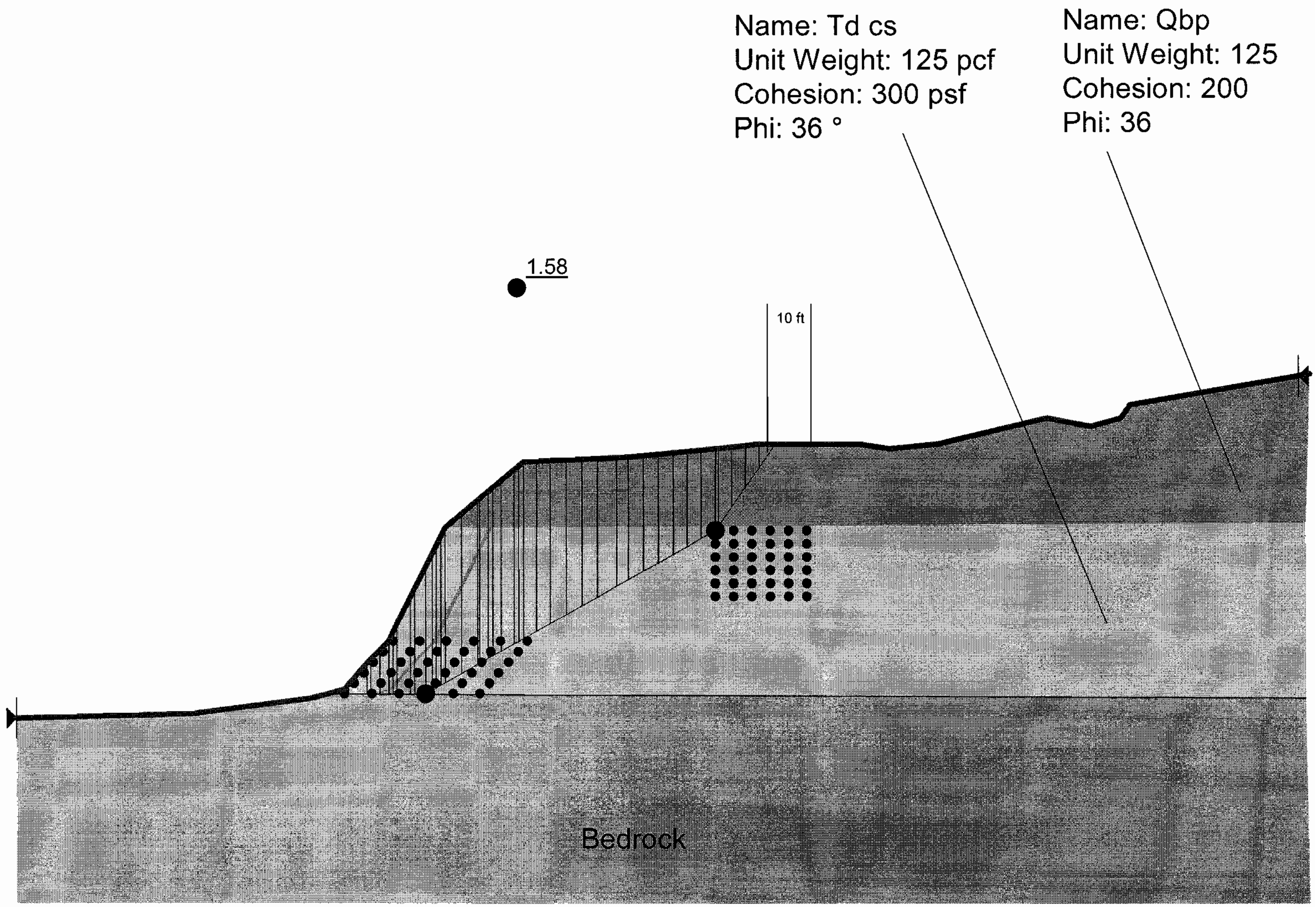
Siesmic Coefficient = 0.28



CROSS SECTION F1-F1’

Del Mar Bluffs Cross Section F1-F1'
Slope Stability Analysis
File Name: Section F1-F1' Static B.gsz
Analysis Method: Spencer

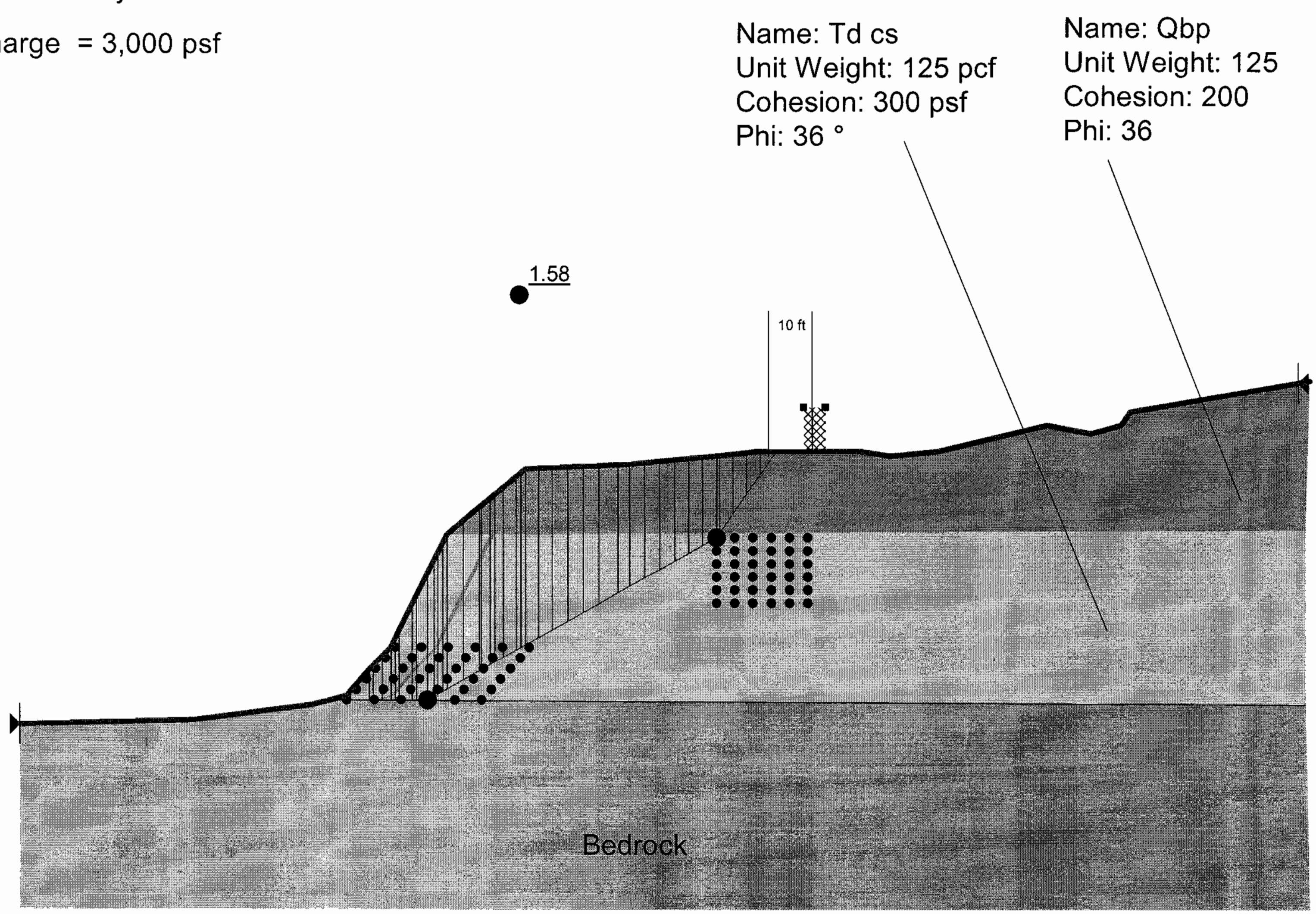
Factor of Safety: 1.58



Del Mar Bluffs Cross Section F1-F1'
Slope Stability Analysis
File Name: Section F1-F1' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.58

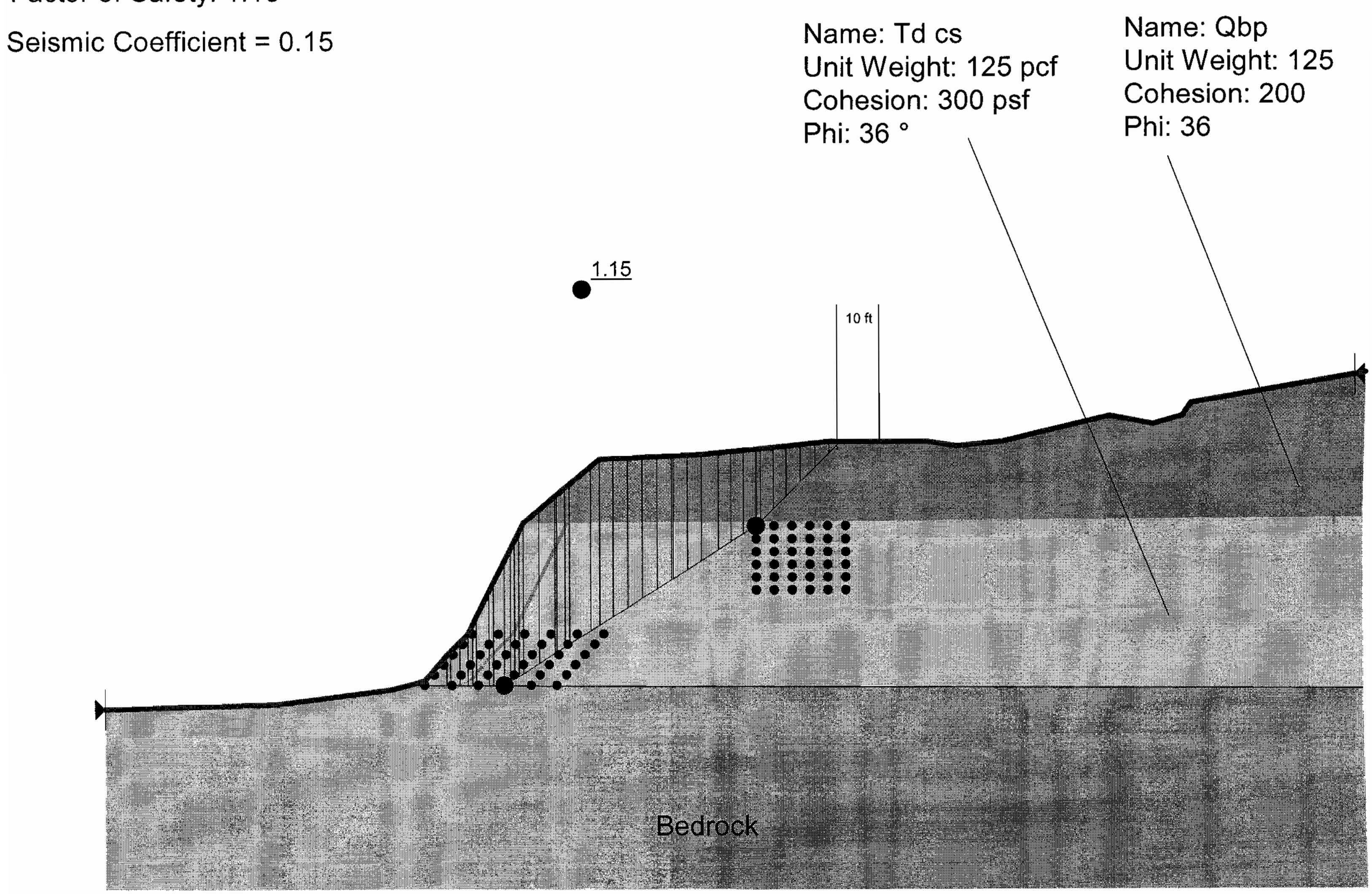
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section F1-F1'
Slope Stability Analysis
File Name: Section F1-F1' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.15

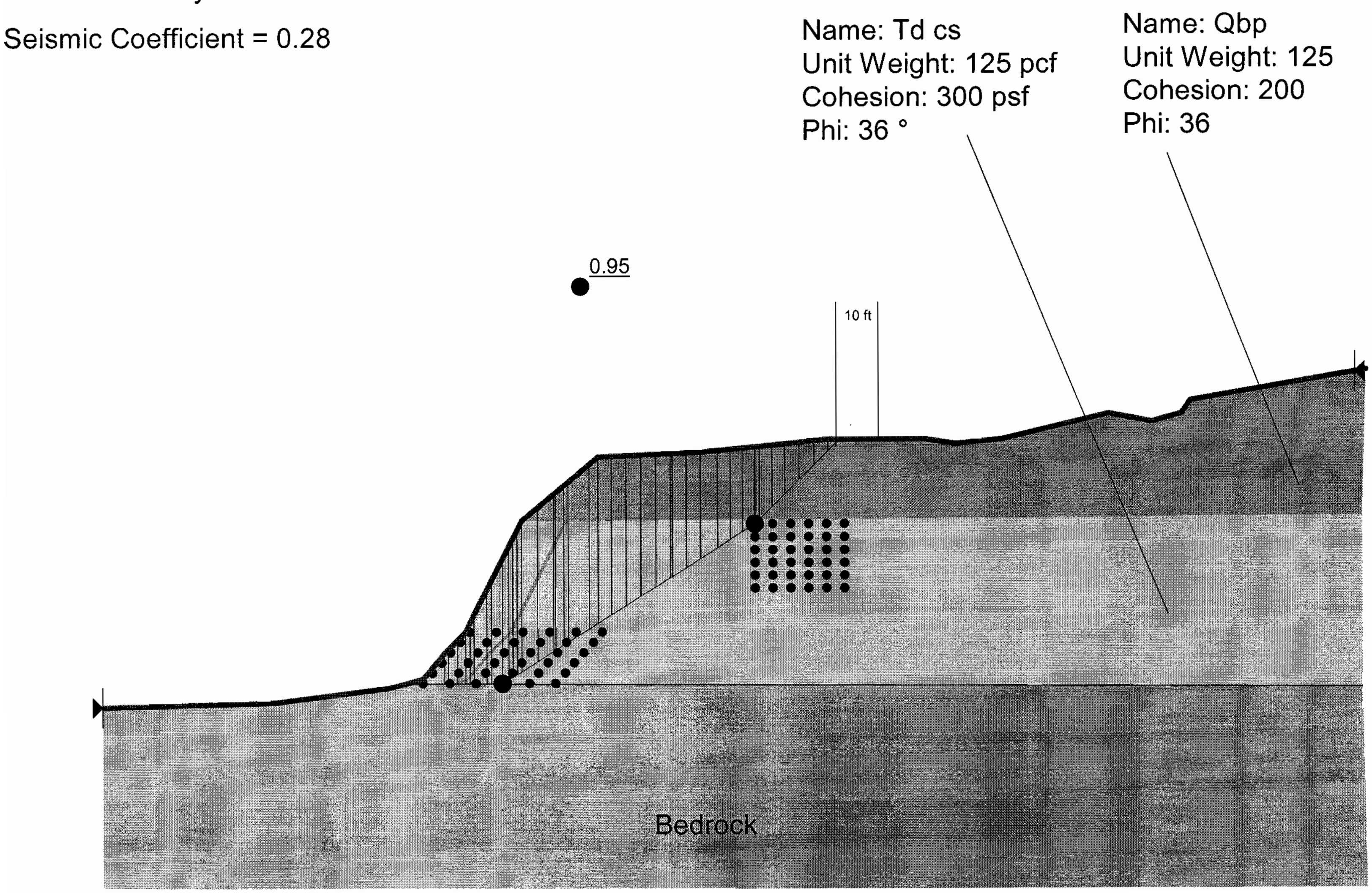
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section F1-F1'
Slope Stability Analysis
File Name: Section F1-F1' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.95

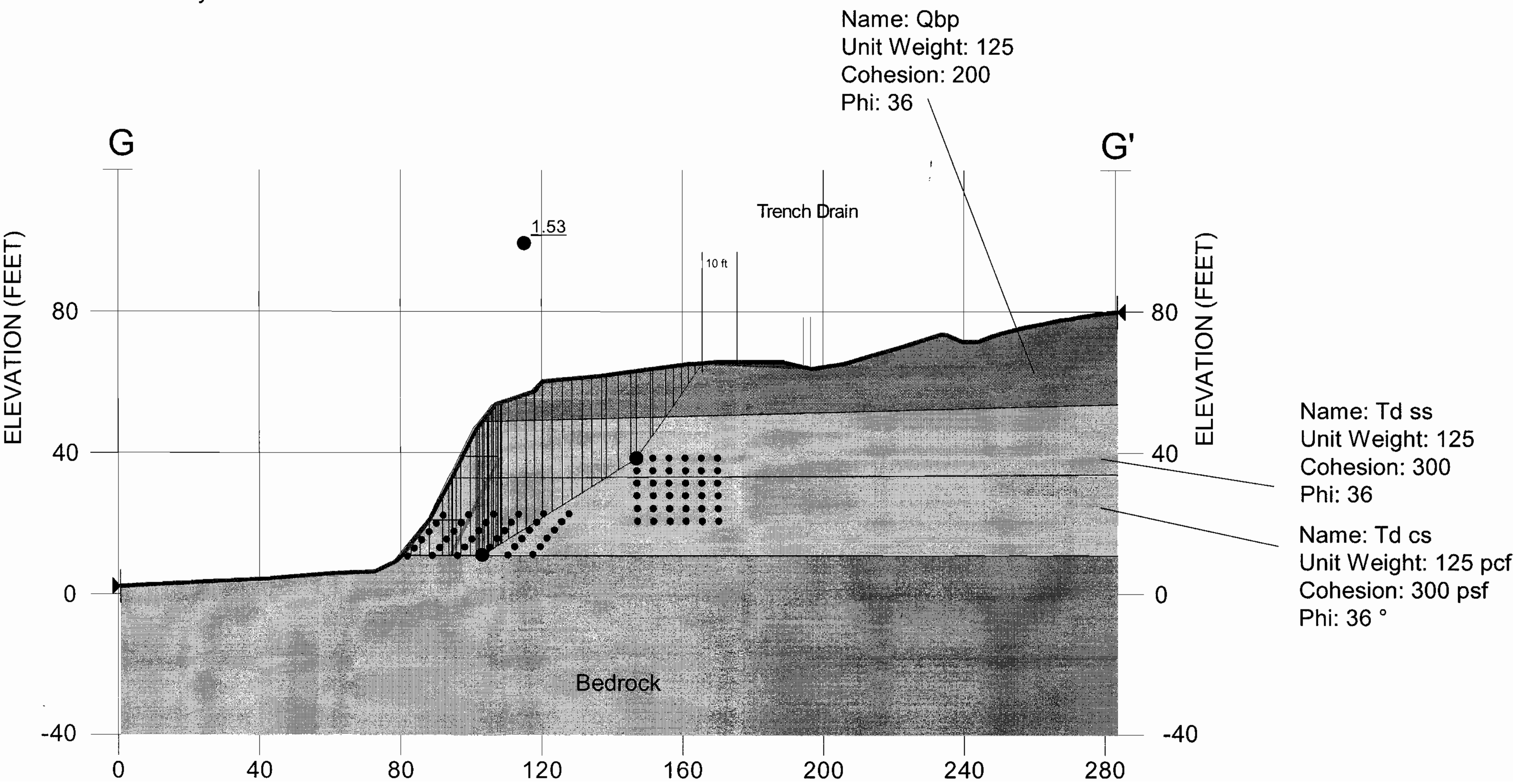
Seismic Coefficient = 0.28



CROSS SECTION G-G'

Del Mar Bluffs Cross Section G-G'
Slope Stability Analysis
File Name: Section G-G' Static B.gsz
Analysis Method: Spencer

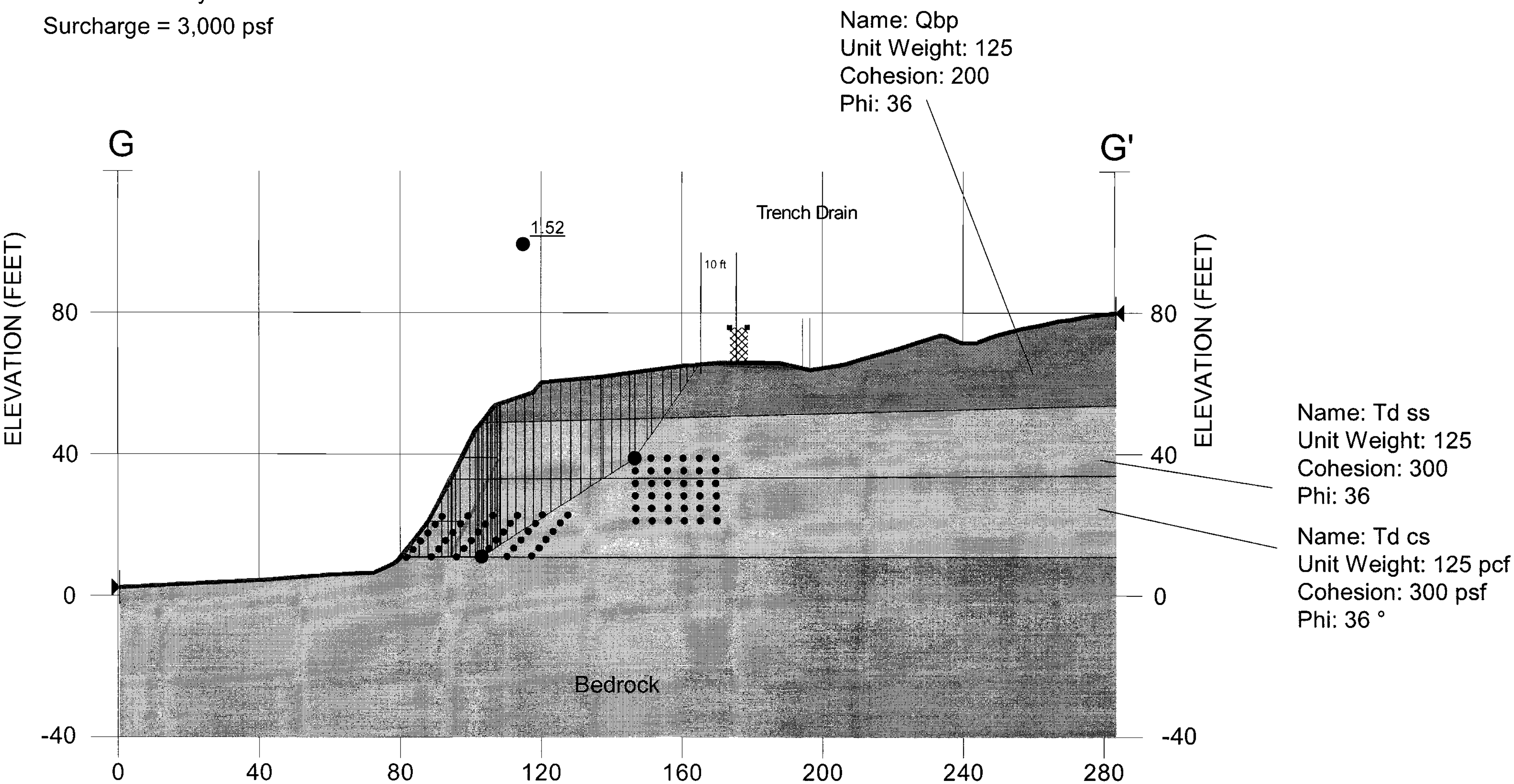
Factor of Safety: 1.53



Del Mar Bluffs Cross Section G-G'
Slope Stability Analysis
File Name: Section G-G' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.52

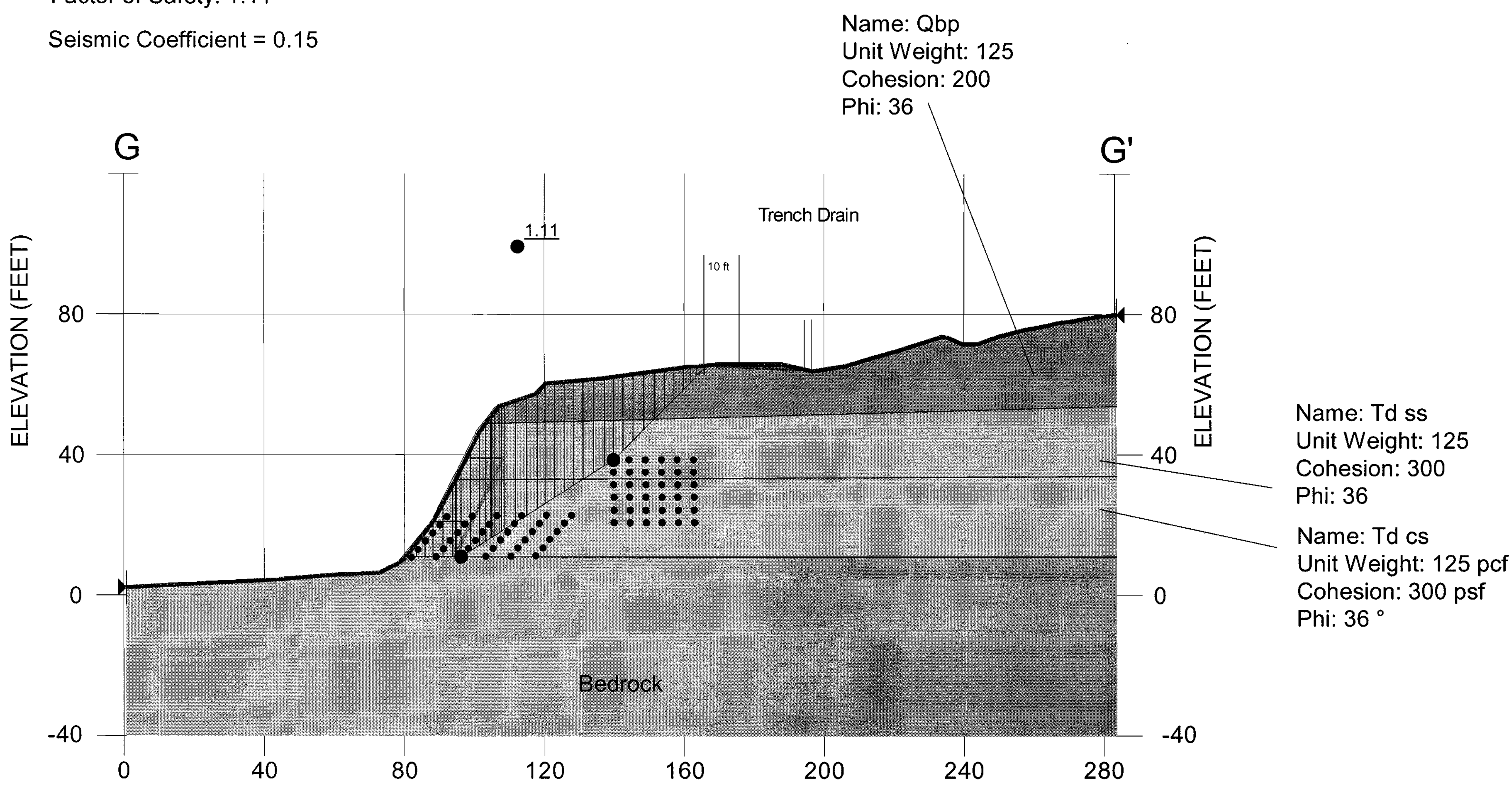
Surcharge = 3,000 psf

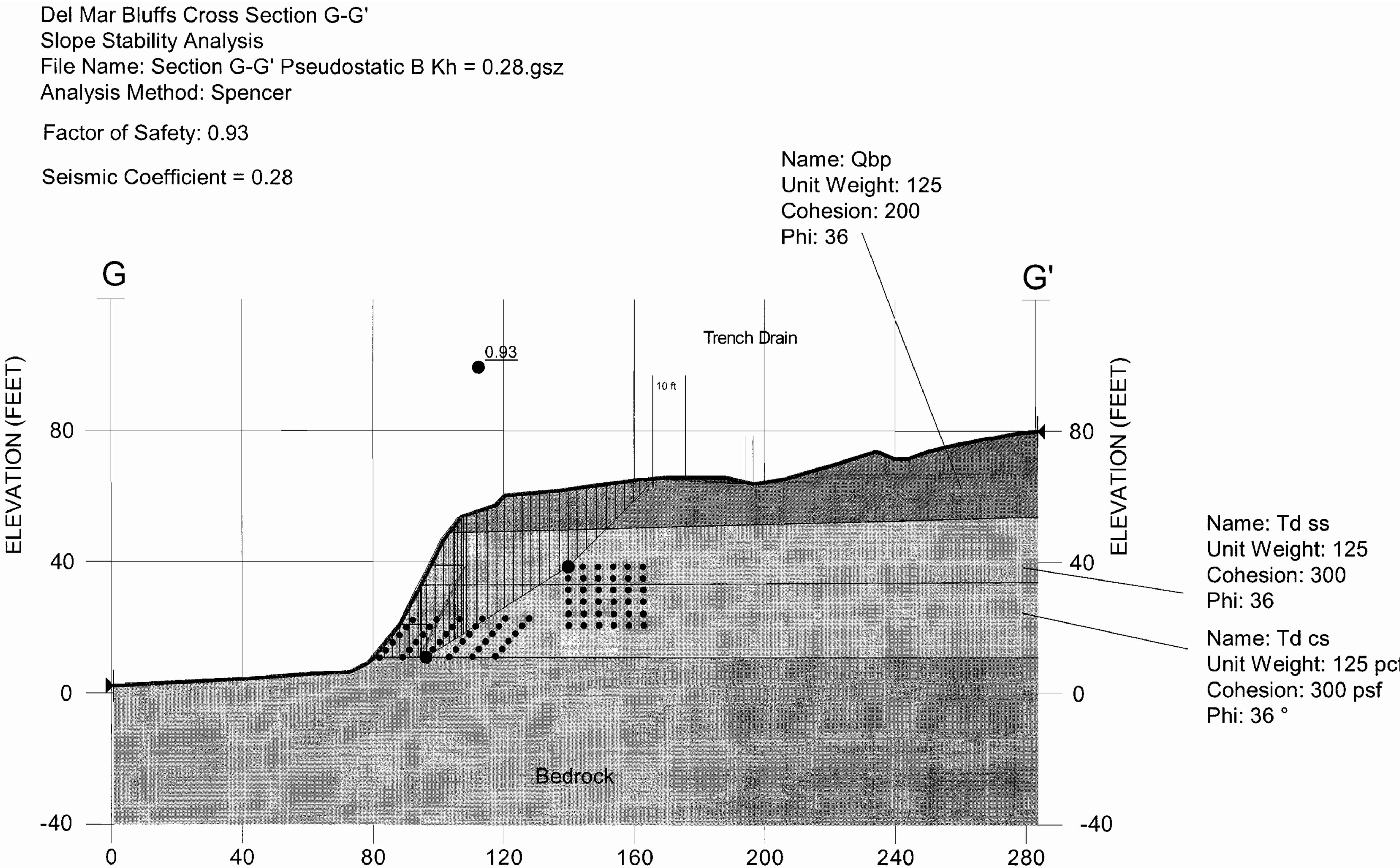


Del Mar Bluffs Cross Section G-G'
Slope Stability Analysis
File Name: Section G-G' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.11

Seismic Coefficient = 0.15



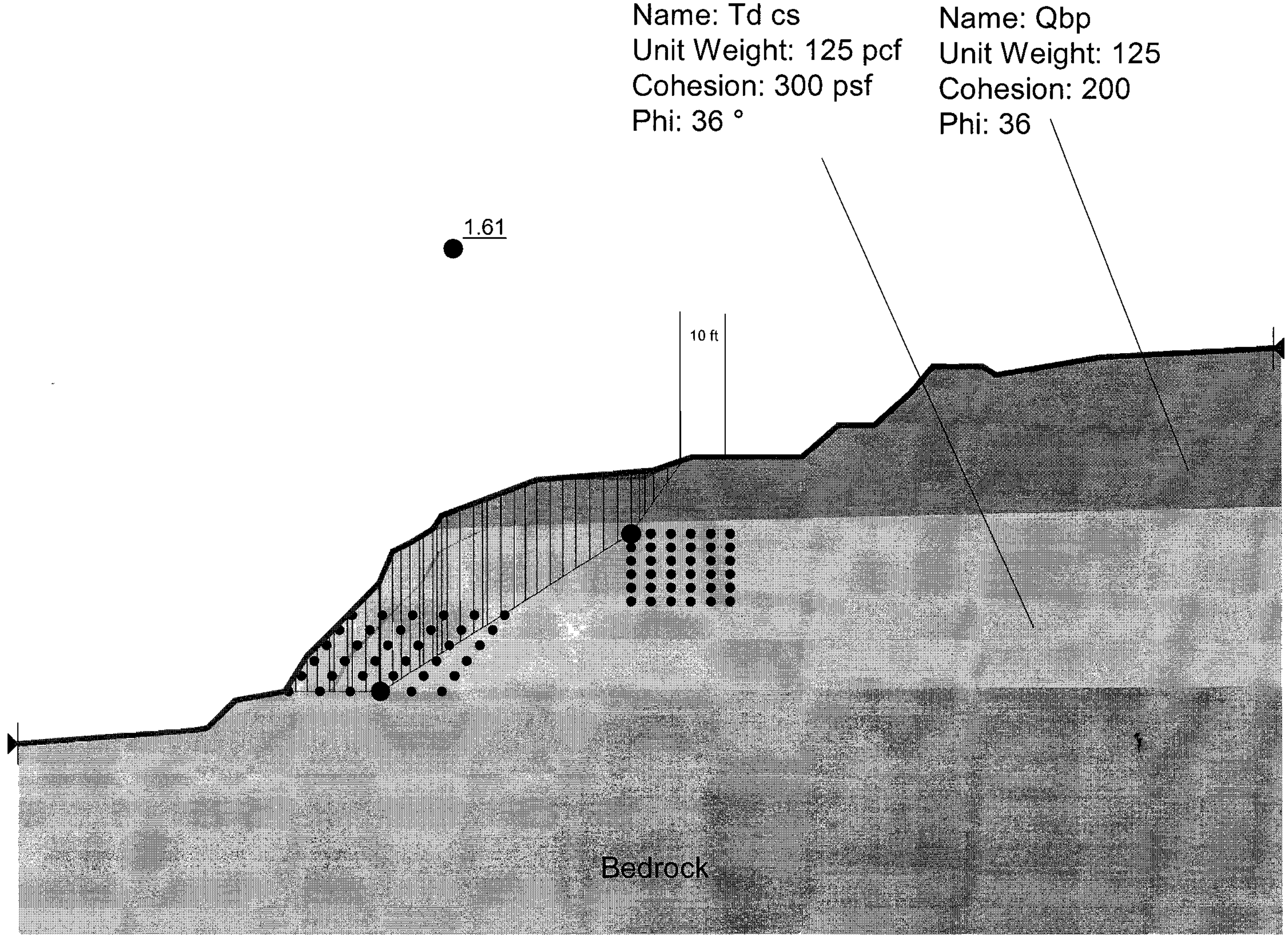


Del Mar Bluffs Cross Section G-G'
Slope Stability Analysis
File Name: Section G-G' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer
Factor of Safety: 0.93
Seismic Coefficient = 0.28
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
G
G'
Trench Drain
0.93
10 ft
ELEVATION (FEET)
80
40
0
-40
Name: Td ss
Unit Weight: 125
Cohesion: 300
Phi: 36
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Bedrock
0
40
80
120
160
200
240
280

CROSS SECTION G1-G1'

Del Mar Bluffs Cross Section G1-G1'
Slope Stability Analysis
File Name: Section G1-G1' Static B.gsz
Analysis Method: Spencer

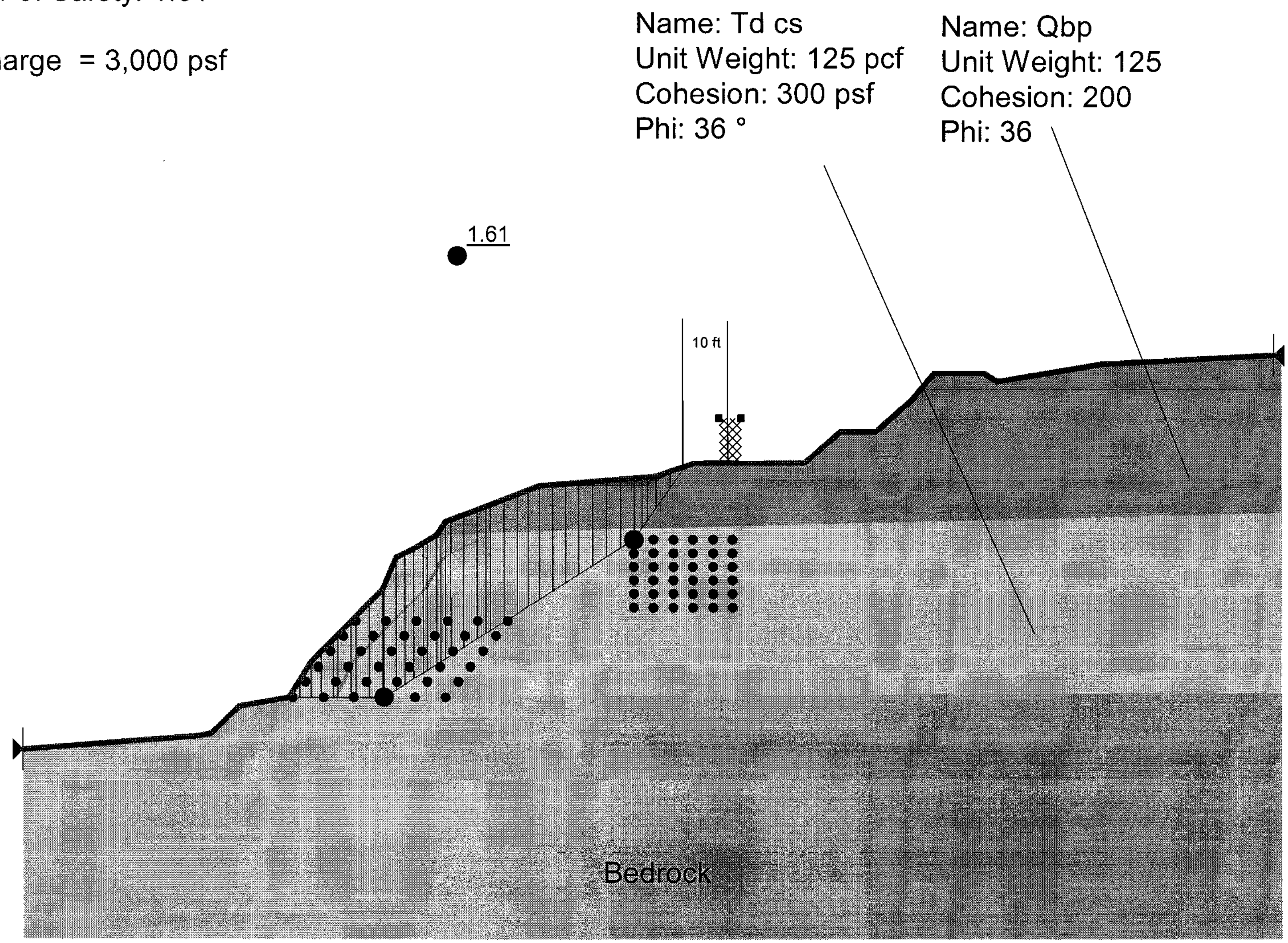
Factor of Safety: 1.61



Del Mar Bluffs Cross Section G1-G1'
Slope Stability Analysis
File Name: Section G1-G1' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.61

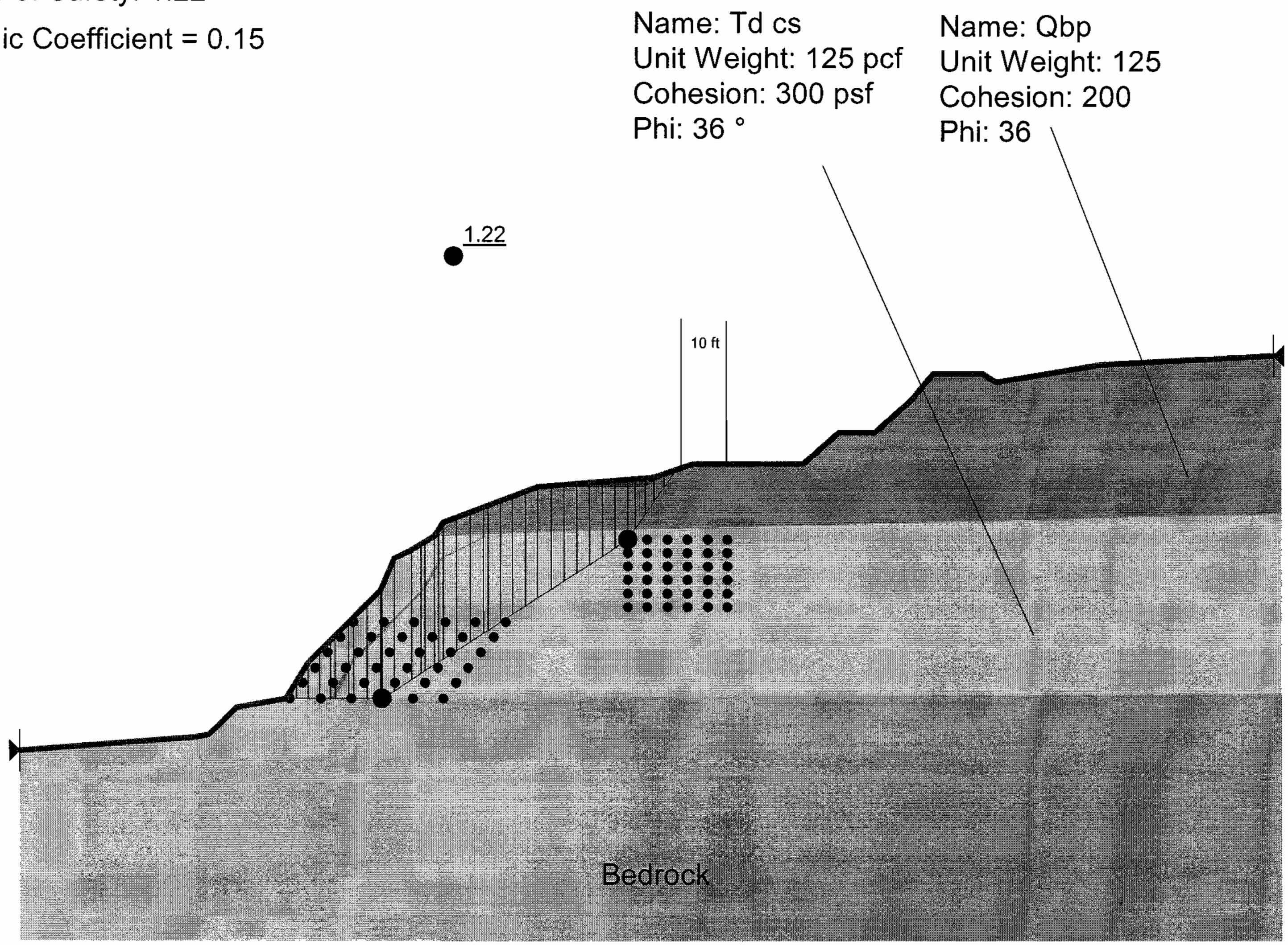
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section G1-G1'
Slope Stability Analysis
File Name: Section G1-G1' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.22

