TerraCosta Consulting Group

*Geotechnical Engineering*
*Coastal Engineering*
*Maritime Engineering*

Project No. 2918
December 19, 2016

Mr. David M. Lessnick
**D. MARIN DEVELOPMENT**
1900 Western Avenue
Las Vegas, Nevada 89102

RESPONSE TO CITY REVIEW COMMENTS
CYCLE TYPE 6 LDR-GEOLOGY
**5228 CHELSEA STREET**
LA JOLLA, CALIFORNIA

CITY OF SAN DIEGO PROJECT NO. 502954
REFERENCE L64A-0038

Dear Mr. Lessnick:

TerraCosta Consulting Group, Inc. (TerraCosta) is responding to the City of San Diego's review comments from Cycle 6 LDR-Geology dated November 18, 2016. For completeness of the record, we have restated the original comments in italics, followed by our response. Only those items requiring responses have been included.

**LDR – GEOLOGY (Jacobe Washburn)**

*Issue No. 20: As previously requested, update the geologic map (Figure 2) to clearly delineate the location of geologic cross sections and plot the geologic structure.*

**Response:** The Geologic Map, specifically Figure 2 of our July 19, 2016, report, has been revised to delineate the location of the two additional geologic cross sections requested in Issue No. 21 and is provided with this response letter.

*Issue No. 21: Provide two additional geologic cross sections. The cross sections should be parallel to cross section 1 in the report dated July 29, 2016 with one cross section passing through Boring 1 and the other passing through Boring 2. Show the distribution of fill, geologic units, and the geologic structure.*

**Response:** The two additional requested geologic cross sections are included as Figures 4 and 5, with the locations of the two additional cross sections shown on the revised

Figure 2 Geologic Map. For completeness of the record, we have also included our existing cross section as Figure 3.

*Issue No. 22: The adjacent property southeast of the site has an accepted coastal bluff edge located at an approximate elevation of 70 feet (MSL) (PTS #125091) based on subsurface exploration. In the current study, the geotechnical consultant has indicated the coastal bluff is at an approximate elevation of 64 feet (MSL). There appears to be a discrepancy of 6 feet elevation at the property line of the two sites. The geotechnical consultant must demonstrate the location of the coastal bluff edge based on additional cross sections and, if necessary, additional subsurface explorations.*

**Response:** While we appreciate that the adjacent property southeast of the site has an accepted coastal bluff edge, we have reviewed in considerable detail Christian Wheeler Engineering's (CWE) May 26, 2016, report, and previously transmitted a copy of CWE's report to Coastal Commission Staff on November 28, 2016, in response to separate questions asked by Dr. Mark Johnsson regarding soil strengths typical of the Mount Soledad Formation. Although CWE excavated 16 test pits to delineate the top-of-bluff, nowhere in their report do they conclude that the coastal bluff edge is located at an approximate elevation of 70 feet. Moreover, we take exception with some of CWE's findings and note that the top-of-bluff delineated on Plate Nos. 3, 4, and 5 of their May 2016 report is not consistent with the City of San Diego's Coastal Bluffs and Beaches Guidelines definition of coastal bluff edge. More to the point, most of CWE's test pits were excavated to delineate the obscured configuration and extent of a bluff-top gully that at one time extended into the adjacent property that was since filled in, and in part stabilized by a brick wall that extended about 3 feet onto 5228 Chelsea Street and previously noted in our site plan, although never mentioned in CWE's report.

As indicated in our response to Issue No. 25, for completeness of the record, we have reproduced Photos 1 and 2 from our October 17, 2016, Response to City Review Comments letter. Photo 3 (attached) provides a low-altitude photograph of both parcels, taken from a drone, clearly showing the brick wall with fill soils extending both north and south onto both adjacent parcels, as noted in both CWE's and TerraCosta's previous reports. Photo 3 is also noteworthy in that, with upwards of 5 feet of fill soils existing near the face of the coastal bluff, even the existing rear yard fence steps down about 3 feet to the north, in part reflecting this previous bluff-top fill that extended out to the face of the coastal bluff on both properties. Photo 4 provides a close-up of the brick wall on 5220 Chelsea Street looking south and showing the bluff-top fill on that property.

In order to provide context between the geotechnical mapping on the adjacent parcel to the south (5220 Chelsea Street) and the subject property (5228 Chelsea Street), we have digitally stitched the two site plans together, as shown on Figure 6. For clarity, and since our mapped top-of-bluff and 25-foot setback were both shown in red, we also highlighted CWE's top-of-bluff and 25-foot setback in red. We added to that graphic our slightly revised top-of-bluff at the southerly corner of 5228 Chelsea Street, shown in green, along with the resulting changes to the 25-foot setback also shown in green. We have also accepted CWE's top-of-bluff and connected our estimated top-of-bluff with theirs.

Please note that, as shown on Figure 6, when constructing a 25-foot setback, the actual setback is determined by the most landward edge of a series of concentric 25-foot radius circles drawn along the entire top-of-bluff line; something that is easily performed with the aid of a computer. Notably, CWE's northerly 10+ feet of their 25-foot setback is in error. Note also that the coastal terrace slopes down to the north at about 0.9 percent, resulting in a slight reduction in at least bluff-top elevations to the north.

*Issue No. 23: As previously requested, submit the computer input parameters and output calculations for the slope stability analyses.*

**Response:** Input and output files for our stability analyses are provided in Attachment A.

*Issue No. 24: The project's geotechnical consultant has indicated that soil strength values were obtained from a site located at 1264 Neptune in Encinitas, California. The geotechnical consultant should provide shear strength parameters based on site specific sampling and testing. Alternatively, the consultant could conduct a parametric analysis to determine the back-calculated strengths of the material, assuming the current coastal bluff has a factor-of-safety of 1.0.*

**Response:** Coastal Commission Staff asked similar questions and we responded to their concerns in our letter dated November 28, 2016, a copy of which is provided in Attachment B (with the appended CWE report omitted). We trust that our response to Dr. Mark Johnsson will adequately address Issue No. 24.

*Issue No. 25: As previously requested, provide photo-quality copies of the historical aerial photographs used to determine 6½ feet of erosion/recession indicated on page 8 of the referenced report dated September 22, 2016. Clearly show where distances were measured on the historical aerial photographs.*

**Response:** As indicated in response to Issue No. 9 in our October 17, 2016, Response to City Review Comments letter, we reviewed U.S. Department of Agriculture stereographic aerial Photograph Nos. AXN-8M-92 and 93, and AXN-7M-186 and 187, flown May 1953. Copies of those four photos are provided in Attachment C. Admittedly, the printouts are not of the same quality as the original USDA photographs.

Also as indicated in our October 17, 2016, Response to Review Comments letter, we used nine images from the California Coastal Records Project (Coastal Records), all of which are copyrighted and, as such, we do not reproduce copies for inclusion in reports, as required by the website's copyright. It should be noted, however, that the Coastal Records website acknowledges the value of their photographs and does authorize the use of reproduction for internal use, which we have done for this project. If the City reviewer wishes to do so, we would encourage them to download the nine previously referenced Coastal Records photographs used in our evaluation of bluff-top retreat. As requested, the last two photographs previously referenced have again been reproduced as Photos 1 and 2 and attached to this letter. Photo 5 provides a current ground shot of the sea stack shown in the 1992 aerial photograph and the location where we measured 6½ feet of erosion between 1987 and September 2016.

*Issue No. 26: The project's geotechnical consultant should clarify if the project will be impacted by coastal bluff erosion as previously requested in Issue No. 12.*

**Response:** It is our opinion that the project will not be impacted by coastal bluff erosion throughout the economic lifespan (next 75 years) of the proposed development.

We trust these responses satisfactorily address the reviewer's comments. If you have any questions, please give us a call.

Very truly yours,

TERRACOSTA CONSULTING GROUP, INC.

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

Gregory A. Spaulding, Project Geologist
C.E.G. 1863, C.H.G. 351, R.G. 5892

WFC/GAS/jg
Attachments

cc: Jacobe Washburn, City of San Diego
Jim Quinn, City of San Diego


**PHOTO 1:** This 1992 photo is the first to show the sea stack, which formed sometime between 1987 and 1992 (see Image 8701192 at www.californiacoastline.org).

TerraCosta Consulting Group

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**PROJECT NO.:**
**2918**

**PHOTO NO.:**
**1**

## 5228 CHELSEA STREET, LA JOLLA


**PHOTO 2:** Sea stack observed in Photo 1 was measured 6-1/2 feet from the front face of the sea stack to the face of the bluff.

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**PROJECT NO.:**
**2918**

**PHOTO NO.:**
**2**

## 5228 CHELSEA STREET, LA JOLLA

**PHOTO 3:** Low-altitude photo of both parcels showing the slope-top brick wall on 5220 Chelsea Street and the bluff-top fill pad that extends out to the face of the bluff.

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**PROJECT NO.:**
**2918**

**PHOTO NO.:**
**3**

## 5228 CHELSEA STREET, LA JOLLA

**PHOTO 4:** Close-up of the brick wall on 5220 Chelsea Street showing the bluff-top fill to the south.

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**PROJECT NO.:**
**2918**

**PHOTO NO.:**
**4**

## 5228 CHELSEA STREET, LA JOLLA

**PHOTO 5:** Close-up of the measured 6½ feet of erosion that occurred between 1987 and 2016

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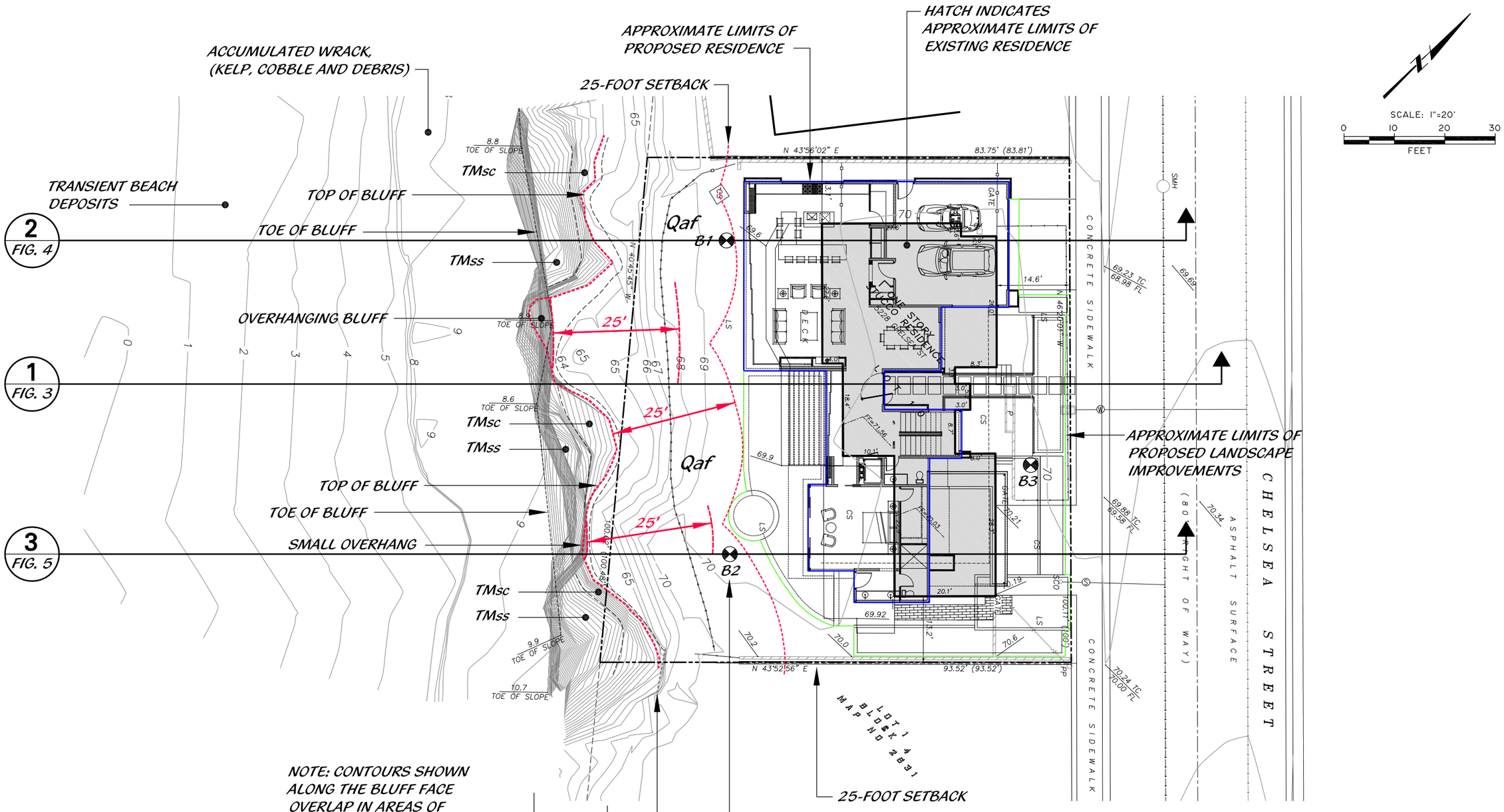
**PROJECT NO.:**
**2918**

**PHOTO NO.:**
**5**

## 5228 CHELSEA STREET, LA JOLLA




DESCRIPTION

| |
|---|
| Qaf - FILL |
| TMsc - MOUNT SOLEDAD FORMATION (CONGLOMERATE) |
| TMss - MOUNT SOLEDAD FORMATION (SANDSTONE) |

## SITE PLAN

SCALE: 1"=20'

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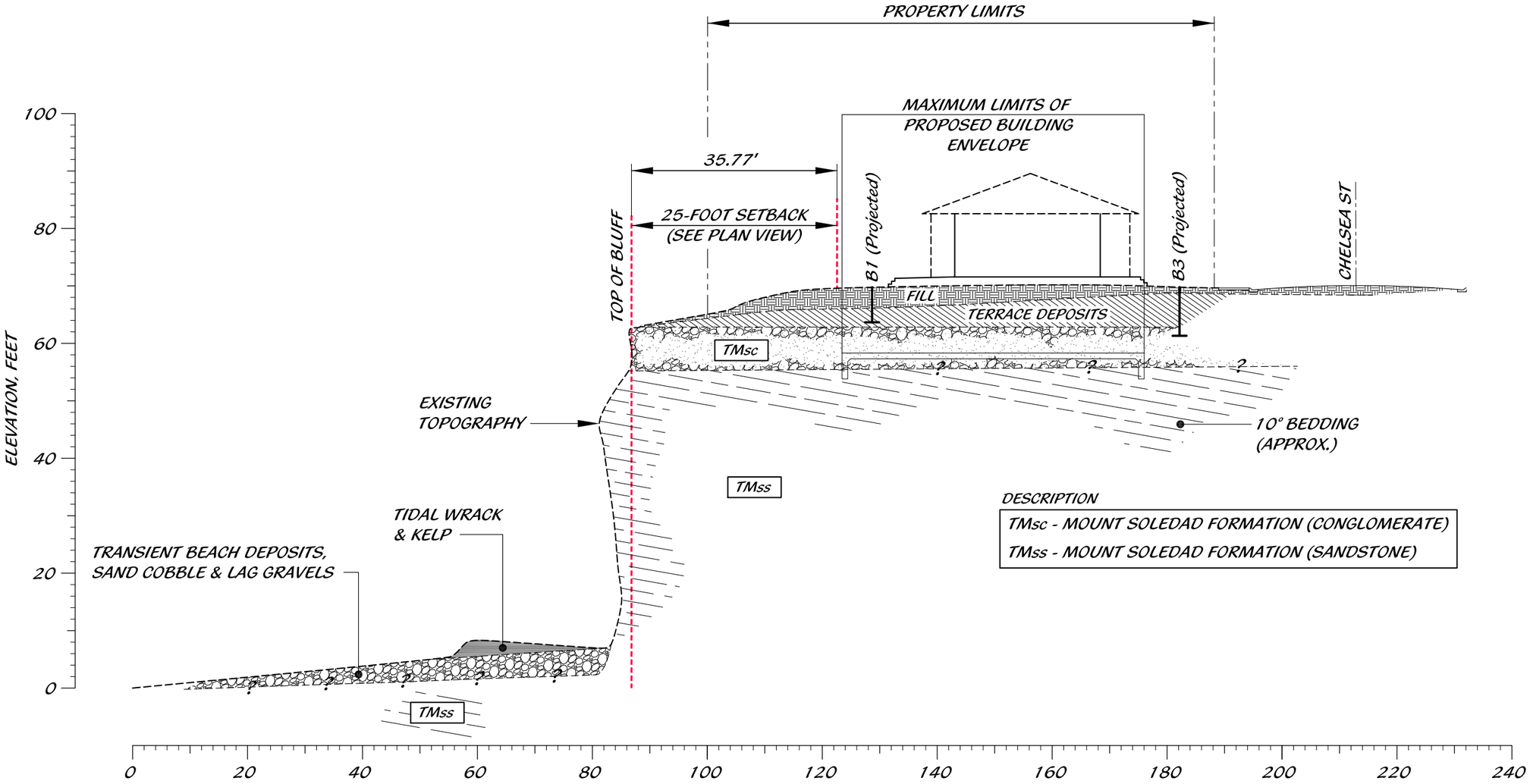
ENGINEERS AND GEOLOGISTS
3890 MURPHY CANYON ROAD, SUITE 200
SAN DIEGO, CA 92123 (858) 573-6900


| FIGURE NUMBER | PROJECT NUMBER |
|---|---|
| 2 | 2918 |

PROJECT NAME: 5228 CHELSEA STREET

# SITE PLAN




DESCRIPTION

TMsc - MOUNT SOLEDAD FORMATION (CONGLOMERATE)

TMss - MOUNT SOLEDAD FORMATION (SANDSTONE)

## EXISTING & PROPOSED CROSS SECTION

SCALE: 1"=20' (HORIZ. & VERT.)

1

---

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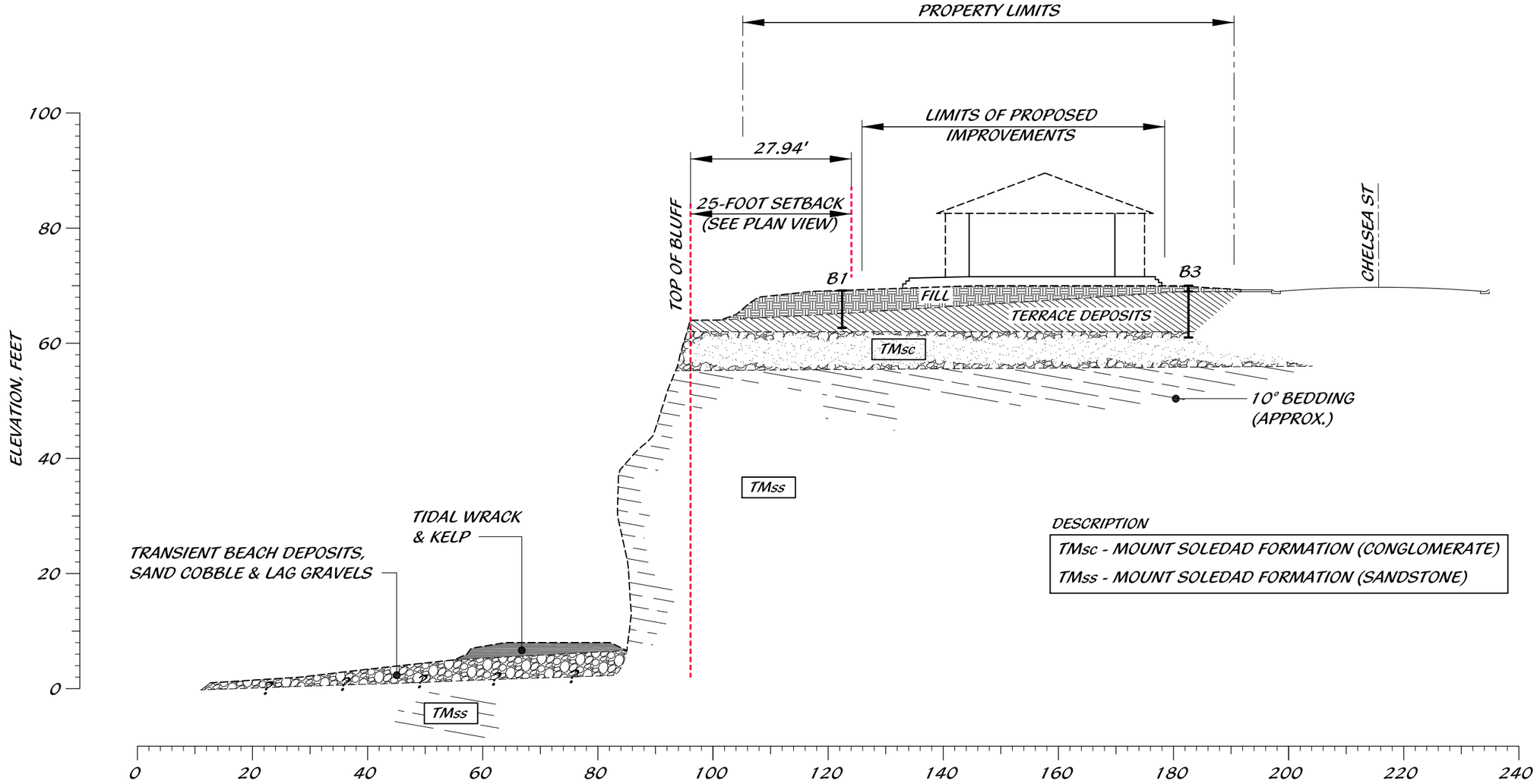
FIGURE NUMBER 3

PROJECT NAME: 5228 CHELSEA STREET

PROJECT NUMBER: 2918

# CROSS SECTION




## EXISTING & PROPOSED CROSS SECTION

SCALE: 1"=20' (HORIZ. & VERT.)

2
---

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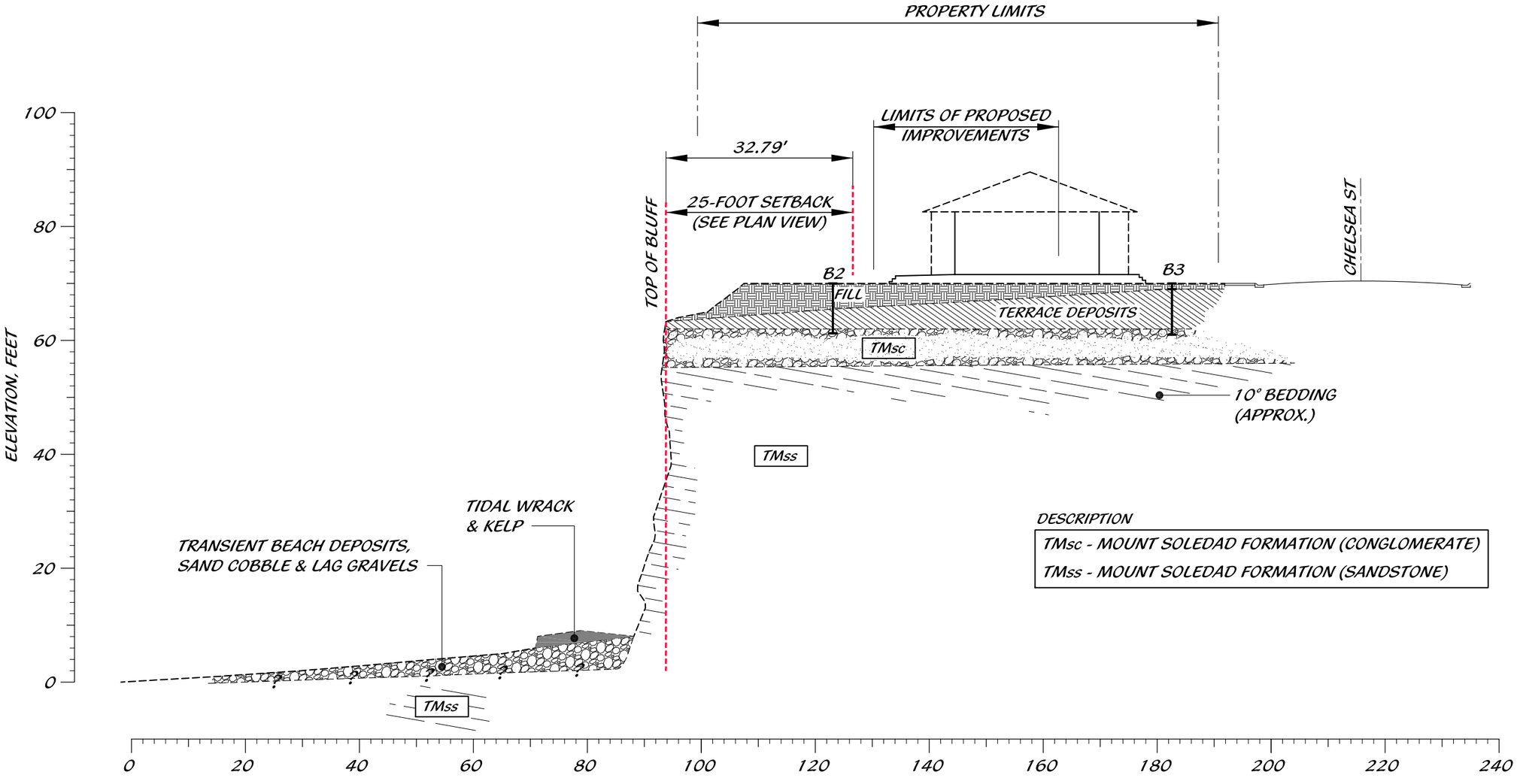
FIGURE NUMBER
4

PROJECT NAME
5228 CHELSEA STREET

PROJECT NUMBER
2918

# CROSS SECTION




DESCRIPTION

TMsc - MOUNT SOLEDAD FORMATION (CONGLOMERATE)

TMss - MOUNT SOLEDAD FORMATION (SANDSTONE)

## EXISTING & PROPOSED CROSS SECTION

SCALE: 1"=20' (HORIZ. & VERT.)

3

---

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FIGURE NUMBER 5

PROJECT NAME: 5228 CHELSEA STREET

PROJECT NUMBER: 2918

# CROSS SECTION

SCALE: 1"=20'
0 10 20 30
FEET

(80' RIGHT OF WAY)

APPROXIMATE LIMITS OF PROPOSED LANDSCAPE IMPROVEMENTS

HATCH INDICATES APPROXIMATE LIMITS OF EXISTING RESIDENCE

APPROXIMATE LIMITS OF PROPOSED RESIDENCE

25-FOOT SETBACK

25-FOOT SETBACK

EXISTING BURIED BRICK WALL

1.5 STATIC F.O.S.
40' SETBACK
1.1 PSEUDO STATIC F.O.S.
25' SETBACK
DAYLIGHT LINE OF PROJECTED 75-YEAR BLUFF TOP EROSION
TOP OF BLUFF

ACCUMULATED WRACK, (KELP, COBBLE AND DEBRIS)

TOP OF BLUFF

TOE OF BLUFF

OVERHANGING BLUFF

TOP OF BLUFF

TOE OF BLUFF

SMALL OVERHANG

TRANSIENT BEACH DEPOSITS

NOTE: CONTOURS SHOWN ALONG THE BLUFF FACE OVERLAP IN AREAS OF OVERHANGS

DESCRIPTION

Qaf - FILL
TMsc - MOUNT SOLEDAD FORMATION (CONGLOMERATE)
TMss - MOUNT SOLEDAD FORMATION (SANDSTONE)

| | |
|---|---|
| TERRACOSTA CONSULTING GROUP<br>ENGINEERS AND GEOLOGISTS<br>3890 MURPHY CANYON ROAD, SUITE 200<br>SAN DIEGO, CA 92123 (858) 573-6900 | FIGURE NUMBER<br>6 |
| PROJECT NAME<br>5228 CHELSEA STREET | PROJECT NUMBER<br>2918 |

# SITE PLAN

# ATTACHMENT A

# STABILITY ANALYSES
# INPUT AND OUTPUT FILES

TerraCosta
Consulting Group

```
PROFIL  C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section -
earthquake-lower.in Version G7v2.005.2 [GSTABL72.EXE]
e
PR NO. 2918 - 5228 Chelsea Street       Section 1 - earthquake -lower
15 9
0. 20. 83. 27. 4
83. 27. 85. 33. 5
85. 33. 85.5 72. 5
85.5 72. 86. 75. 5
86. 75. 86.5 83. 3
86.5 83. 104. 86. 2
104. 86. 109. 88. 1
109. 88. 120. 90. 1
120. 90. 240. 90. 1
104. 86. 174. 88. 2
174. 88. 240. 88. 2
86.5 83. 240. 82. 3
86. 75. 240. 76. 5
0. 20. 81. 22. 5
81. 22. 83. 27. 5
0. 0. 0.
SOIL  Fill    Terrace TMsc    Cobble  TMss
5
120. 120. 100. 30. 0. 0. 0
120. 120. 300. 33. 0. 0. 0
120. 120. 1000. 33. 0. 0. 0
125. 125. 0. 40. 0. 0. 0
120. 120. 3500. 35. 0. 0. 0
LOADS
5
132. 140. 150. 0.
140.1 144. 2000. 0.
144.1 168. 500. 0.
168.1 172. 2000. 0.
172.1 176. 150. 0.
EQUAKE
0.15 0.15 0.0 0. 1
CIRCL2
100 100
83.1 84.9 100. 200.
0. 5. 65. 5.
```

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2A, July 2012 **



**********
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
**********

Analysis Run Date: 7/17/2016
Time of Run: 02:09PM
Run By: Username
Input Data Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - earthquake-lower.in
Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - earthquake-lower.OUT
Unit System: English

Plotted Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - earthquake-lower.PLT

PROBLEM DESCRIPTION: PR NO. 2918 - 5228 Chelsea Street
Section 1 - earthquake -lower

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | 0.00 | 20.00 | 83.00 | 27.00 | 4 |
| 2 | 83.00 | 27.00 | 85.00 | 33.00 | 5 |
| 3 | 85.00 | 33.00 | 85.50 | 72.00 | 5 |
| 4 | 85.50 | 72.00 | 86.00 | 75.00 | 5 |
| 5 | 86.00 | 75.00 | 86.50 | 83.00 | 3 |
| 6 | 86.50 | 83.00 | 104.00 | 86.00 | 2 |
| 7 | 104.00 | 86.00 | 109.00 | 88.00 | 1 |
| 8 | 109.00 | 88.00 | 120.00 | 90.00 | 1 |

| | | | | | |
|---|---|---|---|---|---|
| 9 | 120.00 | 90.00 | 240.00 | 90.00 | 1 |
| 10 | 104.00 | 86.00 | 174.00 | 88.00 | 2 |
| 11 | 174.00 | 88.00 | 240.00 | 88.00 | 2 |
| 12 | 86.50 | 83.00 | 240.00 | 82.00 | 3 |
| 13 | 86.00 | 75.00 | 240.00 | 76.00 | 5 |
| 14 | 0.00 | 20.00 | 81.00 | 22.00 | 5 |
| 15 | 81.00 | 22.00 | 83.00 | 27.00 | 5 |

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| 5 | 120.0 | 120.0 | 3500.0 | 35.0 | 0.00 | 0.0 | 0 |

1

BOUNDARY LOAD(S)

5 Load(s) Specified

| Load No. | X-Left (ft) | X-Right (ft) | Intensity (psf) | Deflection (deg) |
|---|---|---|---|---|
| 1 | 132.00 | 140.00 | 150.0 | 0.0 |
| 2 | 140.10 | 144.00 | 2000.0 | 0.0 |
| 3 | 144.10 | 168.00 | 500.0 | 0.0 |
| 4 | 168.10 | 172.00 | 2000.0 | 0.0 |
| 5 | 172.10 | 176.00 | 150.0 | 0.0 |

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.150(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

1

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

10000 Trial Surfaces Have Been Generated.

100 Surface(s) Initiate(s) From Each Of 100 Points Equally Spaced
Along The Ground Surface Between X = 83.10(ft)
and X = 84.90(ft)

Each Surface Terminates Between X = 100.00(ft)
and X = 200.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

5.00(ft) Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of 5.0
And 65.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 0

Number of Trial Surfaces With Valid FS = 0

Statistical Data On All Valid FS Values:
FS Max = 0.000 FS Min = 500.000 FS Ave = NaN
Standard Deviation = 0.000 Coefficient of Variation = NaN
%

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.209 | 27.627 |
| 2 | 86.839 | 31.066 |
| 3 | 90.454 | 34.520 |
| 4 | 94.055 | 37.989 |
| 5 | 97.640 | 41.475 |
| 6 | 101.209 | 44.976 |
| 7 | 104.764 | 48.492 |
| 8 | 108.303 | 52.024 |
| 9 | 111.827 | 55.571 |
| 10 | 115.336 | 59.133 |
| 11 | 118.829 | 62.711 |

| | | |
|---|---|---|
| 12 | 122.306 | 66.303 |
| 13 | 125.768 | 69.911 |
| 14 | 129.214 | 73.534 |
| 15 | 132.644 | 77.172 |
| 16 | 136.059 | 80.824 |
| 17 | 139.457 | 84.492 |
| 18 | 142.840 | 88.174 |
| 19 | 144.503 | 90.000 |

Circle Center At X = -706.598 ; Y = 865.127 ; and Radius = 1151.173

Factor of Safety
*** 1.963 ***

Individual data on the 0 slices

| Slice No. | Width (ft) | Weight (lbs) | Water Force Top (lbs) | Water Force Bot (lbs) | Tie Force Norm (lbs) | Tie Force Tan (lbs) | Earthquake Force Hor (lbs) | Earthquake Force Ver (lbs) | Surcharge Load (lbs) |
|---|---|---|---|---|---|---|---|---|---|

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.282 | 27.845 |
| 2 | 86.879 | 31.318 |
| 3 | 90.466 | 34.802 |
| 4 | 94.041 | 38.297 |
| 5 | 97.605 | 41.804 |
| 6 | 101.157 | 45.322 |
| 7 | 104.699 | 48.852 |
| 8 | 108.229 | 52.393 |
| 9 | 111.748 | 55.945 |
| 10 | 115.255 | 59.509 |
| 11 | 118.751 | 63.084 |
| 12 | 122.235 | 66.670 |
| 13 | 125.708 | 70.267 |
| 14 | 129.170 | 73.875 |
| 15 | 132.619 | 77.494 |
| 16 | 136.058 | 81.124 |
| 17 | 139.484 | 84.766 |
| 18 | 142.899 | 88.418 |
| 19 | 144.369 | 90.000 |

Circle Center At X = -998.960 ; Y = 1152.647 ; and Radius = 1560.906

Factor of Safety
*** 1.966 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.373 | 28.118 |
| 2 | 87.131 | 31.416 |
| 3 | 90.859 | 34.748 |
| 4 | 94.558 | 38.112 |
| 5 | 98.227 | 41.508 |
| 6 | 101.866 | 44.937 |
| 7 | 105.475 | 48.398 |
| 8 | 109.053 | 51.891 |
| 9 | 112.600 | 55.415 |
| 10 | 116.115 | 58.970 |
| 11 | 119.599 | 62.557 |
| 12 | 123.052 | 66.174 |
| 13 | 126.472 | 69.821 |
| 14 | 129.860 | 73.498 |
| 15 | 133.215 | 77.205 |
| 16 | 136.537 | 80.942 |
| 17 | 139.826 | 84.708 |
| 18 | 143.082 | 88.502 |
| 19 | 144.344 | 90.000 |

Circle Center At X = -288.122 ; Y = 455.176 ; and Radius = 566.027

Factor of Safety
*** 1.967 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.118 | 27.355 |
| 2 | 86.809 | 30.727 |
| 3 | 90.488 | 34.114 |
| 4 | 94.153 | 37.514 |
| 5 | 97.807 | 40.928 |
| 6 | 101.447 | 44.356 |
| 7 | 105.074 | 47.797 |
| 8 | 108.689 | 51.252 |
| 9 | 112.291 | 54.720 |
| 10 | 115.879 | 58.201 |
| 11 | 119.455 | 61.696 |
| 12 | 123.018 | 65.204 |
| 13 | 126.567 | 68.726 |
| 14 | 130.103 | 72.261 |
| 15 | 133.627 | 75.809 |
| 16 | 137.136 | 79.370 |
| 17 | 140.633 | 82.944 |
| 18 | 144.116 | 86.531 |
| 19 | 147.460 | 90.000 |

Circle Center At X = -818.755 ; Y = 1018.020 ; and Radius = 1339.699

Factor of Safety
*** 1.967 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.118 | 27.355 |
| 2 | 86.963 | 30.551 |
| 3 | 90.778 | 33.783 |
| 4 | 94.563 | 37.050 |
| 5 | 98.319 | 40.351 |
| 6 | 102.043 | 43.687 |
| 7 | 105.737 | 47.057 |
| 8 | 109.400 | 50.460 |
| 9 | 113.031 | 53.897 |
| 10 | 116.630 | 57.368 |
| 11 | 120.198 | 60.871 |
| 12 | 123.733 | 64.407 |
| 13 | 127.235 | 67.976 |
| 14 | 130.705 | 71.576 |
| 15 | 134.141 | 75.208 |
| 16 | 137.544 | 78.872 |
| 17 | 140.912 | 82.566 |
| 18 | 144.247 | 86.292 |
| 19 | 147.505 | 90.000 |

Circle Center At X = -262.667 ; Y = 447.168 ; and Radius = 543.885

Factor of Safety
*** 1.968 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.100 | 27.300 |
| 2 | 86.875 | 30.579 |
| 3 | 90.612 | 33.900 |
| 4 | 94.311 | 37.264 |
| 5 | 97.972 | 40.670 |
| 6 | 101.594 | 44.117 |
| 7 | 105.176 | 47.606 |
| 8 | 108.718 | 51.135 |
| 9 | 112.220 | 54.703 |
| 10 | 115.681 | 58.312 |
| 11 | 119.100 | 61.960 |

| | | |
|---|---|---|
| 12 | 122.478 | 65.646 |
| 13 | 125.814 | 69.371 |
| 14 | 129.107 | 73.133 |
| 15 | 132.357 | 76.933 |
| 16 | 135.563 | 80.770 |
| 17 | 138.726 | 84.642 |
| 18 | 141.844 | 88.551 |
| 19 | 142.974 | 90.000 |

Circle Center At X = -202.956 ; Y = 360.445 ; and Radius = 439.106

Factor of Safety
*** 1.968 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.191 | 27.573 |
| 2 | 86.822 | 31.010 |
| 3 | 90.446 | 34.455 |
| 4 | 94.062 | 37.908 |
| 5 | 97.670 | 41.370 |
| 6 | 101.270 | 44.840 |
| 7 | 104.862 | 48.318 |
| 8 | 108.446 | 51.804 |
| 9 | 112.022 | 55.298 |
| 10 | 115.590 | 58.801 |
| 11 | 119.150 | 62.312 |
| 12 | 122.702 | 65.831 |
| 13 | 126.247 | 69.358 |
| 14 | 129.783 | 72.893 |
| 15 | 133.310 | 76.436 |
| 16 | 136.830 | 79.987 |
| 17 | 140.342 | 83.546 |
| 18 | 143.845 | 87.114 |
| 19 | 146.667 | 90.000 |

Circle Center At X = -1419.740 ; Y = 1619.278 ; and Radius = 2189.138

Factor of Safety
*** 1.968 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.191 | 27.573 |
| 2 | 87.002 | 30.810 |

| | | |
|---|---|---|
| 3 | 90.786 | 34.078 |
| 4 | 94.543 | 37.377 |
| 5 | 98.272 | 40.707 |
| 6 | 101.974 | 44.068 |
| 7 | 105.649 | 47.459 |
| 8 | 109.295 | 50.880 |
| 9 | 112.913 | 54.331 |
| 10 | 116.502 | 57.812 |
| 11 | 120.063 | 61.322 |
| 12 | 123.595 | 64.862 |
| 13 | 127.097 | 68.430 |
| 14 | 130.570 | 72.028 |
| 15 | 134.013 | 75.653 |
| 16 | 137.426 | 79.307 |
| 17 | 140.809 | 82.989 |
| 18 | 144.161 | 86.699 |
| 19 | 147.096 | 90.000 |

