GEOTECHNICAL INVESTIGATION
For
Second Avenue between Lopez and North Casanova Street
Carmel-by-the Sea, California

Prepared For
Mr. Richard Guillen
City of Carmel-by-the-Sea
California

Prepared By
HARO, KASUNICH AND ASSOCIATES, INC.
Geotechnical & Coastal Engineers
Project No. M9928
February 2010

HARO, KASUNICH AND ASSOCIATES, INC.

CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

Project No. M9928
8 February 2010

CITY OF CARMEL-BY-THE-SEA
Rich Guillen
City Administrator
P.O. Box CC
Carmel, California 93921

Subject: Geotechnical Investigation Report

Reference: Roadway Distress
Second Avenue between Lopez and North Casanova Streets
Carmel-by-the-Sea, California

Dear Mr. Guillen:

In accordance with your authorization, we have performed a Geotechnical Investigation for the referenced distressed road in Carmel-by-the-Sea, California.

In summary, based on our analysis the currently-closed, distressed road, referenced above, has the potential for slope failure creating additional road way distress which includes potentially jeopardizing support of the existing underground water and sewer utilities. We recommend retaining wall support of the road. Refer to the contents of this report for details.

A very broad estimate for a one-lane road supported by a 200 foot long, 10 -12 foot high soldier beam wall would be on the order of $100 to $200 per face foot. Including roadway re-building and paving the cost could be on the order of $300,000 to $500,000.

Refer to the accompanying report copies which present our discussion, conclusions and recommendations, as well as the results of the geotechnical investigation on which they are based.

If you have any questions concerning the data or conclusions or recommendations presented in this report, please call our office.

Very truly yours,

HARO, KASUNICH & ASSOCIATES, INC.

REGISTERED PROFESSIONAL ENGINEER
VICTORIA CARLIN ODELLO
Exp. 12/2010
CIVIL
STATE OF CALIFORNIA

Vicki Odello
C.E. 5265

VO/sq

Copies: 8 to Addressee
2 to Neill Engineers Corp, Attn: Mr. Sherman Low

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GEOTECHNICAL INVESTIGATION

Introduction

Pursuant to our contract dated 9 October 2009 and Resolution Number 2009-76, this report summarizes the findings and presents the conclusions and recommendations from our geotechnical investigation for the distressed road at Second Avenue between Lopez and North Casanova Streets in Carmel-by-the-Sea, California. Refer to Site Vicinity Map (Figure No. 1) attached to this report.

During our investigation we referenced a topographic map and cross-sections of the study area prepared by Neill Engineers dated November 2009. Refer to Plate 1, 2 and 3. Plate 1 shows the locations of our exploratory borings and cross-section locations.

History

The referenced road section has been closed off for more than ten years as a result of tension cracking and subsidence of the paved roadbed. Based on conversations with Carmel Public Works personnel the distress at the road occurred in a creep like fashion (rather than a catastrophic episode) following the particularly heavy storms of 1998. Ballards were placed at each end of the road blocking off the most distressed section of the road. We understand the road was a dirt road for a long time before it was paved about twenty years ago. About 50 years ago the outboard slope below the road was used as a dump site for construction materials and spoils creating the wide soft shoulder toward the east end of the closed roadway. We understand some of the fill debris could consist of bathtubs, toilets, brick and concrete and typical residential demolition materials. The west end of the road also contains some fill material along its outboard edge but is thinner. Based on conversations with Cal-Am and the Sewer district their respective underground utilities were replaced about 20 years ago. They each indicated their lines are currently functioning well. The new plastic sewer line and sand backfill replaced an old leaky clay vitrified pipe. The old water line was most likely abandoned in place and a new PVC line installed adjacent to the old alignment, as is standard procedure. A nearby leak detector shows no leaks. The water line has several laterals, fire hydrant and a capped lateral. We understand the capped lateral was used as a temporary emergency measure to supply Pebble Beach with water during the 1998 storm by suspending a pipe from Second Avenue over Pescadero Creek to Pebble Beach.

We understand Pescadero Creek can run dry in late summer but can also rage in storm events. The creek blew out Carmel Way, the access road into Pebble Beach, just inside the Carmel Gate during the El Nino storms of 1998. Slopes on the other side of the creek in Pebble Beach have failed as well requiring retaining walls to re-support the roadway.

Purpose and Scope

The purpose of our investigation was to explore and evaluate subsurface conditions at the site, to determine the relative stability of the slope below the road and to provide recommendations as to the viability of re-opening the road to traffic. The specific scope of our services was as follows:

1. Review data provided to us and in our files pertinent to the site.
2. Interview Public Works personnel, past and present, and utility company personnel to get a history of the area.
3. Explore the subsurface conditions at the site with seven exploratory borings drilled to depths of up to 21.5 feet.
4. Develop three cross-sections of the subsurface earth materials within the distressed area.
5. Field and laboratory testing of selected soil samples to determine their strength and pertinent engineering properties.
6. Slope stability analysis for the static and seismic condition on each modeled cross-section.
7. Analyze the resulting data to develop geotechnical conclusions as to the roadbed's stability and present recommendations as to the viability of re-opening the road.
8. Work closely with Mr. Sherman Low at Neill Engineers in preparation of our investigation and analysis.
9. Prepare and present preliminary results at a meeting with Mr. Rich Guillen at City Hall
10. Present the results of our investigation in this report.

Site Location and Conditions

The project site consists of the section of road along Second Avenue between Lopez and North Casanova Streets in Carmel-by-the-Sea. Refer to Plate 1. The distressed road section is about 200 feet in length. The roadbed, at is narrowest, is only one lane wide, about 10 feet wide. Toward the east end it widens to 2 lanes with about a 20-foot wide soft shoulder on the outboard side. The road consists of a cut and fill graded bed. Coincidentally the underground utility line roughly marks the contact between the fill and native material.

The inboard cut exposes steep native sands. It varies in height but on average is about 10 feet high. Above the cut slope are developed residential lots. The outboard slope descends steeply about 40 feet to a narrow canyon at the bottom of which is Pescadero Creek. The topography of this slope shows significant evidence of many small landslides and erosion. Fill debris including bricks and concrete were encountered many places just under the surface of the vegetated slope. At the west end of the road there is a significant erosional gully extending the full height of the slope. It emanates from a drainage culvert at the street level.

The creek banks at the base of the slope show significant scour and typical creek erosion. There is a lot of debris (brick, concrete and steel) in the creek bottom as well.

There are several underground utilities in the roadbed including sewer and water. There may be a parallel abandoned water line as well. There is a short culvert along the inboard edge.

Besides the landslide and erosional distress of the descending outboard slope the roadbed is distressed as well. Pavement distress includes tension cracks situated within the outer half of the roadbed and parallel to the outboard edge. Virtually all tension cracks are confined to the area outboard of the underground utility lines. The tension cracks occur in generally two areas nearest Cross-Sections 2, 4 and 5. There are a series of sub-parallel tension cracks along the outer half of the road. The tension cracks continue beyond the pavement into the fill shoulder. There is vertical displacement as seen in the pavement against the manhole covers of about 2 inches. There appears to be some horizontal as well as vertical displacement of less than an inch between each of the sub parallel tension cracks increasing in displacement the closer to the outer edge of the pavement. The water line location is evidenced by cracks running along its backfilled trench suggesting some settlement of the backfill. The asphalt curb along the outboard edge of the road is cracked and distressed in some areas.

Vegetation at the site consists of thick grass, shrubs and occasional trees.

Drainage at the site consists of some controlled street runoff to a culvert that discharges water on the slope just below the street shoulder. Discharged water descends via a significant incised erosional gully on the slope to Pescadero Creek below. Street runoff also cascades over the soft shoulder onto the slope. At the time of the investigation the creek was running with less than a foot of water and at times during storm events was ragging.

Project Description

The project consists of a geotechnical investigation of the site and analysis of slope stability of the hillside below the referenced section of road for use in determining if slope stabilization is necessary for the road to be re-opened to traffic.

Field Exploration

Subsurface conditions were explored on 1 December 2009 by drilling a total of seven exploratory borings to depths of up to 21.5 feet. The borings were advanced with 6-inch diameter continuous flight auger equipment mounted on a truck.

Representative soil samples were obtained from the exploratory borings at selected depths, or at major strata changes. These samples were recovered using a 3.0 inch O.D. Modified California Sampler (L), or by a Standard Terzaghi Sampler (T). The soils encountered in the borings were continuously logged in the field and described in accordance with the Unified Soil Classification System (ASTM D2488, Visual-Manual

Proceeding). The Logs of Test Borings are included in the Appendix of this report. The logs depict subsurface conditions at the approximate locations shown on the Sheet 1. Subsurface conditions at other locations may differ from those encountered at the explored locations. Stratification lines shown on the logs represent the approximate boundaries between soil types. The actual transitions may be gradual.

The penetration blow counts noted on the boring logs were obtained by driving a down-hole sampler into the soil with a 140-pound hammer dropping through a 30-inch fall. The sampler was driven up to 18 inches into the soil and the number of blows counted for each 6-inch penetration interval. The numbers indicated on the logs are the total number of blows that were recorded for the second and third 6-inch intervals, or the blows that were required to drive the penetration depth shown if high resistance was encountered.

Laboratory Testing

Soil samples obtained from the borings at selected depths were taken to our laboratory for further examination and laboratory testing. The laboratory testing program was directed toward determining pertinent engineering properties of soil underlying the project site.

Natural moisture contents and dry densities were determined on selected samples and are recorded on the boring logs at the appropriate depths. Since water has a significant influence on soil, the natural moisture content provides a rough indicator of the soil's compressibility, strength, and potential expansion characteristics.

The strength parameters of the underlying earth materials were determined from field penetration resistance of the in-situ soil and laboratory direct shear tests.

Atterberg limits and grain size tests were performed to characterize the expansive potential of selected samples.

The in-situ and laboratory test results indicate that upper fill and topsoil has low strength. The underlying native, slightly cemented sandstone layers and bedrock had moderate to strong strength values. Atterberg limits and grain size tests indicated the soil has a low potential for expansion.

The results of the laboratory testing appear on the "Logs of Test Boring" opposite the sample tested.

Subsurface Conditions

Based on our subsurface exploration the general soil profile consists of loose sandy fill and topsoil over slightly cemented native sandstone over a layer of more cohesive conglomeritic sandstone over weathered basaltic bedrock.

The layers are roughly parallel to each other and dip steeply down slope, which is not a favorable condition for stability.

The asphalt pavement and baserock combined thickness varied but was up to 11 inches. The baserock appeared to be decomposed granite (dg).

The exploratory borings drilled on the outboard edge of the road encountered fill material over topsoil ranging in combined thickness of 6 to 19 feet thick. It was difficult to distinguish between the fill and topsoil since the fill is virtually the same topsoil material cut from the inboard side of the road and dumped in place on the outboard side. The fill/topsoil material consists of loose to very loose dark brown sand with occasional fill debris and import soil. The fill/topsoil thickness was thinnest where the roadway shoulder is narrow. The fill layer thickens dramatically where the soft shoulder is wide. The wide shoulder generally consists entirely of the very loose fill. We understand this area was a dumpsite and that old bathtubs and residential demo debris may be included in the fill. We encountered brick and concrete at several locations within the top 2 feet of the surface soils of the hillside.

Below the fill/topsoil the soil graded into a native medium dense to dense, reddish, slightly cemented sandstone. This layer was about 3 to 7 feet thick at the outboard edge of the road however it pinched out and was nonexistent on the out board edge where the shoulder is wide. This material was encountered at grade (below the pavement section) on the inboard side of the road down to a depth of up to 15 feet.

Below that, the material graded to a dense, more cohesive conglomeritic sandstone with gravels and cobbles. This layer varied from 2 to 10 feet thick.

The basement material consisted of very dense weathered volcanic rock (basalt).

Groundwater

No groundwater was encountered in our borings. However, it should be noted that groundwater levels might fluctuate due to variations in rainfall or other factors not evident during our investigation. If groundwater is encountered in the course of construction, additional recommendations may be necessary. The subsurface soils were moist however will vary in moisture content. We did not encounter subsurface seepage but there is potential for natural seepage and perhaps seepage from within the trench back fill or from future leaks of the underground utilities.

Quantitative Slope Stability Analysis

Discussion and General Methodology

Slope failures or landslides can cause problems including undermining and distress of roadbeds. Slope failures occur when stress acting on the soil mass is greater than its internal strength (shear strength). A slope is considered stable when the strength of its soil mass is greater than the stress field acting within it. Some common variables influencing stress are gravity, steep slopes, pore water pressure, bearing pressures (proposed structures), and seismic shaking.

Various methods of analyzing stability of slopes yield a factor of safety. A factor of safety is determined by dividing the resisting forces within the slope soils by the driving forces within the slope (stress field). When a factor of safety less than one (F.S. < 1) is determined, a slope failure is likely. When a factor of safety equal to one (F.S. $= 1$) is determined, the slope is in a state of equilibrium. When a factor of safety greater than one (F.S. >1) is determined, the slope is considered stable. Common practice suggests for a slope to be considered stable the factor of safety should be equal to or greater than 1.2 (F.S. ≥ 1.2) for the seismic condition and a static safety factor should be equal to or greater than 1.5 (F.S. ≥ 1.5).

A quantitative slope stability analysis was performed on three cross sections at various locations on Second Avenue. The cross sections included the Second Avenue roadbed and extended down the slope to Pescadero Creek.

To develop the three slope models to be analyzed we used the topographic cross sections that were developed by Neill Engineers, Corp. Subsurface earth material geometry was developed using laboratory, field and subsurface data derived from our field investigation. The depth and thickness of the subsurface strata delineated on the cross sections were generalized and interpolated from test bore locations. The transition between materials may be more or less gradual than indicated.

Model Confirmation

Initially we analyzed each modeled cross section statically and compared the location of the resulting slip surfaces and associated factor of safety to the actual location of physical tension cracks in the field. The tension cracks are the surface manifestation of failure surfaces with a F.S. ≤ 1. This comparison was performed in an effort to confirm a relatively accurate model. As a result we had to decrease the laboratory strength value of Soil Type 1 of the model to correlative strength values from our insitu blow count readings obtained during exploratory drilling. More precise models were constructed. We believe our models represent a reasonable soil conditions at the site.

The analysis was carried out for both static and pseudo-static (seismic) conditions. Cross Sections 2, 4, and 5 shown on Sheet 2 and 3, were evaluated quantitatively, using the computer program STEDwin2.71. Circular failure surfaces as well as Block and Planar surfaces were analyzed.

The STEDwin2.71 program uses the Janbu, Modified Janbu, and Modified Bishop Method of Slices to determine normal and resistive forces in each slice. The forces in each slice are then summed up for a total force acting on the mass. The computer program STABL6H assumes many failure surfaces using initiation and termination points on the ground surface selected by the user. These chosen points represent the toe and scarp of each potential landslide in relation to the assumed failure surfaces. The Planar failure surfaces were dictated by the user. We also manipulated some Circular failure surfaces to analyze deeper slip surfaces.

Seismic Coefficient

Horizontal forces generated by a design seismic event are typically modeled by applying a seismic coefficient value to the analysis, in order to develop a pseudo-static condition intended to represent earthquake effects on the slope model. A site-specific seismic coefficient was developed for this project using the procedures outlined in *Special Publication 117 Guidelines for Evaluating and Mitigating Seismic Hazards in California* (California Geological Survey, 2008).

For a pseudo-static (seismic) analysis, a seismic coefficient (k_h), in the form of a percentage of the force of gravity, is applied to the slope stability calculation.

To determine the coefficient we first determine the peak ground acceleration as prescribed in the California Building Code (2007 Edition). Using either Section 1613 of the CBC or the USGS web-based Seismic Coefficient Calculator, and inputting the longitude and latitude and Seismic Site Class, the short-duration design spectral response acceleration factor (SDS) is determined. Peak ground acceleration is this value divided by 2.5 (CBC Section 1802.2.7). This method yielded a peak ground acceleration at the site of 0.42. The peak acceleration is then reduced by 2/3 resulting in a (k_h) value of 0.28.

Moisture Condition

There is water running in the creek at the base of the slope. However, we did not encounter groundwater in our borings located on the road bed situated well upslope from the creek bottom. However there is moisture in the soil and a potential for increased moisture in inclement weather. To account for seasonal moisture we increased pore pressure within the slope by 20% (R_U = 0.1). Note that moisture content could be higher than that analyzed.