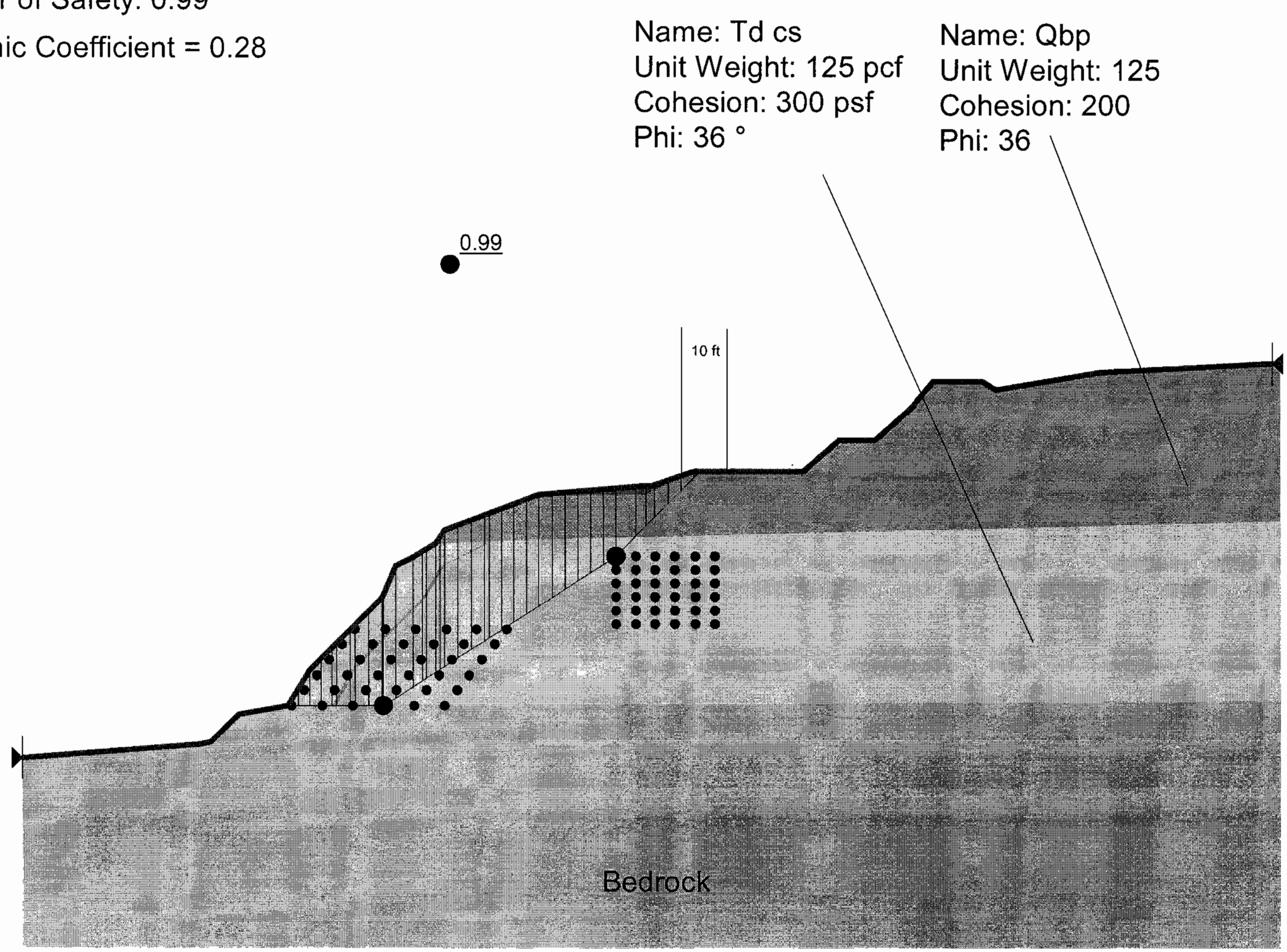
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section G1-G1'
Slope Stability Analysis
File Name: Section G1-G1' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.99

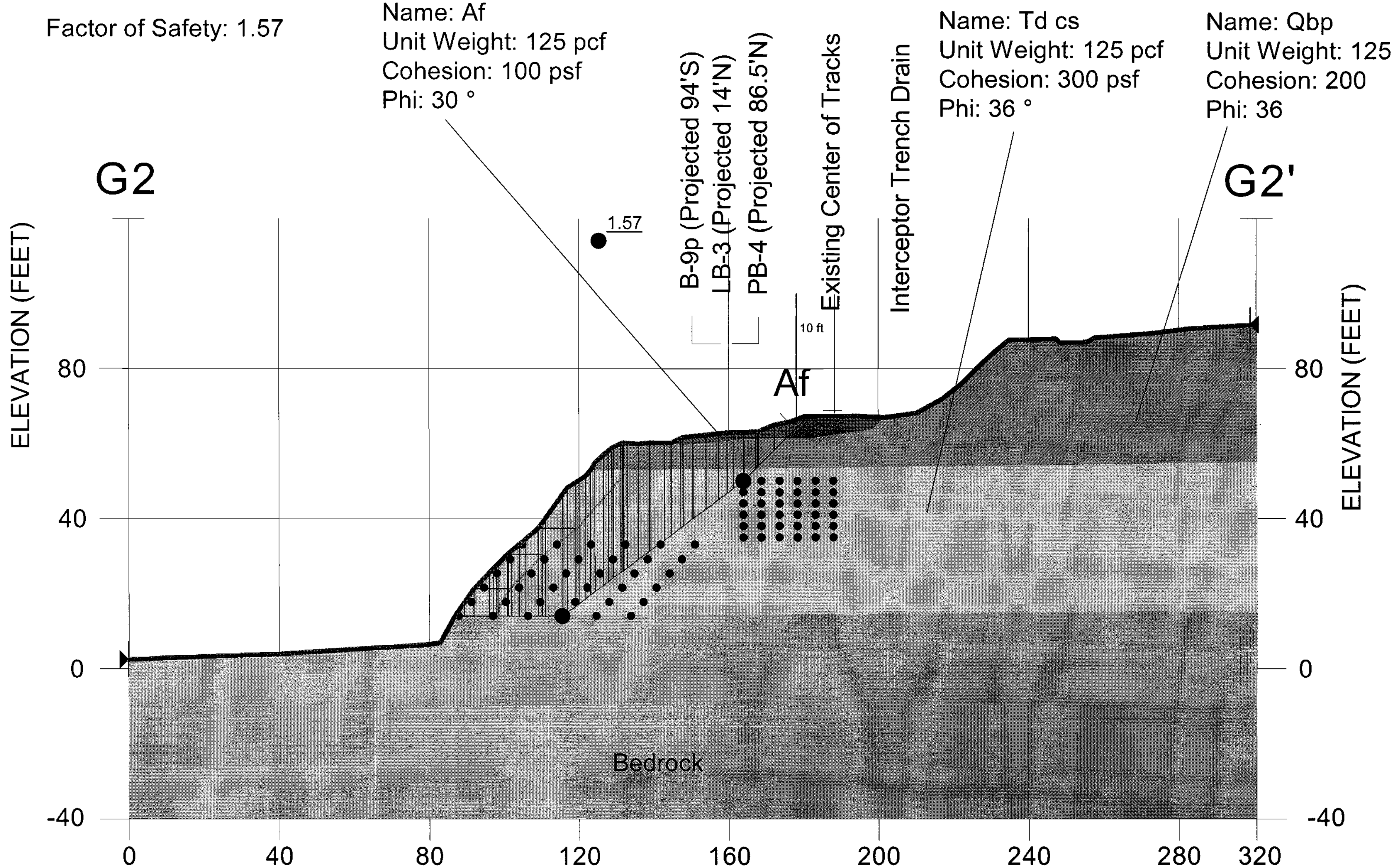
Seismic Coefficient = 0.28



CROSS SECTION G2-G2'

Del Mar Bluffs Cross Section G2-G2'
Slope Stability Analysis
File Name: Section G2-G2' Static B.gsz
Analysis Method: Spencer

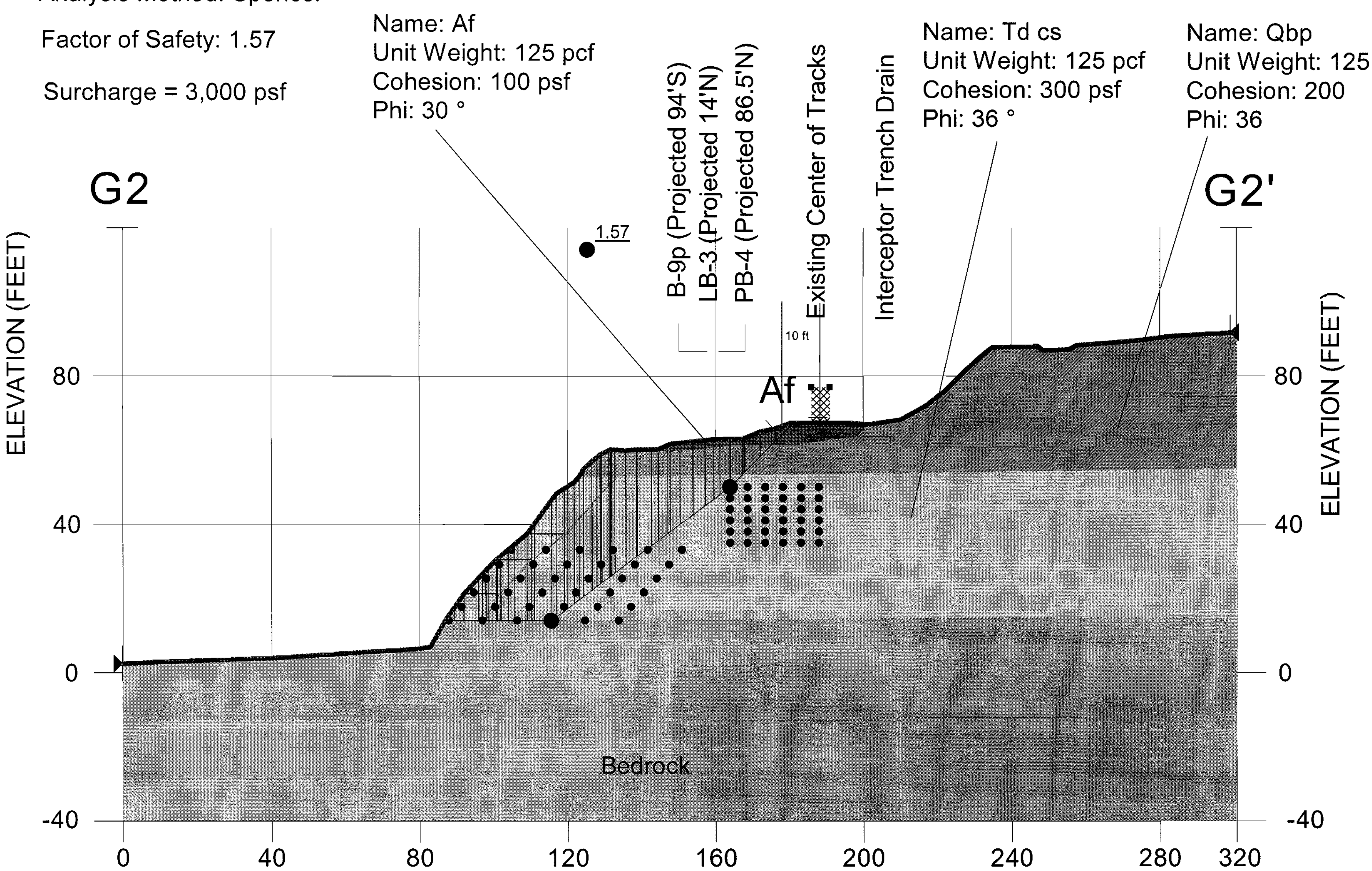
Factor of Safety: 1.57



Del Mar Bluffs Cross Section G2-G2'
Slope Stability Analysis
File Name: Section G2-G2' Static B + Surcharge.gsz
Analysis Method: Spencer

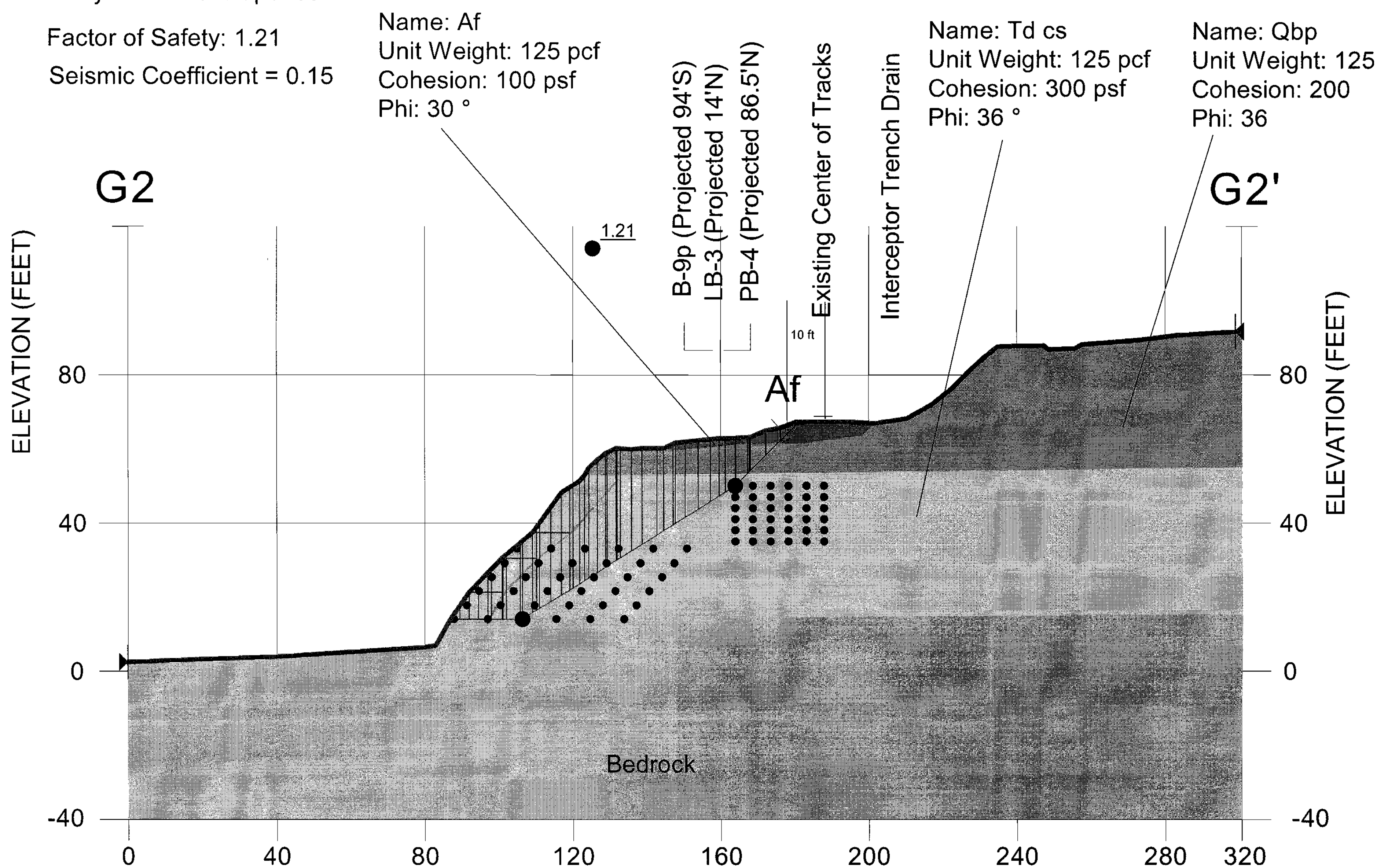
Factor of Safety: 1.57

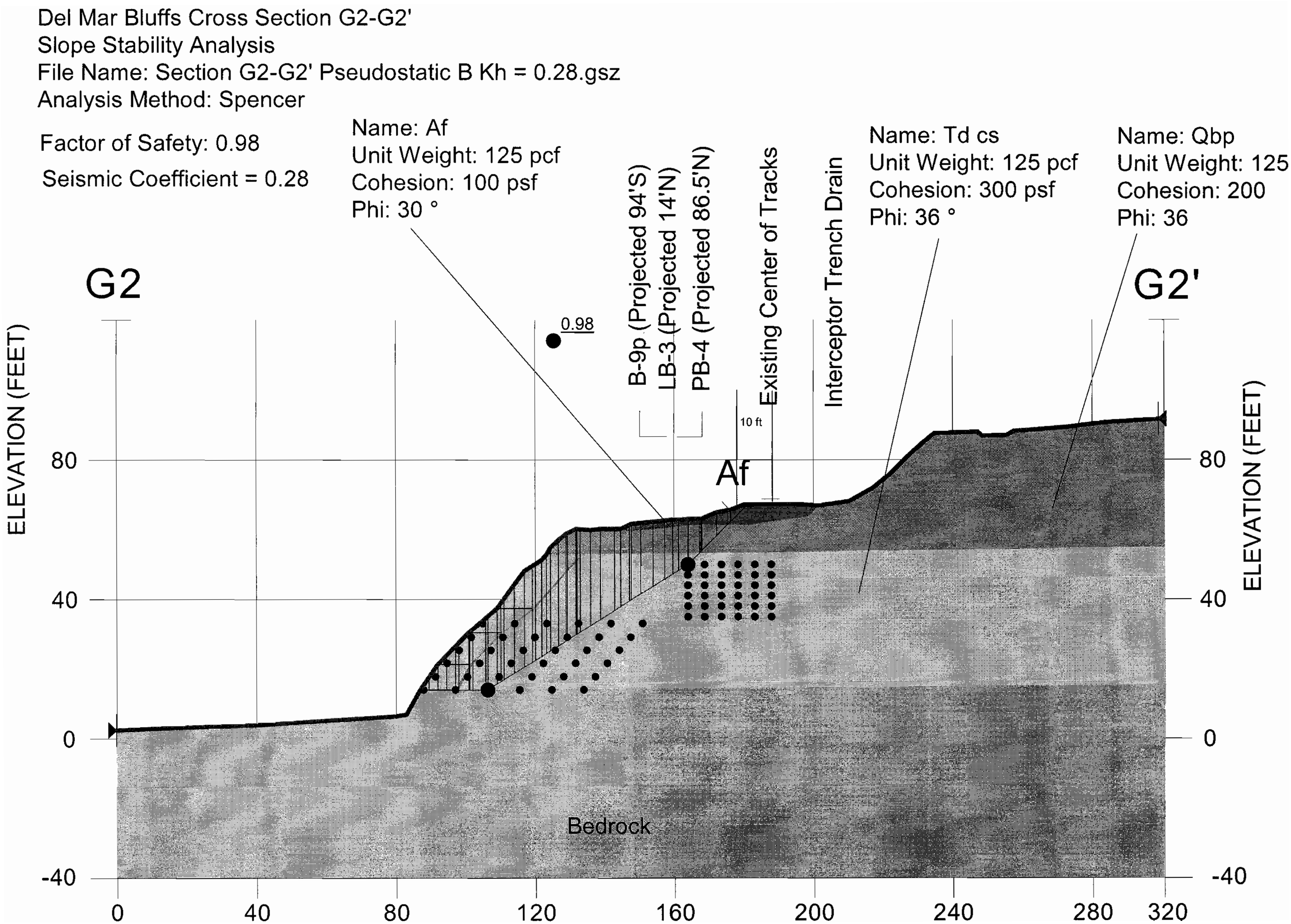
Surcharge = 3,000 psf



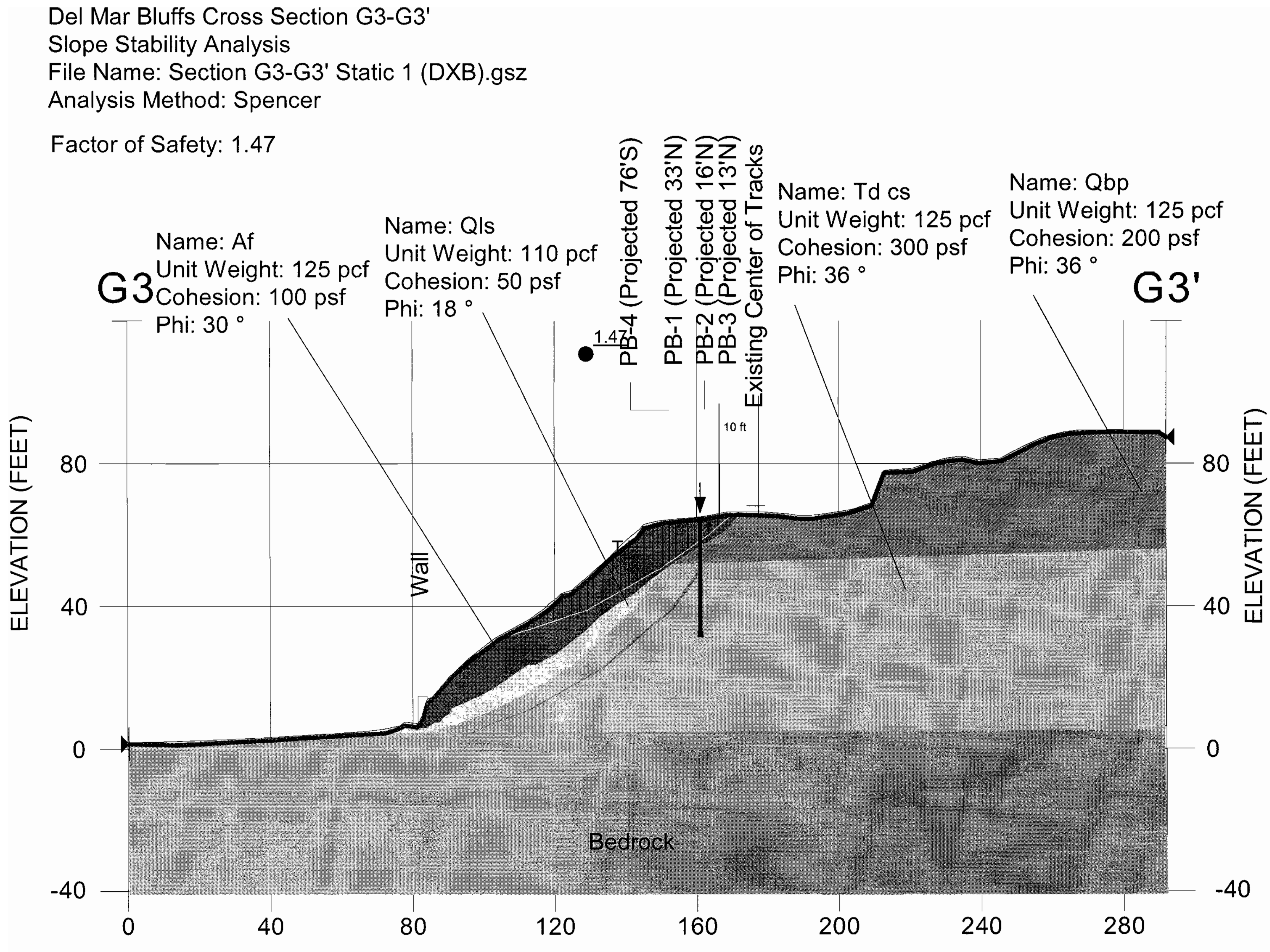
Del Mar Bluffs Cross Section G2-G2'
Slope Stability Analysis
File Name: Section G2-G2' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.21
Seismic Coefficient = 0.15




Del Mar Bluffs Cross Section G2-G2'
Slope Stability Analysis
File Name: Section G2-G2' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer
Factor of Safety: 0.98
Seismic Coefficient = 0.28
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
B-9p (Projected 94'S)
LB-3 (Projected 14'N)
PB-4 (Projected 86.5'N)
Existing Center of Tracks
Interceptor Trench Drain
10 ft
Af
G2
G2'
0.98
Bedrock
ELEVATION (FEET)
80
40
0
-40
0
40
80
120
160
200
240
280
320

CROSS SECTION G3-G3'

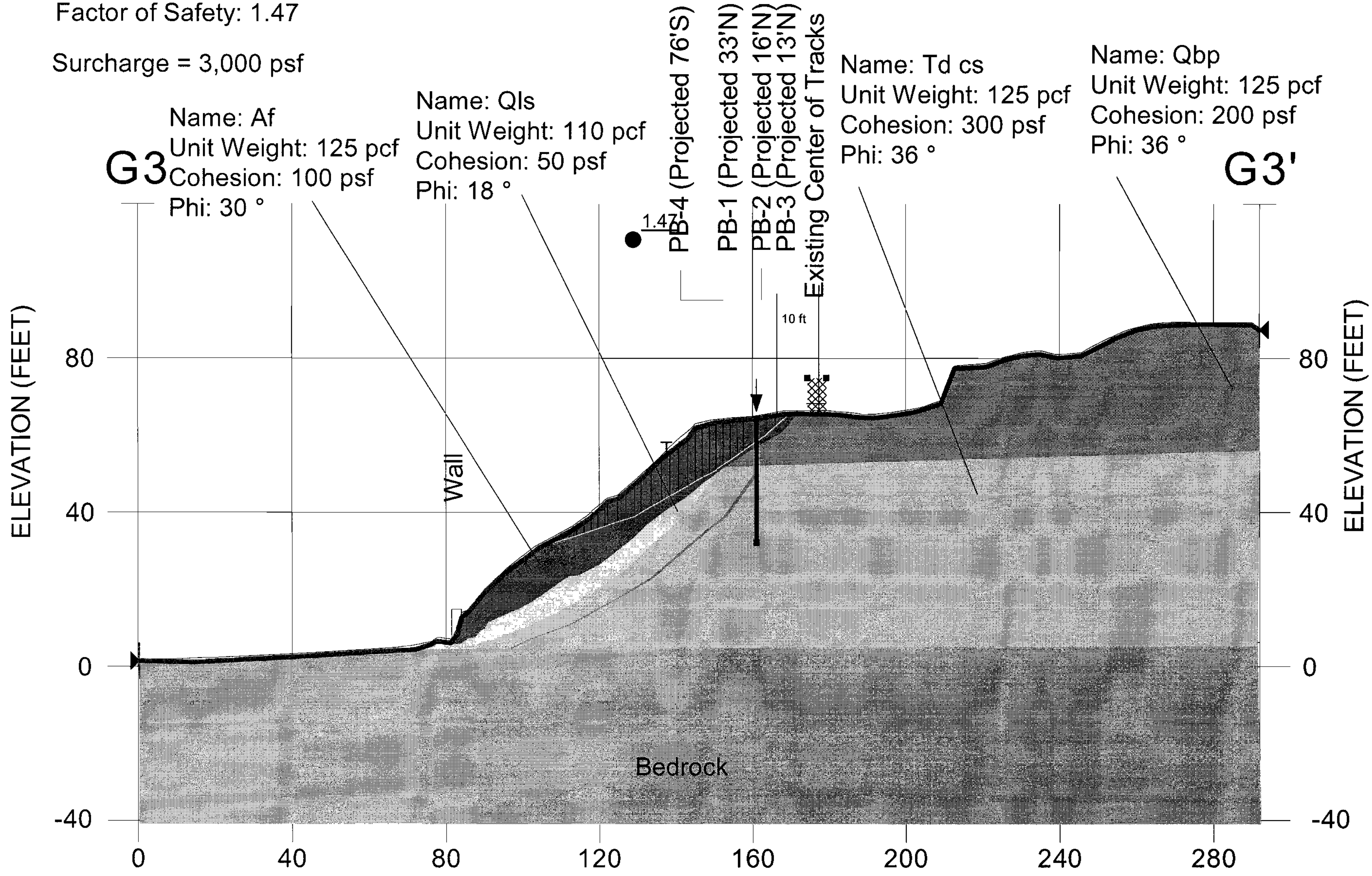


Del Mar Bluffs Cross Section G3-G3'
Slope Stability Analysis
File Name: Section G3-G3' Static 1 (DXB).gsz
Analysis Method: Spencer
Factor of Safety: 1.47
G3
G3'
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Name: Qbp
Unit Weight: 125 pcf
Cohesion: 200 psf
Phi: 36 °
1.47
PB-4 (Projected 76'S)
PB-1 (Projected 33'N)
PB-2 (Projected 16'N)
PB-3 (Projected 13'N)
Existing Center of Tracks
10 ft
Wall
Bedrock
ELEVATION (FEET)
80
40
0
-40
0
40
80
120
160
200
240
280

Del Mar Bluffs Cross Section G3-G3'
Slope Stability Analysis
File Name: Section G3-G3' Static 1 + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.47

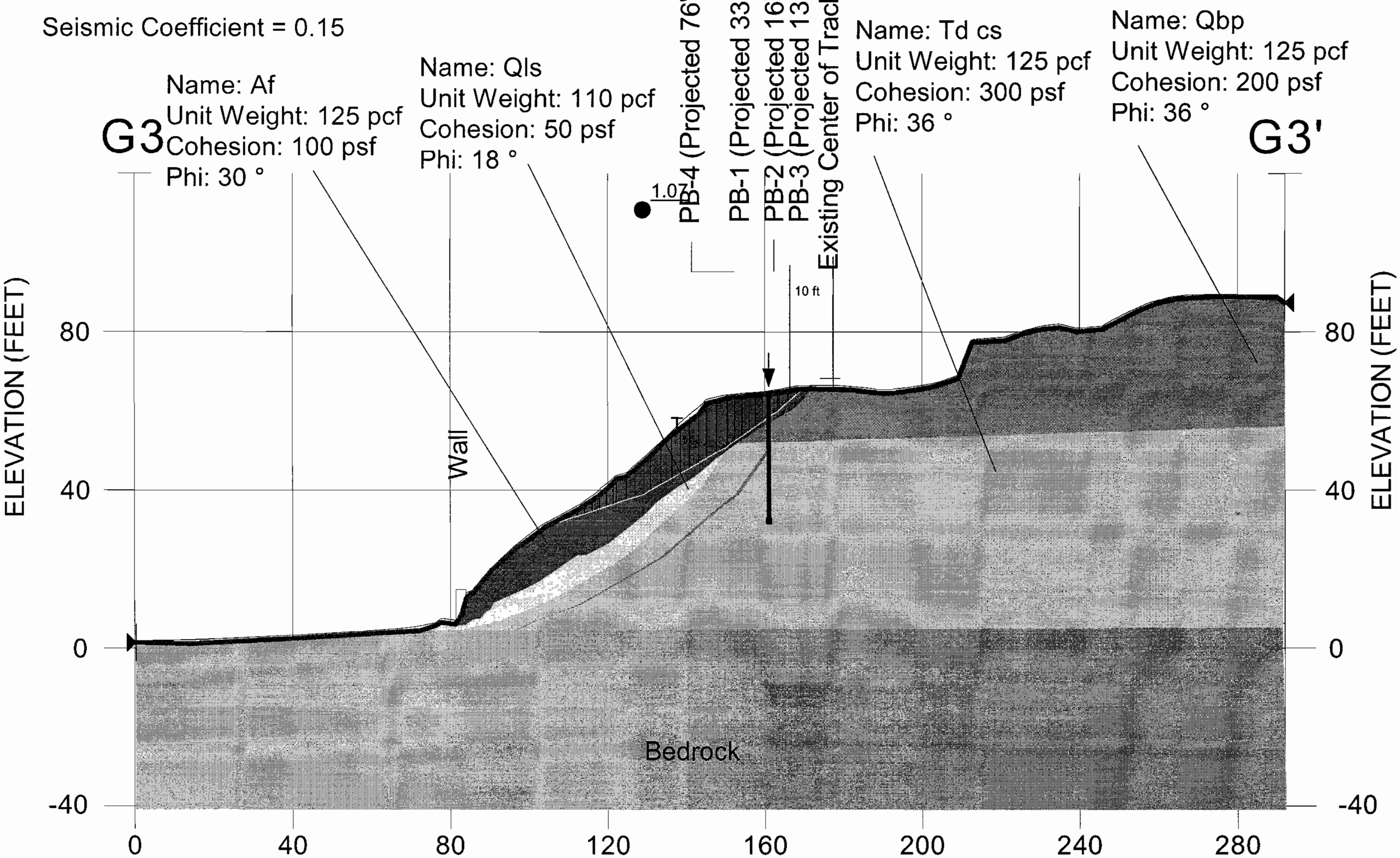
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section G3-G3'
Slope Stability Analysis
File Name: Section G3-G3' Pseudostatic Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.07

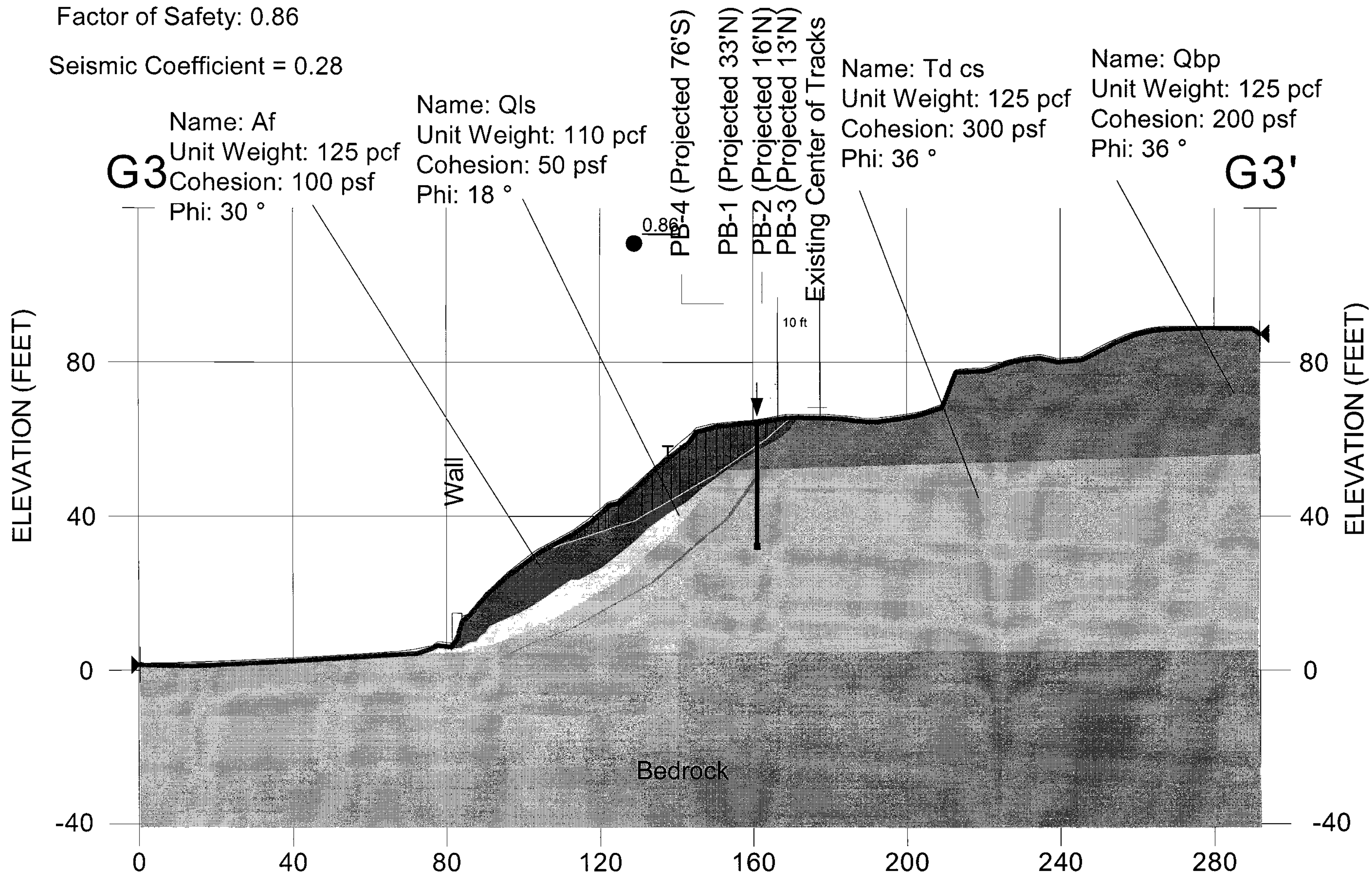
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section G3-G3'
Slope Stability Analysis
File Name: Section G3-G3' Pseudostatic Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.86

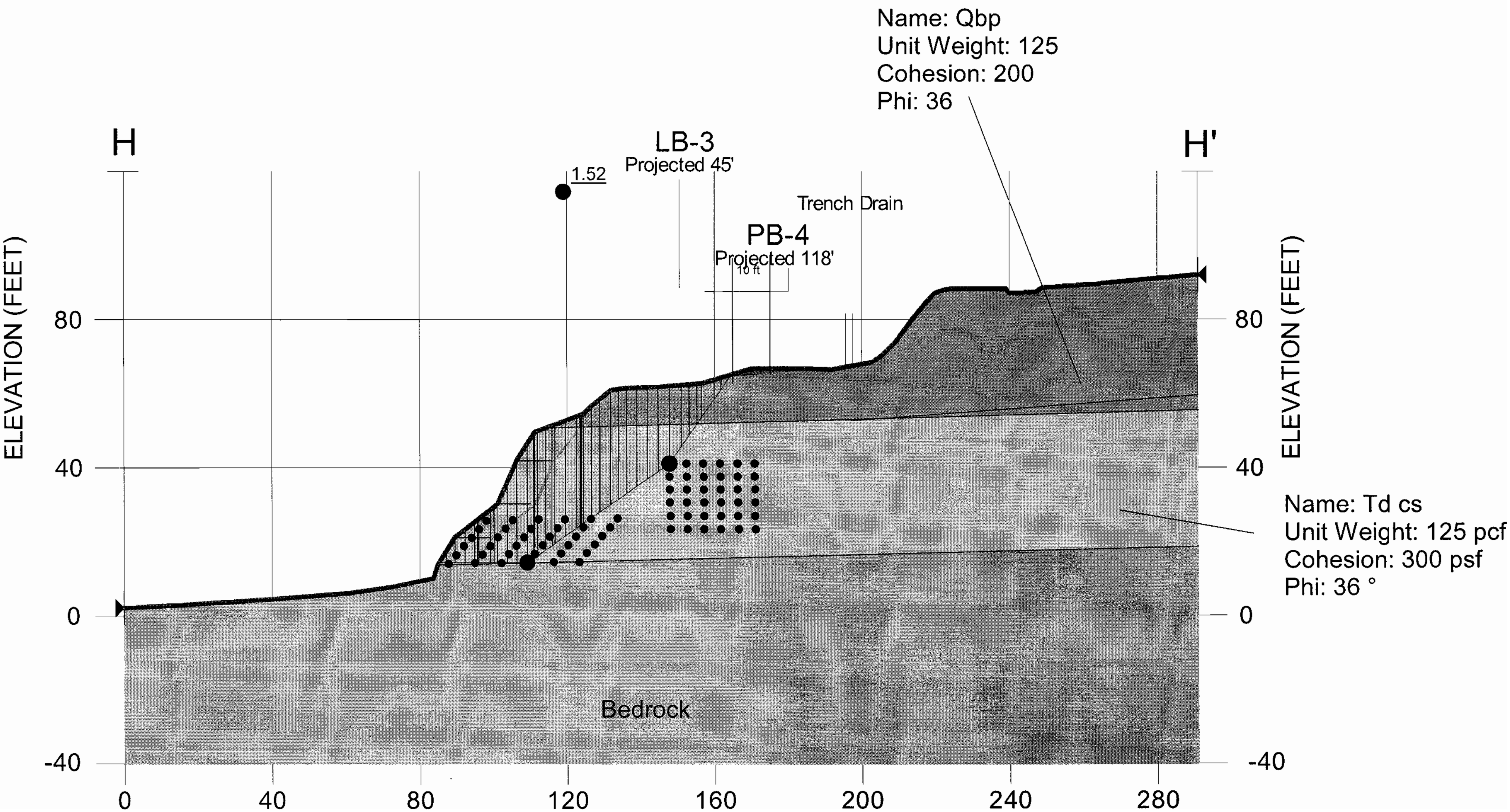
Seismic Coefficient = 0.28



CROSS SECTION H-H’

Del Mar Bluffs Cross Section H-H'
Slope Stability Analysis
File Name: Section H-H' Static B.gsz
Analysis Method: Spencer

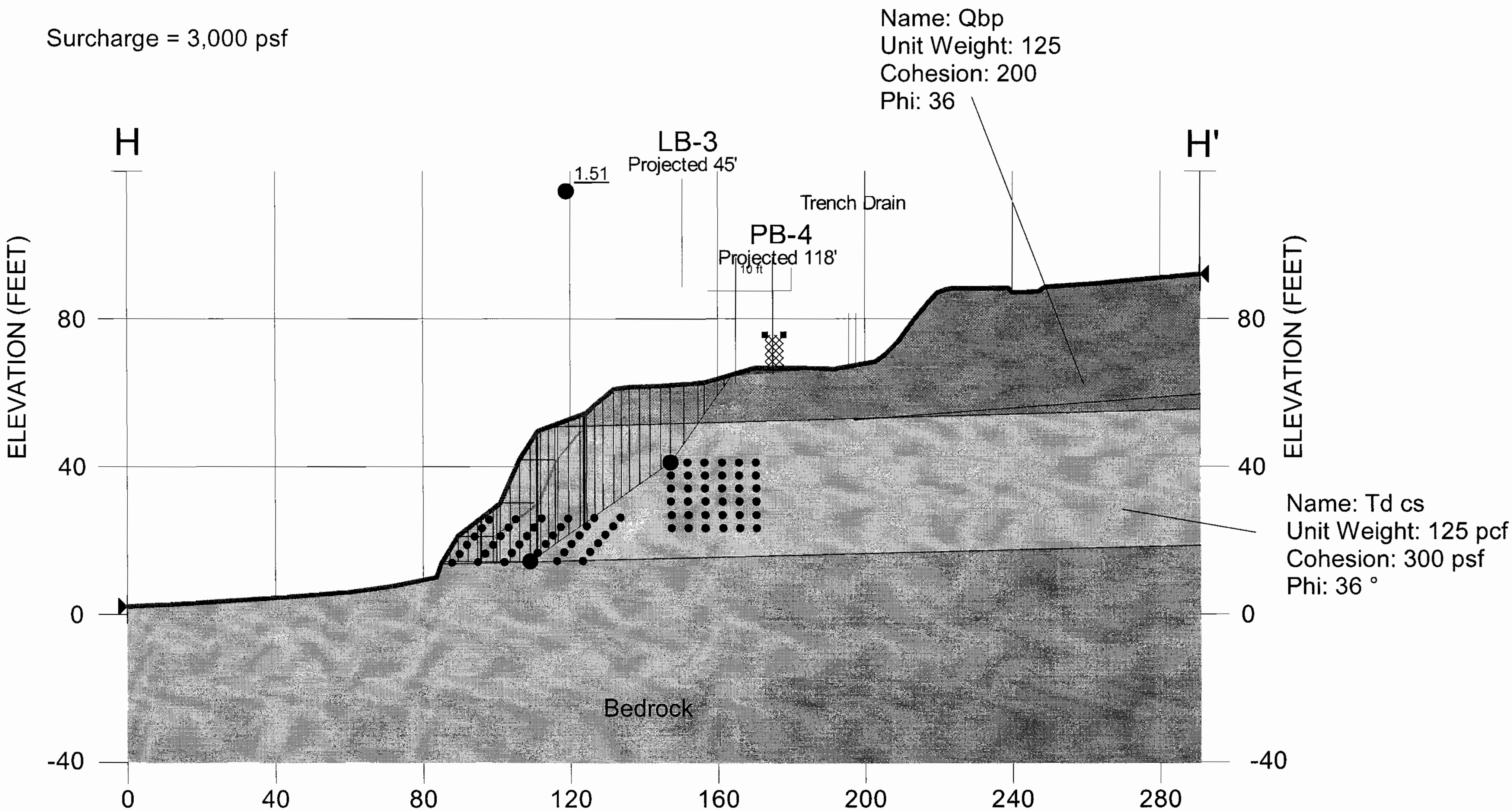
Factor of Safety: 1.52



Del Mar Bluffs Cross Section H-H'
Slope Stability Analysis
File Name: Section H-H' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.51