Circle Center At X = -307.921 ; Y = 491.885 ; and Radius = 607.087

Factor of Safety
*** 1.968 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.355 | 28.064 |
| 2 | 87.059 | 31.422 |
| 3 | 90.744 | 34.801 |
| 4 | 94.410 | 38.201 |
| 5 | 98.057 | 41.622 |
| 6 | 101.683 | 45.064 |
| 7 | 105.290 | 48.527 |
| 8 | 108.877 | 52.011 |
| 9 | 112.443 | 55.515 |
| 10 | 115.990 | 59.039 |
| 11 | 119.516 | 62.584 |
| 12 | 123.022 | 66.149 |
| 13 | 126.508 | 69.734 |
| 14 | 129.973 | 73.338 |
| 15 | 133.417 | 76.963 |
| 16 | 136.840 | 80.608 |
| 17 | 140.242 | 84.272 |
| 18 | 143.623 | 87.955 |
| 19 | 145.479 | 90.000 |

Circle Center At X = -499.940 ; Y = 675.301 ; and Radius = 871.291

Factor of Safety
*** 1.968 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.245 | 27.736 |
| 2 | 86.901 | 31.148 |
| 3 | 90.534 | 34.583 |
| 4 | 94.144 | 38.042 |
| 5 | 97.732 | 41.525 |
| 6 | 101.296 | 45.031 |
| 7 | 104.838 | 48.561 |
| 8 | 108.357 | 52.113 |
| 9 | 111.852 | 55.688 |
| 10 | 115.324 | 59.286 |
| 11 | 118.772 | 62.907 |
| 12 | 122.196 | 66.551 |
| 13 | 125.597 | 70.216 |
| 14 | 128.973 | 73.904 |
| 15 | 132.325 | 77.614 |
| 16 | 135.653 | 81.346 |
| 17 | 138.956 | 85.099 |
| 18 | 142.235 | 88.874 |
| 19 | 143.200 | 90.000 |

Circle Center At X = -436.238 ; Y = 587.996 ; and Radius = 764.037

Factor of Safety
*** 1.969 ***

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

```
PROFIL  C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section -
earthquake-upper.in Version G7v2.005.2 [GSTABL72.EXE]
e
PR NO. 2918 - 5228 Chelsea Street       Section 1 - earthquake -upper
15 9
0. 20. 83. 27. 4
83. 27. 85. 33. 5
85. 33. 85.5 72. 5
85.5 72. 86. 75. 5
86. 75. 86.5 83. 3
86.5 83. 104. 86. 2
104. 86. 109. 88. 1
109. 88. 120. 90. 1
120. 90. 240. 90. 1
104. 86. 174. 88. 2
174. 88. 240. 88. 2
86.5 83. 240. 82. 3
86. 75. 240. 76. 5
0. 20. 81. 22. 5
81. 22. 83. 27. 5
0. 0. 0.
SOIL  Fill    Terrace TMsc    Cobble  TMss
5
120. 120. 100. 30. 0. 0. 0
120. 120. 300. 33. 0. 0. 0
120. 120. 1000. 33. 0. 0. 0
125. 125. 0. 40. 0. 0. 0
120. 120. 3500. 35. 0. 0. 0
LOADS
5
132. 140. 150. 0.
140.1 144. 2000. 0.
144.1 168. 500. 0.
168.1 172. 2000. 0.
172.1 176. 150. 0.
EQUAKE
0.15 0.15 0.0 0. 1
CIRCL2
50 50
85.02 85.49 100. 200.
0. 2. 60. 2.
```

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2A, July 2012 **



**********
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
**********

Analysis Run Date: 7/17/2016
Time of Run: 02:11PM
Run By: Username
Input Data Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - earthquake-upper.in
Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - earthquake-upper.OUT
Unit System: English

Plotted Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - earthquake-upper.PLT

PROBLEM DESCRIPTION: PR NO. 2918 - 5228 Chelsea Street
Section 1 - earthquake -upper

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | 0.00 | 20.00 | 83.00 | 27.00 | 4 |
| 2 | 83.00 | 27.00 | 85.00 | 33.00 | 5 |
| 3 | 85.00 | 33.00 | 85.50 | 72.00 | 5 |
| 4 | 85.50 | 72.00 | 86.00 | 75.00 | 5 |
| 5 | 86.00 | 75.00 | 86.50 | 83.00 | 3 |
| 6 | 86.50 | 83.00 | 104.00 | 86.00 | 2 |
| 7 | 104.00 | 86.00 | 109.00 | 88.00 | 1 |
| 8 | 109.00 | 88.00 | 120.00 | 90.00 | 1 |

| | | | | | |
|---|---|---|---|---|---|
| 9 | 120.00 | 90.00 | 240.00 | 90.00 | 1 |
| 10 | 104.00 | 86.00 | 174.00 | 88.00 | 2 |
| 11 | 174.00 | 88.00 | 240.00 | 88.00 | 2 |
| 12 | 86.50 | 83.00 | 240.00 | 82.00 | 3 |
| 13 | 86.00 | 75.00 | 240.00 | 76.00 | 5 |
| 14 | 0.00 | 20.00 | 81.00 | 22.00 | 5 |
| 15 | 81.00 | 22.00 | 83.00 | 27.00 | 5 |

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| 5 | 120.0 | 120.0 | 3500.0 | 35.0 | 0.00 | 0.0 | 0 |

1

BOUNDARY LOAD(S)

5 Load(s) Specified

| Load No. | X-Left (ft) | X-Right (ft) | Intensity (psf) | Deflection (deg) |
|---|---|---|---|---|
| 1 | 132.00 | 140.00 | 150.0 | 0.0 |
| 2 | 140.10 | 144.00 | 2000.0 | 0.0 |
| 3 | 144.10 | 168.00 | 500.0 | 0.0 |
| 4 | 168.10 | 172.00 | 2000.0 | 0.0 |
| 5 | 172.10 | 176.00 | 150.0 | 0.0 |

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.150(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

1

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

2500 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 85.02(ft)
and X = 85.49(ft)

Each Surface Terminates Between X = 100.00(ft)
and X = 200.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

2.00(ft) Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of 2.0
And 60.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 2500

Number of Trial Surfaces With Valid FS = 2500

Statistical Data On All Valid FS Values:
FS Max = 6.845 FS Min = 2.089 FS Ave = 3.581
Standard Deviation = 0.896 Coefficient of Variation = 25.03
%

Failure Surface Specified By 41 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.599 | 35.787 |
| 3 | 88.172 | 37.023 |
| 4 | 89.736 | 38.269 |
| 5 | 91.294 | 39.523 |
| 6 | 92.843 | 40.788 |
| 7 | 94.386 | 42.061 |
| 8 | 95.920 | 43.343 |
| 9 | 97.447 | 44.635 |
| 10 | 98.967 | 45.936 |
| 11 | 100.478 | 47.246 |

| | | |
|---|---|---|
| 12 | 101.982 | 48.564 |
| 13 | 103.477 | 49.892 |
| 14 | 104.965 | 51.229 |
| 15 | 106.445 | 52.574 |
| 16 | 107.917 | 53.928 |
| 17 | 109.381 | 55.291 |
| 18 | 110.836 | 56.663 |
| 19 | 112.283 | 58.043 |
| 20 | 113.723 | 59.432 |
| 21 | 115.153 | 60.829 |
| 22 | 116.576 | 62.235 |
| 23 | 117.990 | 63.650 |
| 24 | 119.396 | 65.072 |
| 25 | 120.793 | 66.503 |
| 26 | 122.181 | 67.943 |
| 27 | 123.561 | 69.391 |
| 28 | 124.933 | 70.846 |
| 29 | 126.295 | 72.310 |
| 30 | 127.649 | 73.782 |
| 31 | 128.994 | 75.263 |
| 32 | 130.330 | 76.751 |
| 33 | 131.658 | 78.247 |
| 34 | 132.976 | 79.751 |
| 35 | 134.286 | 81.262 |
| 36 | 135.586 | 82.782 |
| 37 | 136.877 | 84.309 |
| 38 | 138.159 | 85.844 |
| 39 | 139.433 | 87.387 |
| 40 | 140.696 | 88.937 |
| 41 | 141.553 | 90.000 |

Circle Center At X = -120.149 ; Y = 300.324 ; and Radius = 335.746

Factor of Safety
*** 2.089 ***

Individual data on the 52 slices

| Slice No. | Width (ft) | Weight (lbs) | Water Force Top (lbs) | Water Force Bot (lbs) | Tie Force Norm (lbs) | Tie Force Tan (lbs) | Earthquake Force Hor (lbs) | Earthquake Force Ver (lbs) | Surcharge Load (lbs) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.5 | 1067.5 | 0.0 | 0.0 | 0. | 0. | 160.1 | 0.0 | 0.0 |
| 2 | 0.5 | 2302.4 | 0.0 | 0.0 | 0. | 0. | 345.4 | 0.0 | 0.0 |
| 3 | 0.5 | 2609.1 | 0.0 | 0.0 | 0. | 0. | 391.4 | 0.0 | 0.0 |
| 4 | 0.1 | 564.2 | 0.0 | 0.0 | 0. | 0. | 84.6 | 0.0 | 0.0 |
| 5 | 1.6 | 8819.2 | 0.0 | 0.0 | 0. | 0. | 1322.9 | 0.0 | 0.0 |
| 6 | 1.6 | 8595.2 | 0.0 | 0.0 | 0. | 0. | 1289.3 | 0.0 | 0.0 |
| 7 | 1.6 | 8370.6 | 0.0 | 0.0 | 0. | 0. | 1255.6 | 0.0 | 0.0 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 8 | 1.5 | 8145.6 | 0.0 | 0.0 | 0. | 0. | 1221.8 | 0.0 |
| 0.0 | | | | | | | | |
| 9 | 1.5 | 7920.1 | 0.0 | 0.0 | 0. | 0. | 1188.0 | 0.0 |
| 0.0 | | | | | | | | |
| 10 | 1.5 | 7694.2 | 0.0 | 0.0 | 0. | 0. | 1154.1 | 0.0 |
| 0.0 | | | | | | | | |
| 11 | 1.5 | 7468.0 | 0.0 | 0.0 | 0. | 0. | 1120.2 | 0.0 |
| 0.0 | | | | | | | | |
| 12 | 1.5 | 7241.6 | 0.0 | 0.0 | 0. | 0. | 1086.2 | 0.0 |
| 0.0 | | | | | | | | |
| 13 | 1.5 | 7014.9 | 0.0 | 0.0 | 0. | 0. | 1052.2 | 0.0 |
| 0.0 | | | | | | | | |
| 14 | 1.5 | 6788.0 | 0.0 | 0.0 | 0. | 0. | 1018.2 | 0.0 |
| 0.0 | | | | | | | | |
| 15 | 1.5 | 6561.1 | 0.0 | 0.0 | 0. | 0. | 984.2 | 0.0 |
| 0.0 | | | | | | | | |
| 16 | 0.5 | 2246.8 | 0.0 | 0.0 | 0. | 0. | 337.0 | 0.0 |
| 0.0 | | | | | | | | |
| 17 | 1.0 | 4100.1 | 0.0 | 0.0 | 0. | 0. | 615.0 | 0.0 |
| 0.0 | | | | | | | | |
| 18 | 1.5 | 6176.4 | 0.0 | 0.0 | 0. | 0. | 926.5 | 0.0 |
| 0.0 | | | | | | | | |
| 19 | 1.5 | 6008.7 | 0.0 | 0.0 | 0. | 0. | 901.3 | 0.0 |
| 0.0 | | | | | | | | |
| 20 | 1.1 | 4334.8 | 0.0 | 0.0 | 0. | 0. | 650.2 | 0.0 |
| 0.0 | | | | | | | | |
| 21 | 0.4 | 1503.5 | 0.0 | 0.0 | 0. | 0. | 225.5 | 0.0 |
| 0.0 | | | | | | | | |
| 22 | 1.5 | 5628.6 | 0.0 | 0.0 | 0. | 0. | 844.3 | 0.0 |
| 0.0 | | | | | | | | |
| 23 | 1.4 | 5403.7 | 0.0 | 0.0 | 0. | 0. | 810.6 | 0.0 |
| 0.0 | | | | | | | | |
| 24 | 1.4 | 5179.1 | 0.0 | 0.0 | 0. | 0. | 776.9 | 0.0 |
| 0.0 | | | | | | | | |
| 25 | 1.4 | 4954.9 | 0.0 | 0.0 | 0. | 0. | 743.2 | 0.0 |
| 0.0 | | | | | | | | |
| 26 | 1.4 | 4731.0 | 0.0 | 0.0 | 0. | 0. | 709.6 | 0.0 |
| 0.0 | | | | | | | | |
| 27 | 1.4 | 4507.5 | 0.0 | 0.0 | 0. | 0. | 676.1 | 0.0 |
| 0.0 | | | | | | | | |
| 28 | 1.4 | 4284.5 | 0.0 | 0.0 | 0. | 0. | 642.7 | 0.0 |
| 0.0 | | | | | | | | |
| 29 | 0.6 | 1781.7 | 0.0 | 0.0 | 0. | 0. | 267.3 | 0.0 |
| 0.0 | | | | | | | | |
| 30 | 0.8 | 2273.6 | 0.0 | 0.0 | 0. | 0. | 341.0 | 0.0 |
| 0.0 | | | | | | | | |
| 31 | 1.4 | 3795.3 | 0.0 | 0.0 | 0. | 0. | 569.3 | 0.0 |
| 0.0 | | | | | | | | |
| 32 | 1.4 | 3532.7 | 0.0 | 0.0 | 0. | 0. | 529.9 | 0.0 |
| 0.0 | | | | | | | | |
| 33 | 1.4 | 3271.7 | 0.0 | 0.0 | 0. | 0. | 490.7 | 0.0 |
| 0.0 | | | | | | | | |
| 34 | 1.4 | 3012.2 | 0.0 | 0.0 | 0. | 0. | 451.8 | 0.0 |
| 0.0 | | | | | | | | |
| 35 | 1.4 | 2754.4 | 0.0 | 0.0 | 0. | 0. | 413.2 | 0.0 |
| 0.0 | | | | | | | | |
| 36 | 1.3 | 2498.2 | 0.0 | 0.0 | 0. | 0. | 374.7 | 0.0 |
| 0.0 | | | | | | | | |
| 37 | 0.0 | 26.5 | 0.0 | 0.0 | 0. | 0. | 4.0 | 0.0 |
| 0.0 | | | | | | | | |
| 38 | 1.3 | 2217.3 | 0.0 | 0.0 | 0. | 0. | 332.6 | 0.0 |
| 0.0 | | | | | | | | |
| 39 | 1.3 | 1991.2 | 0.0 | 0.0 | 0. | 0. | 298.7 | 0.0 |
| 0.0 | | | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 40 | 0.3 | 474.8 | 0.0 | 0.0 | 0. | 0. | 71.2 | 0.0 |
| 0.0 | | | | | | | | |
| 41 | 1.0 | 1265.7 | 0.0 | 0.0 | 0. | 0. | 189.9 | 0.0 |
| 146.4 | | | | | | | | |
| 42 | 1.3 | 1491.7 | 0.0 | 0.0 | 0. | 0. | 223.8 | 0.0 |
| 196.4 | | | | | | | | |
| 43 | 1.2 | 1169.5 | 0.0 | 0.0 | 0. | 0. | 175.4 | 0.0 |
| 182.1 | | | | | | | | |
| 44 | 0.1 | 75.5 | 0.0 | 0.0 | 0. | 0. | 11.3 | 0.0 |
| 13.0 | | | | | | | | |
| 45 | 1.3 | 1000.2 | 0.0 | 0.0 | 0. | 0. | 150.0 | 0.0 |
| 193.7 | | | | | | | | |
| 46 | 1.3 | 757.5 | 0.0 | 0.0 | 0. | 0. | 113.6 | 0.0 |
| 192.3 | | | | | | | | |
| 47 | 1.0 | 410.6 | 0.0 | 0.0 | 0. | 0. | 61.6 | 0.0 |
| 143.5 | | | | | | | | |
| 48 | 0.3 | 106.5 | 0.0 | 0.0 | 0. | 0. | 16.0 | 0.0 |
| 47.4 | | | | | | | | |
| 49 | 0.6 | 154.3 | 0.0 | 0.0 | 0. | 0. | 23.1 | 0.0 |
| 85.1 | | | | | | | | |
| 50 | 0.1 | 22.3 | 0.0 | 0.0 | 0. | 0. | 3.3 | 0.0 |
| 0.0 | | | | | | | | |
| 51 | 0.6 | 102.3 | 0.0 | 0.0 | 0. | 0. | 15.3 | 0.0 |
| 1192.7 | | | | | | | | |
| 52 | 0.9 | 54.6 | 0.0 | 0.0 | 0. | 0. | 8.2 | 0.0 |
| 1713.0 | | | | | | | | |

Failure Surface Specified By 41 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.030 | 35.308 |
| 2 | 86.531 | 36.629 |
| 3 | 88.029 | 37.954 |
| 4 | 89.525 | 39.282 |
| 5 | 91.017 | 40.613 |
| 6 | 92.507 | 41.948 |
| 7 | 93.993 | 43.286 |
| 8 | 95.477 | 44.627 |
| 9 | 96.957 | 45.972 |
| 10 | 98.435 | 47.320 |
| 11 | 99.909 | 48.671 |
| 12 | 101.380 | 50.026 |
| 13 | 102.849 | 51.384 |
| 14 | 104.314 | 52.745 |
| 15 | 105.776 | 54.110 |
| 16 | 107.235 | 55.478 |
| 17 | 108.691 | 56.849 |
| 18 | 110.144 | 58.223 |
| 19 | 111.594 | 59.601 |
| 20 | 113.041 | 60.982 |
| 21 | 114.484 | 62.366 |
| 22 | 115.925 | 63.754 |
| 23 | 117.362 | 65.144 |
| 24 | 118.796 | 66.538 |
| 25 | 120.228 | 67.935 |
| 26 | 121.656 | 69.335 |
| 27 | 123.080 | 70.739 |
| 28 | 124.502 | 72.146 |
| 29 | 125.921 | 73.555 |
| 30 | 127.336 | 74.969 |
| 31 | 128.748 | 76.385 |

| | | |
|---|---|---|
| 32 | 130.157 | 77.804 |
| 33 | 131.563 | 79.227 |
| 34 | 132.966 | 80.653 |
| 35 | 134.365 | 82.081 |
| 36 | 135.761 | 83.513 |
| 37 | 137.154 | 84.949 |
| 38 | 138.544 | 86.387 |
| 39 | 139.930 | 87.828 |
| 40 | 141.314 | 89.273 |
| 41 | 142.007 | 90.000 |

Circle Center At X = -502.285 ; Y = 704.195 ; and Radius = 890.139

Factor of Safety
*** 2.091 ***

1

Failure Surface Specified By 40 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.528 | 35.874 |
| 3 | 88.030 | 37.194 |
| 4 | 89.526 | 38.522 |
| 5 | 91.016 | 39.856 |
| 6 | 92.499 | 41.197 |
| 7 | 93.977 | 42.545 |
| 8 | 95.449 | 43.899 |
| 9 | 96.915 | 45.260 |
| 10 | 98.374 | 46.627 |
| 11 | 99.828 | 48.001 |
| 12 | 101.275 | 49.382 |
| 13 | 102.716 | 50.769 |
| 14 | 104.150 | 52.162 |
| 15 | 105.578 | 53.562 |
| 16 | 107.000 | 54.969 |
| 17 | 108.416 | 56.381 |
| 18 | 109.825 | 57.800 |
| 19 | 111.228 | 59.226 |
| 20 | 112.625 | 60.658 |
| 21 | 114.015 | 62.096 |
| 22 | 115.398 | 63.540 |
| 23 | 116.775 | 64.991 |
| 24 | 118.145 | 66.447 |
| 25 | 119.509 | 67.910 |
| 26 | 120.866 | 69.379 |
| 27 | 122.217 | 70.854 |
| 28 | 123.561 | 72.336 |
| 29 | 124.898 | 73.823 |
| 30 | 126.229 | 75.316 |
| 31 | 127.552 | 76.815 |
| 32 | 128.869 | 78.320 |
| 33 | 130.180 | 79.831 |
| 34 | 131.483 | 81.348 |
| 35 | 132.780 | 82.871 |
| 36 | 134.069 | 84.400 |

| | | |
|---|---|---|
| 37 | 135.352 | 85.934 |
| 38 | 136.628 | 87.474 |
| 39 | 137.897 | 89.020 |
| 40 | 138.693 | 90.000 |

Circle Center At X = -205.754 ; Y = 369.769 ; and Radius = 443.751

Factor of Safety
*** 2.093 ***

Failure Surface Specified By 40 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.556 | 35.841 |
| 3 | 88.085 | 37.130 |
| 4 | 89.607 | 38.427 |
| 5 | 91.122 | 39.733 |
| 6 | 92.630 | 41.047 |
| 7 | 94.130 | 42.370 |
| 8 | 95.623 | 43.700 |
| 9 | 97.109 | 45.039 |
| 10 | 98.588 | 46.385 |
| 11 | 100.059 | 47.740 |
| 12 | 101.523 | 49.103 |
| 13 | 102.980 | 50.473 |
| 14 | 104.429 | 51.852 |
| 15 | 105.870 | 53.239 |
| 16 | 107.304 | 54.633 |
| 17 | 108.730 | 56.035 |
| 18 | 110.149 | 57.445 |
| 19 | 111.559 | 58.862 |
| 20 | 112.962 | 60.288 |
| 21 | 114.358 | 61.721 |
| 22 | 115.745 | 63.161 |
| 23 | 117.125 | 64.609 |
| 24 | 118.496 | 66.065 |
| 25 | 119.860 | 67.528 |
| 26 | 121.216 | 68.998 |
| 27 | 122.563 | 70.476 |
| 28 | 123.903 | 71.961 |
| 29 | 125.234 | 73.453 |
| 30 | 126.558 | 74.953 |
| 31 | 127.873 | 76.460 |
| 32 | 129.180 | 77.974 |
| 33 | 130.478 | 79.495 |
| 34 | 131.768 | 81.023 |
| 35 | 133.050 | 82.558 |
| 36 | 134.324 | 84.101 |
| 37 | 135.589 | 85.650 |
| 38 | 136.845 | 87.206 |
| 39 | 138.093 | 88.768 |
| 40 | 139.066 | 90.000 |

Circle Center At X = -148.982 ; Y = 316.741 ; and Radius = 366.584

Factor of Safety
*** 2.098 ***

1

Failure Surface Specified By 41 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.030 | 35.308 |
| 2 | 86.632 | 36.505 |
| 3 | 88.226 | 37.712 |
| 4 | 89.814 | 38.929 |
| 5 | 91.394 | 40.155 |
| 6 | 92.966 | 41.391 |
| 7 | 94.531 | 42.636 |
| 8 | 96.089 | 43.891 |
| 9 | 97.639 | 45.154 |
| 10 | 99.181 | 46.428 |
| 11 | 100.716 | 47.710 |
| 12 | 102.243 | 49.002 |
| 13 | 103.762 | 50.303 |
| 14 | 105.273 | 51.613 |
| 15 | 106.776 | 52.932 |
| 16 | 108.272 | 54.260 |
| 17 | 109.759 | 55.597 |
| 18 | 111.238 | 56.943 |
| 19 | 112.709 | 58.298 |
| 20 | 114.172 | 59.662 |
| 21 | 115.627 | 61.035 |
| 22 | 117.073 | 62.416 |
| 23 | 118.511 | 63.806 |
| 24 | 119.941 | 65.205 |
| 25 | 121.362 | 66.612 |
| 26 | 122.774 | 68.028 |
| 27 | 124.178 | 69.452 |
| 28 | 125.574 | 70.885 |
| 29 | 126.961 | 72.326 |
| 30 | 128.339 | 73.776 |
| 31 | 129.708 | 75.233 |
| 32 | 131.069 | 76.699 |
| 33 | 132.420 | 78.173 |
| 34 | 133.763 | 79.656 |
| 35 | 135.097 | 81.146 |
| 36 | 136.421 | 82.644 |
| 37 | 137.737 | 84.151 |
| 38 | 139.044 | 85.665 |
| 39 | 140.341 | 87.187 |
| 40 | 141.630 | 88.717 |
| 41 | 142.697 | 90.000 |

Circle Center At X = -112.874 ; Y = 301.732 ; and Radius = 331.885

Factor of Safety
*** 2.102 ***

Failure Surface Specified By 41 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.039 | 36.056 |
| 2 | 86.527 | 37.392 |
| 3 | 88.013 | 38.730 |
| 4 | 89.499 | 40.070 |
| 5 | 90.982 | 41.411 |
| 6 | 92.465 | 42.754 |
| 7 | 93.946 | 44.098 |
| 8 | 95.426 | 45.443 |
| 9 | 96.904 | 46.790 |
| 10 | 98.381 | 48.139 |
| 11 | 99.857 | 49.488 |
| 12 | 101.331 | 50.840 |
| 13 | 102.804 | 52.193 |
| 14 | 104.276 | 53.547 |
| 15 | 105.746 | 54.903 |
| 16 | 107.215 | 56.261 |
| 17 | 108.682 | 57.619 |
| 18 | 110.148 | 58.980 |
| 19 | 111.613 | 60.342 |
| 20 | 113.076 | 61.705 |
| 21 | 114.538 | 63.070 |
| 22 | 115.999 | 64.436 |
| 23 | 117.458 | 65.803 |
| 24 | 118.916 | 67.173 |
| 25 | 120.373 | 68.543 |
| 26 | 121.828 | 69.915 |
| 27 | 123.282 | 71.289 |
| 28 | 124.734 | 72.664 |
| 29 | 126.185 | 74.040 |
| 30 | 127.635 | 75.418 |
| 31 | 129.083 | 76.797 |
| 32 | 130.530 | 78.178 |
| 33 | 131.975 | 79.561 |
| 34 | 133.419 | 80.944 |
| 35 | 134.862 | 82.329 |
| 36 | 136.303 | 83.716 |
| 37 | 137.743 | 85.104 |
| 38 | 139.181 | 86.494 |
| 39 | 140.618 | 87.885 |
| 40 | 142.054 | 89.277 |
| 41 | 142.798 | 90.000 |

Circle Center At X = -1243.445 ; Y = 1516.387 ; and Radius = 1989.032

Factor of Safety
*** 2.104 ***

1

Failure Surface Specified By 38 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.030 | 35.308 |
| 2 | 86.405 | 36.760 |
| 3 | 87.779 | 38.213 |
| 4 | 89.151 | 39.668 |
| 5 | 90.522 | 41.124 |
| 6 | 91.891 | 42.582 |
| 7 | 93.259 | 44.042 |
| 8 | 94.625 | 45.502 |
| 9 | 95.989 | 46.965 |
| 10 | 97.352 | 48.428 |
| 11 | 98.714 | 49.894 |
| 12 | 100.073 | 51.360 |
| 13 | 101.431 | 52.828 |
| 14 | 102.788 | 54.298 |
| 15 | 104.143 | 55.769 |
| 16 | 105.496 | 57.242 |
| 17 | 106.848 | 58.715 |
| 18 | 108.198 | 60.191 |
| 19 | 109.547 | 61.668 |
| 20 | 110.894 | 63.146 |
| 21 | 112.240 | 64.626 |
| 22 | 113.583 | 66.107 |
| 23 | 114.926 | 67.590 |
| 24 | 116.266 | 69.074 |
| 25 | 117.605 | 70.560 |
| 26 | 118.943 | 72.047 |
| 27 | 120.279 | 73.535 |
| 28 | 121.613 | 75.025 |
| 29 | 122.945 | 76.516 |
| 30 | 124.276 | 78.009 |
| 31 | 125.606 | 79.503 |
| 32 | 126.934 | 80.999 |
| 33 | 128.260 | 82.496 |
| 34 | 129.584 | 83.995 |
| 35 | 130.907 | 85.495 |
| 36 | 132.229 | 86.996 |
| 37 | 133.548 | 88.499 |
| 38 | 134.864 | 90.000 |

Circle Center At X = -1260.475 ; Y = 1311.342 ; and Radius = 1854.359

Factor of Safety
*** 2.108 ***

Failure Surface Specified By 43 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.039 | 36.056 |
| 2 | 86.611 | 37.292 |
| 3 | 88.181 | 38.532 |
| 4 | 89.747 | 39.775 |
| 5 | 91.311 | 41.022 |

| | | |
|---|---|---|
| 6 | 92.872 | 42.273 |
| 7 | 94.429 | 43.527 |
| 8 | 95.984 | 44.785 |
| 9 | 97.536 | 46.047 |
| 10 | 99.085 | 47.312 |
| 11 | 100.631 | 48.581 |
| 12 | 102.174 | 49.854 |
| 13 | 103.714 | 51.130 |
| 14 | 105.251 | 52.410 |
| 15 | 106.784 | 53.693 |
| 16 | 108.315 | 54.980 |
| 17 | 109.843 | 56.270 |
| 18 | 111.368 | 57.565 |
| 19 | 112.890 | 58.862 |
| 20 | 114.409 | 60.163 |
| 21 | 115.925 | 61.468 |
| 22 | 117.438 | 62.776 |
| 23 | 118.947 | 64.088 |
| 24 | 120.454 | 65.404 |
| 25 | 121.957 | 66.722 |
| 26 | 123.458 | 68.045 |
| 27 | 124.955 | 69.371 |
| 28 | 126.449 | 70.700 |
| 29 | 127.940 | 72.033 |
| 30 | 129.428 | 73.369 |
| 31 | 130.913 | 74.709 |
| 32 | 132.395 | 76.053 |
| 33 | 133.874 | 77.399 |
| 34 | 135.349 | 78.750 |
| 35 | 136.821 | 80.103 |
| 36 | 138.290 | 81.460 |
| 37 | 139.756 | 82.821 |
| 38 | 141.219 | 84.185 |
| 39 | 142.678 | 85.552 |
| 40 | 144.135 | 86.923 |
| 41 | 145.588 | 88.297 |
| 42 | 147.038 | 89.675 |
| 43 | 147.378 | 90.000 |

Circle Center At X = -442.439 ; Y = 708.619 ; and Radius = 854.737

Factor of Safety
*** 2.121 ***

1

Failure Surface Specified By 46 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.643 | 35.729 |
| 3 | 88.263 | 36.902 |
| 4 | 89.880 | 38.078 |
| 5 | 91.495 | 39.258 |
| 6 | 93.107 | 40.441 |
| 7 | 94.717 | 41.628 |
| 8 | 96.324 | 42.818 |

| | | |
|---|---|---|
| 9 | 97.929 | 44.012 |
| 10 | 99.531 | 45.209 |
| 11 | 101.131 | 46.410 |
| 12 | 102.727 | 47.615 |
| 13 | 104.321 | 48.822 |
| 14 | 105.913 | 50.033 |
| 15 | 107.502 | 51.248 |
| 16 | 109.088 | 52.466 |
| 17 | 110.672 | 53.688 |
| 18 | 112.253 | 54.913 |
| 19 | 113.831 | 56.141 |
| 20 | 115.407 | 57.373 |
| 21 | 116.980 | 58.608 |
| 22 | 118.550 | 59.847 |
| 23 | 120.117 | 61.089 |
| 24 | 121.682 | 62.335 |
| 25 | 123.244 | 63.584 |
| 26 | 124.804 | 64.836 |
| 27 | 126.360 | 66.092 |
| 28 | 127.914 | 67.351 |
| 29 | 129.465 | 68.613 |
| 30 | 131.014 | 69.879 |
| 31 | 132.560 | 71.148 |
| 32 | 134.102 | 72.421 |
| 33 | 135.643 | 73.697 |
| 34 | 137.180 | 74.976 |
| 35 | 138.715 | 76.259 |
| 36 | 140.246 | 77.544 |
| 37 | 141.775 | 78.834 |
| 38 | 143.302 | 80.126 |
| 39 | 144.825 | 81.422 |
| 40 | 146.345 | 82.721 |
| 41 | 147.863 | 84.024 |
| 42 | 149.378 | 85.330 |
| 43 | 150.890 | 86.639 |
| 44 | 152.399 | 87.951 |
| 45 | 153.906 | 89.267 |
| 46 | 154.741 | 90.000 |

Circle Center At X = -451.670 ; Y = 781.057 ; and Radius = 919.399

Factor of Safety
*** 2.127 ***

Failure Surface Specified By 38 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.039 | 36.056 |
| 2 | 86.423 | 37.500 |
| 3 | 87.805 | 38.946 |
| 4 | 89.184 | 40.394 |
| 5 | 90.562 | 41.843 |
| 6 | 91.938 | 43.295 |
| 7 | 93.313 | 44.748 |
| 8 | 94.685 | 46.203 |
| 9 | 96.055 | 47.659 |

| | | |
|---|---|---|
| 10 | 97.424 | 49.118 |
| 11 | 98.790 | 50.578 |
| 12 | 100.155 | 52.040 |
| 13 | 101.518 | 53.504 |
| 14 | 102.879 | 54.970 |
| 15 | 104.238 | 56.437 |
| 16 | 105.595 | 57.906 |
| 17 | 106.950 | 59.377 |
| 18 | 108.303 | 60.850 |
| 19 | 109.655 | 62.324 |
| 20 | 111.004 | 63.800 |
| 21 | 112.352 | 65.278 |
| 22 | 113.697 | 66.758 |
| 23 | 115.041 | 68.239 |
| 24 | 116.383 | 69.722 |
| 25 | 117.722 | 71.207 |
| 26 | 119.060 | 72.694 |
| 27 | 120.396 | 74.182 |
| 28 | 121.730 | 75.672 |
| 29 | 123.062 | 77.164 |
| 30 | 124.392 | 78.658 |
| 31 | 125.720 | 80.153 |
| 32 | 127.047 | 81.650 |
| 33 | 128.371 | 83.149 |
| 34 | 129.693 | 84.650 |
| 35 | 131.014 | 86.152 |
| 36 | 132.332 | 87.656 |
| 37 | 133.648 | 89.161 |
| 38 | 134.380 | 90.000 |

Circle Center At X = -1023.960 ; Y = 1099.948 ; and Radius = 1536.797

Factor of Safety

*** 2.129 ***

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

```
PROFIL  C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section -
static-lower.in Version G7v2.005.2 [GSTABL72.EXE]
e
PR NO. 2918 - 5228 Chelsea Street       Section 1 - Static -lower
15 9
0. 20. 83. 27. 4
83. 27. 85. 33. 5
85. 33. 85.5 72. 5
85.5 72. 86. 75. 5
86. 75. 86.5 83. 3
86.5 83. 104. 86. 2
104. 86. 109. 88. 1
109. 88. 120. 90. 1
120. 90. 240. 90. 1
104. 86. 174. 88. 2
174. 88. 240. 88. 2
86.5 83. 240. 82. 3
86. 75. 240. 76. 5
0. 20. 81. 22. 5
81. 22. 83. 27. 5
0. 0. 0.
SOIL  Fill    Terrace TMsc    Cobble  TMss
5
120. 120. 100. 30. 0. 0. 0
120. 120. 300. 33. 0. 0. 0
120. 120. 1000. 33. 0. 0. 0
125. 125. 0. 40. 0. 0. 0
120. 120. 3500. 35. 0. 0. 0
LOADS
5
132. 140. 150. 0.
140.1 144. 2000. 0.
144.1 168. 500. 0.
168.1 172. 2000. 0.
172.1 176. 150. 0.
EQUAKE
0.15 0.15 0.0 0. 1
EQUAKE
0.
CIRCL2
100 100
83.1 84.9 100. 200.
0. 5. 65. 5.
```

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2A, July 2012 **



**********
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
**********

Analysis Run Date: 7/17/2016
Time of Run: 02:10PM
Run By: Username
Input Data Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - static-lower.in
Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - static-lower.OUT
Unit System: English

Plotted Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - static-lower.PLT

PROBLEM DESCRIPTION: PR NO. 2918 - 5228 Chelsea Street
Section 1 - Static -lower

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | 0.00 | 20.00 | 83.00 | 27.00 | 4 |
| 2 | 83.00 | 27.00 | 85.00 | 33.00 | 5 |
| 3 | 85.00 | 33.00 | 85.50 | 72.00 | 5 |
| 4 | 85.50 | 72.00 | 86.00 | 75.00 | 5 |
| 5 | 86.00 | 75.00 | 86.50 | 83.00 | 3 |
| 6 | 86.50 | 83.00 | 104.00 | 86.00 | 2 |
| 7 | 104.00 | 86.00 | 109.00 | 88.00 | 1 |
| 8 | 109.00 | 88.00 | 120.00 | 90.00 | 1 |

| | | | | | |
|---|---|---|---|---|---|
| 9 | 120.00 | 90.00 | 240.00 | 90.00 | 1 |
| 10 | 104.00 | 86.00 | 174.00 | 88.00 | 2 |
| 11 | 174.00 | 88.00 | 240.00 | 88.00 | 2 |
| 12 | 86.50 | 83.00 | 240.00 | 82.00 | 3 |
| 13 | 86.00 | 75.00 | 240.00 | 76.00 | 5 |
| 14 | 0.00 | 20.00 | 81.00 | 22.00 | 5 |
| 15 | 81.00 | 22.00 | 83.00 | 27.00 | 5 |

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| 5 | 120.0 | 120.0 | 3500.0 | 35.0 | 0.00 | 0.0 | 0 |

1

BOUNDARY LOAD(S)

5 Load(s) Specified

| Load No. | X-Left (ft) | X-Right (ft) | Intensity (psf) | Deflection (deg) |
|---|---|---|---|---|
| 1 | 132.00 | 140.00 | 150.0 | 0.0 |
| 2 | 140.10 | 144.00 | 2000.0 | 0.0 |
| 3 | 144.10 | 168.00 | 500.0 | 0.0 |
| 4 | 168.10 | 172.00 | 2000.0 | 0.0 |
| 5 | 172.10 | 176.00 | 150.0 | 0.0 |

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.150(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

EARTHQUAKE DATA HAS BEEN SUPPRESSED

1

A Critical Failure Surface Searching Method, Using A Random

Technique For Generating Circular Surfaces, Has Been Specified.