Soil Properties

The slope profiles were modeled with four soil types. On the basis of pre-saturated moisture conditioned direct shear laboratory and insitu testing of soil samples, strength values were assigned to the four soil types. However Soil Type 1 was reduced from the laboratory strength value to correlate field strength values to field conditions (existing tension cracks-refer to Model Confirmation Section above). The assigned strength values are as follows:

Soil Type	Cohesion (c)	Phi Angle (ø)
1. Combined Fill/Topsoil	89 psf	30
2. Native Slightly Cemented Sandstone	318 psf	37
3. Native Cohesive Conglomeritic Sandstone	1418 psf	28
4. Weathered Volcanic Bedrock	500 psf	44

Slope Stability Analysis Results

Three different shapes of potential failure surfaces were analyzed: Circular, Block and Planar. The Circular and Block modes worked best for Section 2 and only worked well for the outboard edge of the road for Sections 4 and 5. The Planar mode worked best in analyzing the inboard part of the road especially for Section 4 and 5. Refer to Table 1 for a summary of results.

Statically, the outboard half of the road, which is coincident with the distress observed, yielded Factors of Safety ranging between failing to barely stable (FS= 0.83 to 1.41) suggesting potential instability of this section of the road. Refer to Figures B-1, B-4, B-8, and B-9. However for the inboard side of the road the static Factors of Safety ranged from barely stable to stable (FS= 1.16 to1.69). Refer to Figures B-2, B-5 and B-11in Appendix B.

Seismically, the outboard edge of the road is deemed unstable (FS=0.57 to 0.85) Refer to B-2, B-6 and B-10. The inboard side of the road has Factors of Safety indicating instability as well (FS=0.74 to 0.99). Refer Figures B-B-2, B-3, B-6, B-7, B-10 and B-12 in Appendix B.

The slope stability analysis suggests the uppermost soil layer is weak and that potential instability appears to be confined to this uppermost layer. We ran a forced planar slip surface through the underlying soil layers to confirm this and our forced analysis yielded a seismic FS = 2.02 (Figure B-13) indicating the underlying material is stable.

Table 1: Slope Stability Results 20% Increased Pore Pressure

Cross-Section 2

Fig. B-1	Static Condition Circular Failure Surface		
		Outboard side of road:	min. FS=1.41
		Inboard side of road:	min. FS=1.69
Fig. B-2	Seismic Condition Circular Failure Surface		
		Outboard side of road:	min. FS=0.85
		Inboard side of road:	min. FS=0.99
Fig B-3	Seismic Condition Planar Failure Surface		min. FS=0.92

Cross-Section 4

Fig. B-4	Static Condition Circular Failure Surface		
		Outboard side of road:	min. FS=1.04
Fig. B-5	Static Condition, Circular Failure Surface		
		Inboard side of road:	min. FS=1.16-1.42
Fig. B-6	Seismic Condition, Circular Failure Surface		
		Outboard side of road:	min. FS=0.70
		Inboard side of road:	min. FS=0.78
Fig. B-7	Seismic Condition, Planar Failure Surface:		min. FS=0.76

Cross-Section 5

Fig. B-8	Static Condition, Circular Failure Surface		
		Outermost edge of Road:	min. FS=0.83
Fig. B-9	Static Condition, Circular Failure Surface		
		Outboard side of road:	min. FS=0.97
Fig. B-10	Seismic Condition, Circular Failure Surface		
		Outboard side of road:	min. FS=0.57
Fig. B-11	Static Condition Planar, Failure Surface		
		Inboard side of Road:	min. FS=1.30
Fig. B-12	Seismic Condition, Planar Failure Surface:		min. FS=0.74
Fig. B-13	Seismic Condition, Planar Failure Surface		
		Failure Plane through Soil Layer #3	min. FS2.02

FS< 1 suggests instability, FS= 1 suggests barely stable, FS> 1 suggest stable

Industry suggests > 1.5 for static stablility and > 1.2 for seismic stability

Graphical results of our slope stability analysis are presented in Appendix B of this report.

The results of our analysis indicate that the subject road is potentially statically unstable and seismically unstable.

It must be cautioned that slope stability analysis is an inexact science; and that the mathematical models of the slopes and soils contain many simplifying assumptions, not the least of which is homogeneity. Density, moisture content and shear strength may vary within a soil type. There may be localized areas of low strength or perched ground water within a soil. Slope stability analyses and the generated factors of safety should be used as indicating trend lines. A slope with a safety factor less than one will not necessarily fail, but the probability of slope movement will be greater than a slope with a higher safety factor. Conversely, a slope with a safety factor greater than one may fail, but the probability of stability is higher than a slope with a lower safety factor.

Liquefaction

Due to lack of shallow ground water table within the upper sandy soil the potential for liquefaction is nil.

Settlement

Some of the road distress can be attributed to settlement of the trench backfill of the underground utilities as seen in the asphalt cracks along the sewer and water lines. Settlement of the loosely placed outboard fill wedge has also contributed to the road distress. Use of properly compacted fill, keyed or retained, would minimize this effect should the road ever be rebuilt.

Erosion

In addition to landslide failures the surface of the hillside below the road is also experiencing erosion from direct rainfall, road runoff, and creek scour.

CONCLUSIONS AND RECOMMENDATIONS

Based on our investigation the slope supporting the subject road is unstable. In summary, several geologic processes contribute to current and future instability of the roadway, including seepage, settlement, erosion and slope instability (landsliding). Landsliding is the overriding geotechnical issue for the road. Parts of the slope are currently failing in a static condition as creep. The slope supporting the road could fail further during the design seismic event, incurring additional distress to the roadbed and possibly jeopardizing support of the underground utilities.

There are several possibilities discussed below,

Options regarding the subject road are:

- Do nothing. Keep the road closed. Maintain as needed with asphalt patches and overlays.
- Relocate the road and/or utilities by cutting into the firm native inboard edge of the right of way where space allows. Abandon and allow the unstable outboard edge to continue to creep, fail downslope. An inboard retaining wall with a conventional foundation on the order of 10 feet high would be required. Protection of the underground utilities from instability of the outboard side would not be attained. A hybrid version of this option is the road bed is moved toward the inboard edge with a shorter wall on the inside and a retaining wall on the outboard side. Designers could analyze this option to develop the optimum

combination of inboard and outboard wall heights relative to road width and road position. Property lines and tree removal would have to be determined.

- Hilfiker walls and tieback shotcrete walls are not a good option due to the existing underground utilities.
- Pin Pile walls are not a viable option because there is not enough arching capability of the loose sandy soil comprising the upper soil layer. Where the roadbed is located within firm native ground (the inboard side) a pin pile wall could be an option.
- A soldier pile-lagging retaining wall positioned on the out board edge could be used. Only the upper fill/topsoil layer needs retainment, which for a one lane road width would amount to a 10 to 12 foot high wall. Tree removal and property lines would have to be determined.

If the road is to be re-opened to traffic, we recommend the soldier pile option retaining wall be used to support the outboard edge of a one-lane road with a 10 to 12 foot high soldier pile wall. This design will support the distressed road and utilities. The alignment of this single wall is versatile. Uphill property line issues can be avoided. A preliminary engineer's cost estimate for construction of this type of wall is $300,000 to $500,000.

It is not necessary to support the entire large outboard fill wedge if a one-lane road is proposed. The slope beyond the retained one lane road width would remain and be allowed to fail. Where the existing road is wider than one-lane; the soldier pile wall could be built below grade in a subexcavated bench.

Backfill behind the wall will consist of a conventional engineered backfill, drainage rock and pavement section. The existing underground utilities may need to be shored temporarily.

We have prepared general geotechnical criteria for such a wall.

<u>Soldier Pile Foundation System</u>

1. Piers must penetrate all loose fill/topsoil and embed into firm native bedrock the prescribed amount as per the structural engineer and accommodate geotechnical lateral criteria needs.
2. In order to gain lateral support for the proposed wall, the piers must be deepened to accommodate at least 15 feet of horizontal separation between the base of the piers and the daylight of the adjacent slope. The portion of pier above this point should be neglected in passive resistance calculations.
3. An upper portion of the piers as well as the wall should be designed to accommodate an active lateral earth pressure.
4. The geotechnical engineer should be present during pier drilling to verify anticipated subsurface conditions and verifying adequate pier depths. <u>Prior to</u>

placing steel and concrete, all pier excavations should be thoroughly cleaned and observed by the geotechnical engineer.

5. Prior to pouring concrete excavations should be thoroughly moisture conditioned so that the soil is allowed to absorb the water.

Retaining Wall Lateral Pressures

6. Retaining walls allowed to yield should be designed to resist active lateral earth pressures, seismic loads and any additional surcharge loads (e.g. traffic).
7. Walls should be fully drained to prevent hydrostatic pressure behind the walls.

California Building Code (2007) Seismic Design Parameters

8. The latest CBC (2007) edition design considerations, specifically the seismic factors and design spectral response acceleration parameters from Chapter 16, should be followed in the design of the proposed wall. Based on our investigation the Site Class is C - Very Dense Soil and Soft Rock.

Site Drainage and Erosion

9. Surface drainage must be collected and discharged in a way so as not to cause erosion. Discharge should be conveyed via tight line to the creek at the base of the slope. The storm drain pipe should be well secured to the surface of the slope.

10. Slopes should be protected from erosion by preventing runoff from spilling over slopes. Lined V-ditches and/or berms at the out board edge of the road may be considered. Runoff must not be allowed to cascade over the steep slope.

Utility Trenches

11. Trenches must be properly shored and braced during construction or laid back at an appropriate angle to prevent sloughing and caving at sidewalls. The project plans and specifications should direct the attention of the contractor to all CAL OSHA and local safety requirements and codes dealing with excavations and trenches.

12. New trenches should be backfilled with slurry so as not to allow seepage into the backfill.

13. Temporary cuts should be shored or braced. Exposed utilities may need to be shored or braced as well.

Flexible Pavements

14. Asphaltic concrete, aggregate base and subbase sections should be designed by a civil engineer based on R-value tests and should conform to and be placed in accordance with the Caltrans Standard Specifications, latest edition, except that the test method for compaction should be determined by ASTM D1557-78.

15. To have the selected sections perform to their greatest efficiency, it is important that the following items be considered:

A. Grading should not be performed during inclement weather.

B. Subexcavate unsuitable material.

C. Scarify exposed base, moisture and compact to a relative compaction of 95 percent at about 2 percent over optimum moisture content.

D. Engineered fill should be placed in thin lifts and compacted to 95% at about 2 percent over optimum moisture content.

E. Provide sufficient gradient to direct runoff to inboard side of road.

F. Use only quality materials of the type and thickness (minimum) specified. Base rock should meet Caltrans Standard Specifications for Class II Aggregate Base, and be angular in shape.

G. Compact the upper 6 inches of engineered fill and base rock sections to a relative dry density of 95 percent.

H. Place the asphaltic concrete during periods of fair weather when the free air temperature is within prescribed limits per Caltrans specifications.

I. Provide a routine maintenance program.

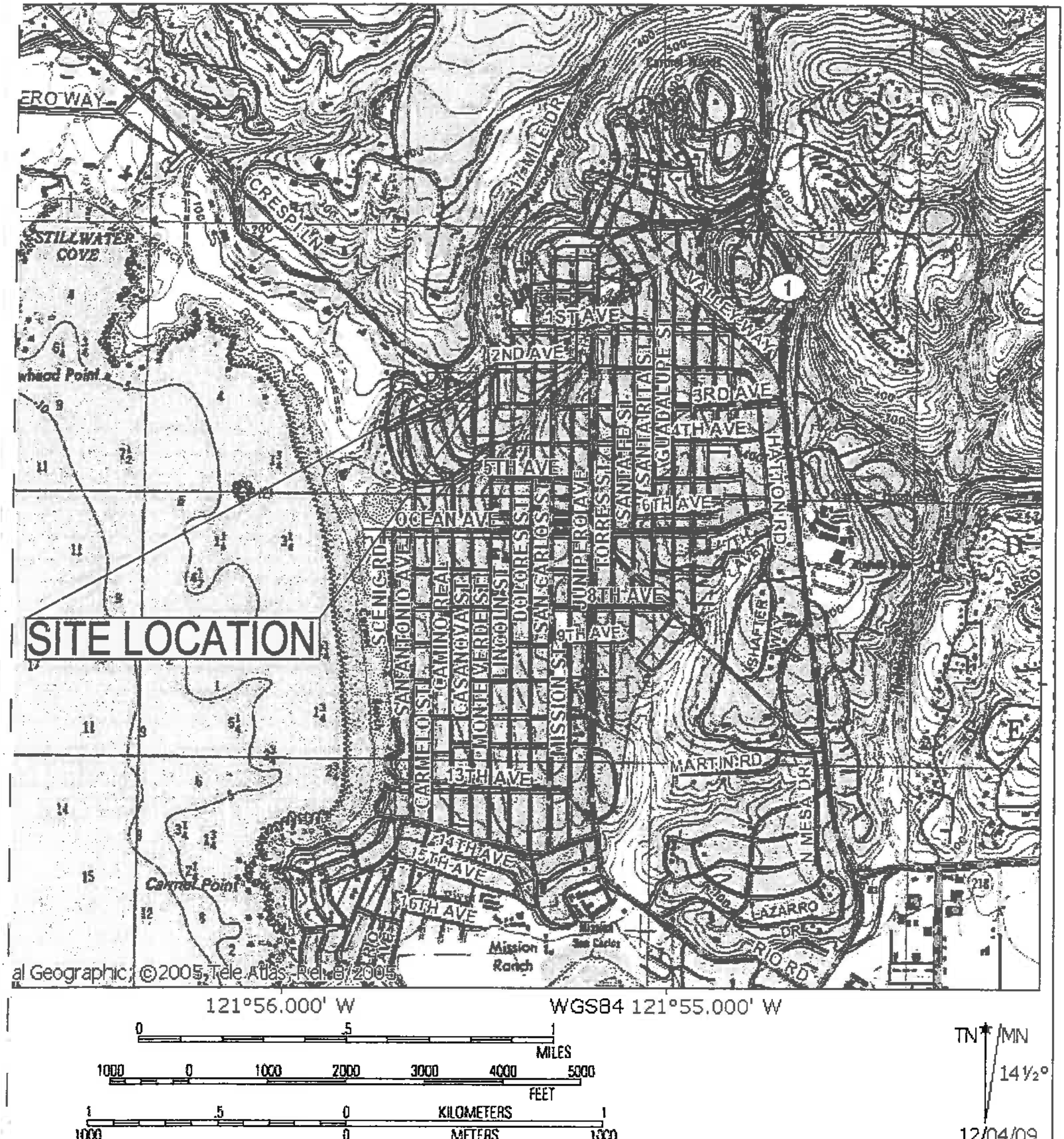
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice. No other warranty expressed or implied is made.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by a geotechnical engineer.