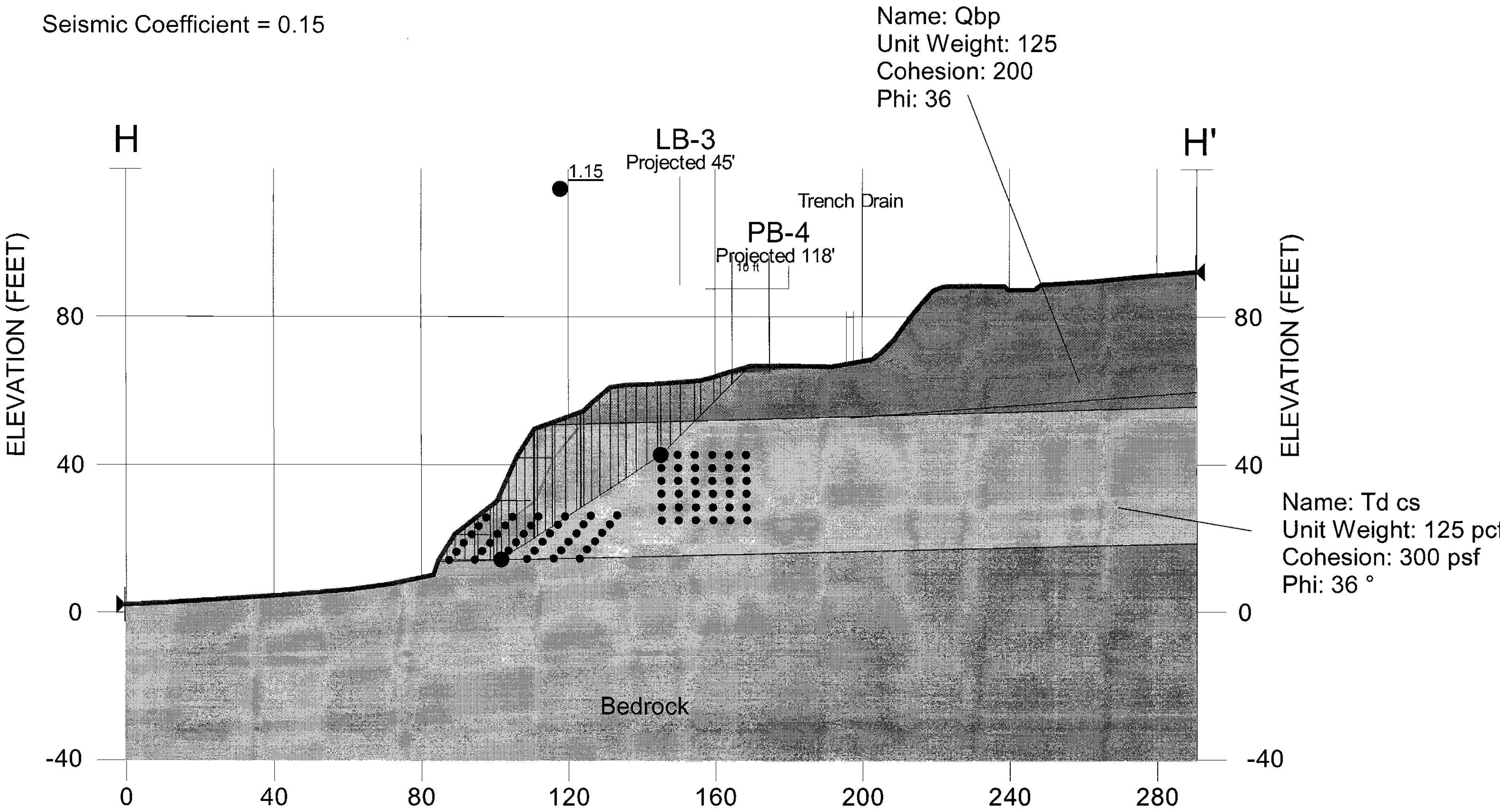
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section H-H'
Slope Stability Analysis
File Name: Section H-H' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.15

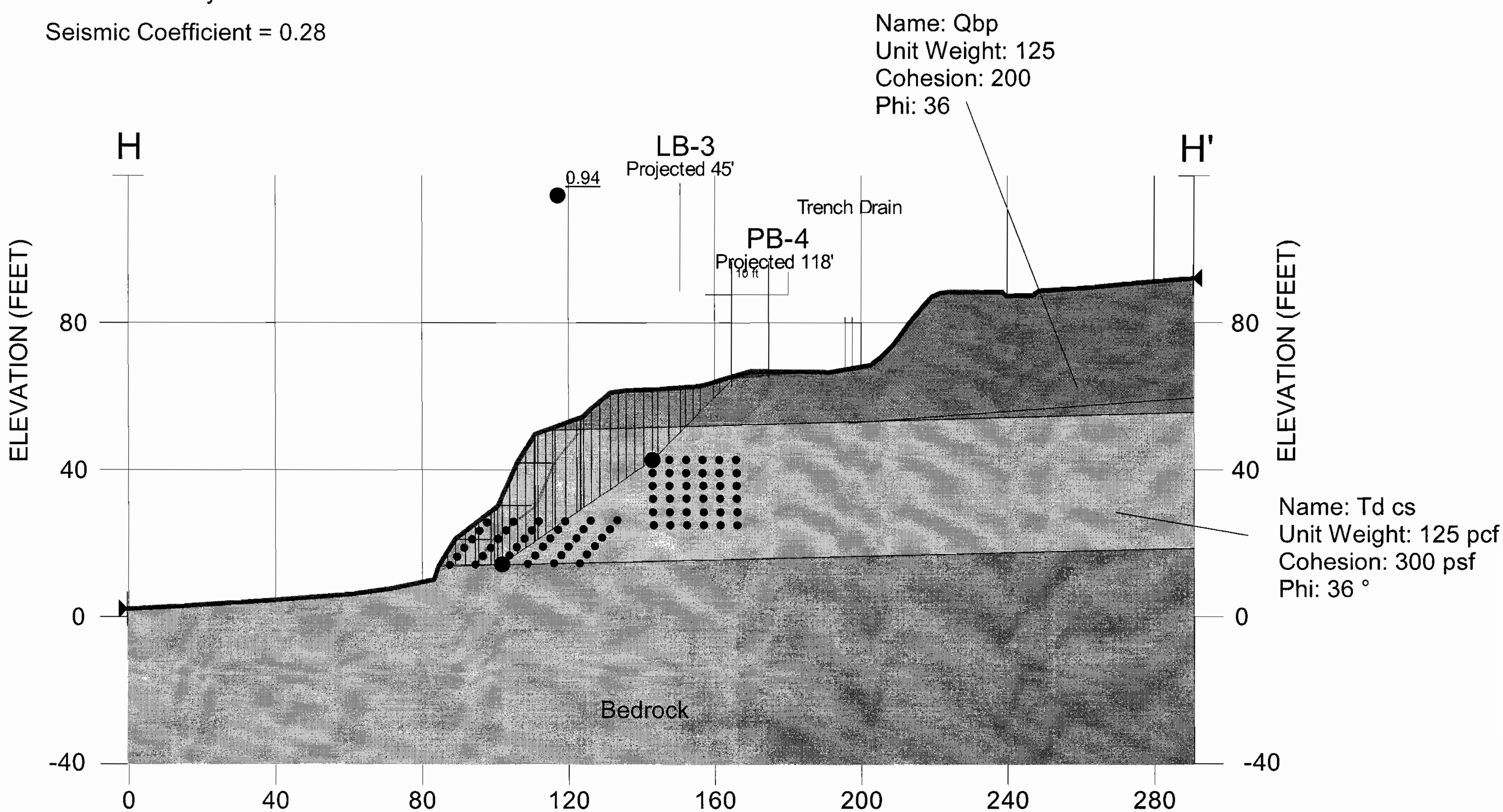
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section H-H'
Slope Stability Analysis
File Name: Section H-H' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.94

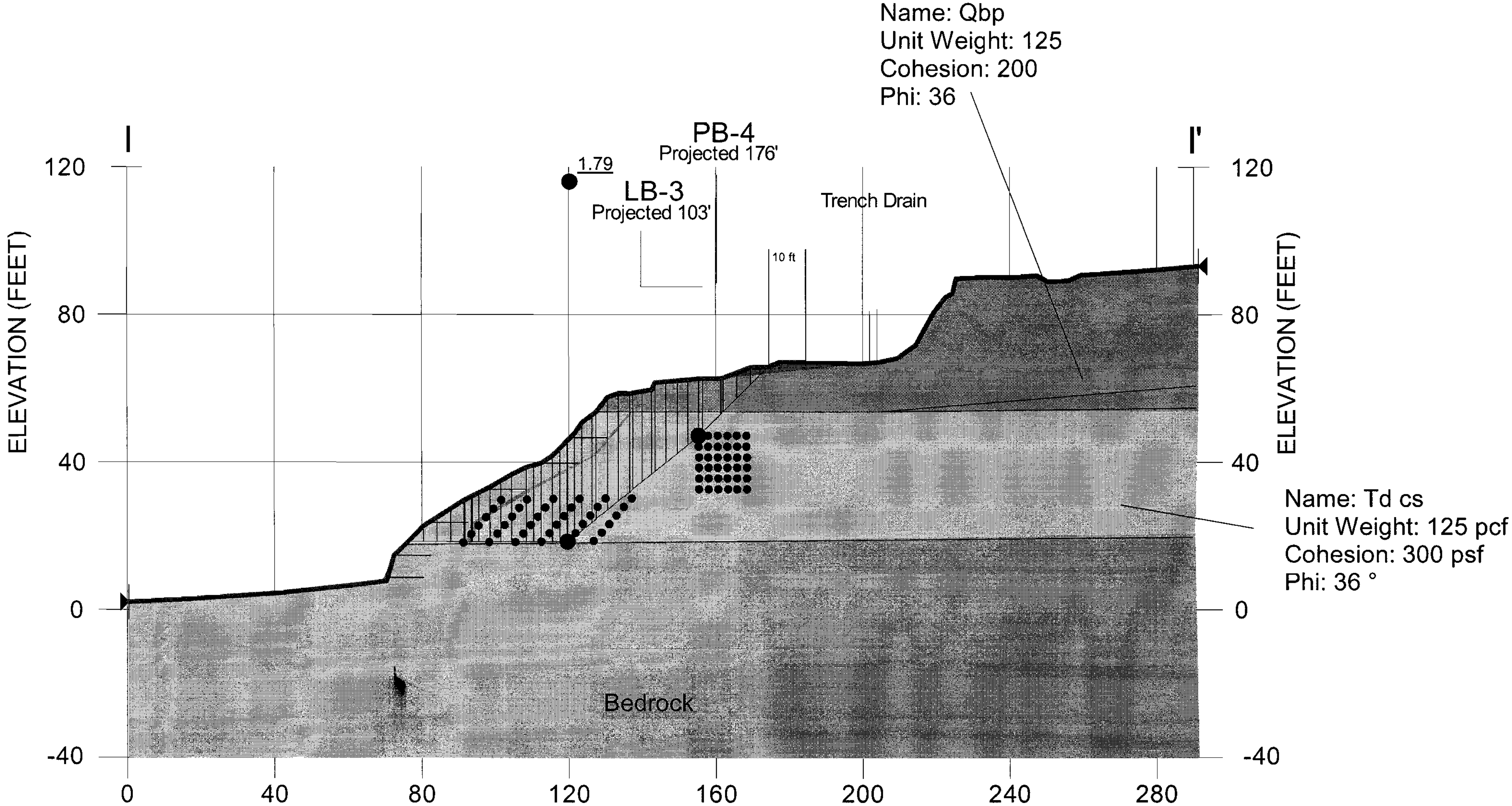
Seismic Coefficient = 0.28



CROSS SECTION I-I'

Del Mar Bluffs Cross Section I-I'
Slope Stability Analysis
File Name: Section I-I' Static B.gsz
Analysis Method: Spencer

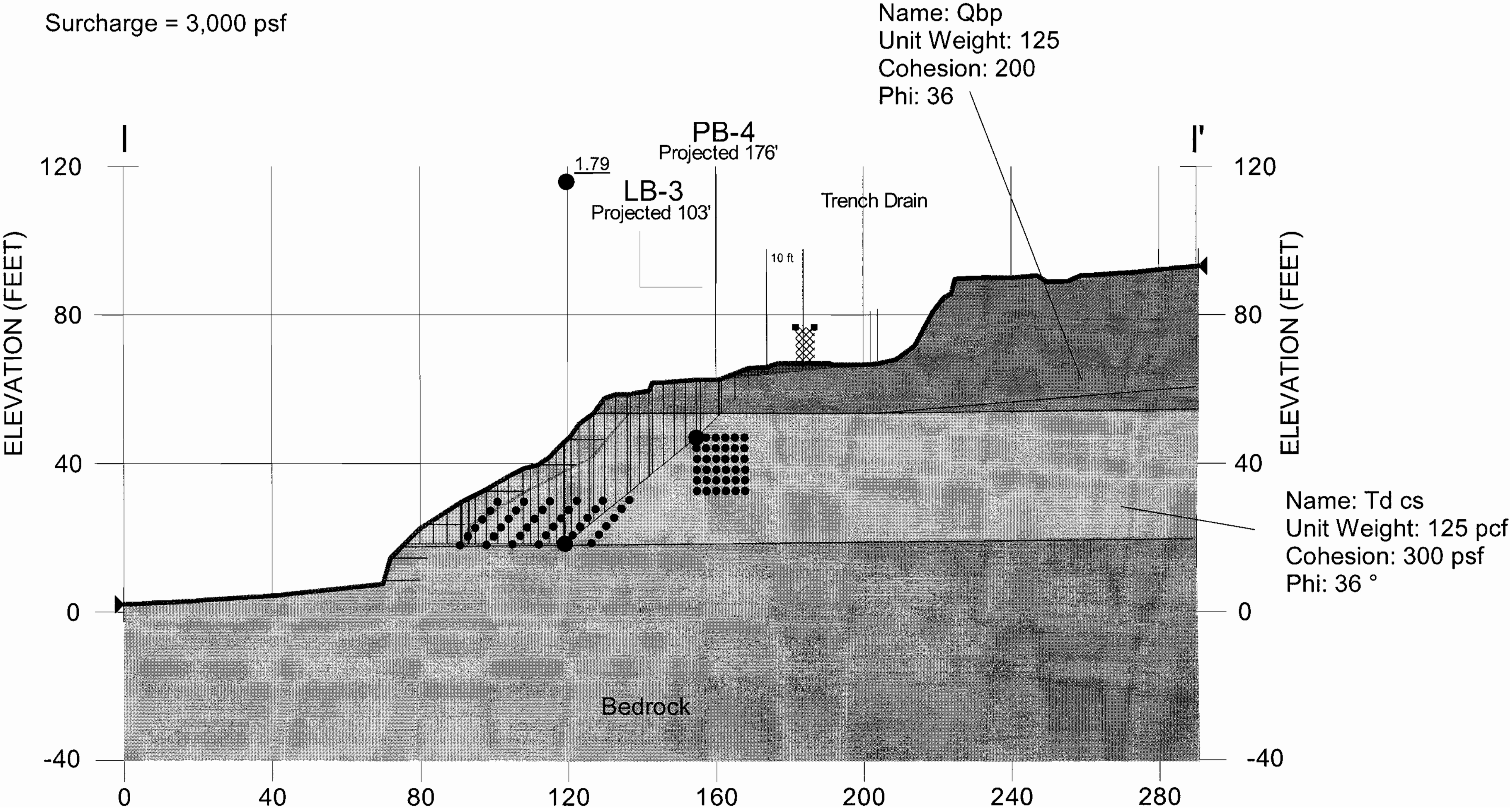
Factor of Safety: 1.79



Del Mar Bluffs Cross Section I-I'
Slope Stability Analysis
File Name: Section I-I' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.79

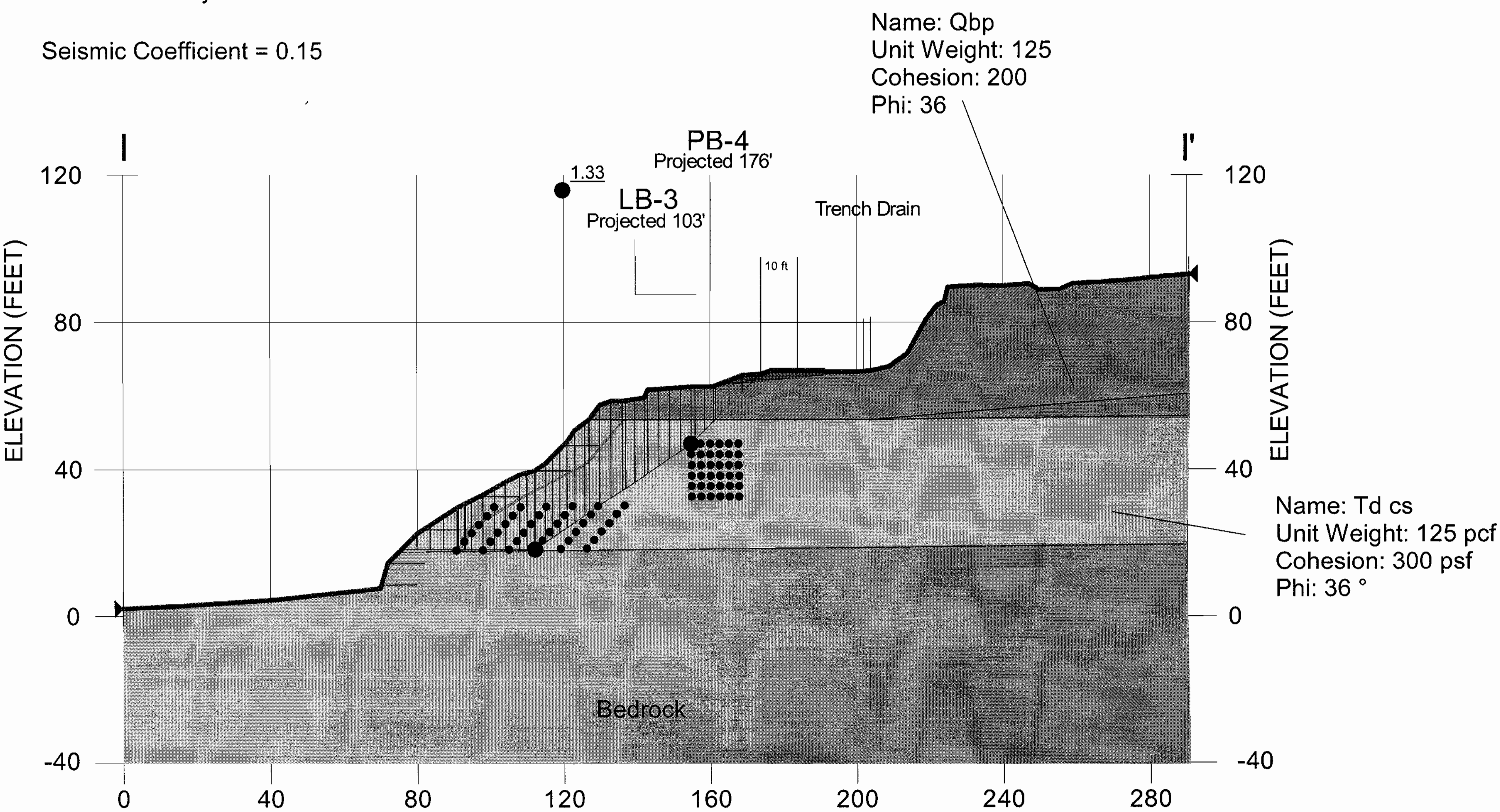
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section I-I'
Slope Stability Analysis
File Name: Section I-I' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.33

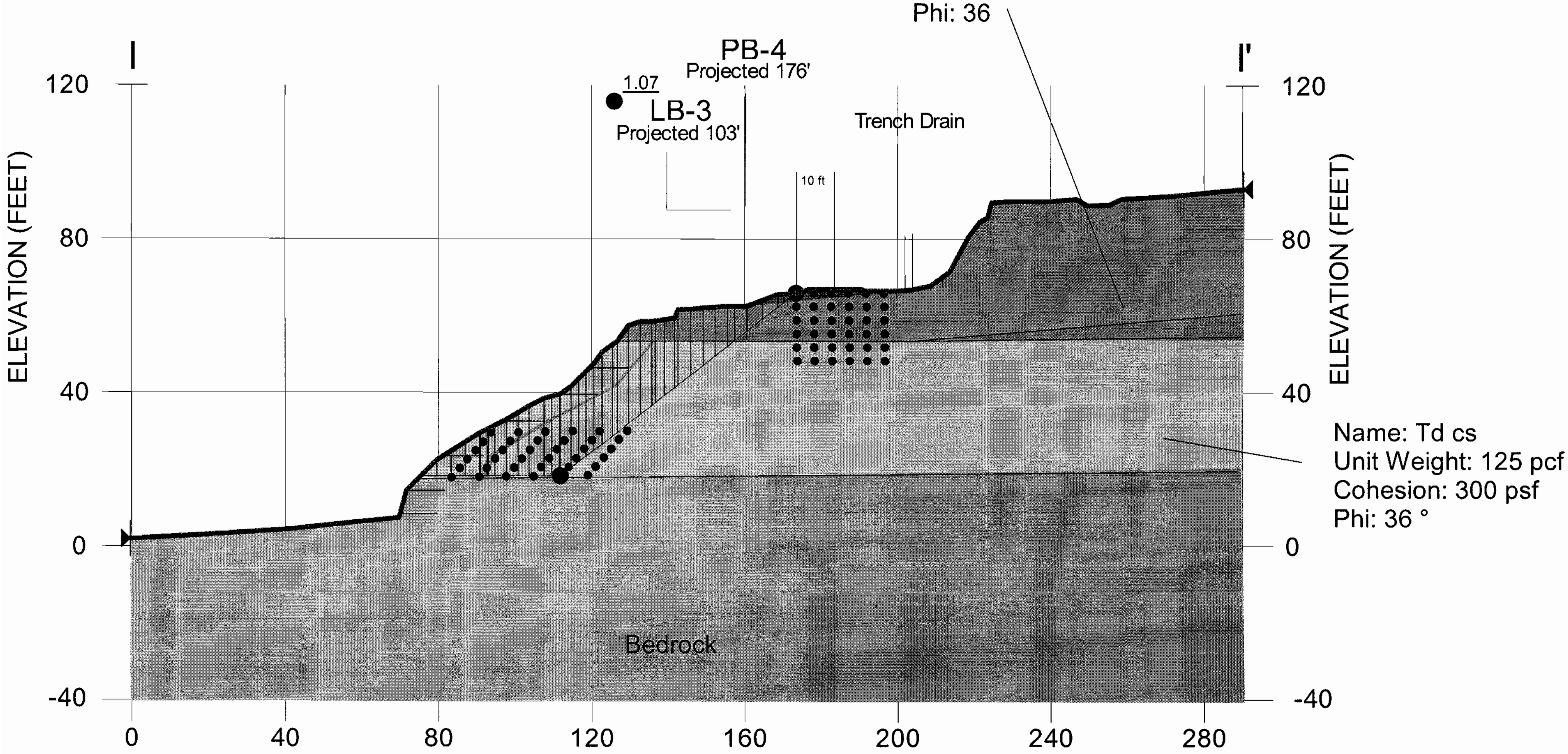
Seismic Coefficient = 0.15



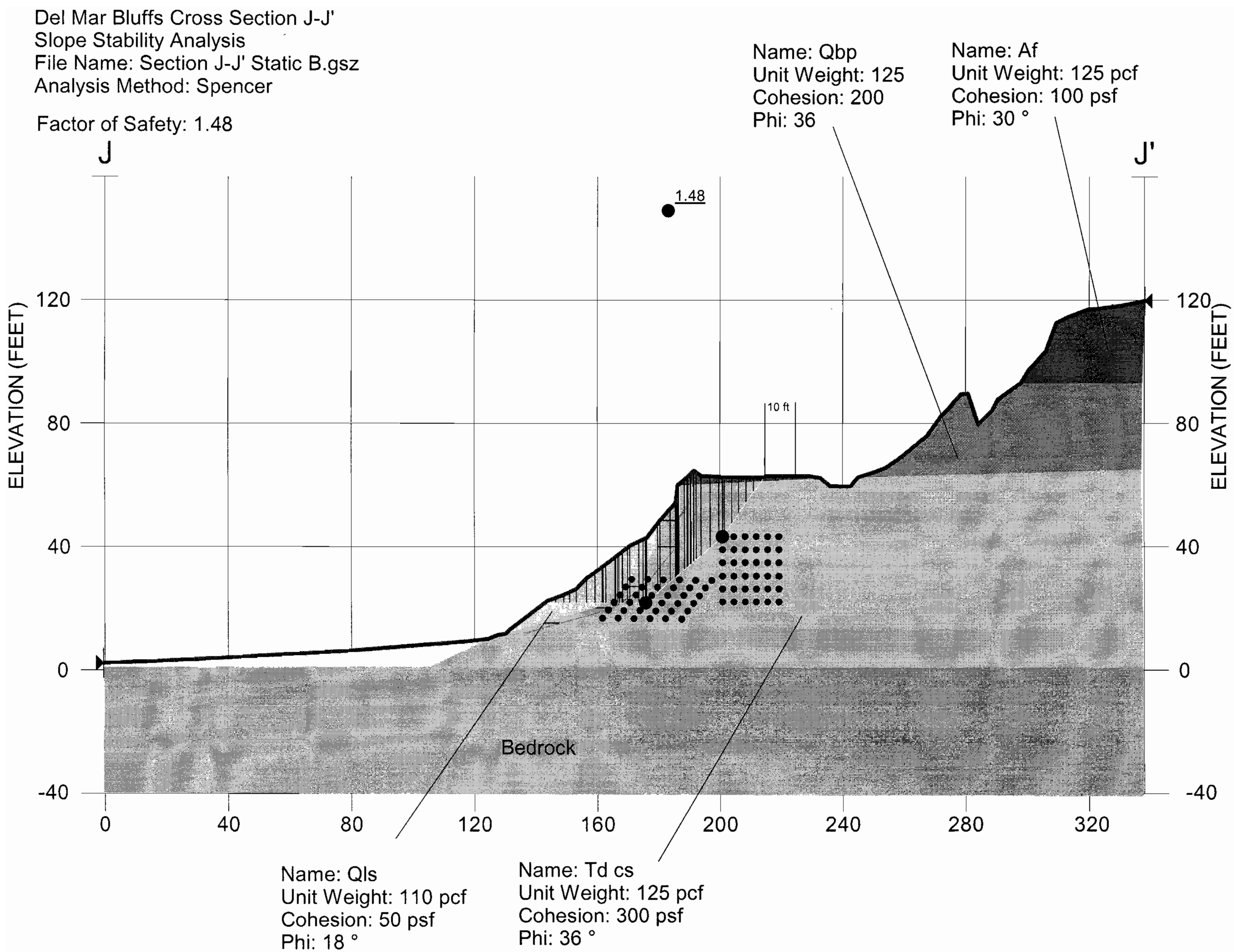
Del Mar Bluffs Cross Section I-I'
Slope Stability Analysis
File Name: Section I-I' Pseudostatic Kh = 0.28.gsz
Analysis Method: Spencer

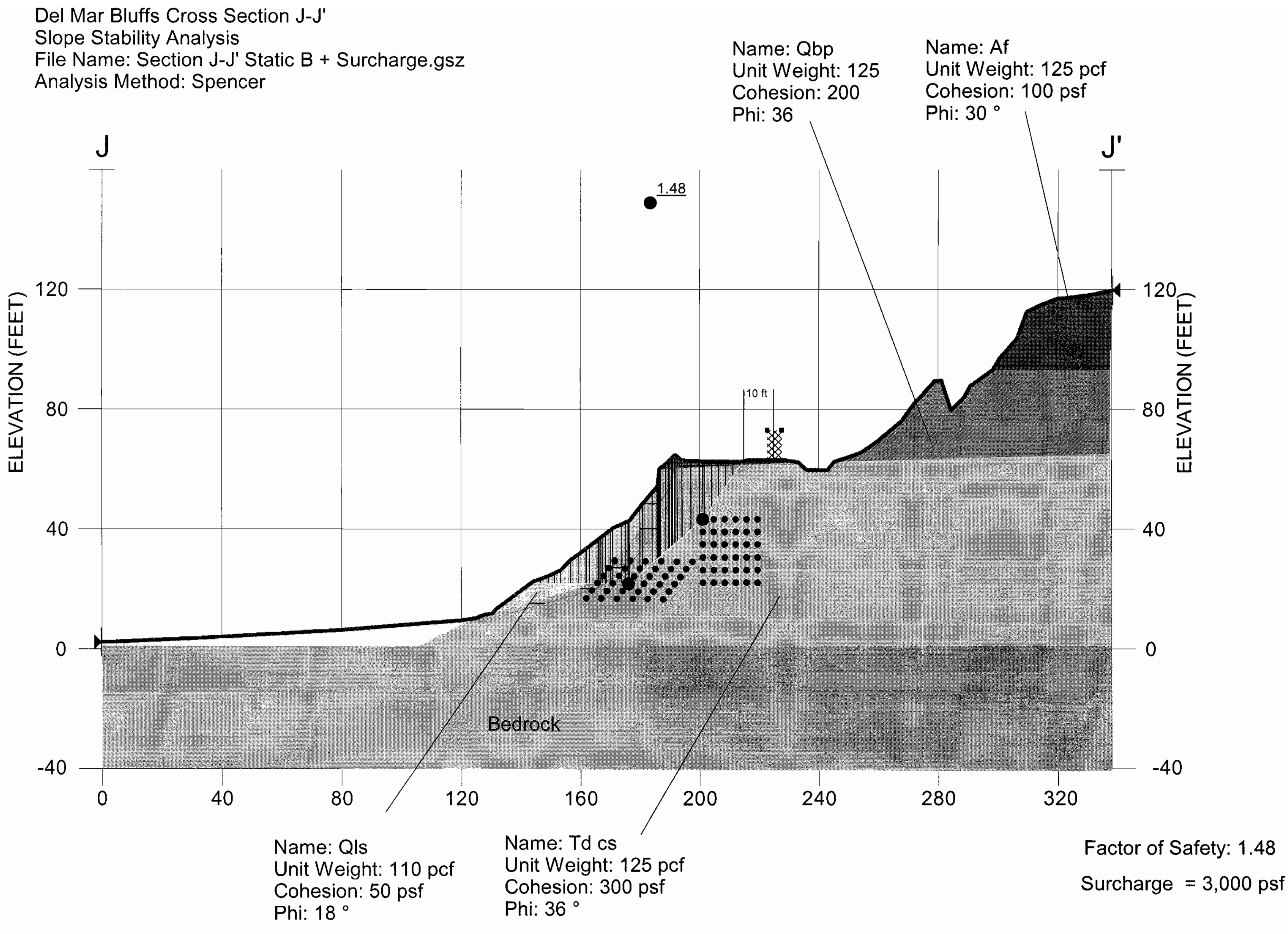
Factor of Safety: 1.07

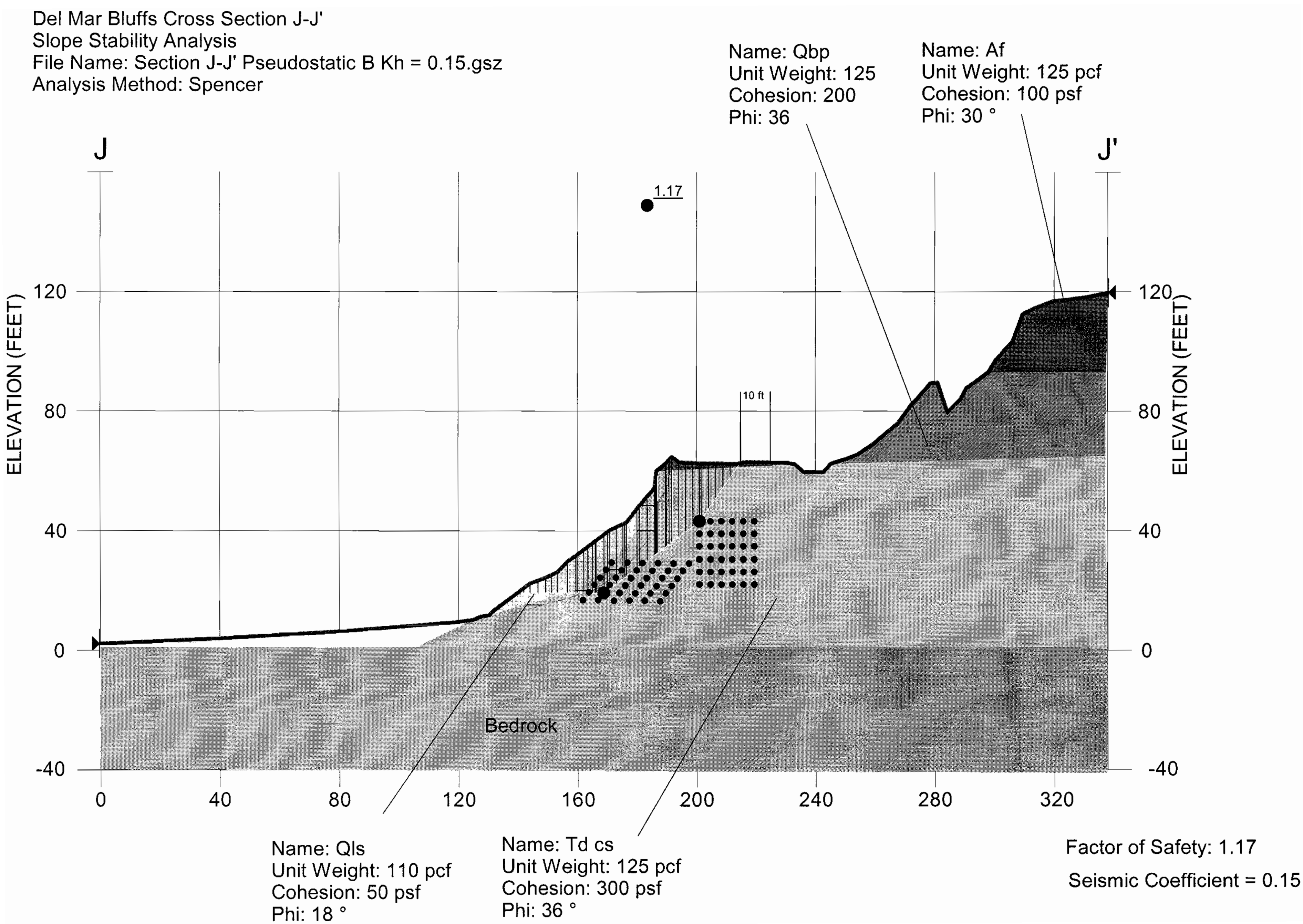
Seismic Coefficient = 0.28

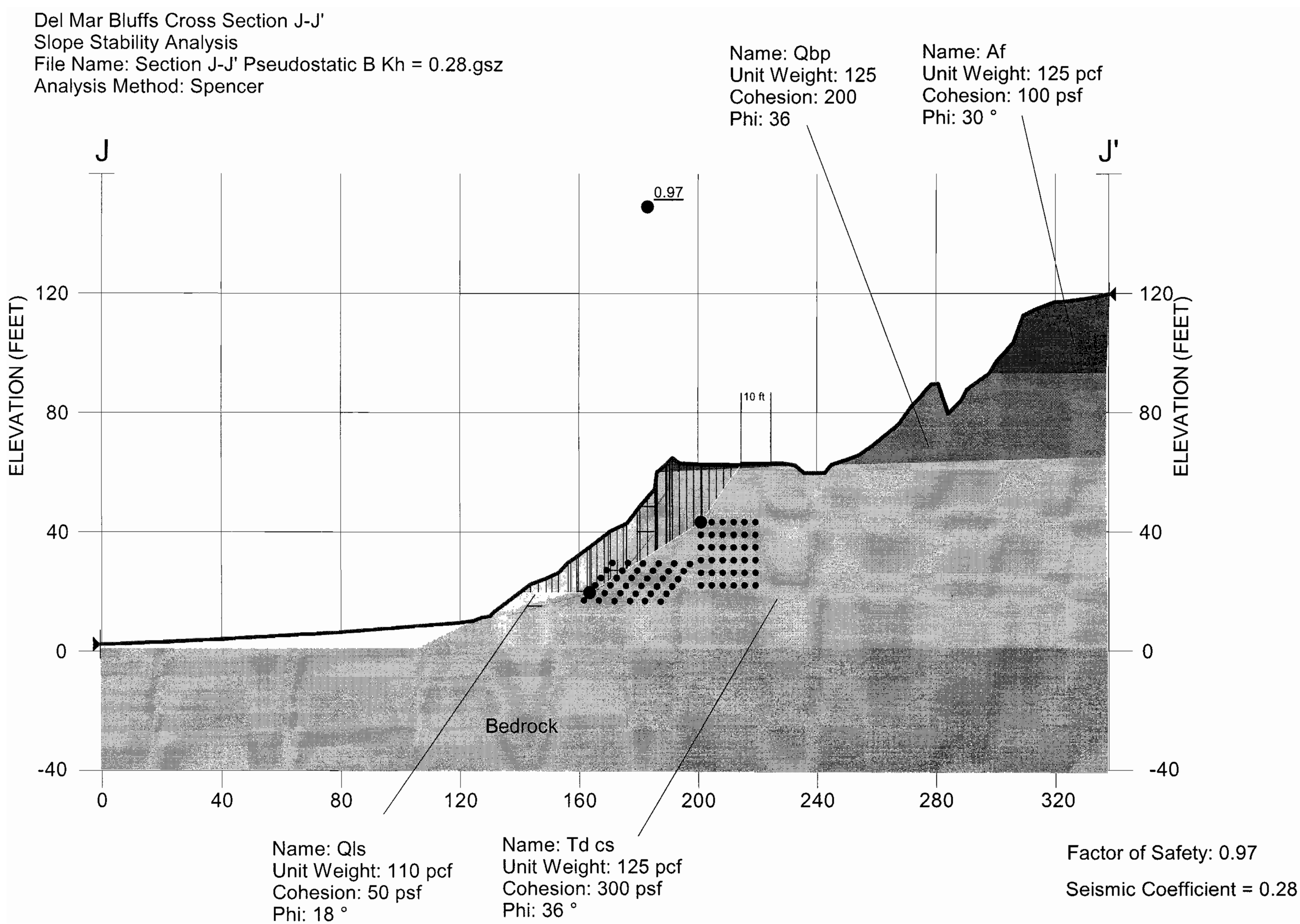


CROSS SECTION J-J'

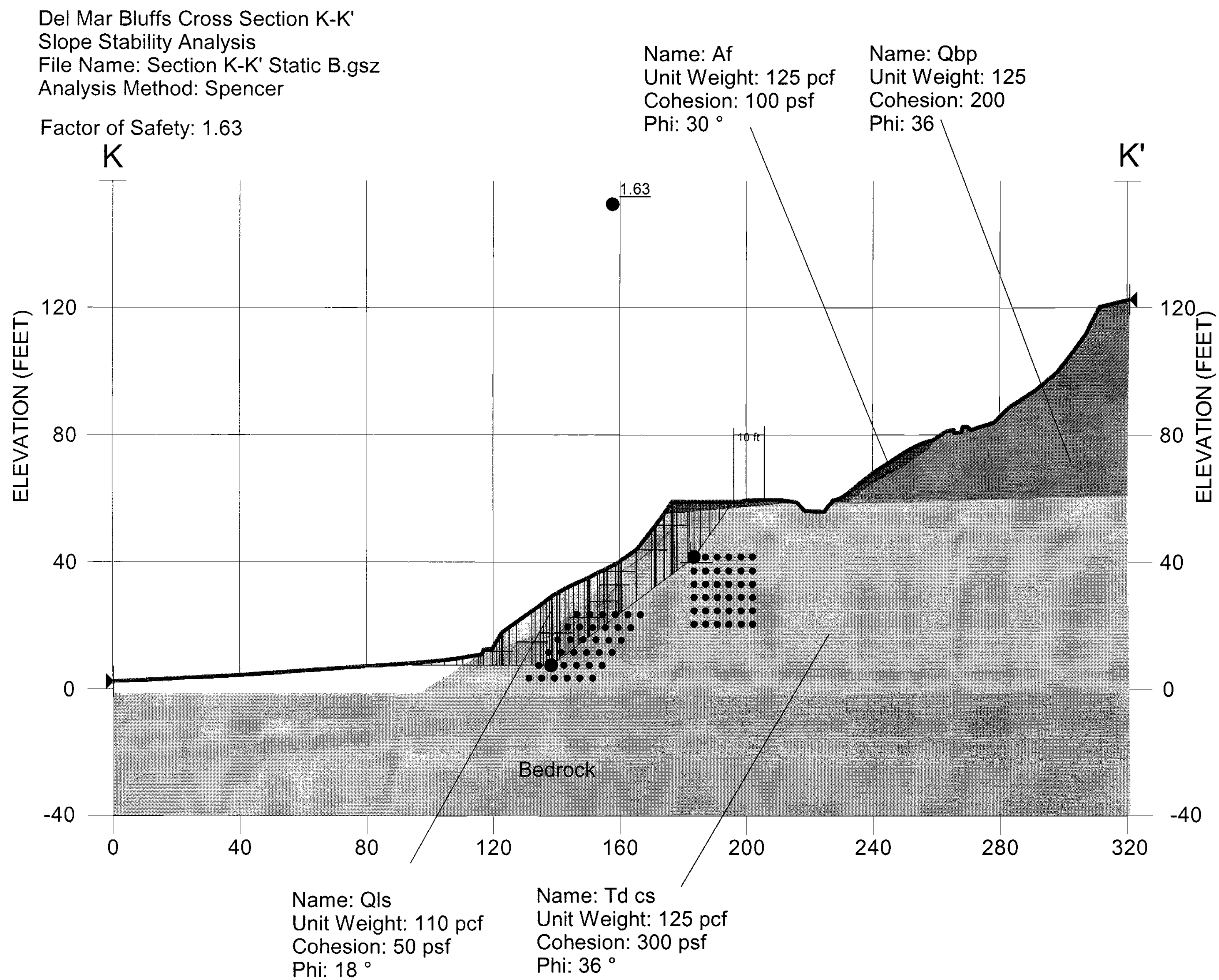

Del Mar Bluffs Cross Section J-J'
Slope Stability Analysis
File Name: Section J-J' Static B.gsz
Analysis Method: Spencer
Factor of Safety: 1.48
J
J'
1.48
10 ft
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
Bedrock
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
ELEVATION (FEET)
120
80
40
0
-40
0
40
80
120
160
200
240
280
320


Del Mar Bluffs Cross Section J-J'
Slope Stability Analysis
File Name: Section J-J' Static B + Surcharge.gsz
Analysis Method: Spencer
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
J
J'
1.48
10 ft
ELEVATION (FEET)
120
80
40
0
-40
Bedrock
0
40
80
120
160
200
240
280
320
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 1.48
Surcharge = 3,000 psf