10000 Trial Surfaces Have Been Generated.

100 Surface(s) Initiate(s) From Each Of 100 Points Equally Spaced Along The Ground Surface Between X = 83.10(ft)
and X = 84.90(ft)

Each Surface Terminates Between X = 100.00(ft)
and X = 200.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)

5.00(ft) Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of 5.0
And 65.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 0

Number of Trial Surfaces With Valid FS = 0

Statistical Data On All Valid FS Values:
FS Max = 0.000 FS Min = 500.000 FS Ave = NaN
Standard Deviation = 0.000 Coefficient of Variation = NaN
%

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.100 | 27.300 |
| 2 | 86.875 | 30.579 |
| 3 | 90.612 | 33.900 |
| 4 | 94.311 | 37.264 |
| 5 | 97.972 | 40.670 |
| 6 | 101.594 | 44.117 |
| 7 | 105.176 | 47.606 |
| 8 | 108.718 | 51.135 |
| 9 | 112.220 | 54.703 |

| | | |
|---|---|---|
| 10 | 115.681 | 58.312 |
| 11 | 119.100 | 61.960 |
| 12 | 122.478 | 65.646 |
| 13 | 125.814 | 69.371 |
| 14 | 129.107 | 73.133 |
| 15 | 132.357 | 76.933 |
| 16 | 135.563 | 80.770 |
| 17 | 138.726 | 84.642 |
| 18 | 141.844 | 88.551 |
| 19 | 142.974 | 90.000 |

Circle Center At X = -202.956 ; Y = 360.445 ; and Radius = 439.106

Factor of Safety
*** 2.334 ***

Individual data on the 0 slices

| Slice No. | Width (ft) | Weight (lbs) | Water Force Top (lbs) | Water Force Bot (lbs) | Tie Force Norm (lbs) | Tie Force Tan (lbs) | Earthquake Force Hor (lbs) | Earthquake Force Ver (lbs) | Surcharge Load (lbs) |
|---|---|---|---|---|---|---|---|---|---|

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.209 | 27.627 |
| 2 | 86.839 | 31.066 |
| 3 | 90.454 | 34.520 |
| 4 | 94.055 | 37.989 |
| 5 | 97.640 | 41.475 |
| 6 | 101.209 | 44.976 |
| 7 | 104.764 | 48.492 |
| 8 | 108.303 | 52.024 |
| 9 | 111.827 | 55.571 |
| 10 | 115.336 | 59.133 |
| 11 | 118.829 | 62.711 |
| 12 | 122.306 | 66.303 |
| 13 | 125.768 | 69.911 |
| 14 | 129.214 | 73.534 |
| 15 | 132.644 | 77.172 |
| 16 | 136.059 | 80.824 |
| 17 | 139.457 | 84.492 |
| 18 | 142.840 | 88.174 |
| 19 | 144.503 | 90.000 |

Circle Center At X = -706.598 ; Y = 865.127 ; and Radius = 1151.173

Factor of Safety
*** 2.337 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.373 | 28.118 |
| 2 | 87.131 | 31.416 |
| 3 | 90.859 | 34.748 |
| 4 | 94.558 | 38.112 |
| 5 | 98.227 | 41.508 |
| 6 | 101.866 | 44.937 |
| 7 | 105.475 | 48.398 |
| 8 | 109.053 | 51.891 |
| 9 | 112.600 | 55.415 |
| 10 | 116.115 | 58.970 |
| 11 | 119.599 | 62.557 |
| 12 | 123.052 | 66.174 |
| 13 | 126.472 | 69.821 |
| 14 | 129.860 | 73.498 |
| 15 | 133.215 | 77.205 |
| 16 | 136.537 | 80.942 |
| 17 | 139.826 | 84.708 |
| 18 | 143.082 | 88.502 |
| 19 | 144.344 | 90.000 |

Circle Center At X = -288.122 ; Y = 455.176 ; and Radius = 566.027

Factor of Safety
*** 2.340 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.245 | 27.736 |
| 2 | 86.901 | 31.148 |
| 3 | 90.534 | 34.583 |
| 4 | 94.144 | 38.042 |
| 5 | 97.732 | 41.525 |
| 6 | 101.296 | 45.031 |
| 7 | 104.838 | 48.561 |
| 8 | 108.357 | 52.113 |
| 9 | 111.852 | 55.688 |
| 10 | 115.324 | 59.286 |
| 11 | 118.772 | 62.907 |
| 12 | 122.196 | 66.551 |
| 13 | 125.597 | 70.216 |
| 14 | 128.973 | 73.904 |
| 15 | 132.325 | 77.614 |
| 16 | 135.653 | 81.346 |
| 17 | 138.956 | 85.099 |
| 18 | 142.235 | 88.874 |
| 19 | 143.200 | 90.000 |

Circle Center At X = -436.238 ; Y = 587.996 ; and Radius = 764.037

Factor of Safety
*** 2.340 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.173 | 27.518 |
| 2 | 87.049 | 30.676 |
| 3 | 90.877 | 33.892 |
| 4 | 94.656 | 37.167 |
| 5 | 98.383 | 40.500 |
| 6 | 102.060 | 43.888 |
| 7 | 105.684 | 47.333 |
| 8 | 109.255 | 50.833 |
| 9 | 112.772 | 54.387 |
| 10 | 116.234 | 57.995 |
| 11 | 119.640 | 61.655 |
| 12 | 122.990 | 65.366 |
| 13 | 126.283 | 69.129 |
| 14 | 129.518 | 72.941 |
| 15 | 132.694 | 76.803 |
| 16 | 135.811 | 80.713 |
| 17 | 138.867 | 84.670 |
| 18 | 141.863 | 88.673 |
| 19 | 142.825 | 90.000 |

Circle Center At X = -121.213 ; Y = 282.407 ; and Radius = 326.714

Factor of Safety
*** 2.341 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.282 | 27.845 |
| 2 | 86.879 | 31.318 |
| 3 | 90.466 | 34.802 |
| 4 | 94.041 | 38.297 |
| 5 | 97.605 | 41.804 |
| 6 | 101.157 | 45.322 |
| 7 | 104.699 | 48.852 |
| 8 | 108.229 | 52.393 |
| 9 | 111.748 | 55.945 |
| 10 | 115.255 | 59.509 |

| | | |
|---|---|---|
| 11 | 118.751 | 63.084 |
| 12 | 122.235 | 66.670 |
| 13 | 125.708 | 70.267 |
| 14 | 129.170 | 73.875 |
| 15 | 132.619 | 77.494 |
| 16 | 136.058 | 81.124 |
| 17 | 139.484 | 84.766 |
| 18 | 142.899 | 88.418 |
| 19 | 144.369 | 90.000 |

Circle Center At X = -998.960 ; Y = 1152.647 ; and Radius = 1560.906

Factor of Safety
*** 2.341 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.173 | 27.518 |
| 2 | 86.876 | 30.878 |
| 3 | 90.548 | 34.271 |
| 4 | 94.189 | 37.698 |
| 5 | 97.797 | 41.159 |
| 6 | 101.374 | 44.653 |
| 7 | 104.918 | 48.180 |
| 8 | 108.429 | 51.740 |
| 9 | 111.908 | 55.331 |
| 10 | 115.353 | 58.955 |
| 11 | 118.764 | 62.611 |
| 12 | 122.142 | 66.298 |
| 13 | 125.485 | 70.016 |
| 14 | 128.793 | 73.764 |
| 15 | 132.067 | 77.543 |
| 16 | 135.306 | 81.353 |
| 17 | 138.510 | 85.191 |
| 18 | 141.678 | 89.060 |
| 19 | 142.433 | 90.000 |

Circle Center At X = -278.465 ; Y = 429.890 ; and Radius = 541.004

Factor of Safety
*** 2.343 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.227 | 27.682 |

| | | |
|---|---|---|
| 2 | 86.838 | 31.141 |
| 3 | 90.429 | 34.619 |
| 4 | 94.002 | 38.117 |
| 5 | 97.556 | 41.634 |
| 6 | 101.091 | 45.170 |
| 7 | 104.607 | 48.725 |
| 8 | 108.104 | 52.299 |
| 9 | 111.582 | 55.891 |
| 10 | 115.040 | 59.502 |
| 11 | 118.479 | 63.132 |
| 12 | 121.898 | 66.780 |
| 13 | 125.298 | 70.447 |
| 14 | 128.678 | 74.131 |
| 15 | 132.037 | 77.834 |
| 16 | 135.377 | 81.555 |
| 17 | 138.697 | 85.294 |
| 18 | 141.997 | 89.050 |
| 19 | 142.822 | 90.000 |

Circle Center At X = -558.353 ; Y = 700.920 ; and Radius = 929.987

Factor of Safety
*** 2.344 ***

1

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.300 | 27.900 |
| 2 | 87.257 | 30.956 |
| 3 | 91.165 | 34.076 |
| 4 | 95.021 | 37.258 |
| 5 | 98.826 | 40.502 |
| 6 | 102.577 | 43.808 |
| 7 | 106.275 | 47.173 |
| 8 | 109.918 | 50.598 |
| 9 | 113.505 | 54.081 |
| 10 | 117.035 | 57.622 |
| 11 | 120.508 | 61.220 |
| 12 | 123.921 | 64.873 |
| 13 | 127.276 | 68.581 |
| 14 | 130.570 | 72.342 |
| 15 | 133.802 | 76.157 |
| 16 | 136.973 | 80.023 |
| 17 | 140.081 | 83.940 |
| 18 | 143.125 | 87.906 |
| 19 | 144.678 | 90.000 |

Circle Center At X = -103.747 ; Y = 274.206 ; and Radius = 309.279

Factor of Safety
*** 2.345 ***

Failure Surface Specified By 19 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 83.300 | 27.900 |
| 2 | 87.288 | 30.916 |
| 3 | 91.222 | 34.001 |
| 4 | 95.101 | 37.156 |
| 5 | 98.924 | 40.379 |
| 6 | 102.689 | 43.669 |
| 7 | 106.396 | 47.025 |
| 8 | 110.042 | 50.446 |
| 9 | 113.628 | 53.931 |
| 10 | 117.151 | 57.479 |
| 11 | 120.611 | 61.088 |
| 12 | 124.007 | 64.758 |
| 13 | 127.337 | 68.488 |
| 14 | 130.601 | 72.276 |
| 15 | 133.797 | 76.121 |
| 16 | 136.925 | 80.021 |
| 17 | 139.983 | 83.977 |
| 18 | 142.971 | 87.986 |
| 19 | 144.418 | 90.000 |

Circle Center At X = -85.295 ; Y = 255.004 ; and Radius = 282.844

Factor of Safety
*** 2.345 ***

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

```
PROFIL  C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section -
static-upper.in Version G7v2.005.2 [GSTABL72.EXE]
e
PR NO. 2918 - 5228 Chelsea Street       Section 1 - Static -upper
15 9
0. 20. 83. 27. 4
83. 27. 85. 33. 5
85. 33. 85.5 72. 5
85.5 72. 86. 75. 5
86. 75. 86.5 83. 3
86.5 83. 104. 86. 2
104. 86. 109. 88. 1
109. 88. 120. 90. 1
120. 90. 240. 90. 1
104. 86. 174. 88. 2
174. 88. 240. 88. 2
86.5 83. 240. 82. 3
86. 75. 240. 76. 5
0. 20. 81. 22. 5
81. 22. 83. 27. 5
0. 0. 0.
SOIL  Fill    Terrace TMsc    Cobble  TMss
5
120. 120. 100. 30. 0. 0. 0
120. 120. 300. 33. 0. 0. 0
120. 120. 1000. 33. 0. 0. 0
125. 125. 0. 40. 0. 0. 0
120. 120. 3500. 35. 0. 0. 0
LOADS
5
132. 140. 150. 0.
140.1 144. 2000. 0.
144.1 168. 500. 0.
168.1 172. 2000. 0.
172.1 176. 150. 0.
EQUAKE
0.15 0.15 0.0 0. 1
EQUAKE
0.
CIRCL2
50 50
85.02 85.49 100. 200.
0. 2. 60. 2.
```

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2A, July 2012 **



**********
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
**********

Analysis Run Date: 7/17/2016
Time of Run: 02:11PM
Run By: Username
Input Data Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - static-upper.in
Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - static-upper.OUT
Unit System: English

Plotted Output Filename: C:\Project Files\2900-2999\2918 - 5228 Chelesea Street\2918 section - static-upper.PLT

PROBLEM DESCRIPTION: PR NO. 2918 - 5228 Chelsea Street
Section 1 - Static -upper

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | 0.00 | 20.00 | 83.00 | 27.00 | 4 |
| 2 | 83.00 | 27.00 | 85.00 | 33.00 | 5 |
| 3 | 85.00 | 33.00 | 85.50 | 72.00 | 5 |
| 4 | 85.50 | 72.00 | 86.00 | 75.00 | 5 |
| 5 | 86.00 | 75.00 | 86.50 | 83.00 | 3 |
| 6 | 86.50 | 83.00 | 104.00 | 86.00 | 2 |
| 7 | 104.00 | 86.00 | 109.00 | 88.00 | 1 |
| 8 | 109.00 | 88.00 | 120.00 | 90.00 | 1 |

| | | | | | |
|---|---|---|---|---|---|
| 9 | 120.00 | 90.00 | 240.00 | 90.00 | 1 |
| 10 | 104.00 | 86.00 | 174.00 | 88.00 | 2 |
| 11 | 174.00 | 88.00 | 240.00 | 88.00 | 2 |
| 12 | 86.50 | 83.00 | 240.00 | 82.00 | 3 |
| 13 | 86.00 | 75.00 | 240.00 | 76.00 | 5 |
| 14 | 0.00 | 20.00 | 81.00 | 22.00 | 5 |
| 15 | 81.00 | 22.00 | 83.00 | 27.00 | 5 |

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

5 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| 5 | 120.0 | 120.0 | 3500.0 | 35.0 | 0.00 | 0.0 | 0 |

1

BOUNDARY LOAD(S)

5 Load(s) Specified

| Load No. | X-Left (ft) | X-Right (ft) | Intensity (psf) | Deflection (deg) |
|---|---|---|---|---|
| 1 | 132.00 | 140.00 | 150.0 | 0.0 |
| 2 | 140.10 | 144.00 | 2000.0 | 0.0 |
| 3 | 144.10 | 168.00 | 500.0 | 0.0 |
| 4 | 168.10 | 172.00 | 2000.0 | 0.0 |
| 5 | 172.10 | 176.00 | 150.0 | 0.0 |

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.150(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

EARTHQUAKE DATA HAS BEEN SUPPRESSED

1

A Critical Failure Surface Searching Method, Using A Random

Technique For Generating Circular Surfaces, Has Been Specified.

2500 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced Along The Ground Surface Between X = 85.02(ft)
and X = 85.49(ft)

Each Surface Terminates Between X = 100.00(ft)
and X = 200.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)

2.00(ft) Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of 2.0
And 60.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 2500

Number of Trial Surfaces With Valid FS = 2500

Statistical Data On All Valid FS Values:
FS Max = 12.374 FS Min = 2.474 FS Ave = 4.553
Standard Deviation = 1.414 Coefficient of Variation = 31.05
%

Failure Surface Specified By 36 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.030 | 35.308 |
| 2 | 86.452 | 36.714 |
| 3 | 87.861 | 38.133 |
| 4 | 89.258 | 39.565 |
| 5 | 90.642 | 41.009 |
| 6 | 92.013 | 42.465 |
| 7 | 93.370 | 43.933 |
| 8 | 94.715 | 45.413 |
| 9 | 96.047 | 46.906 |

| | | |
|---|---|---|
| 10 | 97.365 | 48.410 |
| 11 | 98.670 | 49.926 |
| 12 | 99.961 | 51.453 |
| 13 | 101.238 | 52.992 |
| 14 | 102.502 | 54.543 |
| 15 | 103.751 | 56.104 |
| 16 | 104.987 | 57.676 |
| 17 | 106.209 | 59.260 |
| 18 | 107.416 | 60.854 |
| 19 | 108.610 | 62.459 |
| 20 | 109.788 | 64.075 |
| 21 | 110.953 | 65.701 |
| 22 | 112.103 | 67.337 |
| 23 | 113.238 | 68.984 |
| 24 | 114.358 | 70.641 |
| 25 | 115.464 | 72.307 |
| 26 | 116.555 | 73.984 |
| 27 | 117.631 | 75.670 |
| 28 | 118.691 | 77.365 |
| 29 | 119.737 | 79.070 |
| 30 | 120.767 | 80.785 |
| 31 | 121.782 | 82.508 |
| 32 | 122.781 | 84.240 |
| 33 | 123.765 | 85.981 |
| 34 | 124.734 | 87.731 |
| 35 | 125.687 | 89.490 |
| 36 | 125.957 | 90.000 |

Circle Center At X = -71.847 ; Y = 195.363 ; and Radius = 224.115

Factor of Safety
*** 2.474 ***

Individual data on the 44 slices

| Slice No. | Width (ft) | Weight (lbs) | Water Force Top (lbs) | Water Force Bot (lbs) | Tie Force Norm (lbs) | Tie Force Tan (lbs) | Earthquake Force Hor (lbs) | Earthquake Force Ver (lbs) | Surcharge Load (lbs) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.5 | 1022.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 2 | 0.5 | 2248.8 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 3 | 0.5 | 2282.8 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 4 | 0.0 | 266.4 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 5 | 1.4 | 7458.8 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 6 | 1.4 | 7459.1 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 7 | 1.4 | 7191.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 8 | 1.4 | 6924.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |
| 9 | 1.4 | 6658.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 | 0.0 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 10 | 1.3 | 6393.4 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 11 | 1.3 | 6129.6 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 12 | 1.3 | 5867.0 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 13 | 1.3 | 5605.8 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 14 | 1.3 | 5346.2 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 15 | 1.3 | 5088.3 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 16 | 1.3 | 4832.2 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 17 | 1.2 | 4578.1 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 18 | 0.2 | 886.6 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 19 | 1.0 | 3452.9 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 20 | 1.2 | 4129.9 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 21 | 1.2 | 3922.2 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 22 | 1.2 | 3715.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 23 | 0.4 | 1180.4 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 24 | 0.8 | 2321.6 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 25 | 1.2 | 3264.2 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 26 | 1.1 | 3027.4 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 27 | 1.1 | 2793.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 28 | 1.1 | 2562.8 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 29 | 1.1 | 2335.3 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 30 | 1.1 | 2111.1 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 31 | 0.8 | 1386.9 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 32 | 0.3 | 503.6 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 33 | 1.1 | 1673.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 34 | 1.0 | 1460.3 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 35 | 0.3 | 337.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 36 | 0.8 | 907.0 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 37 | 1.0 | 1017.4 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 38 | 0.2 | 133.2 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 39 | 0.8 | 661.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 40 | 1.0 | 577.3 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 41 | 0.3 | 146.5 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 42 | 0.6 | 218.8 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 43 | 1.0 | 158.9 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |
| 44 | 0.3 | 8.3 | 0.0 | 0.0 | 0. | 0. | 0.0 | 0.0 |
| 0.0 | | | | | | | | |

Failure Surface Specified By 35 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.496 | 35.910 |
| 3 | 87.949 | 37.284 |
| 4 | 89.379 | 38.682 |
| 5 | 90.786 | 40.103 |
| 6 | 92.169 | 41.548 |
| 7 | 93.528 | 43.015 |
| 8 | 94.863 | 44.505 |
| 9 | 96.173 | 46.016 |
| 10 | 97.458 | 47.549 |
| 11 | 98.717 | 49.103 |
| 12 | 99.950 | 50.677 |
| 13 | 101.157 | 52.272 |
| 14 | 102.338 | 53.886 |
| 15 | 103.491 | 55.520 |
| 16 | 104.618 | 57.172 |
| 17 | 105.717 | 58.843 |
| 18 | 106.788 | 60.532 |
| 19 | 107.832 | 62.239 |
| 20 | 108.847 | 63.962 |
| 21 | 109.833 | 65.702 |
| 22 | 110.790 | 67.458 |
| 23 | 111.718 | 69.229 |
| 24 | 112.617 | 71.016 |
| 25 | 113.486 | 72.817 |
| 26 | 114.325 | 74.633 |
| 27 | 115.134 | 76.462 |
| 28 | 115.913 | 78.304 |
| 29 | 116.661 | 80.159 |
| 30 | 117.379 | 82.026 |
| 31 | 118.065 | 83.904 |
| 32 | 118.720 | 85.794 |
| 33 | 119.344 | 87.694 |
| 34 | 119.937 | 89.604 |
| 35 | 120.052 | 90.000 |

Circle Center At X = 4.119 ; Y = 124.474 ; and Radius = 120.953

Factor of Safety
*** 2.478 ***

1

Failure Surface Specified By 35 Coordinate Points

| Point | X-Surf | Y-Surf |
|---|---|---|

| No. | (ft) | (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.614 | 35.768 |
| 3 | 88.182 | 37.009 |
| 4 | 89.726 | 38.281 |
| 5 | 91.243 | 39.584 |
| 6 | 92.733 | 40.918 |
| 7 | 94.196 | 42.281 |
| 8 | 95.631 | 43.674 |
| 9 | 97.038 | 45.096 |
| 10 | 98.415 | 46.546 |
| 11 | 99.763 | 48.024 |
| 12 | 101.080 | 49.529 |
| 13 | 102.367 | 51.060 |
| 14 | 103.622 | 52.617 |
| 15 | 104.846 | 54.199 |
| 16 | 106.037 | 55.806 |
| 17 | 107.195 | 57.437 |
| 18 | 108.320 | 59.090 |
| 19 | 109.411 | 60.766 |
| 20 | 110.468 | 62.464 |
| 21 | 111.490 | 64.183 |
| 22 | 112.477 | 65.923 |
| 23 | 113.429 | 67.682 |
| 24 | 114.345 | 69.460 |
| 25 | 115.224 | 71.256 |
| 26 | 116.067 | 73.070 |
| 27 | 116.873 | 74.900 |
| 28 | 117.642 | 76.747 |
| 29 | 118.373 | 78.608 |
| 30 | 119.066 | 80.484 |
| 31 | 119.721 | 82.374 |
| 32 | 120.337 | 84.277 |
| 33 | 120.915 | 86.191 |
| 34 | 121.453 | 88.117 |
| 35 | 121.939 | 90.000 |

Circle Center At X = 26.257 ; Y = 113.705 ; and Radius = 98.575

Factor of Safety
*** 2.490 ***

Failure Surface Specified By 40 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.528 | 35.874 |
| 3 | 88.030 | 37.194 |
| 4 | 89.526 | 38.522 |
| 5 | 91.016 | 39.856 |
| 6 | 92.499 | 41.197 |
| 7 | 93.977 | 42.545 |
| 8 | 95.449 | 43.899 |
| 9 | 96.915 | 45.260 |
| 10 | 98.374 | 46.627 |

| | | |
|---|---|---|
| 11 | 99.828 | 48.001 |
| 12 | 101.275 | 49.382 |
| 13 | 102.716 | 50.769 |
| 14 | 104.150 | 52.162 |
| 15 | 105.578 | 53.562 |
| 16 | 107.000 | 54.969 |
| 17 | 108.416 | 56.381 |
| 18 | 109.825 | 57.800 |
| 19 | 111.228 | 59.226 |
| 20 | 112.625 | 60.658 |
| 21 | 114.015 | 62.096 |
| 22 | 115.398 | 63.540 |
| 23 | 116.775 | 64.991 |
| 24 | 118.145 | 66.447 |
| 25 | 119.509 | 67.910 |
| 26 | 120.866 | 69.379 |
| 27 | 122.217 | 70.854 |
| 28 | 123.561 | 72.336 |
| 29 | 124.898 | 73.823 |
| 30 | 126.229 | 75.316 |
| 31 | 127.552 | 76.815 |
| 32 | 128.869 | 78.320 |
| 33 | 130.180 | 79.831 |
| 34 | 131.483 | 81.348 |
| 35 | 132.780 | 82.871 |
| 36 | 134.069 | 84.400 |
| 37 | 135.352 | 85.934 |
| 38 | 136.628 | 87.474 |
| 39 | 137.897 | 89.020 |
| 40 | 138.693 | 90.000 |

Circle Center At X = -205.754 ; Y = 369.769 ; and Radius = 443.751

Factor of Safety
*** 2.495 ***

1

Failure Surface Specified By 38 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.030 | 35.308 |
| 2 | 86.405 | 36.760 |
| 3 | 87.779 | 38.213 |
| 4 | 89.151 | 39.668 |
| 5 | 90.522 | 41.124 |
| 6 | 91.891 | 42.582 |
| 7 | 93.259 | 44.042 |
| 8 | 94.625 | 45.502 |
| 9 | 95.989 | 46.965 |
| 10 | 97.352 | 48.428 |
| 11 | 98.714 | 49.894 |
| 12 | 100.073 | 51.360 |
| 13 | 101.431 | 52.828 |
| 14 | 102.788 | 54.298 |
| 15 | 104.143 | 55.769 |
| 16 | 105.496 | 57.242 |

| | | |
|---|---|---|
| 17 | 106.848 | 58.715 |
| 18 | 108.198 | 60.191 |
| 19 | 109.547 | 61.668 |
| 20 | 110.894 | 63.146 |
| 21 | 112.240 | 64.626 |
| 22 | 113.583 | 66.107 |
| 23 | 114.926 | 67.590 |
| 24 | 116.266 | 69.074 |
| 25 | 117.605 | 70.560 |
| 26 | 118.943 | 72.047 |
| 27 | 120.279 | 73.535 |
| 28 | 121.613 | 75.025 |
| 29 | 122.945 | 76.516 |
| 30 | 124.276 | 78.009 |
| 31 | 125.606 | 79.503 |
| 32 | 126.934 | 80.999 |
| 33 | 128.260 | 82.496 |
| 34 | 129.584 | 83.995 |
| 35 | 130.907 | 85.495 |
| 36 | 132.229 | 86.996 |
| 37 | 133.548 | 88.499 |
| 38 | 134.864 | 90.000 |

Circle Center At X = -1260.475 ; Y = 1311.342 ; and Radius = 1854.359

Factor of Safety
*** 2.498 ***

Failure Surface Specified By 41 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.599 | 35.787 |
| 3 | 88.172 | 37.023 |
| 4 | 89.736 | 38.269 |
| 5 | 91.294 | 39.523 |
| 6 | 92.843 | 40.788 |
| 7 | 94.386 | 42.061 |
| 8 | 95.920 | 43.343 |
| 9 | 97.447 | 44.635 |
| 10 | 98.967 | 45.936 |
| 11 | 100.478 | 47.246 |
| 12 | 101.982 | 48.564 |
| 13 | 103.477 | 49.892 |
| 14 | 104.965 | 51.229 |
| 15 | 106.445 | 52.574 |
| 16 | 107.917 | 53.928 |
| 17 | 109.381 | 55.291 |
| 18 | 110.836 | 56.663 |
| 19 | 112.283 | 58.043 |
| 20 | 113.723 | 59.432 |
| 21 | 115.153 | 60.829 |
| 22 | 116.576 | 62.235 |
| 23 | 117.990 | 63.650 |
| 24 | 119.396 | 65.072 |
| 25 | 120.793 | 66.503 |

| | | |
|---|---|---|
| 26 | 122.181 | 67.943 |
| 27 | 123.561 | 69.391 |
| 28 | 124.933 | 70.846 |
| 29 | 126.295 | 72.310 |
| 30 | 127.649 | 73.782 |
| 31 | 128.994 | 75.263 |
| 32 | 130.330 | 76.751 |
| 33 | 131.658 | 78.247 |
| 34 | 132.976 | 79.751 |
| 35 | 134.286 | 81.262 |
| 36 | 135.586 | 82.782 |
| 37 | 136.877 | 84.309 |
| 38 | 138.159 | 85.844 |
| 39 | 139.433 | 87.387 |
| 40 | 140.696 | 88.937 |
| 41 | 141.553 | 90.000 |

Circle Center At X = -120.149 ; Y = 300.324 ; and Radius = 335.746

Factor of Safety
*** 2.499 ***

1

Failure Surface Specified By 36 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.644 | 35.727 |
| 3 | 88.244 | 36.927 |
| 4 | 89.817 | 38.162 |
| 5 | 91.364 | 39.429 |
| 6 | 92.884 | 40.729 |
| 7 | 94.377 | 42.061 |
| 8 | 95.840 | 43.424 |
| 9 | 97.275 | 44.817 |
| 10 | 98.680 | 46.241 |
| 11 | 100.054 | 47.694 |
| 12 | 101.397 | 49.176 |
| 13 | 102.709 | 50.685 |
| 14 | 103.989 | 52.223 |
| 15 | 105.235 | 53.786 |
| 16 | 106.449 | 55.376 |
| 17 | 107.628 | 56.991 |
| 18 | 108.773 | 58.631 |
| 19 | 109.883 | 60.295 |
| 20 | 110.958 | 61.982 |
| 21 | 111.997 | 63.691 |
| 22 | 112.999 | 65.421 |
| 23 | 113.965 | 67.173 |
| 24 | 114.894 | 68.944 |
| 25 | 115.784 | 70.735 |
| 26 | 116.637 | 72.544 |
| 27 | 117.451 | 74.371 |
| 28 | 118.227 | 76.214 |
| 29 | 118.963 | 78.074 |
| 30 | 119.660 | 79.948 |

| | | |
|---|---|---|
| 31 | 120.317 | 81.837 |
| 32 | 120.934 | 83.740 |
| 33 | 121.511 | 85.655 |
| 34 | 122.047 | 87.582 |
| 35 | 122.542 | 89.519 |
| 36 | 122.654 | 90.000 |

Circle Center At X = 30.669 ; Y = 111.948 ; and Radius = 94.567

Factor of Safety
*** 2.499 ***

Failure Surface Specified By 35 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.039 | 36.056 |
| 2 | 86.508 | 37.413 |
| 3 | 87.962 | 38.787 |
| 4 | 89.399 | 40.177 |
| 5 | 90.821 | 41.583 |
| 6 | 92.228 | 43.006 |
| 7 | 93.617 | 44.444 |
| 8 | 94.991 | 45.898 |
| 9 | 96.348 | 47.367 |
| 10 | 97.688 | 48.852 |
| 11 | 99.011 | 50.351 |
| 12 | 100.318 | 51.866 |
| 13 | 101.607 | 53.395 |
| 14 | 102.878 | 54.939 |
| 15 | 104.132 | 56.497 |
| 16 | 105.369 | 58.069 |
| 17 | 106.587 | 59.654 |
| 18 | 107.788 | 61.254 |
| 19 | 108.970 | 62.867 |
| 20 | 110.134 | 64.494 |
| 21 | 111.279 | 66.133 |
| 22 | 112.406 | 67.785 |
| 23 | 113.514 | 69.450 |
| 24 | 114.604 | 71.128 |
| 25 | 115.674 | 72.817 |
| 26 | 116.725 | 74.519 |
| 27 | 117.756 | 76.232 |
| 28 | 118.768 | 77.957 |
| 29 | 119.761 | 79.694 |
| 30 | 120.734 | 81.441 |
| 31 | 121.687 | 83.200 |
| 32 | 122.620 | 84.969 |
| 33 | 123.533 | 86.748 |
| 34 | 124.426 | 88.538 |
| 35 | 125.135 | 90.000 |

Circle Center At X = -34.150 ; Y = 166.525 ; and Radius = 176.715

Factor of Safety

*** 2.502 ***

1

Failure Surface Specified By 40 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.020 | 34.560 |
| 2 | 86.556 | 35.841 |
| 3 | 88.085 | 37.130 |
| 4 | 89.607 | 38.427 |
| 5 | 91.122 | 39.733 |
| 6 | 92.630 | 41.047 |
| 7 | 94.130 | 42.370 |
| 8 | 95.623 | 43.700 |
| 9 | 97.109 | 45.039 |
| 10 | 98.588 | 46.385 |
| 11 | 100.059 | 47.740 |
| 12 | 101.523 | 49.103 |
| 13 | 102.980 | 50.473 |
| 14 | 104.429 | 51.852 |
| 15 | 105.870 | 53.239 |
| 16 | 107.304 | 54.633 |
| 17 | 108.730 | 56.035 |
| 18 | 110.149 | 57.445 |
| 19 | 111.559 | 58.862 |
| 20 | 112.962 | 60.288 |
| 21 | 114.358 | 61.721 |
| 22 | 115.745 | 63.161 |
| 23 | 117.125 | 64.609 |
| 24 | 118.496 | 66.065 |
| 25 | 119.860 | 67.528 |
| 26 | 121.216 | 68.998 |
| 27 | 122.563 | 70.476 |
| 28 | 123.903 | 71.961 |
| 29 | 125.234 | 73.453 |
| 30 | 126.558 | 74.953 |
| 31 | 127.873 | 76.460 |
| 32 | 129.180 | 77.974 |
| 33 | 130.478 | 79.495 |
| 34 | 131.768 | 81.023 |
| 35 | 133.050 | 82.558 |
| 36 | 134.324 | 84.101 |
| 37 | 135.589 | 85.650 |
| 38 | 136.845 | 87.206 |
| 39 | 138.093 | 88.768 |
| 40 | 139.066 | 90.000 |

Circle Center At X = -148.982 ; Y = 316.741 ; and Radius = 366.584

Factor of Safety
*** 2.502 ***

Failure Surface Specified By 36 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 85.039 | 36.056 |
| 2 | 86.553 | 37.363 |
| 3 | 88.052 | 38.687 |
| 4 | 89.537 | 40.026 |
| 5 | 91.008 | 41.382 |
| 6 | 92.463 | 42.754 |
| 7 | 93.904 | 44.141 |
| 8 | 95.330 | 45.543 |
| 9 | 96.740 | 46.962 |
| 10 | 98.135 | 48.395 |
| 11 | 99.514 | 49.843 |
| 12 | 100.877 | 51.306 |
| 13 | 102.225 | 52.784 |
| 14 | 103.556 | 54.277 |
| 15 | 104.872 | 55.783 |
| 16 | 106.171 | 57.304 |
| 17 | 107.453 | 58.839 |
| 18 | 108.718 | 60.388 |
| 19 | 109.967 | 61.950 |
| 20 | 111.199 | 63.526 |
| 21 | 112.414 | 65.115 |
| 22 | 113.611 | 66.717 |
| 23 | 114.791 | 68.331 |
| 24 | 115.953 | 69.959 |
| 25 | 117.098 | 71.599 |
| 26 | 118.225 | 73.251 |
| 27 | 119.334 | 74.916 |
| 28 | 120.424 | 76.592 |
| 29 | 121.497 | 78.280 |
| 30 | 122.551 | 79.980 |
| 31 | 123.587 | 81.691 |
| 32 | 124.604 | 83.413 |
| 33 | 125.602 | 85.146 |
| 34 | 126.582 | 86.889 |
| 35 | 127.543 | 88.644 |
| 36 | 128.267 | 90.000 |

Circle Center At X = -34.860 ; Y = 176.427 ; and Radius = 184.607

Factor of Safety

*** 2.506 ***

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

# ATTACHMENT B

# TERRACOSTA RESPONSE TO COASTAL COMMISSION REVIEW COMMENTS Dated November 28, 2016 (Appendix B omitted)

TerraCosta Consulting Group

TerraCosta
Consulting Group

*Geotechnical Engineering*
*Coastal Engineering*
*Maritime Engineering*

Project No. 2918
November 28, 2016

Mr. Alex Llerandi
**CALIFORNIA COASTAL COMMISSION**
7575 Metropolitan Drive, Suite 103
San Diego, California 92108

RESPONSE TO COASTAL COMMISSION
REVIEW COMMENTS
**5228 CHELSEA DRIVE**
LA JOLLA, CALIFORNIA

Dear Mr. Llerandi:

TerraCosta Consulting Group, Inc. (TerraCosta) is responding to your October 21, 2016, email requesting additional information specific to the soil strength parameters cited in our July 19, 2016, Geotechnical Investigation and Bluff Stability Study report for the subject property. For completeness of the record, and as indicated in your October 21 email, previous concerns regarding the location of the bluff edge and the effects of sea level rise have been adequately addressed, with the only item remaining being Item 2, specifically "*Soil strength parameter: The study cited from which soil strengths were derived is from a project almost 20 miles away with very different geologic conditions. The soil strength parameters derived there are not transferrable to this site. Thus, the soil strength parameters in the original report remain unsupported.*"

In response to these concerns, on October 24, 2016, we spoke with Dr. Mark Johnsson, the Coastal Commission's Staff Geologist, about the Mt. Soledad Formation and its relatively limited exposures along the San Diego County coastline limited to the area between Tourmaline Park and just south of Sea Ridge Drive, a distance of no more than 1,300 feet. Actually, the northern extent of the exposed Mt. Soledad Formation is a mapped fault previously reported in the California Division of Mines and Geology Bulletin 200 – Geology of the San Diego Metropolitan Area, California, published in 1975. In Figure 1, we have reproduced a portion of the City of San Diego April 3, 2008, Seismic Safety Study – Geologic Hazards and Faults, which shows the unnamed fault located about 280 feet northwest of the subject property. Bulletin 200 also provides an

excellent description of both the Cabrillo Formation exposed to the north of the fault, along with the Mt. Soledad Formation exposed to the south of the fault, with Figure 2 showing the actual unconformity between the Mt. Soledad and Cabrillo Formations reproduced from Bulletin 200. For completeness of the record, in Appendix A, we have provided copies of Pages 15 and 16 of Bulletin 200, which describe these two geologic units, with the Cretaceous-age Cabrillo Formation being approximately 66.4 million years old and the Eocene-age Mt. Soledad Formation being approximately 57.8 million years old. The geologic time scale is provided by the Geological Society of America.

In our discussions with Dr. Johnsson on October 24, he indicated that the approximately 800-foot offset in the shoreline might be the result of changes in geologic stratigraphy, and specifically the result of differential erosion rates between the Cretaceous Cabrillo Formation and the Eocene Mt. Soledad Formation. Dr. Mark Johnsson volunteered that this opinion was based on discussions with the San Diego City Geologist, Mr. Jim Quinn, and I indicated that I would contact Mr. Quinn to independently confirm Mr. Quinn's opinion regarding this 800-foot offset in the La Jolla shoreline.

In subsequent discussions with Mr. Quinn, he indicated that he did not offer to anyone that the 800-foot offset in the shoreline was a result of differential erosion between the Cabrillo and Mt. Soledad Formations, and then offered that the photograph contained in Bulletin 200 (Figure 2, attached) actually shows the unconformity between the geologic units, with no erosion offset between the Cretaceous Cabrillo Formation and the Eocene Mt. Soledad Formation. If, for some reason, Dr. Johnsson would like to discuss this further with Mr. Quinn, he can be reached at (619) 446-5334.

In our discussions with Dr. Johnsson, he indicated that some testing on the Mt. Soledad Formation would be very helpful. Prior to embarking upon an expensive drilling and testing program, we performed additional research and found a geotechnical report dated May 26, 2016, for a proposed new single-family residence located at 5220 Chelsea Drive, the adjacent lot to the southeast, prepared by Christian Wheeler Engineering (CWE). A copy of that report is provided in Appendix B. Notably, CWE obtained a chunk sample of the Mt. Soledad Formation from the cobble beach below the subject property, took the chuck to the lab and carved samples, on which they performed direct shear testing, specifically Plate No. 26 of their attached report, with the reported angle of internal friction being 38 degrees and a cohesion intercept of 1,500 psf.

We discussed with Dr. Johnsson the importance of obtaining unweathered soft rock samples to obtain unweathered specimens for strength testing, with the weathering process typically substantially degrading the unweathered strength of most geologic units. In San Diego, the Eocene Ardath Shale is particularly susceptible to strength degradation from weathering, resulting in considerable construction defect litigation resulting associated with substantial changes in engineering properties from the unweathered state to its highly weathered state.

In Appendix C, we have provided an April 1980 Federal Highway Administration report (FHWA-TS-80-219) titled, "Design and Construction of Shale Embankments: Summary," which discusses the importance of good quality core borings to obtain unweathered samples for subsequent evaluation in susceptibility of soil degradation. While the majority of the report is more specific to fill performance, for which we have considerable data on the substantial loss of soil strength in the Ardath Shale due to weathering, the fact remains that testing on a loose chunk sample of Mt. Soledad Formation taken from within the surf zone and likely exposed to possibly years of weathering, would underestimate the actual unweathered soil strength of this geologic unit previously estimated and reported in our July 19, 2016, geotechnical report.

Given the soil strengths reported by CWE for the adjacent parcel to the south, we conducted additional stability analyses on the same geologic section reported in our July 19, 2016, report, with the factor of safety, using weathered soil strengths, being 1.39. By increasing the cohesion intercept by 250 psf, the resulting factor of safety is above 1.5. Recognizing that soil strength can be mathematically described as $C + \sigma \tan \varphi$, where $\sigma$ would be the average overburden pressure, when using a friction angle of 38 degrees and a cohesion intercept of 1,500 psf and 40 feet of overburden (considered representative of the mid section of the slope stability profile), the available soil strength would be 5,406 psf. By increasing the cohesion intercept to 1,750 psf, the available soil strength becomes 5,656 psf, with the 5 percent increase in soil strength increasing the factor of safety to in excess of 1.5. Copies of our stability analyses are provided in Appendix D.

Given the preceding, and as we discussed with Dr. Johnsson, we remain of the opinion that if we spent the money, we would still find unweathered soil strengths well in excess of the values reported by CWE on their weathered chunk sample, and more along the lines of the Eocene sediments of the younger Santiago Formation tested and reported in

our July 2016 geotechnical report. As we discussed with Dr. Johnsson, if necessary, we will conduct the expensive core boring and additional strength testing. However, we remain confident that given the totality of all of the data, the opinions, conclusions, and recommendations contained in our July 2016 report are still appropriate. Moreover, the developer notes that their City Permit will not allow the construction of future shoreline stabilization measures with the currently proposed 25-foot bluff-top setback, and as a result, question the need for yet additional and costly confirmation testing.