APPENDIX A



12/04/09

SITE VICINITY MAP
2ND AVENUE BETWEEN LOGAN AND PALOU

SCALE AS NOTED	CARMEL, CALIFORNIA
DRAWN BY MH	
DATE 4 DECEMBER 2009	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
REVISED	
JOB NO M9928	
FIGURE NO. 1	SHEET NO

LATITUDE: 36.5609 N.
LONGITUDE: 121.9631 W

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0	1-1 (L)		Fill Medium brown SAND, loose, damp trace roots	SM	11	ts/Fill	80	17	Saturated Direct Shear C = 142 osf 0 = 37 Ms = 38%
			Topsoil dark brown SAND	SM					
5	1-2 (L)				31				
			Native - Light brown weakly cemented SAND occasional Gravels, medium dense, damp	SM/SP		Qoa			
10	1-3 (L)		Unweathered SANDSTONE, brown grey coarse grain, slightly cemented, moist, some cohesion occasional Gravels of Siltstone and Granite	BR	82	Tus	109	12	% Passing #200 Sieve = 22.1 7.5% Clay
15	1-4 (L)		Red brown very coarse slightly cemented SANDSTONE, some cohesion, moist, hard		85/10"				
20	1-5 (T)		Grey basalt (weathered)	Tvb	74+	Tvb			
25	1-6 (T)		Very hard, colored, weathered basalt, damp		50+				
			Boring terminated at 26.5 feet						
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk | FIGURE NO. 2

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Fill 2" AC 11" BR	SM					
	2-1 (L)		Very loose, damp, brown SAND		7	ts/Fill	103	8	
			Medium orange brown coarse SAND, moist	SM					
	2-2 (T)		Dark brown SAND, Topsoil, very loose, damp terrace organics		3			10	% Passing #200 Sieve = 16.8
5			"Coffee Ground" dry						
10	2-3 (L)				32				
			Native Red brown SAND (almost no cementation) damp/dry medium dense	SM		Qoa			
	2-7 (T)				16			16	% Passing #200 Sieve = 32.6
15	2-4 (L)		Colorful SANDSTONE clasts of Siltstone, purple weathered basalt, (Tus?) cohesion areas, moist, medium dense (conglomerate)	BR	52	Tus	108	17	Direct Shear Saturated C = 1418 psf ϕ = 28 m = 25%
	2-5 (T)		Orange grey brown mottled SANDSTONE, slightly cemented, moist	BR	45			14	% Passing #200 Sieve = 20.6
20	2-6 (L)		Grey black Basalt, damp, very hard		50+	Tvb		14	
			Boring terminated at 21 feet						
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 3

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-3

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			3" AB	SM					
			Native		24				
	3-1 (L)		Red brown cleanish SAND (weakly cemented) medium dense, damp			Qoa	101	12	Saturated Direct Shear C = 318 psf 0 = 37 Ms = 24.1%
5	3-2 (L)		Increase cementation, dense, moist		64				
10	3-3 (T)		Increase cohesion and grey color, moist, medium dense		32			13	% Passing #200 Sieve = 29.6 Clay = 9.8%
15	3-4 (T)		Grey colorful cohesive SANDSTONE congolmerate, moist, hard	BR	72+	Tus			
	3-5 (T)		Grey Basalt	Tvb	50/3"	Tvb			
20			Boring terminated at 20 feet						
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 4

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-4

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Fill, light brown SAND with some aggregate, lense of beach SAND	SM	22	Ts/Fill	99	8	
	4-1 (L)							
	4-2 (T)	Dark brown SAND, moist, very loose Fill	SM	6				
5	4-3 (L)			15		101	11	Saturated Direct Shear C = 89 psf 0 = 40 Ms - 24
	4-4 (T)	Medium brown SAND, dry, loose		14				
10	4-5 (L)	Increase in cohesion some wood		16				
		Fill, drilling = fast	SM					
15	4-6 (T)	Very loose, some wood		2				
		Drilling = fast Coffee Grounds						
20	4-7 (T)	Native Harder Drilling	BR	50/8"	Tus			
		Bouncing on basalt floater Basalt scraping drilling at 20 feet	Tvb					
		Boring terminated at 21.5 feet						
25								
30								
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 5

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-5

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			AC 1 1/2" AB	SM					
	5-1 (L)		Fill, light brown SAND with Gravels, lense		5	TS/Fill			
			Brown SAND, loose, damp						
5	5-2 (L)		Dark brown coffee grounds, trace roots hairs		7				
10	5-3 (T)		Native Medium brown SAND some cohesion slight cementation	SM	31	Qoa		20	Atterberg Limits PI = 16 LL = 33%
15	5-4 (L)		Increase colorful, volcanic clasts cohesion SANDSTONE, slightly cemented, moist, hard	BR	72+	Tvs			
20	5-5 (T)		Weathered cohesive, colorful Basalt, hard, moist	Tvb	80+				
			Boring terminated at 21.00 feet						
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 6

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-6

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Topsoil/Fill?	SM					
	6-1 (T)		Dark brown fine uniform SAND "coffee grounds" dry, loose		4	TS/Fill			
5			Grades to medium brown						
	6-2 (T)				4				
			Native Red brown uniform SAND, damp, dense	SM		Qoa			
10	6-3 (T)		Slightly cemented		7				
	6-4 (T)		Increase in Cohesion		72+				
15			Weathered Basalt, coloful, hard	Tvb		Tus/Tvb			
			Boring terminated at 16.5 feet						
20									
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 7

Haro, Kasunich & Associates, Inc.

2nd Avenue Between Lopez & North Casanov

PROJECT NO. M9928

LOGGED BY VO **DATE DRILLED** December 1, 2009 **BORING DIAMETER** 6" **BORING NO.** B-7

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0	7-1 (L)		Dark red brown SAND then light brown SAND, slightly cemented, moist, dense	SM	38				
5	7-2 (T)		Light red brown slightly cemented, damp, very dense		79	Qoa		10	% Passing #200 Sieve = 24.6
10	7-3 (T)		Slightly darker brown, slightly more cemented		48				
15	7-4 (T)		Increase in cohesion SILTSTONE fragments, grey brown cohesion conglomerate SANDSTONE, moist, dense	BR	43	Tus		14	PI = 11 LL = 26.4
20	7-5 (T)		Grey weathered basalt hard		50+	Tvb			
			Boring terminated at 21.5 feet						
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

FIGURE NO. 8

Direct Shear

Project:	Second Avenue
Sample #	1-1-1
Description	Dark Brown Silty Sand,Wood Fragments

Date	12/8/2009
Tested By:	JR/MA

Test Number	1	2	3	4
Normal Pressure (KSF)	1000	2000	4000	8000
Max Shear Stress	27.3	53	98.8	
Shear Stress (PSF)	880.6	1709.7	3187.1	

Equation of Trendline	
Intercept	Slope
141.9	0.7645

C (PSF)	PHI
142	37

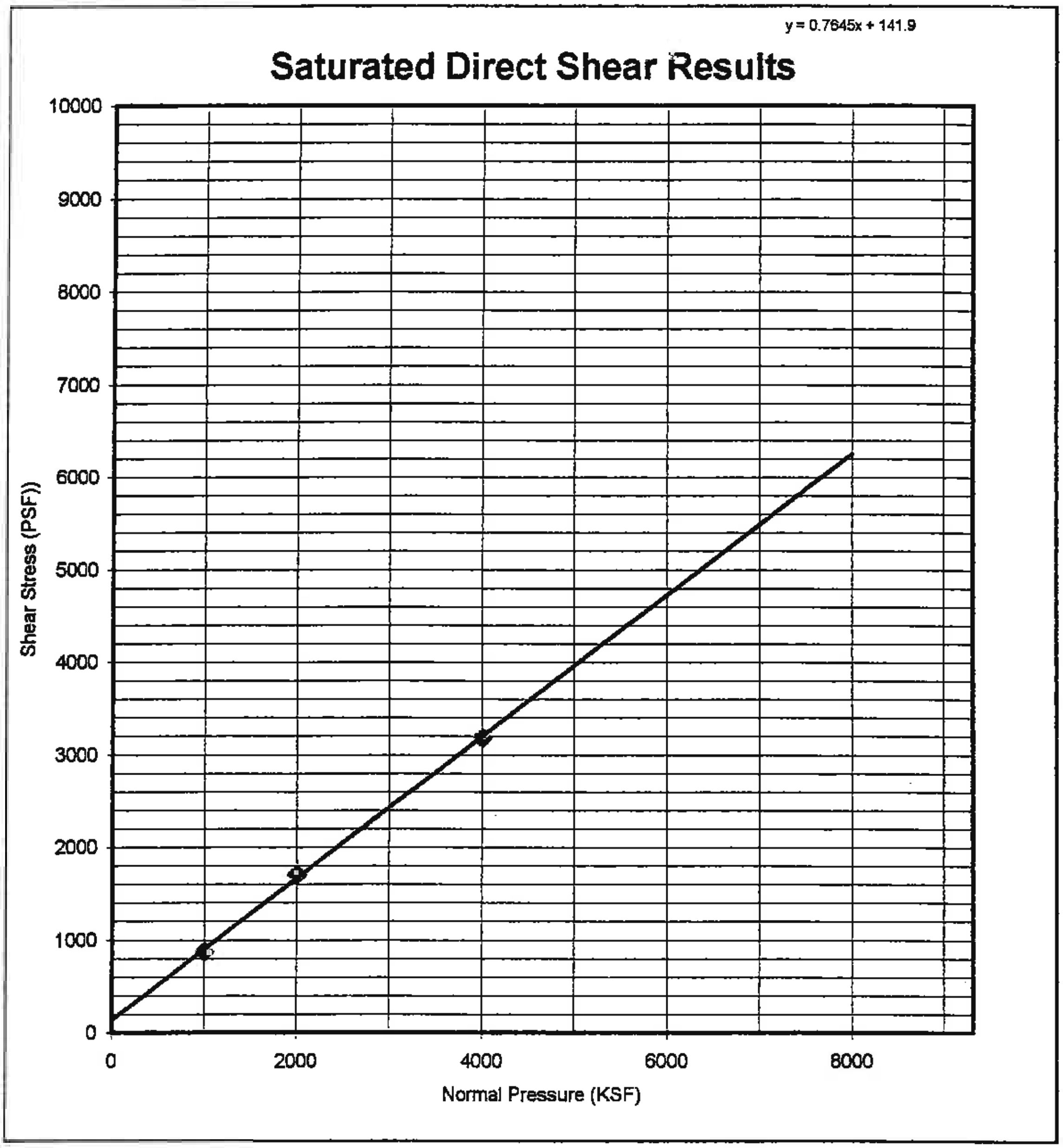


Figure No. 9

Direct Shear

Project:	Second Avenue		Date	12/7/2009
Sample #	2-4-1		Tested By:	JR/MA
Description	Brown Clayey Sand Small ,Large Gravels			

Test Number	1	2	3	4
Normal Pressure (KSF)	1000	2000	4000	8000
Max Shear Stress	57.3	60.6	124	165.5
Shear Stress (PSF)	1848.4	1954.8	4516.1	5335.7

Equation of Trendline	
Intercept	Slope
1417.9	0.5322

C (PSF)	PHI
1418	28

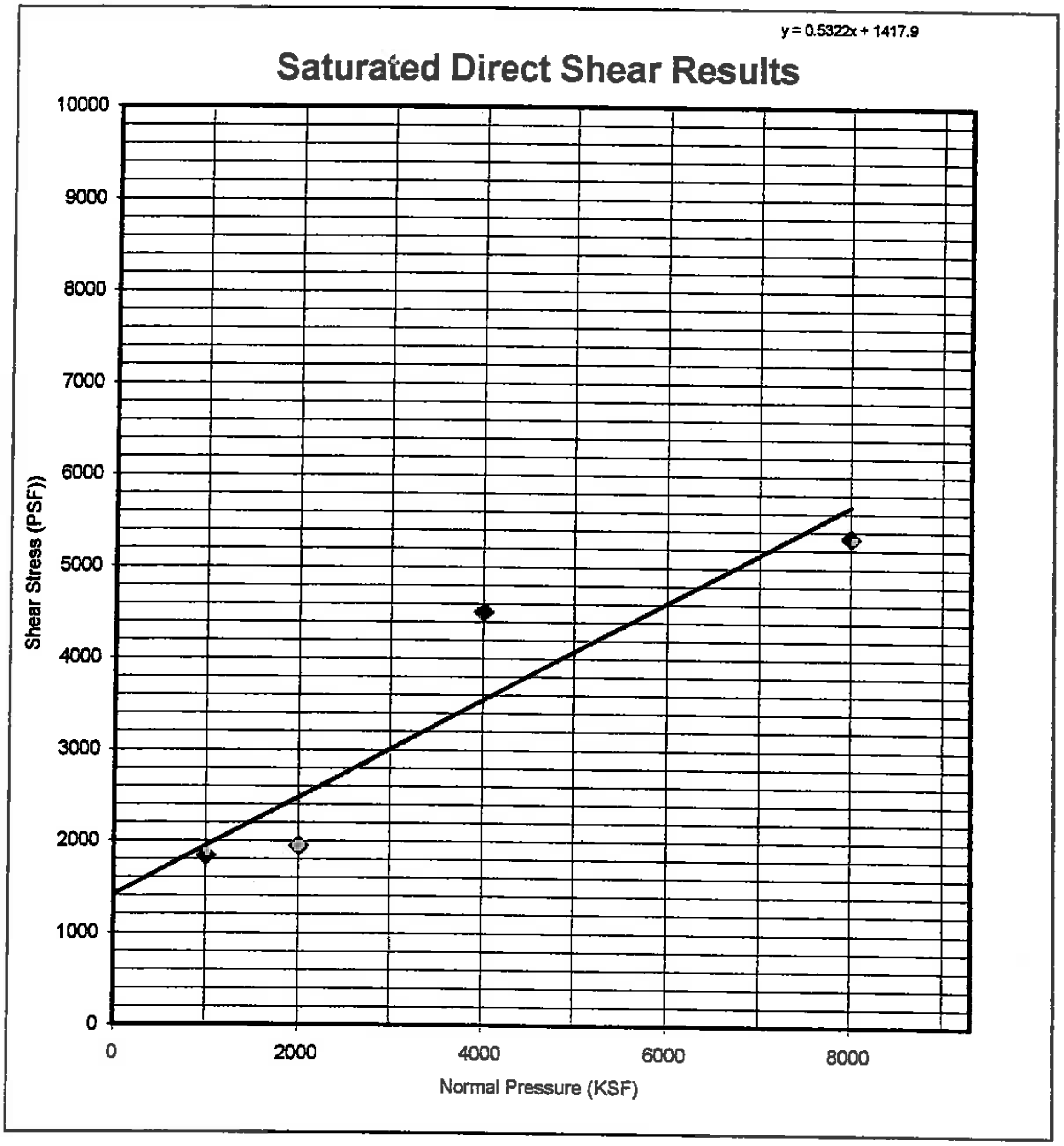


Figure No. 10

Direct Shear

Project:	Second Avenue
Sample #	3-1-1
Description	Rusty Brown Silty Sand

Date	12/7/2009
Tested By:	JR/MA

Test Number	1	2	3	4
Normal Pressure (KSF)	1000	2000	4000	8000
Max Shear Stress	29.9	61.1	101.2	
Shear Stress (PSF)	964.5	1971	3264.5	

Equation of Trendline	
Intercept	Slope
317.75	0.7495

C (PSF)	PHI
318	37

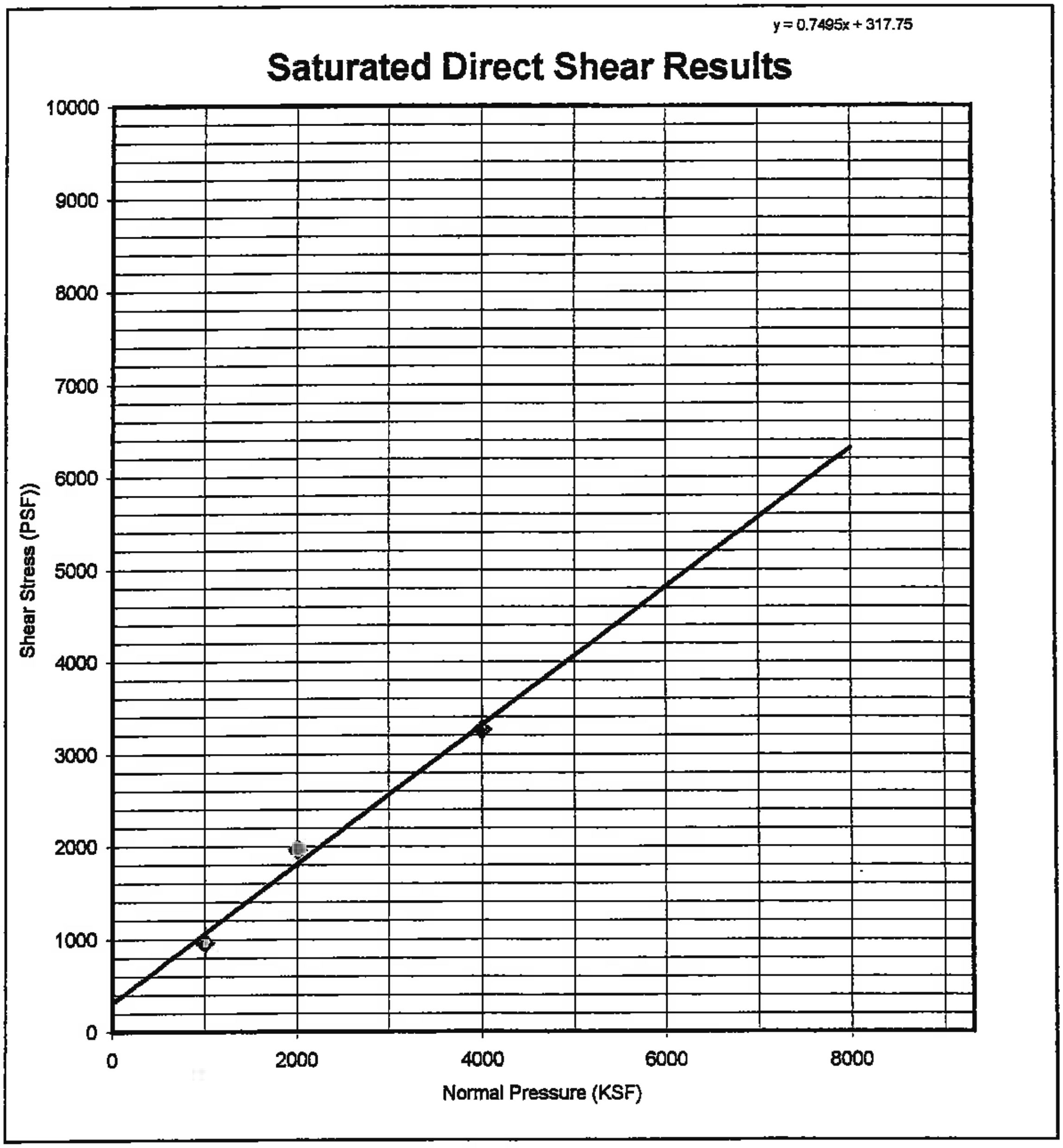


Figure No. 11

Direct Shear

Project:	Second Avenue
Sample #	4-3-1
Description	Brown Dark Brown Silty Sand

Date	12/8/2009
Tested By:	JR/MA

Test Number	1	2	3	4
Normal Pressure (KSF)	1000	2000	4000	8000
Max Shear Stress	32.3	49.3	108.4	
Shear Stress (PSF)	1041.9	1590.3	3496.8	