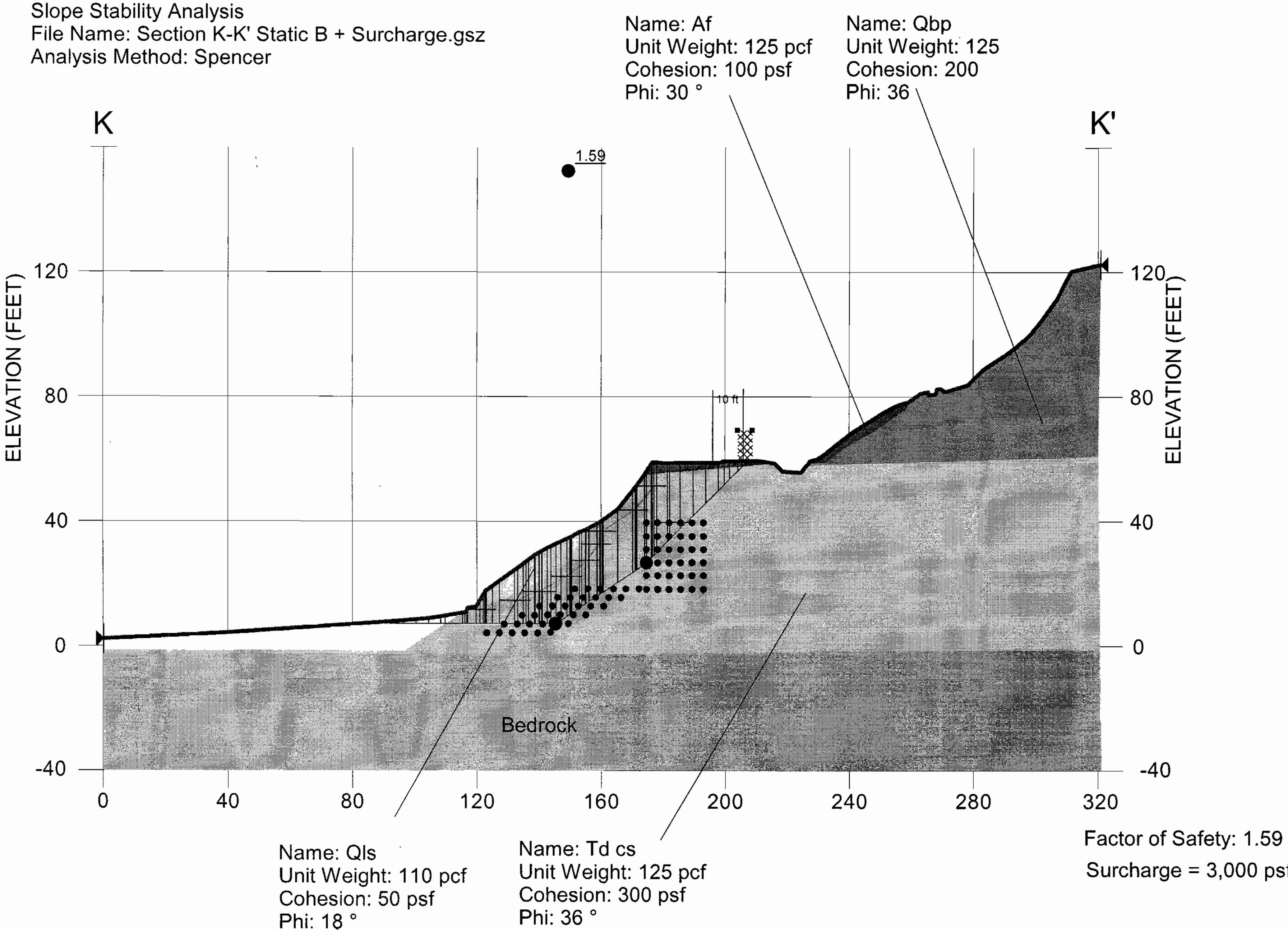

Del Mar Bluffs Cross Section J-J'
Slope Stability Analysis
File Name: Section J-J' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
J
J'
1.17
10 ft
ELEVATION (FEET)
120
80
40
0
-40
Bedrock
0
40
80
120
160
200
240
280
320
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 1.17
Seismic Coefficient = 0.15


Del Mar Bluffs Cross Section J-J'
Slope Stability Analysis
File Name: Section J-J' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
J
J'
0.97
10 ft
ELEVATION (FEET)
120
80
40
0
-40
Bedrock
0
40
80
120
160
200
240
280
320
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 0.97
Seismic Coefficient = 0.28

CROSS SECTION K-K'

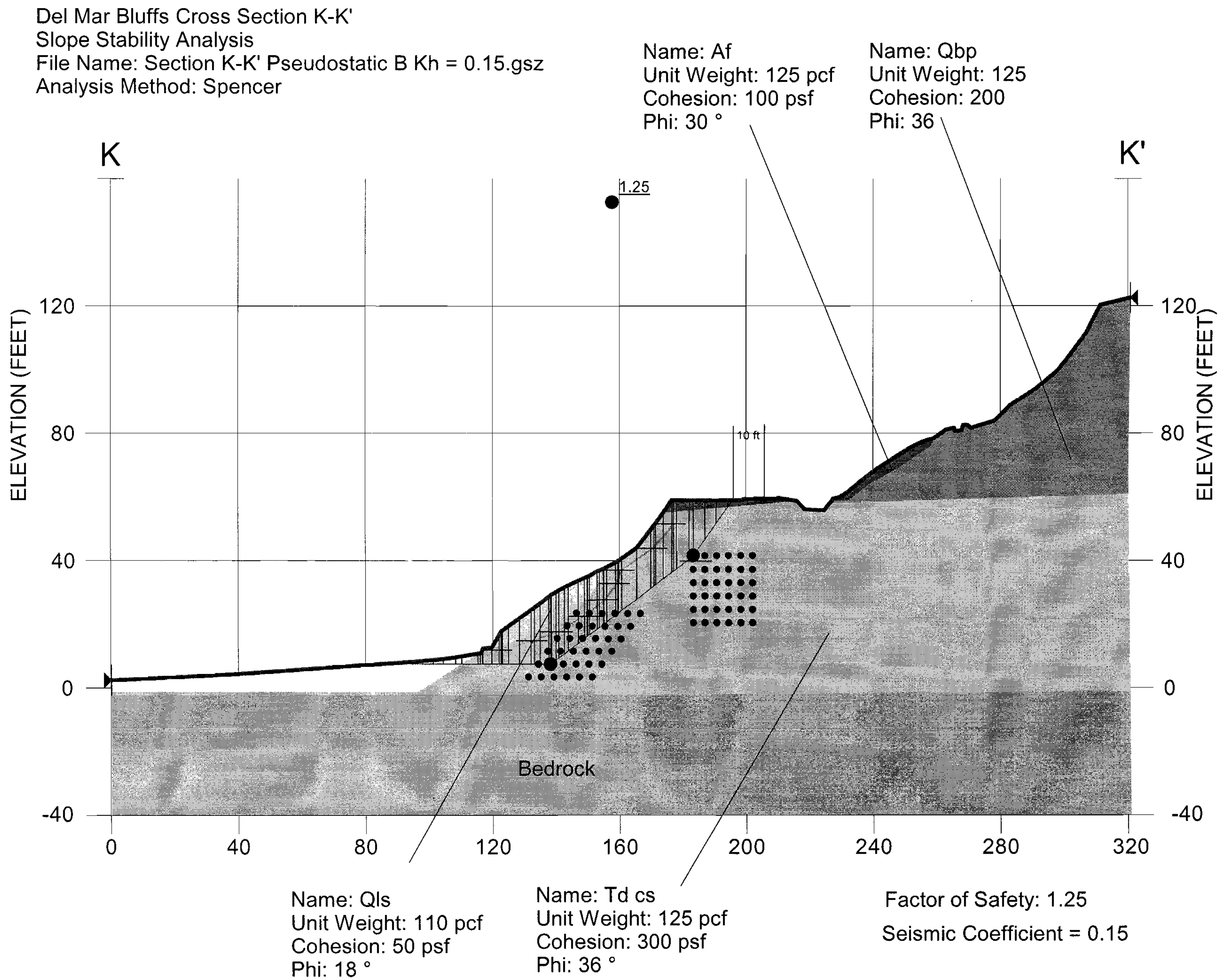

Del Mar Bluffs Cross Section K-K'
Slope Stability Analysis
File Name: Section K-K' Static B.gsz
Analysis Method: Spencer
Factor of Safety: 1.63
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
K
K'
1.63
10 ft
Bedrock
ELEVATION (FEET)
120
80
40
0
-40
0
40
80
120
160
200
240
280
320
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °

Del Mar Bluffs Cross Section K-K'
Slope Stability Analysis
File Name: Section K-K' Static B + Surcharge.gsz
Analysis Method: Spencer

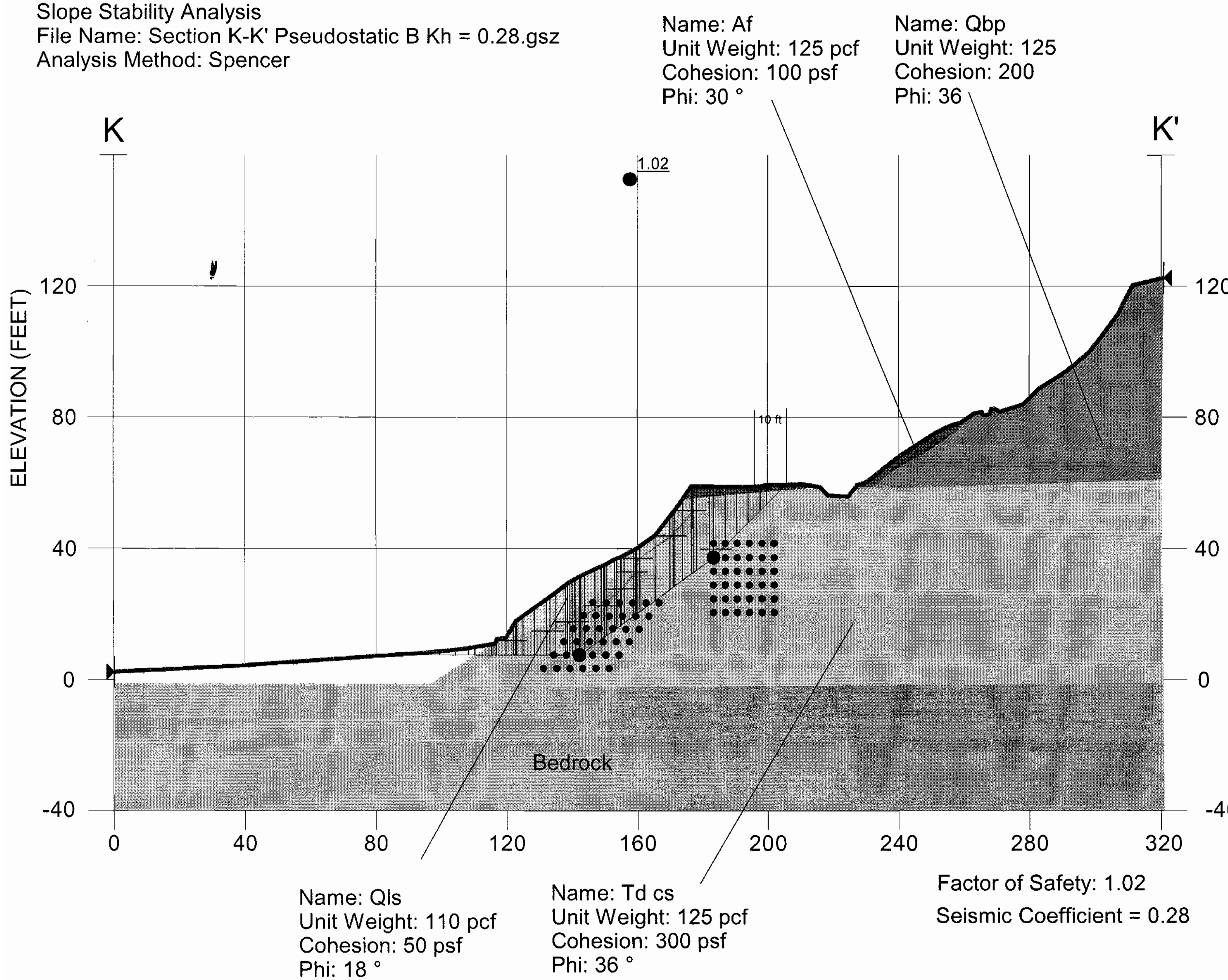


Factor of Safety: 1.59

Surcharge = 3,000 psf


Del Mar Bluffs Cross Section K-K'
Slope Stability Analysis
File Name: Section K-K' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer
Name: Af
Unit Weight: 125 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
K
K'
1.25
10 ft
ELEVATION (FEET)
120
80
40
0
-40
Bedrock
0
40
80
120
160
200
240
280
320
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 1.25
Seismic Coefficient = 0.15

Del Mar Bluffs Cross Section K-K'
Slope Stability Analysis
File Name: Section K-K' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

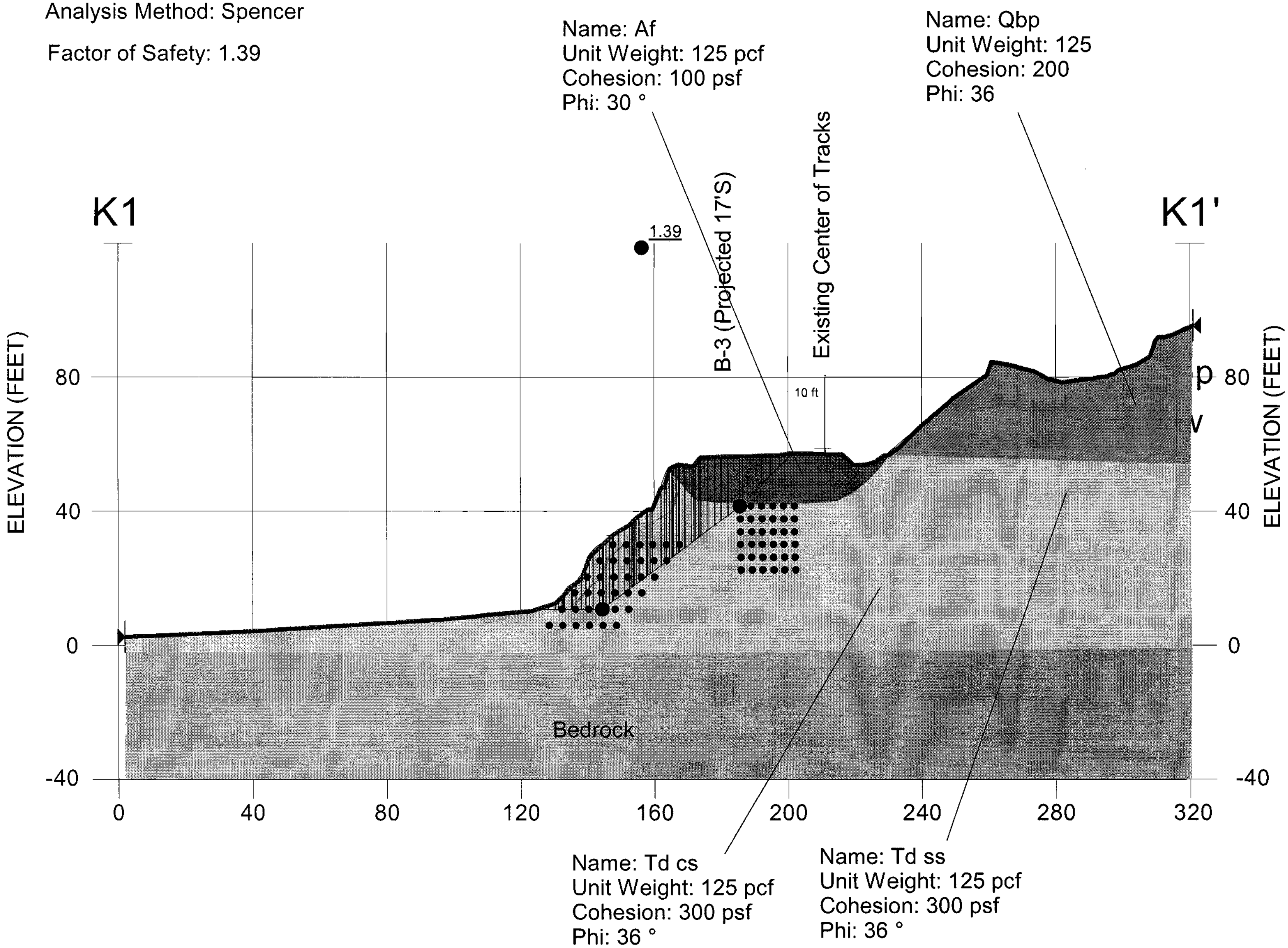


Factor of Safety: 1.02
Seismic Coefficient = 0.28

CROSS SECTION K1-K1′

Del Mar Bluffs Cross Section K1-K1'
Slope Stability Analysis
File Name: Section K1-K1' Static B.gsz
Analysis Method: Spencer

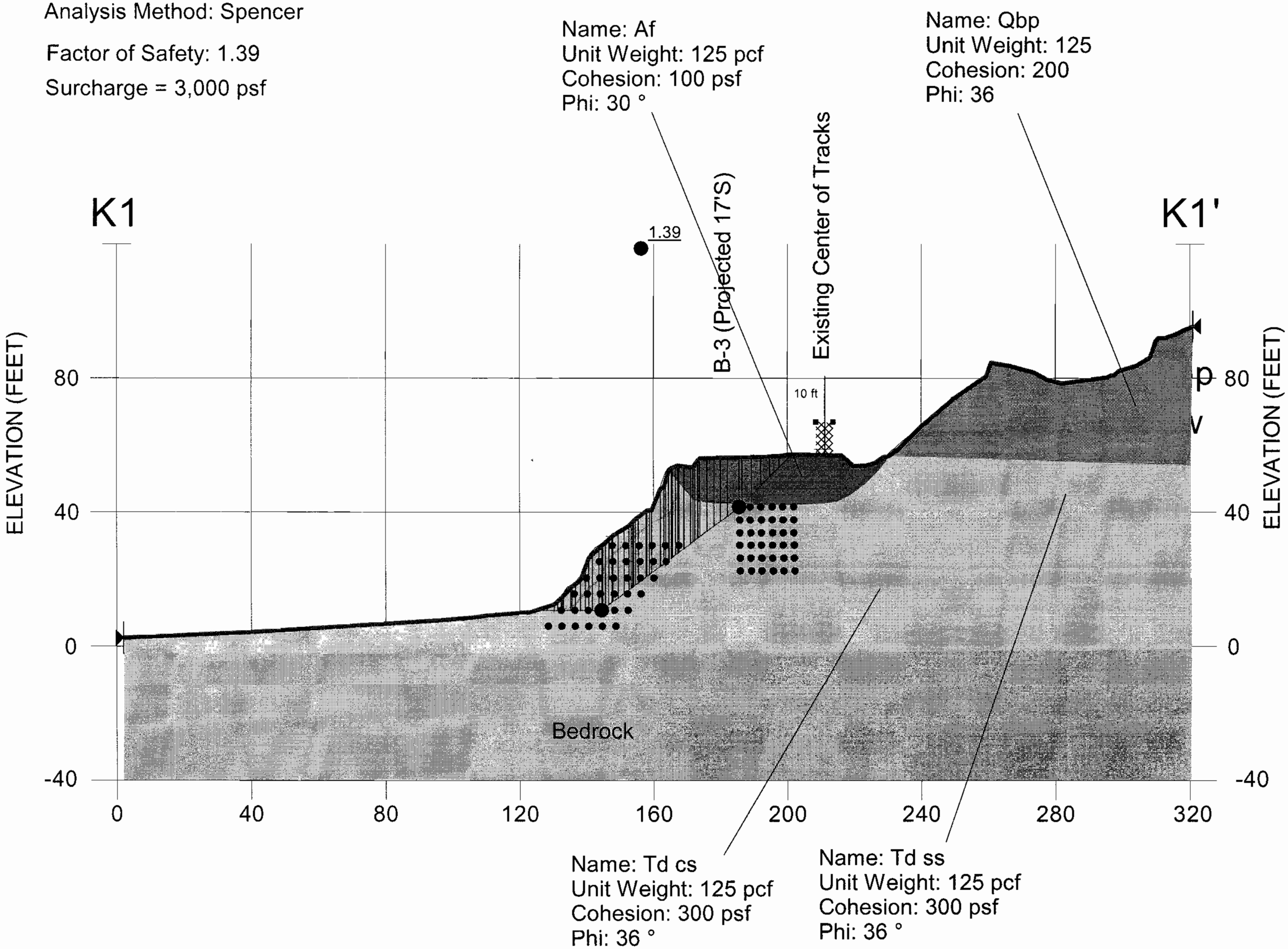
Factor of Safety: 1.39



Del Mar Bluffs Cross Section K1-K1'
Slope Stability Analysis
File Name: Section K1-K1' Static B + Surcharge.gsz
Analysis Method: Spencer

Factor of Safety: 1.39

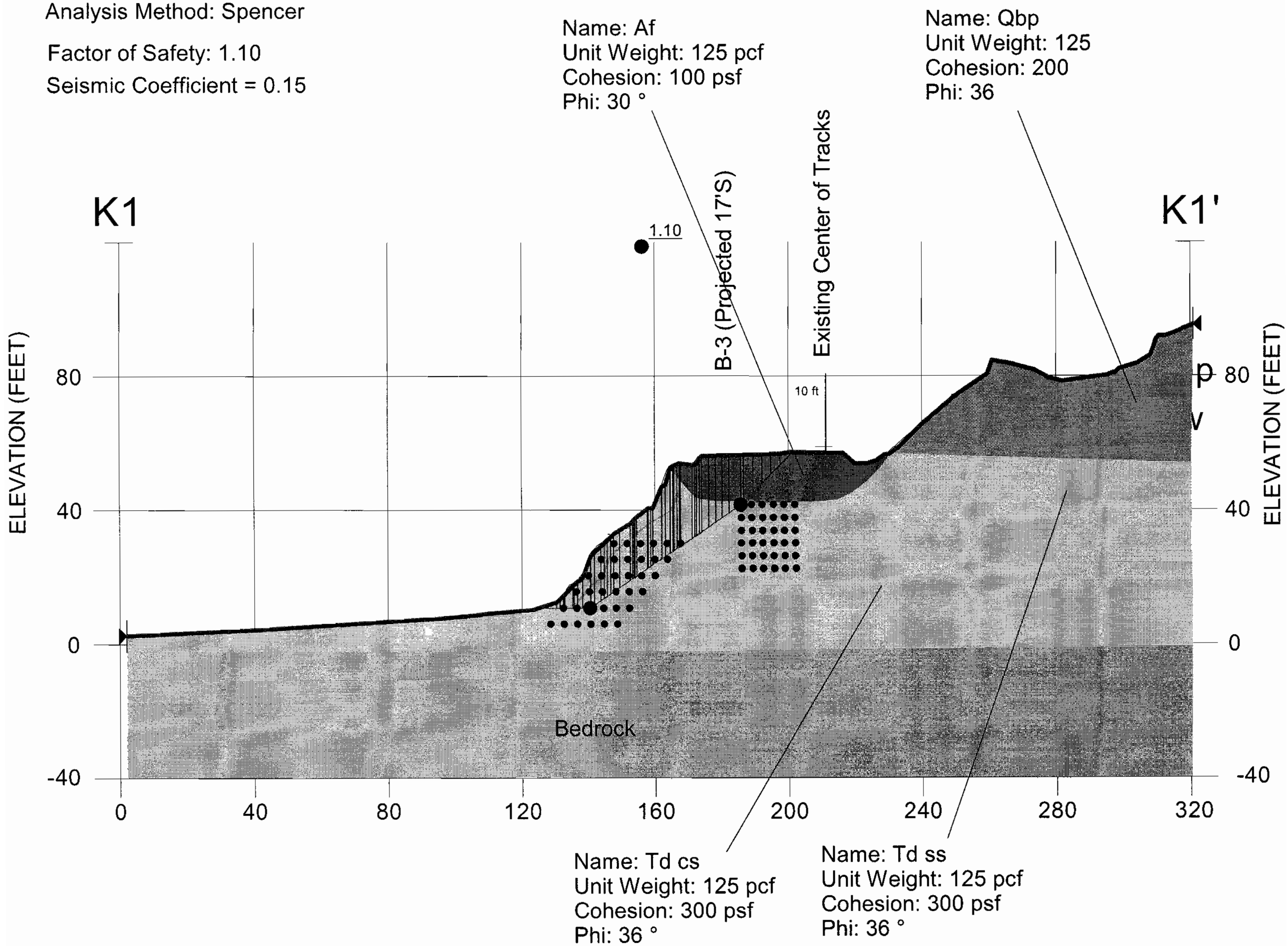
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section K1-K1'
Slope Stability Analysis
File Name: Section K1-K1' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer

Factor of Safety: 1.10

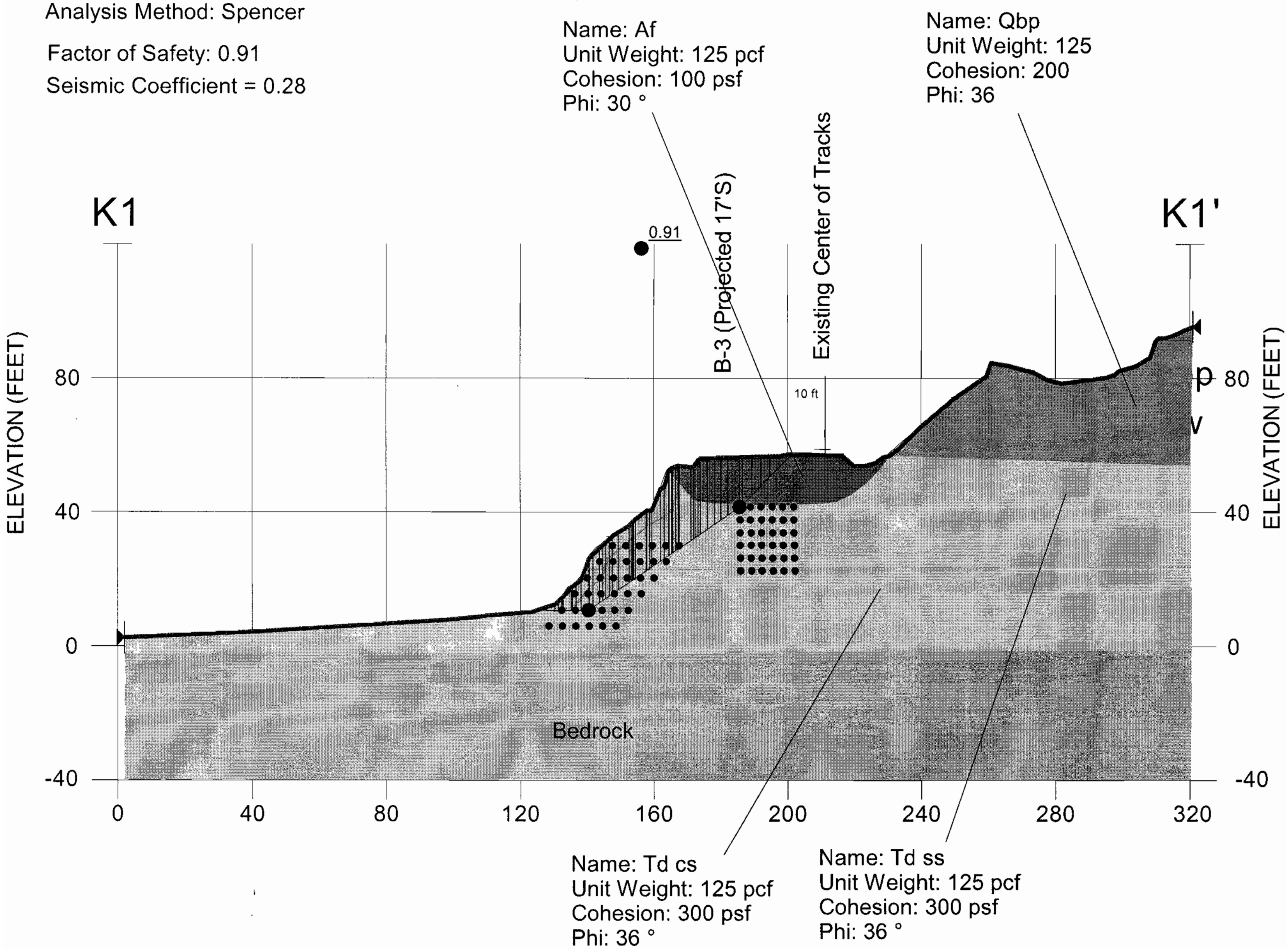
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section K1-K1'
Slope Stability Analysis
File Name: Section K1-K1' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.91

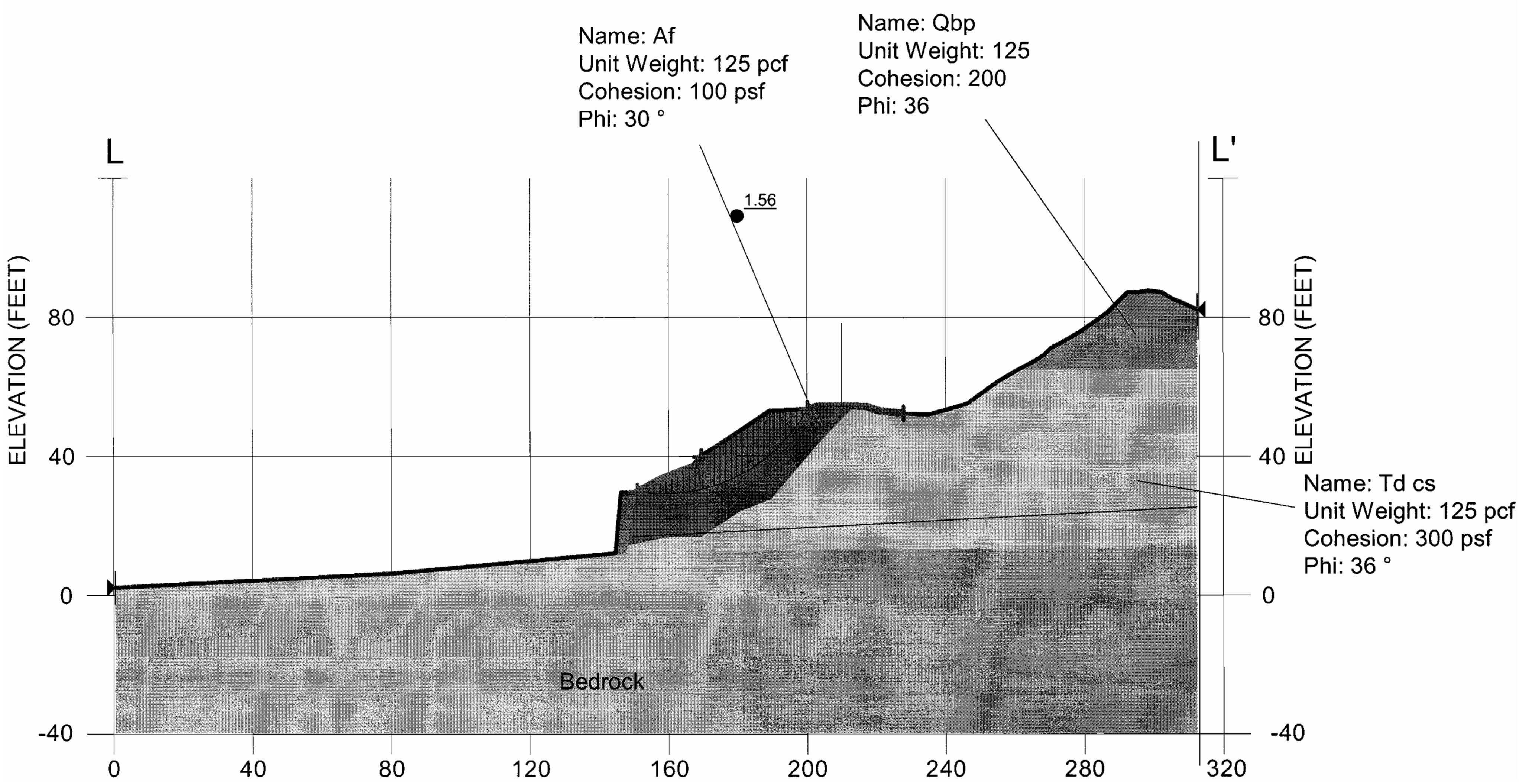
Seismic Coefficient = 0.28



CROSS SECTION L-L’

Del Mar Bluffs Cross Section L-L'
Slope Stability Analysis
File Name: Section L-L' Static 1.gsz
Analysis Method: Spencer

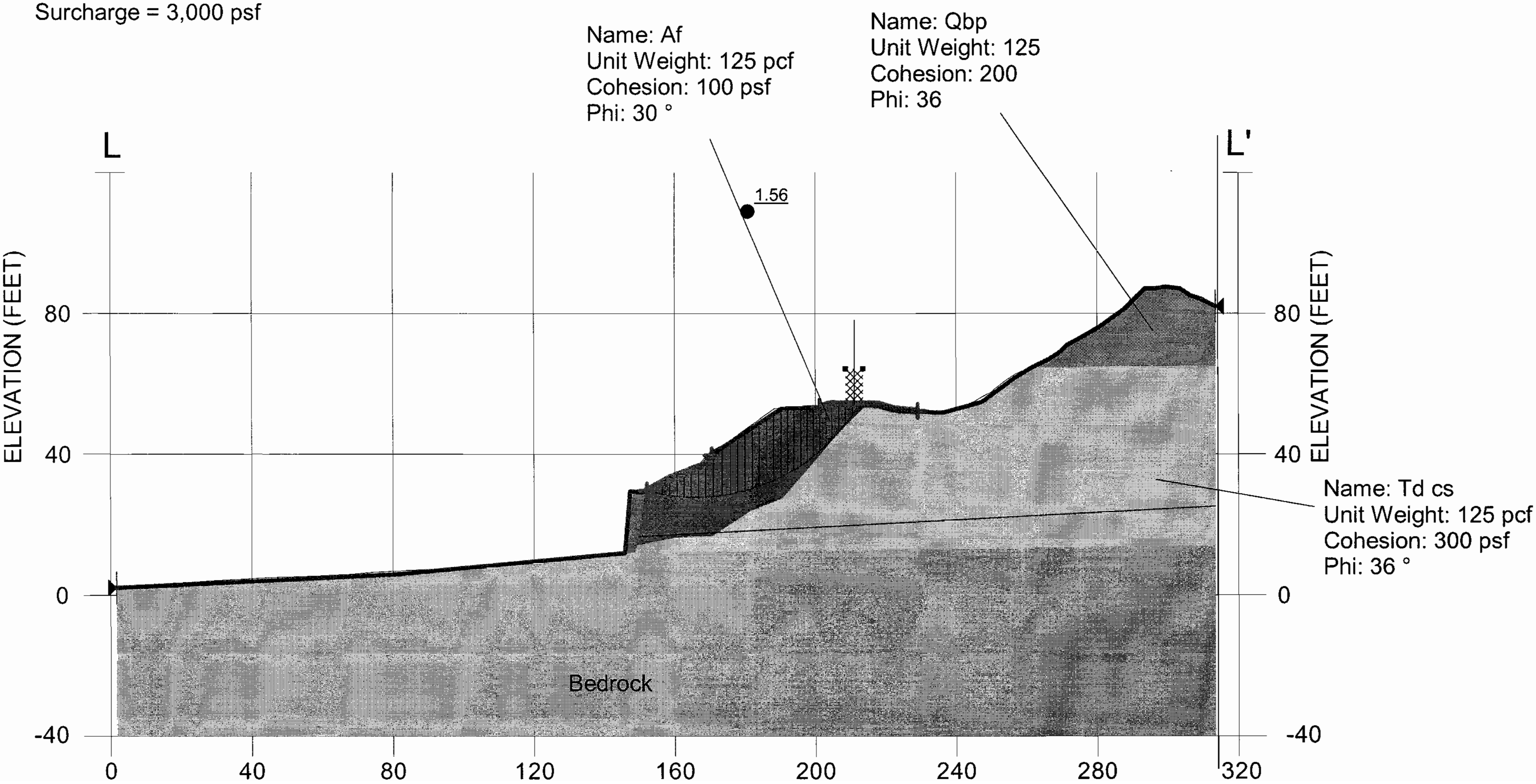
Factor of Safety: 1.56



Del Mar Bluffs Cross Section L-L'
Slope Stability Analysis
File Name: Section L-L' Static 1 + Surcharge.gsz
Analysis Method: Bishop

Factor of Safety: 1.56

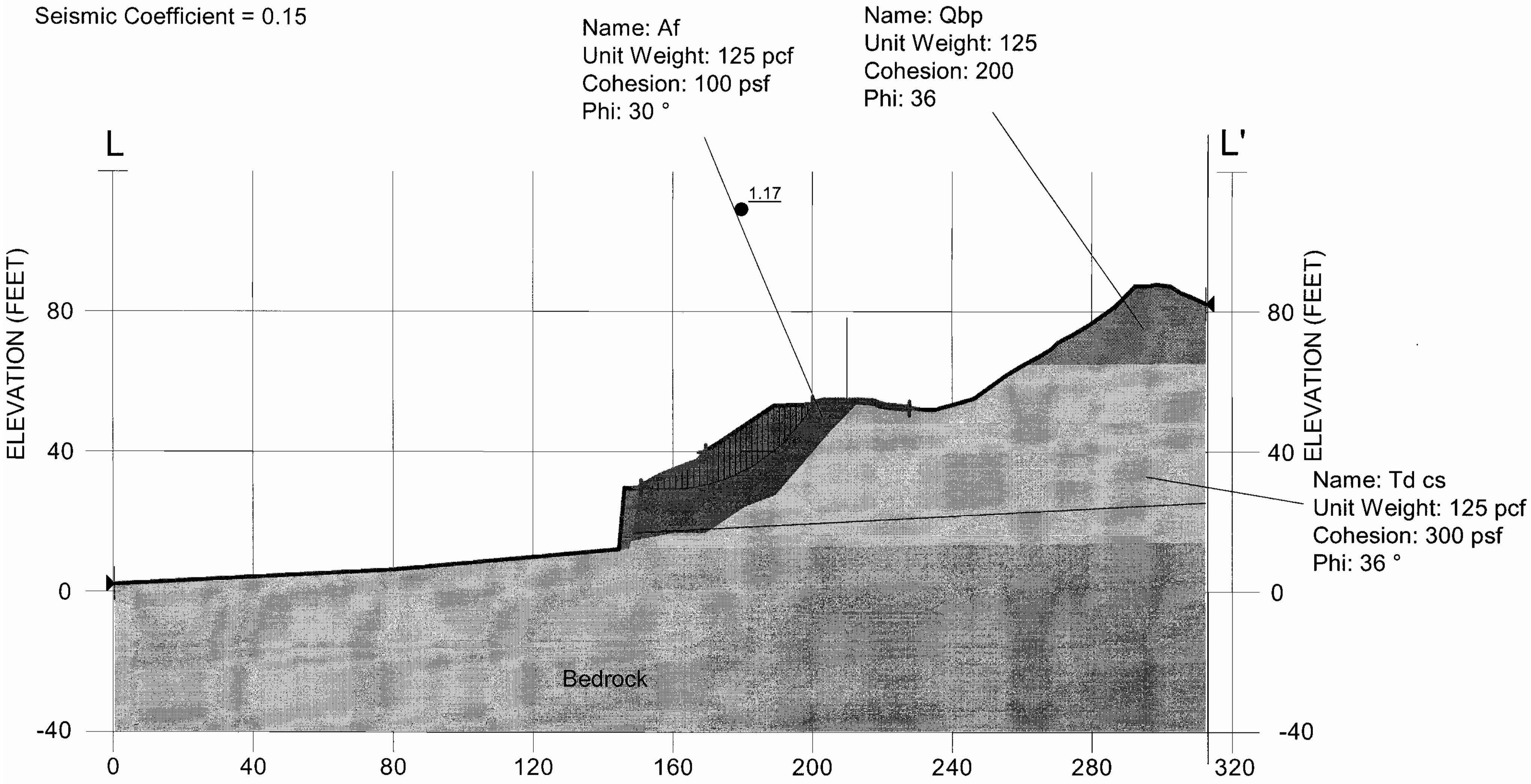
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section L-L'
Slope Stability Analysis
File Name: Section L-L' Pseudostatic Kh = 0.15.gsz
Analysis Method: Bishop

Factor of Safety: 1.17

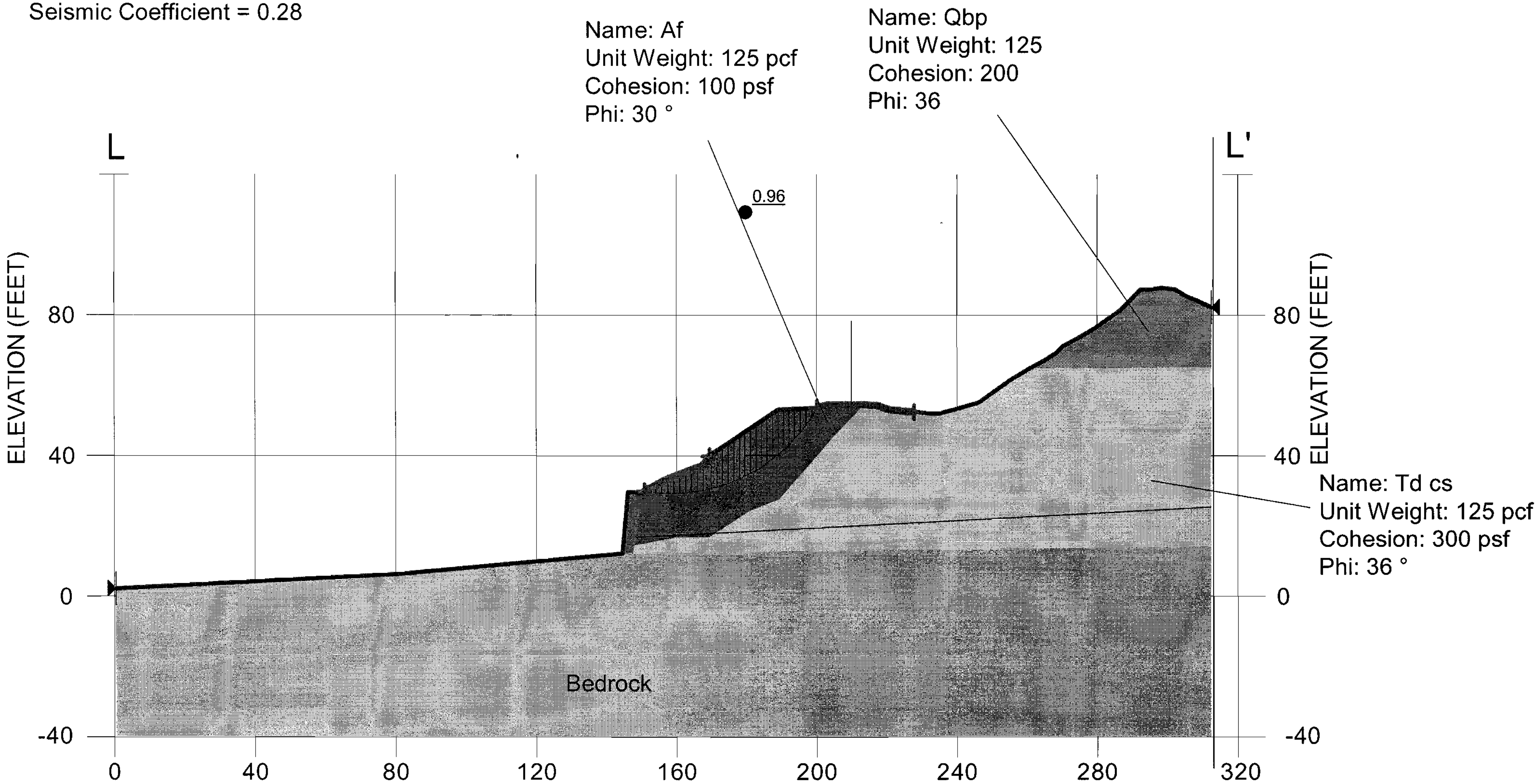
Seismic Coefficient = 0.15



Del Mar Bluffs Cross Section L-L'
Slope Stability Analysis
File Name: Section L-L' Pseudostatic Kh = 0.28.gsz
Analysis Method: Spencer

