State of California

Business, Transportation and Housing Agency

Memorandum

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To: Jiwanjit Palaha – District 07
Project Manager

Date: January 16, 2012

File: 07-LA-10-PM 42.5
EA: 07-3X0101
Slope Stability Analysis

Attention: Giap Hoang, Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1
Branch B

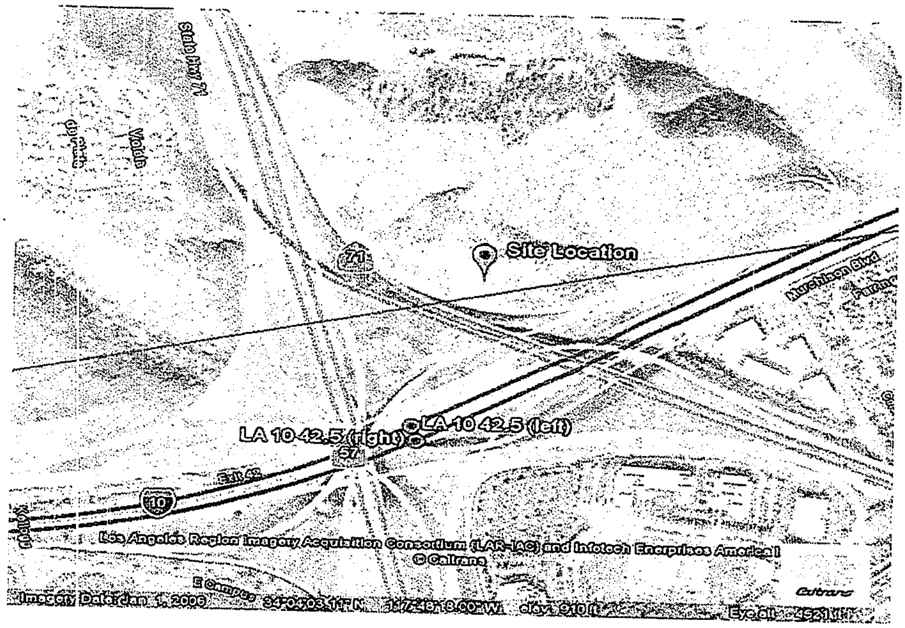
Subject: Revised Geotechnical Investigation Report for 57-10 Interchange Cut Slope near Station 15+00

Per your request, the Office of Geotechnical Design South 1 (OGDS-1), Branch B has prepared this revised geotechnical investigation study report for slope stability analysis near the 57 and 10 Interchange. This revised report provides for an adjustment to the location identification of the analyzed slope from the original December 19, 2011. The adjusted slope location analyzed is near station 15+00 per the latest layout plans provided (Updated January 17, 2012). This is changed from the previous Station 9+00.

The purpose of this report is to summarize the results of our slope stability investigation and to provide recommendations for any additional work required to keep the slope in stable condition. This report follows a September 8, 2011 Slope Stability Evaluation Report for the buttress fill located north of this cut slope at PM 42.6.

2.0 SITE DESCRIPTION

The site is located east of the I-10 WB to SR-57 NB Connector facing the westerly direction. The slope is in a cut located at postmile 42.5 near I-10 WB to SR 57 NB Connector Alignment Station 15+00. The top of the slope being evaluated is approximately 250 feet above the ramp roadway grade. Based on measurements made in the field and cross sections provided to our office the slope is at a 1.7:1 grade within the bottom 100 feet and steepens to a 1.5:1 grade within 100 to 200 feet above the travel way elevation. At the present time, no distress was observed along the slope face or within the travel way at the toe of the slope. Photos of the cut slope are attached to this memo in Appendix A. A Vicinity Map is shown in Figure 1 below.



Vicinity Map – Figure 1

3.0 BACKGROUND

As discussed in the September 8th report, the buttress fill located about 300 feet to the north of this site had failed in February 2010. Subsequently, a portion of the slope was rebuilt at a 2:1 grade with a buttress fill. The slope face was covered by hydroseeding and erosion mats.

This site located about 300 feet to the south, was apparently undisturbed during the re-grading occurring to the north. However, upon completion of the grading process, the slope at this location was left slightly steeper than the surrounding grade. As mentioned previously, no distress has been observed at this location, however, as the steepest part of the cut slope along the 10 W/B to 57 N/B connector concern was raised about the stability.