Equation of Trendline	
Intercept	Slope
88.65	0.8376

C (PSF)	PHI
89	40

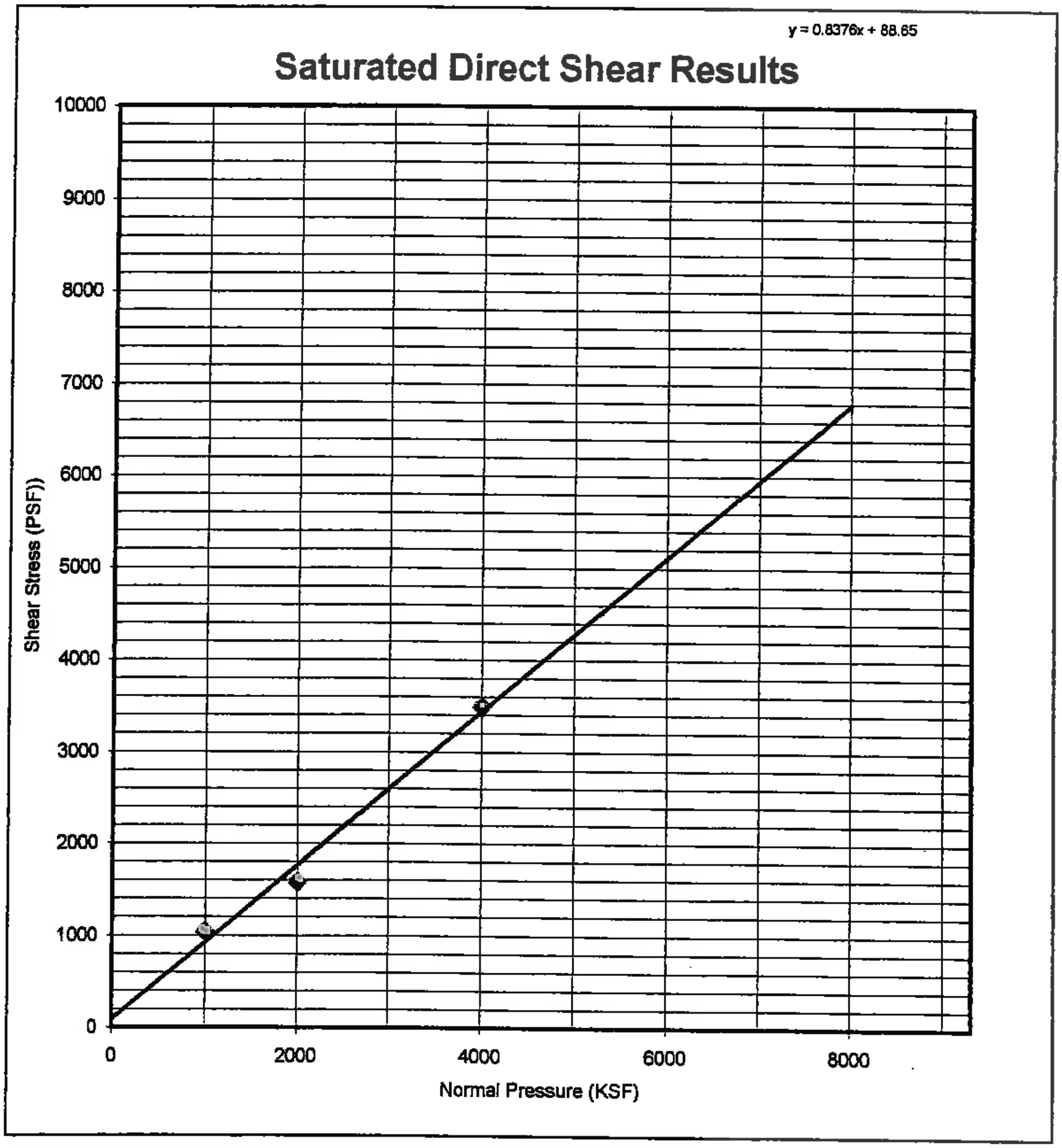


Figure No. 12

FULL GRADATION				Project Name:	2nd Ave.	
				File No.:	M 9928	
Moisture Density				Sample No.:	2-2	
Height Of Sample (in) or Enter "Bag"		Bag		Date:	December 8, 2009	
Tare No.		399		By:	JR/MA	
Gross Wet Weight		317.5		Sample Description:		
Gross Dry Weight		295.8		Dark Brown Silty Sand		
Tare Weight		72.7				
Net Dry Weight		223.1		Group Symbol:	SM	
Weight of Water		21.7		Gravel Content:	0.0%	
% Moisture		9.7%		Sand Content:	83.2%	
Dry Density		#VALUE!		Fines Content:	16.8%	
Sieve	Weight Retained	% Retained	Cumulative Percent			Specs
			Retained	Passing		
2"	0.0	0.0%	0.0%	100.0%		
1½"	0.0	0.0%	0.0%	100.0%		
1"	0.0	0.0%	0.0%	100.0%		
¾"	0.0	0.0%	0.0%	100.0%		
½"	0.0	0.0%	0.0%	100.0%		
3/8"	0.0	0.0%	0.0%	100.0%		
No. 4	0.0	0.0%	0.0%	100.0%		
No. 8	0.8	0.4%	0.4%	99.6%		
No. 10	0.2	0.1%	0.4%	99.6%		
No. 16	1.0	0.4%	0.9%	99.1%		
No. 30	18.8	8.4%	9.3%	90.7%		
No. 40	44.2	19.8%	29.1%	70.9%		
No. 50	56.5	25.3%	54.5%	45.5%		
No. 100	52.6	23.6%	78.0%	22.0%		
No. 200	11.6	5.2%	83.2%	16.8%		
Pan	37.2 0.2	16.8%	100.0%	0.0%		
Total	223.1	100.0%		100.0%		
Before	223.1		After			
Dry Wt.			Dry Wt.	258.6		
Tare			Tare	72.7		
			185.9			

Haro Kasunich and Associates
Geotechnical and Coastal Engineers

Test Report Prepared By HKA Lab
Date :12/8/2009

Figure No. 13

FULL GRADATION		Project Name:	2nd Ave.
		File No.:	M 9928
Moisture Density		Sample No.:	2-5
Height Of Sample (in) or Enter "Bag"	Bag	Date:	December 8, 2009
Tare No.	127	By:	JR/MA
Gross Wet Weight	493.2	Sample Description:	
Gross Dry Weight	441.5	Rust Brown Clayey Sand	
Tare Weight	76.0		
Net Dry Weight	365.5	Group Symbol:	SC
Weight of Water	51.7	Gravel Content:	1.1%
% Moisture	14.1%	Sand Content:	78.3%
Dry Density	#VALUE!	Fines Content:	20.6%

Sieve	Weight Retained	% Retained	Cumulative Percent Retained	Cumulative Percent Passing	Specs
2"	0.0	0.0%	0.0%	100.0%	
1½"	0.0	0.0%	0.0%	100.0%	
1"	0.0	0.0%	0.0%	100.0%	
¾"	0.0	0.0%	0.0%	100.0%	
½"	3.0	0.8%	0.8%	99.2%	
3/8"	0.0	0.0%	0.8%	99.2%	
No. 4	1.2	0.3%	1.1%	98.9%	
No. 8	9.2	2.5%	3.7%	96.3%	
No. 10	6.0	1.6%	5.3%	94.7%	
No. 16	30.2	8.3%	13.6%	86.4%	
No. 30	93.7	25.6%	39.2%	60.8%	
No. 40	47.1	12.9%	52.1%	47.9%	
No. 50	42.3	11.6%	63.7%	36.3%	
No. 100	45.2	12.4%	76.0%	24.0%	
No. 200	12.4	3.4%	79.4%	20.6%	
Pan	74.5 0.7	20.6%	100.0%	0.0%	
Total	365.5	100.0%		100.0%	

Before	365.5	After	
Dry Wt.		Dry Wt.	367.0
Tare		Tare	76
		291.0	

Test Report Prepared By HKA Lab
Date :12/8/2009

Haro Kasunich and Associates
Geotechnical and Coastal Engineers

Figure No. 14

HARO KASUNICH & ASSOCIATES
Soil Testing and Inspection - Field and Laboratory

FULL GRADATION				Project Name:	2nd Ave.	
				File No.:	M 9928	
Moisture Density				Sample No.:	2-7	
Height Of Sample (in) or Enter "Bag"		Bag		Date:	December 8, 2009	
Tare No.		116		By:	JR/MA	
Gross Wet Weight		343.1		Sample Description:		
Gross Dry Weight		305.6		Brown Clayey Sand		
Tare Weight		75.5				
Net Dry Weight		230.1		Group Symbol:	SC	
Weight of Water		37.5		Gravel Content:	1.2%	
% Moisture		16.3%		Sand Content:	66.2%	
Dry Density		#VALUE!		Fines Content:	32.6%	

Sieve	Weight Retained	% Retained	Cumulative Percent Retained	Cumulative Percent Passing	Specs
2"	0.0	0.0%	0.0%	100.0%	
1½"	0.0	0.0%	0.0%	100.0%	
1"	0.0	0.0%	0.0%	100.0%	
¾"	0.0	0.0%	0.0%	100.0%	
½"	0.0	0.0%	0.0%	100.0%	
3/8"	0.0	0.0%	0.0%	100.0%	
No. 4	2.7	1.2%	1.2%	98.8%	
No. 8	4.5	2.0%	3.1%	96.9%	
No. 10	0.6	0.3%	3.4%	96.6%	
No. 16	8.2	3.6%	7.0%	93.0%	
No. 30	20.8	9.0%	16.0%	84.0%	
No. 40	23.1	10.0%	26.0%	74.0%	
No. 50	31.1	13.5%	39.5%	60.5%	
No. 100	39.5	17.2%	56.7%	43.3%	
No. 200	24.5	10.6%	67.4%	32.6%	
Pan	70.0 5.1	32.6%	100.0%	0.0%	
Total	230.1	100.0%		100.0%	

Before	230.1	After	
Dry Wt.		Dry Wt.	235.6
Tare		Tare	75.5
		160.1	

Test Report Prepared By HKA Lab
Date :12/8/2009

Haro Kasunich and Associates
Geotechnical and Coastal Engineers

Figure No. 15

FULL GRADATION		Project Name:	2nd Ave.
		File No.:	M 9928
Moisture Density		Sample No.:	7-2
Height Of Sample (in) or Enter "Bag"	Bag	Date:	December 8, 2009
Tare No.	240	By:	JR/MA
Gross Wet Weight	517.0	Sample Description:	
Gross Dry Weight	476.0	Rusty Brown Silty Sand	
Tare Weight	81.2		
Net Dry Weight	394.8	Group Symbol:	SM
Weight of Water	41.0	Gravel Content:	0.0%
% Moisture	10.4%	Sand Content:	75.4%
Dry Density	#VALUE!	Fines Content:	24.6%

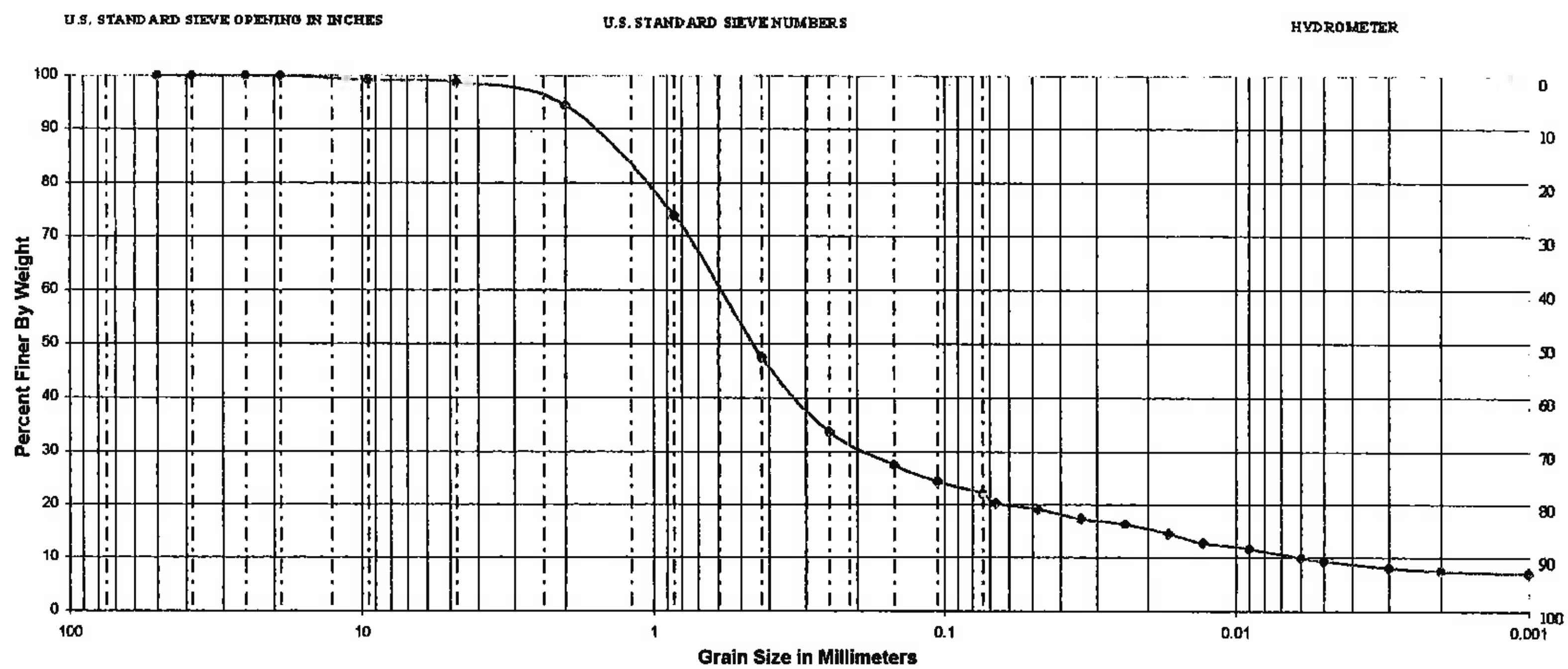
Sieve	Weight Retained	% Retained	Cumulative Percent Retained	Cumulative Percent Passing	Specs
2"	0.0	0.0%	0.0%	100.0%	
1½"	0.0	0.0%	0.0%	100.0%	
1"	0.0	0.0%	0.0%	100.0%	
¾"	0.0	0.0%	0.0%	100.0%	
½"	0.0	0.0%	0.0%	100.0%	
3/8"	0.0	0.0%	0.0%	100.0%	
No. 4	0.0	0.0%	0.0%	100.0%	
No. 8	0.5	0.1%	0.1%	99.9%	
No. 10	0.1	0.0%	0.2%	99.8%	
No. 16	1.0	0.3%	0.4%	99.6%	
No. 30	27.9	7.1%	7.5%	92.5%	
No. 40	69.6	17.6%	25.1%	74.9%	
No. 50	95.1	24.1%	49.2%	50.8%	
No. 100	79.9	20.2%	69.4%	30.6%	
No. 200	23.5	6.0%	75.4%	24.6%	
Pan	96.4 0.8	24.6%	100.0%	0.0%	
Total	394.8	100.0%		100.0%	