We trust this adequately addresses your concerns regarding the soil strength of the Mt. Soledad Formation. If you have any questions or require additional information, please give us a call.

Very truly yours,

TERRACOSTA CONSULTING GROUP, INC.

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

Gregory A. Spaulding, Project Geologist
C.E.G. 1863, C.H.G. 351, R.G. 5892

WFC/GAS/jg
Attachments

cc: David Lessnick, D. Marin Development
Chandra Slaven, Blue Heron
Claude Anthony Marengo, Magengo Morton Architects

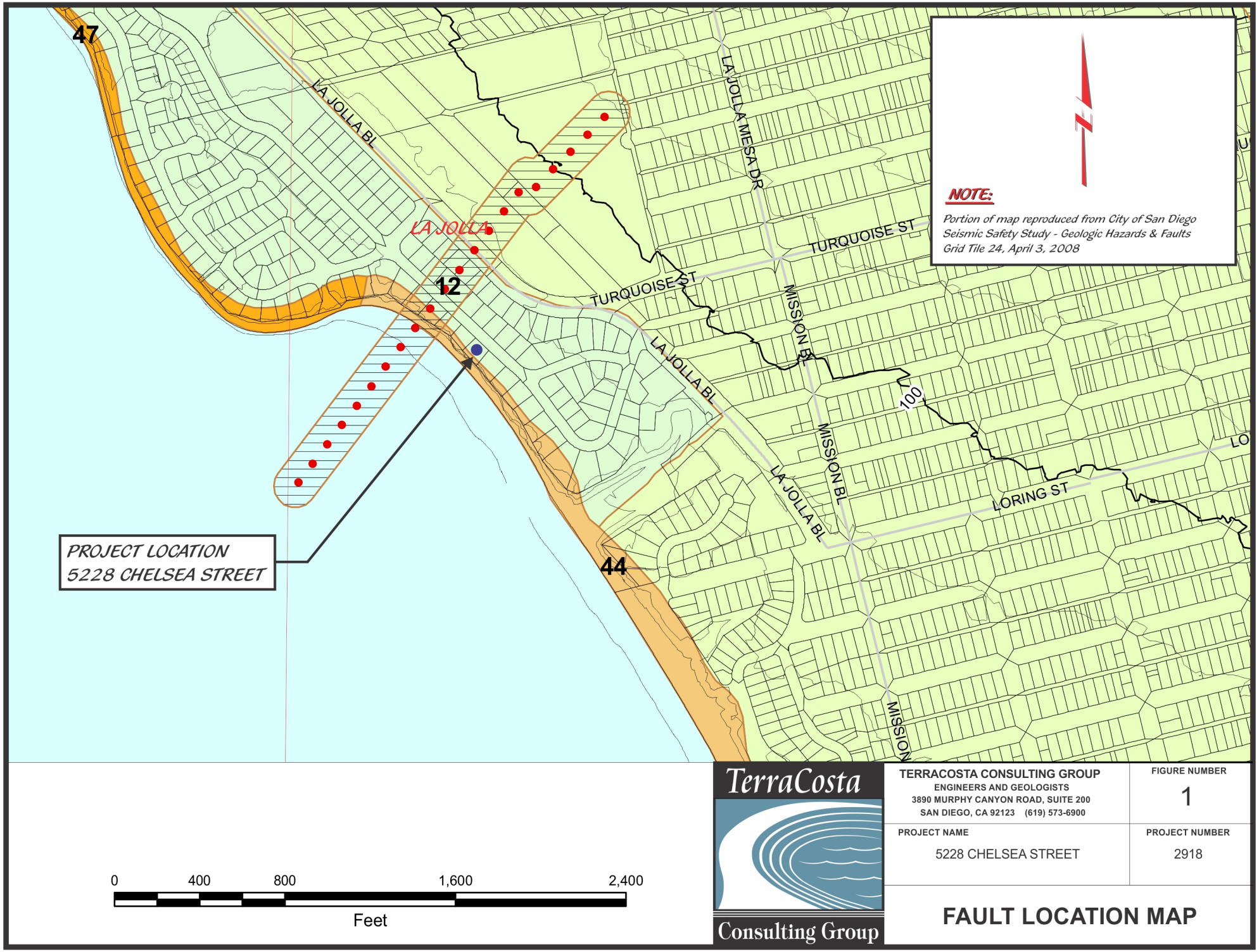


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FIGURE NUMBER
1

PROJECT NAME
5228 CHELSEA STREET

PROJECT NUMBER
2918

# FAULT LOCATION MAP

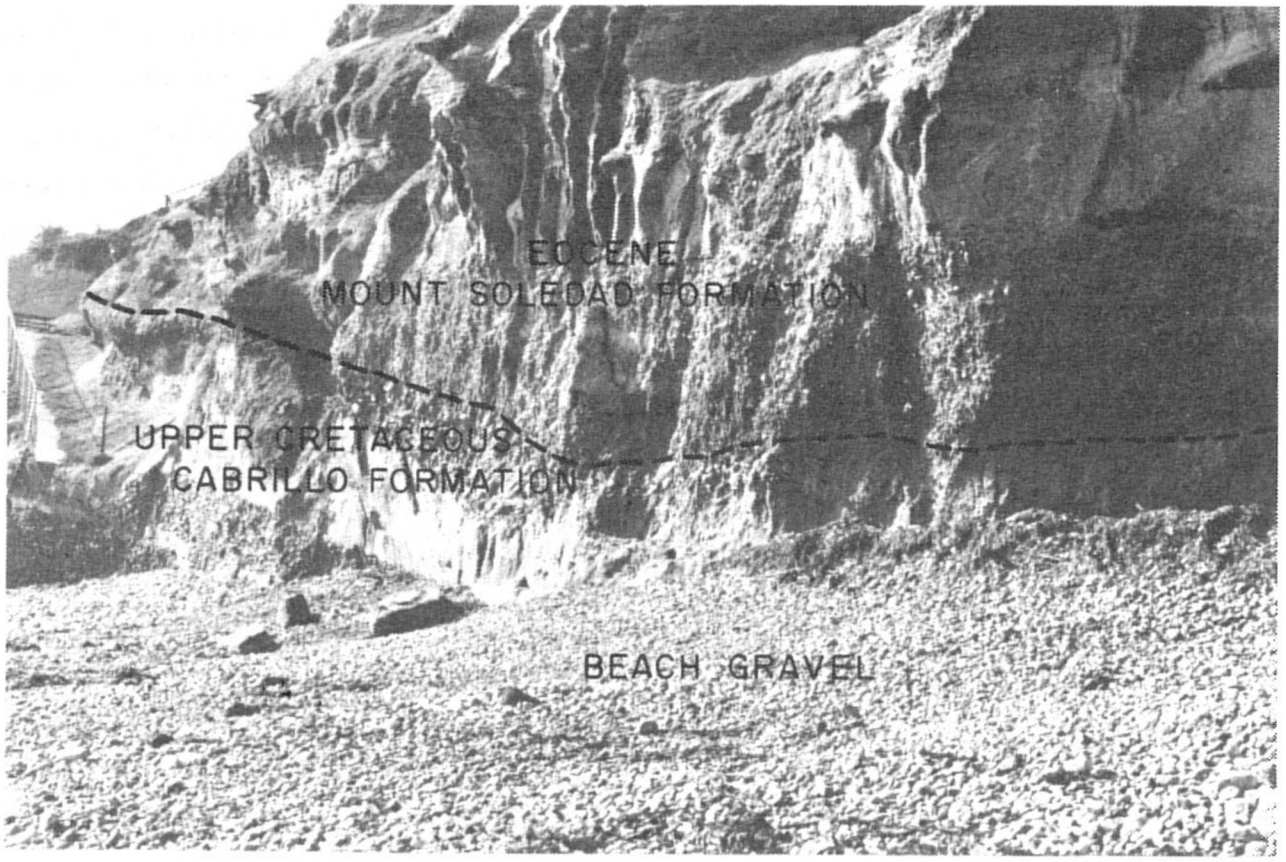


Photo 2. Unconformity between the Eocene and Upper Cretaceous rocks located 300 meters north of Tourmaline Street in Pacific Beach, looking northeast. The Eocene rocks contain soda rhyolite-tuff clasts not found in the Upper Cretaceous conglomerates.

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FIGURE NUMBER
2

PROJECT NAME
5228 CHELSEA STREET

PROJECT NUMBER
2918

**BULLETIN 200 PHOTOGRAPH**
**MOUNT SOLEDAD & CABRILLO FORMATION UNCONFORMITY**

# APPENDIX A

# CALIFORNIA DIVISION OF MINES AND GEOLOGY BULLETIN 200 1975 PAGES 15 AND 16

TerraCosta Consulting Group

## Rosario Group

The Upper Cretaceous Rosario Group is composed of clastic sedimentary rocks of marine and nonmarine origin assigned to the Lusardi, Point Loma, and Cabrillo Formations.

### Lusardi Formation

The basal formation of the Rosario Group, the Lusardi Formation, was named by Nordstrom (1970) for exposures of boulder conglomerate near the confluence of Lusardi Creek and the San Dieguito River, 2 km nórth of the area in the Rancho Santa Fe quadrangle.

These rocks consist of cobble and boulder conglomerate, with occasional thin lenses of medium-grained sandstone. Some of the clasts are 10 m in diameter. The Lusardi Formation at the exposures within the northeast quarter of the Del Mar quadrangle, and within the type area to the north, has a maximum thickness of 125 meters. At the Holderness No. 1 well, 17 km southeast of the tip of the Point Loma Peninsula, rocks considered to belong to the Lusardi Formation are 82 m thick, whereas at the Point Loma No. 1 well, 10 km north of Point Loma, these rocks are 295-376 m thick (Hertlein and Grant, 1944, p. 38). The Lusardi Formation at its type area is unconformably overlain by Eocene rocks, but 16 km to the north near Carlsbad, it is overlain conformably by siltstone and sandstone of the Point Loma Formation (Kennedy and Moore, 1971b).

The Lusardi Formation is considered to be Late Cretaceous in age because it contains quartz diorite boulders eroded from the mid-Cretaceous southern California batholith, which has a minimum age of 105 ± 10 million years (Bushee *et al.*, 1963), and it is overlain by the Point Loma Formation which contains Upper Cretaceous (Campanian) Foraminifera (Sliter, 1968).

The Lusardi Formation is lithologically equivalent to the Trabuco Formation of the Santa Ana Mountains on the north (Nordstrom, 1970), to an unnamed fanglomerate near the base of the Williams Formation, also in the Santa Ana Mountains (Morton, 1972, p. 39), and to the Redondo Formation of Flynn (1970) in northern Baja California.

### Point Loma Formation

The Point Loma Formation, the intermediate part of the Rosario Group, crops out along the sea cliffs on the west side of the Point Loma Peninsula, and in the La Jolla sea cliffs from Bird Rock to La Jolla Shores Beach (plates 2A,3A). At its type locality at the tip of Point Loma, it has an exposed thickness of 83 meters. The rocks there are interbedded fine-grained dusky-yellow sandstone and olive-gray clay shale that occur in graded beds about 30 centimeters (cm) thick.

Scuba-diving observations 1860 m offshore from the type locality show that ledgy pavement-like sandstone, similar to that in the lower half of the exposed section, continues to a depth of at least 37 m (Turner *et al.*, 1968, p. 8). With a shoreline dip of 6° E., it is postulated that 190 m of section may be added below low-tide level to the observed thickness of the formation. This submarine information, combined with interpolation from well logs, suggests that the total thickness of the Point Loma Formation at its type locality is about 300 m (section D-D').

Fossil Foraminifera and calcareous nannoplankton indicate a Late Cretaceous age for the Point Loma Formation (Sliter, 1968; Bukry and Kennedy, 1969). Foraminifera from near the base of the formation at Carlsbad are middle to upper Campanian in age, whereas those from the uppermost beds are lower Maestrichtian in age (Sliter, 1968).

The exposed part of the Point Loma Formation correlates with the Williams Formation and the upper part of the Ladd Formation in the Santa Ana Mountains (Popenoe *et al.*, 1960) and with the middle part of Beal's (1924) Formacion Rosario in northern Baja California.

### Cabrillo Formation

The Cabrillo Formation, the uppermost unit of the Rosario Group, is exposed on the Point Loma Peninsula from the southern tip north to Sunset Cliffs. At Pacific Beach in the sea cliffs, it is exposed from 300 m south of False Point to Bird Rock on the north and at La Jolla in an S-shaped belt around the noses of the Pacific Beach syncline and Mount Soledad anticline. In the sea cliff at its type section 250 m east of the new Point Loma lighthouse, it consists of massive medium-grained sandstone and cross-bedded cobble conglomerate containing fresh plutonic and metavolcanic clasts but lacking red porphyritic rhyolite-tuff cobbles characteristic of nearby Eocene rocks.

Throughout the mapped area, the Cabrillo Formation conformably overlies the Point Loma Formation. The formation is 81 m thick at its type locality, where it is unconformably overlain by Pleistocene deposits. Along the sea cliff at False Point, it has a thickness of 170 meters.

A clam from the east flank of Mount Soledad within the lower 5 m of the Cabrillo Formation has been identified as *"Pharella" alta* (Gabb) and assigned to the Maestrichtian (L. Saul, written communication, 1969). The Cabrillo Formation correlates with the upper part of the Formacion Rosario of Beal (1924) in northern Baja California and possibly with the upper part of the Williams Formation in the Santa Ana Mountains.

# EOCENE DEPOSITS

## La Jolla Group

The La Jolla Group (La Jolla Formation of Hanna, 1926) ranges from moderately deep-water, fine-grained siltstone, to sandy beach and lagoonal facies, and coarse-grained continental sandstone and conglomerate. Deep water fine-grained facies predominate to the southwest, whereas the lagoonal

and continental facies are more abundant to the northeast. These units include six partly intertonguing and partially time equivalent formations, which from oldest to youngest, are the Mount Soledad Formation, Delmar Formation, Torrey Sandstone, Ardath Shale, Scripps Formation, and Friars Formation (figure 4).

### Mount Soledad Formation

Southwest of the Rose Canyon fault, which in Rose Canyon displaces rocks on its southwest side (figure 4; plate 2A; section B-B′ ), an Eocene marine cobble conglomerate and sandstone unit, designated the Mount Soledad Conglomerate (part of the Rose Canyon Shale Member of Hanna, 1926), rests unconformably on Upper Cretaceous rocks of the Cabrillo Formation. This formation crops out around the Mount Soledad anticline in La Jolla and northern Pacific Beach and south of Mission Bay on the southwest flank of the Pacific Beach syncline (plates 2A, 3A). Block diagrams 3, 5, and 6 (figure 4) show the stratigraphic relationship of the Mount Soledad Formation and the overlying and partly intertonguing Ardath Shale, Delmar Formation, and Torrey Sandstone. At its type locality on Mount Soledad, the formation is 69 m thick and consists of cobble conglomerate with minor beds of sandstone. The conglomerate content of the formation is variable to the southeast where it is locally composed entirely of medium-grained sandstone. The conglomerate commonly overlies similar Upper Cretaceous conglomerate of the Cabrillo Formation. The presence of distinctive red porphyritic, soda rhyolite-tuff clasts in the Mount Soledad Formation differentiates it from the Cabrillo Formation. This difference is easily seen at a sea-cliff exposure 300 m northwest of the end of Tourmaline Street in Pacific Beach where the two conglomerates are in contact (photo 2). The sandstone is moderately well sorted, subangular to subrounded, poorly indurated, and well bedded. It consists of quartz (75-80 percent), potassium feldspar (20-25 percent), plagioclase (1-2 percent), biotite (1-2 percent), and a trace of epidote, pyroxene, and hematite.

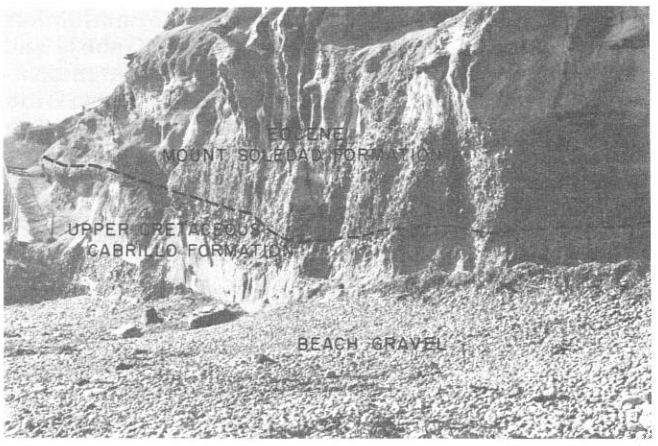


Photo 2. Unconformity between the Eocene and Upper Cretaceous rocks located 300 meters north of Tourmaline Street in Pacific Beach, looking northeast. The Eocene rocks contain soda rhyolite-tuff clasts not found in the Upper Cretaceous conglomerates.

The Ardath Shale, conformably overlying the Mount Soledad Formation, contains fossils which are lower middle Eocene in age (Bukry and Kennedy, 1969). The Mount Soledad Formation correlates with the basal part of the Santiago Formation in the Santa Ana Mountains (Kennedy and Moore, 1971a).

### Delmar Formation

The Delmar Formation (Delmar Sand Member of Hanna, 1926) is exposed from the northern edge of the area mapped for 9 km south to Soledad Valley where it is overlain by younger rocks (plate 1A). The stratigraphic relationship of the Delmar Formation with the Mount Soledad Formation, Torrey Sandstone, and Ardath Shale is shown in figure 4.

Most of the Delmar Formation is dusky yellowish-green sandy claystone interbedded with medium-gray coarse-grained sandstone. Several resistant beds composed of *Ostrea idriaensis* Gabb and other brackish-water mollusks indicate a lagoonal origin. The sandstone is typically composed of quartz (80-85 percent), potassium feldspar (10-15 percent), plagioclase (1-2 percent), biotite (2-3 percent), and a trace of hematite, topaz, glauconite, and pyroxene. The claystone is composed of montmorillonite and kaolinite.

The base of the formation is not exposed but is presumed to rest unconformably on Upper Cretaceous or older rocks (section A-A′ ) or conformably on the Mount Soledad Formation as do correlative formations to the north and south (section B-B′ ). In its type section near the town of Del Mar and throughout the area, it is overlain gradationally by the Torrey Sandstone, with which it is also partly equivalent (figure 4). In the subsurface 15 km north near Carlsbad, the Delmar Formation grades into the Santiago Formation, and its boundary with the Santiago Formation occurs directly below the northernmost depositional limit of the overlying Torrey Sandstone (Kennedy and Moore, 1971a).

The Delmar Formation is considered to be middle Eocene in age because it is correlative in part with the Mount Soledad Formation on the south, the Santiago Formation on the north, and contains a rich Domengine molluscan assemblage.

### Torrey Sandstone

The Torrey Sandstone (Torrey Sand Member of Hanna, 1926) crops out continuously from the northern boundary of the area 12 km south to Torrey Pines Golf Course and inland about 10 km (plates 1A, 2A). It has a maximum thickness of 60 m and is composed of arkosic sandstone which is white to light brown, medium to coarse grained, subangular, and moderately well indurated. It is massive and broadly cross-bedded. The sandstone consists of quartz (85-90 percent), orthoclase (5-10 percent), plagioclase (less than 1 percent), biotite (1-5 percent), and a trace of hematite, epidote, zircon, tourmaline, pyroxene, and amphibole. At the type section at

APPENDIX B
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FROM THIS COPY

# APPENDIX B

GEOTECHNICAL REPORT
PREPARED BY
CHRISTIAN WHEELER ENGINEERING
PROPOSED SINGLE-FAMILY RESIDENCE
5220 CHELSEA DRIVE
DATED MAY 26, 2016

TerraCosta
Consulting Group

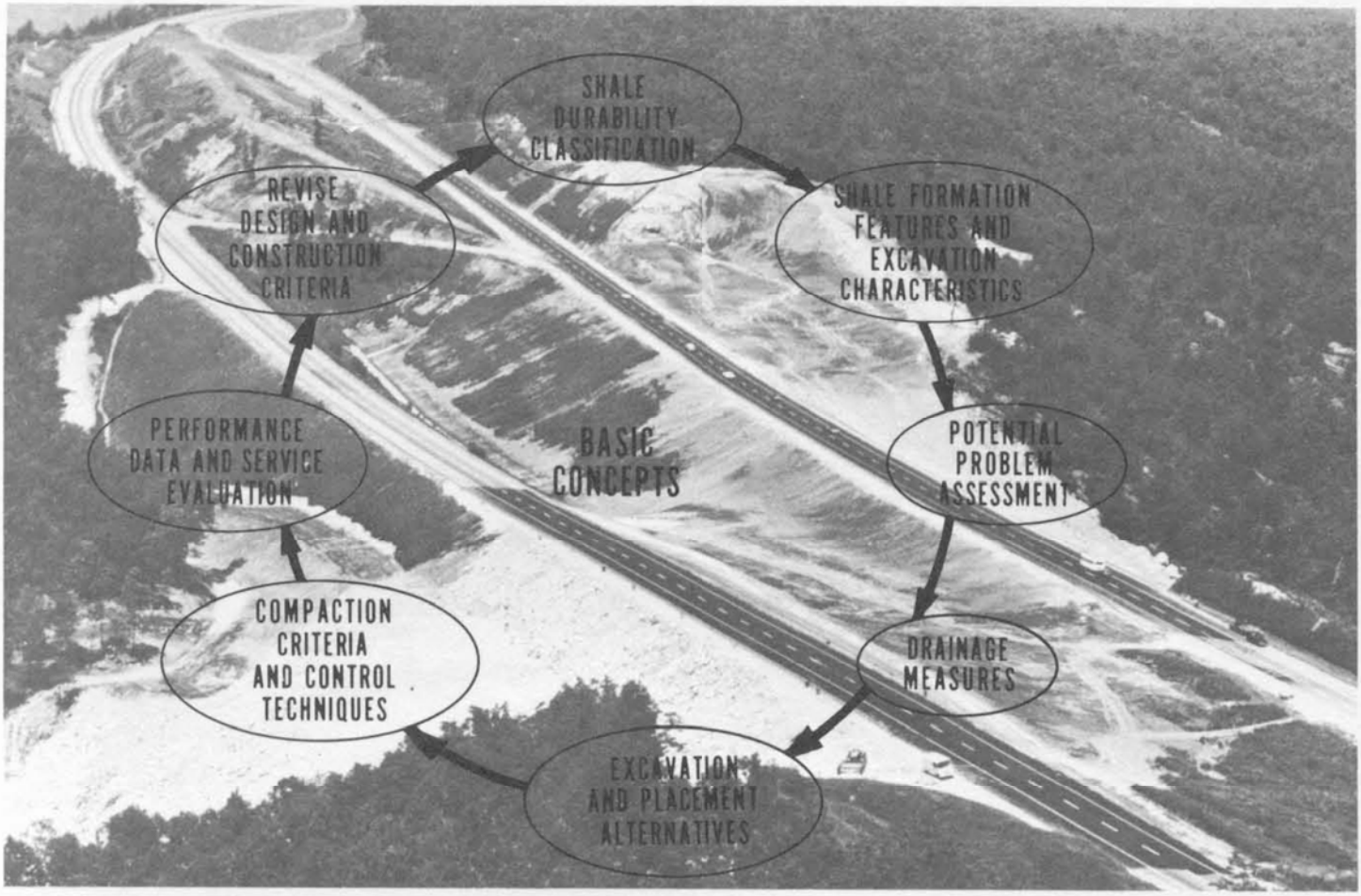
# APPENDIX C

# FEDERAL HIGHWAY ADMINISTRATION REPORT FHWA TS 80 219 DESIGN AND CONSTRUCTION OF SHALE EMBANKMENTS: SUMMARY APRIL 1980

TerraCosta Consulting Group

FHWA-TS-80-219

# Design and Construction of SHALE EMBANKMENTS: Summary




April 1980

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration
Offices of Research and Development
Implementation Division

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This report was prepared by the Department of the Army Waterways Experiment Station. Mr. W. E. Strohm was the principle investigator.

CONTENTS

# DESIGN AND CONSTRUCTION OF SHALE EMBANKMENTS

## SUMMARY

## INTRODUCTION

### The Problem

Construction of the modern highway system has required large, high embankments using economically available fill from adjacent cuts or nearby borrow sources. Because of their widespread occurrence, shales and other weak, fine-grained sedimentary rock (siltstone, claystone, mudstone, etc.) were the main source of fill for many embankments from the Appalachian region to the Pacific Coast.

*Excessive Settlement.* The use of shale materials has caused excessive settlements of 1 to 3 ft (0.3 to 0.9 m) in many embankments. Frequent overlaying and raising of bridge abutments have been required to maintain grade. In some shale embankments continuing settlements have led to large slides. The more severe problems have occurred in the East Central States where the climate is humid.

*Costly Repairs.* The lack of early remedial treatment often resulted in expensive repairs, amounting to nearly $2 million at one location for three slides where reconstruction required 18 months and numerous lane closures. Prevention of a large slide was often precluded by the lack of suitable techniques for detecting the source of major distress, defining the cause and existing stability, and determining the most appropriate type of remedial treatment.

*Underlying Cause.* The underlying cause of excessive settlement and slides in highway shale embankments is deterioration of certain shales by infiltrating water with time after construction. Some shales are rocklike when excavated, but when placed as rock fill, slake or soften upon wetting into weak soil. Other shales, often interbedded with limestone or sandstone, break down when excavated, but large-size durable rocks often prevent adequate compaction and large settlements occur upon wetting.

*Main Problem.* The main problem is determining which shales are durable enough to be placed as rock fill in thick lifts and which shales must be broken down and compacted as soil in thin lifts. The absence of proven criteria for classifying shale durability and predicting the long-term performance has led some highway agencies to adopt a conservative approach where all shale materials are treated as soil.

## Local-Level Action

In 1963, a conservative approach of requiring shales to be compacted in 8-in. (0.2-m) lifts was adopted in Ohio after studies of excessive settlements in bridge approach fills and marginal results with stabilization by cement grout. In the early 1970's, following slides on I-74, the Indiana State Highway Commission initiated a cooperative research program with Purdue University. This work led the way in developing criteria for identifying nondurable shales to be compacted as soil from durable shales suitable for rock fill.

During the same period, settlement and slide investigation studies and research on causes of distress were underway at various levels of effort in Kentucky, Tennessee, West Virginia, Virginia, Kansas, Oklahoma, and Montana. While suitable methods for repair of slides were being developed, proven techniques for determining the source of distress, existing and future stability, and treatment methods for unstable slopes were not readily available. In addition, many shale formations were suspected and resulted in overdesign with durable shales often treated as soil. In other cases, lack of reliable criteria and tests for defining nondurable shales often resulted in underdesign and inadequate compaction.

## Research Program

To provide the highway geotechnical engineer with needed technical guidelines, the Federal Highway Administration sponsored a comprehensive four-year research program in 1974. The three-phase program, accomplished at the U. S. Army Engineer Waterways Experiment Station (WES) was completed in 1978. The results are published in five volumes (Figure 1).

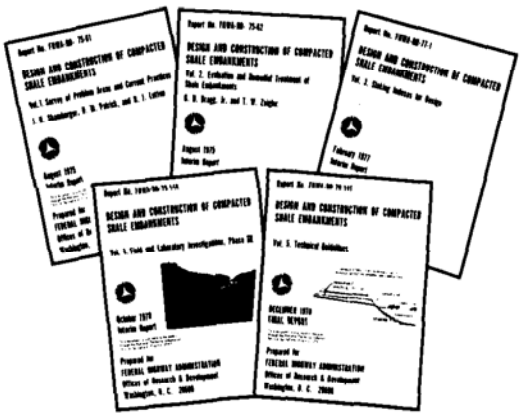



Figure 1. Research Reports.

The technical guidelines draw heavily from the experience of State Highway Agencies, Purdue research, Corps of Engineer work, and WES field and laboratory investigations of selected shale embankments and tests on sampled shales. An advisory group provided valuable guidance during the research work.

## Results

Discussions with highway geotechnical engineers of 16 States revealed fewer problems in the Western States where the younger shales were softer and usually compacted as soil in thin lifts. Problem shale formations, especially in the East Central States (Figure 2), were identified by State agencies from past experience. The need for special provisions was recognized in many

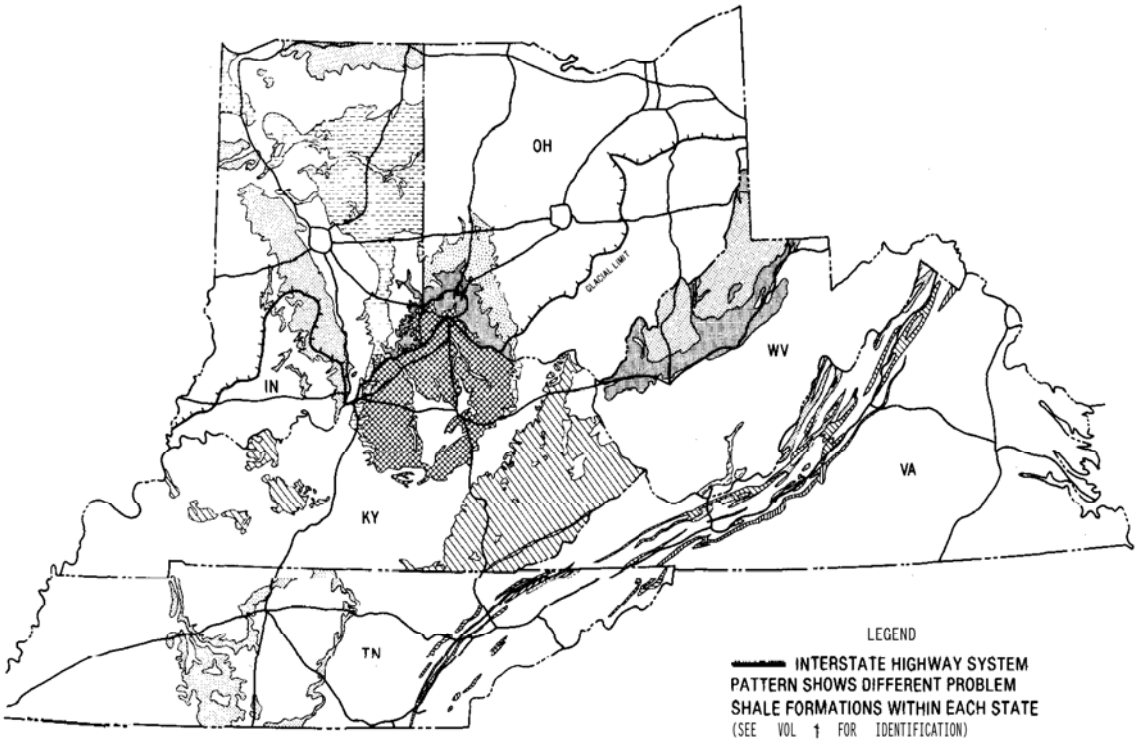



Figure 2. Problem Shale Formations, East Central States.

States. The more severe settlements and slides were related to three main causes:

- Use of nondurable shales as rock fill, which allowed infiltrating surface water or subsurface seepage to progressively slake and soften the shale into small fragments or soft clay.

- Mixing shale and overburden soils with harder rock, which prevented adequate compaction and led to progressive wetting and softening of the fill materials by infiltrating water.

- Lack of adequate benching and drainage of underlying slopes, especially on **side-**hill locations, which caused a progressive buildup of subsurface seepage in the embankment base and **upslope** foundation area.

*Key Findings.* Causes of distress and contributing factors are listed in Table 1. Uncontrolled grading was a major contributor to poor embankment performance. Blasted shale and rock from one cut were often placed in the same lift with overburden soils from another cut. **This** practice produced random zones of pervious rock and shale, and loose to dense soil. Infiltrating water following erratic paths induced nonuniform settlements. Poor compaction of outer slopes caused shallow slides that progressed into deeper slides with time. Lack of benching at the rear of bridge approach fills, when the

remainder of the embankment was constructed, produced weak zones that caused transverse cracking and excessive settlement of the roadway.

Table 1. **Key** Findings.

| CAUSES OF DISTRESS |
|---|
| • Inadequate compaction<br>• Infiltration and saturation<br>• Shale deterioration |
| **CONTRIBUTING FACTORS** |
| • Inadequate foundation benching and drainage<br>• Lack of reliable index tests and criteria for defining nondurable shales<br>• Difficulties in breaking down hard shales and rock prior to compaction<br>• Uncontrolled mixing of soil, shale, and large rock in the same lift<br>• Use of excessive lift thicknesses<br>• Lack of specific compaction requirements and procedures<br>• Lake of adequate **measures** to prevent infiltration of surface water |

*Good Practices.* Controlled grading and the use of test pads to develop compaction procedures were the two main practices that were evident for good embankment performance. Other basic criteria for good shale embankments are:

- Increased use of foundation benching and drainage.

  Define nondurable shale strata in cut sections.
- Increased use of selective excavation and placement to separate nondurable shale from durable shale and rock.
- Increased use of durable hard shale and rock for drainage layers at base and/or in outer sections of embankments.
- Increased compaction of nondurable shale and soil in thin lifts.
- Increased use of impervious layers beneath median and shoulders, paved median ditches, and shoulder curbing to prevent surface water infiltration.

*Index Tests.* Two index tests, found suitable for identifying nondurable shales, are the slake-durability test and a simple jar-soaking test. With these tests, criteria for classifying shales into the **soillike** or **rocklike** category were recommended.

*Special Tests.* The WES research also developed a soaked-compression index test to predict settlement potential of compacted shale. The soaked compression was related to density and the slake-durability index, and provides the geotechnical engineer a means of estimating the compaction required to minimize settlement.

*Existing Embankments.* Rapid techniques for the evaluation of existing shale embankments includes air photo surveys, ground inspections, and the use of a pressuremeter test to measure in situ strengths of distressed sections. Early application of drainage measures was found to be an economical means of preventing large slides.

### Scope of Report

Guidelines for the use of shales in new construction, evaluation of existing embankments, and remedial treatment of distressed shale embankments are briefly described in the remainder of this summary report.

## GUIDELINES FOR NEW CONSTRUCTION

The successful use of shales in highway embankments requires adequate compaction in all fill materials and sufficient drainage to prevent harmful saturation after construction. These two main requirements are not easily achieved, especially in shale formations with complex and variable type stratification. Thus, several basic concepts, unique to shale embankments, should be considered in planning new construction.

### Basic Concepts

The most important concept in planning highway projects across shale formations is the identification of nondurable shale (siltstone, claystone, etc.) strata. The location and extent of these layers in relation to durable shale and rock layers have a direct influence on the proper use of materials to achieve a stable embankment. For example, thick strata of nondurable shale and sandstone can be excavated separately with the shale compacted as soil in the central portion and sandstone placed as rock fill shells with steep slopes. Hard nondurable shale requires extra blasting during excavation or use of impact equipment on the fill to break down large pieces for proper compaction in thin lifts.


Figure 3. Basic Concepts.

*Drainage.* Where subsurface seepage feeds into the embankment area, durable rock should be used for a rock fill drainage layer on benched slopes under the fill to prevent harmful saturation and high seepage levels. In shale formations containing steeply dipping layers or thin layers of interbedded shale and limestone, selective excavation and placement are impractical, and all materials need to be broken down during excavation and placement for compaction as soil. In this case, underdrains on benched foundation slopes will require sand or gravel backfill.

*Formation Features.* The main basic concepts in logical order, as shown in Figure 3, start with the geotechnical investigation to obtain core samples of shales for

durability classification and to provide a complete picture of shale formation features in each cut. The features include hardness, thickness, and dip of major strata with depth and seepage conditions below the grade line. Excavation characteristics related to rippability and blasting requirements are important in determining special procedures needed during excavation and placement to break down hard materials and limit the maximum rock size to the specified lift thickness.

Potential *Problem* Evaluation. Information on shale durability classification, shale formation features, and excavation characteristics should be compared with recent design, construction, and service performance experience for similar projects. This process leads to a logical evaluation of potential problem areas and the need for special features, such as extensive foundation benching and drainage, special excavation and placement procedures, and compaction requirements, to meet settlement limitations and stability requirements associated with the allowable risk and type of project. Major embankments in areas of high rainfall may require impervious layers beneath the median and shoulders, pavement subdrains, paved median ditches, and shoulder curbing to prevent surface water infiltration.

*Compaction and Control.* Field compaction procedures, should be developed from field test pads during construction when experience is lacking for a particular shale formation. Important construction control techniques involve visual inspection and tests to ensure compliance with special provisions, periodic air photos to document construction practices, and visits by the **geotechnical** staff to solve unforeseen problems.

*Monitoring Performance.* Major embankments should be monitored after construction to obtain settlement and stability performance data. Evaluation of these data against design measures used and construction procedures will provide a sound basis for revision of design and construction criteria and changes to special provisions for future shale embankments.

## Field Exploration and Sampling

During initial studies, all pertinent geologic and soils information available for the project area should be reviewed, and field reconnaissance made to determine the optimum field boring program needed to fill data gaps. Aerial photographs (color, and color infrared photos and thermal infrared imagery) provide valuable geologic information, surface drainage patterns, and subsurface seepage exiting from hillsides along the center line under proposed embankment areas.

*Core Borings.* At least two core borings, in addition to the usual auger borings, are required in each cut or borrow area to obtain unweathered shale samples and to define soil and weathered shale depths and the thickness and inclination of each major strata of different material (shale, claystone, siltstone, limestone, sandstone, etc.). The core

borings should be deep enough to detect seepage layers below grade in cuts that drain towards adjacent embankment areas (Figure 4).

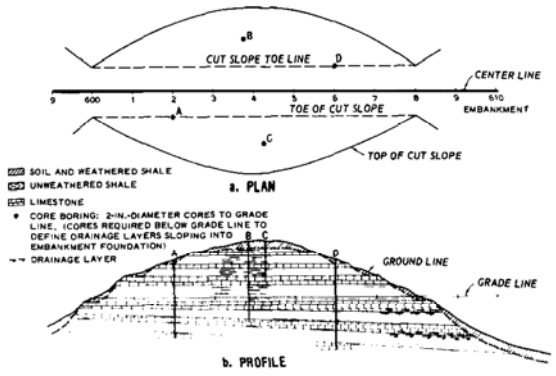



Figure 4. Location of Core Borings.

*Shale Testing.* Core samples of each major shale layer are needed for durability index tests, natural water content (if cores are sealed when recovered), and compaction tests. Special soaked-compression index tests and strength tests on compacted shale samples may be needed for critical embankments over 50 ft (15 m) high. These test requirements, estimated during the preliminary design phase, may require additional core borings to obtain a sufficient amount of shale cores.

*Seepage Conditions.* Measurement of groundwater variations before and after heavy rains can be used to estimate the amount of subsurface seepage that would enter the embankment foundation. This information is used for estimating foundation drainage locations and size requirements.

## Shale Durability Classification

Selection and testing of shales should be done under the supervision of a geotechnical engineer. Representative unweathered cores of chunk samples from each major shale layer should be tested unless durability and compaction properties have been established for the same shales on another project.

*Durability Categories.* Major shale strata in each cut along the project need to be classified into the following durability categories:

- Soft nondurable - soillike.
- Hard nondurable - soillike.
- Hard durable - rocklike.

The hard nondurable category is needed to define those shales to be compacted as soil in thin lifts. Thus, hand shales will require extra blasting or processing to break them down for proper compaction.