4.0 GEOTECHNICAL INVESTIGATION

The geotechnical investigation for this project consisted of obtaining soil boring information from the investigation performed for the September 8th Report. From that investigation, two exploratory borings were drilled within the buttress repair area to the north. The borings, labeled as BH-1 and BH-2 were drilled along the slope face to depths of 55 feet and 48 feet, respectively. The borings from this buttress area were applied to the project area due to the similarity of the formational materials. The borings were advanced utilizing a mud rotary method with a CS 2000 drill rig for BH-1 and CS-1000 track mounted rig for BH-2. Both rigs were equipped with an automatic hammer. Boring locations were determined based upon field observations. The elevation of the boring locations is based on layout plans provided by District

Design. A third boring, BH-3, was performed to collect a bulk sample at the toe of the slope. Figure 2 included as an attachment shows the location of the borings.

4.1 Subsurface Conditions:

The Glendora Volcanics formation was encountered in the borings. The formation is a conglomerate with hard to very hard clasts of andesite ranging from gravel to boulder sized. The matrix is intensely weathered to decomposed tuff.

No ground water was encountered to the deepest depth of 55 ft. explored during drilling.

6.0 LABORATORY TESTING

Unconfined Compression test results were used to develop parameters for slope stability analysis. The test results were taken from the investigation used for the September 8th report. Unconfined compression results on the formational material, based on Test Method CTM 221 ranged from 1071 to 4728 psi. A lower value of 20 psi was also considered, however, that result was not used in the parameter evaluation due to its extremely lower result when compared to the other two results.

7.0 GEOLOGY

7.1 Regional Geology

The project site is located in the Transverse Ranges Geomorphic Province at the southern base of the San Jose Hills, north of the Puente Hills. The Transverse Ranges Province is characterized by east-west trending mountain ranges associated with east-west trending faults.

7.2 Site Geology

The slope failure and surrounding hillside have been mapped as the Glendora Volcanic Rocks – Volcanic Conglomerate (T.W. Dibblee, 2002). The conglomerate consists mainly of andesitic clasts ranging from an average of 1 to 12 inches in size, with blocks up to 6 feet. The conglomerate is poorly lithified. The matrix is highly tuffaceous, typically lapilli tuff (J.S. Shelton, 1955).

8.0 SEISMICITY

The nearest mapped fault is the San Jose Fault, which may have been exposed during the excavation for the buttress. The San Jose Fault is listed as a reverse fault dipping 25° to the north. The maximum magnitude of the San Jose Fault is 7.3. The site PGA was determined to be 0.5g.

8.1 Liquefaction Potential

The potential for liquefaction is estimated to be very low to negligible since the material is hard formational bedrock and also that ground water was not encountered in either boring.

9.0 SLOPE STABILITY ANAYLSES

Slope stability analyses were performed using the Slope/W 2004 Version Program. The specific method for analysis used was the Morgenstern-Price Method due to its inclusion of both shear and inter-slice forces. This method provides a lower factor of safety than the Bishop's simplified method which ignores these inter-slice forces. The results of the slope stability analysis are included in Appendix B. The following table provides a summary of the unconfined compression results considered and strength parameters used for the analysis.

Table 1- Summary of Unconfined Compression Results

Boring	Sample/Depth	Material	qu (psi)
BH-1	S-1/0-5 ft	Sedimentary Rock – Conglomerate	20
BH-1	S-2/5-10 ft	Sedimentary Rock – Conglomerate	1071
BH-1	S-3/10-15 ft	Sedimentary Rock - Conglomerate	4728

Notes: Samples for S-2 and S-3 were less than 2 for the L/D ratio. Subsequently, the results were reduced by a proportional amount.

An average unconfined compression of the results for S-2 and S-3 was determined to be 1,790 psi. This result was used in the procedure for determining shear strength based on closely jointed rock masses (Hoek and Bray, 3rd Ed. 1992, pg 104). Using rock mass quality descriptions of moderately weathered and moderately fractured for sandstone type rock empirical constants were obtained to determine the shear strength (Hoek and Bray, 3rd Ed 1992, pg 108). Based on this analysis the resulting undrained shear strength used for slope stability analysis was 5,000 psf. The unit weight and friction angle used were 120 pcf and 0 degrees, respectively. The shear strengths were based on a depth and overburden pressure of 50 feet and 6000 psf, respectively.