Factor of Safety: 0.96

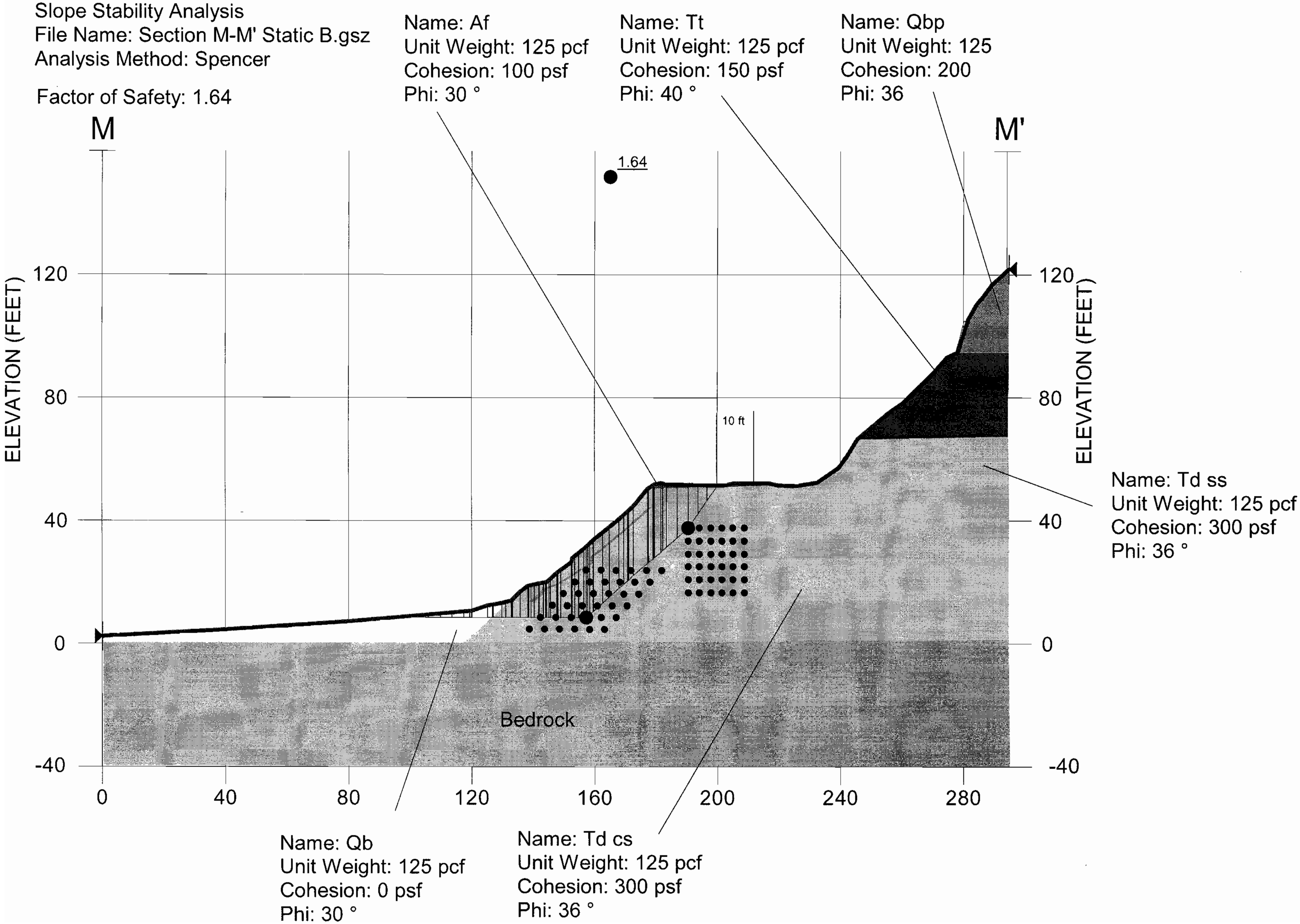
Seismic Coefficient = 0.28



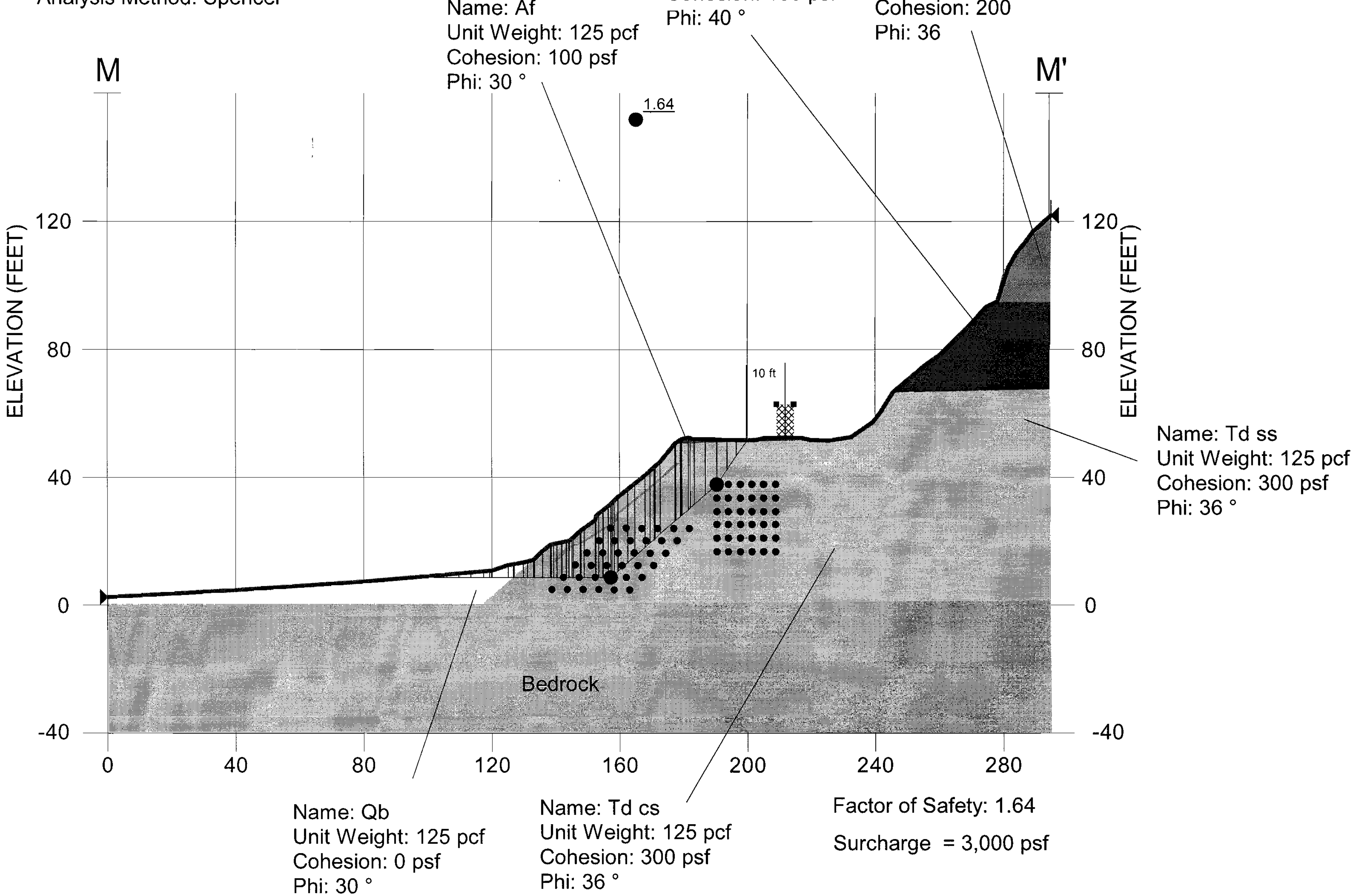
CROSS SECTION M-M’

Del Mar Bluffs Cross Section M-M'
Slope Stability Analysis
File Name: Section M-M' Static B.gsz
Analysis Method: Spencer

Factor of Safety: 1.64



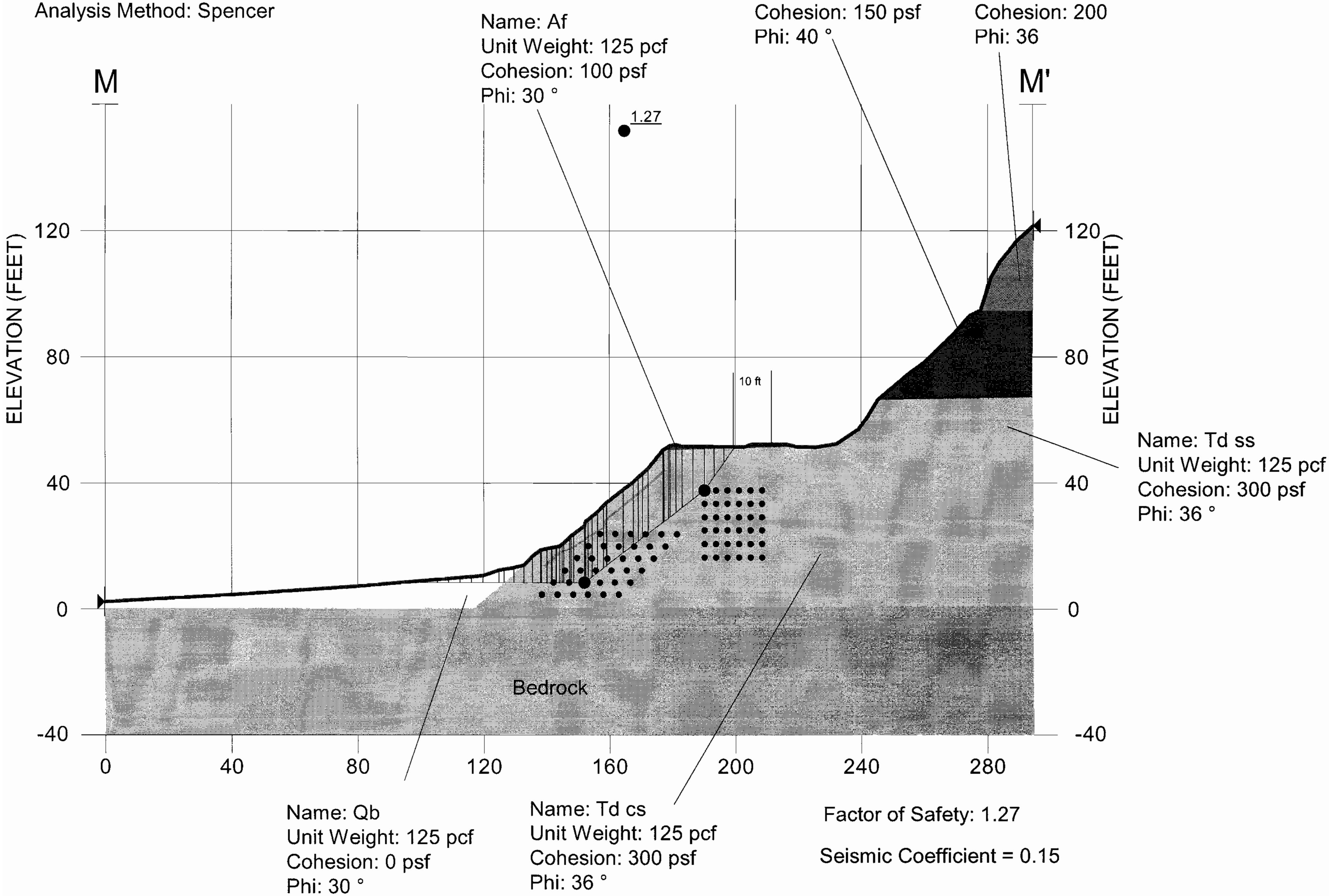
Del Mar Bluffs Cross Section M-M'
Slope Stability Analysis
File Name: Section M-M' Static B + Surcharge.gsz
Analysis Method: Spencer



Factor of Safety: 1.64

Surcharge = 3,000 psf

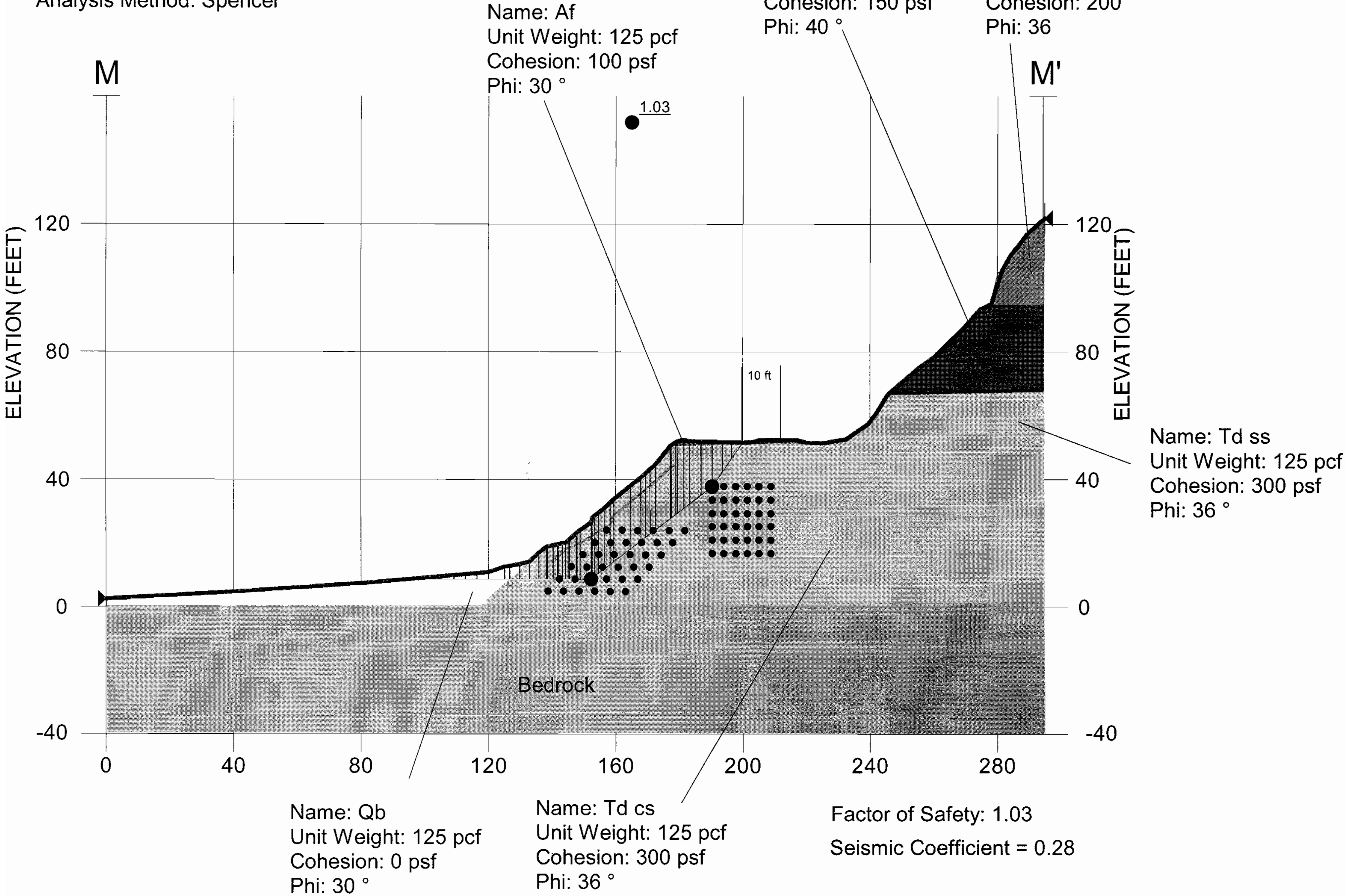
Del Mar Bluffs Cross Section M-M'
Slope Stability Analysis
File Name: Section M-M' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer



Factor of Safety: 1.27

Seismic Coefficient = 0.15

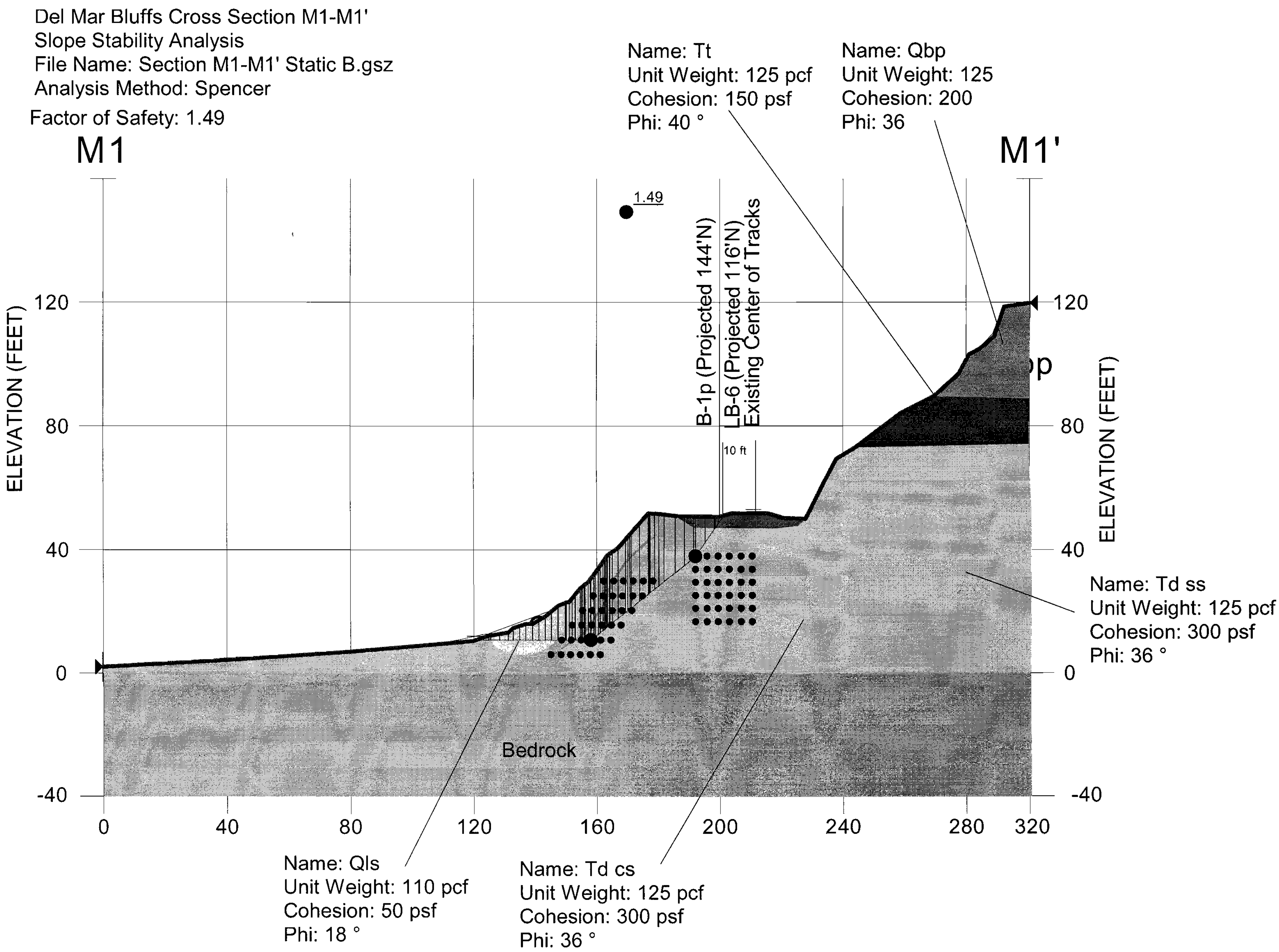
Del Mar Bluffs Cross Section M-M'
Slope Stability Analysis
File Name: Section M-M' Pseudostatic B Kh = 0.28.gsz
Analysis Method: Spencer



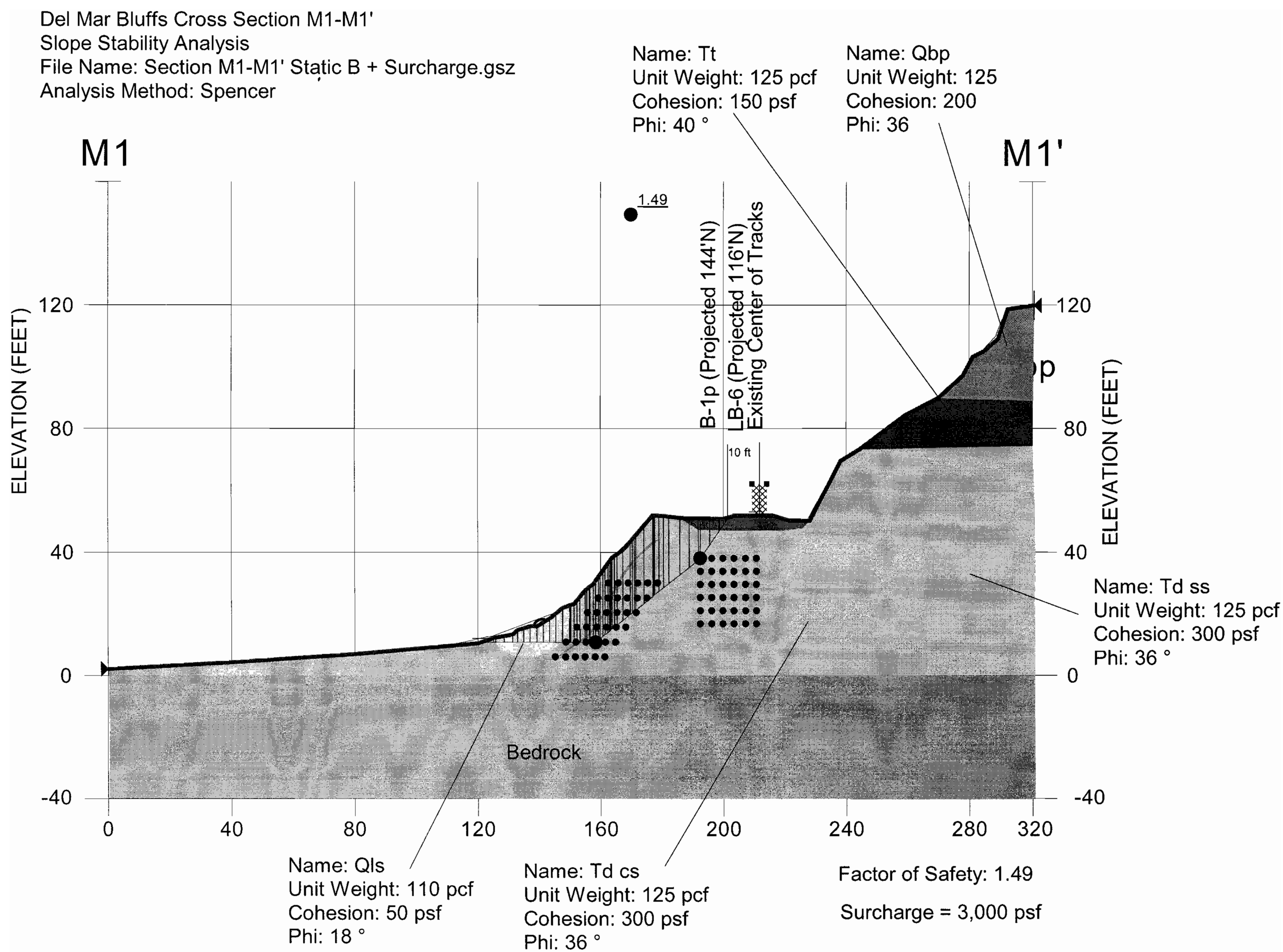
Factor of Safety: 1.03

Seismic Coefficient = 0.28

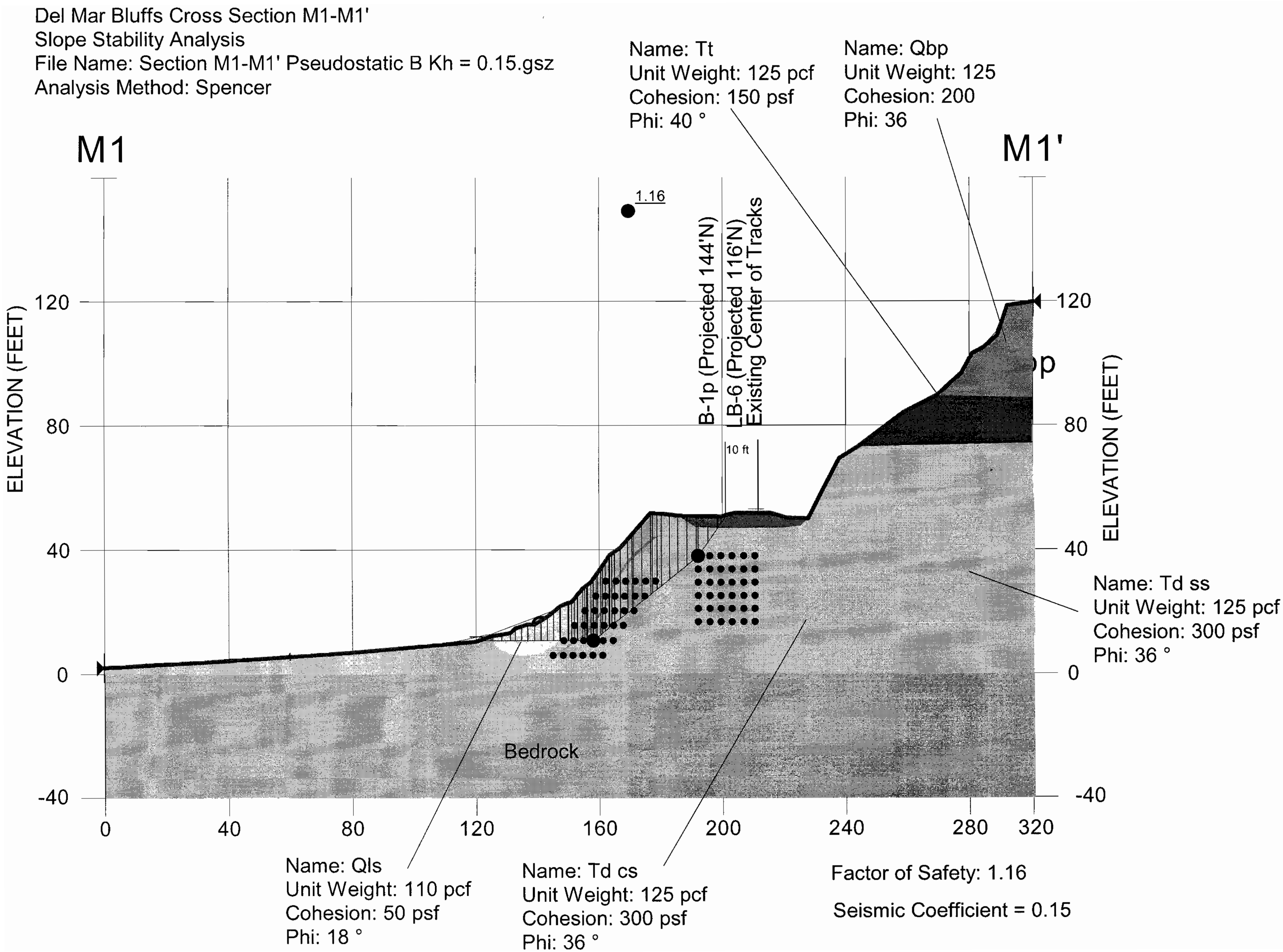
CROSS SECTION M1-M1′



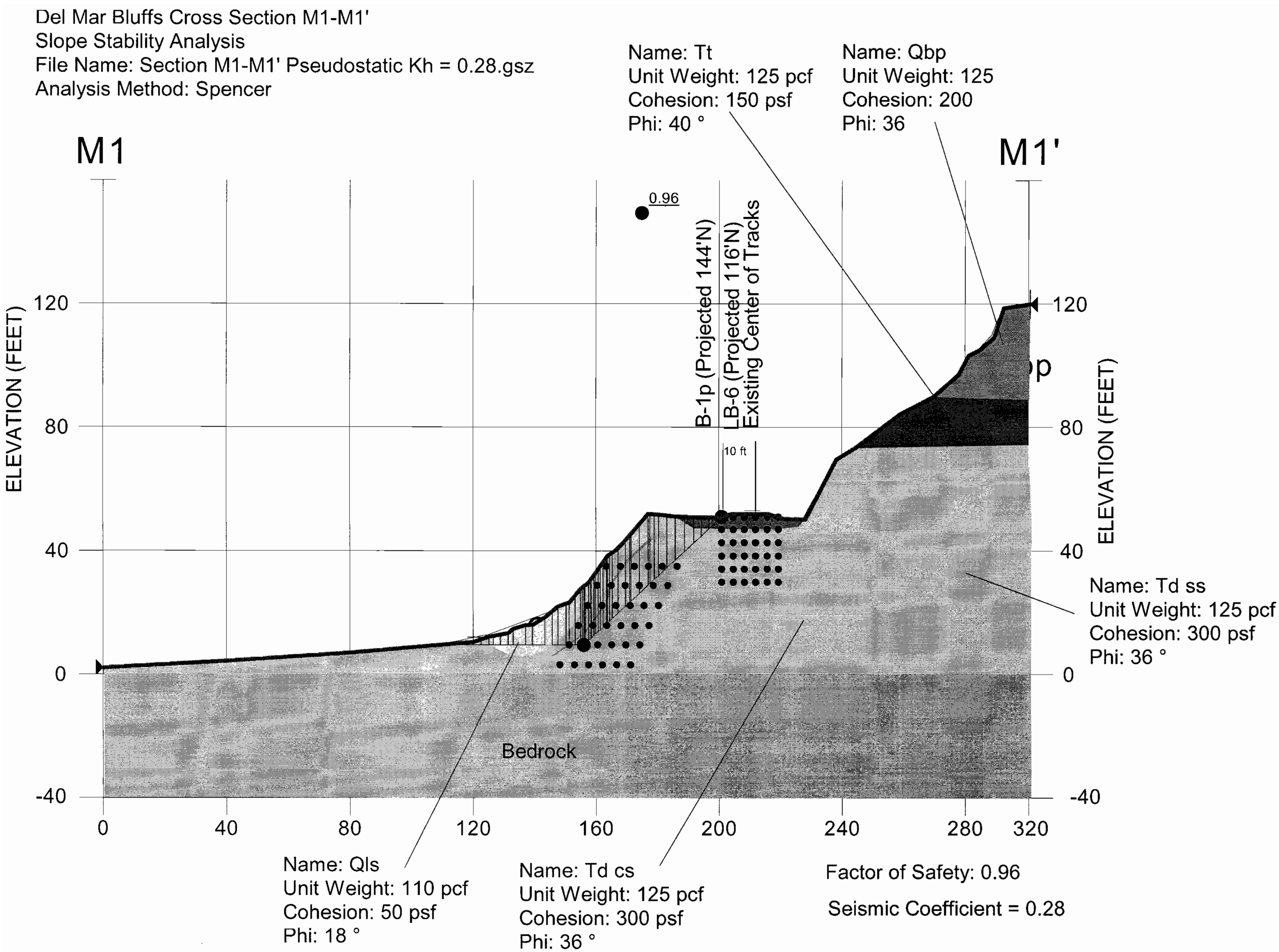
Del Mar Bluffs Cross Section M1-M1'
Slope Stability Analysis
File Name: Section M1-M1' Static B.gsz
Analysis Method: Spencer
Factor of Safety: 1.49
M1
M1'
1.49
Name: Tt
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 40 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
B-1p (Projected 144'N)
LB-6 (Projected 116'N)
Existing Center of Tracks
10 ft
ELEVATION (FEET)
120
80
40
0
-40
Bedrock
0
40
80
120
160
200
240
280
320
Name: Td ss
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °



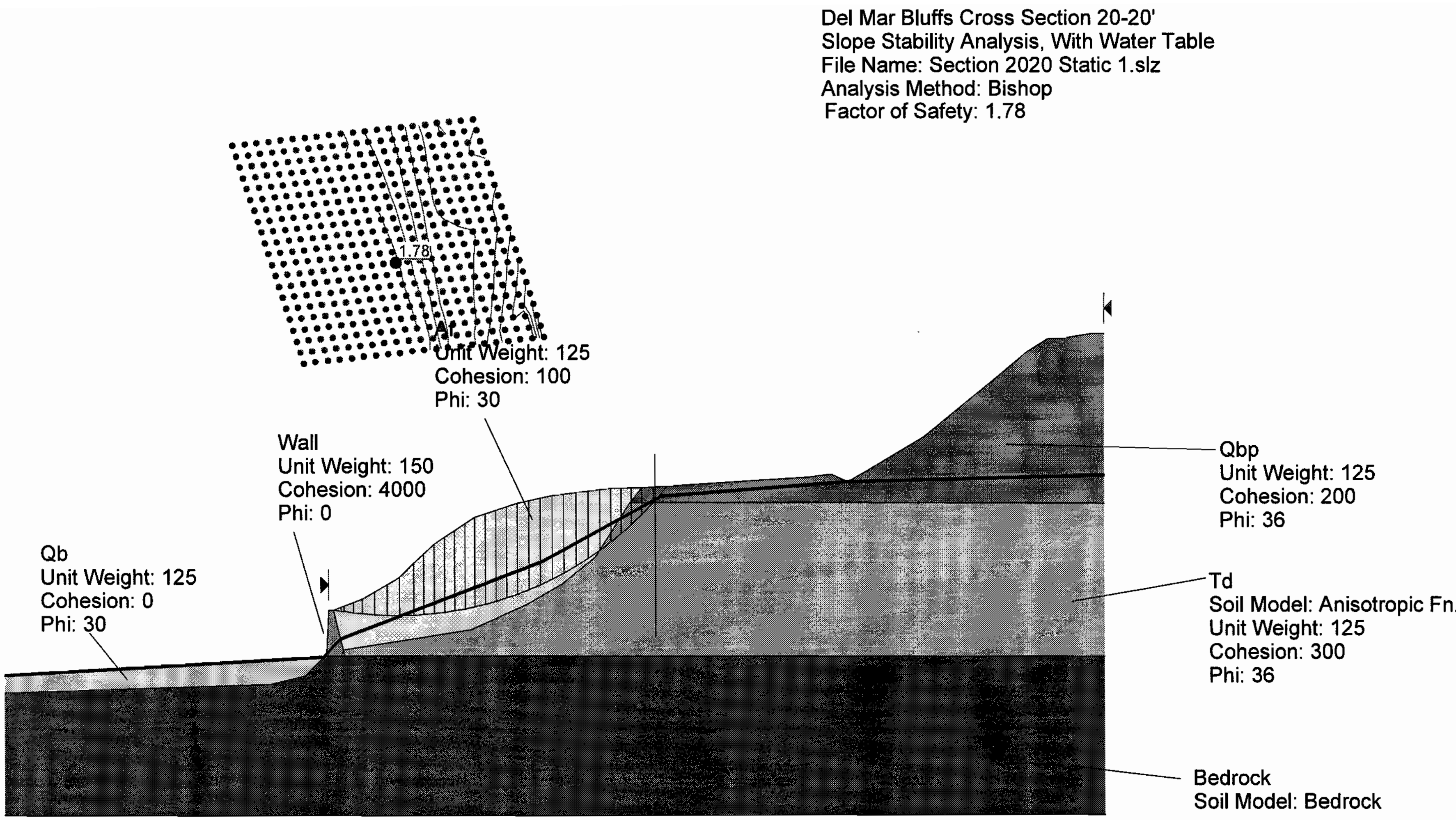
Del Mar Bluffs Cross Section M1-M1'
Slope Stability Analysis
File Name: Section M1-M1' Static B + Surcharge.gsz
Analysis Method: Spencer
Name: Tt
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 40 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
M1
M1'
1.49
B-1p (Projected 144'N)
LB-6 (Projected 116'N)
Existing Center of Tracks
10 ft
ELEVATION (FEET)
120
80
40
0
-40
Name: Td ss
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Bedrock
0
40
80
120
160
200
240
280
320
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 1.49
Surcharge = 3,000 psf

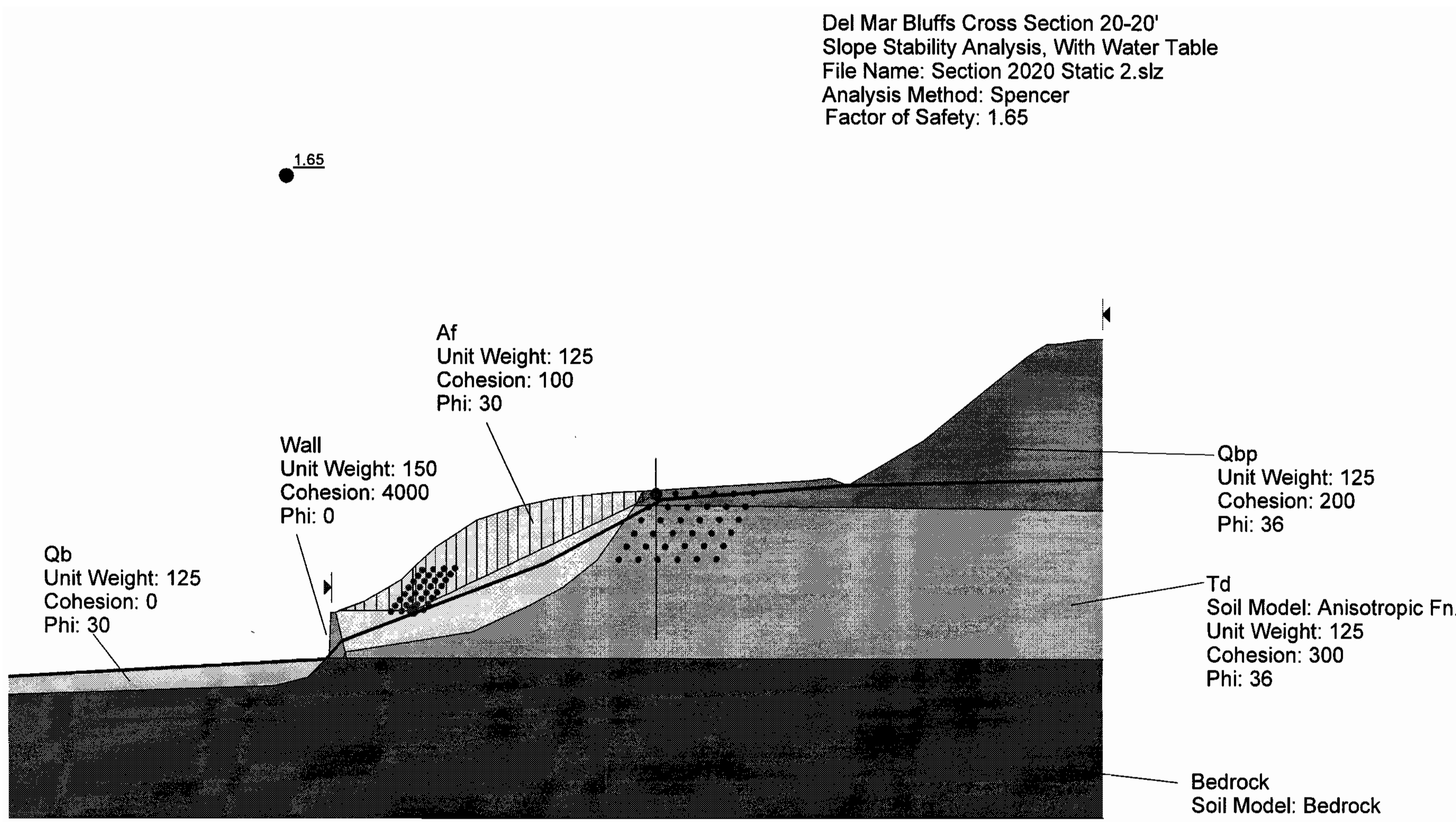


Del Mar Bluffs Cross Section M1-M1'
Slope Stability Analysis
File Name: Section M1-M1' Pseudostatic B Kh = 0.15.gsz
Analysis Method: Spencer
Name: Tt
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 40 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
M1
M1'
1.16
B-1p (Projected 144'N)
LB-6 (Projected 116'N)
Existing Center of Tracks
10 ft
ELEVATION (FEET)
Name: Td ss
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Bedrock
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 1.16
Seismic Coefficient = 0.15

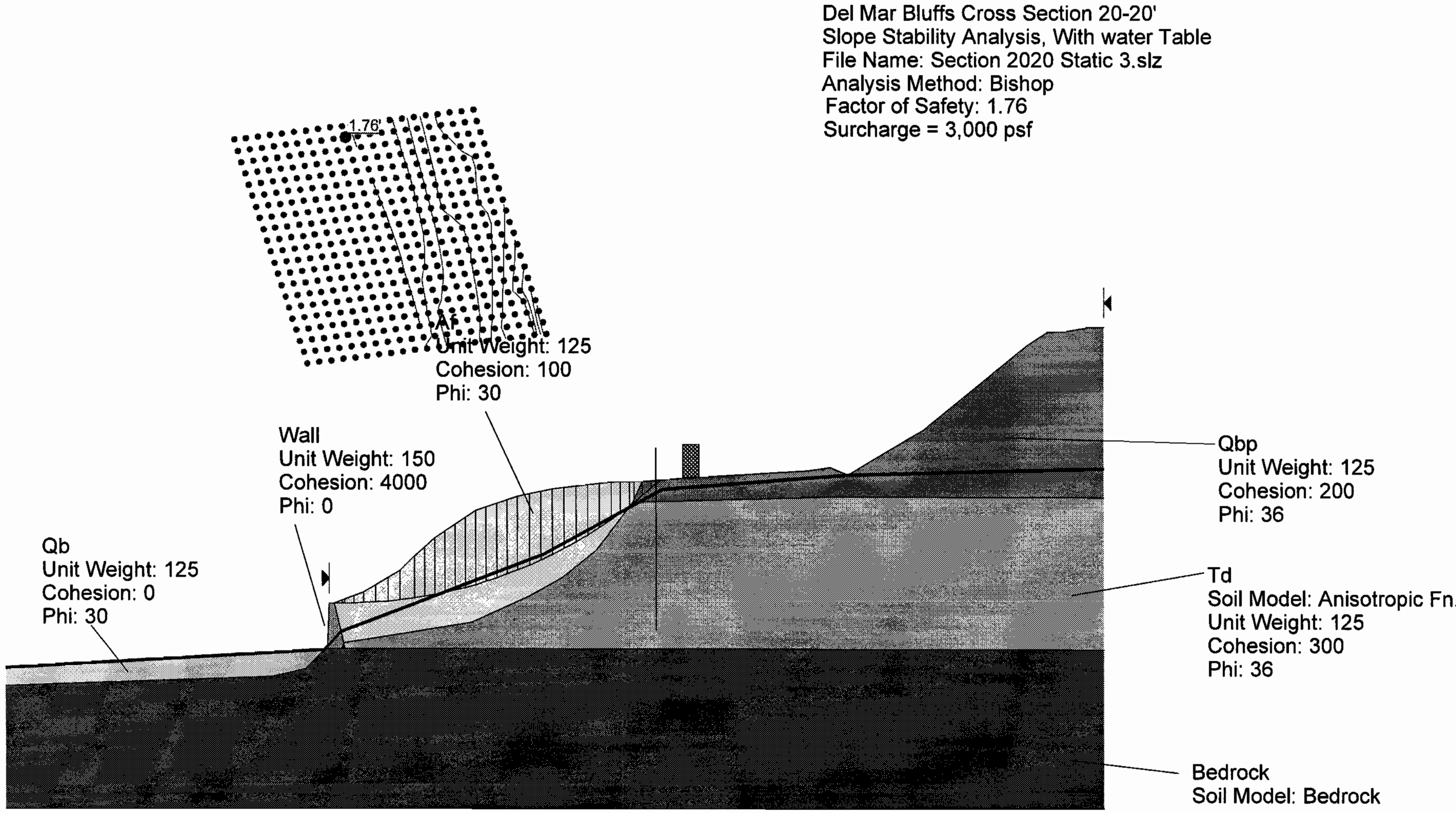

Del Mar Bluffs Cross Section M1-M1'
Slope Stability Analysis
File Name: Section M1-M1' Pseudostatic Kh = 0.28.gsz
Analysis Method: Spencer
Name: Tt
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 40 °
Name: Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
M1
M1'
0.96
B-1p (Projected 144'N)
LB-6 (Projected 116'N)
Existing Center of Tracks
10 ft
ELEVATION (FEET)
ELEVATION (FEET)
120
80
40
0
-40
0
40
80
120
160
200
240
280
320
Bedrock
Name: Td ss
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Name: Qls
Unit Weight: 110 pcf
Cohesion: 50 psf
Phi: 18 °
Name: Td cs
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 36 °
Factor of Safety: 0.96
Seismic Coefficient = 0.28