*Index Tests.* The two primary tests for durability classification are the jar-slake test and slake-durability test. The simple jar-slake test can be performed on many core pieces as a rapid screening test. Ovendry pieces are soaked in water, and a jar-slake index is assigned using the descriptive behavior noted in the chart. An $I_J$ value of 1 or 2 indicates an obvious soft nondurable shale. Values greater than 2 require slake-durability testing.

| JAR-SLAKE INDEX | |
|---|---|
| $I_J$ | Descriptive Behavior |
| 1 | Degrades into a pile of flakes or mud |
| 2 | Breaks rapidly and/or forms many chips |
| 3 | Breaks rapidly and/or forms few chips |
| 4 | Breaks slowly and/or forms several fractures |
| 5 | Breaks slowly and/or develops few fractures |
| 6 | No change |

In the slake-durability test, ten 3/4- to 1-in. (19- to 25-mm) pieces of ovendry, unweathered shale are placed in a wire-screen drum that is submerged in water and rotated at 20 rpm for 10 minutes (Figure 5).

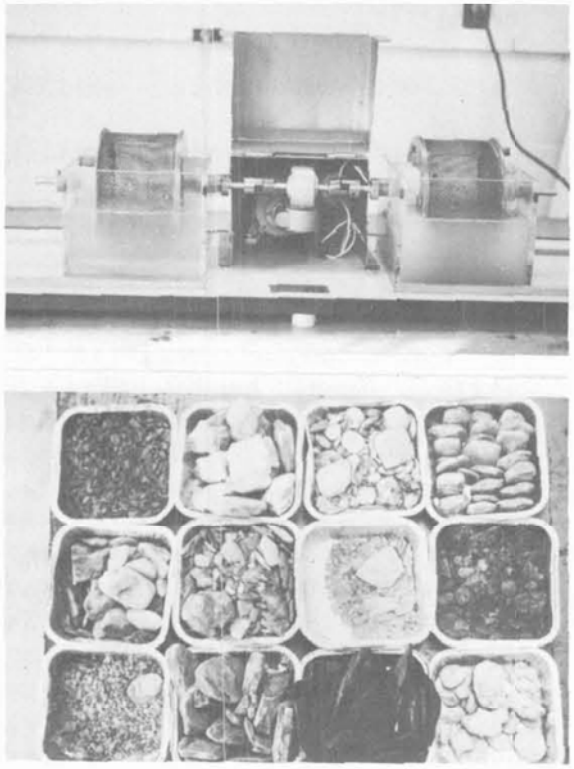


Figure 5. Slake-Durability Apparatus and Different Shales After Testing.

The procedure is repeated on the material remaining in the drum, again after oven-drying. The slake-durability index $I_D$ is the percent of material retained after the two-cycle test.

$$I_D = \frac{\text{Dry wt after 2 cycles}}{\text{Dry wt before test}} \quad 100$$

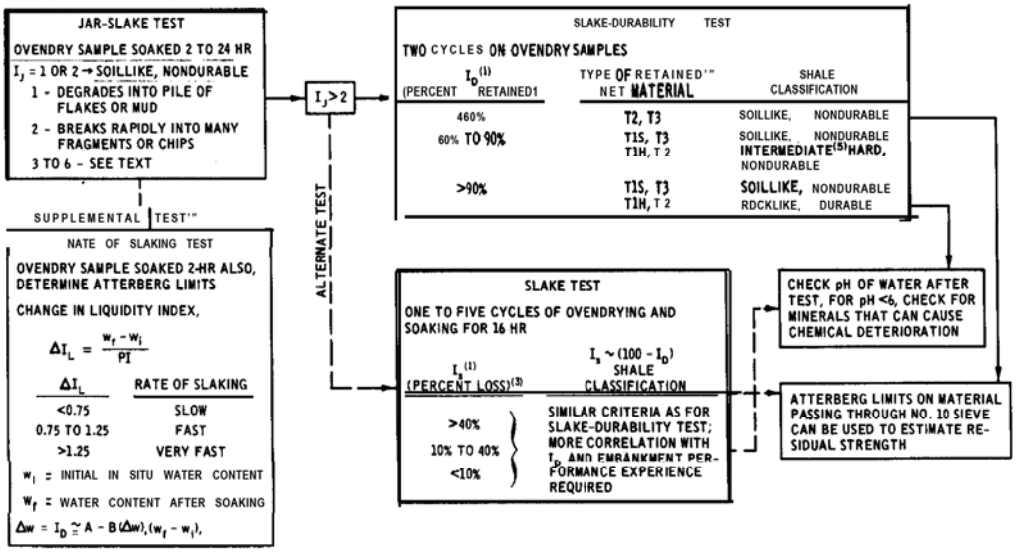
The total test time is about two hours when a rapid means of drying (such as a microwave oven) is used. Low $I_D$ values of 10 to 60 percent indicate low durability, i.e., high susceptibility to deterioration. However, this index does not indicate hardness nor type of breakdown. Since an $I_D$ value near 100 percent for these conditions would be misleading, supplemental letter-number groups are used to denote the condition of the retained wet material.

*Classification.* Suggested durability classification criteria are shown in Figure 6. A supplemental and an alternate test are included. Although these tests take longer to perform, they each have an advantage. The rate of slaking test can aid in defining hard nondurable shale, and the slake test requires only a glass funnel, beaker, and filter paper.

In all tests, a low pH of the water after testing hard shales indicates an acid shale and possible chemical deterioration by alteration of minerals. For example, the mineralogy of dark shales should be checked for chlorite. Soaking of hard dark shales in a sulfuric acid solution can also be used to define potential chemical deterioration.

### Shale Property Tests

*Compaction.* Shales classified as soillike require compaction tests




NOTE: (1) DIFFERENT LIMITING VALUES MAY NE JUSTIFIED ON BASIS OF LOCAL EMBANKMENT PERFORMANCE EXPERIENCE.

(2) TYPE T1 - NO SIGNIFICANT BREAKDOWN OF ORIGINAL PIECES.
TYPE T1S - SOFT, CAN NE BROKEN APART ON REMOLDED WITH FINGERS.
TYPE T1H "AND, CANNOT NE BROKEN APART.
TYPE T2 - RETAINED PARTICLES CONSIST OF LANGE AND SNALL "AND PIECES.
TYPE T3 - RETAINED PARTICLES ARE ALL SNALL FRAGMENTS.

"USING NO. 10 SIEVE (2 MM)

(4) CAN BE PERFORMED ON JAR-SLAKE TEST SAMPLES IF IN SITU MATURAL WATER CONTENT IS KNOWN. PI SENSITIVE TO DEGREE OF PULVERIZATION.

(5) REQUIRES SPECIAL PROCEDURES TO ASSURE GOOD DRAINAGE AND ADEQUATE COMPACTION (95% T-W) FOR LOOSE LIFT THICKNESS UP TO 24-IN. (0.6-M) MAXIMUM.

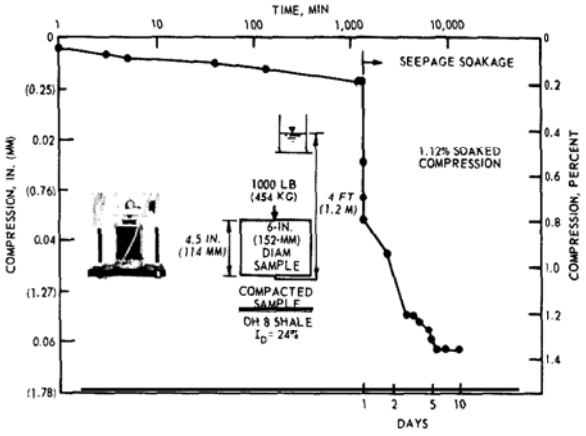
Figure 6. Recommended Durability Index Tests and Suggested Classification Criteria for Shales Used In Highway Embankments.

to define their optimum water content and maximum density for field compaction control of end result specifications or for developing compaction procedures from test pad construction. The American Association of State Highway Officials (AASHTO) T-99 (Method D) compaction test can be used for shales with oversize particles by scalping plus **3/4-in. (19-mm)** material from the test sample. New material should be used for each test point.

*Compression.* Soaked-compression index tests on compacted samples can provide a relative measure of expected performance for large embankments where settlement of fill materials is of major concern. Compacted samples at different densities are subjected to a vertical pressure equivalent to that for one-half the fill height, then allowed to soak until the measured compression stabilizes. The effect of increase in density and decrease in compression can be related to the slake-durability index and used to evaluate the feasibility of greater compaction in reducing settlements to tolerable limits.

***Permeability.*** Measurements of permeability can also be made during the soaked-compression test. The permeability at a given density provides an indication whether

the compacted shale will act as a relatively impermeable barrier to surface water infiltration and also block subsurface seepage water, if foundation drainage measures are not included in the design.




***Shear Strength.*** For special embankments 100 ft (30 m) or higher, shear strength tests on **2.8-in.-(71-mm)** diameter compacted shale samples using modeled gradations may be warranted. The results would be needed for stability analyses and possible finite element analyses to predict settlement and lateral deformations.

## Data Storage

Consideration should be given to a computerized system of data storage and retrieval for geologic and test data on shales to develop an expanding source of information for future projects. The advantages include reduction of testing for common shale formations and the potential for correlating index properties with compaction and settlement properties, shear strength, excavation characteristics, and service performance.

## Potential Problem Assessment

***Evaluation.*** A realistic assessment of potential problems (other than poor foundations) with embankments constructed in shale formations requires a thorough understanding of the causes of distress and the role of contributing factors listed in Table 2.

***Judgment.*** Although a number of problem shale formations have been identified, the influence of construction procedures and the relative importance of factors that contribute to inadequate compaction, saturation, and shale deterioration are not well defined. Consequently, considerable judgment is required in assessing potential problems.

***Service Data.*** A valuable guide can be established if the **slake-**durability index, lift thickness, compaction procedures, and performance data are collected and correlated. An example of a preliminary correlation between slake-durability index and lift thickness is summarized in Figure 7. The criterion was established using performance experience (mainly settlement) for 83 embankments in 15 States. The choice of lift thickness, say 8 versus 24 in. (0.2 or 0.6 m) for a range of $I_D$ values from 40 to 60 percent **in one** cut depends on the consequences of **post-**construction problems and acceptable maintenance costs.

Table 2. Assessment of Potential Problems.

| IMPORTANT FACTORS | POTENTIAL PROBLEM CATEGORY | | | INFLUENCE OF SPECIAL REQUIREMENTS |
|---|---|---|---|---|
| | MINOR | MODERATE | MAJOR | |
| **Formation Features** | | | | |
| Soil and weathered rock depth: | Thin, less than 1.5 m | Shallow, 1.5 m | Deep, greater than 1.5 m | Benching depth and need to stockpile |
| Shale and rock layer thickness: | Less than 0.9 m | 0.9 to 1.5 m | Greater than 1.5 m | Feasible use of selective grading |
| Interbedding of shale and limestone: | None | Few thin beds | Many thin beds | Breakdown of rock for compaction |
| Dip of layers into fill areas | Horizontal | Five-degree dip | Ten-degree dip | Potential seepage into embankment |
| Joint spacing: | Wide, greater than 3 m | Medium, 1.5 to 3 m | Close, 0.3 to 1.5 m | Size of excavated rock |
| Seepage from cut into fill areas: | Low | Moderate | High | Potential wetting of shale embankment |
| **Excavation Characteristics** | Easily ripped | Hard ripping | Blasting required | Maximum size of rock and compaction problems |
| **Shale Durability** | | | | |
| Type of breakdown: | Few cracks | Hard pieces | Slakes to soil | Lift thickness and |
| Classification: | Durable | Durable | Nondurable | compaction requirements |
| **Type Embankment:** | cross valley (through) | Skew | Sidehill | Benching and foundation drainage required |
| **Current Design and Construction Practices** | | | | |
| Slope benching and drainage: | Detailed Plans | Not shown in detail | No special requirements, left to project engineer's judgment | Type and detail of information on plan drawings and sections |
| Rock and durable shale for foundation drainage: | Specified on plans | Hard to identify | | |
| Mixing nondurable shale and soil with rock: | Controlled to prevent | Complex formation | | Fill processing and control techniques |
| Field durability classification of shales | Rapid test used | Time-consuming test | No reliable method | Identifying nondurable shale on plans and use of rapid field test |
| Lift thickness and compaction | specified and controlled | Too few inspectors | Uncontrolled | Compaction requirements, procedures, and control techniques |
| **Performance Experience** | | | | |
| Settlements | less than 0.18 m | 0.18 to 0.3 m | Greater than 0.3 m | Cost of maintenance versus construction costs |
| Slope stability | No problem | Shallow slides | Large slides | Cost of repairs or reconstruction and lane closures |

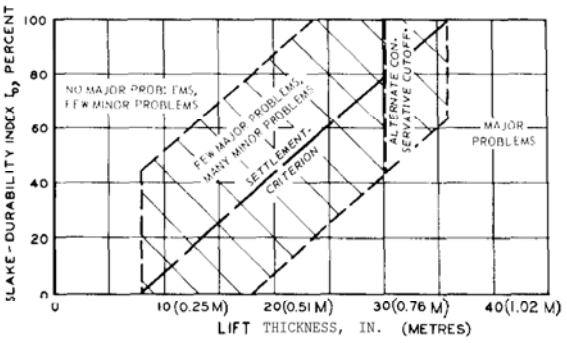



Figure 7. Preliminary Correlation Between Slake-Durability Index and Lift Thickness.

*Past Experience.* Considerable experience with shale construction projects and a good knowledge of formation geology and excavation characteristics are required to determine, for example, whether normal construction practices will cause undesirable mixing of nondurable shale and soil with rock because of variable stratigraphy and lead to unacceptable settlements, Valuable information on specific projects and regional conditions is contained in State Highway Agency internal reports on slide investigations and repairs and in research reports and published papers by State geotechnical engineers. The greater the detail on geologic conditions and shale durability developed during the project field investigation, the less will be the degree of conservatism required in assessing

potential problems and the need for extensive use of special measures.

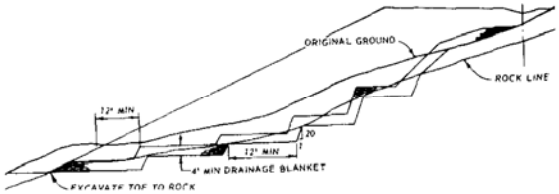
*Problem Definition.* Problem areas should be well defined before the final alignment is established. Where the final alignment or grade cannot be shifted to avoid or minimize the problem, extensive stabilization measures, such as deep drainage trenches and/or horizontal drains or retaining structures, may be required on hillside locations to avoid a costly failure after construction. Complex geologic conditions in cuts may require extra blasting to reduce all materials to an acceptable size for compaction as soil in thin lifts. In a cut-fill transition area, spring drains, subdrains, and benching may be required to drain excess seepage from pervious strata that are dipping out of the foundation surface.

Along **sidehill** problem locations deeper than normal borings, rock coring, and sampling may be required over a wider area to define depths of weak materials, stratification sequence, bedding inclination, and groundwater seepage conditions that will affect the stability of the foundation area. In geologically complex cuts, more extensive explorations and sampling may be needed to define stratification sequence, bedding orientation and inclination, spacing of joints, fractures and bedding planes, groundwater depths, and shale durability.

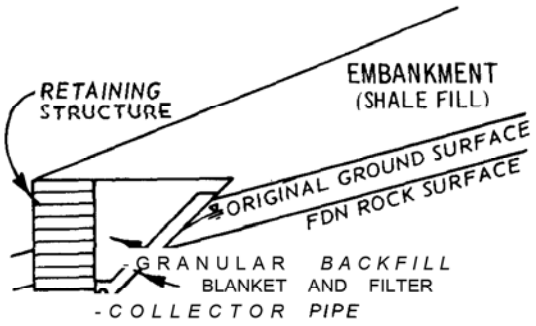
## Special Design Measures

*Special Features.* Special design features especially for problem locations, such as sloping ground on deep weathered material, narrow right-of-way in areas of high fills, and areas of excessive seepage, include the following:

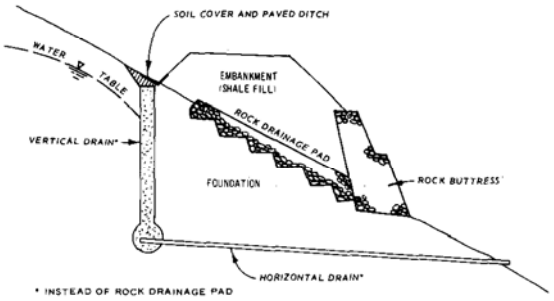
- Foundation benching and rock drainage blanket




- Berms
- Retaining structures
  - Rock buttress
  - Reinforced earth wall
  - **Gabion** and crib walls

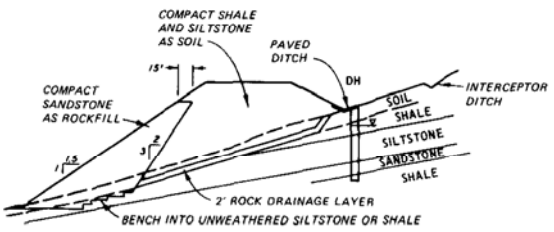



- Drainage measures
  - Underdrains
  - Rock drainage pad
  - Horizontal drains
  - Vertical drains




*Drainage.* Drainage measures are needed for all retaining structures and require sand or gravel filters of the proper gradation to prevent movement of soil particles out of the shale fill or foundation. Otherwise, clogging of the drains and piping in the fill could cause detrimental settlements. Filter fabrics can be substituted for sand as a filter component between **soillike** fill and gravel drainage material when sand placement is uneconomical.

*Special Provisions.* Special excavation, selective grading, foundation benching and drainage, and compaction provisions may be necessary for major embankments to ensure stability and prevent settlement. In areas of deep weathering, stockpiling may be necessary to obtain durable shale and rock for drainage pads at the base of the fill. Special *cross* sections in the plans may be needed to designate the different layers in a cut for use in certain sections of the fill.




Special compaction requirements including **types** and minimum weight of compaction equipment, maximum lift thickness, number of coverages, and processing procedures may be necessary to achieve require densities for hard nondurable shales. Where experience is lacking for a shale formation, test pad construction should be specified to develop required compaction procedures.

*Material Properties.* Selection of compaction settlement and strength properties for long-term performance evaluation can be obtained from laboratory tests on compacted samples or estimated from index tests and past experience. An example of settlement related to slake-durability index and the effect of increased density is shown in Figure 8.

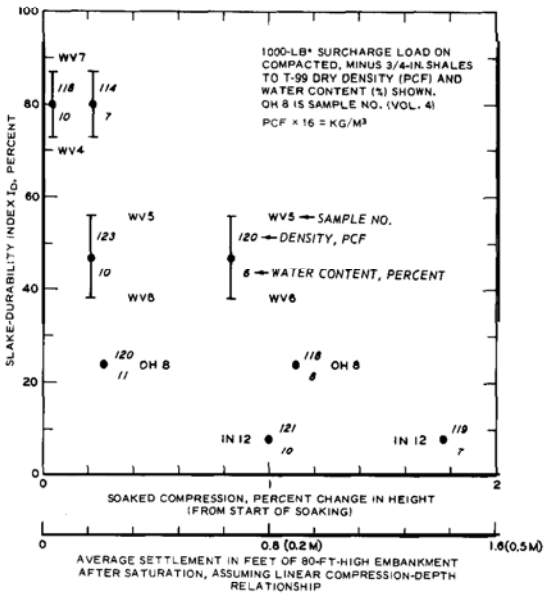



Figure 8. Soaked Compression of Minus **3/4-in.** Compacted Shale Samples Related to Slake-Durability Index.

The shear strength needed for a factor of safety of 1.2 is related to slope height in Figure 9.

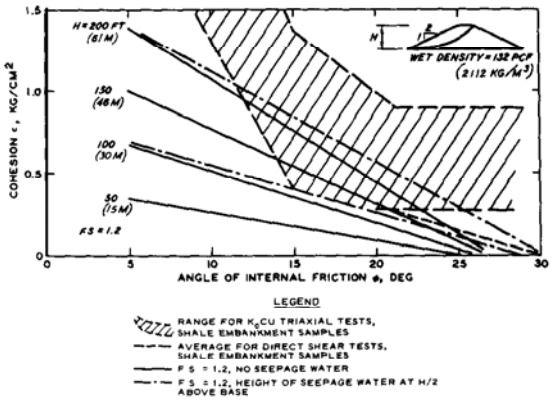



Figure 9. Shear Strengths to Slope Height for FS = 1.2 and 2:1 Side Slopes.

Shales compacted to 95 percent of AASHTO T-99 maximum densities generally will have adequate strength for 2:1 side slopes. Compaction in excess of 95 percent of AASHTO T-99 maximum density may be required for coarse-graded nondurable shales that often develop large strains before the maximum shear strength is developed. Flatter slopes, such as 3:1 side slopes, may be required for bridge approach fills to reduce lateral deformations and associated settlements.

### Construction Grading

*Grading Sequence. The* overall grading sequence is a major consideration in construction of shale embankments to achieve selective excavation and placement as directed by the plans and special provisions. Surface soil, weathered shale, and nondurable shales from cuts and foundation benches need to be compacted in thin, relatively impervious layers in cross-valley fills where foundation drainage is not required or in the central zone of fills above a drainage layer.

Rock, such as sandstone and hard durable shales, needs to be used for drainage layers. These layers are needed on sidehill benched foundation slopes and transverse slopes beneath the cut-fill transition to intercept and drain subsurface seepage.

The usual procedure of placing materials as they are excavated from a cut in the next fill from the bottom up is not suitable for shale embankments.

Preconstruction training of the project engineer staff and inspectors should cover the type and extent of selectively grading and the major items requiring special control as outlined below.

### Construction Control

*Foundation Preparation. Key* items for foundation preparation control are:

1 - Visual inspection to ensure benches are cut into unweathered shale or rock in proper sequence.

2 - Checking durability of rocklike shales used for drainage rock by the simple jar-slake test or point-load test (Figure 10) if preconstruction correlation with slake durability is established.

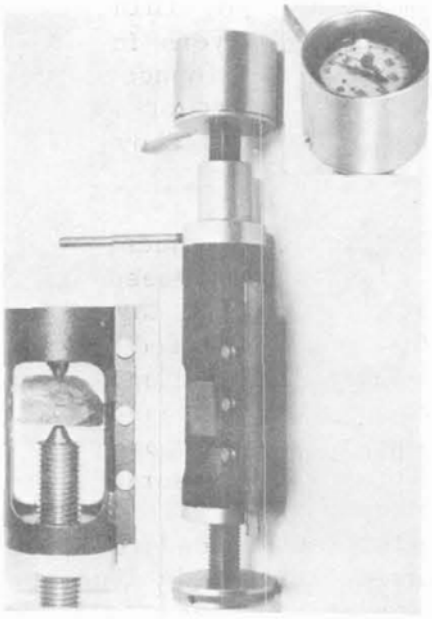


Figure 10. Point Load Tester.

3 - Preventing excess fines in drainage rock.

4 - Ensuring adequate filter zone on top of rock drainage layer by surface degradation under dozer tracks or use of carefully installed filter fabric.

5 - Proper installation of fabricated bench drains and spring drains.

*Excavation Procedures.* Control of excavation requires the following:

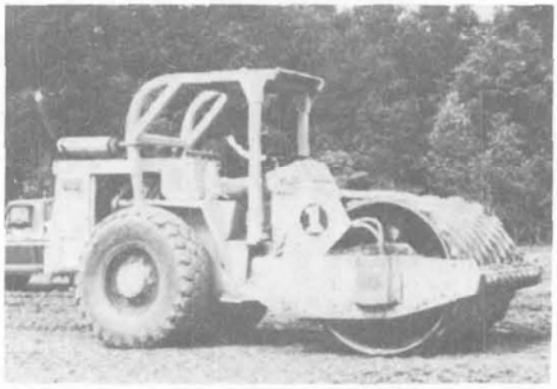
1 - Adequate fragmentation or ripping of nondurable shale for proper compaction.

2 - Separate excavation of different layers and routing to designated parts of fill as required by special provisions.

3 - Checking durability classification of shales.

*Compaction Equipment Capabilities.* Compaction equipment should be checked to ensure:

1 - Proper types and weight for further breakdown of nondurable shales and compaction to required densities for lift thickness allowed.



2 - Sufficient amount of compaction equipment to keep up with fill placement rate.

| KEY COMPACTION CONTROL TECHNIQUES |
|---|
| ● Visual inspection: |
| • Material type, gradation, fill processing |
| • Type and number of compaction machines |
| ◦ Maximum speed |
| ◦ Number of coverages |
| ● Lift thickness measurement |
| ● Moisture-density testing |
| ● Geotechnical staff visits |

*Compaction Procedures.* Proper compaction for shales require the following steps:

1 - Following established practices for test pad construction and testing when used to establish procedural-type compaction requirements.

2 - Breaking down or removing excess large rock from shales compacted as soil.

3 - Disking and adding water as required for proper compaction of shales.

4 - Checking for proper types of equipment and number of coverages established by special provisions or test pad results.

*Compaction Control.* Control for adequate compaction involves:

1 - Checking material type, lift thickness, water content, and density for end result provisions.

2 - Checking compaction equipment type, weight, lift thickness, number of coverages, and maximum speed for procedural-type provisions.

## Construction of Special Features

Frequent inspection of special feature construction is needed to ensure that critical items listed below are accomplished.

1 - Foundation shear trenches and underdrains: (a) construction in segments to prevent slides in weak foundation materials, and (b) proper type of drainage and filter materials and compaction of backfill.

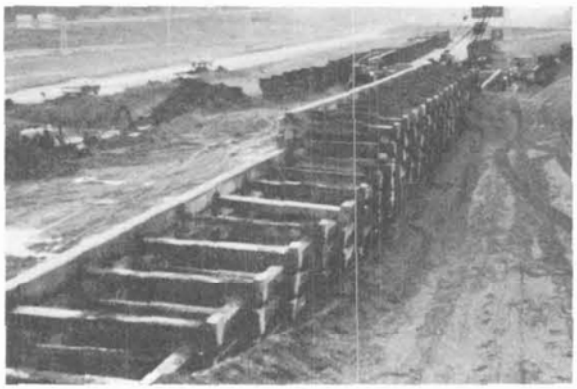
2 - Berms: adequate foundation benching and drainage and proper compaction.

3 - Buttresses: use of hard, durable rock and adequate filter materials along base and behind the buttress to prevent erosion of fill into rock and clogging of drainage paths.

4 - Reinforced earth walls: use of free draining backfill materials (clean sands or gravelly sands).

5 - Gabion or crib walls: use of clean stone backfill and proper filter zone behind wall to prevent erosion of

embankment fill into rock and clogging of drainage paths.



Frequent coordination is required between the geotechnical and construction staffs to solve unforeseen problems such as less than anticipated quantities of durable rock for drainage layers, need for additional test pads to resolve compaction problems, or difficulties in classifying the durability of shale strata.

### Construction Records

Construction records with specific information on actual procedures and compaction equipment used, photographs, and test data are valuable sources of materials. The records can be used with long-term performance results to achieve optimum requirements for drainage measures, compaction procedures, and control techniques for future construction at a minimum cost.

### Post Construction Monitoring

Monitoring of critical shale embankments should be considered for two reasons:

- To detect as soon as possible distress and apply corrective measures.
- To verify predicted performance and provide data for selection of optimum criteria for economical construction of future shale embankments.

Settlement is a major problem, and embankments over 50 ft (15 m) high with low margins of safety should have roadway settlement surveys two to three times a year. Settlement increases would warrant a geotechnical investigation to define the cause.

Major embankments higher than 50 ft (15 m) may warrant additional observations. Installation of piezometers may be needed for seepage detection. Inclinometer surveys may be necessary to detect lateral movements and possible slope failure.

## EVALUATION OF EXISTING SHALE EMBANKMENTS

### Early Detection of Distress

Early detection of distress in shale embankments is important in preventing a large slide. One means of detecting distress is by periodic low-level aerial color photography. Color photographs for many miles of a highway can be quickly scanned for telltale signs of embankment distress such as pavement overlays, cracks along pavement and shoulders, misaligned guardrail, shoulder and slope sloughing, erroded slopes and surface drains, and seepage areas indicated by unusual plant growth. Selected

embankments can then be inspected on the ground, and the seriousness evaluated.

Distress, in the form of continuing settlement, cracking along pavement edges and shoulders, small shoulder slides, and slope sloughing, has often developed within 1 to 10 years into a large slide requiring expensive reconstruction, especially along sidehill locations. In many cases, the distress has been handled at the District level by maintenance forces without the assistance of the State geotechnical staff until a problem reaches major proportions.

Several States are cataloging distressed shale embankments and assigning priorities according to the seriousness of distress and consequences of failure. The States have established a continuing program of limited field investigations to evaluate the distressed embankments in order of priority and available funds.

## Cause of Distress

The primary cause of shale embankment distress is saturation and progressive softening and deterioration of nondurable shales (often mixed with rock and soil) by surface water and/or subsurface seepage that enters the embankment during periods of prolonged rainfall. Because of the heterogeneous mixture of shales, rock, and soil in many shale embankments, infiltrating water follows erratic flow paths, depending on the relative porosity of different layers and the pattern of cracks caused by settlement and deformation. Consequently, it is often difficult to define the pattern of infiltration, location of soft shale or soil zones, and the extent of distress without extensive subsurface investigations.

## Evaluation

The major objective in evaluating the future behavior of distressed shale embankments is to determine whether settlements will eventually stop or will continue and develop into a large slide. The first step should be a thorough field evaluation by a geotechnical engineer to determine the surface extent and seriousness of the distress and the probable effectiveness of immediate remedial measures in reducing further distress.

An immediate measure that is usually inexpensive is improvement of surface drainage to reduce infiltration. Other steps in the evaluation process for major embankments include the following:

- Review of available information on (a) the stratification and attitude of bedding at the cut-fill transition and uphill of sidehill embankments, (b) groundwater seepage patterns, (c) placement sequence and amount of mixing of different materials during construction, (d) quantity and durability of shales placed in the embankment, and (e) past experience with similar embankments in the same formation and local area to establish possible seepage sources and locations of weak zones.

- Periodic roadway center-line and cross-section elevation surveys to monitor the rate of settlement and lateral deformation with time.

- Disturbed sample borings to define the depth and thickness of predominant types of materials (i.e., shale, shale chunks and clay soil, soft shale with some limestone, limestone with few shale chunks and sandstone); type and amount of shale deterioration (e.g., softened into clay, fragmented into hard silty chips and gravel sizes, or friable clayey chunks); location of wet or saturated zones; and type of foundation material (e.g., sandstone drainage layer, thick weathered shale, hard shale, and limestone strata).

- Installation of piezometers and/or slope inclinometer casing in selected borings to monitor groundwater levels (or pore water pressures from piezometers) and lateral movements and development of a slip zone.

- Logging of selected borings with portable nuclear moisture density equipment to locate wet, low density zones.

- In situ pressuremeter tests and/or undisturbed sampling and laboratory testing to estimate shear strengths for evaluation of stability.

The scope of the field investigation and monitoring program depends on the type of available information and experience in the area. The results of the evaluation should provide necessary information for decisions on the need for remedial measures and the selection and design of appropriate treatment methods.

| EXAMPLE CRITERIA FOR TREATMENT ACTION | | |
|---|---|---|
| TYPE OF DISTRESS | EMBANKMENT CONDITIONS | ACTION NEEDED |
| Settlement minor (<0.3 m) | Good surface drainage, no cracks or signs of slope instability | Continue to monitor |
| Excessive settlement (>0.3 m) | Eroded ditches; drainage inlets above median surface; embankment does not contain soft clay, wet zones, or high groundwater; no signs of slope instability | Repair and improve surface drainage, overlay and continue to monitor |
| Excessive settlement (>0.3 m) | Same as above but embankment contains soft wet zones or high groundwater, cracks exist along shoulders and slope instability is apparent | Design and install drainage measures, evaluate stability and select major treatment alternatives |

## REMEDIAL TREATMENT OF DISTRESSED EMBANKMENTS

### Drainage Measures

The primary consideration in remedial treatment of shale embankments should be surface and subsurface drainage methods. Drainage methods are an integral part of most remedial treatment methods. Remedial treatment plans should include surface treatment and drains designed to minimize infiltration of surface water. Subsurface drainage is essential in treatment of side-hill and transitional fills. Certain types of subsurface drains can be rapidly installed (i.e., horizontal drains and pumped vertical wells) and are used when temporary (or emergency) support is required. Early installation of subsurface drains, when feasible, can halt embankment distress and prevent an extensive failure. Proven remedial measures are:

- Drainage
  - Surface drains (repairs and additions)
  - Horizontal drains
  - Vertical drains (upslope of embankment)
  - Trench drains at embankment toe
  - Pumped wells (temporary)
  - Drainage blanket (under reconstructed embankment)
- Impervious layer beneath shoulders and median
- Slope-flattening
- Berms
- Shear trenches
- Retaining structures

### Measures for Unstable Slopes

Remedial treatment, in addition to drainage methods, will often be necessary when significant improvement in slope stability is required. Primary consideration should be given to constructing berms. Retaining structures for supporting slope-flattening or berm fills should be considered where right-of-way and/or suitable borrow materials are limited. As a special type of retaining method one or two rows of closely spaced piles can be rapidly installed as a temporary or permanent support (when properly designed and required to maintain traffic).

Where embankment distress is caused largely by foundation shear failure, foundation shear trenches may be required to supplement slope-flattening or berm fills.

Embankment reconstruction involving combinations of material replacement, flatter slopes and berms, and shear trenches should

be considered where large settlements, shear displacements, and/or shale degradation have severely weakened embankment and/or foundation materials.

Specialized stabilization methods, including cement grouting and other cement, lime, and chemical treatments, may be successful under certain conditions. Cement grouting should be considered when embankment settlements have been attributed to a high percentage of interconnected voids. Other cement, lime, or chemical treatments should be considered only on a trial basis at selected sites where risk of failure is minimal and substantial savings over more conventional remedial treatment methods can be realized. Expert guidance is required in design and application of these methods.

## Design Considerations

Design of remedial treatment alternatives should be based on sound geotechnical engineering principles, combined with engineering experience, judgment, and ingenuity. Design investigations should include a review of site evaluation data, past experience, and stability analyses based on in situ strengths.

As an essential feature in the design of economical and effective remedial treatment plans, stability analyses aid in determining the significance and interaction of design variables and provide a quantitative basis for designing remedial treatment methods consistent with engineering judgment and experience. Furthermore, these analyses should be conducted in the design of permanent or temporary support (including temporary stability of slopes excavated in construction of permanent remedial treatment). Acceptable factors of safety can vary, depending on the accuracy and confidence in design parameters and the consequence of failure. Factors of safety for permanent remedial treatment range from 1.25 to 1.5 and from 1.1 to 1.3 for temporary support.

## Construction Control

Repair or reconstruction of shale embankments requires constant inspection to ensure compliance with design requirements. Important items for inspection include the following:

- Proper type materials for drainage and filter layers, trench backfill, berms, and retaining structures such as reinforced earth, gabion, and crib walls.

- Adequate compaction of backfill, berm fill, and fill for retaining structures.
- Spacing and depth of horizontal drains and vertical drains.
- Proper type and placement of drainage backfill for vertical drains and wells.

It is particularly important that shear trenches or trench drains at the embankment toe be constructed in short segments without delay to prevent back slope slides into the excavation before backfilling is started.

### Treatment Monitoring

A plan for monitoring and maintenance should be implemented following repair or reconstruction. The following items should be observed and additional remedial treatment or maintenance applied as necessary:

- Embankment settlement and lateral deformation.
- Pavement distress and surface cracking.
- Erosion and cracking of embankment slopes.
- Function and discharge from surface and subsurface drains.
- Groundwater table elevation and embankment and foundation pore water pressures.

The intensity of site monitoring will depend on site conditions, risk of failure, and critical factors in the remedial treatment design. It is particularly important to check and maintain the operation and effectiveness of subsurface drainage installations to ensure that groundwater levels and pore water pressures do not exceed safe values determined from design stability analyses. Site monitoring and maintenance plans should be modified periodically based on accumulated data.

## FUTURE DIRECTION

The future direction in design and construction of shale embankments should be based on service performance. Refinements in shale durability classification criteria, use of selective grading, extensive benching and drainage measures, and procedural-type compaction provisions based on test pads can best be achieved by evaluation of embankment performance within each State. The ultimate goal should be development of optimum criteria for economical design and construction of stable shale embankments.