Results of the analysis are shown in Appendix B as Figures B-1 and B-2. Figure B-1 shows the static case were the factor of safety is 1.86. Per the Bridge Design Specifications, BDS (April 2000, Section 4.4.9) the minimum factor of safety for static conditions is 1.5. Figure B-2 shows stability results under seismic conditions. Based on a PGA of 0.5g a pseudostatic coefficient of 0.2 was used in the analysis. The Factor of Safety generated was 1.22. the minimum factor of safety under seismic conditions should exceed 1.1.

10.0 DISSCUSSION AND RECOMMENDATIONS

Based on our slope stability analysis, the slope at Station 15+00 appears to be stable for the present time. However, the following recommendations should be followed to help maintain the slope:

1) The slope should be monitored regularly for sign of distress and erosion. After periods of heavy rainfall the slope should be checked for signs of erosion or tension cracking.

2) An erosion cover such as an erosion control mat should be considered for application over the slope face. This will help prevent erosion gullies from forming after periods of heavy rainfall. The mat should extend to Station 15+00 from the latest Layout Plan Sheet.

3) In order to control rainfall runoff extending eastward the existing lower bench of the buttress along with the concrete v-ditch is also advisable. The lower bench should extend to Station 15+00 from the latest Layout Plan Sheet.

4) If you have any questions or comments, please call Sam Sukiasian at or 213-620-2135 or Kristopher Barker at 213-620-2334..

Prepared by: Date: 1-17-12

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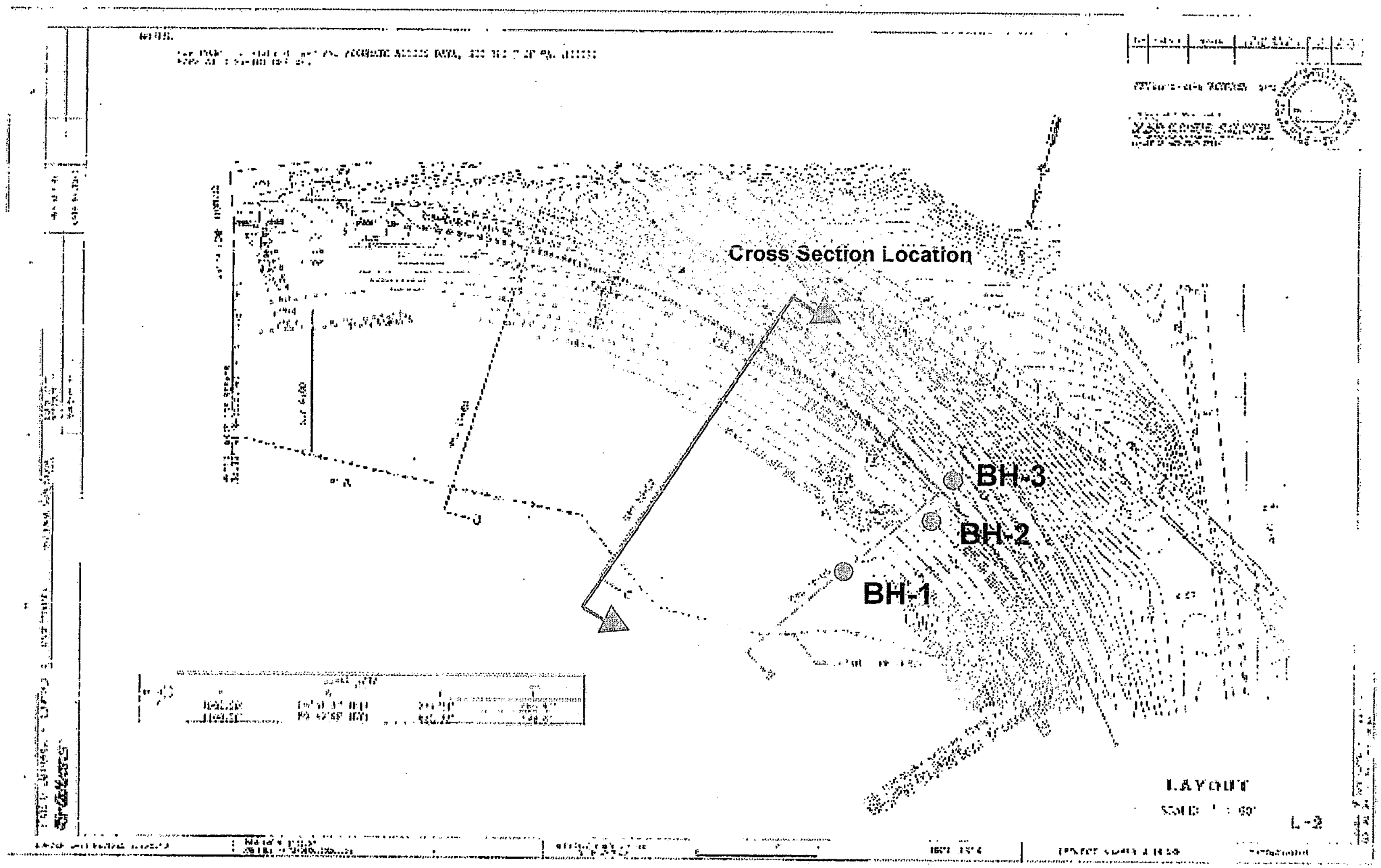