Before	394.8	After	
Dry Wt.		Dry Wt.	379.6
Tare		Tare	81.2
		298.4	

Haro Kasunich and Associates
Geotechnical and Coastal Engineers

Test Report Prepared By HKA Lab
Date :12/8/2009

Figure No. 16



Gravel Content:	**1.2%**
Sand Content:	**76.7%**
Silt Content	**14.6%**
Clay Content	**7.5%**
Cumulative Sum	**100.0%**

Sample Description: Orange Brn Green Brn Clayey Sand
Group Symbol: SC

Haro, Kasunich & Associates, Inc.
116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

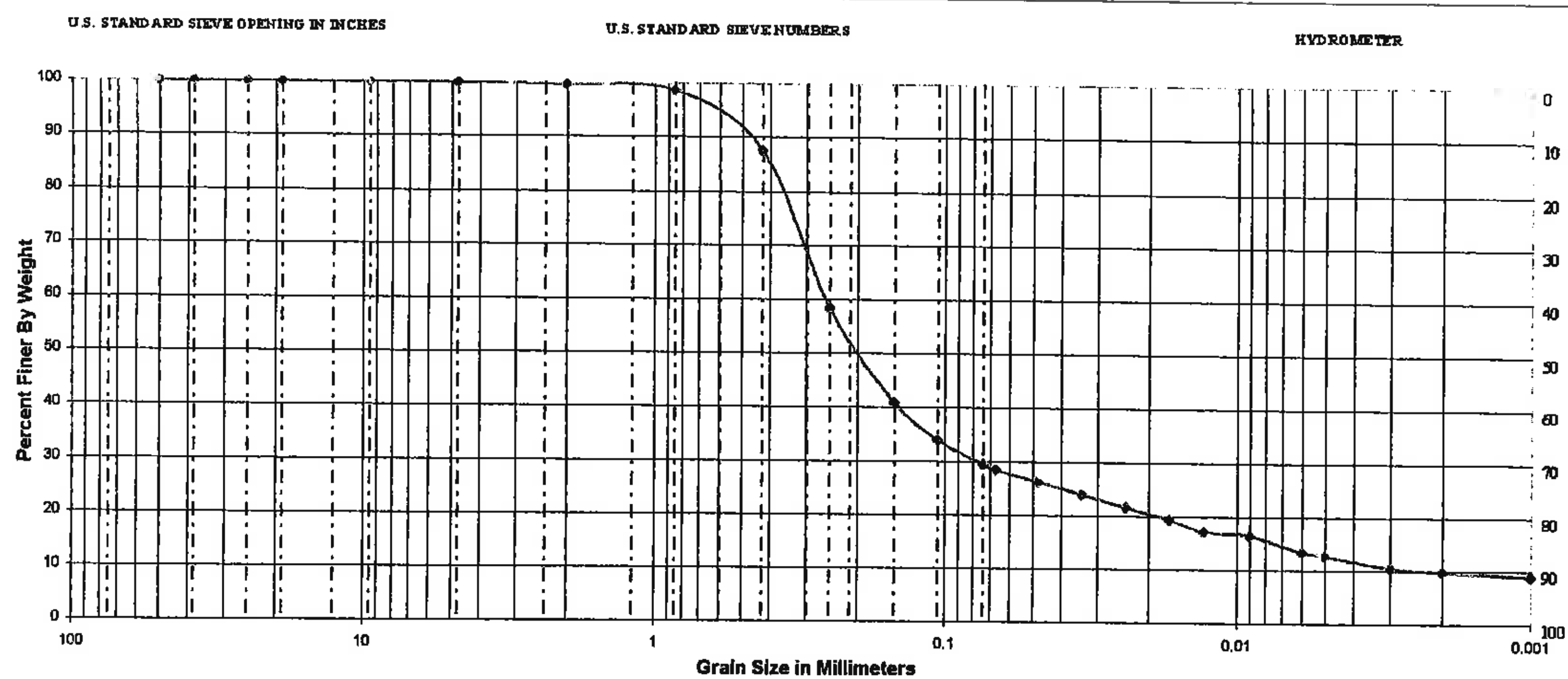
HKA Project No:	M 9928
Sample No:	1-3-1
Date:	December 9, 2009

MECHANICAL AND HYDROMETER GRAIN SIZE ANALYSIS

Second Avenue

Figure No. 17

44



Gravel Content:	**0.0%**
Sand Content:	**70.4%**
Silt Content	**19.8%**
Clay Content	**9.8%**
Cumulative Sum	**100.0%**

Sample Description: Brown Clayey Sand
Group Symbol: SC

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers
116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

HKA Project No:	M 9928
Sample No:	3-3
Date:	December 9, 2009

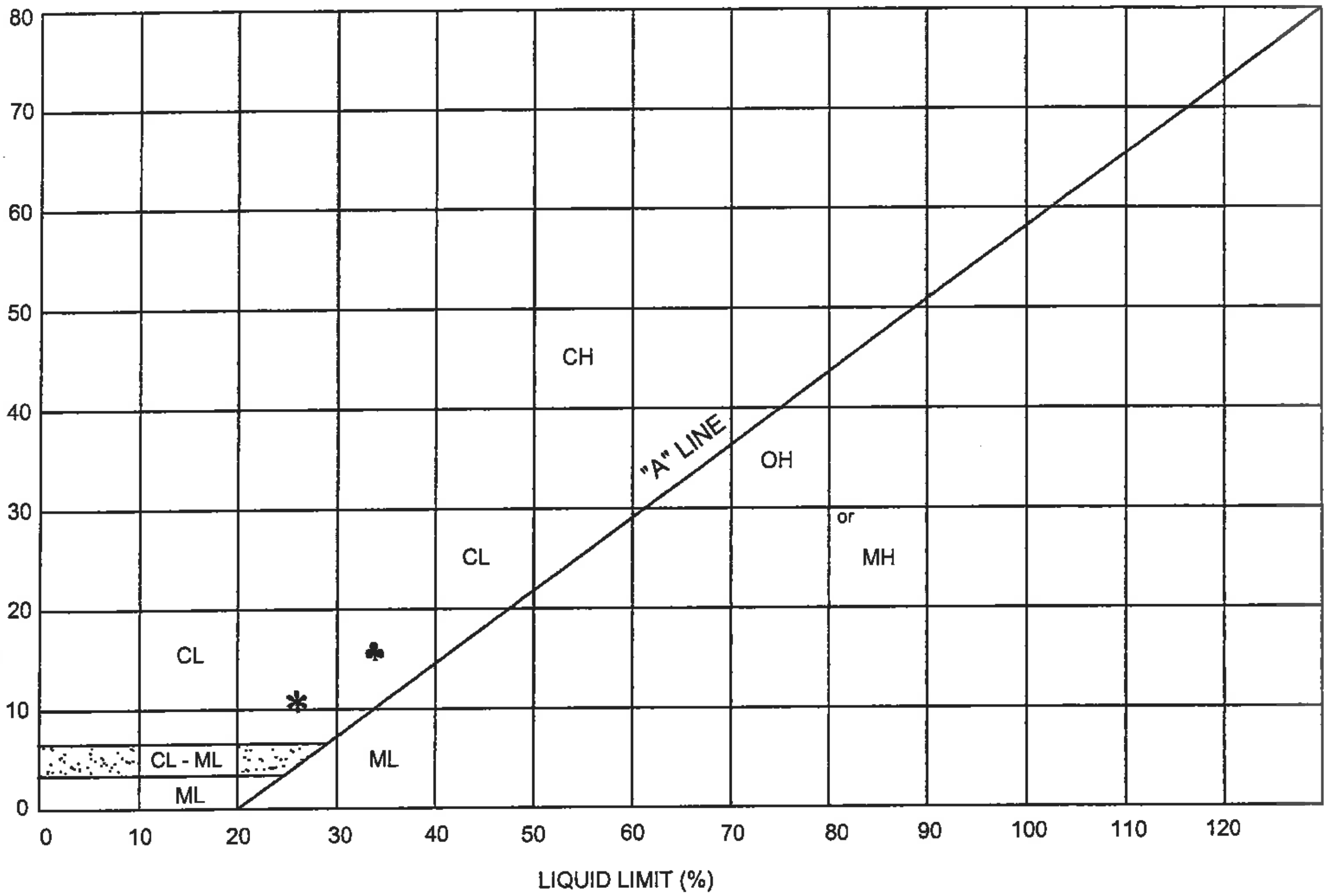
MECHANICAL AND HYDROMETER GRAIN SIZE ANALYSIS

Second Avenue

Figure No. 18

45

PLASTICITY CHART



PLASTICITY DATA

Key Symbol	Sample Number	Depth (feet)	Natural Water Content W(%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	Liquidity Index W - PL / LL -PL	Unified Soil Classification Symbol
♣	5-3	11	19.5	17.0	32.9	**16**	0.16	CL
*	7-4	16	14.0	15.5	26.4	**11**	-0.14	CL

ATTERBERG LIMITS TEST RESULTS
SECOND AVENUE

SCALE	NA	CARMEL, CALIFORNIA
DRAWN BY	MH	
DATE	22 DECEMBER 2009	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-1475
REVISED		
JOB NO	M9928	

FIGURE NO. 19 | SHEET NO OF 1

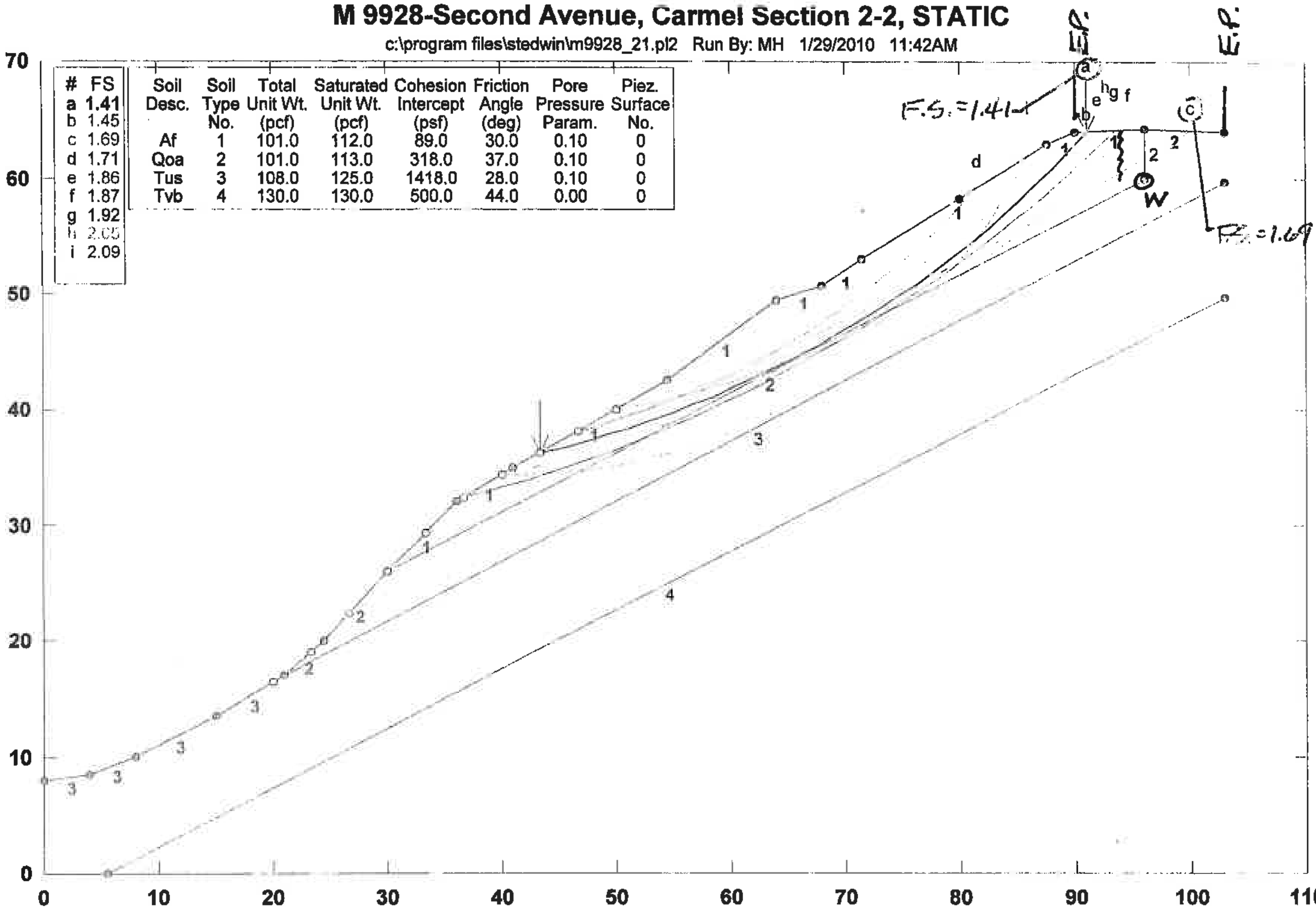
Project No. M9928
8 February 2010

APPENDIX B

Slope Stability Analysis

M 9928-Second Avenue, Carmel Section 2-2, STATIC

c:\program files\stedwin\m9928_21.pl2 Run By: MH 1/29/2010 11:42AM



STABL6H FSmin=1.41

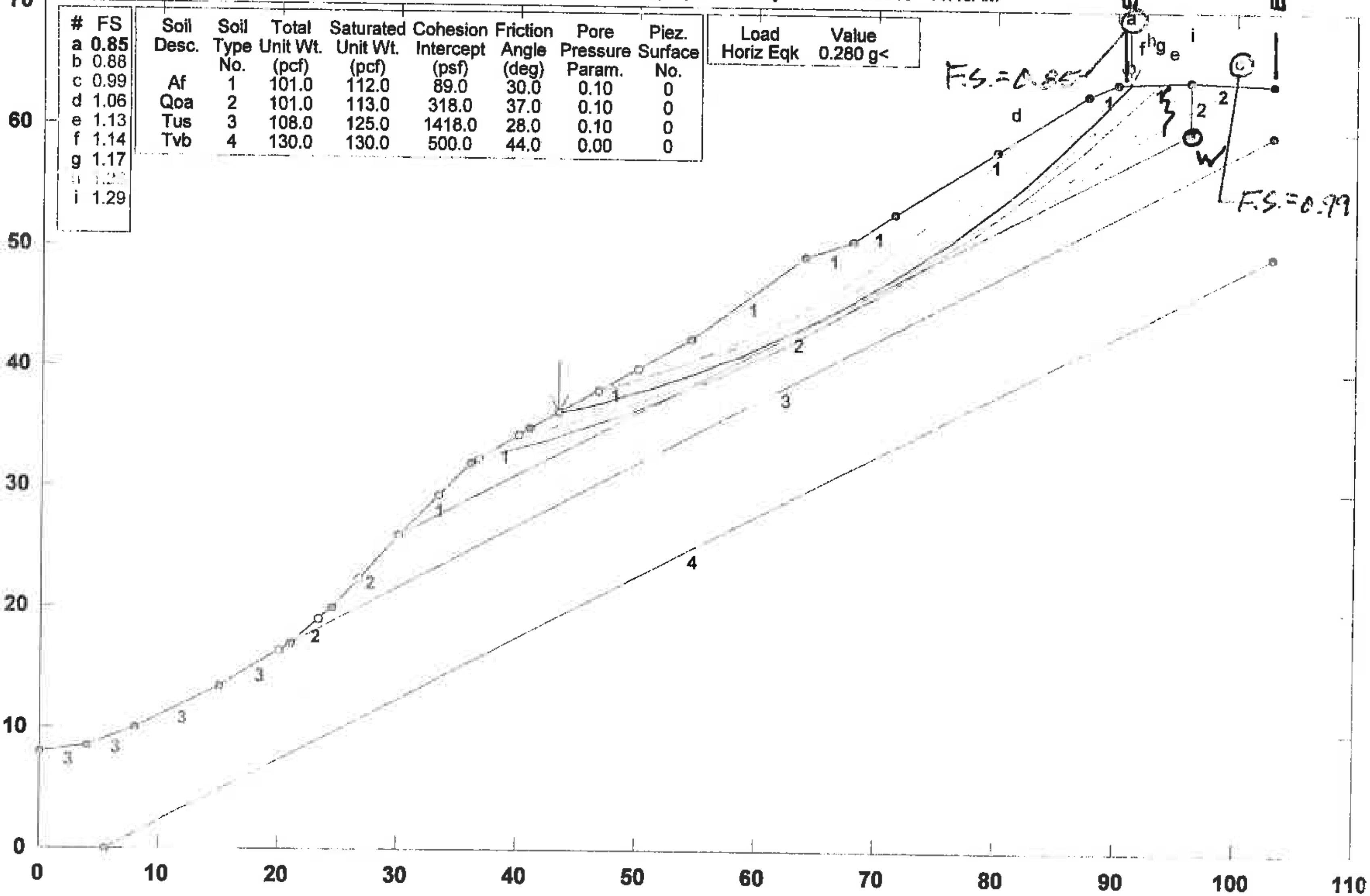
Safety Factors Are Calculated By The Modified Janbu Method

STED

Figure B1.