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# APPENDIX D

# STABILITY ANALYSES

TerraCosta Consulting Group

# PR NO. 2918 - 5228 Chelsea Street Section 1 - Static -lower-mod

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| c | 1.521 |
| d | 1.521 |
| e | 1.522 |
| f | 1.522 |
| g | 1.522 |
| h | 1.523 |
| i | 1.523 |
| j | 1.523 |

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
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| TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| TMss | 5 | 125.0 | 135.0 | 1750.0 | 38.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
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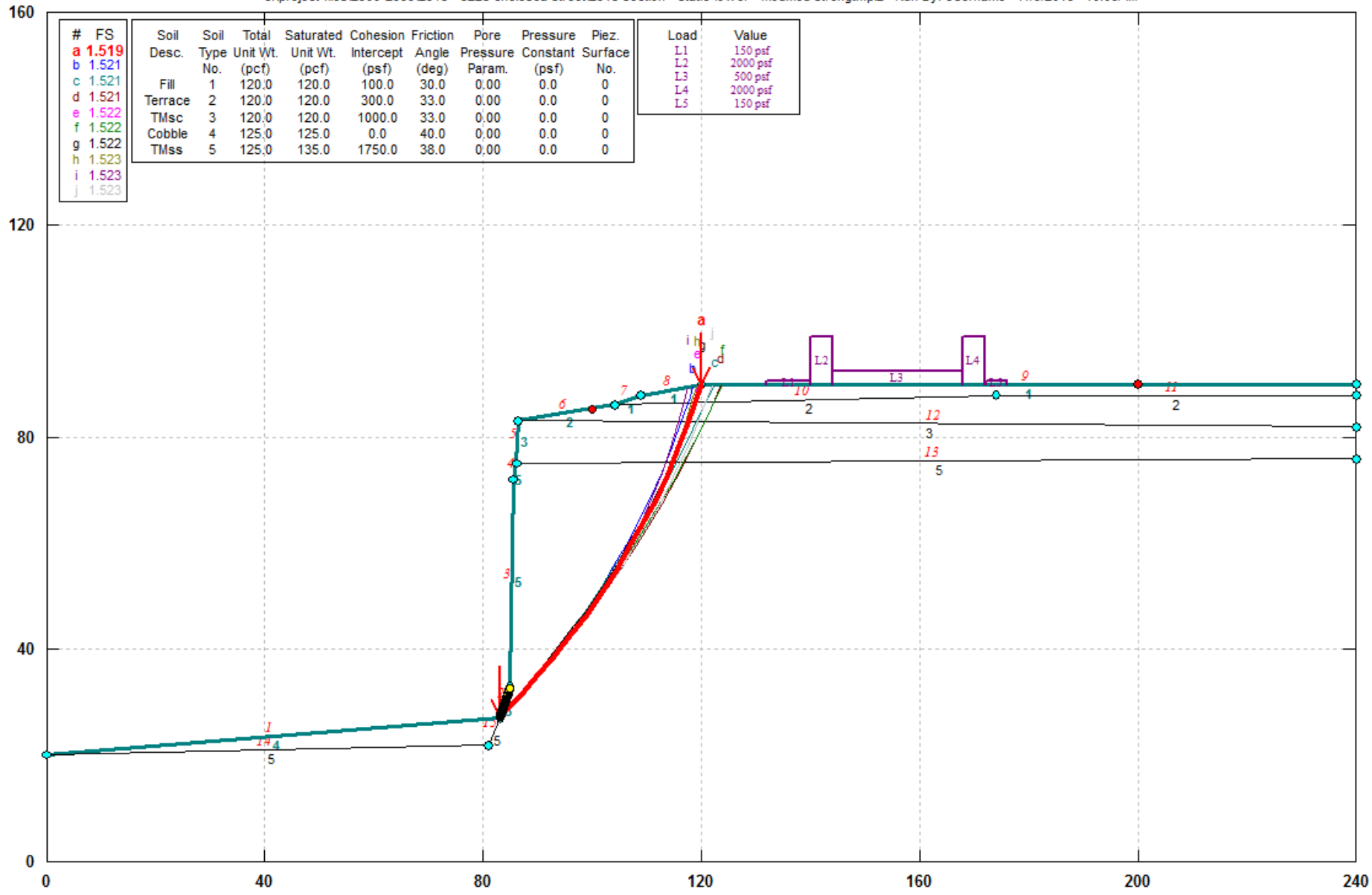
| L2 | 2000 psf |
| L3 | 500 psf |
| L4 | 2000 psf |
| L5 | 150 psf |

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Safety Factors Are Calculated By The Modified Bishop Method

# PR NO. 2918 - 5228 Chelsea Street Section 1 - Static -lower-mod

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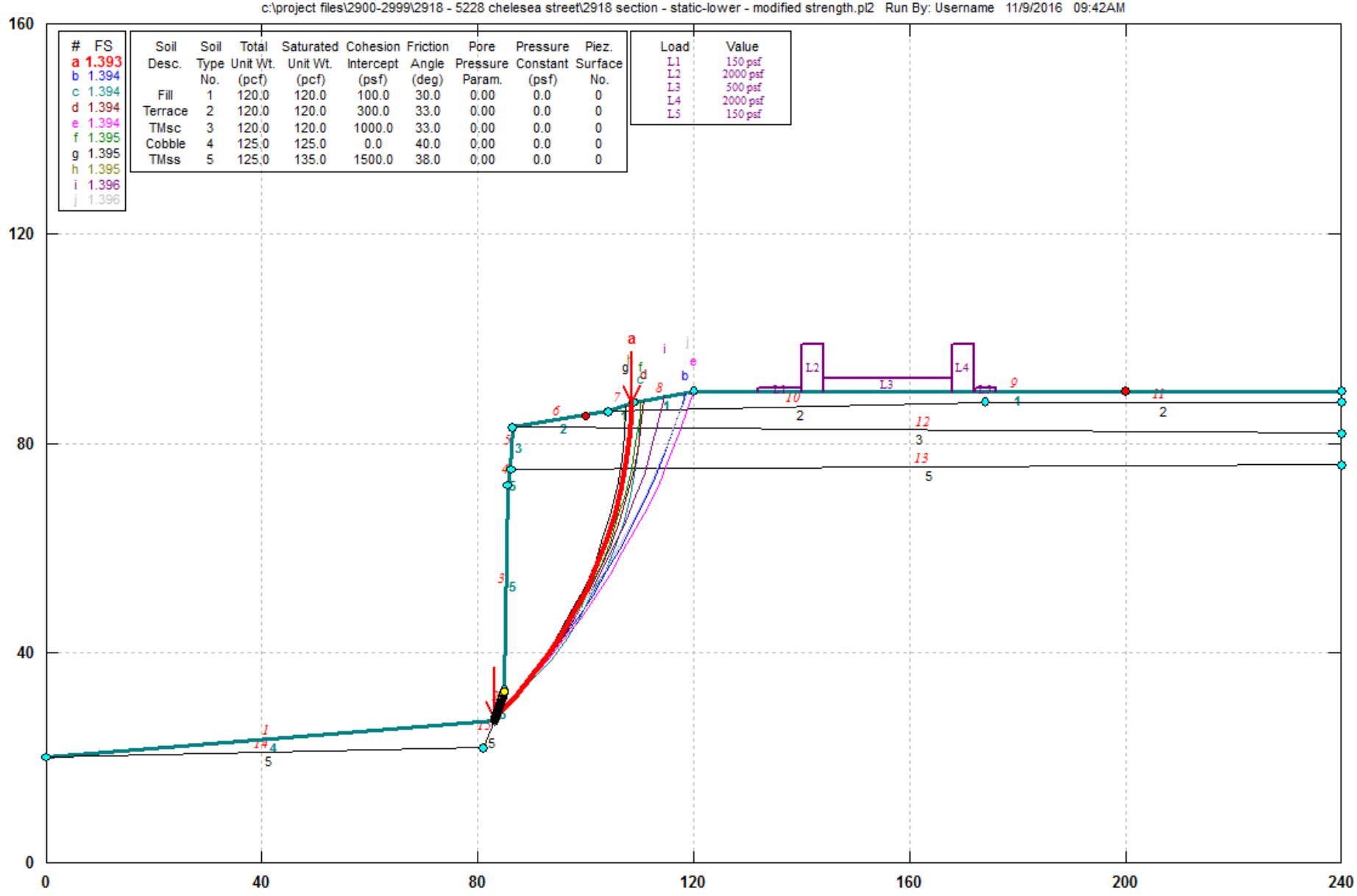
| d | 1.394 |
| e | 1.394 |
| f | 1.395 |
| g | 1.395 |
| h | 1.395 |
| i | 1.396 |
| j | 1.396 |

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Fill | 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| Terrace | 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| TMss | 5 | 125.0 | 135.0 | 1500.0 | 38.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| L1 | 150 psf |
| L2 | 2000 psf |
| L3 | 500 psf |
| L4 | 2000 psf |
| L5 | 150 psf |




GSTABL7 v.2 FSmin=1.393

Safety Factors Are Calculated By The Modified Bishop Method

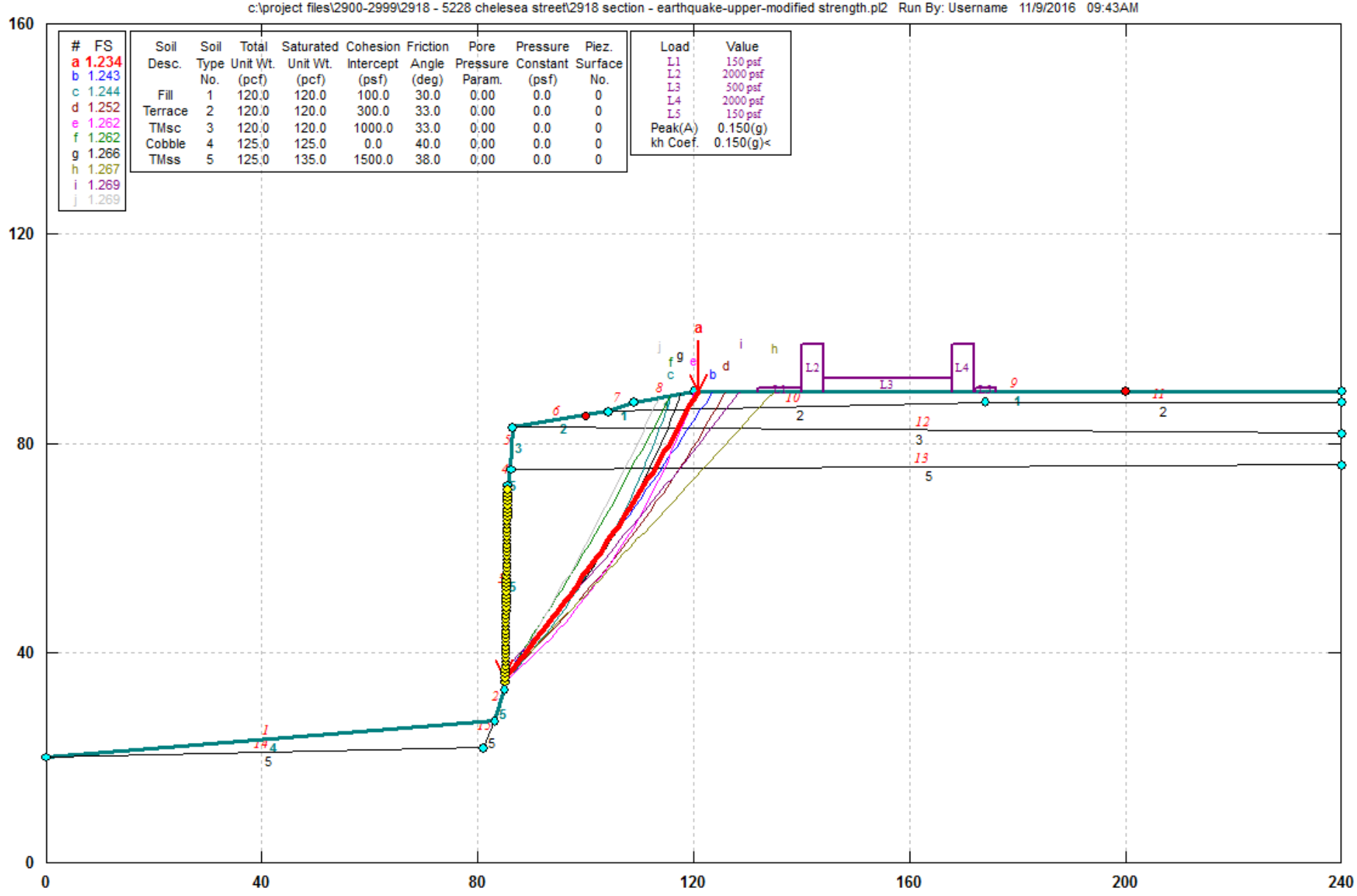
# PR NO. 2918 - 5228 Chelsea Street Section 1 - earthquake -upper-mod

c:\project files\2900-2999\2918 - 5228 chelesea street\2918 section - earthquake-upper-modified strength.pl2 Run By: Username 11/9/2016 09:43AM

| # | FS |
|---|---|
| a | 1.234 |
| b | 1.243 |
| c | 1.244 |
| d | 1.252 |
| e | 1.262 |
| f | 1.262 |
| g | 1.266 |
| h | 1.267 |
| i | 1.269 |
| j | 1.269 |

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Fill | 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| Terrace | 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| TMss | 5 | 125.0 | 135.0 | 1500.0 | 38.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| L1 | 150 psf |
| L2 | 2000 psf |
| L3 | 500 psf |
| L4 | 2000 psf |
| L5 | 150 psf |
| Peak(A) | 0.150(g) |
| kh Coef. | 0.150(g)< |

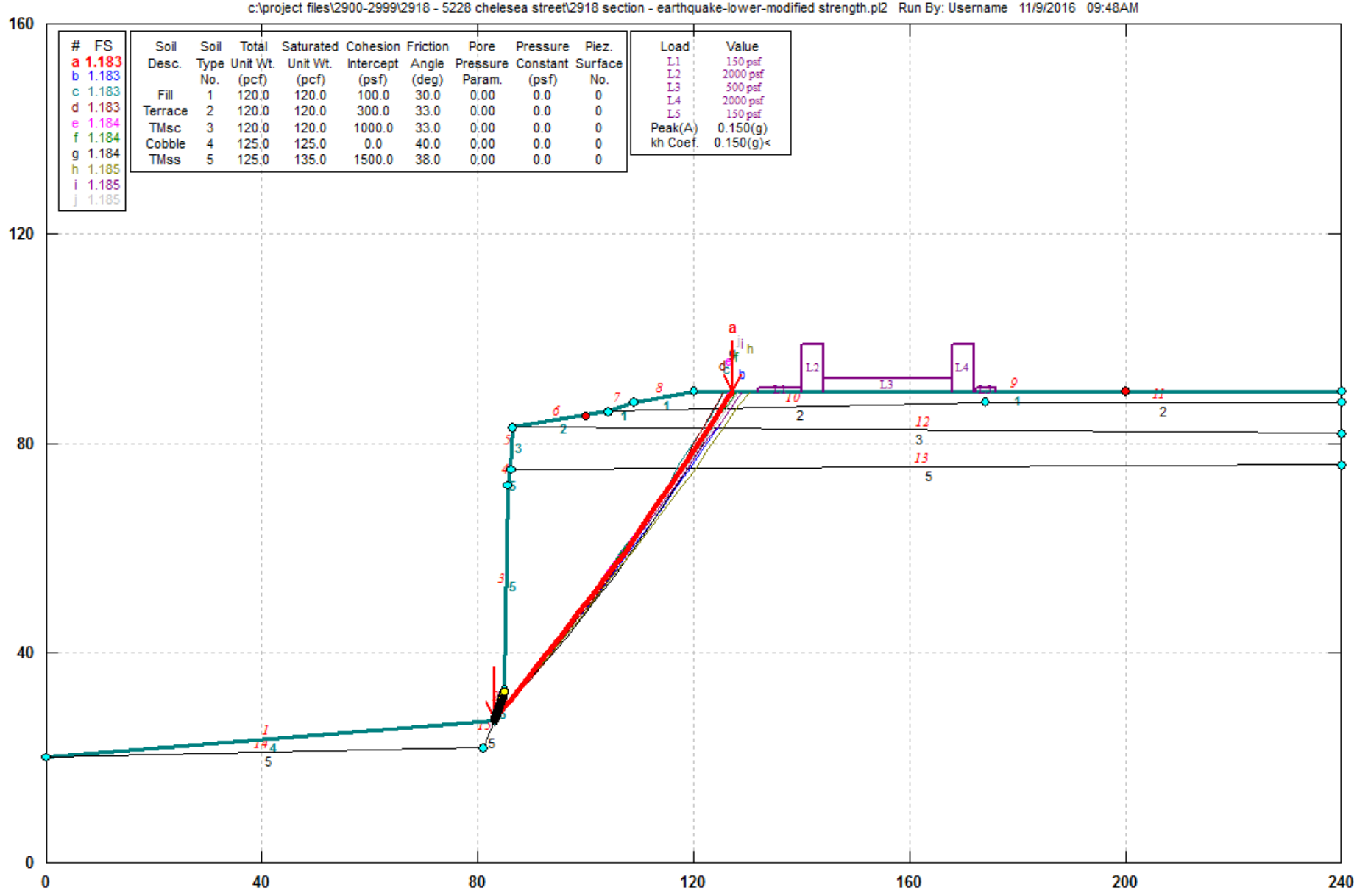



GSTABL7 v.2 FSmin=1.234

Safety Factors Are Calculated By The Modified Bishop Method

# PR NO. 2918 - 5228 Chelsea Street Section 1 - earthquake -lower-mod

c:\project files\2900-2999\2918 - 5228 chelesea street\2918 section - earthquake-lower-modified strength.pl2 Run By: Username 11/9/2016 09:48AM




| # | FS |
|---|---|
| a | 1.183 |
| b | 1.183 |
| c | 1.183 |
| d | 1.183 |
| e | 1.184 |
| f | 1.184 |
| g | 1.184 |
| h | 1.185 |
| i | 1.185 |
| j | 1.185 |

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Fill | 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| Terrace | 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| TMss | 5 | 125.0 | 135.0 | 1500.0 | 38.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| L1 | 150 psf |
| L2 | 2000 psf |
| L3 | 500 psf |
| L4 | 2000 psf |
| L5 | 150 psf |
| Peak(A) | 0.150(g) |
| kh Coef. | 0.150(g)< |

GSTABL7 v.2 FSmin=1.183

Safety Factors Are Calculated By The Modified Bishop Method

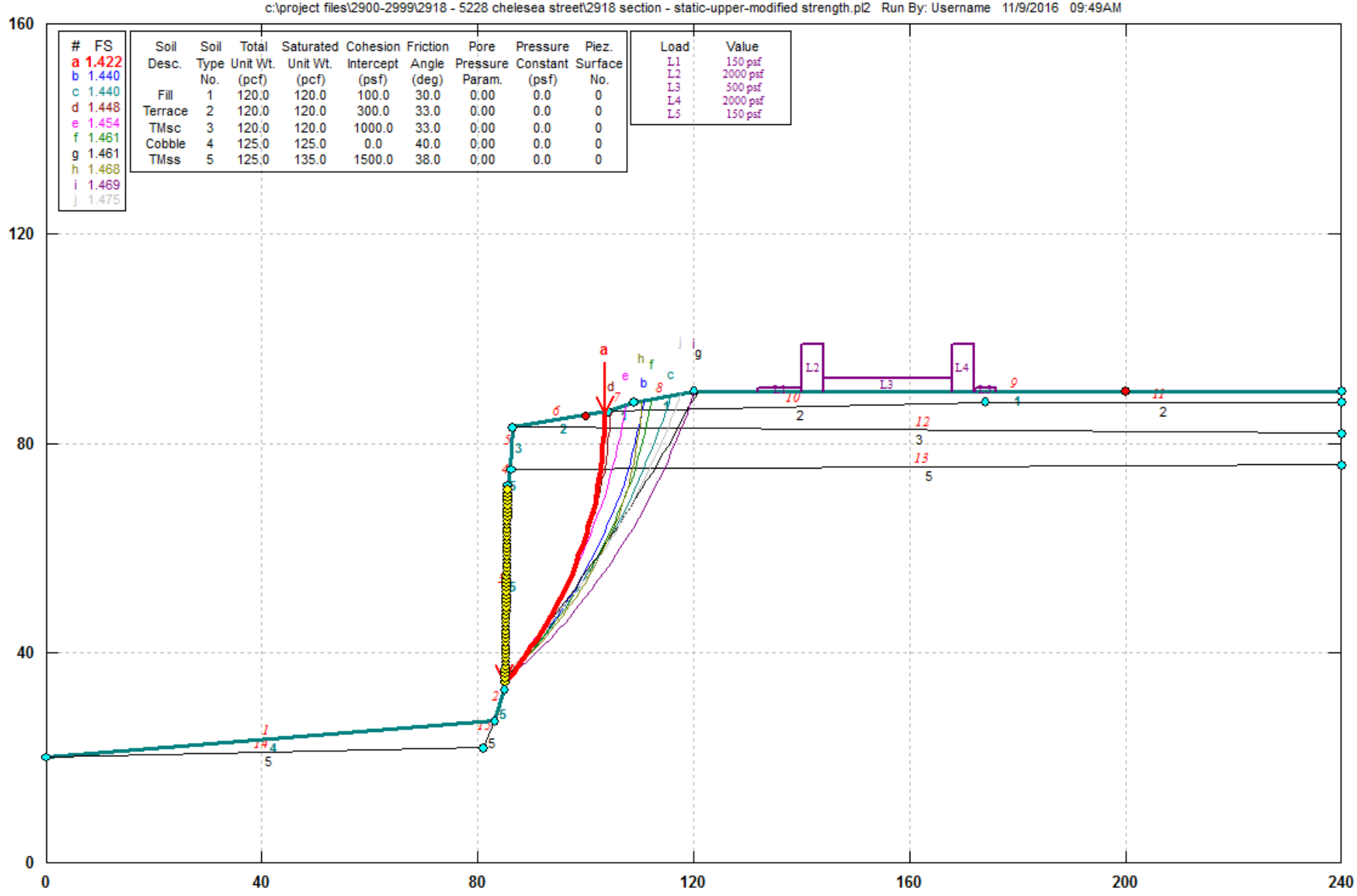
# PR NO. 2918 - 5228 Chelsea Street Section 1 - Static -upper mod

c:\project files\2900-2999\2918 - 5228 chelesea street\2918 section - static-upper-modified strength.pl2 Run By: Username 11/9/2016 09:49AM

| # | FS |
|---|---|
| a | 1.422 |
| b | 1.440 |
| c | 1.440 |
| d | 1.448 |
| e | 1.454 |
| f | 1.461 |
| g | 1.461 |
| h | 1.468 |
| i | 1.469 |
| j | 1.475 |

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Fill | 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0 |
| Terrace | 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0 |
| TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0 |
| Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0 |
| TMss | 5 | 125.0 | 135.0 | 1500.0 | 38.0 | 0.00 | 0.0 | 0 |

| Load | Value |
|---|---|
| L1 | 150 psf |
| L2 | 2000 psf |
| L3 | 500 psf |
| L4 | 2000 psf |
| L5 | 150 psf |




GSTABL7 v.2 FSmin=1.422

Safety Factors Are Calculated By The Modified Bishop Method

# ATTACHMENT C

# USDA STEREOGRAPHIC AERIAL PHOTOS
# AXN-8M-92 AND 93
# AND
# AXN-7M-186 AND 187
# FLOWN MAY 1953

TerraCosta
Consulting Group


4-11-53
AXN-8M-93


4-11-53
AXN-8M-92


5-2-53
AXN-7M-186


5-2-53
AXN-7M-187

TerraCosta Consulting Group

*Geotechnical Engineering*
*Coastal Engineering*
*Maritime Engineering*

Project No. 2918
September 22, 2016

Mr. David M. Lessnick
**D. MARIN DEVELOPMENT**
1900 Western Avenue
Las Vegas, Nevada 89102

RESPONSE TO CITY REVIEW COMMENTS
CYCLE TYPE 2 LDR-GEOLOGY
**5228 CHELSEA DRIVE**
LA JOLLA, CALIFORNIA

CITY OF SAN DIEGO PROJECT NO. 502954
REFERENCE L64A-0038

Dear Mr. Lessnick:

TerraCosta Consulting Group, Inc. (TerraCosta) is responding to the City of San Diego's review comments from Cycle 2 LDR-Geology dated September 15, 2016. For completeness of the record, we have restated the original comments in italics, followed by our response. Only those items requiring responses have been included.

**LDR – GEOLOGY (Jacobe Washburn)**

*Issue No. 2: Submit an addendum geotechnical report or update letter that specifically addresses the proposed development for the purposes of environmental review and the following:*

*Issue No. 3: Update the geologic map (Figure 2) to clearly delineate the 1.5 factor-of-safety line determined via slope stability analyses with respect to the coastal bluff, the location of geologic cross sections, and the geologic structure.*

**Response:** As indicated from the topography shown on the Site Plan (Figure 2 of our July 19, 2016, report), the section selected for analysis is the most critical, as it is near-vertical as shown in cross section on Figure 3 of our report and in our summary stability analyses, with the first summary printout for static conditions having a factor of safety of 2.334. The slope stability summary graphic has been reproduced below (Figure 1) to

facilitate review. As indicated on Figure 1, the two red circles on the bluff top (the first located approximately 14 feet back from the top of the bluff, and the second located approximately 112 feet back from the top of the bluff) represent the computational calculation boundary limits, within which all sample calculations are run. As indicated below, the minimum factor of safety for this slope is 2.334, and thus there is no location on the face of the bluff with a factor of safety at or below 1.5. Therefore, and given the California Building Code and the City of San Diego's Municipal Code requirements for all engineered structures to be located landward of that line delineating the 1.5 factor of safety atop the coastal bluff, and given the site geology, there is no 1.5 factor of safety bluff-top setback requirement, with the controlling setback requirement being the 25-foot coastal bluff edge setback.

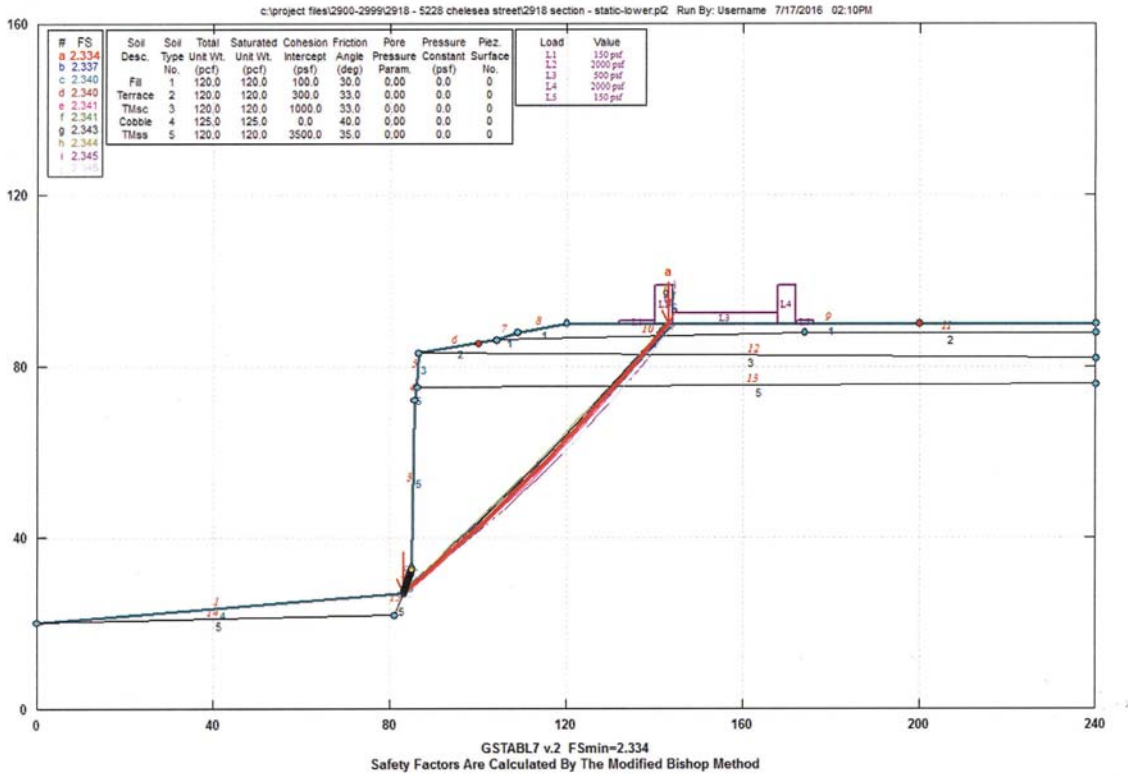



**Figure 1. Critical static slope stability analysis, reproduced from Appendix C of our July 2016 geotechnical report.**

*Issue No. 4: Three cross sections are typical for coastal bluff properties. The cross sections are typically located at the property lines and one intermediate cross section aligned orthogonal to the bluff edge. Show the distribution of geologic units, depict geologic structure, and delineate the location of the static 1.5 factor-of-safety line with respect to the coastal bluff on the cross sections.*

**Response:** See our response to Issue No. 3. The preparation of additional cross sections located at the northern and southern property lines would result in more gentle slope inclinations and even higher factors of safety. It was for this reason that we did not prepare additional geologic cross sections and remain of the opinion that Cross Section 1 as shown on Figure 3 of our July 2016 report adequately characterizes both the geology and stability of the coastal bluff.

*Issue No. 5: The project's geotechnical consultant should clarify the methods used to determine to location of the coastal bluff edge for the site. Clarify why, for example, the coastal bluff edge was not identified at the 70-foot elevation contour. Also, clarify if a gully is present onsite that has been filled during previous grading activities.*

**Response:** The coastal bluff edge was determined by the undersigned Project Geologist based on field mapping augmented by the results of our field investigation, which included the excavation of three 6-inch-diameter hollow-stem auger borings, the results of which are described in our July 2016 report for the subject property. Given the reviewer's question pertaining to the 70-foot elevation contour, it might be appropriate to first define the coastal bluff edge. We have provided below the California Coastal Commission definition, the City definition, and what we have developed as a geologic definition, also acknowledged by Coastal Commission Staff as a proper characterization of the coastal bluff.

Coastal Commission Definition

A bluff or cliff is a scarp or steep face of rock, decomposed rock, sediment or soil resulting from erosion, faulting, folding or excavation of the land mass. The cliff or bluff may be simple planar or curved surface, or it may be steplike in section. For the purposes of this guideline, "cliff" or "bluff" is limited to those features having vertical relief of ten feet or more, and "seacliff" is a cliff whose toe is or may be subject to marine erosion. "Bluff edge" or "cliff edge" is the upper termination of a bluff, cliff or seacliff. When the top edge of the cliff is rounded away from the face of the cliff as a result of erosional processes related to the presence of the steep cliff face, the edge shall be defined as that point nearest the cliff beyond which the downward gradient of the land surface increases more or less continuously until it reaches the general gradient of the cliff. In a case where there is a steplike feature at the top of the cliff face, the landward edge of the topmost riser shall be taken to be the cliff edge.

City of San Diego Definition
The upper termination of a coastal bluff. When the top edge of the coastal bluff is rounded away from the face of the coastal bluff, the edge shall be defined as that point nearest the coastal bluff beyond which the downward gradient of the land surface increases more or less continuously until it reaches the general gradient of the coastal bluff. In a case where there is a steplike feature at the top of the coastal bluff, the landward edge of the topmost riser shall be considered the bluff edge.

Geologic Definition
The boundary between the coastal bluff and the coastal terrace. Specifically, this boundary is represented by the landward extent of increased subaerial erosion due to the presence of the coastal bluff. Subaerial erosion, in its broadest sense, encompasses all of the natural geologic processes and human actions that contribute to erosion, excluding marine erosion. A coastal bluff represents the rising ground bordering the sea, which may include a sea cliff, but is characterized by an upper, moderately-sloping section ending at a coastal terrace. A coastal terrace can be defined as that geologic feature that was formed during a higher stillstand of the sea, and represents a wave-cut abrasion surface often characterized by a long, narrow, relatively level surface, bounded along the shoreward edge by the coastal bluff. Higher relic coastal terraces representing earlier stillstands of the sea commonly extend well inland. However, for the purpose of this guideline, the top of the coastal bluff is defined as that boundary of the coastal terrace that was developed during the last stillstand of the sea, which occurred approximately 125,000 years ago.

The City of San Diego Land Development Code has published the Coastal Bluffs and Beaches Guidelines, in which they define the coastal bluff edge. In Section 3 of the Guidelines, the City provides details on determining the location of the bluff edge for sensitive coastal bluffs and measuring the required bluff edge setback. Diagram III-1 illustrates a simple bluff in cross section (Figure 2), with Diagram III-4 illustrating the modified landform.

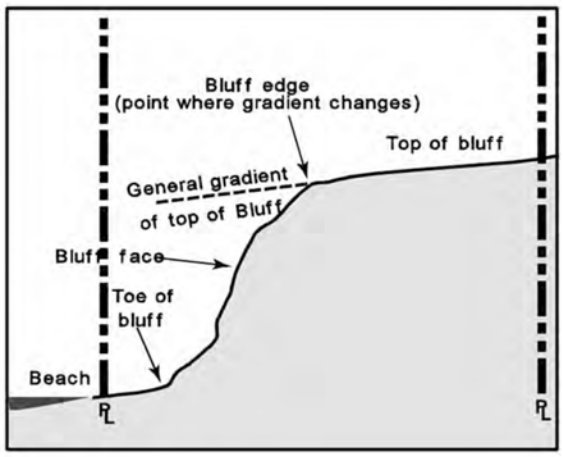



Diagram III-1 – Cross Section A-A

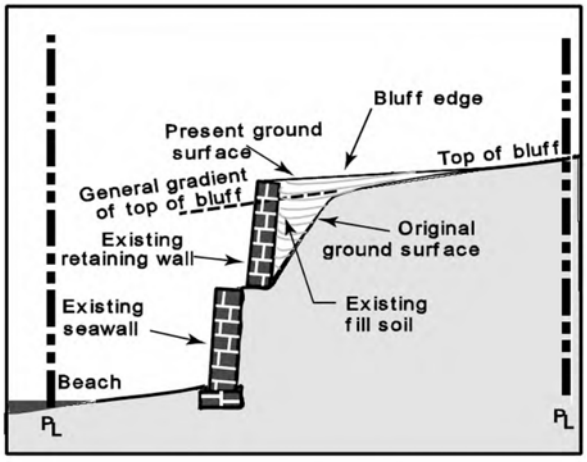



Diagram III-4 – Modified Landform

**Figure 2. Bluff edge diagrams reproduced from the City of San Diego's Coastal Bluffs and Beaches Guidelines.**

As indicated in our geologic cross section (Figure 3 of our July 2016 report and reproduced below as Figure 3), the top-of-bluff was determined first with a geologic reconnaissance and then augmented by the field exploratory program, with the stratigraphy from test borings enabling a characterization of the modified landform, along with the sloping terrace surface that is interrupted at its seaward edge by marine erosion processes, with the landward extent of marine erosion intersecting the coastal terrace then being the top-of-bluff.

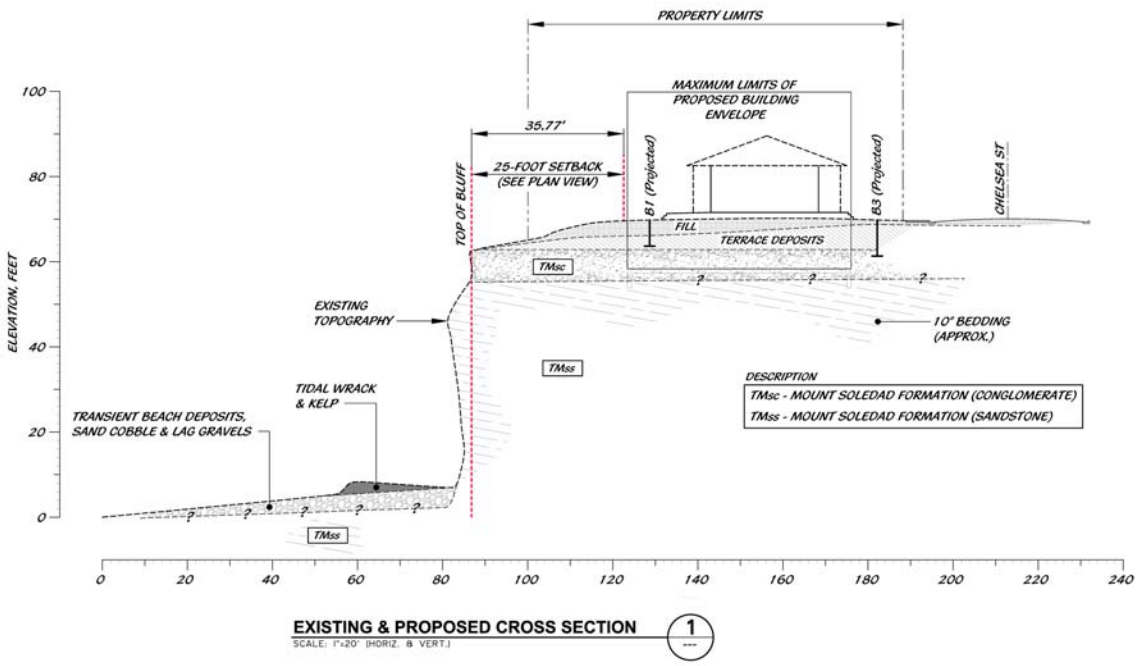



**Figure 3. Geologic cross section reproduced from our July 2019 geotechnical report.**

Given the preceding, the elevation 70 foot contour represents the modified landform and is irrelevant to the actual bluff edge.

Specific to the possibility of a gully present on site that has been filled in during previous grading activities, with the exception of the existing buried brick wall along the southern property line, our detailed geologic mapping of the bluff face did not indicate the presence of any additional filled gullies on site. Specific to the southerly property line buried brick wall, we were able to excavate immediately behind the wall and expose formational terrace deposits, again confirming the seaward edge of the coastal terrace defining the coastal bluff edge.

*Issue No. 6: Submit the computer input parameters and output calculations for the previously submitted slope stability analyses.*

**Response:** Input parameters and output files for our stability analyses are provided in Attachment A.

*Issue No. 7: Provide the reference or source material for the soil strength values used in the slope stability analyses.*

**Response:** Similar issues have been raised by Coastal Commission Staff, specifically Dr. Mark Johnsson, the Commission's Staff Geologist. Specific to the Eocene strata along the San Diego County coastline, and in an effort to achieve a consensus as to what an allowable strength would be for the Eocene strata, TerraCosta completed a study for Dr. Johnsson to develop an acceptable lower bound for strengths within these strata that would be acceptable for slope stability calculations. These data can also be utilized for marine erosion estimates. In that study, a total of five consolidated drained direct shear tests were performed, along with three unconfined compression, seven Atterberg limits, and 15 moisture content/dry density tests on samples from the middle Eocene Santiago Formation. Since this formational unit interfingers with the Del Mar and Torrey Sandstone, it can be argued that this unit has similar strength characteristics as other Eocene units along the San Diego County coastline. Specific to the fairly linear Encinitas shoreline, since erosion resistance is generally a function of unconfined compressive strength, the linearity of the Encinitas shoreline would suggest that all three of these Eocene-age cliff-forming geologic units have similar unconfined compressive strengths.

During testing, all five direct shear tests, including testing on a clay seam, were terminated prior to failure due to the fact that the core samples specimen strength exceeded the maximum limit of the proving ring. The results of this limited testing would suggest that, for the Santiago Formation, reasonable soil strengths would include a friction angle of 35 degrees and a cohesion intercept of 3,500 psf. We have reproduced below the results from the Coastal Commission study showing the lower-bound strengths of the direct shear tests on the 15 specimens taken at five sample locations (Figure 4). This is an informative plot, as the strength of any soil, when used for stability analysis, is the summation of cohesion intercept plus overburden pressure x tangent phi. Also shown on the figure is the typical strength envelope that we routinely use to approximate the strength of the Eocene cliff-forming units. The area of interest is also ballooned,

approximating an 85-foot-high slope for the Coastal Commission study and, for this condition, it is quite obvious that the existing soil strengths are in excess of twice the strengths assigned by φ = 35° and C = 3,500 pcf. We would also anticipate the Mount Soledad Formation to exhibit similar or even higher soil strengths. Copies of laboratory test results are included in Attachment B.

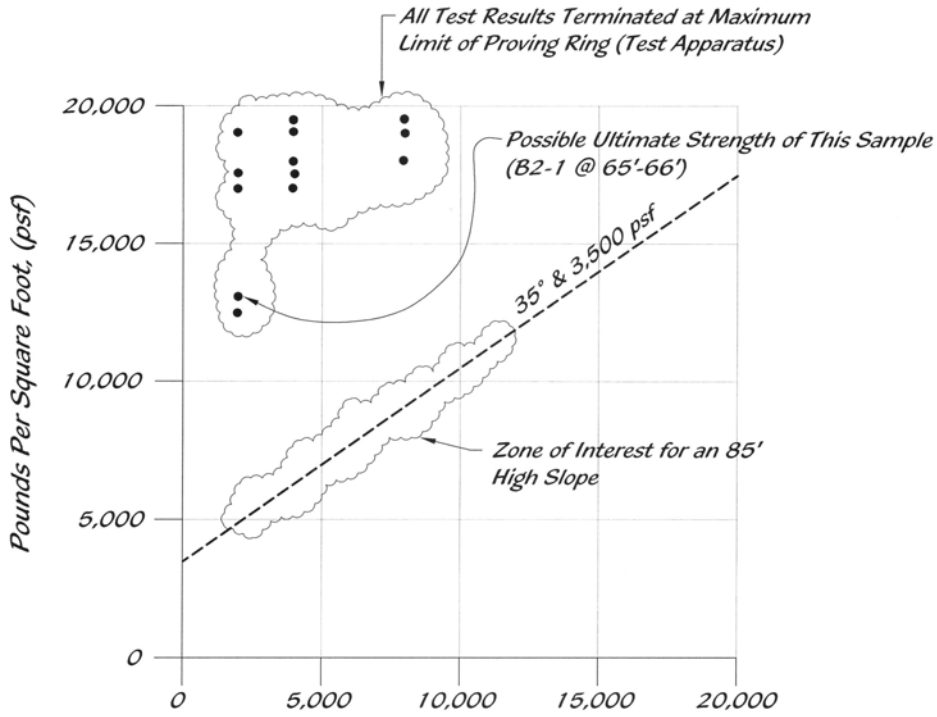



**Figure 4. Shear strength test results from a Coastal Commission study.**

*Issue No. 8: The project's geotechnical consultant must also demonstrate that the area immediately landward of the 25-foot coastal bluff edge setback onsite will have a factor-of-safety of 1.5 or greater for gross stability. Provide updated slope stability analyses and show the 25-foot setback on the plots.*

**Response:** See our responses to Issue Nos. 3 and 4.

*Issue No. 9: The project's geotechnical consultant has indicated they estimate that 5-10 feet of erosion has occurred at the site in the last 45+ years based on a review of historical photographs. Provide copies of the aerial photographs or historic maps used to determine coastal bluff recession rates. Clearly show where distances were measured on the aerial photographs or historic maps.*

**Response:** As indicated in Section 8 of our July 2016 report, erosion rates were determined using both empirical and analytical techniques to arrive at a rate of erosion. As part of our study, we reviewed U.S. Department of Agriculture stereographic aerial Photograph Nos. AXN-8M-92 and 93, and AXN-7M-186 and 187, flown May 1953. In addition, we reviewed historical photographs dating back to the 1970s, available on the California Coastal Records Project website, as listed on Table 1. On that website are historical photographs dating from 1972 to 2013 (the most current data set). Based on our review of the photographs beginning with 1972, the property to the north of the subject site had an apparent failure, as evidenced by the talus pile on the beach at the base of the bluff. A review of the 1979 and 1987 photographs indicates the talus pile had been eroded. A review of a City of San Diego 1992 aerial Image No. 8153-51 (from a three-volume aerial photo set purchased from the City of San Diego) indicates that another bluff failure had occurred on the property to the north. In this photograph, a sea stack, also exposed at the base of the bluff, is visible (Photo 1). This same sea stack can also be seen in a 2002 Coastal Records photograph (see Table 1), as well as in a December 21, 2015 (Photo 2). Our measurements between the face of the sea stack and the face of the bluff indicated 6½ feet of erosion has occurred some time between 1987 and September 2016.