Boring Location Plan- Figure 2

APPENDIX A
Site Photos

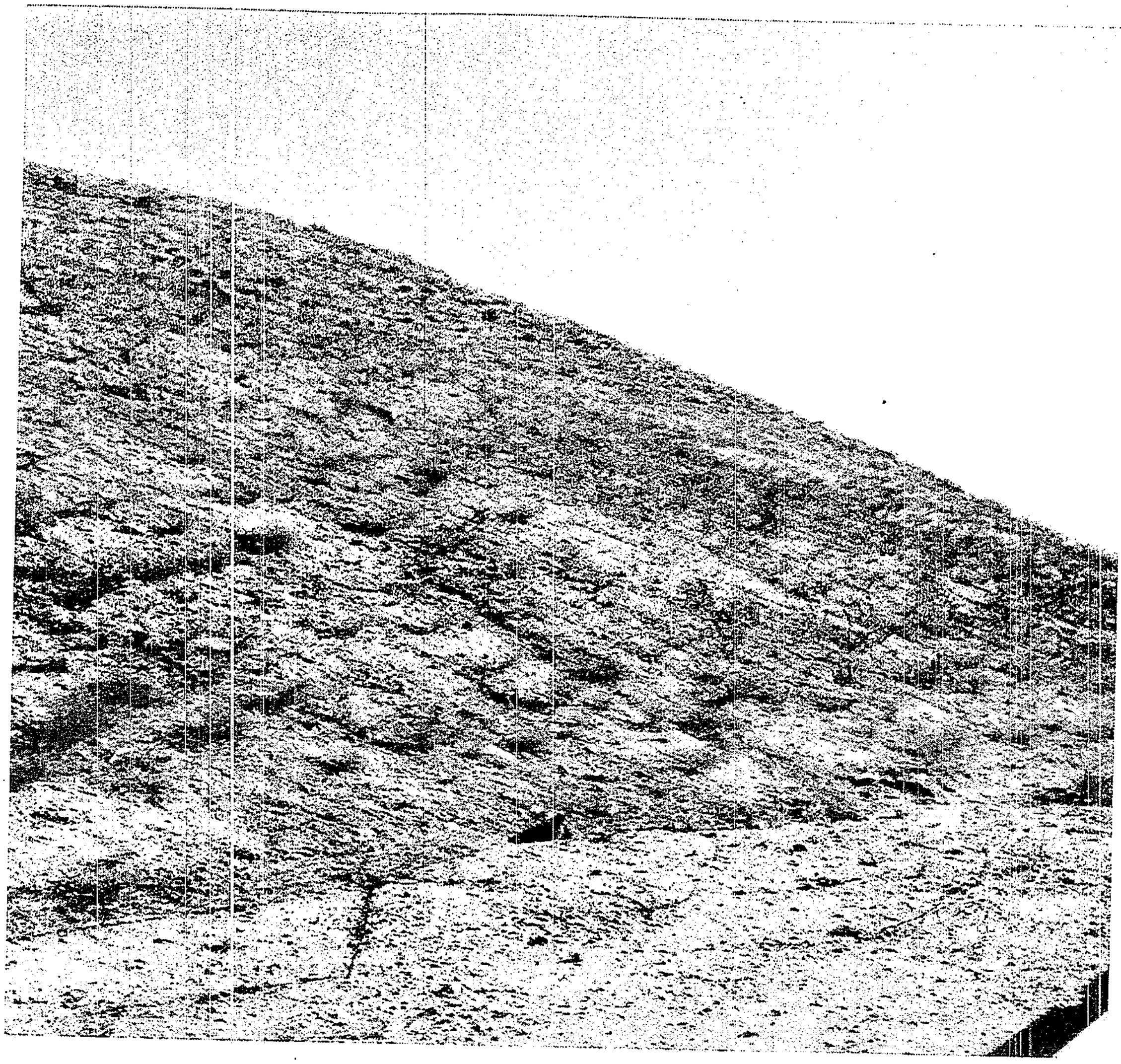

Photo 1 – View of Slope from Northwest

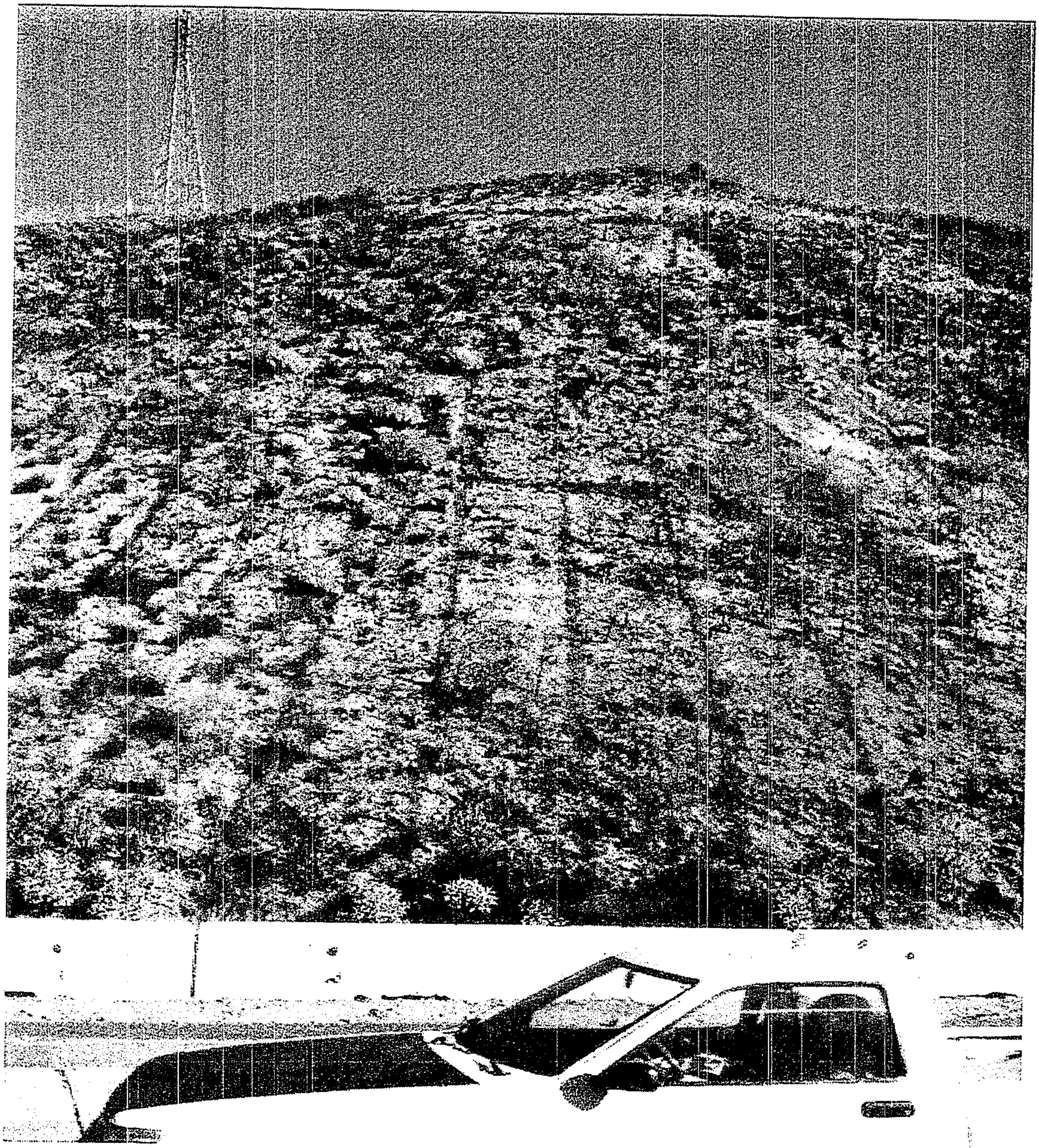