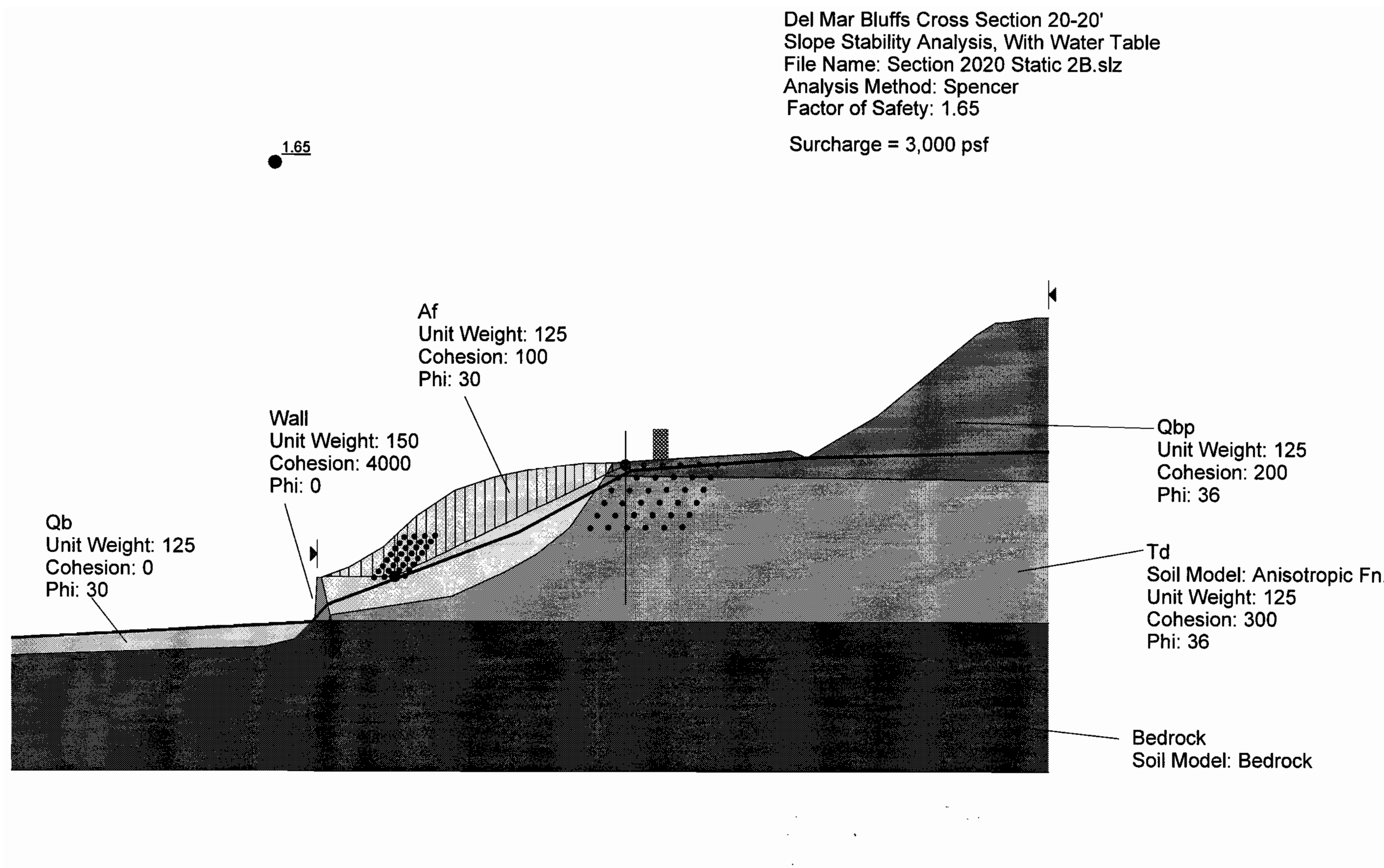
Cross Section 20-20’

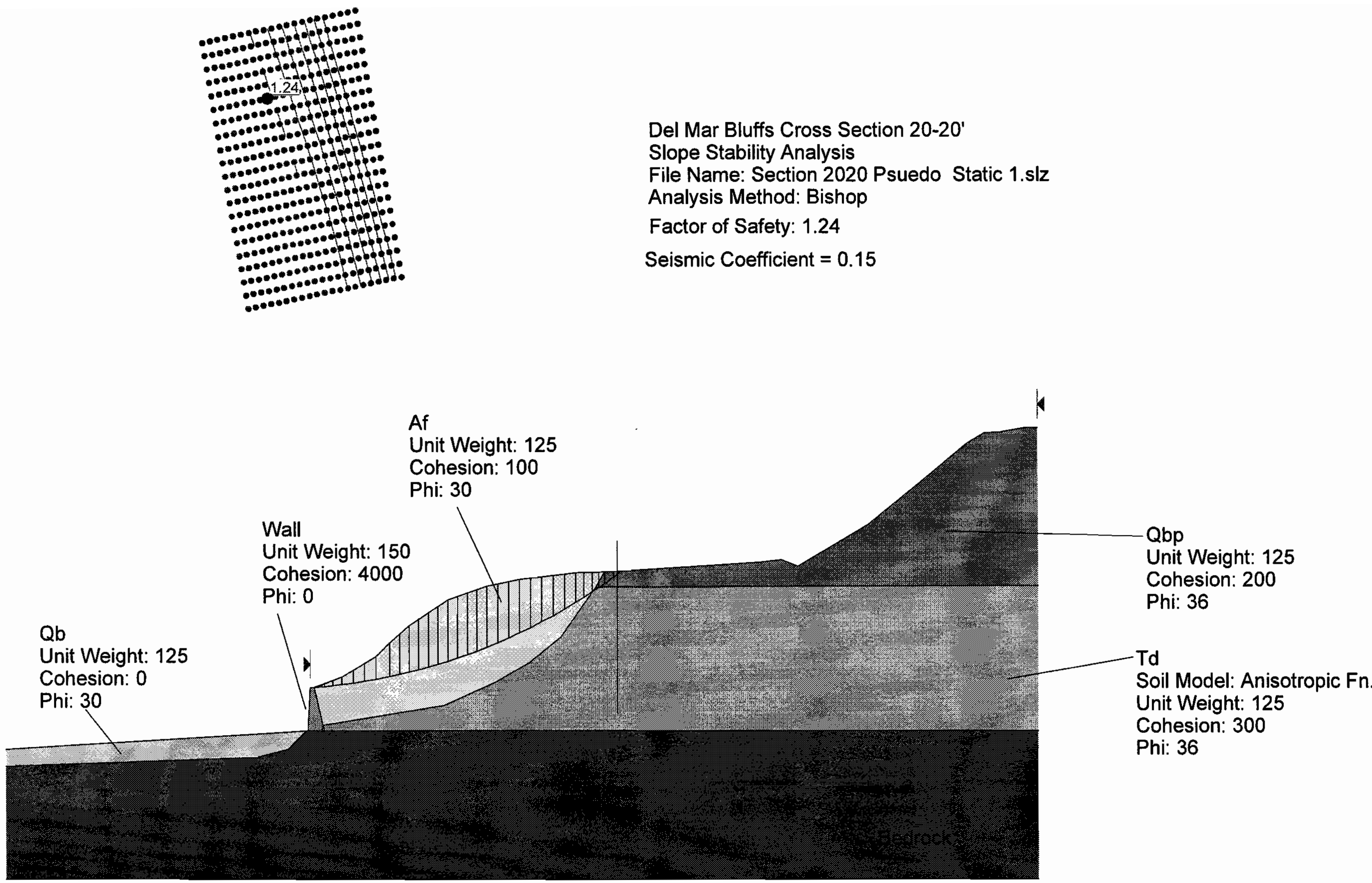

Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis, With Water Table
File Name: Section 2020 Static 1.slz
Analysis Method: Bishop
Factor of Safety: 1.78
1.78
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock
Soil Model: Bedrock

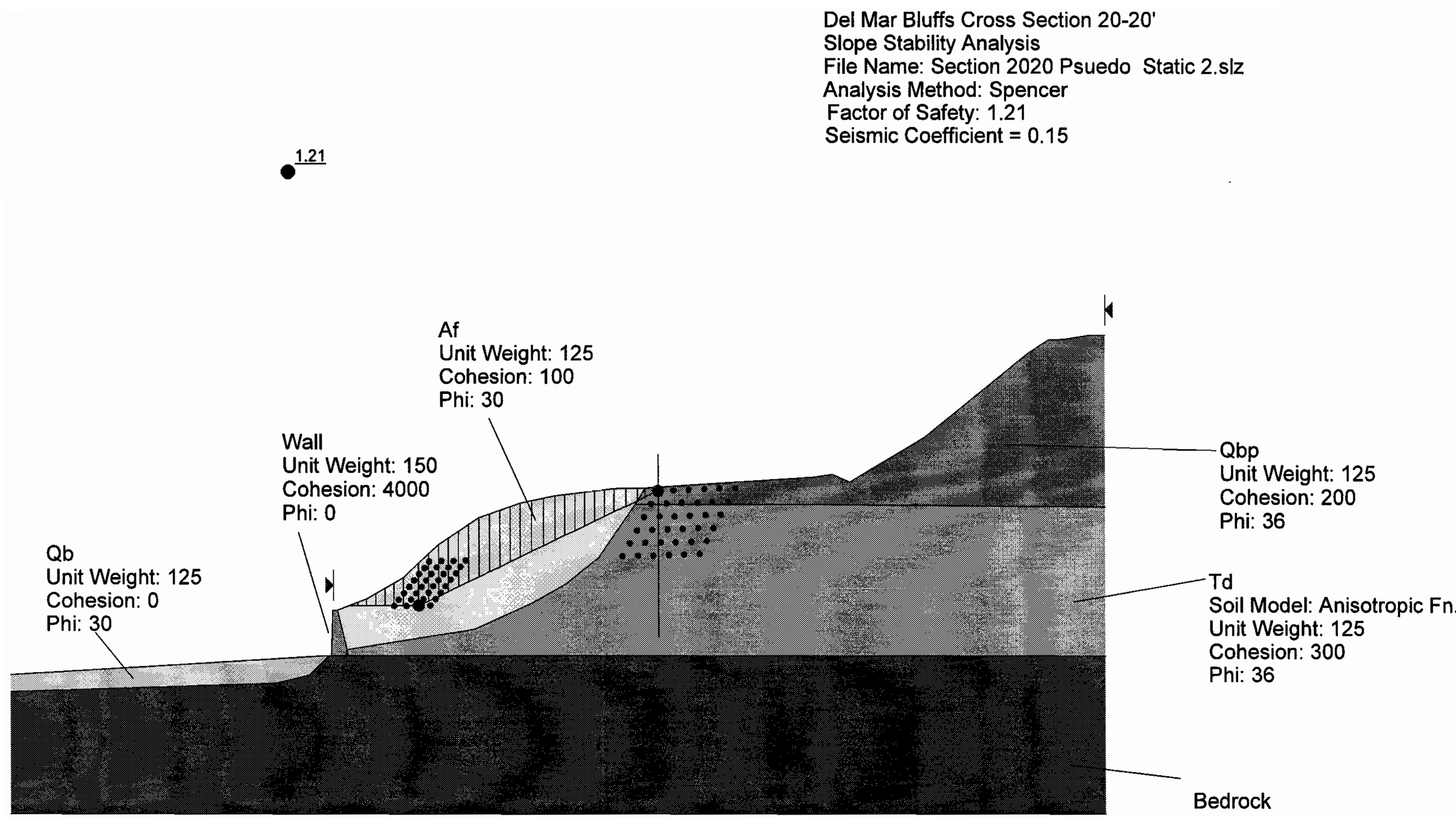


Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis, With Water Table
File Name: Section 2020 Static 2.slz
Analysis Method: Spencer
Factor of Safety: 1.65
1.65
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock
Soil Model: Bedrock

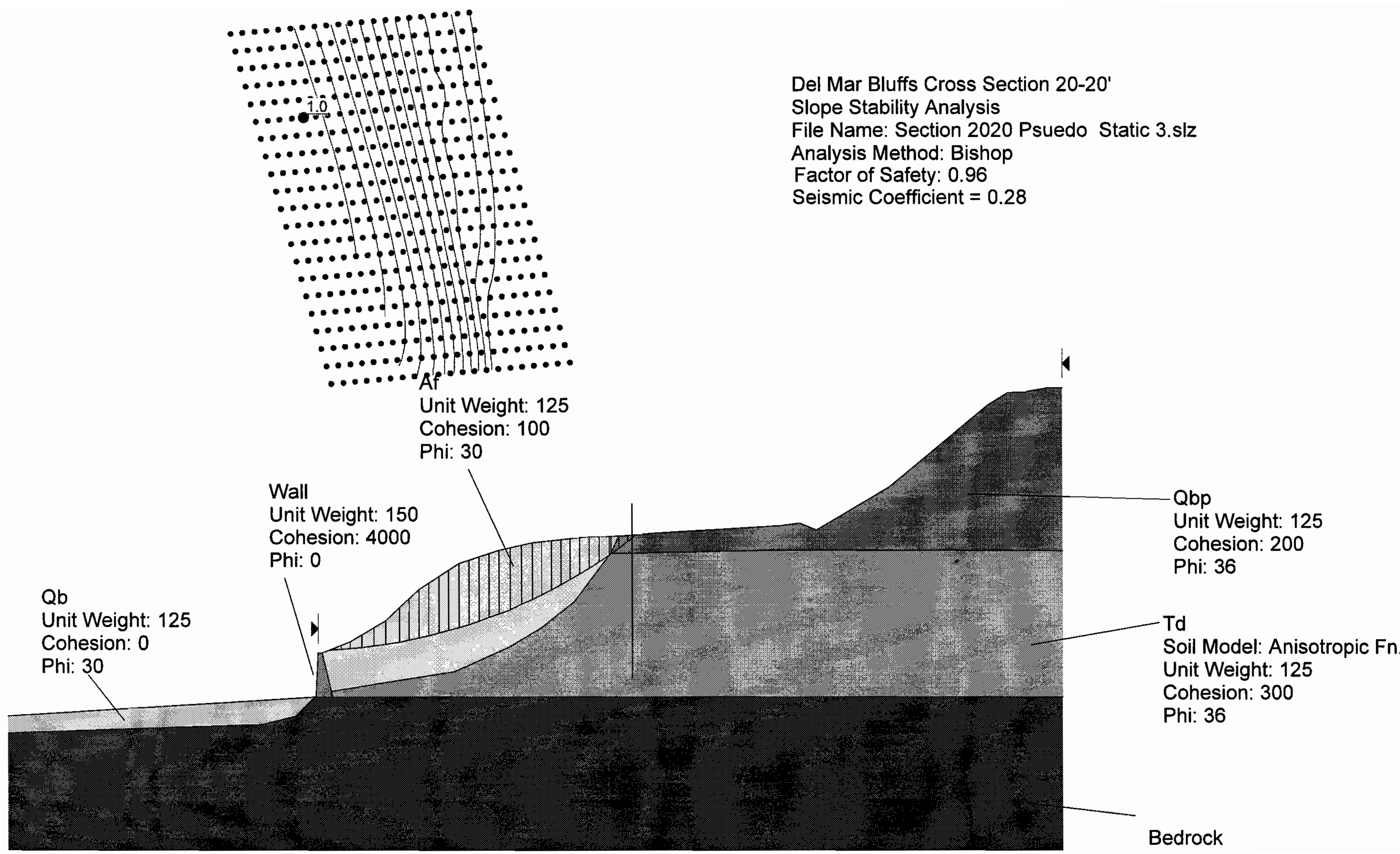

Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis, With water Table
File Name: Section 2020 Static 3.slz
Analysis Method: Bishop
Factor of Safety: 1.76
Surcharge = 3,000 psf
1.76
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock
Soil Model: Bedrock


Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis, With Water Table
File Name: Section 2020 Static 2B.slz
Analysis Method: Spencer
Factor of Safety: 1.65
Surcharge = 3,000 psf
1.65
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock
Soil Model: Bedrock

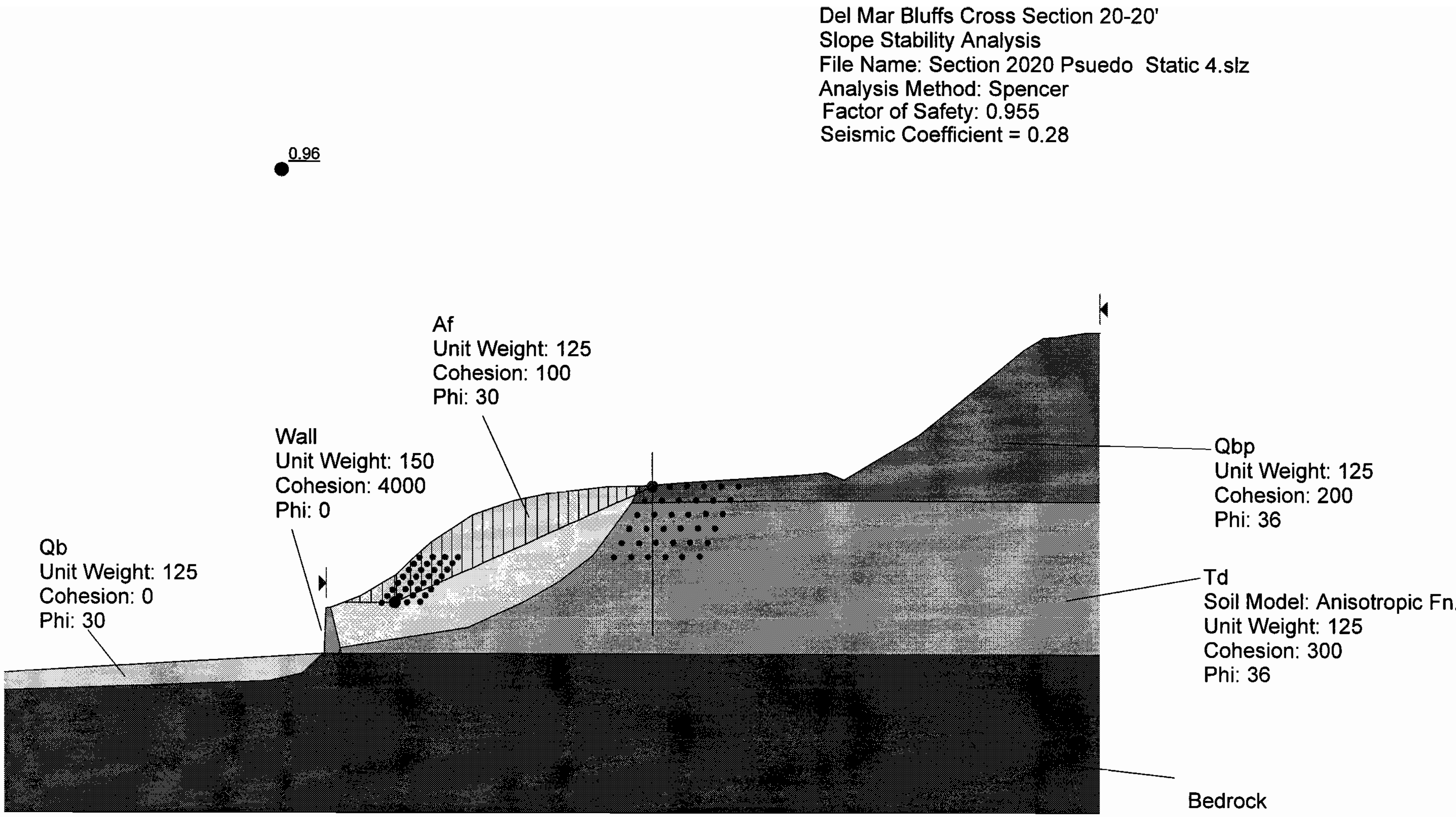

1.24
Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis
File Name: Section 2020 Psuedo Static 1.slz
Analysis Method: Bishop
Factor of Safety: 1.24
Seismic Coefficient = 0.15
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock



Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis
File Name: Section 2020 Psuedo Static 2.slz
Analysis Method: Spencer
Factor of Safety: 1.21
Seismic Coefficient = 0.15
1.21
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock



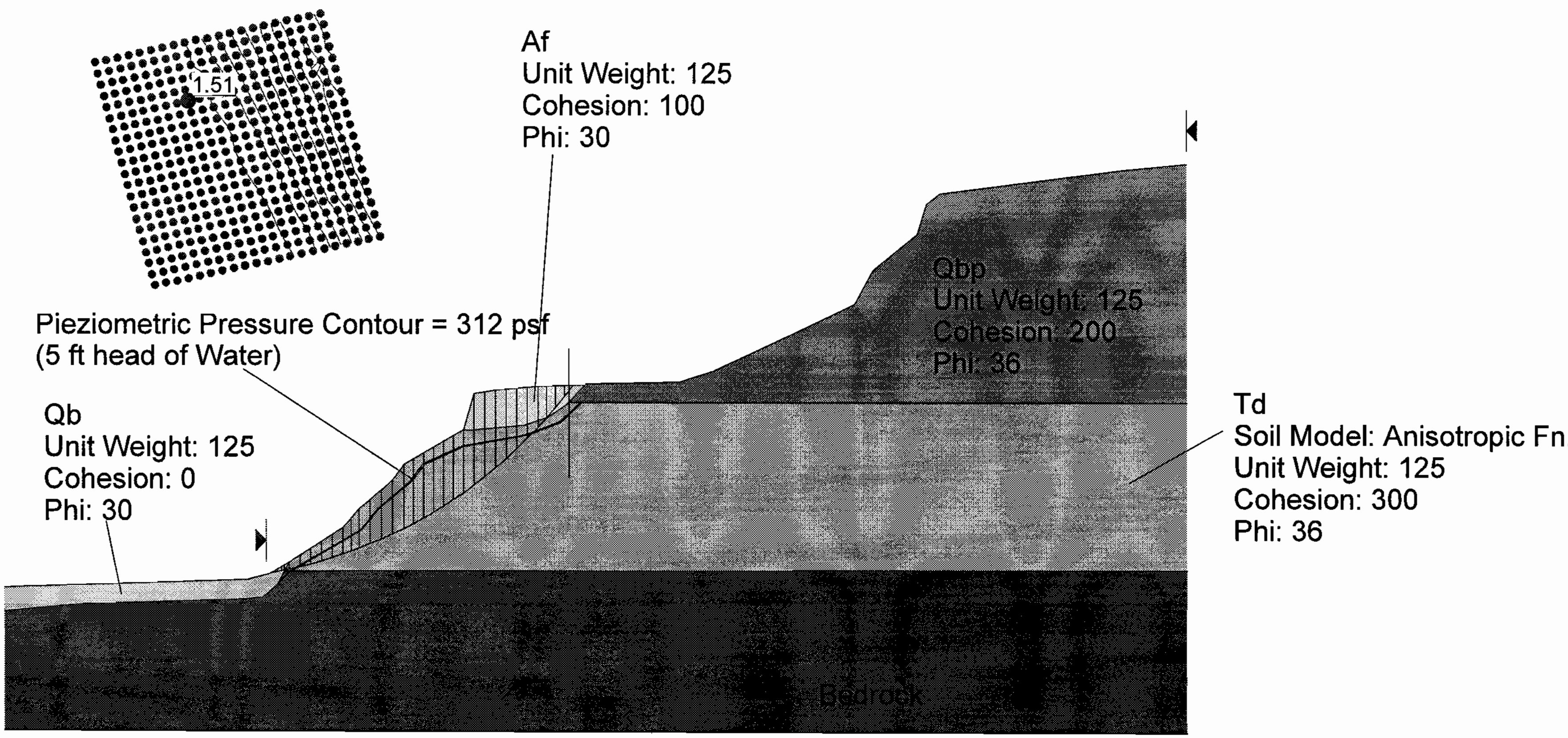
Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis
File Name: Section 2020 Psuedo Static 3.slz
Analysis Method: Bishop
Factor of Safety: 0.96
Seismic Coefficient = 0.28
1.0
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock



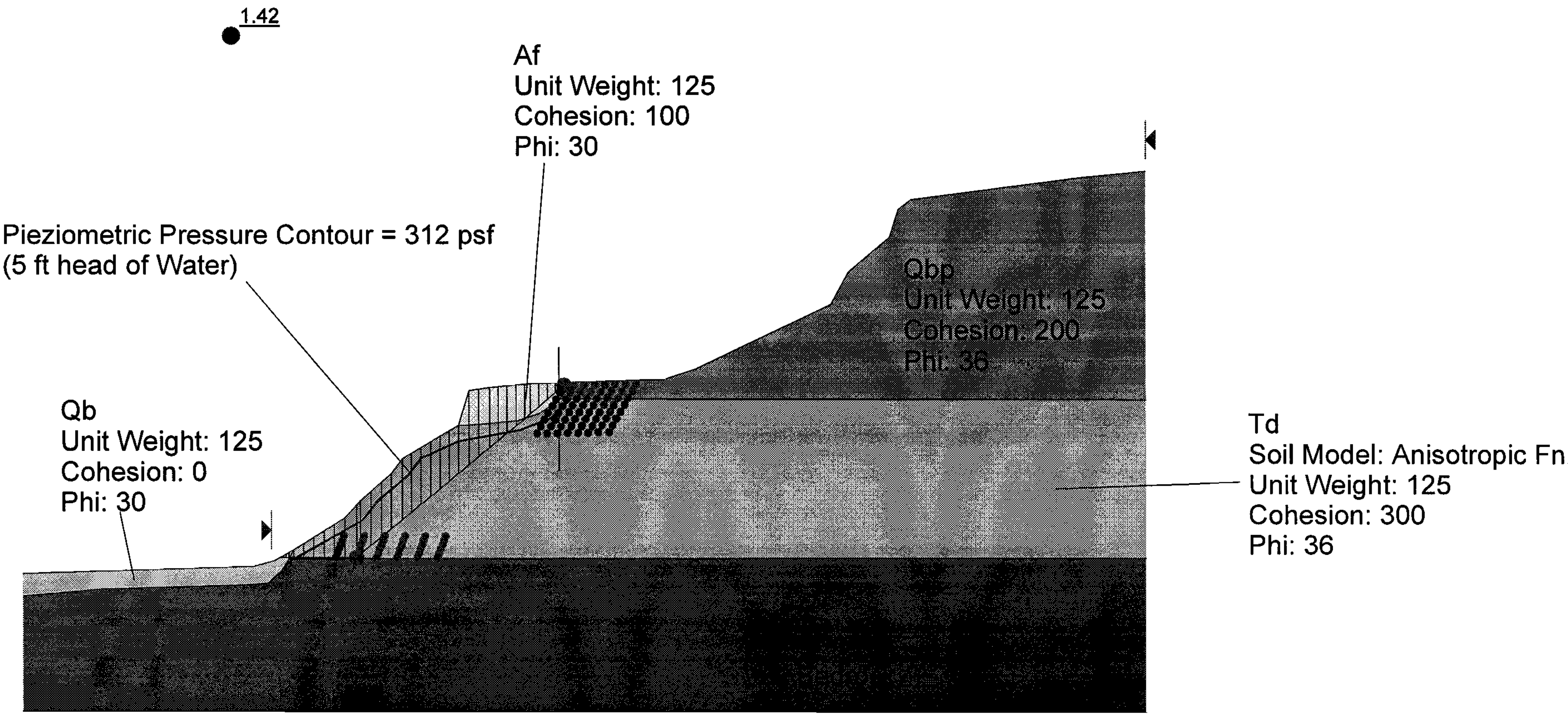
Del Mar Bluffs Cross Section 20-20'
Slope Stability Analysis
File Name: Section 2020 Psuedo Static 4.slz
Analysis Method: Spencer
Factor of Safety: 0.955
Seismic Coefficient = 0.28
0.96
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Wall
Unit Weight: 150
Cohesion: 4000
Phi: 0
Qb
Unit Weight: 125
Cohesion: 0
Phi: 30
Qbp
Unit Weight: 125
Cohesion: 200
Phi: 36
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Bedrock

Cross Section 21-21’

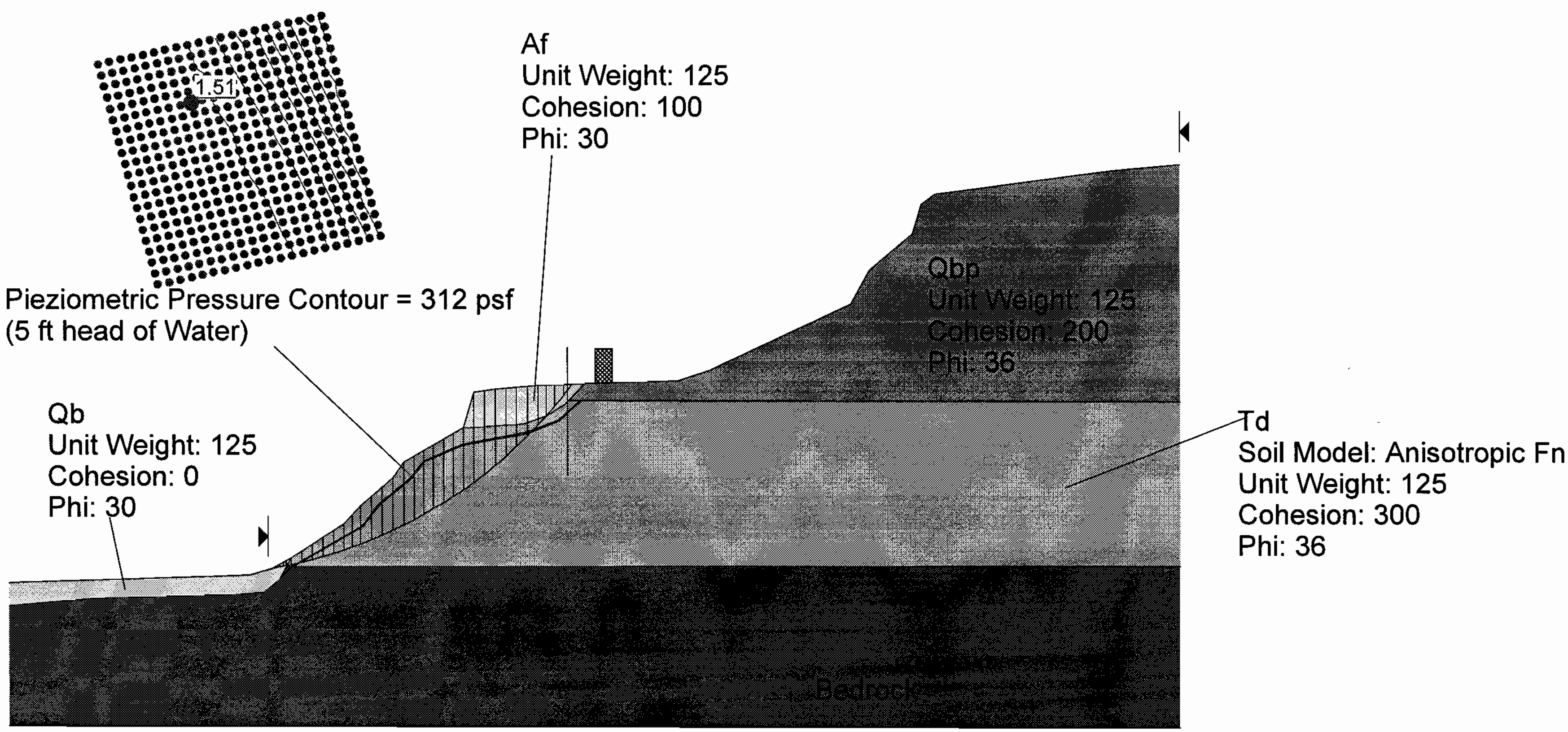
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 5 ft Water Static 1.slz
Analysis Method: Bishop
Factor of Safety: 1.51



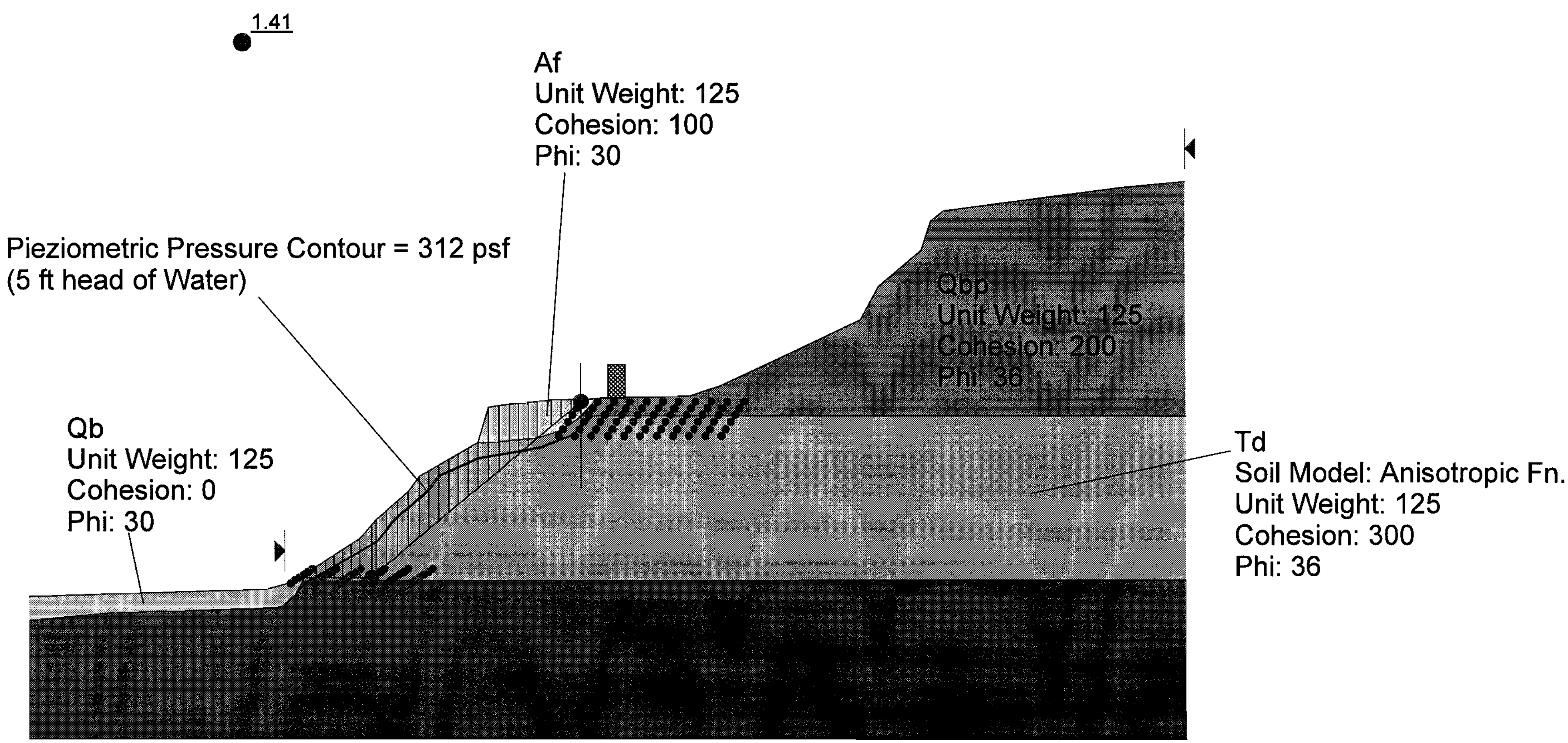
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 5 ft Water Static 2.slz
Analysis Method: Spencer
Factor of Safety: 1.42



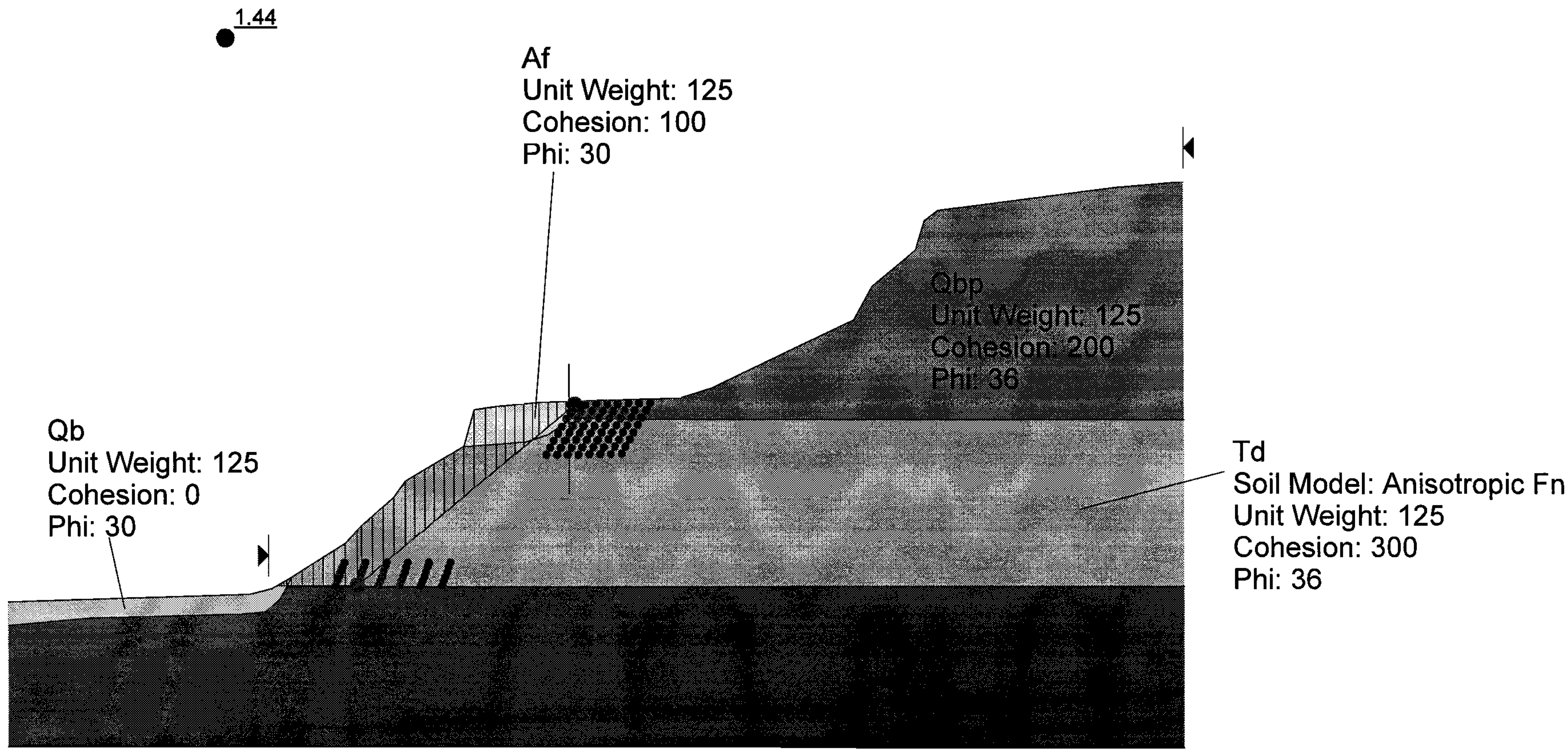
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 5 ft Water Static 3.slz
Analysis Method: Bishop
Factor of Safety: 1.51
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 5 ft Water Static 4.slz
Analysis Method: Spencer
Factor of Safety: 1.41
Surcharge = 3,000 psf

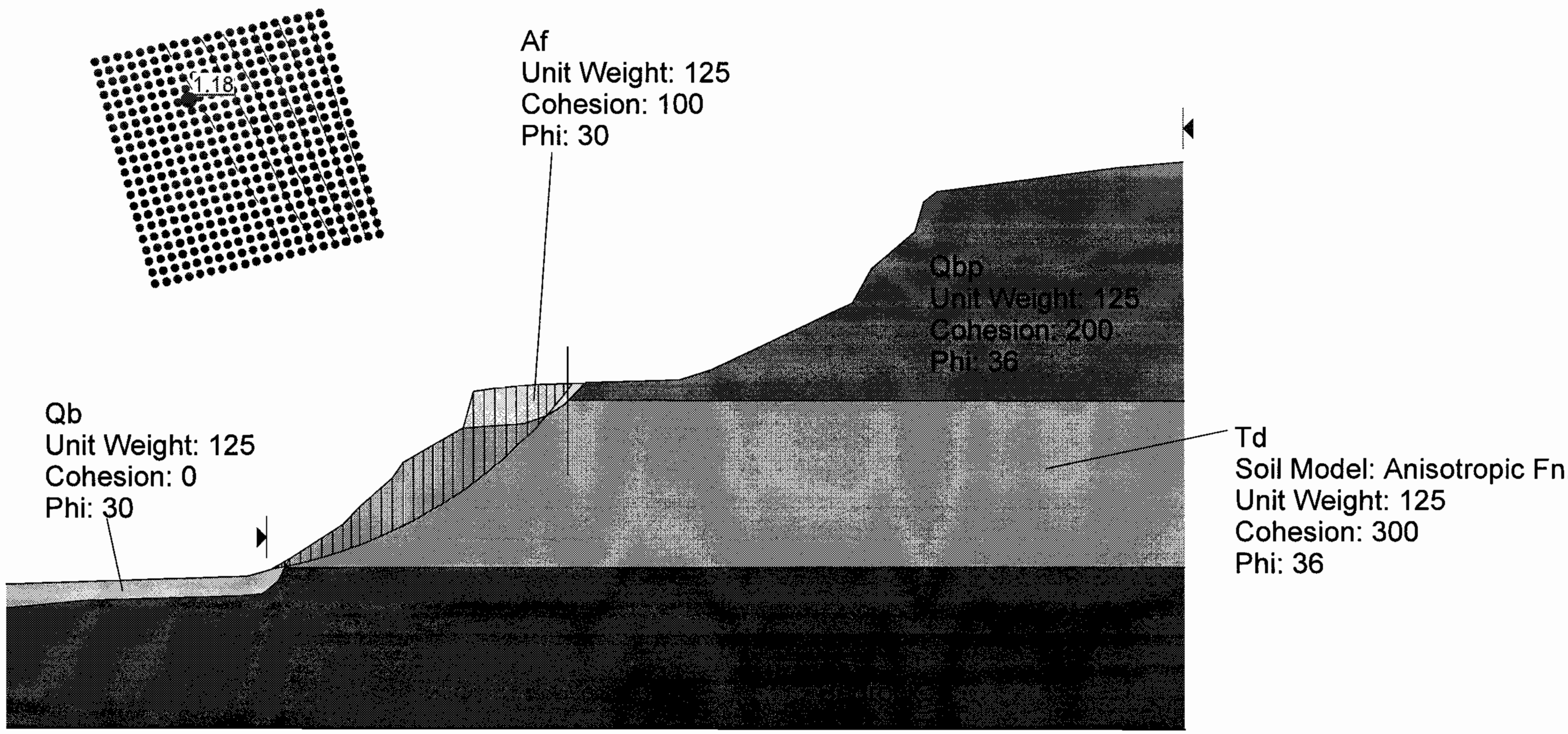


Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis, No Water
File Name: Section 2121 Static 2.slz
Analysis Method: Spencer
Factor of Safety: 1.44