M 9928-Second Avenue, Carmel Section 2-2, SEISMIC

c:\program files\stedwin\m9928_21.pl2 Run By: MH 1/29/2010 11:40AM



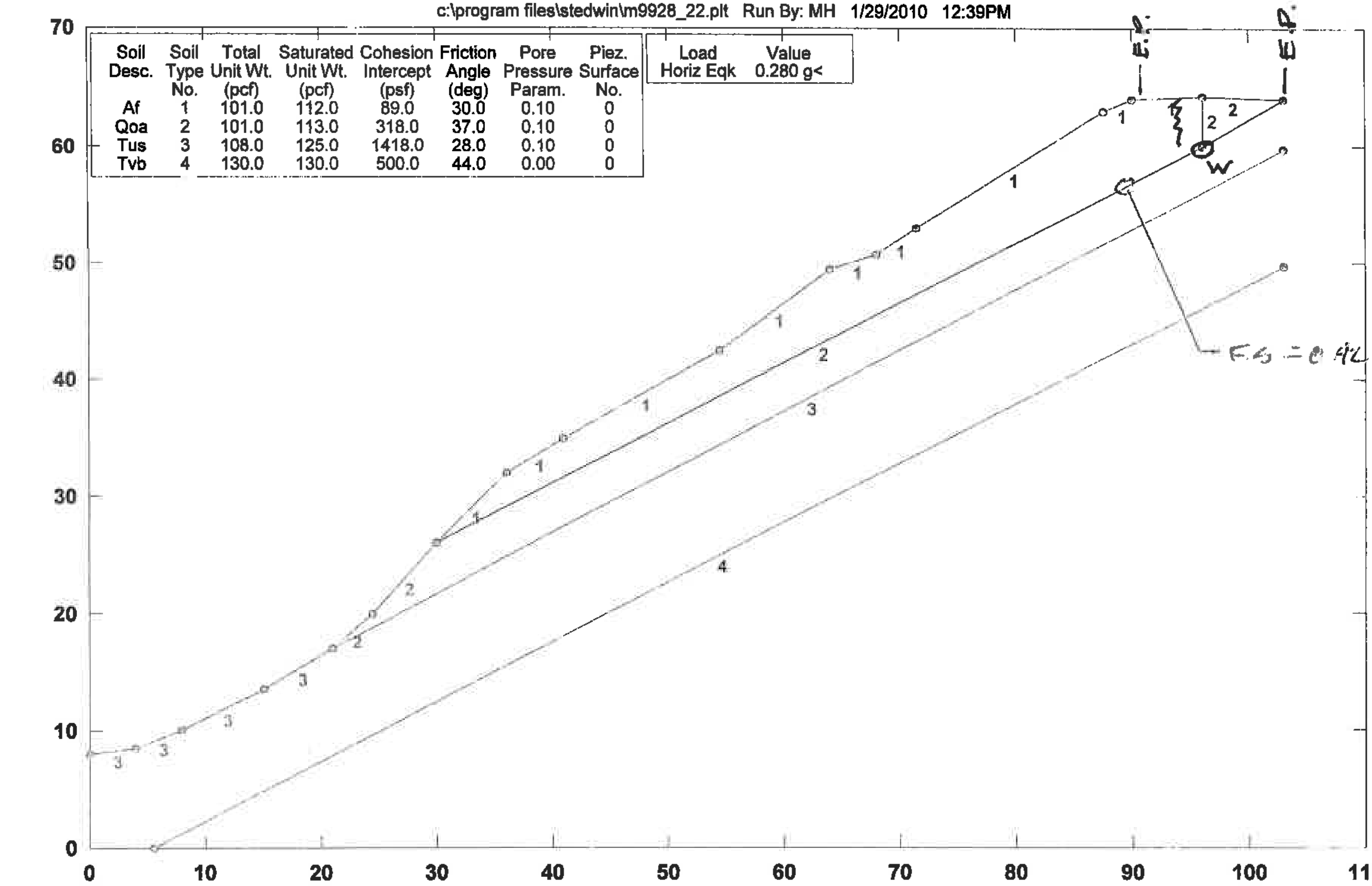
STABL6H FSmin=0.85
Safety Factors Are Calculated By The Modified Janbu Method

STED

Figure B-2.

M 9928-Second Avenue, Carmel Section 2-2, SEISMIC,planar,inbd ssmh

c:\program files\stedwin\m9928_22.plt Run By: MH 1/29/2010 12:39PM



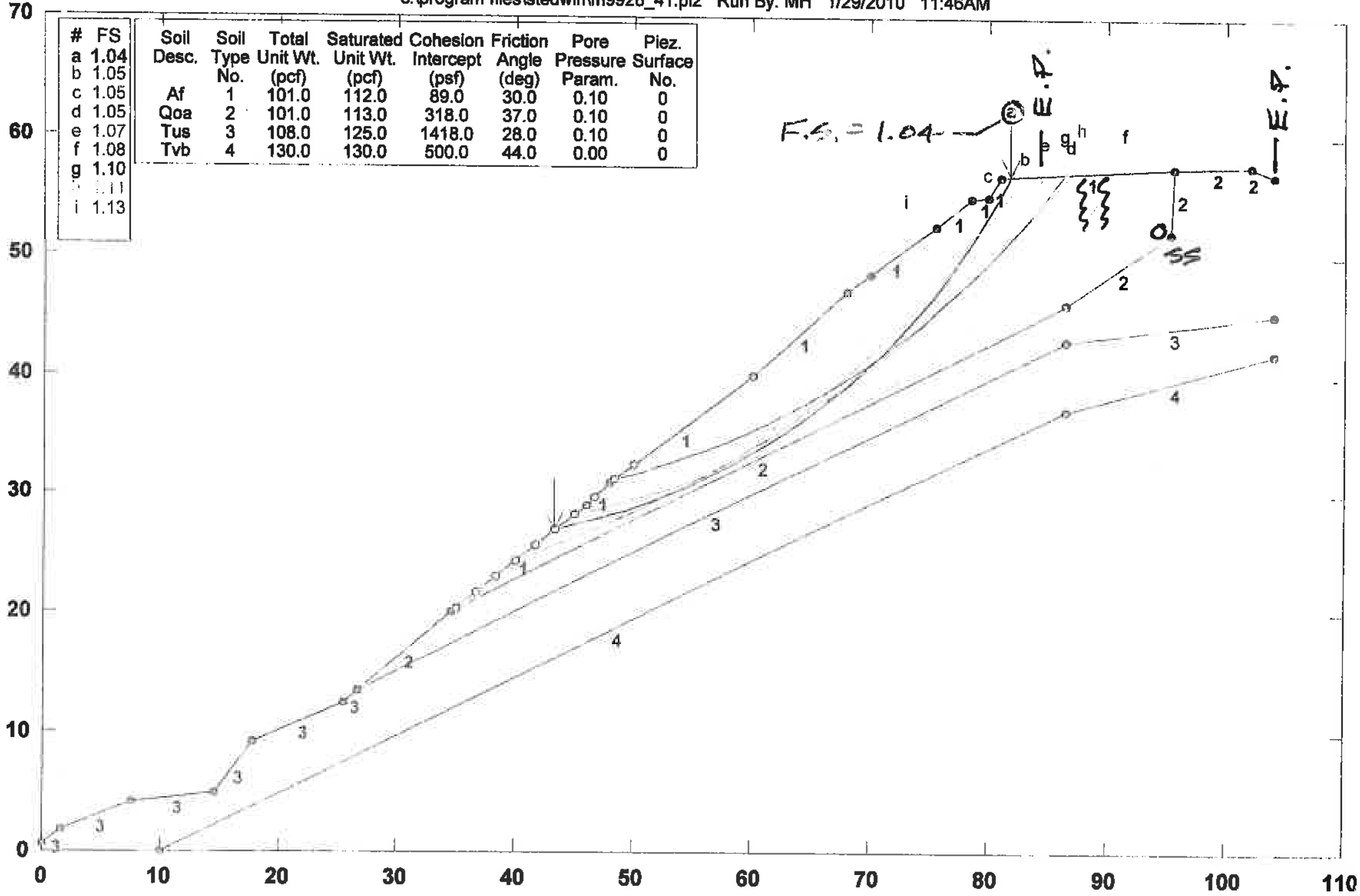
STABL6H FSmin=0.92
Factors of Safety Calculated by Janbu Method

STED

Figure B-3.

M9928-Second Avenue, Carmel Section 4-4, static,outboard

c:\program files\stedwin\m9928_41.pl2 Run By: MH 1/29/2010 11:46AM



#	FS
a	1.04
b	1.05
c	1.05
d	1.05
e	1.07
f	1.08
g	1.10
h	1.11
i	1.13

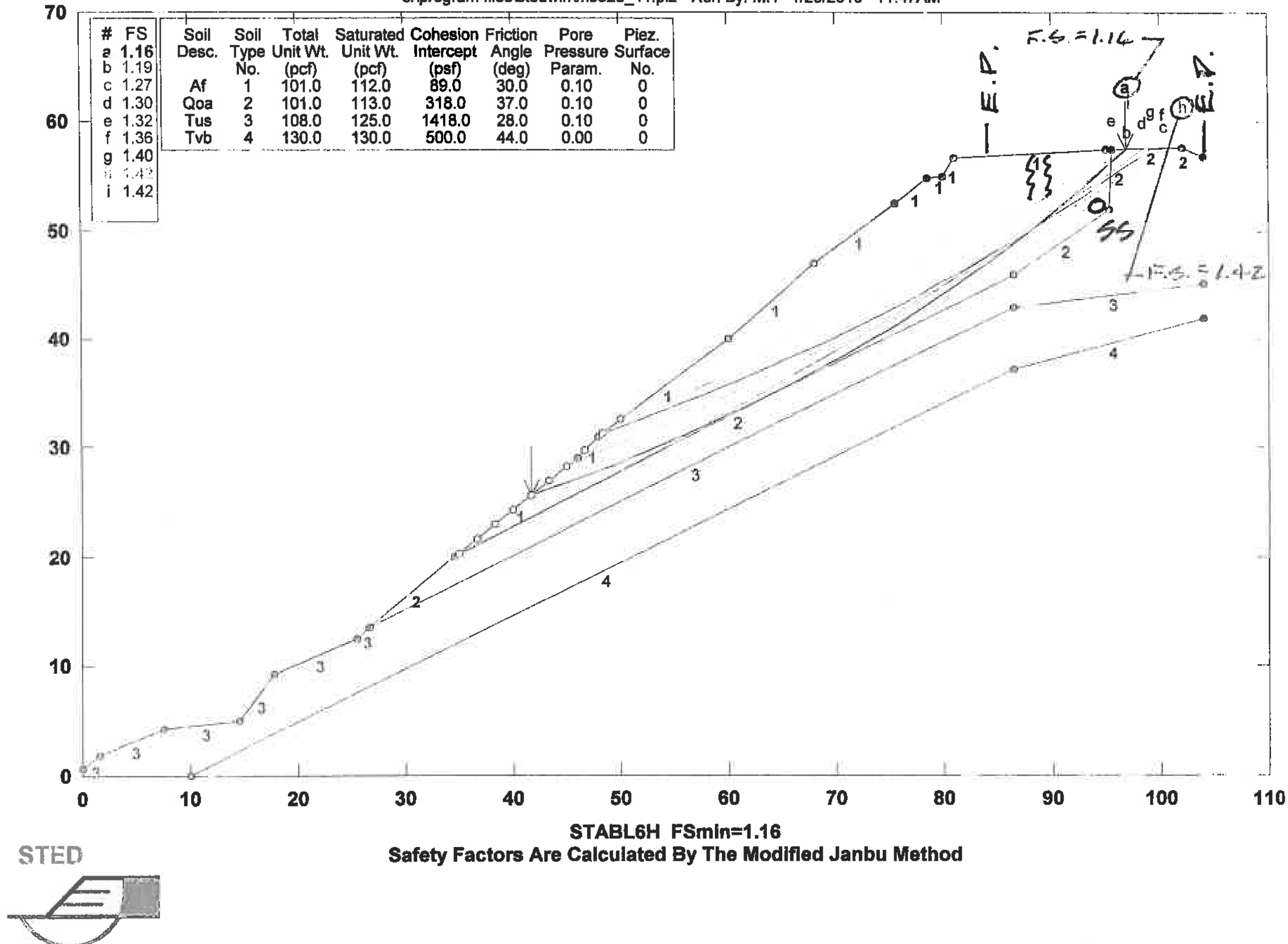
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0

STED

Figure B-4.