Photo 2 – View of Slope from West

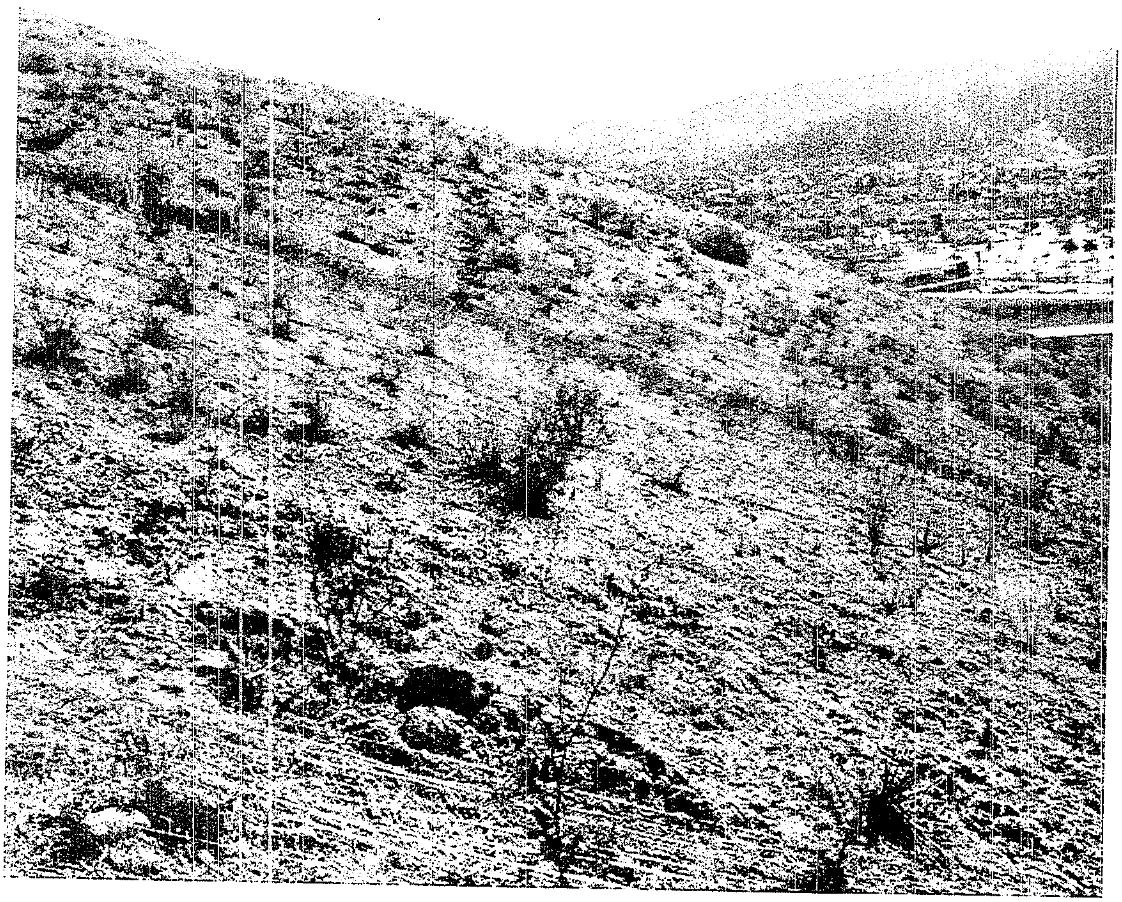

Photo 3 – View of Slope from North

APPENDIX B
Slope Stability Analysis