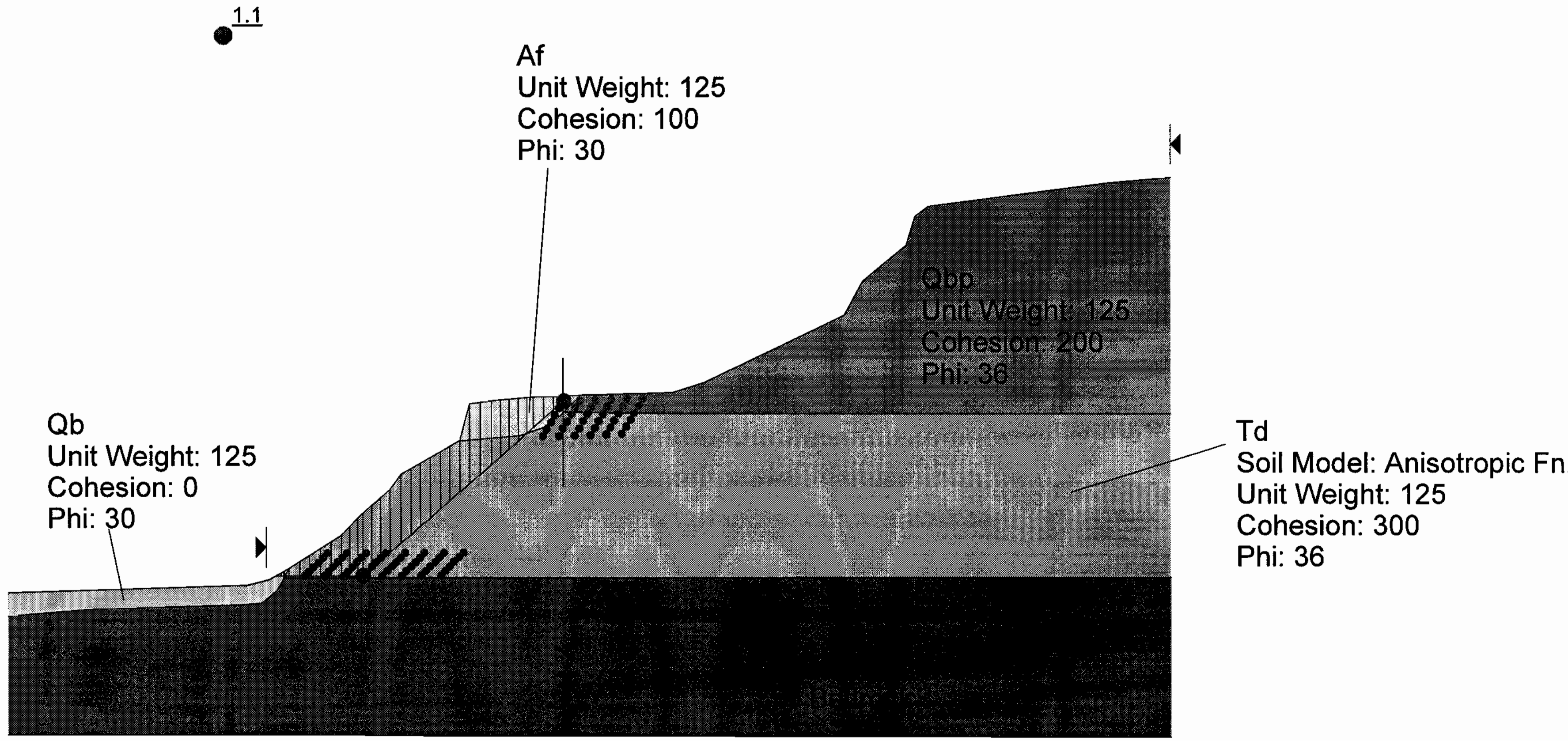


Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 Psuedo Static 1.slz
Analysis Method: Bishop
Factor of Safety: 1.18

Seismic Coefficient = 0.15

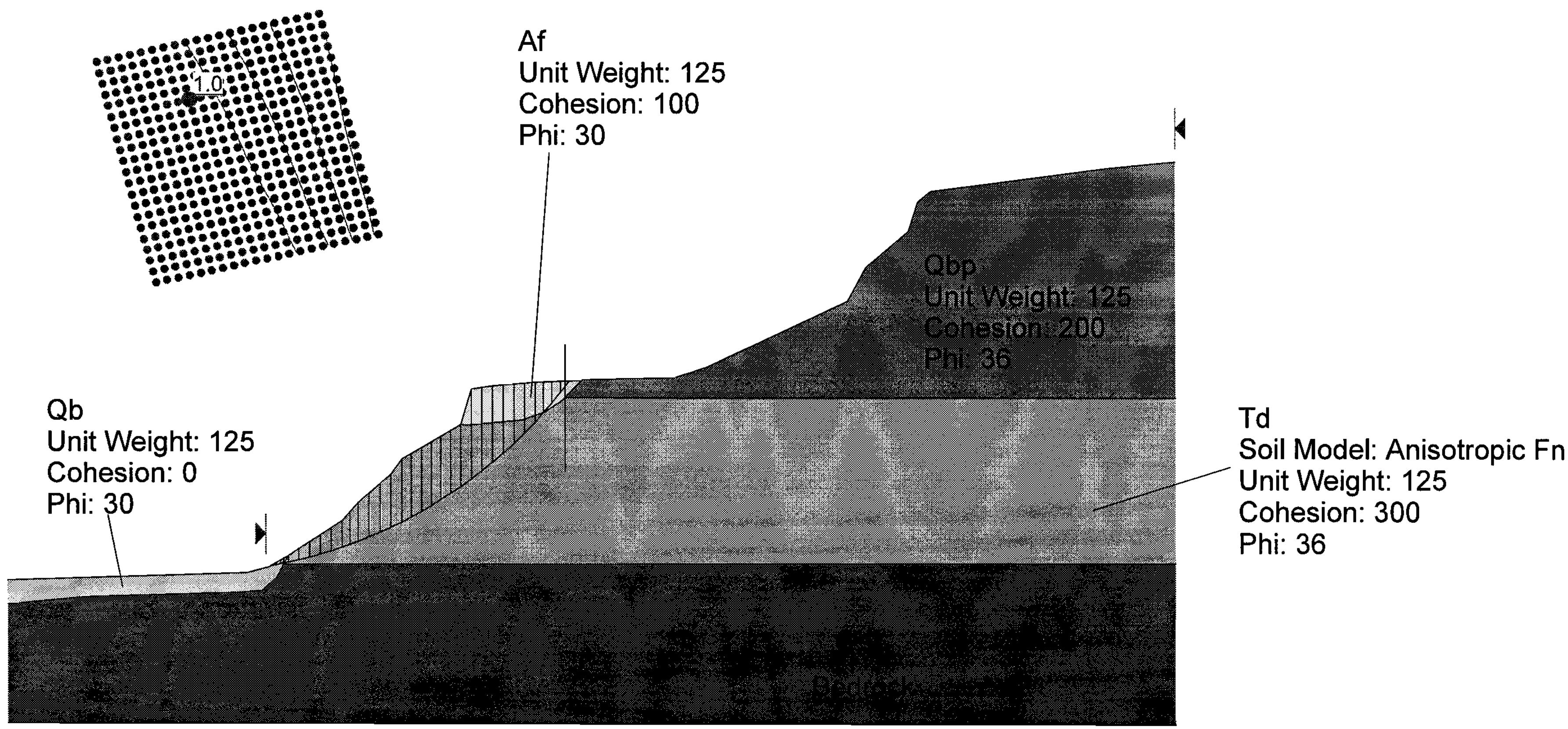


Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 Psuedo Static 2.slz
Analysis Method: Spencer
Factor of Safety: 1.1
Seismic Coefficient = 0.15

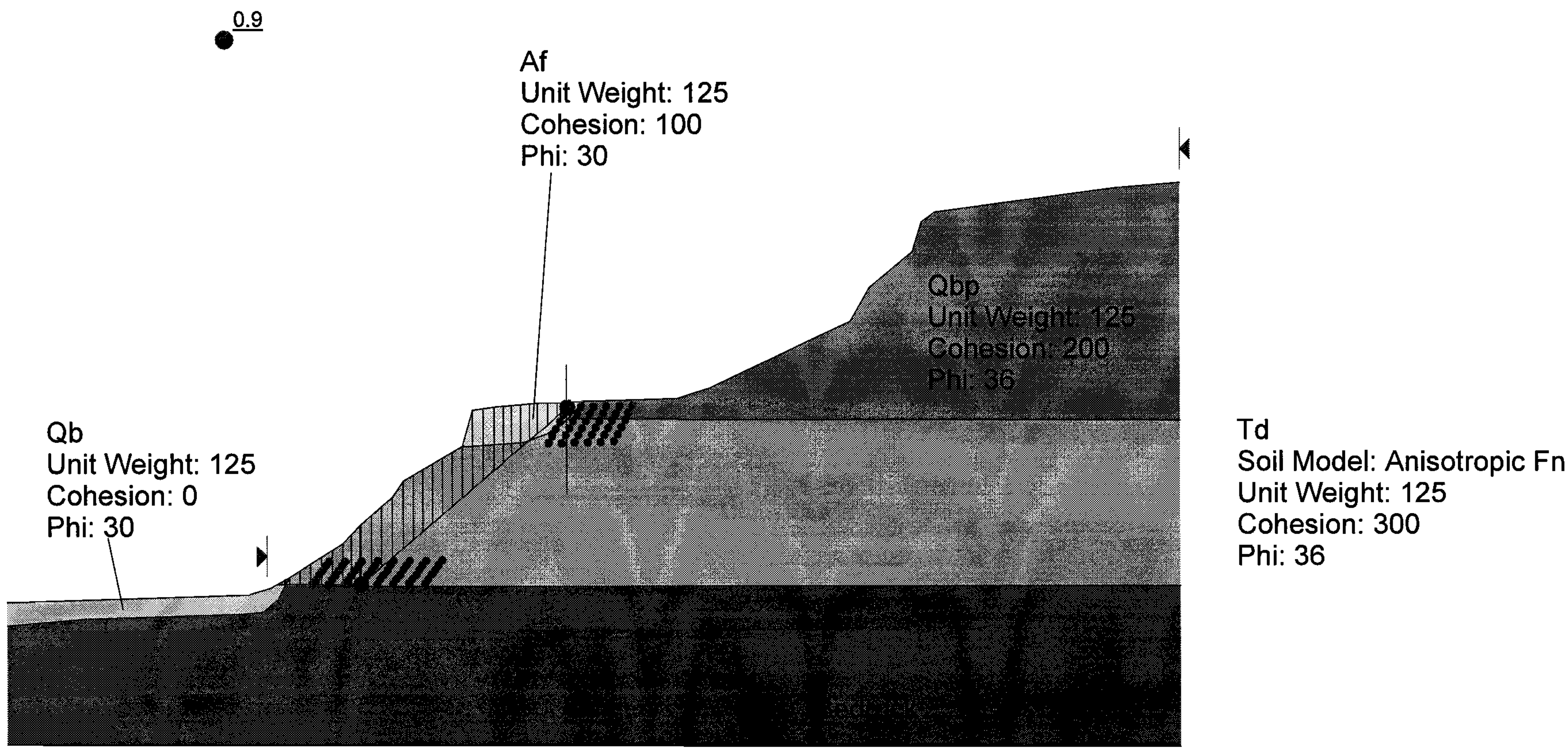


Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 Psuedo Static 3.slz
Analysis Method: Bishop
Factor of Safety: 0.97

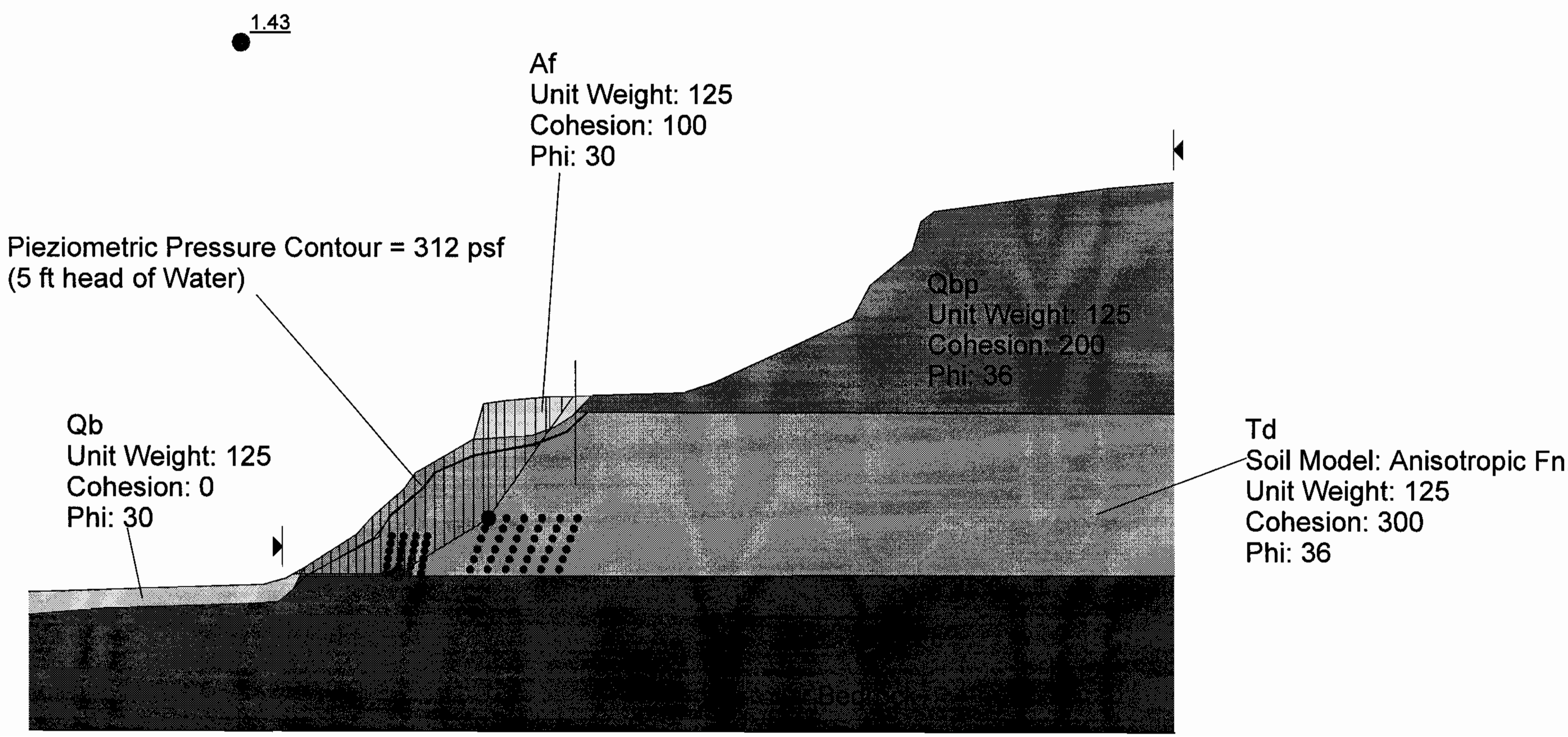
Seismic Coefficient = 0.28



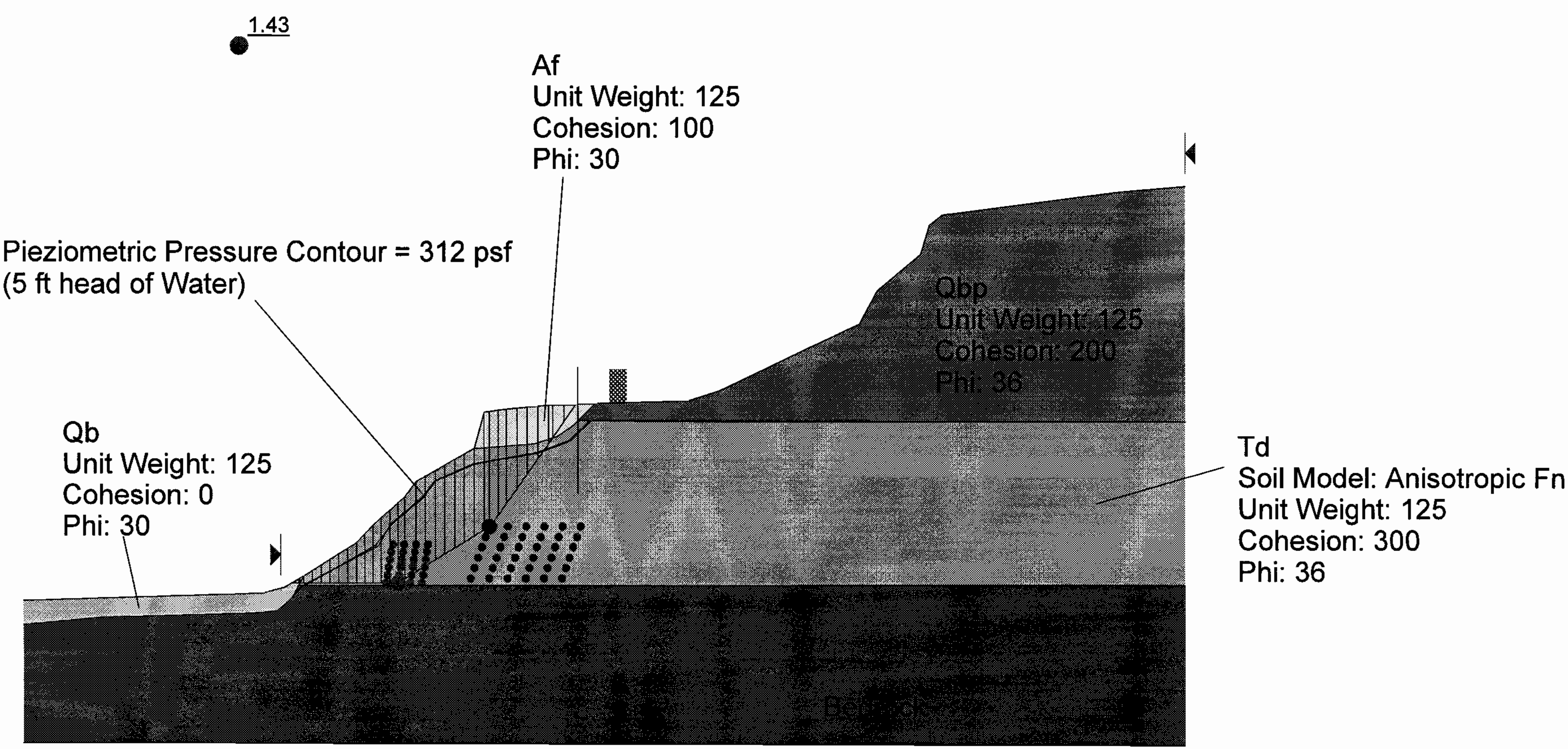
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 Psuedo Static 4.slz
Analysis Method: Spencer
Factor of Safety: 0.92
Seismic Coefficient = 0.28



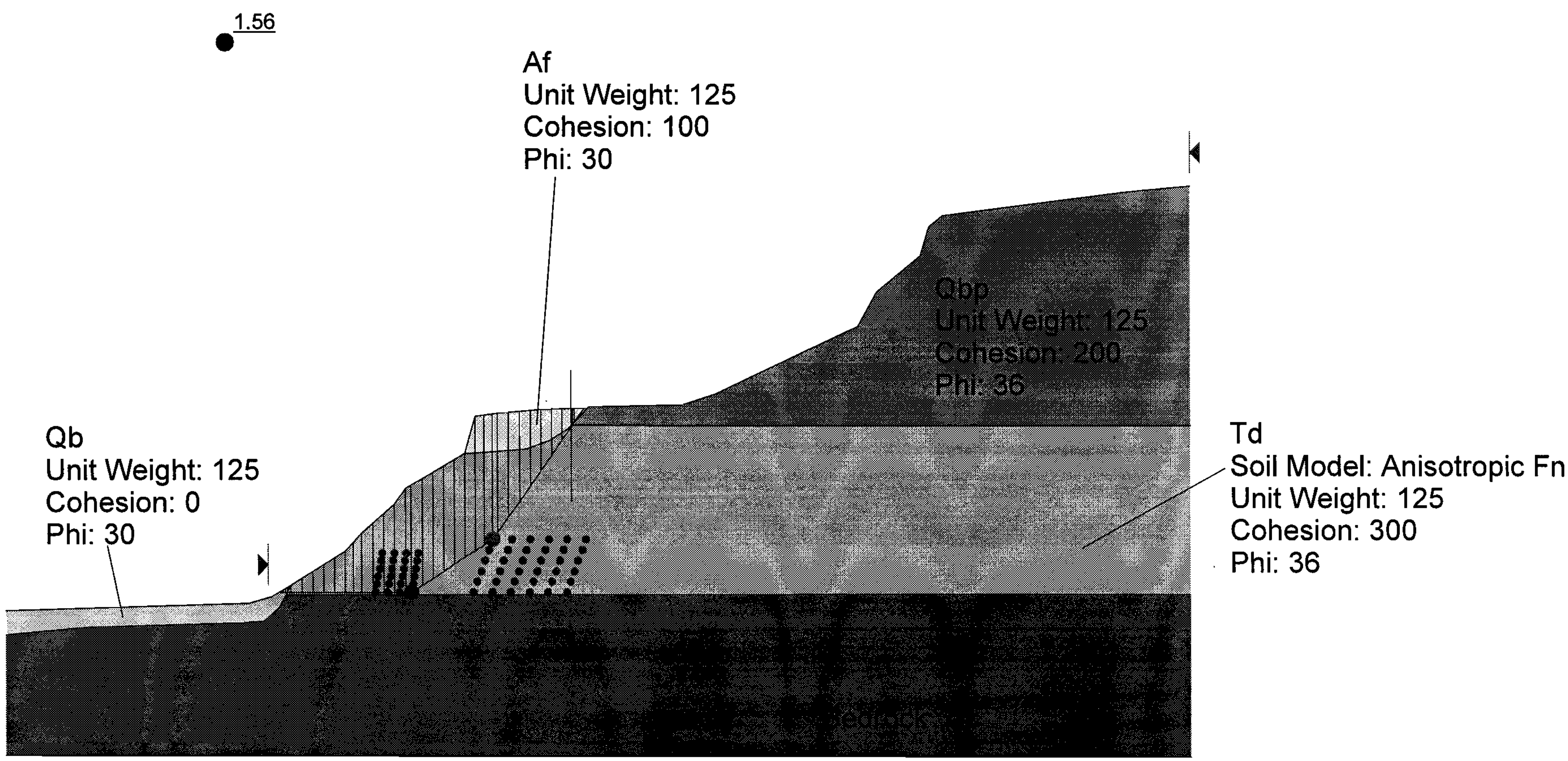
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 5 ft Water Static 2B.slz
Analysis Method: Spencer
Factor of Safety: 1.43



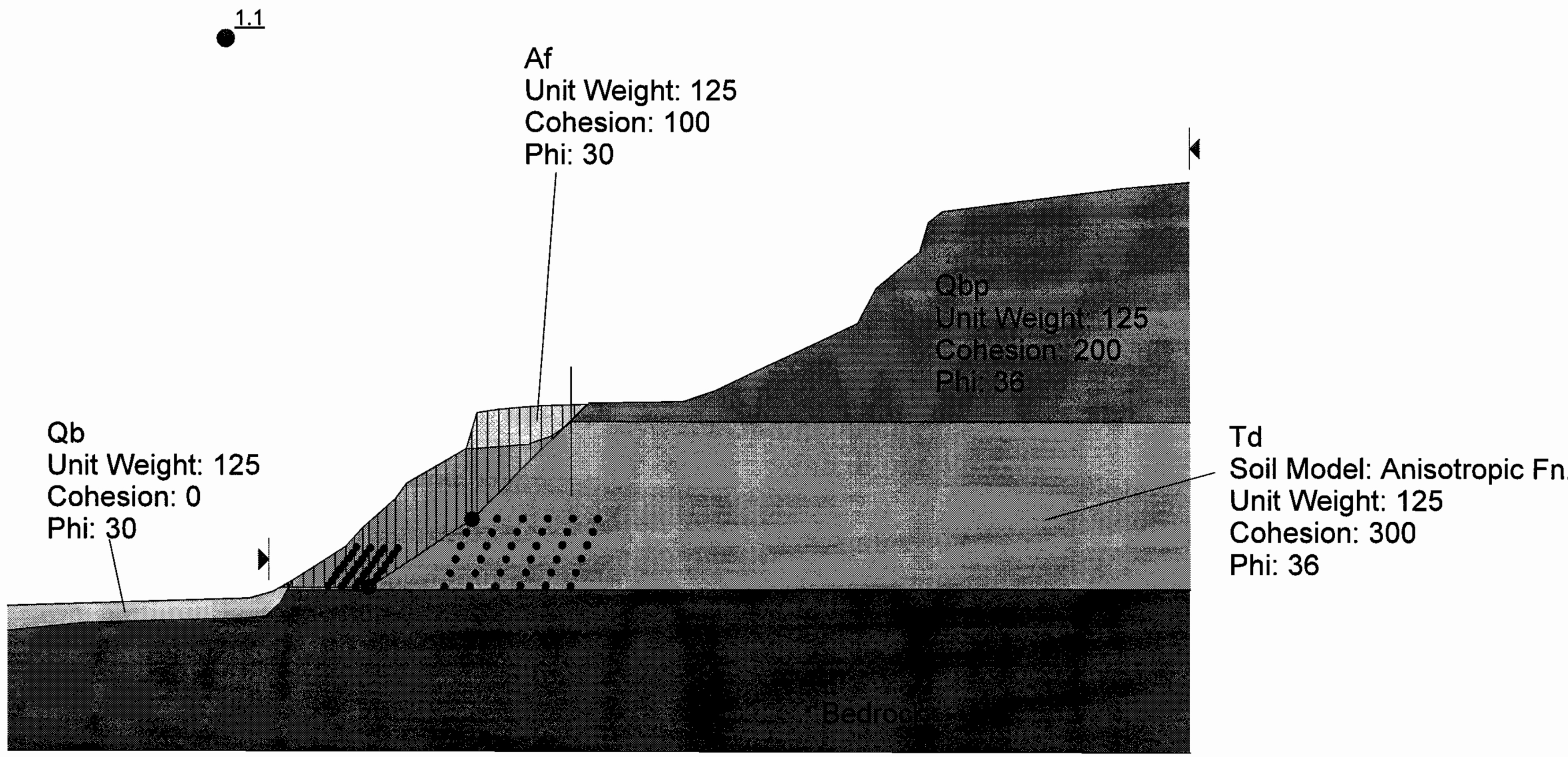
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 5 ft Water Static 4B.slz
Analysis Method: Spencer
Factor of Safety: 1.43
surcharge = 3,000 psf



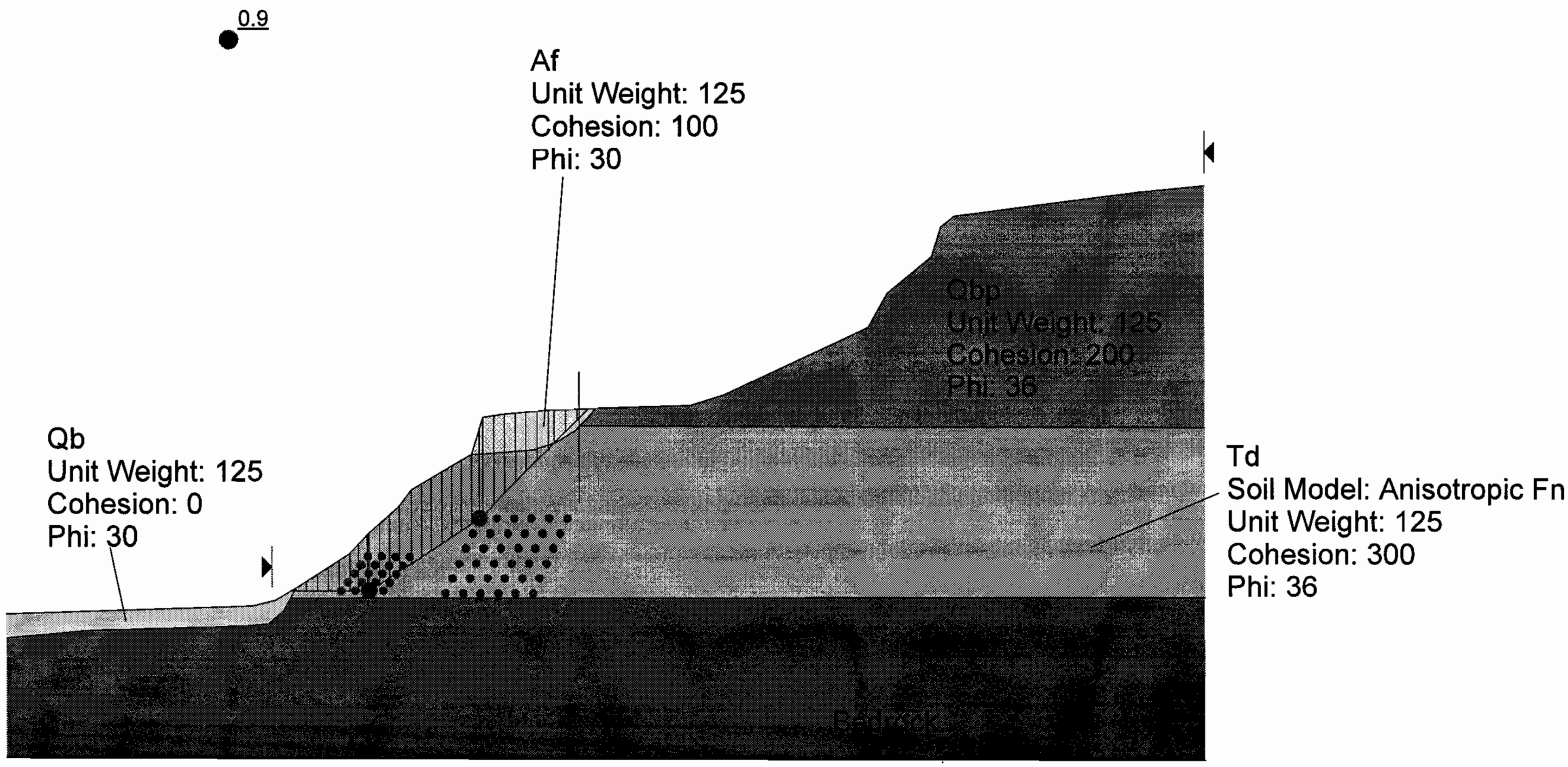
Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis, No Water
File Name: Section 2121 Static 2B.slz
Analysis Method: Spencer
Factor of Safety: 1.56



Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 Psuedo Static 2B.slz
Analysis Method: Spencer
Factor of Safety: 1.1
Seismic Coefficient = 0.15

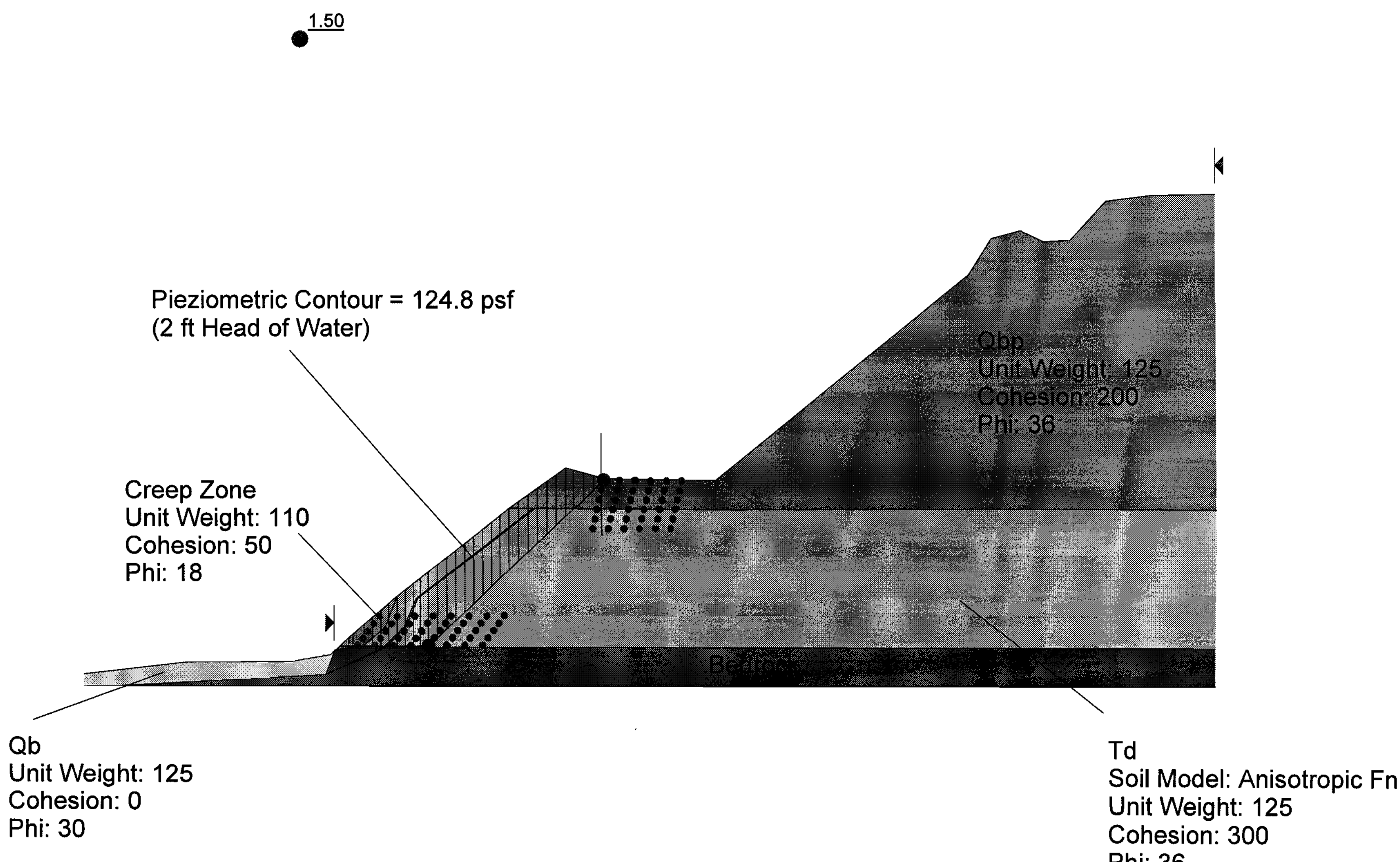


Del Mar Bluffs Cross Section 21-21'
C: Slope Stability Analysis
File Name: Section 2121 Psuedo Static 4B.slz
Analysis Method: Spencer
Factor of Safety: 0.91
Seismic Coefficient = 0.28

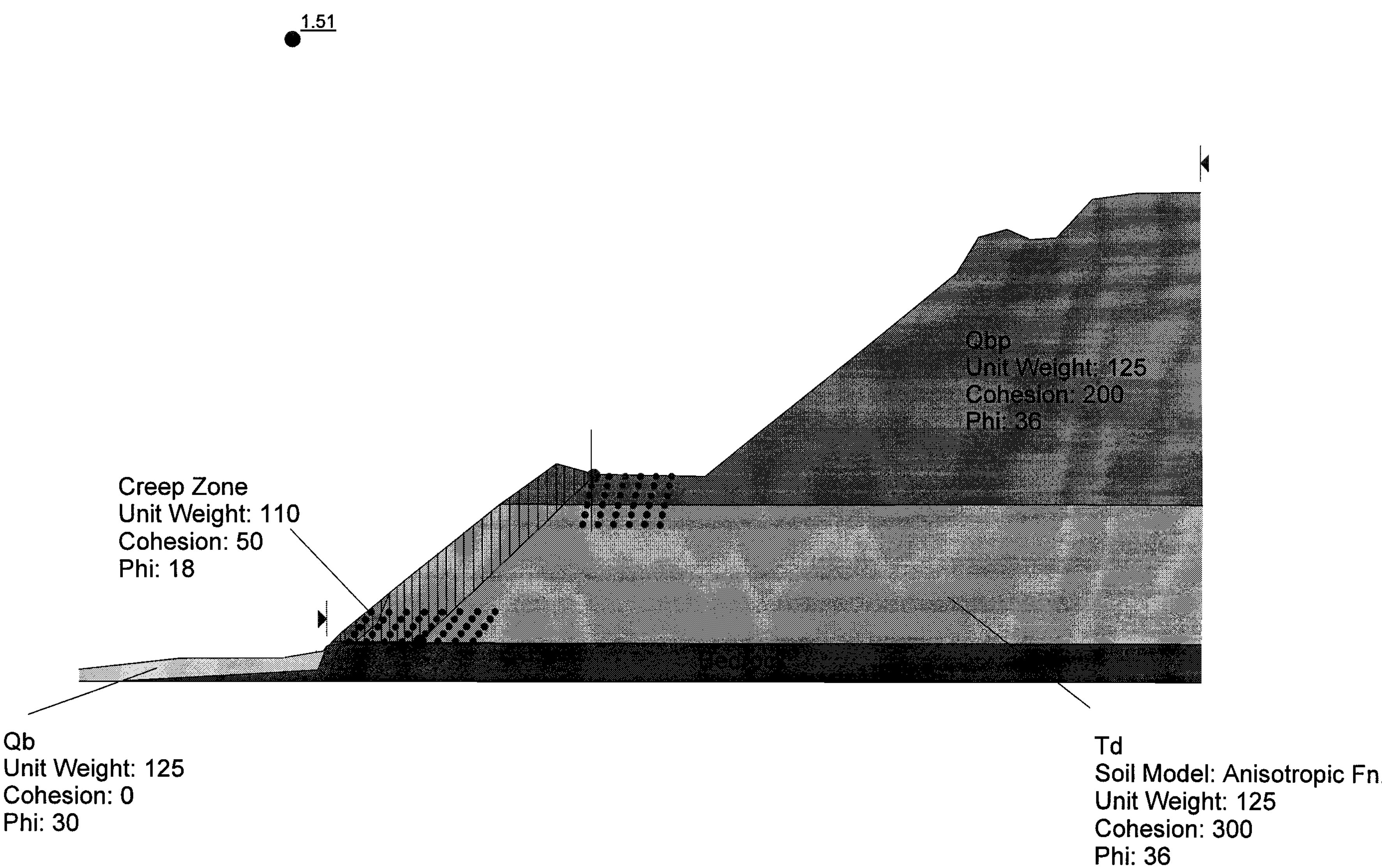


Cross Section 22-22’

Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Static 2.slz
Analysis Method: Spencer
Factor of Safety: 1.5

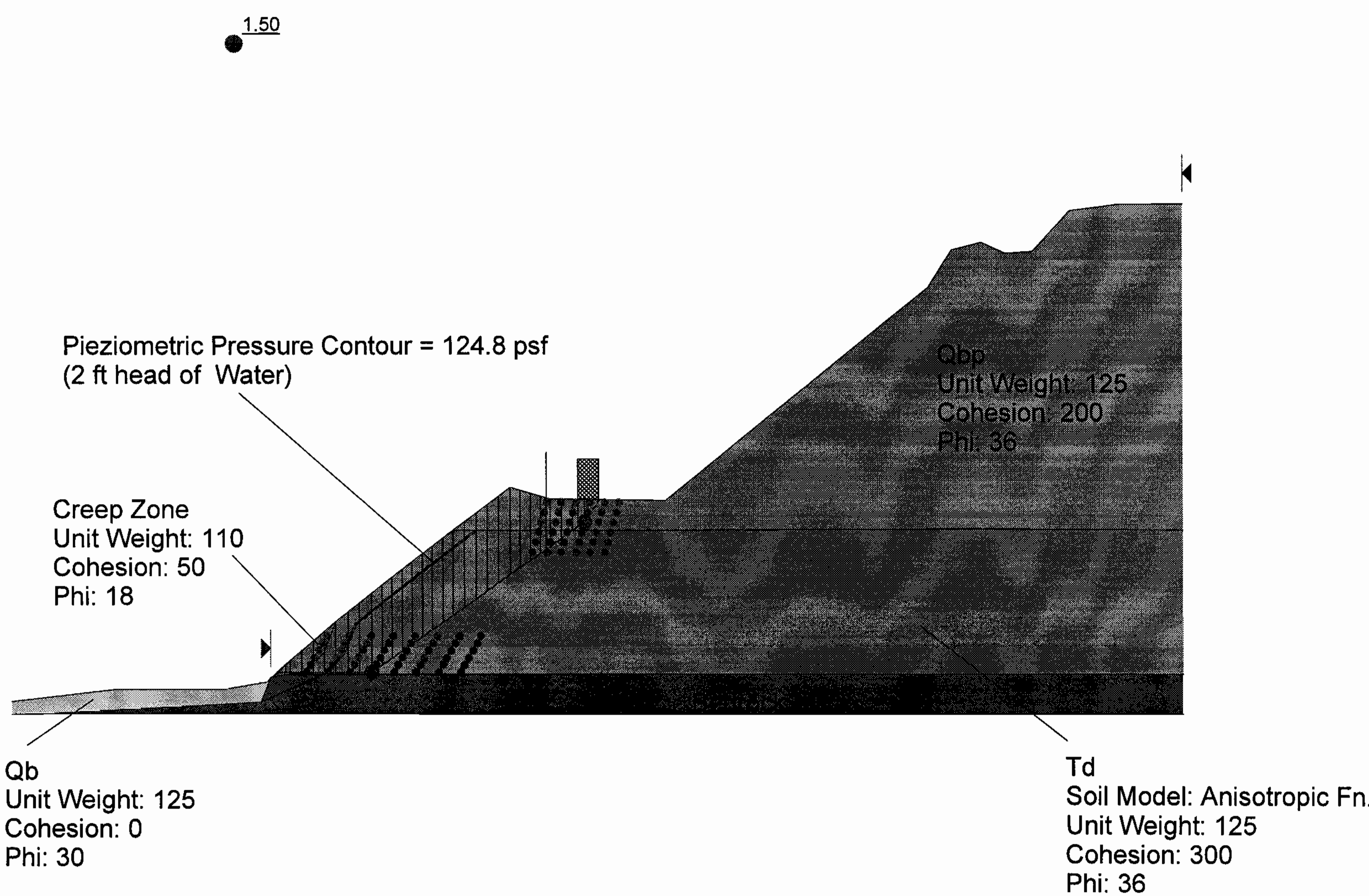


Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Static 2 No Water.slz
Analysis Method: Spencer
Factor of Safety: 1.51