*Issue No. 10: The geologic/geotechnical report must include an analysis of the potential effects on bluff stability of rising sea levels, using latest scientific date (SDMC 143.0143(B)) and an analysis of the potential effects of past and projected EI Nino events on bluff stability (SDMC 143.0143(C). The report must also provide an analysis of whether this section of coastline is under a process of retreat (SDMC 143.0143(D).*

**Response:** The stability of the coastal bluff was evaluated using the computer software GSTABL7. GSTABL7 is a graphical program that uses limit equilibrium theory to compute the factor of safety for earth and rock slopes. The Modified Bishop Method was selected for analyses of the subject slope.

Slope stability analyses indicate that the existing static factor of safety with regard to slope stability is greater than 1.5, with a computed factor of safety of 2.33 for Section 1.

Under pseudo-static conditions corresponding to a horizontal seismic coefficient of 0.15 g, the slope has a computed factor of safety greater than 1.1, with a computed factor of safety of 1.96 for Section 1. As such, from both a static and pseudo-static perspective, the slope is considered stable. Results of the stability analyses are included in Attachment A.

The effects of rising sea levels and El Niño events were also discussed in our July 2016 report and described in more detail below in our response to Issue No. 11.

As discussed in our report, this section of coastline (along with the entire California coast) is currently eroding at an annualized erosion rate of about 2 to 3 inches per year.

*Issue No. 11: Per SDMC 143.0143(f)(1)(B), reductions from a 40-foot coastal bluff edge setback requires an analysis of the potential effects on bluff stability 0f rising sea levels, using latest scientific information. The published California Coastal Commission policy provides guidance regarding this required analysis. (http://documents.coastal.ca.gov/assets/slr/guidance/August2015/O_Full_Adopted_Sea_Level_Rise_Policy_Guidance.pdf.*

**Response:** Sea level rise can influence coastal erosion, and when using relatively coarse time scales, that is, hundreds to thousands of years, the rate of cliff erosion over a given time is equal to the rate of sea level rise divided by the shore platform slope. This sea level model takes the following form (Marine Board, 1987)[1]:

$$dx/dt = (L + E) / \text{platform gradient} \qquad (1)$$

where, $dx/dt$ is the horizontal rate of erosion, $L$ is the local tectonic rate of subsidence or uplift, and $E$ is the eustatic sea level rise.

One of the most contemporary efforts at evaluating the effects of sea level rise on future bluff retreat has been developed by Young, et al. (2014)[2], in their paper titled, "Estimating Cliff Retreat in Southern California Considering Sea Level Rise Using a Sand Balance Approach."

[1] Marine Board, National Research Council, 1987, Responding to changes in sea level: engineering implications. National Academy Press, Washington, D.C.
[2] Young, A.P.,. R.E. Flick, W.C. O'Reilly, D.B. Chadwick, W.F. Crampton, J.J. Helly, 2014, Estimating Cliff Retreat in Southern California Considering Sea Level Rise Using a Sand Balance Approach. Marine Geology, 348, p. 5-26.

In summarizing Young's work, "Coastal recession associated with MSLR is often estimated using Bruun rule concepts of equilibrium profile and volume conservation (Bruun, 1962).[3]" In Young's work, they started with the basic sea level rise model above (Equation 1) represented by Equation 2 below:

$$R = \frac{SL - G}{P\left(\overline{H} + h\right)} \tag{2}$$

$R$ = Cliff retreat (m)
$S$ = MSLR (m)
$L$ = Length of active profile (m)
$G$ = External sand surplus or deficit ($m^3/m$)
$P$ = Percent of coarse cliff sand
$h$ = Closure depth (m)
$\overline{H}$ = Cliff height average over retreat distance (m)

While additional equations are provided in Young's paper, for more generalized conditions, if one simply assumes a level terrace atop the bluff, the erosion of the sea cliff can be described as follows:

$$R = \frac{SL - G}{P\left(H_o + \frac{S}{2} + h\right)} \tag{3}$$

From this, future rates of cliff retreat as a function of MSLR can be written as follows:

$$R_2 = R_1\left(\frac{S_2}{S_1}\right)^m \tag{4}$$

Where $R_1$, $R_2$ represent past and future cliff retreat, and $S_1$, $S_2$ are past and future MSLR, respectively.

The exponent $m$ may be assumed to equal 0.5 (Walkden and Dickson, 2006)[4].

[3] Bruun, P.M., 1962, "Sea-Level Rise as a Cause of Shore Erosion," Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 88, pp. 117-130.
[4] Walkden, Mike, and M. Dickson, 2006, The Response of Soft Rock Shore Profiles to Increased Sea-Level Rise, Tyndall Centre for Climate Change Research, Working Paper 105, March 2006.

In the January 2007 Edition of Science Magazine, Stefan Rahmstorf published his often cited paper, "A Semi-Empirical Approach to Projecting Future Sea Level Rise,"[5] which investigated the likely relationship between global sea level and a global rise in temperature. In Rahmstorf's plots, he uses 1990 as the zero datum for sea level rise projections through the year 2100. Figure 5 is a plot of both Rahmstorf's data and Meehl, et al.'s data (2007), reproduced from the ICLEI paper, "Sea Level Rise Adaptation Strategy for San Diego Bay,"[6] published January 2012.

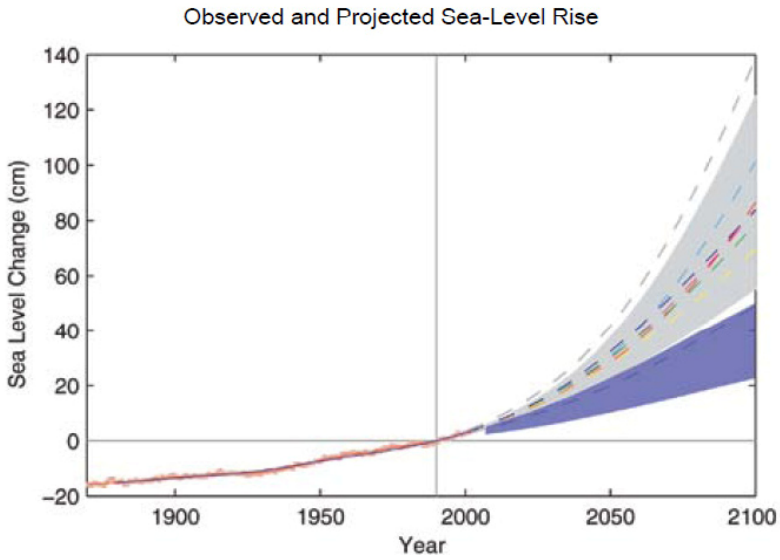



**Figure 5. Historic global sea-level observations (red) and future projections (dashed lines). The blue shaded area is from Meehl, et al., 2007 analysis and the higher grey projections are from Rahmstorf, S. 2007 analysis.**

Notably, Rahmstorf's past sea level and sea level projections from 1990 through 2100 are based on global mean temperature projections of the IPCC TAR. The grey uncertainty range spans the range of temperature rise from 1.4 degrees to 5.8 degrees Celsius. The dashed grey lines show the added uncertainty due to the statistical error of the fit in plotting temperature versus rate of sea level change, described further in Rahmstorf's 2007 paper. Colored dashed lines are the individual scenarios, with the light blue line being the A1Fl scenario, with the yellow line being the B1 scenario. In other words, the

[5] Rahmstorf, Stefan, 2007, A Semi-Empirical Approach to Projecting Future Sea-Level Rise. Science, Vol. 315, January 19, 2007, p. 368 - 370.

dashed grey lines showing 140 cm of sea level change by the year 2100 represents the maximum range in uncertainty associated with Rahmstorf's data.

If we conservatively use Rahmstorf's B1 scenario, as shown in Figure 5, the estimated sea level rise from 1990 through the year 2100 is approximately 67 cm. From 1990 through the year 2090 (the 75-year design life), the B1 scenario has a sea level rise of approximately 58 cm.

As indicated in our July 2016 report, which provides additional background information on the coastal environment consistent with the Coastal Commission's Sea Level Rise Policy Guidance Document approved August 12, 2015, the Mount Soledad Formation along this section of coastline typically has erosion rates on the order of 2 to 3 inches per year. Using Equation 4 and a rate of sea level rise consistent with Rahmstorf's B1 scenario would result in an annualized erosion rate in the year 2090 of 40 percent more than the current erosion rate. However, annualized over the next 75 years, the annualized increase in erosion rate considering sea level rise would be about 20 percent above the contemporary value. If we were to assume a contemporary annualized erosion rate of 2.5 inches per year, when spread over the next 75 years assuming Rahmstorf's B1 sea level rise scenario, the annualized erosion rate would be 3 inches per year. This corresponds to 18.75 feet of erosion in 75 years, which was reported in our July 2016 report.

*Issue No. 12: The Coastal Bluffs and Beaches Guidelines states that "development may be located less than 40 feet but not less than 25 feet from the coastal bluff edge if there is evidence in a geology report that the site is stable enough to support the proposed distance and if the development will neither be subjected to nor contribute to significant geologic instability or require a shoreline or bluff erosion control device..." Clarify if the project will be impacted by coastal bluff instability or recession during the reasonable economic life of the development (next 75-year period).*

**Response:** Based on our studies of the site and surrounding area, it is our opinion that the proposed development will neither be subjected to nor contribute to significant geologic instability and will not require construction of shoreline protection measures throughout the economic lifespan (next 75-year period) of the proposed development.

[6] ICLEI - Local Governments for Sustainability, January 2012, Sea Level Rise Adaptation Strategy for San Diego Bay.

*Issue No. 13: The geotechnical consultant must address if a shoreline or bluff erosion control devise will be necessary to protect the proposed development during the reasonable economic life 01 the development (next 75-year period). Note that if a development is approved with a Jess than 40-loot distance to the coastal bluff-edge, future erosion control measures are precluded.*

**Response:** See our response to Issue No. 12.

*Issue No. 14: According the San Diego Seismic Safety Study Geologic Hazard Maps, the site is located in geologic hazard category 53, indicating unfavorable geologic structure. The project's geotechnical consultant must indicate if the overall geologic structure is favorable or unfavorable for the proposed development as designed or provide recommendations to mitigate the geologic hazards to an acceptable level.*

**Response:** Based on our observations, as well as our review of published geologic maps, there are no indications of adverse geologic conditions, such as faulting or landsliding, that would affect the proposed development.

*Issue No. 15: The project's geotechnical consultant must indicate if the site is suitable for the currently proposed development.*

**Response:** It is our opinion that the site is suitable for the currently proposed development.

*Issue No. 16: The project's geotechnical consultant should provide a conclusion regarding if the proposed development will destabilize or result in settlement of adjacent property or the Right-of-Way.*

**Response:** Based on our review of plans and other documents for the proposed development, it is our opinion that the proposed improvements will not destabilize or result in settlement of adjacent property or right-of-way.

*Issue No. 17: Storm Water Requirements for the proposed conceptual development will be evaluated by LDR-Engineering review. Priority Development Projects (PDPs) may require an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). Check with your LDR-Engineering reviewer on requirements. LDR-Engineering may determine that LDR-Geology review of a storm water infiltration evaluation is required.*

**Response:** Based on our experience with coastal development, as well as state and local regulations, we do not recommend that any concentrated stormwater be allowed to infiltrate into the site and we recommend that all runoff should be directed away from the bluff top.

We trust these responses satisfactorily address the reviewer's comments. If you have any questions, please give us a call.

Very truly yours,

TERRACOSTA CONSULTING GROUP, INC.

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

Gregory A. Spaulding, Project Geologist
C.E.G. 1863, C.H.G. 351, R.G. 5892

WFC/GAS/jg
Attachments

TABLE 1

**LIST OF PHOTOS REVIEWED**

**CALIFORNIA COASTAL RECORDS PROJECT**
(www.californiacoastline.org)

| Year | Image No. |
|---|---|
| 1972 | 7241111 |
| 1979 | 7955099 |
| 1987 | 8701192 |
| 2002 | 9601 |
| 2004 | 200407867 |
| 2006 | 200604593 |
| 2008 | 200804782 |
| 2010 | 20100471 |
| 2013 | 201312517 |

**CITY OF SAN DIEGO OBLIQUE AERIAL**

| Year | Image No. |
|---|---|
| 1992 | 8153-51 |

**TERRACOSTA CONSULTING GROUP**

| Date | Image No. |
|---|---|
| 12-21-2015 | PC210032 |


**PHOTO 1:** This 1992 photo is the first to show the sea stack, which formed sometime between 1987 and 1992 (see Image 8701192 at www.californiacoastline.org).

TerraCosta Consulting Group

**TERRACOSTA CONSULTING GROUP**
3890 Murphy Canyon Road, Suite 200
San Diego, California 92123 (858) 573-6900

**PROJECT NO.:**
**2918**

**PHOTO NO.:**
**1**

## 5228 CHELSEA STREET, LA JOLLA


**PHOTO 2:** Sea stack observed in Photo 1 was measured 6-1/2 feet from the front face of the sea stack to the face of the bluff.

TerraCosta Consulting Group

**TERRACOSTA CONSULTING GROUP**
3890 Murphy Canyon Road, Suite 200
San Diego, California 92123 (858) 573-6900

**PROJECT NO.:**
**2918**

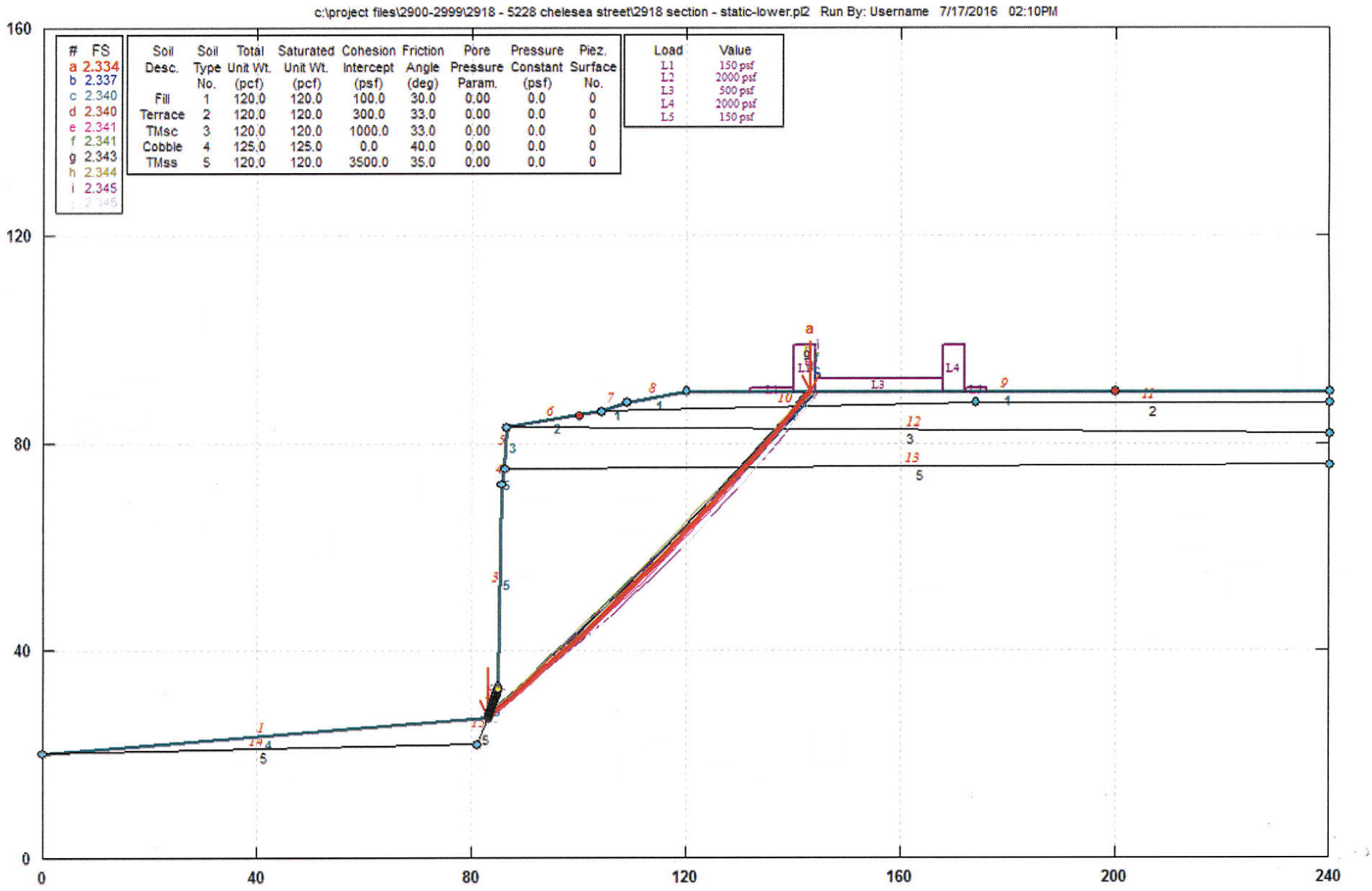
**PHOTO NO.:**
**2**

## 5228 CHELSEA STREET, LA JOLLA

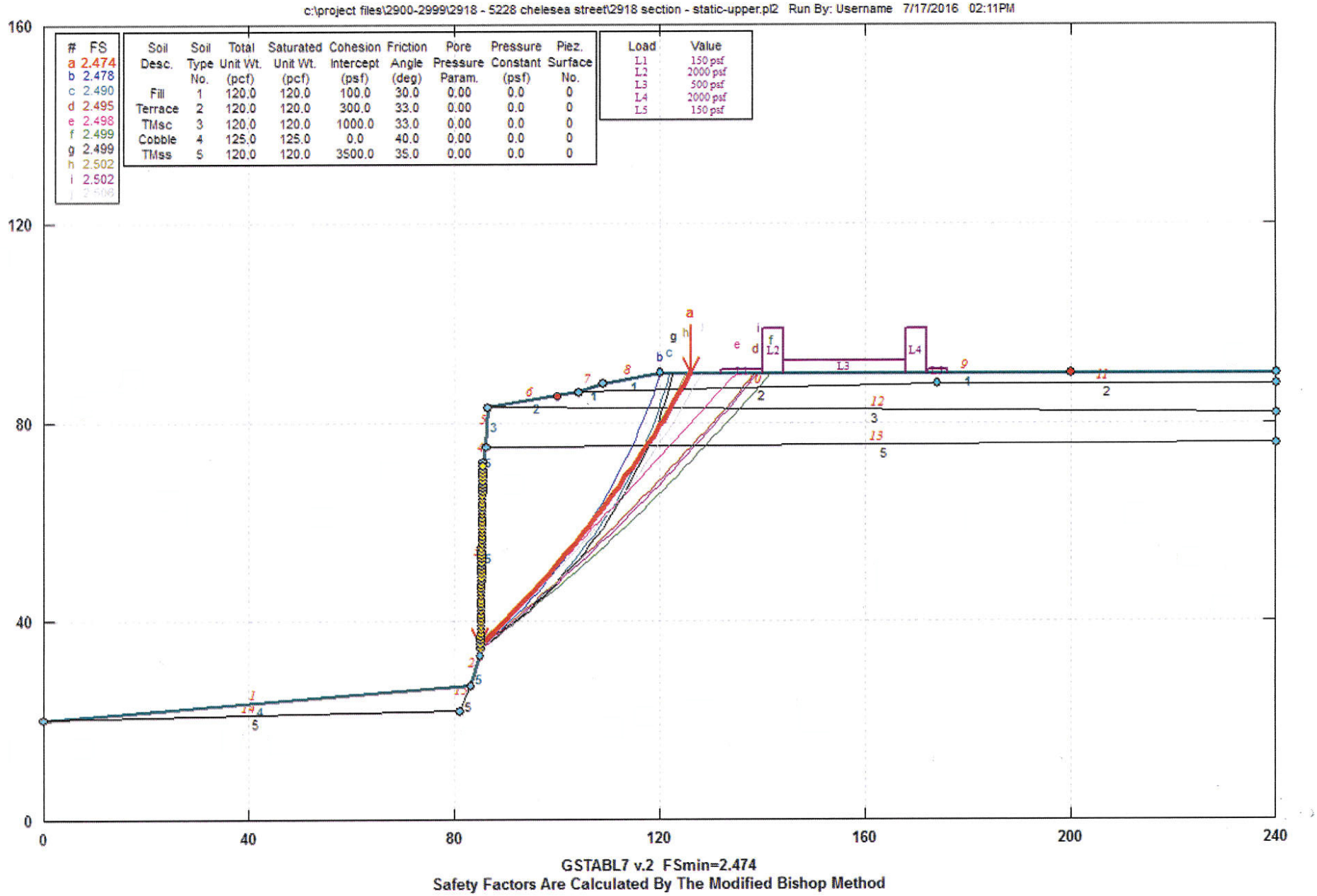
# ATTACHMENT A

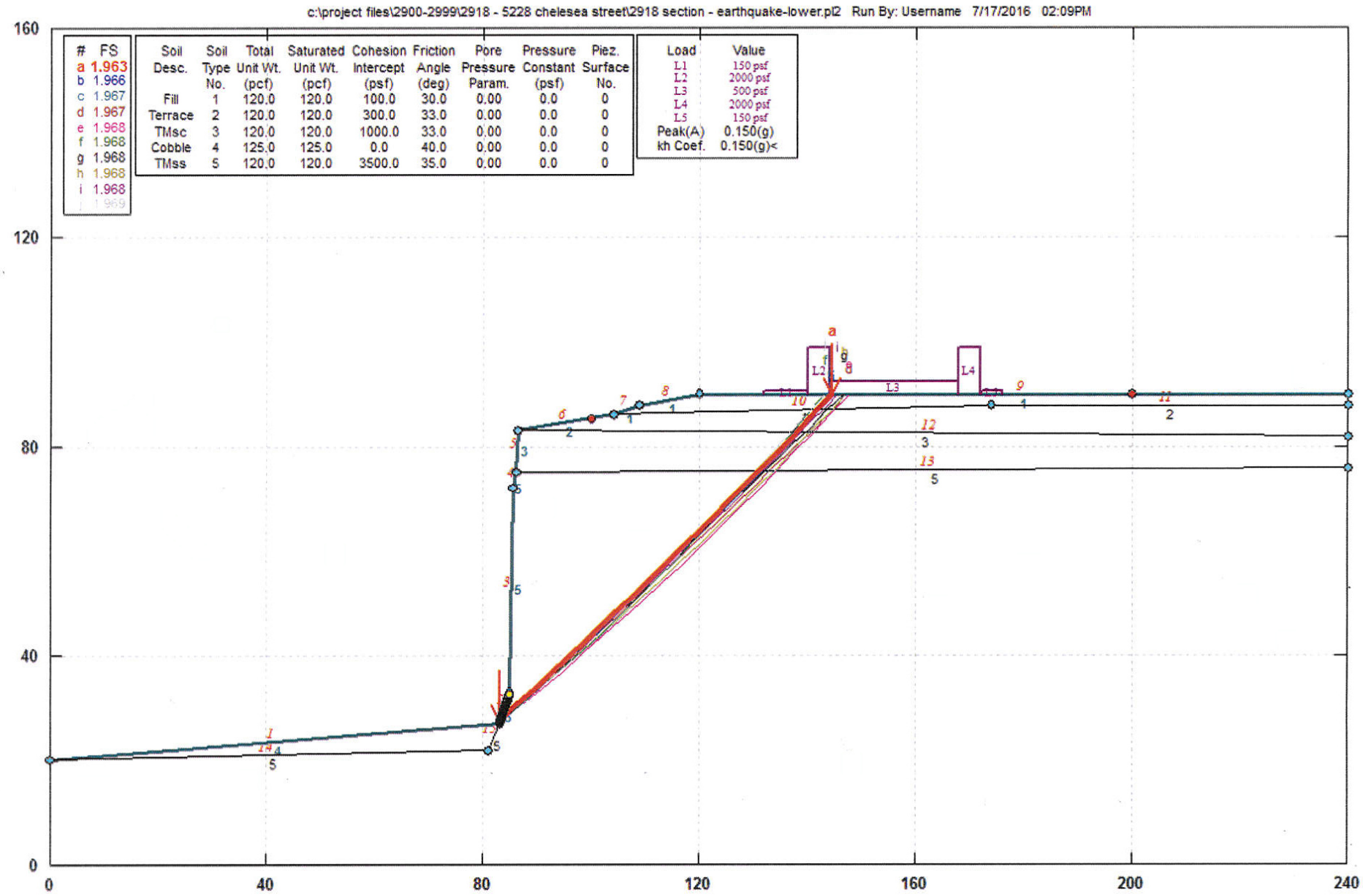
# STABILITY ANALYSES
# INPUT AND OUTPUT FILES

TerraCosta
Consulting Group



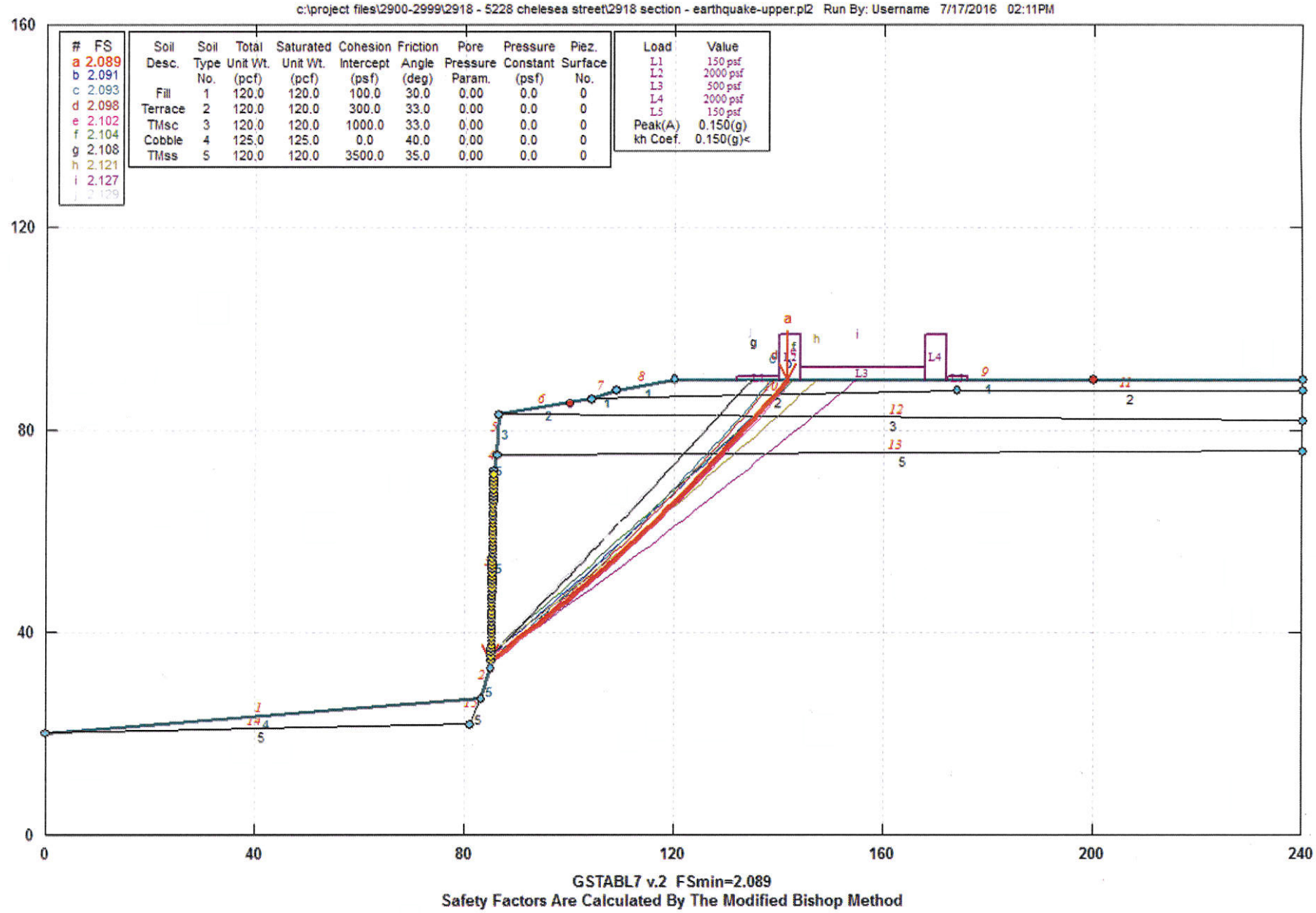

GSTABL7 v.2 FSmin=2.334
Safety Factors Are Calculated By The Modified Bishop Method


c:\project files\2900-2999\2918 - 5228 chelesea street\2918 section - static-upper.pl2 Run By: Username 7/17/2016 02:11PM
# FS
a 2.474
b 2.478
c 2.490
d 2.495
e 2.498
f 2.499
g 2.499
h 2.502
i 2.502
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No.
Fill | 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0
Terrace | 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0
TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0
Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0
TMss | 5 | 120.0 | 120.0 | 3500.0 | 35.0 | 0.00 | 0.0 | 0
Load | Value
L1 150 psf
L2 2000 psf
L3 500 psf
L4 2000 psf
L5 150 psf
160
120
80
40
0
0
40
80
120
160
200
240
GSTABL7 v.2 FSmin=2.474
Safety Factors Are Calculated By The Modified Bishop Method




GSTABL7 v.2 FSmin=1.963

Safety Factors Are Calculated By The Modified Bishop Method


c:\project files\2900-2999\2918 - 5228 chelesea street\2918 section - earthquake-upper.pl2 Run By: Username 7/17/2016 02:11PM
# FS
a 2.089
b 2.091
c 2.093
d 2.098
e 2.102
f 2.104
g 2.108
h 2.121
i 2.127
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No.
Fill | 1 | 120.0 | 120.0 | 100.0 | 30.0 | 0.00 | 0.0 | 0
Terrace | 2 | 120.0 | 120.0 | 300.0 | 33.0 | 0.00 | 0.0 | 0
TMsc | 3 | 120.0 | 120.0 | 1000.0 | 33.0 | 0.00 | 0.0 | 0
Cobble | 4 | 125.0 | 125.0 | 0.0 | 40.0 | 0.00 | 0.0 | 0
TMss | 5 | 120.0 | 120.0 | 3500.0 | 35.0 | 0.00 | 0.0 | 0
Load | Value
L1 150 psf
L2 2000 psf
L3 500 psf
L4 2000 psf
L5 150 psf
Peak(A) 0.150(g)
kh Coef. 0.150(g)<
160
120
80
40
0
0
40
80
120
160
200
240
GSTABL7 v.2 FSmin=2.089
Safety Factors Are Calculated By The Modified Bishop Method

# ATTACHMENT B

# LABORATORY TEST RESULTS FROM 2002 STUDY PERFORMED FOR CALIFORNIA COASTAL COMMISSION

TerraCosta
Consulting Group

# PHYSICAL PROPERTIES OF SOILS

| | | | |
|---|---|---|---|
| PROJECT: TerraCosta Consulting Group | LAB NO.: | 12957 | PROJECT NO.: 70341.1.0081.22.831 |
| | SAMPLED BY: | G. Spaulding | DATE: 04/16/02 |
| Project #2112 | SUBMITTED BY: | G. Spaulding | DATE: 04/16/02 |
| 1264 Neptune | AUTHORIZED BY: | W. Crampton | DATE: 04/16/02 |
| | REVIEWED BY: | L. Shuler | REPORT DATE: 05/15/02 |

| Sample I.D. | Depth (ft.) | Liquid Limit/ Plastic Limit ASTM D4318-00 | Expansion Index ASTM 4829-95 | Percent Passing #200 Sieve ASTM D1140-00 | Dry Density (pcf) | Moisture Content (%), as received ASTM D2216 |
|---|---|---|---|---|---|---|
| B2-6 | 101' | 39/35 | * | * | 123.8 | 9.0 |
| B2-6 | 101' | * | * | * | 119.7 | 9.0 |
| B2-6 | 101' | * | * | * | 121.5 | 9.0 |
| B2-2 | 75-76' | 44/18 | * | * | 119.5 | 10.4 |
| B2-2 | 75-76' | * | * | * | 117.6 | 10.4 |
| B2-2 | 75-76' | * | * | * | 118.3 | 10.4 |
| B2-1 | 65-66' | 66/26 | * | * | 110.4 | 16.6 |
| B2-1 | 65-66' | * | * | * | 106.8 | 16.6 |
| B2-1 | 65-66' | * | * | * | 73.8 | 16.6 |
| B2-4 | 83.5-84' | 56/22 | * | * | 118.6 | 13.4 |
| B2-4 | 83.5-84' | * | * | * | 116.6 | 13.4 |
| B2-4 | 83.5-84' | * | * | * | 115.4 | 13.4 |

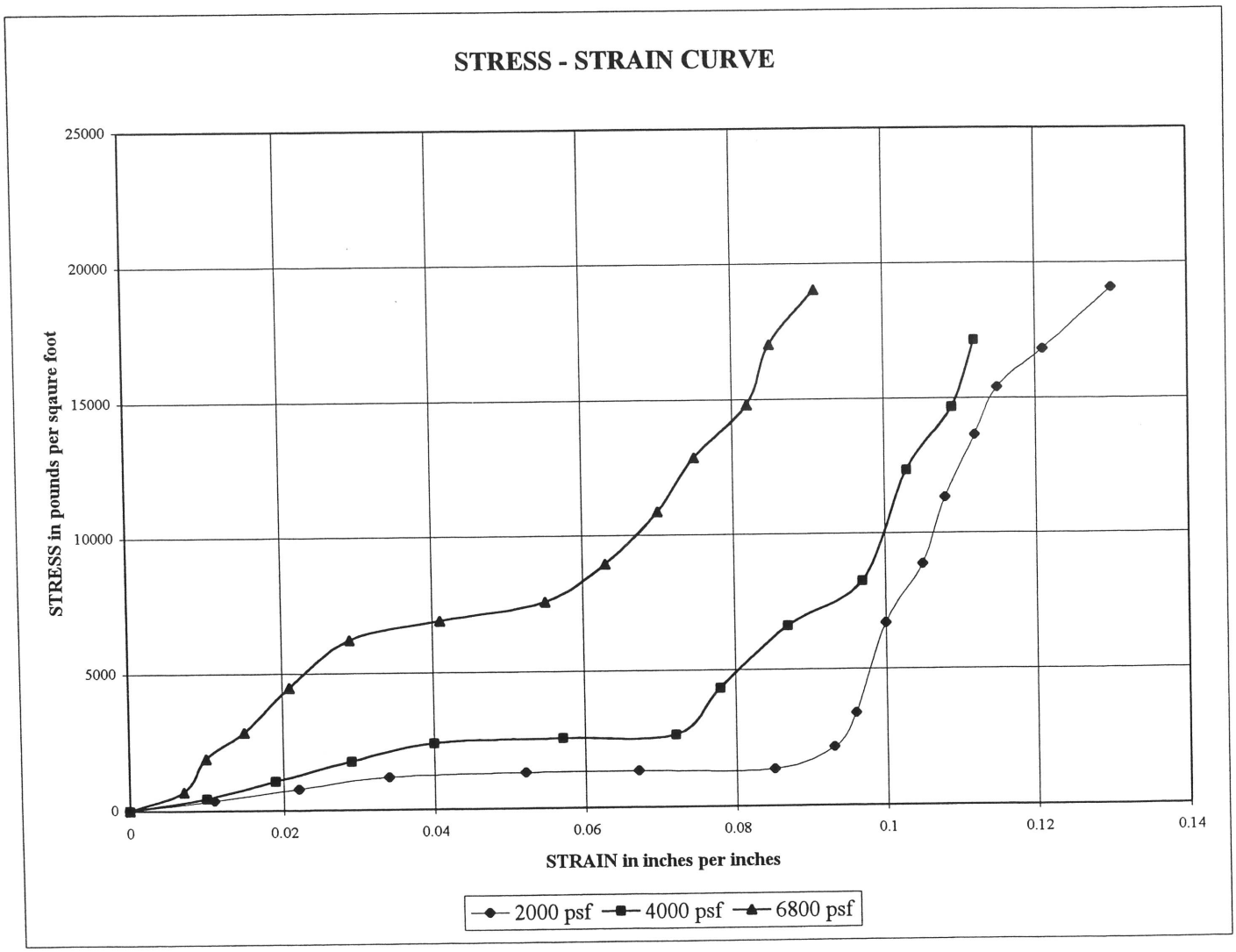
*Indicates test not requested

Distribution: TerraCosta/ W. Crampton
TerraCosta/ G. Spaulding

Submitted By: Clifford Craft, P.E.

# REPORT OF DIRECT SHEAR TEST DATA

| | | | | | |
|---|---|---|---|---|---|
| PROJECT: | Terra Costa #2112 | LAB NO.: | 12957 | JOB NO: | 70341-1-0081.22.820 |
| COMPACTION SAMPLE: | ** | SUBMITTED BY: | Terra Costa | DATE: | ** |
| DEPTH: | ** | AUTHORIZED BY: | Terra Costa | DATE: | ** |
| BORING: | B2 | TESTED BY: | RV | DATE: | April 30, 2002 |
| DEPTH: | 75-76 | COMPUTED BY: | RV | DATE: | May 3, 2002 |
| COMPACTION PREPARED AT: | | REVIEWED BY: | ELS | DATE: | May 15, 2002 |
| OPT. MOISTURE CONTENT: | | SOIL TYPE: | | | |
| MAX. DRY DENSITY: | | SOURCE: | | | |
| REPORT DATE: | 5/15 | MOISTURE CONTENT TESTED AT: | | | |



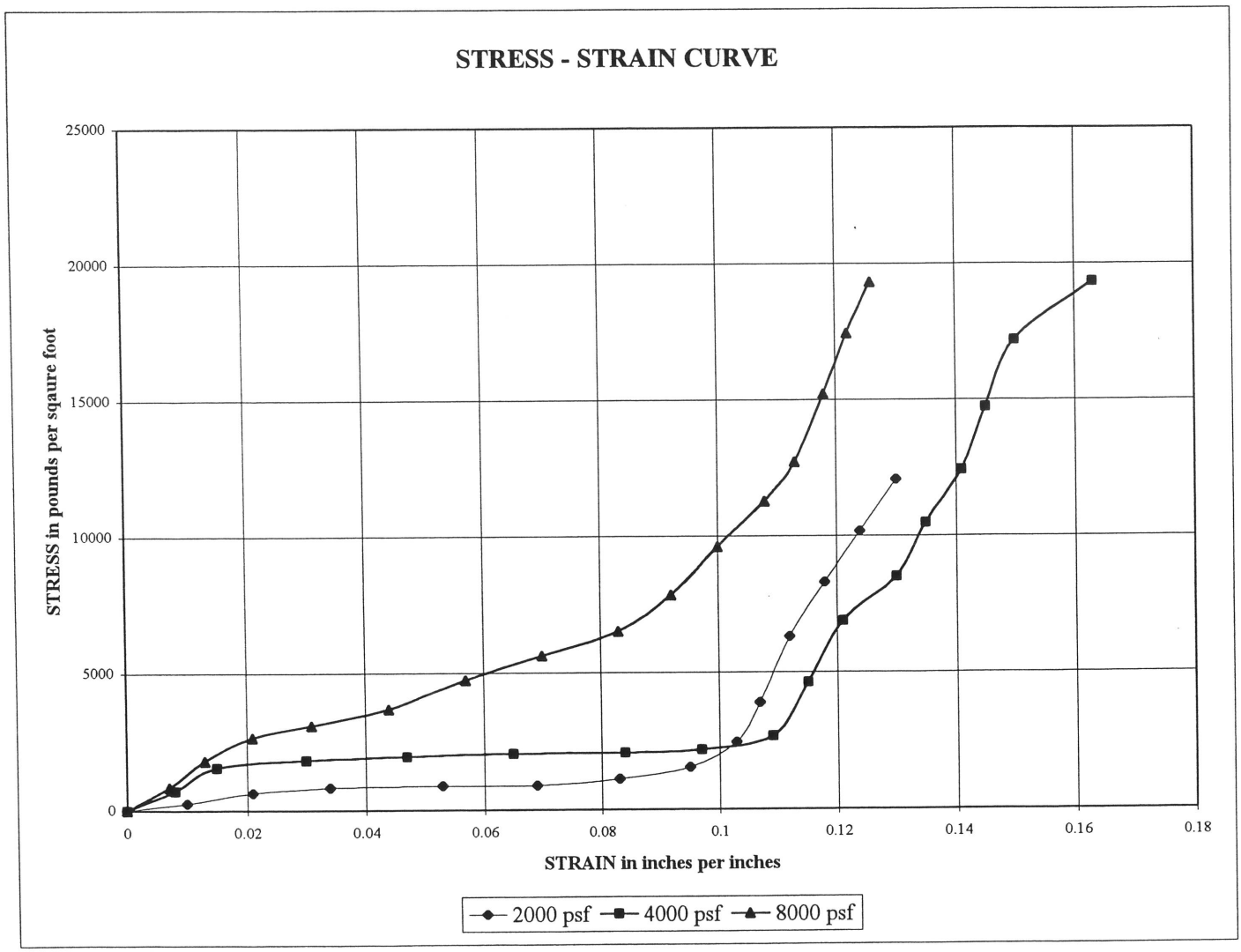

Note: Test terminated at maximum limit of proving ring.