M9928-Second Avenue, Carmel Section 4-4, static,inboard

c:\program files\stedwin\m9928_41.pl2 Run By: MH 1/29/2010 11:47AM

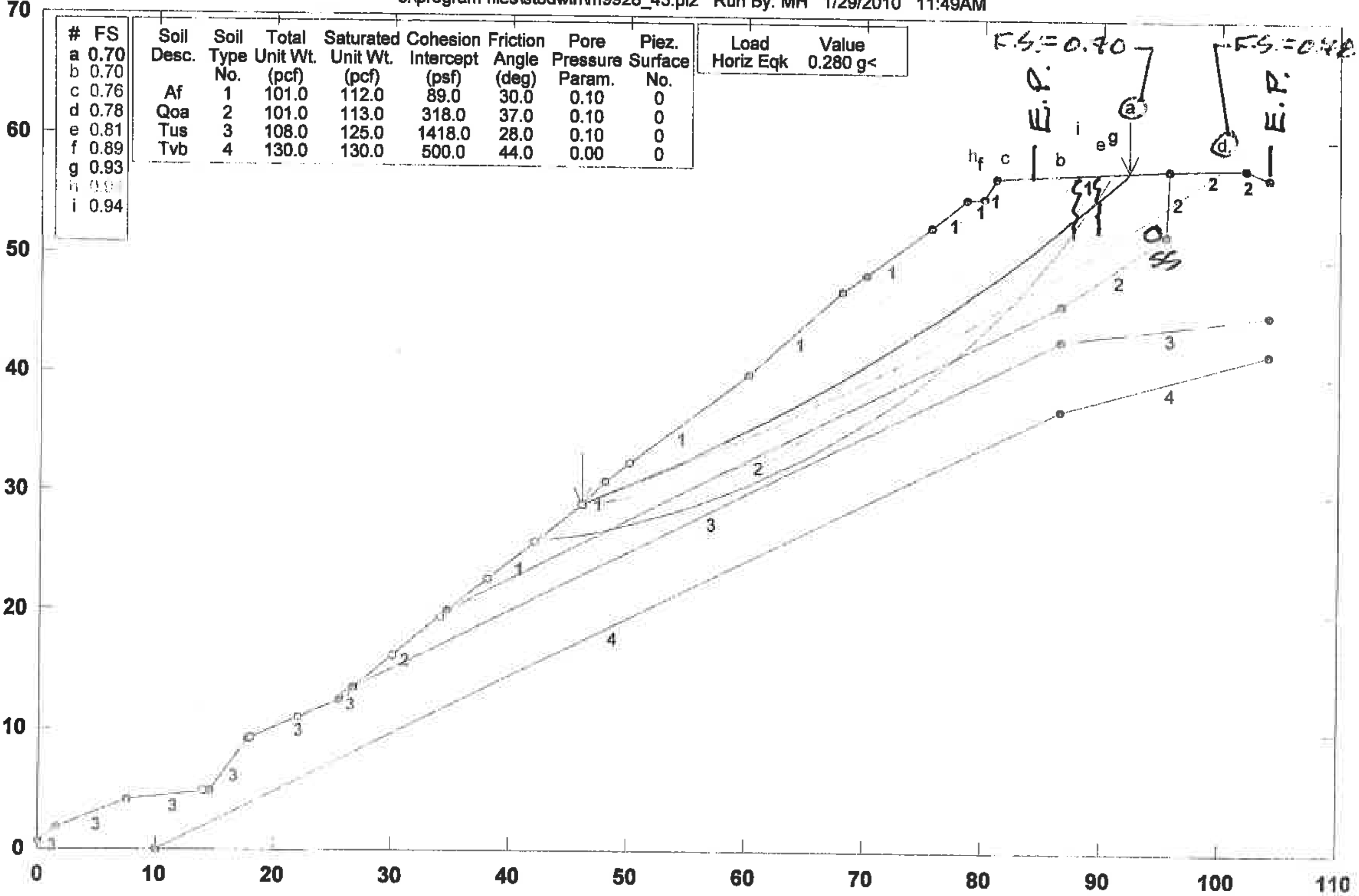


STABL6H FSmin=1.16
Safety Factors Are Calculated By The Modified Janbu Method

Figure B-5.

M9928-Second Avenue, Carmel Section 4-4, SEISMIC

c:\program files\stedwin\m9928_43.pl2 Run By: MH 1/29/2010 11:49AM



#	FS
a	0.70
b	0.70
c	0.76
d	0.78
e	0.81
f	0.89
g	0.93
h	[illegible]
i	0.94

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0

Load	Value
Horiz Eqk	0.280 g<

STABL6H FSmin=0.70

Safety Factors Are Calculated By The Modified Janbu Method

STED

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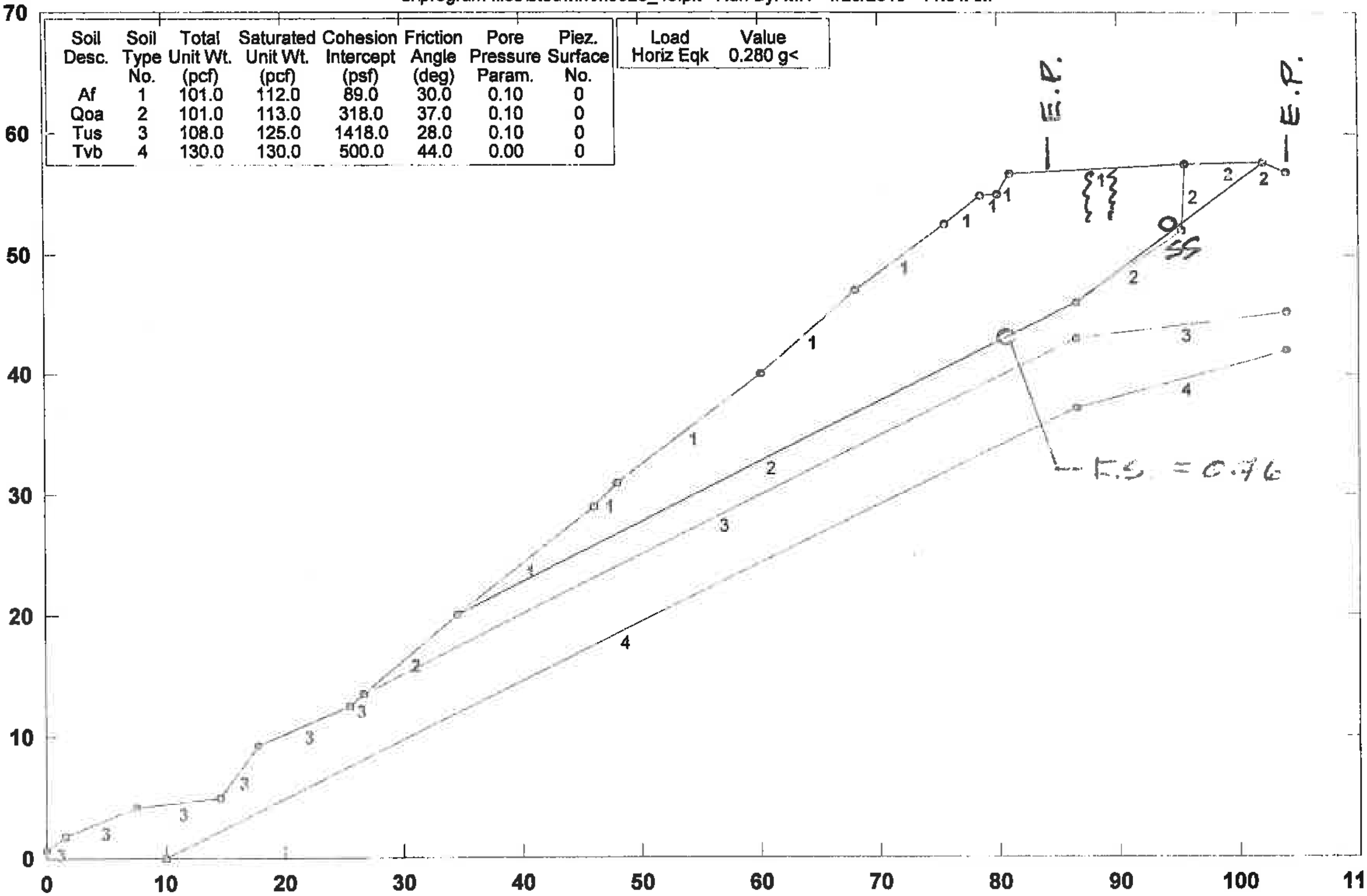
Figure B-6.

M9928-Second Avenue, Carmel Section 4-4, SEISMIC,planar

c:\program files\stedwin\m9928_45.plt Run By: MH 1/29/2010 11:51AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0

Load	Value
Horiz Eqk	0.280 g<



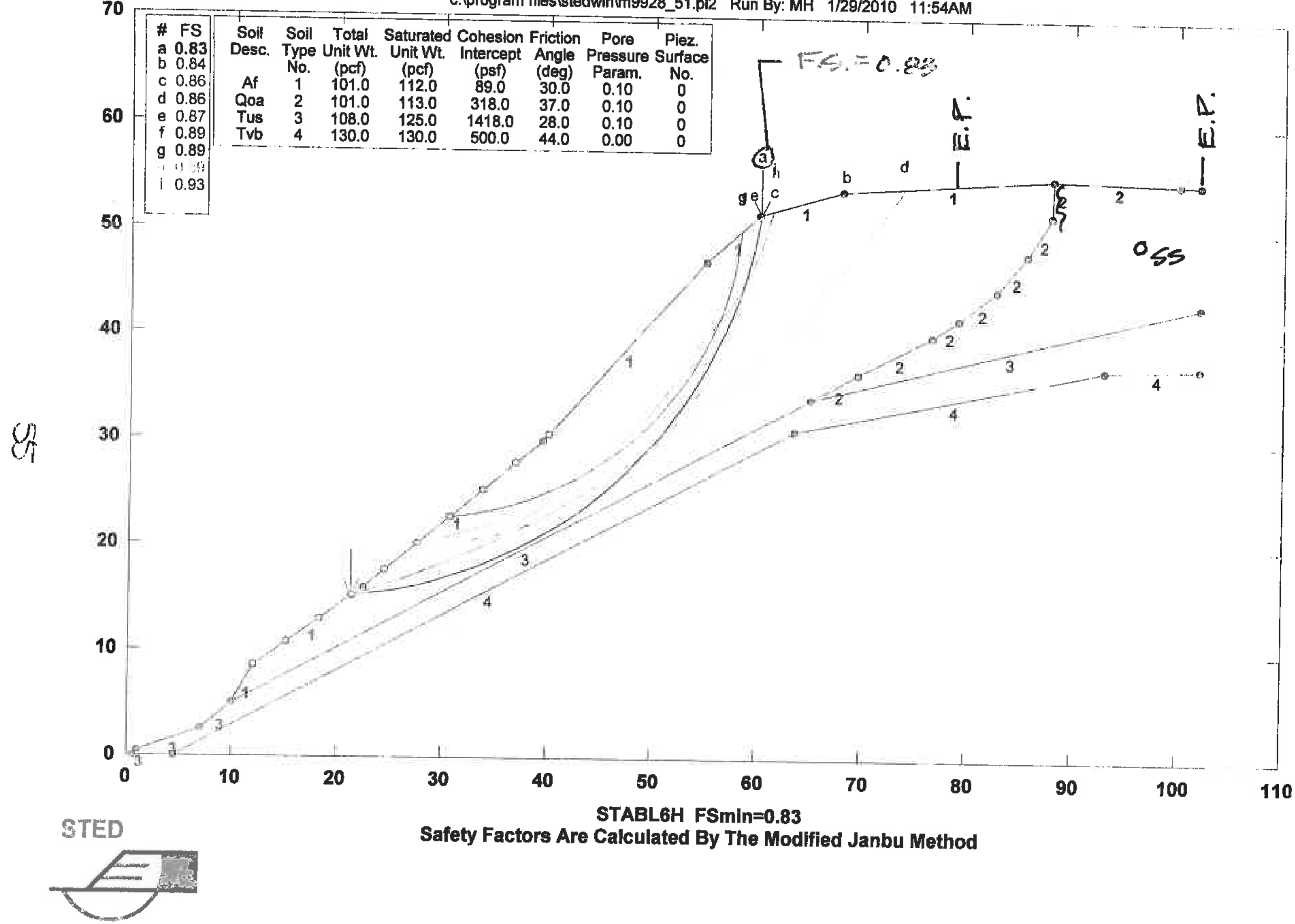
STABL6H FSmin=0.76
Factors of Safety Calculated by Janbu Method

STED

Figure B-7.

M9928-Second Avenue, Carmel Section 5-5, STATIC,very outboard

c:\program files\stedwin\m9928_51.pl2 Run By: MH 1/29/2010 11:54AM

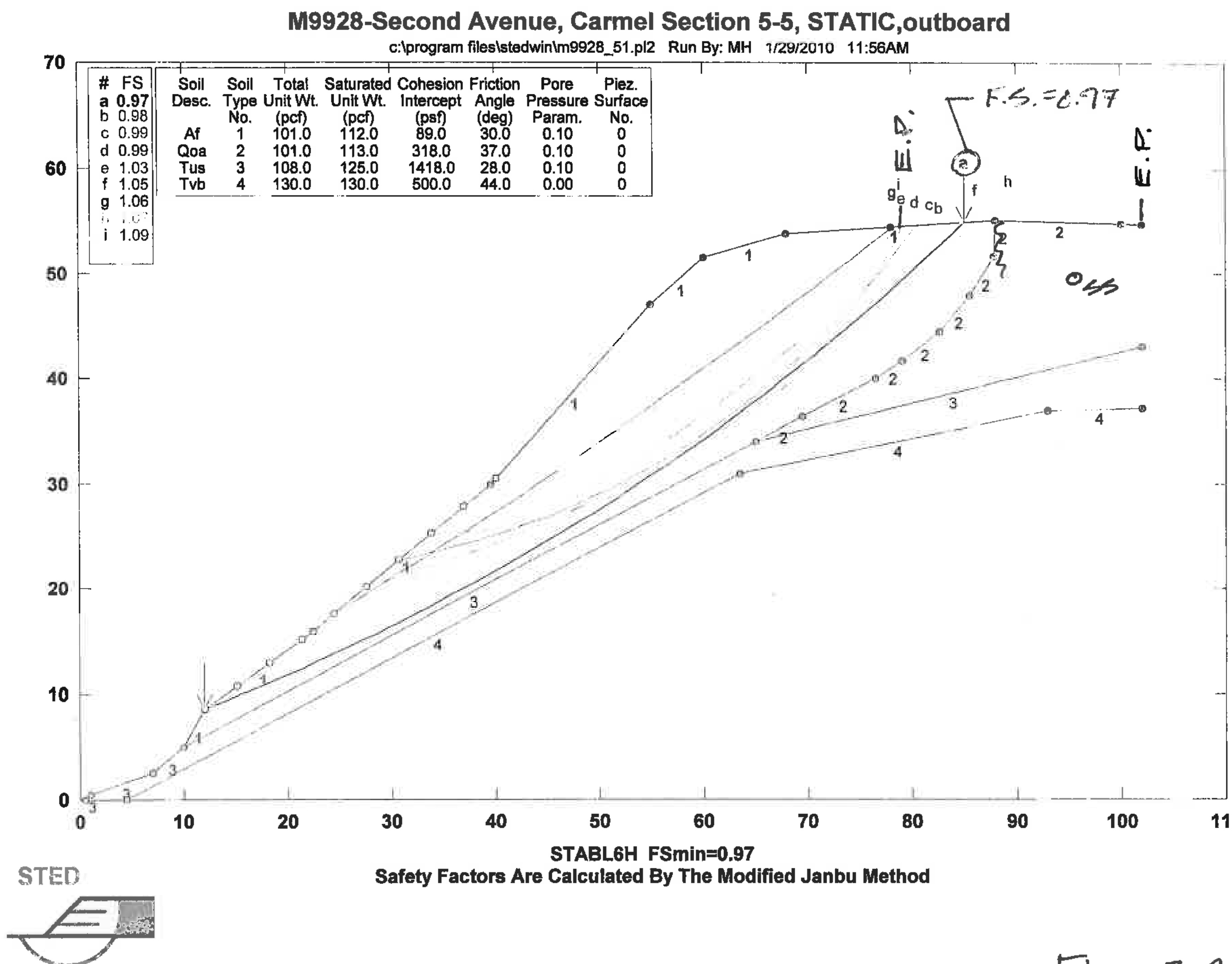


STABL6H FSmin=0.83

Safety Factors Are Calculated By The Modified Janbu Method

Figure B-8.