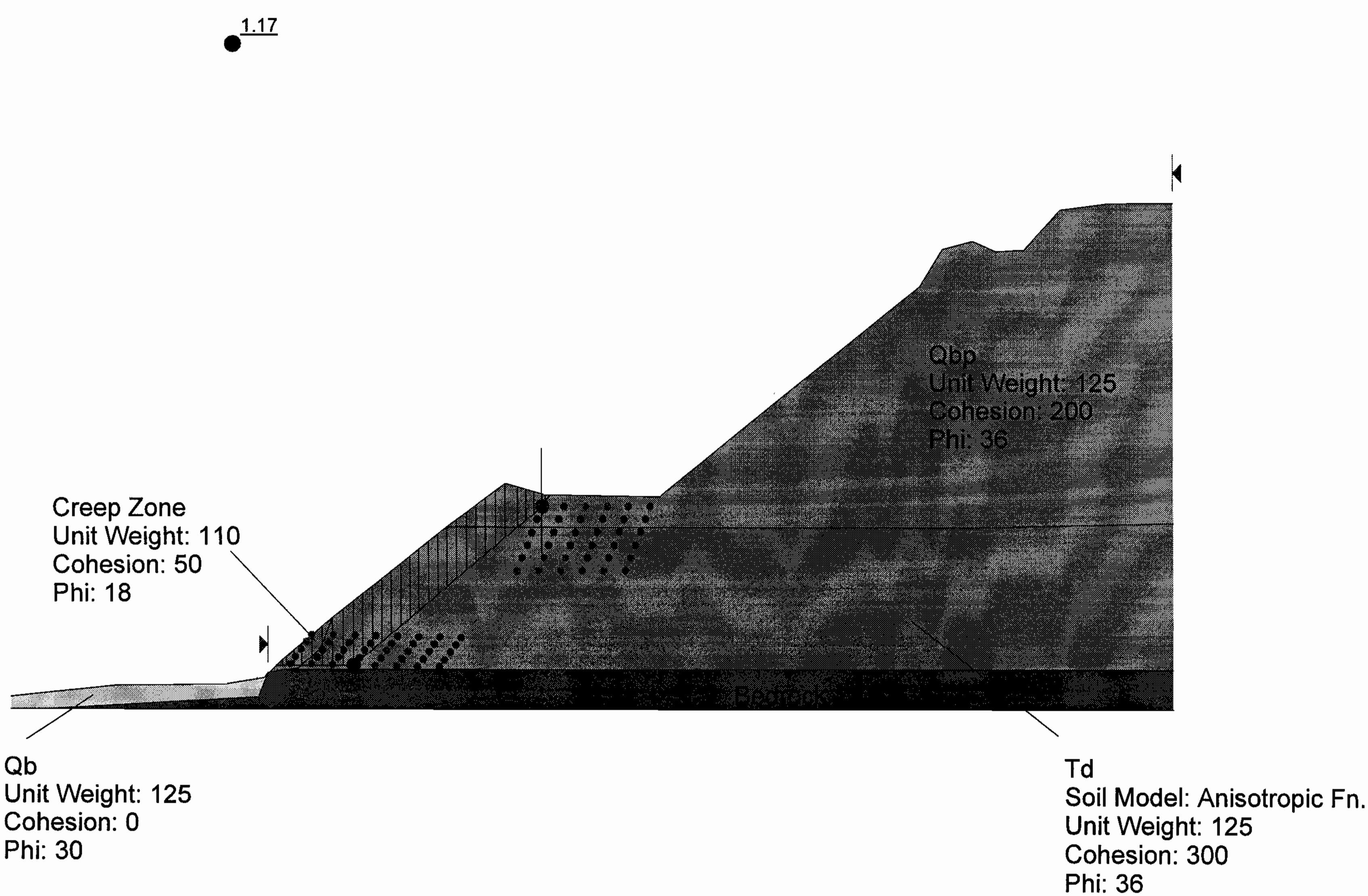


Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Static 4.slz
Analysis Method: Spencer
Factor of Safety: 1.5

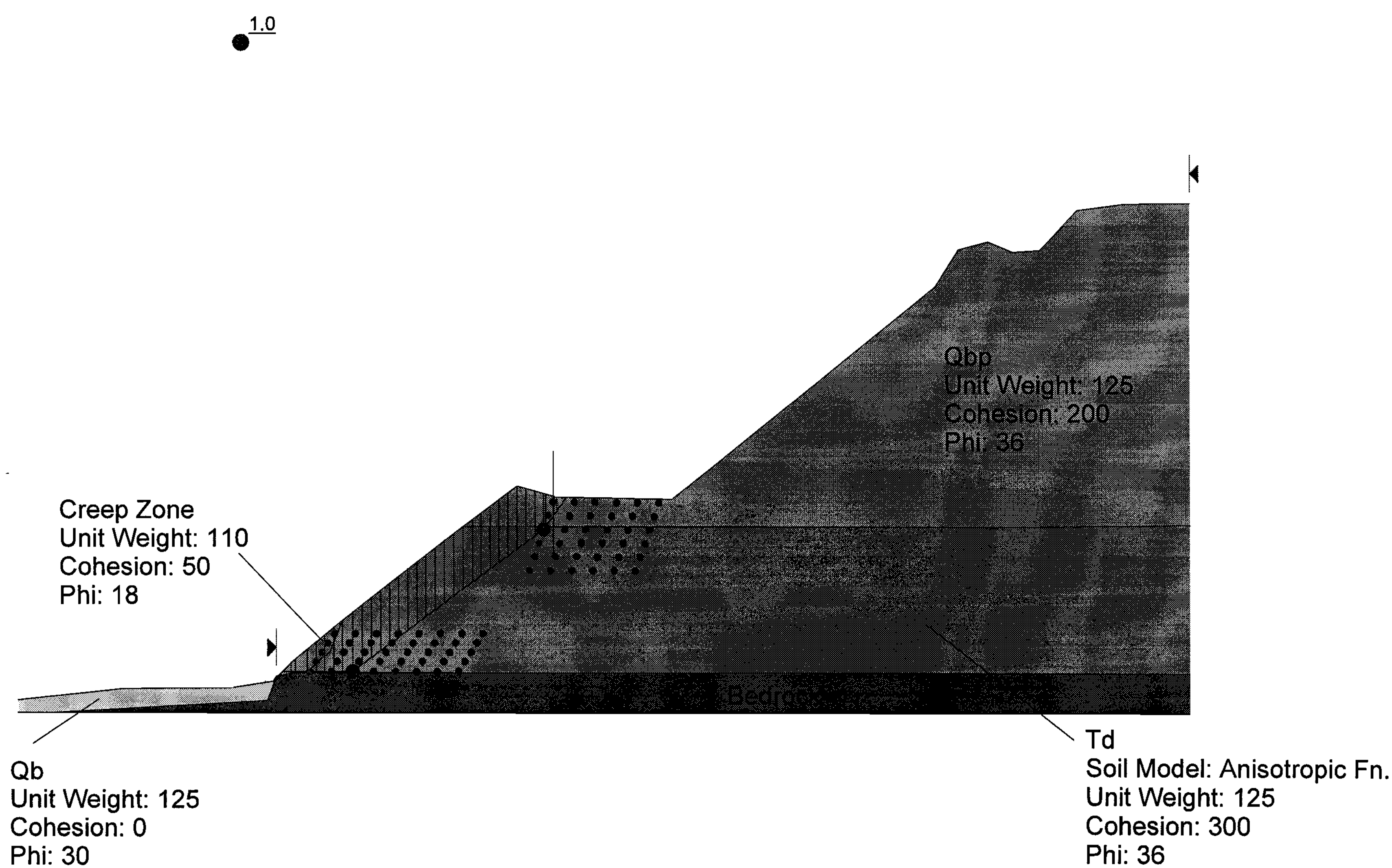
Surcharge = 3,000 psf



Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Psuedo Static 1.slz
Analysis Method: Spencer
Factor of Safety: 1.17
Seismic Coefficient = 0.15

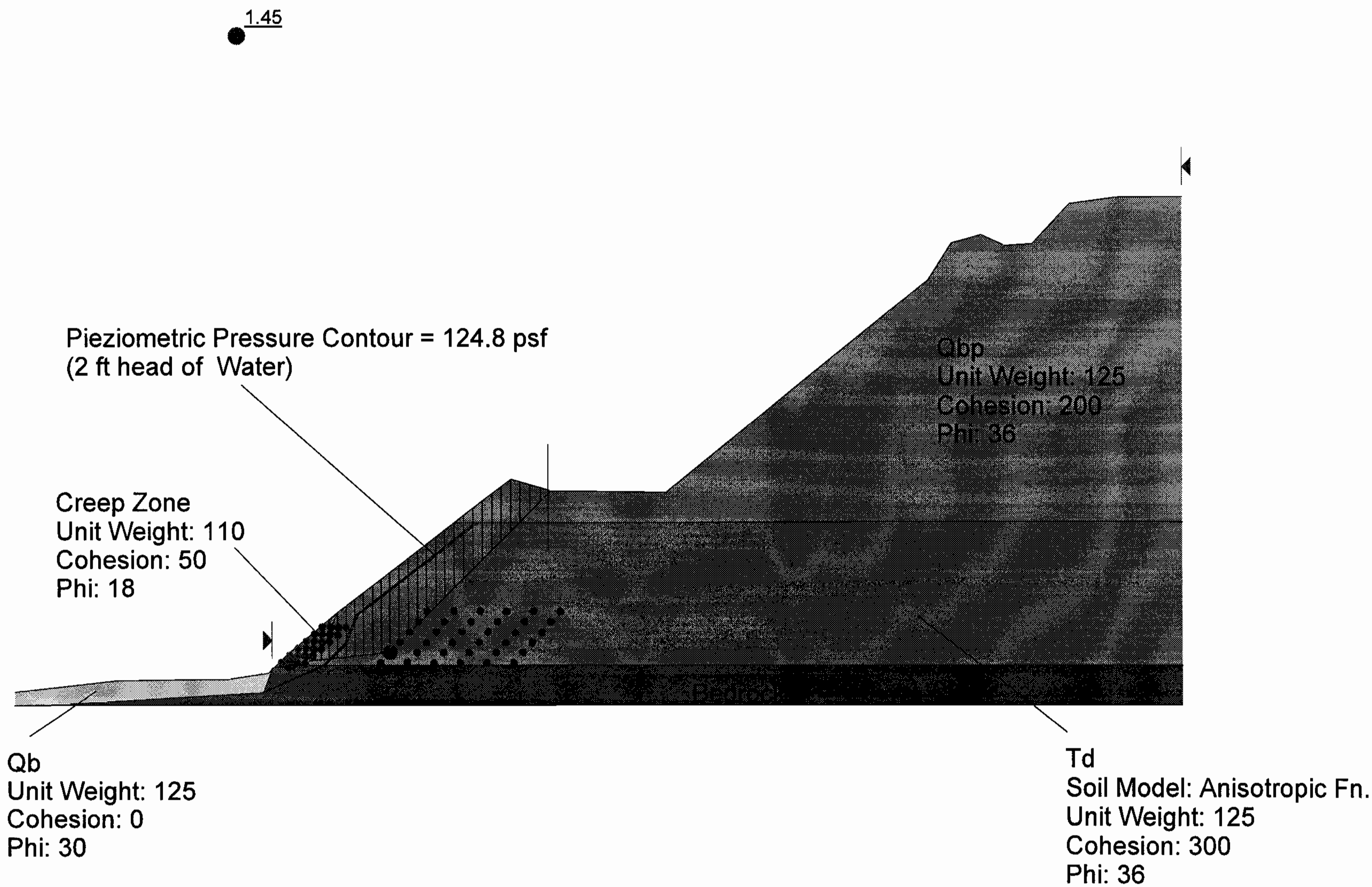


Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Psuedo Static 2.slz
Analysis Method: Spencer
Factor of Safety: 0.97
Siesmic Coefficient = 0.28

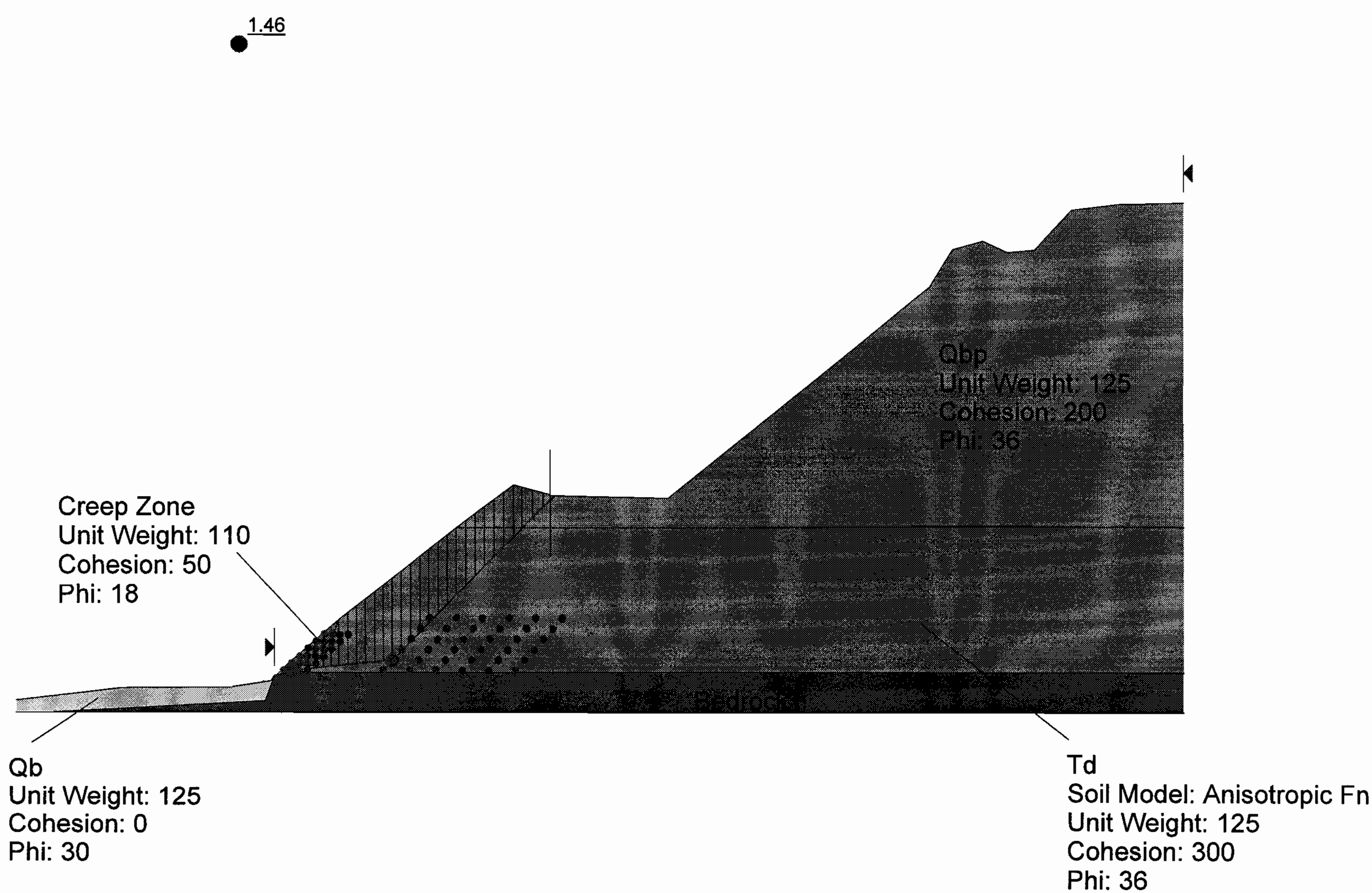


Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Static 2B.slz
Analysis Method: Spencer
Factor of Safety: 1.45

Surcharge = 3,000 psf

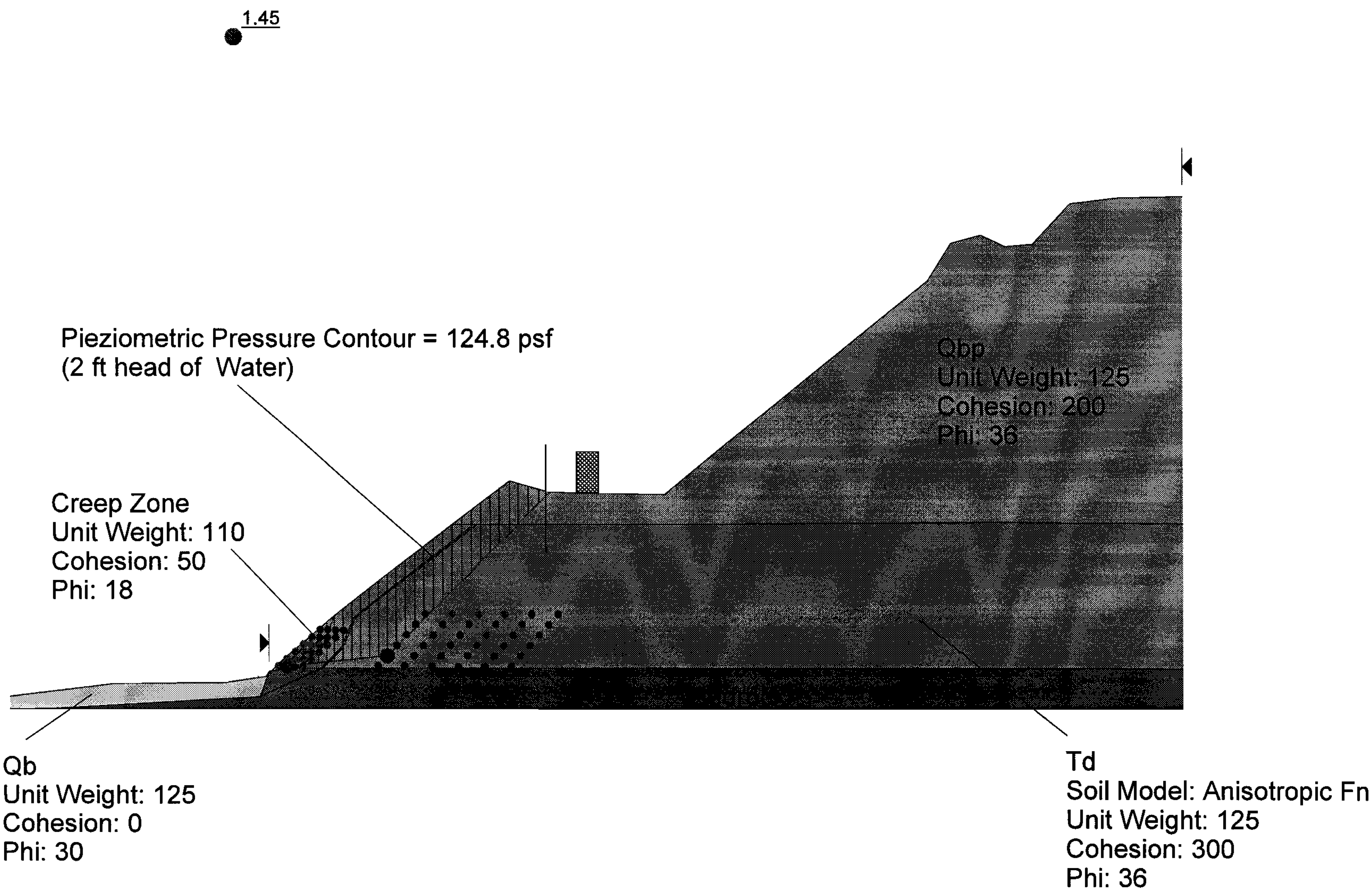


Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Static 2B No Water.slz
Analysis Method: Spencer
Factor of Safety: 1.46



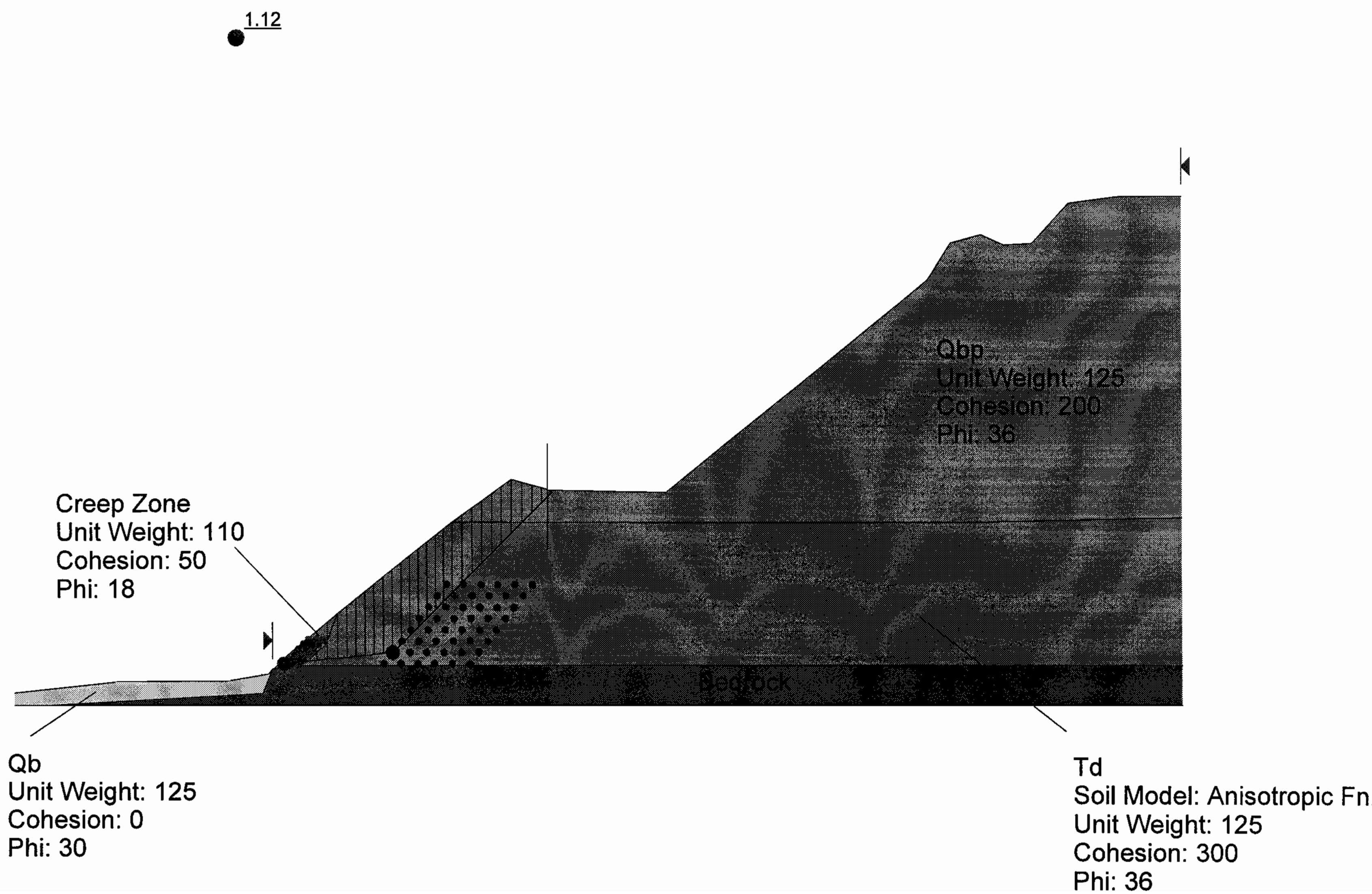
Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Static 4B.slz
Analysis Method: Spencer
Factor of Safety: 1.45

Surcharge = 3,000 psf



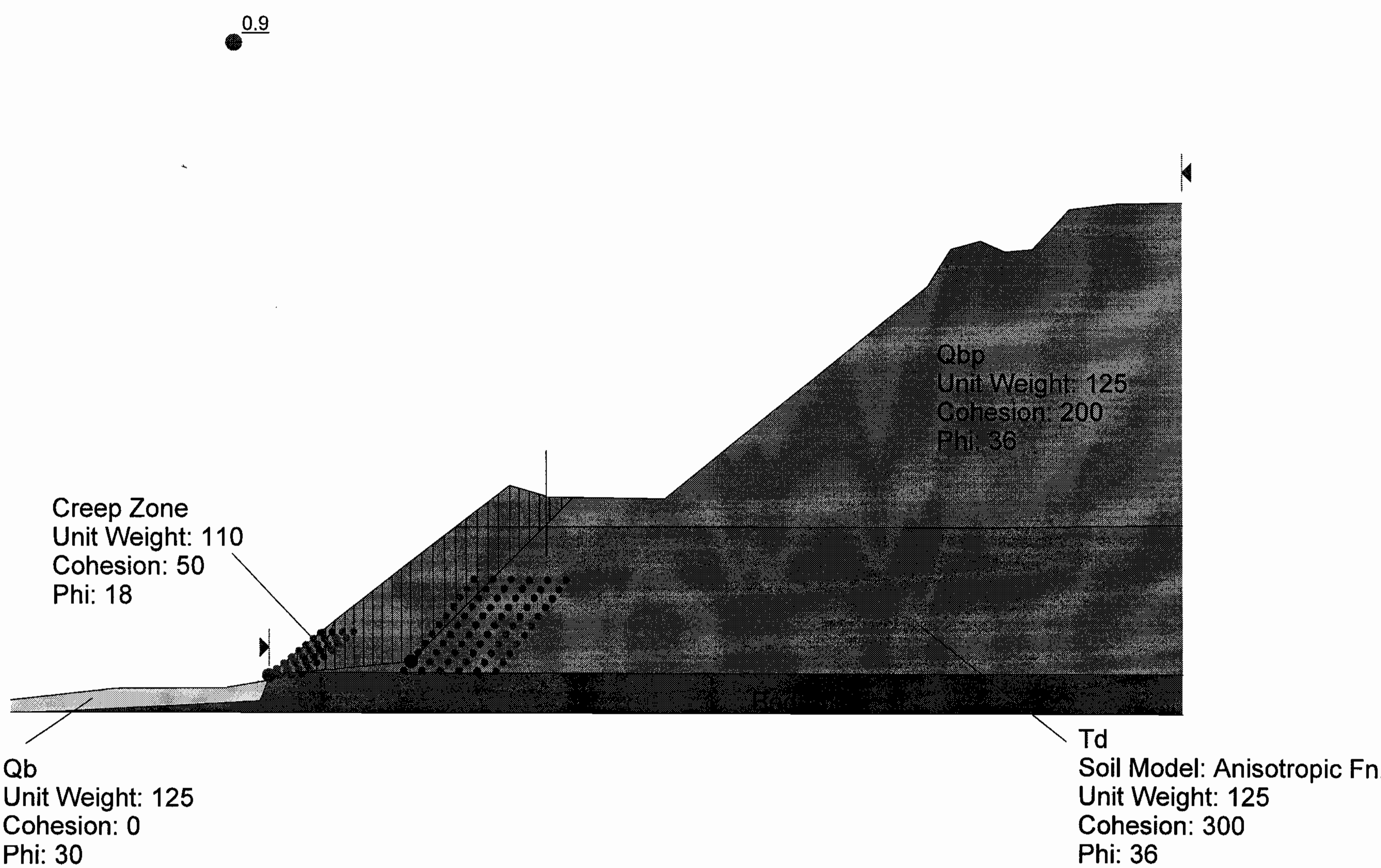
Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Psuedo Static 1B.slz
Analysis Method: Spencer
Factor of Safety: 1.12

Seismic Coefficient = 0.15

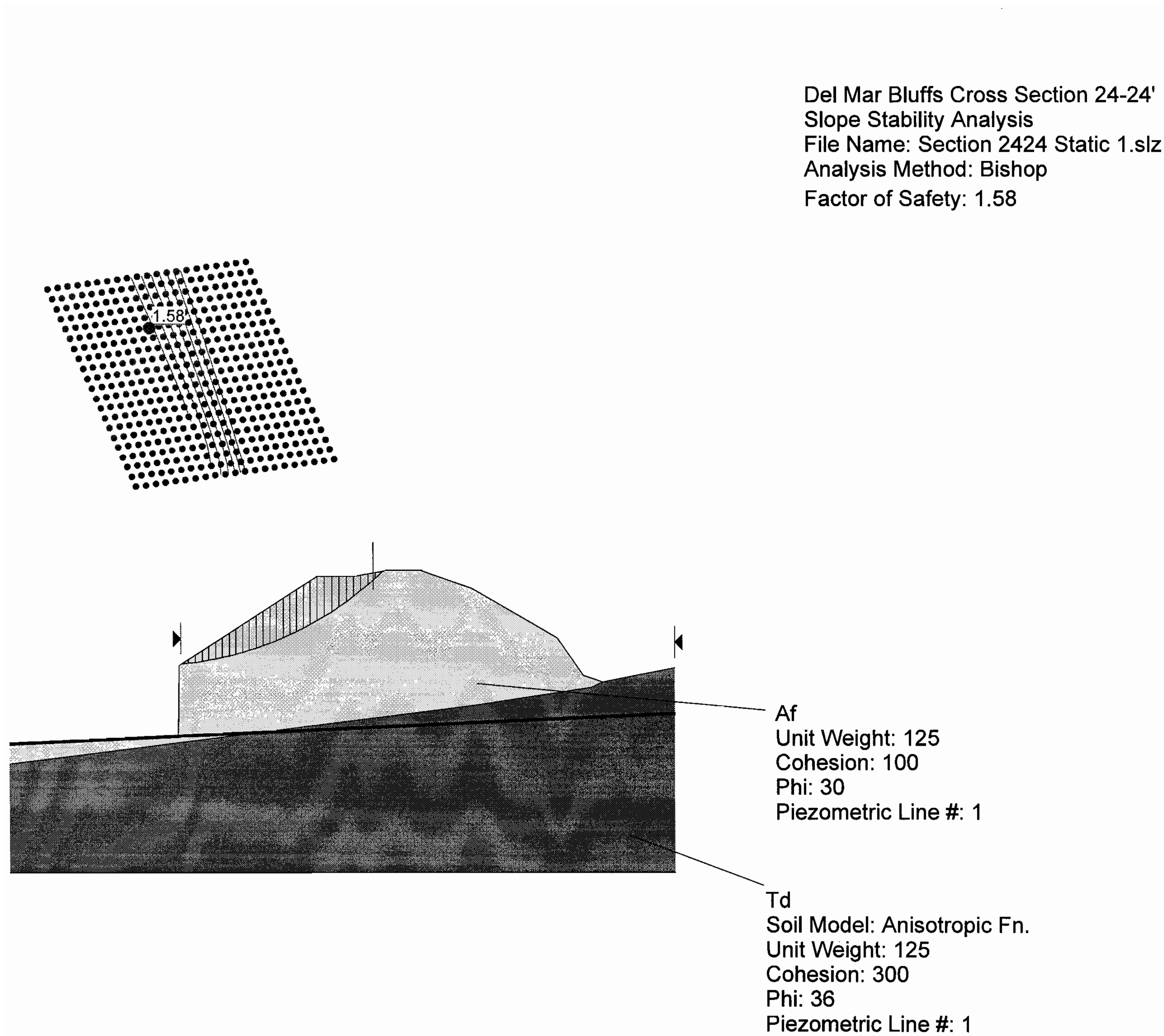


Del Mar Bluffs Cross Section 22-22'
Slope Stability Analysis
File Name: Section 2222 Psuedo Static 2B.slz
Analysis Method: Spencer
Factor of Safety: 0.94

Siesmic Coefficient = 0.28

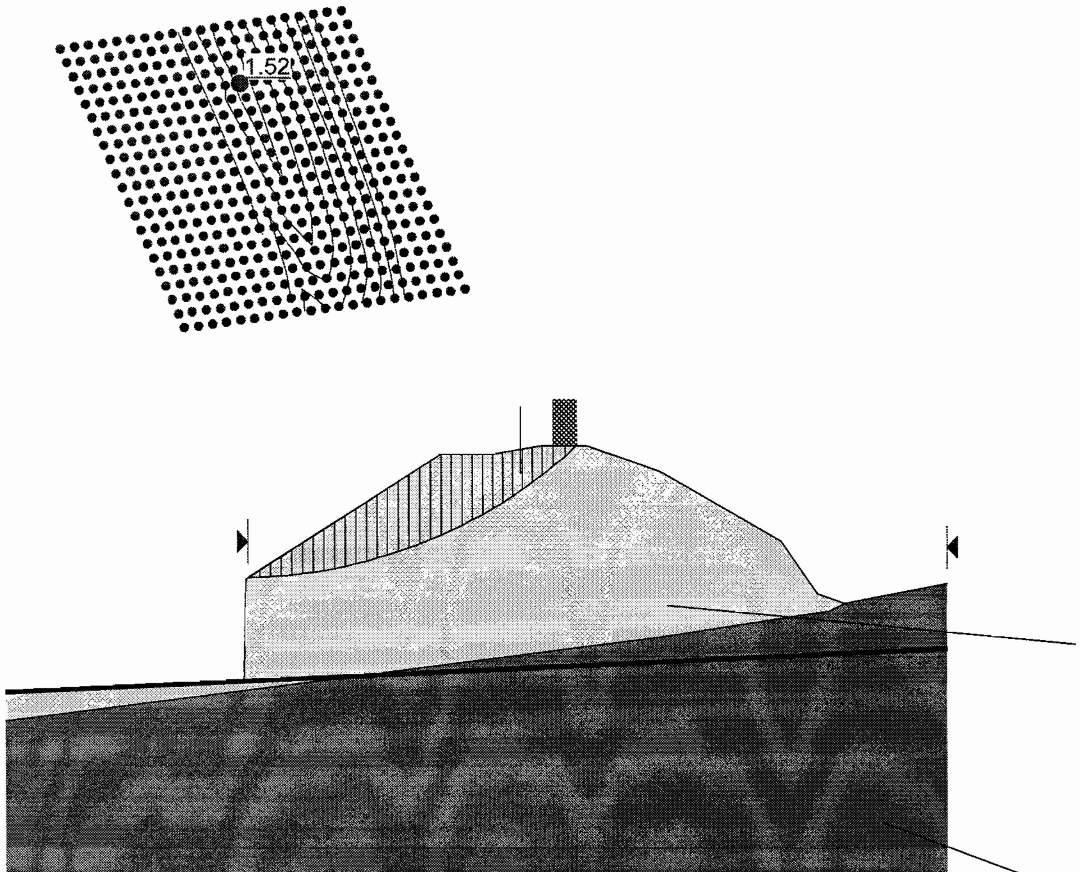


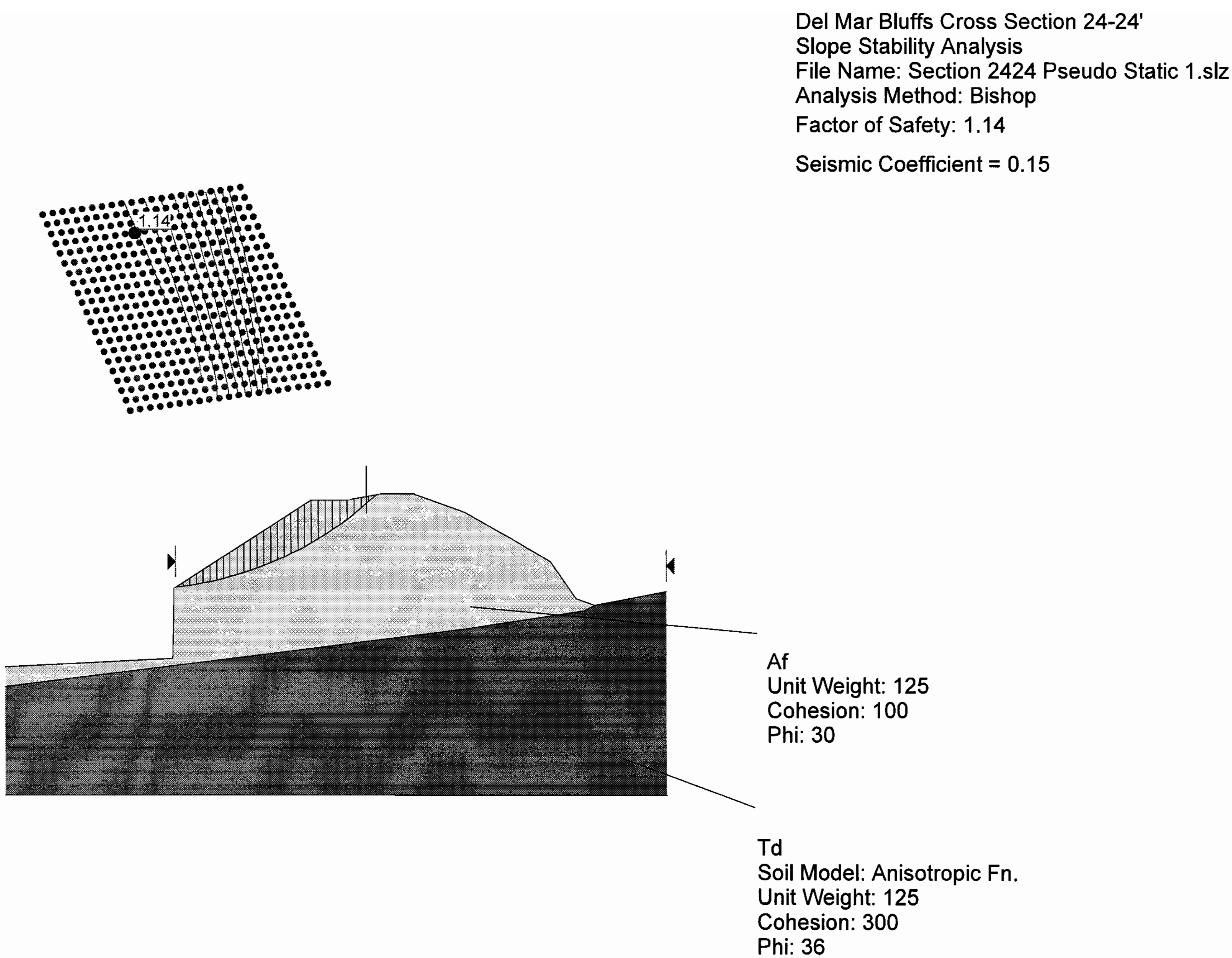
Cross Section 24-24'

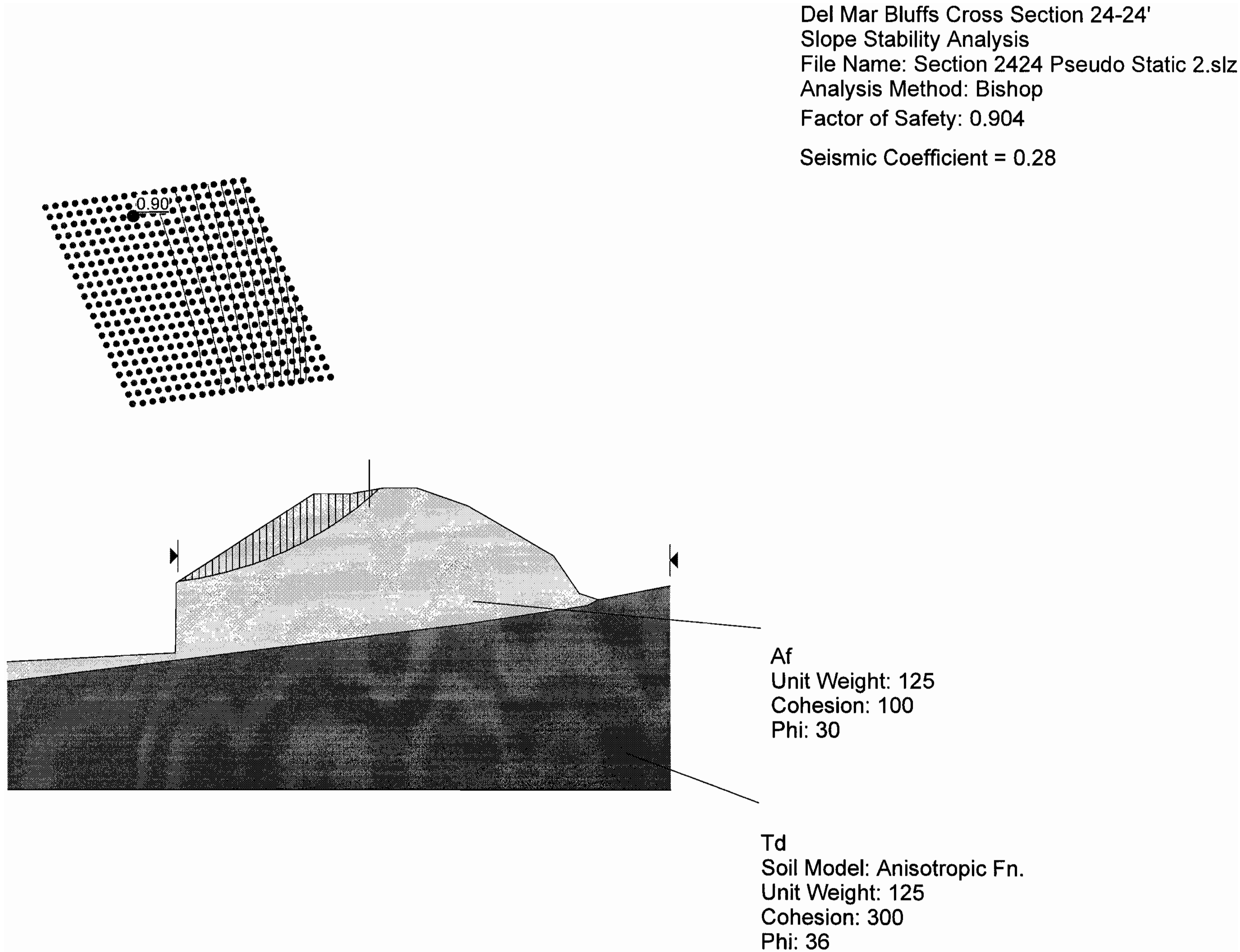

Del Mar Bluffs Cross Section 24-24'
Slope Stability Analysis
File Name: Section 2424 Static 1.slz
Analysis Method: Bishop
Factor of Safety: 1.58
1.58
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Piezometric Line #: 1
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36
Piezometric Line #: 1

Del Mar Bluffs Cross Section 24-24'
Slope Stability Analysis
File Name: Section 2424 Static 2.slz
Analysis Method: Bishop
Factor of Safety: 1.52

Surcharge = 3,000 psf




Del Mar Bluffs Cross Section 24-24'
Slope Stability Analysis
File Name: Section 2424 Pseudo Static 1.slz
Analysis Method: Bishop
Factor of Safety: 1.14
Seismic Coefficient = 0.15
1.14
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36


Del Mar Bluffs Cross Section 24-24'
Slope Stability Analysis
File Name: Section 2424 Pseudo Static 2.slz
Analysis Method: Bishop
Factor of Safety: 0.904
Seismic Coefficient = 0.28
0.90
Af
Unit Weight: 125
Cohesion: 100
Phi: 30
Td
Soil Model: Anisotropic Fn.
Unit Weight: 125
Cohesion: 300
Phi: 36