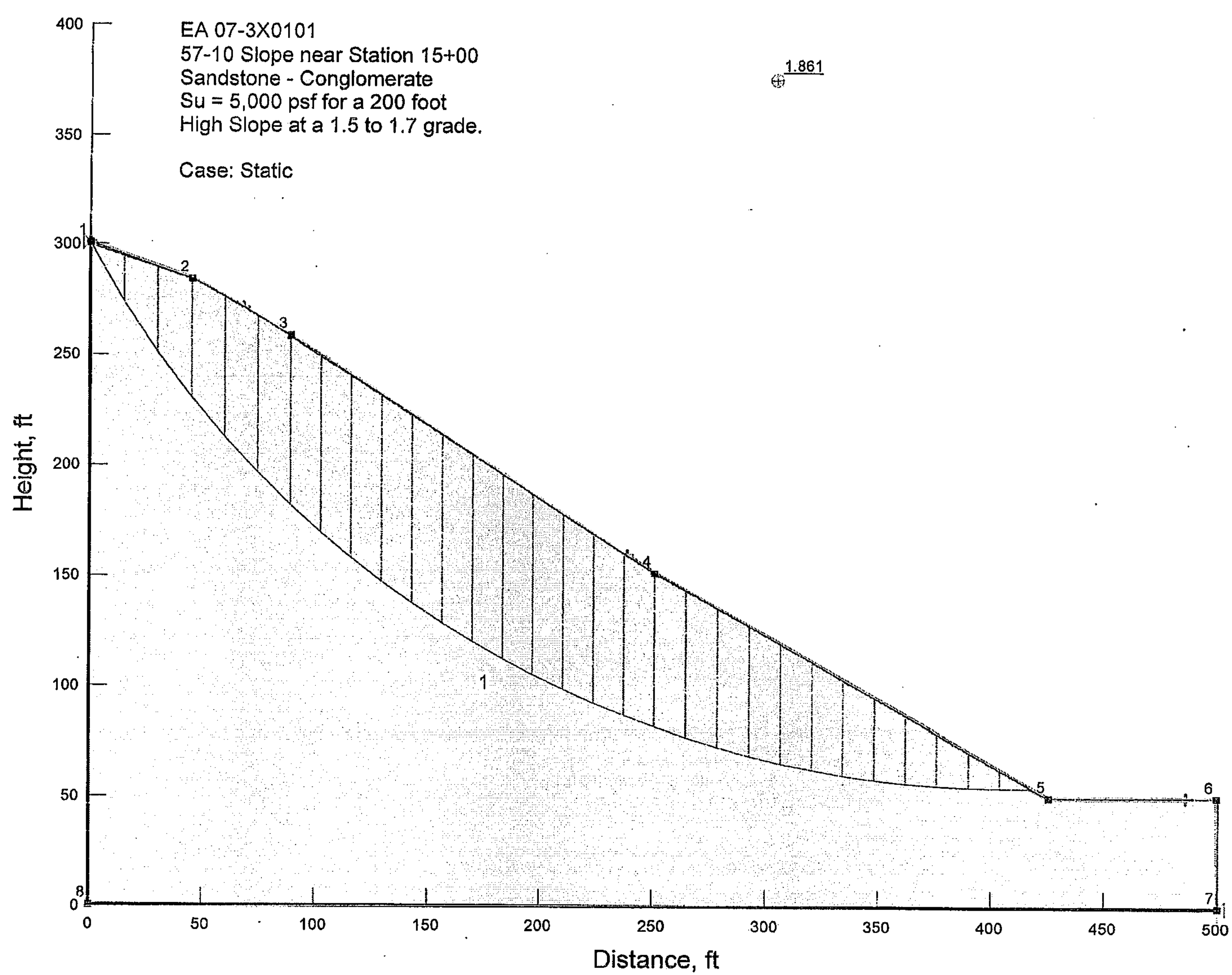
EA 07-3X0101
57-10 Slope near Station 15+00
Sandstone - Conglomerate
Su = 5,000 psf for a 200 foot
High Slope at a 1.5 to 1.7 grade.
Case: Static
1.861
Height, ft
Distance, ft
400
350
300
250
200
150
100
50
0
0
50
100
150
200
250
300
350
400
450
500
1
2
3
4
5
6
7
8