Respectfully Submitted,

Clifford A. Craft, P.E.

# REPORT OF DIRECT SHEAR TEST DATA

| | | | | | |
|---|---|---|---|---|---|
| PROJECT: | Terra Costa #2112 | LAB NO.: | 12957 | JOB NO: | 70341-1-0081.22.820 |
| COMPACTION SAMPLE: | ** | SUBMITTED BY: | Terra Costa | DATE: | ** |
| DEPTH: | ** | AUTHORIZED BY: | Terra Costa | DATE: | ** |
| BORING: | B2-4 | TESTED BY: | RV | DATE: | April 30, 2002 |
| DEPTH: | 83.5-84 | COMPUTED BY: | RV | DATE: | May 3, 2002 |
| COMPACTION PREPARED AT: | | REVIEWED BY: | ELS | DATE: | May 15, 2002 |
| OPT. MOISTURE CONTENT: | | SOIL TYPE: | | | |
| MAX. DRY DENSITY: | | SOURCE: | | | |
| REPORT DATE: | May 15, 2002 | MOISTURE CONTENT TESTED AT: | | | |



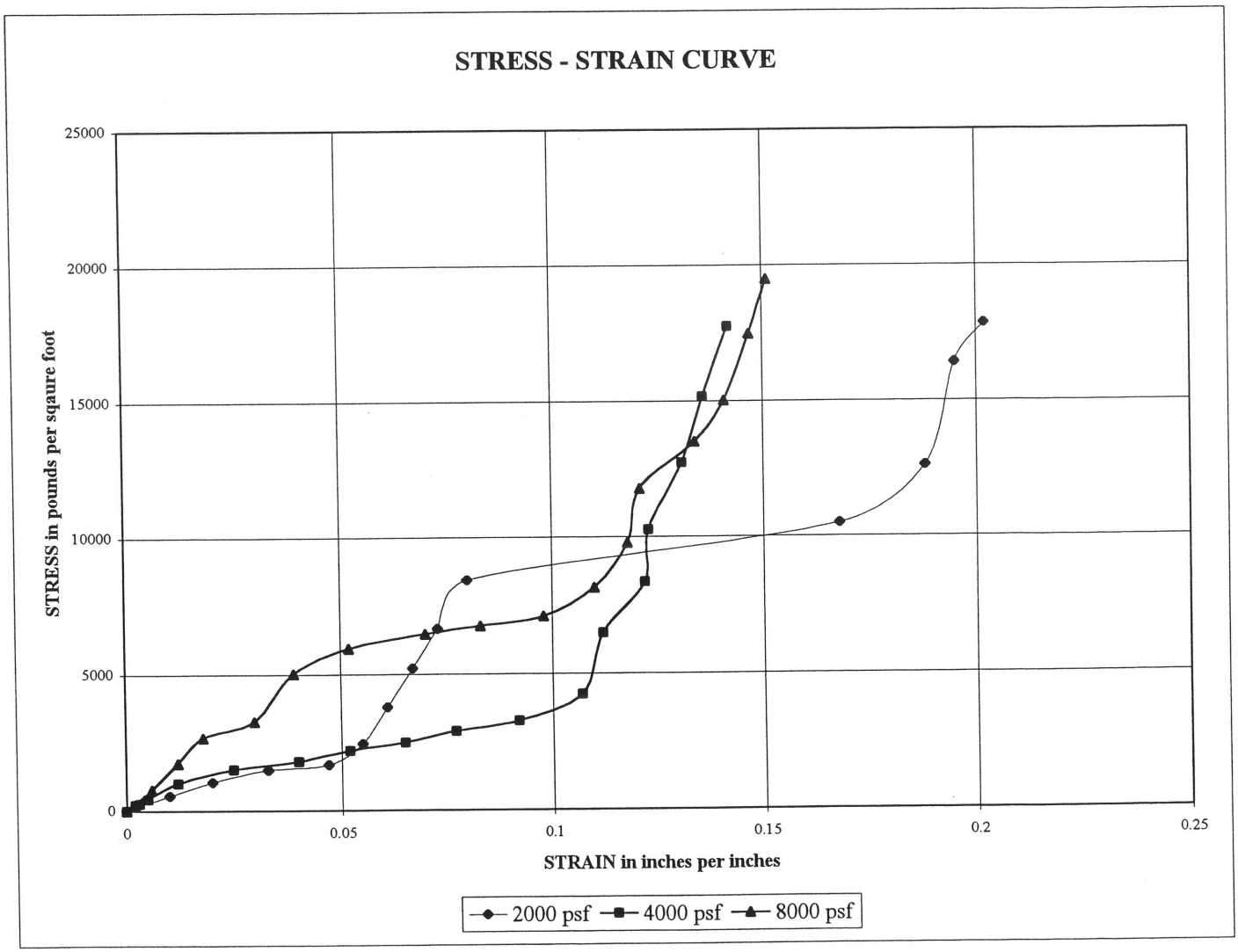

Note: Test terminated at maximum limit of proving ring.

Respectfully Submitted,

Clifford A. Craft, P.E.

# REPORT OF DIRECT SHEAR TEST DATA

| | | | | | |
|---|---|---|---|---|---|
| PROJECT: | Terra Costa #2112 | LAB NO.: | 12957 | JOB NO: | 70341-1-0081.22.820 |
| COMPACTION SAMPLE: | ** | SUBMITTED BY: | Terra Costa | DATE: | ** |
| DEPTH: | ** | AUTHORIZED BY: | Terra Costa | DATE: | ** |
| BORING: | B2 | TESTED BY: | RV | DATE: | April 30, 2002 |
| DEPTH: | 93-94 | COMPUTED BY: | RV | DATE: | May 3, 2002 |
| COMPACTION PREPARED AT: | | REVIEWED BY: | ELS | DATE: | May 15, 2002 |
| OPT. MOISTURE CONTENT: | | SOIL TYPE: | | | |
| MAX. DRY DENSITY: | | SOURCE: | | | |
| REPORT DATE: | May 15, 2002 | MOISTURE CONTENT TESTED AT: | | | |



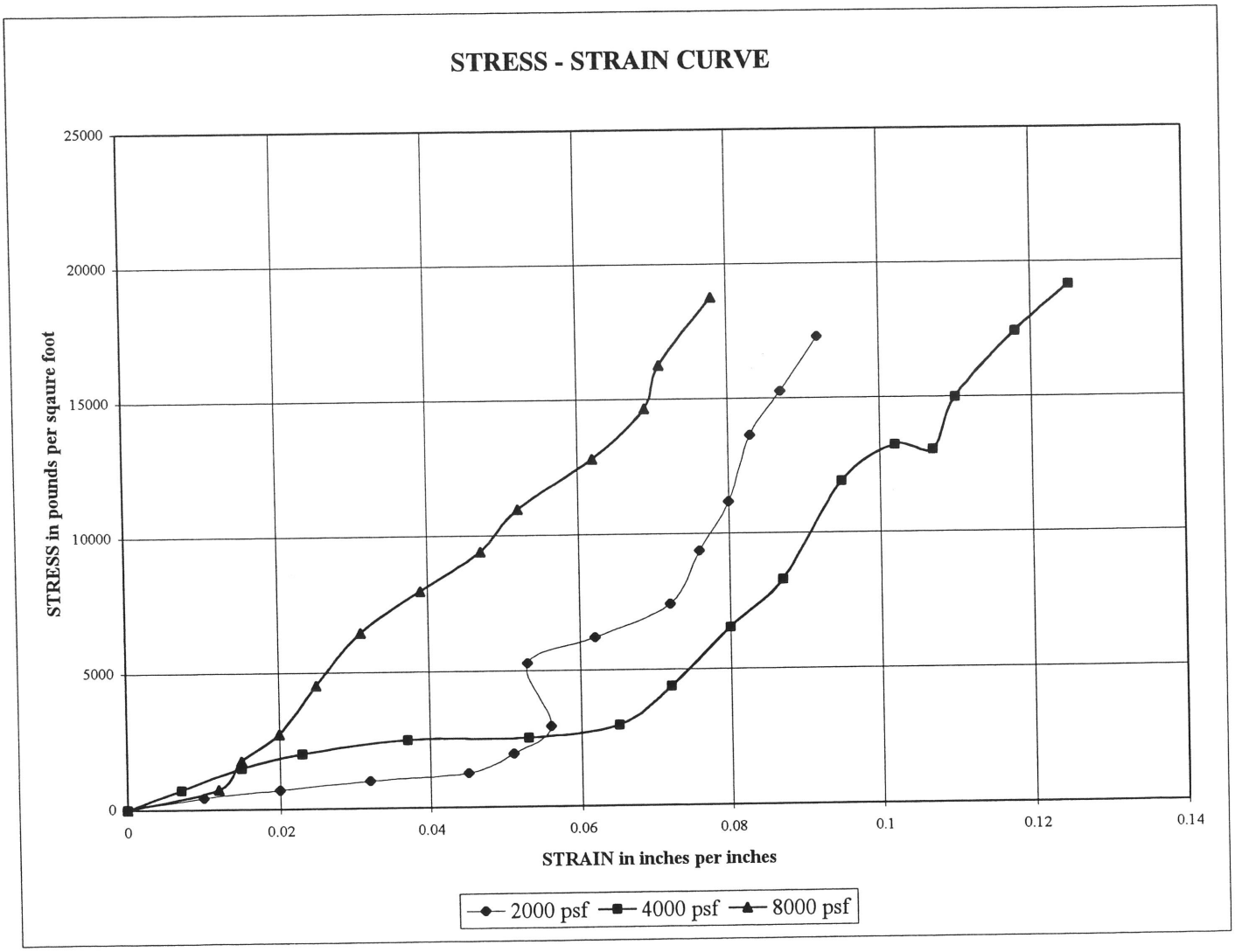

Note: Test terminated at maximum limit of proving ring.

Respectfully Submitted,

Clifford A. Craft, P.E.

# REPORT OF DIRECT SHEAR TEST DATA

| | | | | | |
|---|---|---|---|---|---|
| PROJECT: | Terra Costa #2112 | LAB NO.: | 12957 | JOB NO: | 70341-1-0081.22.820 |
| COMPACTION SAMPLE: | ** | SUBMITTED BY: | Terra Costa | DATE: | ** |
| DEPTH: | ** | AUTHORIZED BY: | Terra Costa | DATE: | ** |
| BORING: | B2 | TESTED BY: | RV | DATE: | April 30, 2002 |
| DEPTH: | 101 | COMPUTED BY: | RV | DATE: | May 3, 2002 |
| COMPACTION PREPARED AT: | | REVIEWED BY: | ELS | DATE: | May 15, 2002 |
| OPT. MOISTURE CONTENT: | | SOIL TYPE: | | | |
| MAX. DRY DENSITY: | | SOURCE: | | | |
| REPORT DATE: | May 15, 2002 | MOISTURE CONTENT TESTED AT: | | | |




Note: Test terminated at maximum limit of proving ring.

Respectfully Submitted,

Clifford A. Craft, P.E.

LAW
RESOURCES CREATING SOLUTIONS

A Division of
Law Engineering and Environmental Services, Inc.

9177 Sky Park Court
San Diego, California

# UNCONFINED COMPRESSION

ASTM D -2166

| | |
|---|---|
| PROJECT | Terra Costa - 1264 Neptune #2112 |
| LOCATION | |
| JOB No. | 70341-1-0081.22 |
| BORING / SAMPLE No. | B2-2 |
| SAMPLE DEPTH | 76' |
| SOIL DESCRIPTION | |
| TESTED BY | RV |
| DATE OF TESTING | |
| REVIEWED BY | ELS |
| DATE OF REVIEW | 5/16/2002 |

| | | | | |
|---|---|---|---|---|
| DIAMETER, $D_0$ | 2.37 | in | 60.198 | mm |
| AREA, $A_0$ | 4.4115 | $in^2$ | 28.4613 | $cm^2$ |
| HEIGHT, $L_0$ | 5.773 | in | 146.634 | mm |
| VOLUME, $V_0$ | 25.4676 | $in^3$ | 417.339 | $cm^3$ |
| WATER CONT. | | % | | |

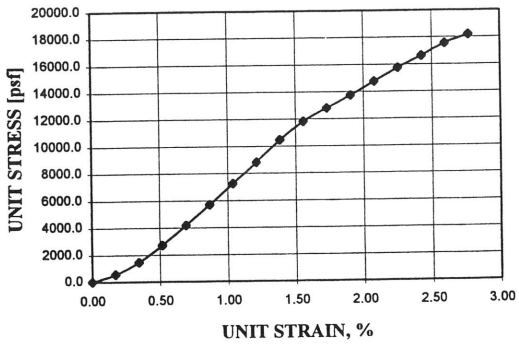
| | | |
|---|---|---|
| WET SOIL SAMPLE | | g |
| WET SOIL+ PAN MASS | | g |
| DRY SOIL + PAN MASS | | g |
| PAN MASS | | g |
| WET DENSITY | | $lb/ft^3$ |
| DRY DENSITY | | $lb/ft^3$ |

| | | | | |
|---|---|---|---|---|
| UNCONFINED COMPRESSION STRENGTH, $q_u$ | 18150.00 | psf | 869.02 | kPa |
| | 126.04 | psi | | |
| COHESION, $S_U = q_U / 2$ = | 9075.00 | psf | 434.51 | kPa |

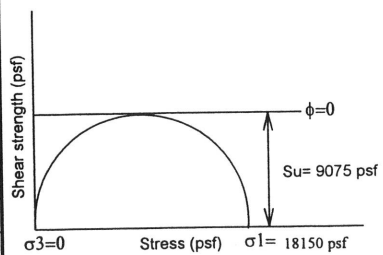
Ring Calibration (lbs/div): 1

| Sample Def., ΔL (in) | Proving Ring Dial (div) | Total Load on Sample (lb) | Unit Strain, ε ΔL/$L_0$ ( x $10^{-2}$ ) | Area Cor. Factor ( 1 - ε ) | Corrected Area, A' ( $in^2$ ) | Sample Stress, $\sigma_1$ (psf) | Sample Stress, $\sigma_1$ (kPa) |
|---|---|---|---|---|---|---|---|
| 0.0000 | 0 | 0.00 | 0.0000 | 1.0000 | 4.4115 | 0.00 | 0.00 |
| 0.0100 | 17 | 17.00 | 0.1732 | 0.9983 | 4.4192 | 553.95 | 26.52 |
| 0.0200 | 45 | 45.00 | 0.3464 | 0.9965 | 4.4268 | 1463.80 | 70.09 |
| 0.0300 | 84 | 84.00 | 0.5197 | 0.9948 | 4.4345 | 2727.67 | 130.60 |
| 0.0400 | 129 | 129.00 | 0.6929 | 0.9931 | 4.4423 | 4181.63 | 200.22 |
| 0.0500 | 176 | 176.00 | 0.8661 | 0.9913 | 4.4500 | 5695.22 | 272.69 |
| 0.0600 | 224 | 224.00 | 1.0393 | 0.9896 | 4.4578 | 7235.80 | 346.45 |
| 0.0700 | 273 | 273.00 | 1.2125 | 0.9879 | 4.4657 | 8803.20 | 421.50 |
| 0.0800 | 324 | 324.00 | 1.3858 | 0.9861 | 4.4735 | 10429.43 | 499.36 |
| 0.0900 | 367 | 367.00 | 1.5590 | 0.9844 | 4.4814 | 11792.83 | 564.64 |
| 0.1000 | 398 | 398.00 | 1.7322 | 0.9827 | 4.4893 | 12766.45 | 611.26 |
| 0.1100 | 428 | 428.00 | 1.9054 | 0.9809 | 4.4972 | 13704.55 | 656.17 |
| 0.1200 | 461 | 461.00 | 2.0786 | 0.9792 | 4.5051 | 14735.14 | 705.52 |
| 0.1300 | 493 | 493.00 | 2.2519 | 0.9775 | 4.5131 | 15730.09 | 753.16 |
| 0.1400 | 522 | 522.00 | 2.4251 | 0.9757 | 4.5211 | 16625.88 | 796.05 |
| 0.1500 | 551 | 551.00 | 2.5983 | 0.9740 | 4.5292 | 17518.38 | 838.78 |
| 0.1600 | 572 | 572.00 | 2.7715 | 0.9723 | 4.5373 | 18153.71 | 869.20 |

## UNCONFINED COMPRESSION TEST




Mohr's Circle




Remarks: Material failed suddenly at maximum reported loading.

Respectfully Submitted,

Clifford A. Craft, P.E.

LAW
RESOURCES CREATING SOLUTIONS

A Division of
Law Engineering and Environmental Services, Inc.

9177 Sky Park Court
San Diego, California

# UNCONFINED COMPRESSION

ASTM D -2166

| | |
|---|---|
| PROJECT | Terra Costa - 1264 Neptune #2112 |
| LOCATION | |
| JOB No. | 70341-1-0081.22 |
| BORING / SAMPLE No. | B2-3 |
| SAMPLE DEPTH | 78' |
| SOIL DESCRIPTION | |
| TESTED BY | RV |
| DATE OF TESTING | |
| REVIEWED BY | ELS |
| DATE OF REVIEW | 5/16/2002 |

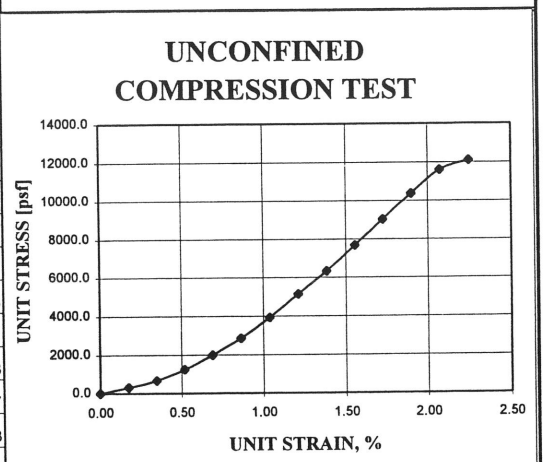
| | | |
|---|---|---|
| DIAMETER, $D_0$ | 2.367 in | 60.1218 mm |
| AREA, $A_0$ | 4.40034 $in^2$ | 28.3892 $cm^2$ |
| HEIGHT, $L_0$ | 5.782 in | 146.863 mm |
| VOLUME, $V_0$ | 25.4428 $in^3$ | 416.932 $cm^3$ |
| WATER CONT. | % | |

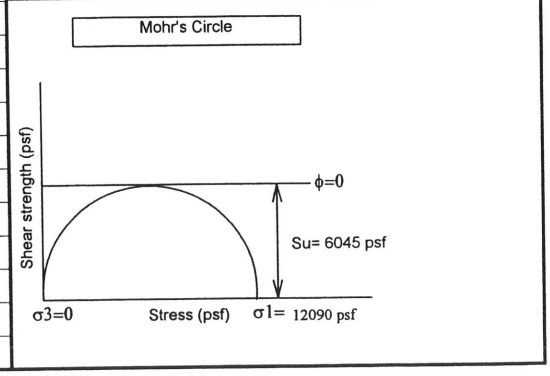
| | |
|---|---|
| WET SOIL SAMPLE | g |
| WET SOIL+ PAN MASS | g |
| DRY SOIL + PAN MASS | g |
| PAN MASS | g |
| WET DENSITY | $lb/ft^3$ |
| DRY DENSITY | $lb/ft^3$ |

| | | | | |
|---|---|---|---|---|
| UNCONFINED COMPRESSION STRENGTH, $q_u$ | 12090.00 | psf | 578.87 | kPa |
| | 83.96 | psi | | |
| COHESION, $S_U = q_U / 2 =$ | 6045.00 | psf | 289.43 | kPa |

Ring Calibration (lbs/div): 1

| Sample Def., ΔL (in) | Proving Ring Dial (div) | Total Load on Sample (lb) | Unit Strain, ε ΔL/$L_0$ ( x $10^{-2}$ ) | Area Cor. Factor, ( 1 - ε ) | Corrected Area, A' ( $in^2$ ) | Sample Stress, $\sigma_1$ (psf) | Sample Stress, $\sigma_1$ (kPa) |
|---|---|---|---|---|---|---|---|
| 0.0000 | 0 | 0.00 | 0.0000 | 1.0000 | 4.4003 | 0.00 | 0.00 |
| 0.0100 | 9 | 9.00 | 0.1730 | 0.9983 | 4.4080 | 294.01 | 14.08 |
| 0.0200 | 20 | 20.00 | 0.3459 | 0.9965 | 4.4156 | 652.23 | 31.23 |
| 0.0300 | 38 | 38.00 | 0.5189 | 0.9948 | 4.4233 | 1237.09 | 59.23 |
| 0.0400 | 61 | 61.00 | 0.6918 | 0.9931 | 4.4310 | 1982.40 | 94.92 |
| 0.0500 | 88 | 88.00 | 0.8648 | 0.9914 | 4.4387 | 2854.87 | 136.69 |
| 0.0600 | 121 | 121.00 | 1.0377 | 0.9896 | 4.4465 | 3918.60 | 187.62 |
| 0.0700 | 159 | 159.00 | 1.2107 | 0.9879 | 4.4543 | 5140.24 | 246.11 |
| 0.0800 | 196 | 196.00 | 1.3836 | 0.9862 | 4.4621 | 6325.30 | 302.86 |
| 0.0900 | 238 | 238.00 | 1.5566 | 0.9844 | 4.4699 | 7667.25 | 367.11 |
| 0.1000 | 280 | 280.00 | 1.7295 | 0.9827 | 4.4778 | 9004.45 | 431.13 |
| 0.1100 | 323 | 323.00 | 1.9025 | 0.9810 | 4.4857 | 10369.00 | 496.47 |
| 0.1200 | 362 | 362.00 | 2.0754 | 0.9792 | 4.4936 | 11600.49 | 555.43 |
| 0.1300 | 378 | 378.00 | 2.2484 | 0.9775 | 4.5016 | 12091.83 | 578.96 |




Remarks: Material failed suddenly at maximum reported loading.

Respectfully Submitted,

Clifford A. Craft, P.E.

LAW
RESOURCES CREATING SOLUTIONS

A Division of
Law Engineering and Environmental Services, Inc.

9177 Sky Park Court
San Diego, California

# UNCONFINED COMPRESSION

ASTM D -2166

| | |
|---|---|
| PROJECT | Terra Costa - 1264 Neptune #2112 |
| LOCATION | |
| JOB No. | 70341-1-0081.22 |
| BORING / SAMPLE No. | B2-4 |
| SAMPLE DEPTH | 86' |
| SOIL DESCRIPTION | |
| TESTED BY | RV |
| DATE OF TESTING | |
| REVIEWED BY | ELS |
| DATE OF REVIEW | 5/16/2002 |

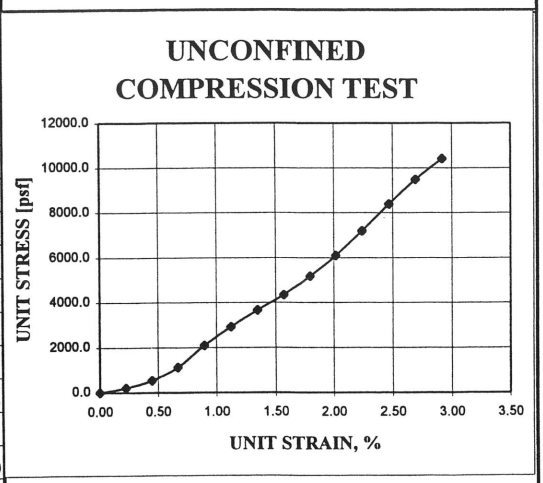
| | | | | | |
|---|---|---|---|---|---|
| DIAMETER, $D_0$ | 2.362 | in | 59.9948 | mm | |
| AREA, $A_0$ | 4.38177 | $in^2$ | 28.2694 | $cm^2$ | |
| HEIGHT, $L_0$ | 4.456 | in | 113.182 | mm | |
| VOLUME, $V_0$ | 19.5252 | $in^3$ | 319.96 | $cm^3$ | |
| WATER CONT. | | % | | | |

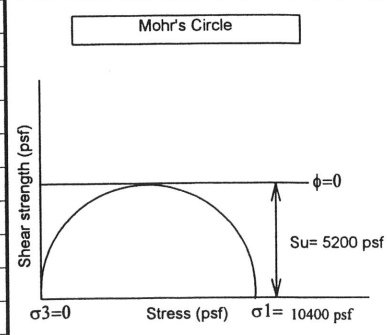
| | | |
|---|---|---|
| WET SOIL SAMPLE | | g |
| WET SOIL+ PAN MASS | | g |
| DRY SOIL + PAN MASS | | g |
| PAN MASS | | g |
| WET DENSITY | | $lb/ft^3$ |
| DRY DENSITY | | $lb/ft^3$ |

| | | | | |
|---|---|---|---|---|
| UNCONFINED COMPRESSION STRENGTH, $q_u$ | 10400.00 | psf | 497.95 | kPa |
| | 72.22 | psi | | |
| COHESION, $S_U = q_U / 2 =$ | 5200.00 | psf | 248.98 | kPa |

Ring Calibration (lbs/div): 1

| Sample Def., ΔL (in) | Proving Ring Dial (div) | Total Load on Sample (lb) | Unit Strain, ε ΔL/L0 (x 10-2) | Area Cor. Factor, (1 - ε) | Corrected Area, A' ($in^2$) | Sample Stress, $\sigma_1$ (psf) | Sample Stress, $\sigma_1$ (kPa) |
|---|---|---|---|---|---|---|---|
| 0.0000 | 0 | 0.00 | 0.0000 | 1.0000 | 4.3818 | 0.00 | 0.00 |
| 0.0100 | 6 | 6.00 | 0.2244 | 0.9978 | 4.3916 | 196.74 | 9.42 |
| 0.0200 | 16 | 16.00 | 0.4488 | 0.9955 | 4.4015 | 523.45 | 25.06 |
| 0.0300 | 34 | 34.00 | 0.6732 | 0.9933 | 4.4115 | 1109.83 | 53.14 |
| 0.0400 | 64 | 64.00 | 0.8977 | 0.9910 | 4.4215 | 2084.38 | 99.80 |
| 0.0500 | 90 | 90.00 | 1.1221 | 0.9888 | 4.4315 | 2924.52 | 140.03 |
| 0.0600 | 113 | 113.00 | 1.3465 | 0.9865 | 4.4416 | 3663.56 | 175.41 |
| 0.0700 | 134 | 134.00 | 1.5709 | 0.9843 | 4.4517 | 4334.52 | 207.54 |
| 0.0800 | 160 | 160.00 | 1.7953 | 0.9820 | 4.4619 | 5163.75 | 247.24 |
| 0.0900 | 189 | 189.00 | 2.0197 | 0.9798 | 4.4721 | 6085.74 | 291.39 |
| 0.1000 | 223 | 223.00 | 2.2442 | 0.9776 | 4.4824 | 7164.08 | 343.02 |
| 0.1100 | 261 | 261.00 | 2.4686 | 0.9753 | 4.4927 | 8365.61 | 400.55 |
| 0.1200 | 296 | 296.00 | 2.6930 | 0.9731 | 4.5030 | 9465.61 | 453.21 |
| 0.1300 | 326 | 326.00 | 2.9174 | 0.9708 | 4.5134 | 10400.92 | 498.00 |




Remarks: Material failed suddenly at maximum reported loading.

Respectfully Submitted,

Clifford A. Craft, P.E.

TerraCosta
Consulting Group

*Geotechnical Engineering*
*Coastal Engineering*
*Maritime Engineering*

Project No. 2918
March 29, 2017

Mr. David M. Lessnick
**D. MARIN DEVELOPMENT**
1900 Western Avenue
Las Vegas, Nevada 89102

RESPONSE TO CITY REVIEW COMMENTS
CYCLE TYPE 12 LDR-GEOLOGY
**5228 CHELSEA STREET**
LA JOLLA, CALIFORNIA

CITY OF SAN DIEGO PROJECT NO. 502954
REFERENCE L64A-0038

Dear Mr. Lessnick:

TerraCosta Consulting Group, Inc. (TerraCosta) is responding to the City of San Diego's review comments from Cycle 12 LDR-Geology dated March 24, 2017, along with five additional questions from Glen Gargas's March 25, 2017, email to the undersigned (Subject: Geology Rev. Comments – Blue Heron-Chelsea CDP SDP – Project No. 502954). For completeness of the record, we have restated the original comments in italics, followed by our responses. Only those items requiring responses are included below.

**LDR – GEOLOGY (Jacobe Washburn)**

*Issue No. 20: As previously requested, update the geologic map (Figure 2) to clearly delineate the location of geologic cross sections and plot the geologic structure.*

**Response:** The requested Geologic Map was provided in our December 19, 2016, Response to City Review Comments.

*Issue No. 29: As previously requested, update the geologic map (Figure 6) to show the site specific geologic structure. Also, clearly delineate the 1.5 factor-of-safety line determined via the updated slope stability analyses with respect to the coastal bluff edge.*

**Response:** We responded to this issue in our October 17, 2016, Response to Comments letter, specifically in response to Issue Nos. 3 and 4, and in our March 15, 2017, letter. As previously stated, the entire coastal bluff has a static factor of safety in excess of 1.5, and thus the 1.5 factor of safety line coincides with the coastal bluff edge. Since Figure 6 provided with our December 19, 2016, Response to Comments Letter was slightly revised and reproduced in our March 15, 2017, letter, we have revised the Geologic Map to clearly delineate the >1.5 factor of safety line as coinciding with the existing top-of-bluff.

*Issue No. 30: In the referenced report dated December 19, 2016, the project's geotechnical consultant has provided updated slope stability analyses using soil strength parameters obtained from a report prepared by Christian Wheeler Engineering on the adjacent parcel to the south. The results indicate a factor-of-safety that is less than 1.5 landward of a 25-foot coastal bluff edge setback. LDR-Geology is unable to accept a 25-foot coastal bluff-top setback based on the information provided.*

**Response:** This issue was addressed in our March 15, 2017, letter. We provided additional clarity in our responses to Mr. Gargas's five emailed questions. Those responses are provided below, starting on Page 3.

*Issue No. 31: The project's geotechnical consultant has opined that unweathered soil strengths would be well in excess of the values reported by Christian Wheeler Engineering. The geotechnical consultant has the option of providing additional analyses using site specific soil strength parameters in order to demonstrate a factor-of-safety of 1.5 or greater to justify a 25-foot coastal bluff-top setback. The consultant could consider using anisotropic strengths to model geologic structure in their slope stability analyses.*

**Response:** This issue was addressed in our March 15, 2017, letter. We provided additional clarity in our responses to Mr. Gargas's five emailed questions. Those responses are provided below, starting on Page 3.

*Issue No. 32: The project's engineer should update the appropriate plans (proposed site plan, etc.) and sections to clearly delineate the revised coastal bluff edge and coastal bluff edge setbacks depicted on Figure 6 of the referenced geotechnical report.*

**Response:** As indicated in our March 15, 2017, letter, the entire coastal bluff has a static factor of safety in excess of 1.5, and thus the 25-foot setback controls, the location of which is shown on the attached Figure 1 (which is a reproduction of Figure 6 of our December 19, 2016, Response to Comments letter), with the inclusion of the 1.5 factor of safety line coinciding with the top-of-bluff.

## ADDITIONAL ISSUES IN GLEN GARGAS'S MARCH 24, 2017, EMAIL

*Issue No. 1: A review of ASTM test methods indicates that ASTM C805-13 is a Standard Test Method for Rebound Number of Hardened Concrete; however, ASTM D5873 is a Standard Test Method for Determining Rock Hardness by Rebound Hammer Method. Clarify the method used at the subject site in the updated report.*

At TerraCosta, we use ASTM C805. This is consistent with the test method used by Katz, et al.[1], whose paper was included in our March 15, 2017, Benumof & Griggs[2], whose paper was included in our March 15, 2017, letter; Selby[3]; and others.

ASTM Standard Test Method D5873 is limited to the determination of rock hardness, a parameter used in the analysis of rock mechanics.

[1] Katz, O., Z. Reches, and J.-C. Roegiers, 1999, Evaluation of Mechanical Rock Properties Using a Schmidt Hammer, Technical Note in International Journal of Rock Mechanics and Mining Sciences 37 (2000) 723-728.

[2] Benumof, Benjamin T., and Gary B. Griggs, 1999, The Dependence of Seacliff Erosion Rates on Cliff Material Properties and Physical Processes: San Diego County, California in Shore & Beach, Vol. 67, No. 4, October 1999, Pages 29-41.

[3] Selby, M.J., 1980, A Rock Mass Strength Classification for Geomorphic Purposes: with Tests from Antarctica and New Zealand. Annals of Geomorphology, 24, p. 31 - 51: Gebruder Borntraeger, Berlin - Stuttgart.

*Issue No. 2: A review of section 4.4 of ASTM D5873 indicates that the results of the test method are not intended for conversion to strength data suitable for design. The project's geotechnical consultant should address this apparent discrepancy and provide a professional opinion regarding whether or not conversion of the test results to strength data is appropriate for evaluation of stability of the coastal bluff and analyses used to determine factor of safety.*

As indicated above, ASTM D5873 is not appropriate for evaluating the rock strengths typical of San Diego County coastal bluffs. As explained in our March 15, 2017, letter, we use the approach developed by Katz, et al., to assess rock strength.

*Issue No. 3: Provide a professional opinion on how the relationship between uniaxial strength and hammer rebound units developed from prepared hard rock samples applies to the bedded and jointed soft sedimentary rock shear strengths profile of the coastal bluff on and adjacent to the subject site.*

At TerraCosta, we typically use the rock mass strength classification developed by Selby and included as Table 2 in Benumof & Griggs' paper (a copy of which was provided with our March 15, 2017, letter).

*Issue No. 4: Clarify if the Schmidt Hammer testing is appropriate for the field conditions and material types, and the shear strength profile determined from this testing is representative of the weakest material controlling stability of the coastal bluff on or adjacent to the subject site.*

Based on our geologic mapping, we consider the coastal bluff below the subject property, although bedded and with occasional tight joints, to be massive and in general representative of a homogeneous, although layered, cliff-forming bedrock geologic unit. The Schmidt Hammer testing is appropriate for the field conditions and material types at the site, particularly given the correction coefficients developed from Shelby and others. Notably, Dr. Benumof's 1999 Ph.D. dissertation titled, "The Dynamics, Kinematics, and Geomorphic Evolution of the San Diego, California, Coastline," described the use of the Schmidt Hammer exclusively to characterize the San Diego County cliff-forming geologic units[4]. Dr. Benumof's use of the Schmidt Hammer to measure intact rock strengths is also discussed in his paper presented in *Shore & Beach*[2] (a copy of which was provided in our March 15, 2017, letter). The Benumof & Griggs paper, to this day, is still

[4] Benumof, B. T., 1999, The Dynamics, Kinematics, and Geomorphic Evolution of the San Diego, California Coastline, Ph.D. dissertation, University of California, Santa Cruz.

used by the California Coastal Commission in their evaluation of San Diego County coastal bluffs.

*Issue No. 5: The submitted geotechnical report must be signed and sealed.*

We have wet-stamped and sealed our July 19, 2016, geotechnical report. Notably, there are no substantive changes to our original July 2016 geotechnical report. If requested, we can reissue our three Response to Review Comments letters dated October 17, 2016, December 19, 2016, and March 15, 2017, wet-stamped and sealed. Per your request, we have submitted three additional copies of our July 19, 2016, geotechnical report.

We trust these responses satisfactorily address the reviewer's comments. If you have any questions, please give us a call.

Very truly yours,

TERRACOSTA CONSULTING GROUP, INC.

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

Gregory A. Spaulding, Project Geologist
C.E.G. 1863, C.H.G. 351, R.G. 5892

WFC/GAS/jg
Attachment

cc: Chandra Slaven, Blue Heron
Jacobe Washburn, City of San Diego
Jim Quinn, City of San Diego
Glenn Gargas, City of San Diego
Laura Black, City of San Diego
Gregory Hopkins, City of San Diego

# SITE PLAN

SCALE: 1"=20'
0 10 20 30
FEET

(80' RIGHT OF WAY)

CHELSEA STREET

CONCRETE SIDEWALK

CONCRETE SIDEWALK

APPROXIMATE LIMITS OF PROPOSED LANDSCAPE IMPROVEMENTS

HATCH INDICATES APPROXIMATE LIMITS OF EXISTING RESIDENCE

APPROXIMATE LIMITS OF PROPOSED RESIDENCE

ONE STORY STUCCO RESIDENCE 5228 CHELSEA ST

DECK

25-FOOT SETBACK

25-FOOT SETBACK

EXISTING BURIED BRICK WALL

NOTE: CONTOURS SHOWN ALONG THE BLUFF FACE OVERLAP IN AREAS OF OVERHANGS

1.5 STATIC F.O.S.
40' SETBACK
1.1 PSEUDO STATIC F.O.S.
25' SETBACK
DAYLIGHT LINE OF PROJECTED 75-YEAR BLUFF TOP EROSION
TOP OF BLUFF

ACCUMULATED WRACK, (KELP, COBBLE AND DEBRIS)

TOP OF BLUFF
(BLUFF TOP FACTOR OF SAFETY LINE > 1.5)

TOE OF BLUFF

OVERHANGING BLUFF

TOP OF BLUFF
(BLUFF TOP FACTOR OF SAFETY LINE > 1.5)

TOE OF BLUFF

SMALL OVERHANG

SEEP

TRANSIENT BEACH DEPOSITS

Qaf Qbp Tms TMsc TMss Qb

DESCRIPTION

Qaf - FILL
TMsc - MOUNT SOLEDAD FORMATION (CONGLOMERATE)
TMss - MOUNT SOLEDAD FORMATION (SANDSTONE)

| | |
|---|---|
| TERRACOSTA CONSULTING GROUP<br>ENGINEERS AND GEOLOGISTS<br>3890 MURPHY CANYON ROAD, SUITE 200<br>SAN DIEGO, CA 92123 (858) 573-6900 | FIGURE NUMBER<br>1 |
| PROJECT NAME<br>5228 CHELSEA STREET | PROJECT NUMBER<br>2918 |

TerraCosta Consulting Group