55

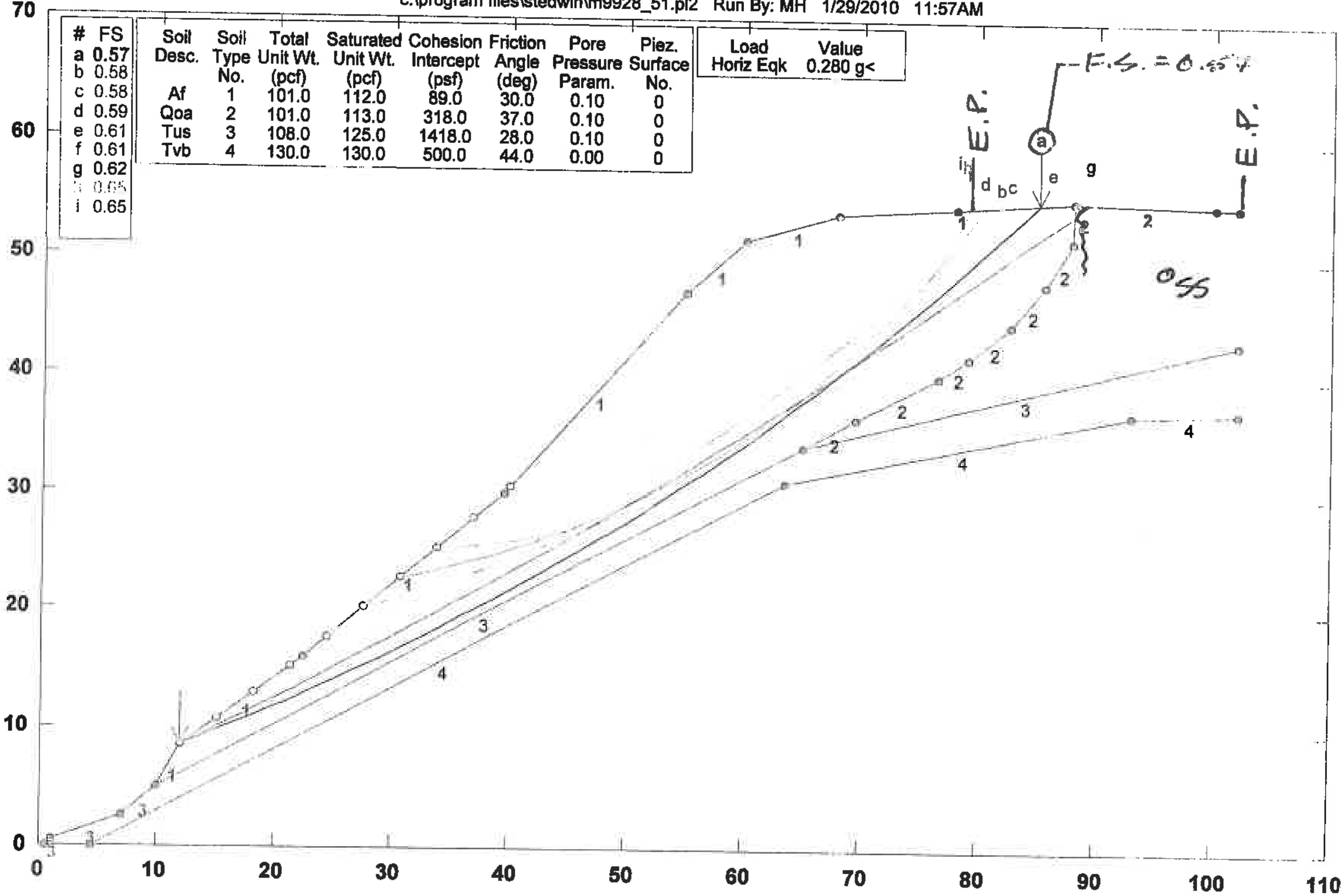


M9928-Second Avenue, Carmel Section 5-5, STATIC,outboard
c:\program files\stedwin\m9928_51.pl2 Run By: MH 1/29/2010 11:56AM
FS
a 0.97
b 0.98
c 0.99
d 0.99
e 1.03
f 1.05
g 1.06
i 1.09
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Piez. Surface No.
Af 1 101.0 112.0 89.0 30.0 0.10 0
Qoa 2 101.0 113.0 318.0 37.0 0.10 0
Tus 3 108.0 125.0 1418.0 28.0 0.10 0
Tvb 4 130.0 130.0 500.0 44.0 0.00 0
F.S.=0.97
E.P.
E.P.
STABL6H FSmin=0.97
Safety Factors Are Calculated By The Modified Janbu Method
STED

Figure B-9.

M9928-Second Avenue, Carmel Section 5-5, SEISMIC,outboard

c:\program files\stedwin\m9928_51.pl2 Run By: MH 1/29/2010 11:57AM



#	FS
a	0.57
b	0.58
c	0.58
d	0.59
e	0.61
f	0.61
g	0.62
h	0.65
i	0.65

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0

Load	Value
Horiz Eqk	0.280 g<

STABL6H FSmin=0.57

Safety Factors Are Calculated By The Modified Janbu Method

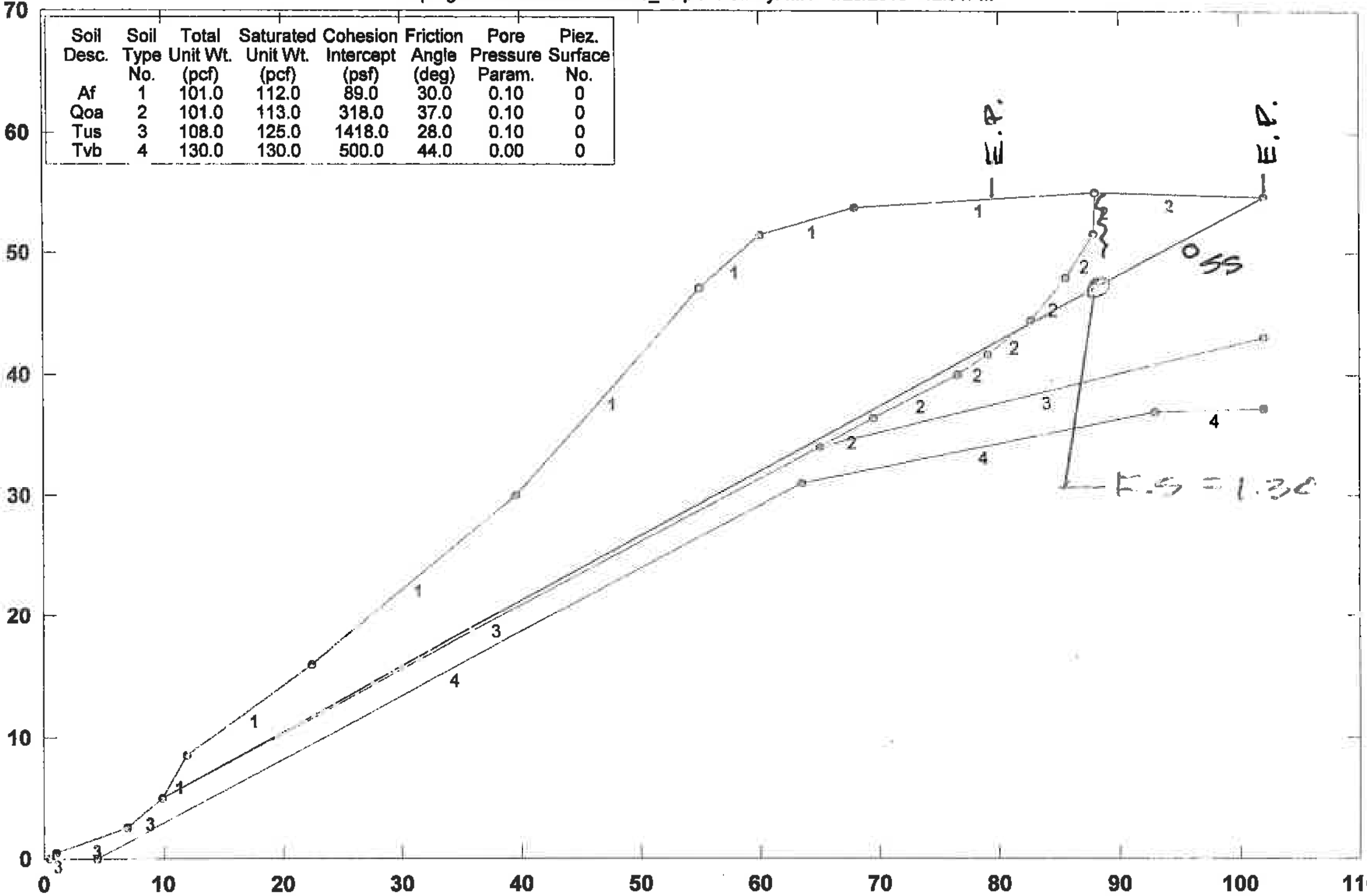
STED

Figure B-10.

M9928-Second Avenue, Carmel Section 5-5,STATIC, planar

c:\program files\stedwin\m9928_53.plt Run By: MH 1/29/2010 12:01PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0



STABL6H FSmin=1.30

Factor Of Safety Is Calculated By The Modified Bishop Method

STED

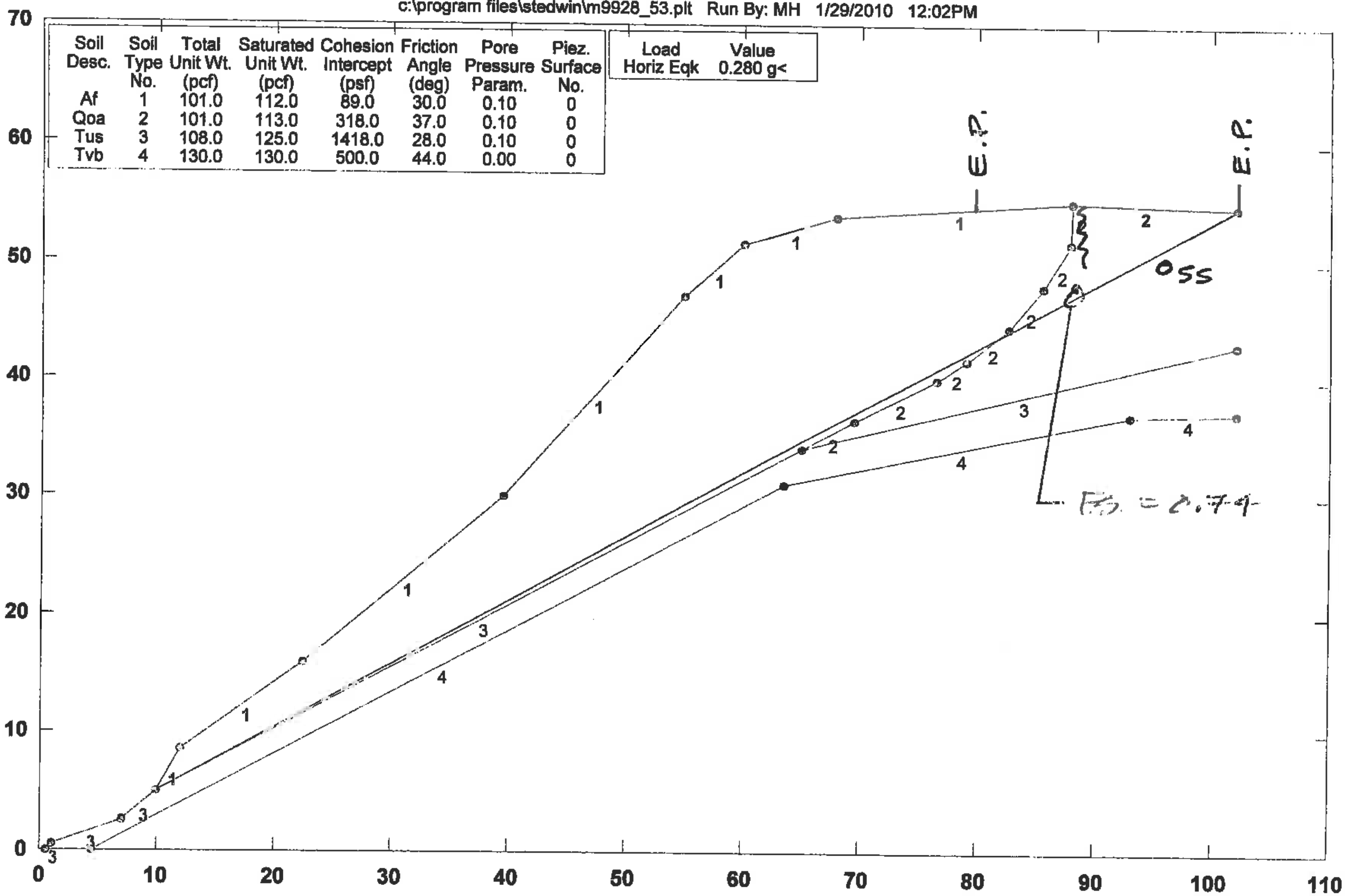
Figure B-11

M9928-Second Avenue, Carmel Section 5-5,SEISMIC, planar

c:\program files\stedwin\m9928_53.plt Run By: MH 1/29/2010 12:02PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0

Load	Value
Horiz Eqk	0.280 g<



STABL6H FSmin=0.74
Factor Of Safety Is Calculated By The Modified Bishop Method

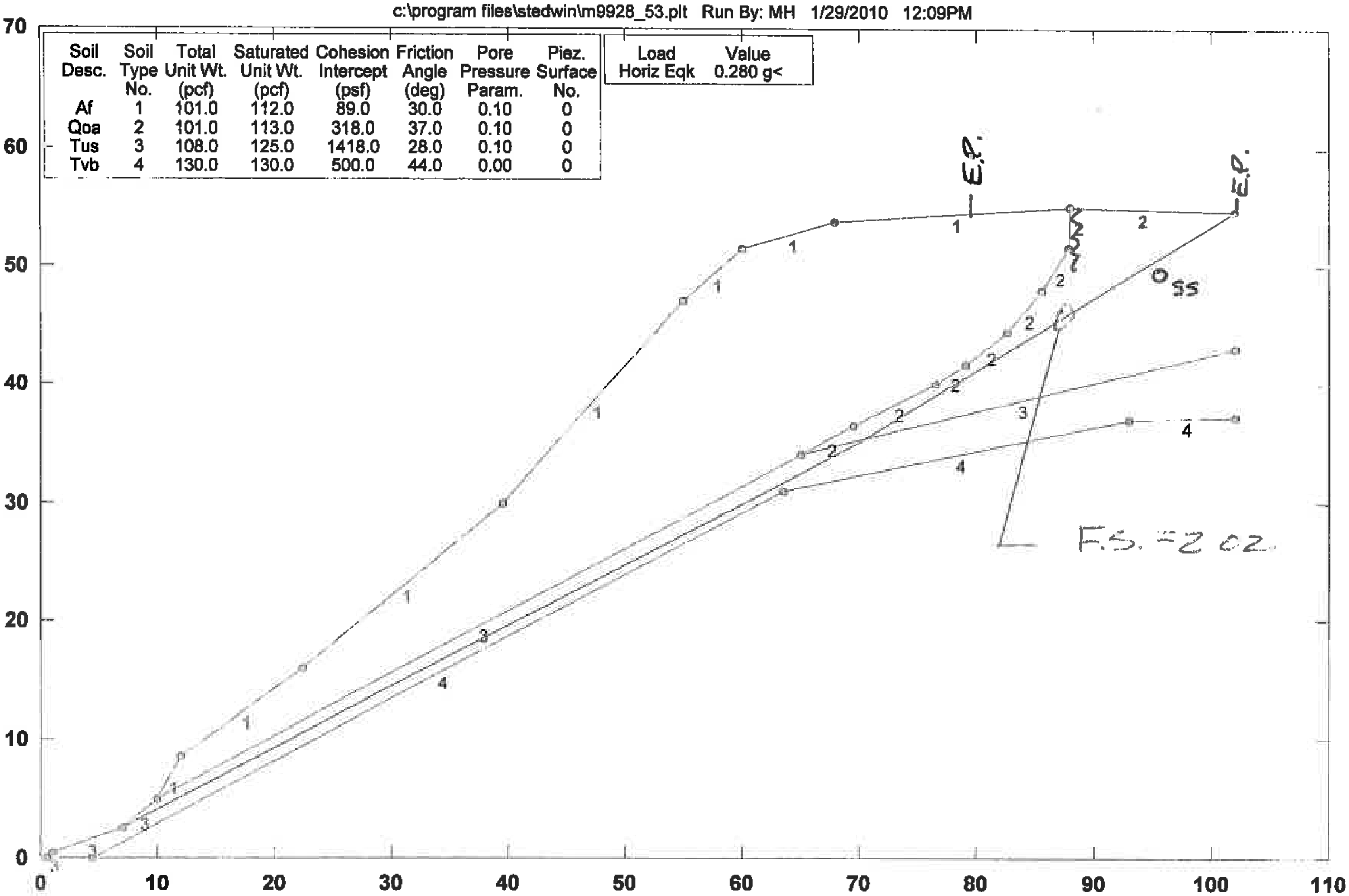
STED

59

Figure B-12.

M9928-Second Avenue, Carmel Section 5-5,SEISMIC,planar,thru Soil 2

c:\program files\stedwin\m9928_53.plt Run By: MH 1/29/2010 12:09PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Af	1	101.0	112.0	89.0	30.0	0.10	0
Qoa	2	101.0	113.0	318.0	37.0	0.10	0
Tus	3	108.0	125.0	1418.0	28.0	0.10	0
Tvb	4	130.0	130.0	500.0	44.0	0.00	0

Load	Value
Horiz Eqk	0.280 g<

STABL6H FSmin=2.02

Factor Of Safety Is Calculated By The Modified Bishop Method

STED

60

Figure B-13.