Figure A-1

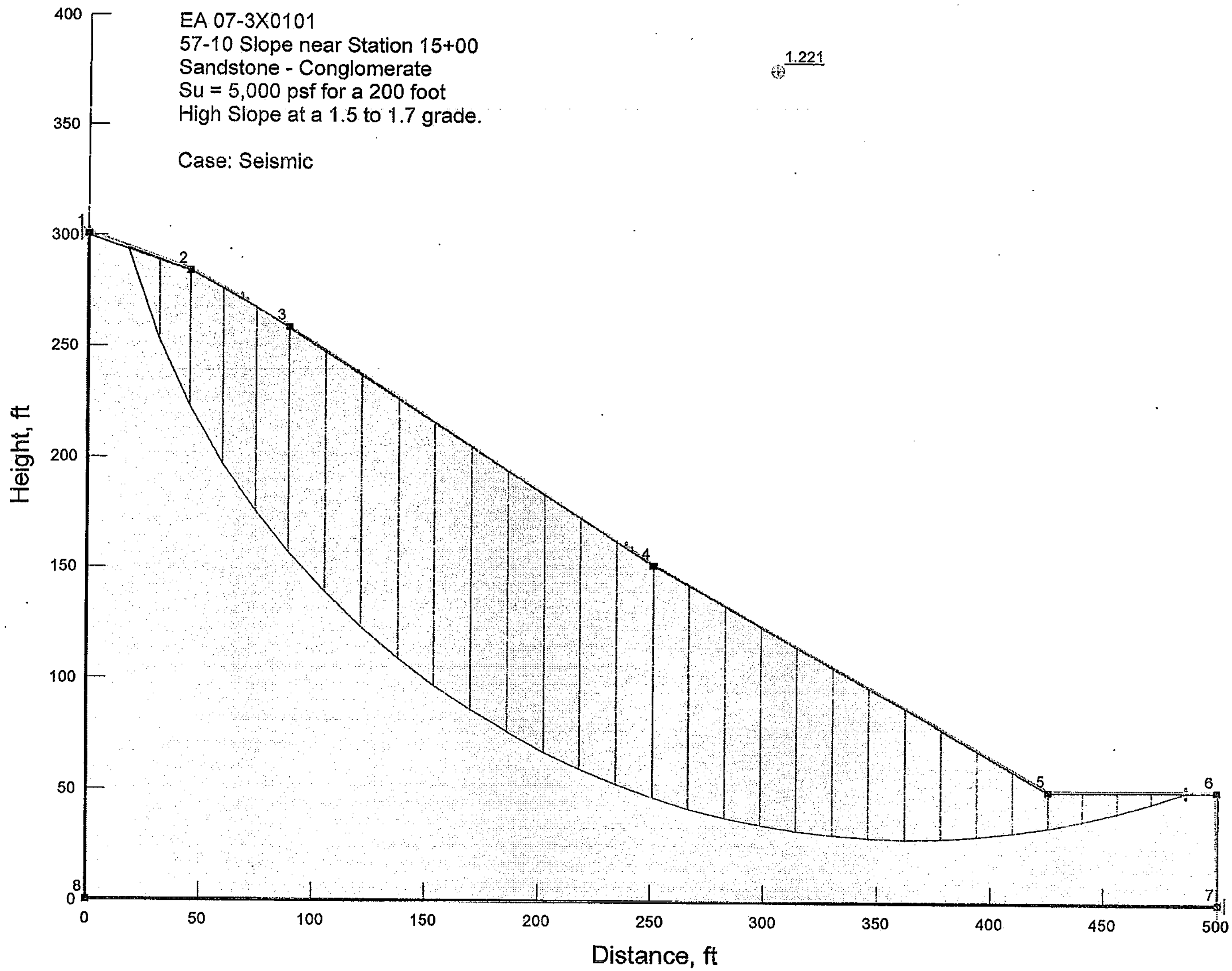
EA 07-3X0101
57-10 Slope near Station 15+00
Sandstone - Conglomerate
Su = 5,000 psf for a 200 foot
High Slope at a 1.5 to 1.7 grade.
Case: Seismic
1.221
Height, ft
Distance, ft
0
50
100
150
200
250
300
350
400
450
500
1
2
3
4
5
6
7
8

Figure A-2