February 13, 2015
File No.: 20152361.001A

Omni-Means, Ltd.
330 Hartnell Avenue, Suite B
Redding, California 96002

Attention: Russ Wenham, PE

SUBJECT: Foundation Report
Proposed SR99/Fulkerth Avenue Interchange Project
Tie-Back Anchor Retaining Walls
Wall Numbers 38-E0005 (South Wall) and 38E0006 (North Wall)
Turlock, California

Mr. Wenham:

The attached Foundation Report presents the results of the geotechnical study for the proposed anchored retaining walls located at Fulkerth Avenue and State Route (SR) 99 in Turlock, California. This report supersedes Kleinfelder's report dated September 4, 2014, describes the study and provides conclusions and recommendations for use in foundation design and construction. A separate Geotechnical Design Report dated February 13, 2015 was also prepared by Kleinfelder for the project.

Kleinfelder appreciates the opportunity to provide geotechnical engineering services to Omni-Means, Ltd., the City of Turlock, and other project designers. It is trusted this information will meet your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,

KLEINFELDER, INC.

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FOUNDATION REPORT
PROPOSED SR99/FULKERTH AVENUE
INTERCHANGE PROJECT
TIE- BACK ANCHOR RETAINING WALLS
WALLS 38E0005 (SOUTH) AND 38E0006 (NORTH)
TURLOCK, CALIFORNIA

A report prepared for:

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330 Hartnell Avenue, Suite B
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FOUNDATION REPORT
PROPOSED SR99/FULKERTH AVENUE INTERCHANGE PROJECT
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WALLS 38E0005 (SOUTH WALL) and 38E0006 (NORTH WALL)
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Kleinfelder Job No.: 20152361.001A

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1. INTRODUCTION

1.1. GENERAL

This Foundation Report (FR) presents the results of a geotechnical investigation for the proposed State Route (SR) 99 and Fulkerth Avenue Interchange Project located in Turlock, California. The overall project includes: relocation of the southbound on- and off-ramps approximately 260 feet west of the existing ramps; the widening of Fulkerth Avenue below SR99 with construction of anchored and standard retaining walls near SR99 bridge abutments; and, construction of three storm water drainage basins on the west side of SR99. This FR was prepared to provide conclusions and recommendations for use in foundation design and construction of the planned tie-back anchor retaining walls, Retaining Walls 38E0005 and 38E0006. A separate Geotechnical Design Report (GDR) for the overall Fulkerth Interchange project roadway work has also been prepared.

1.2. PROJECT DESCRIPTION

The proposed project includes construction of anchored and standard retaining walls adjacent to the SR99 bridge abutments in order to widen Fulkerth Avenue below SR99. Wall information used in our study was based on the Tie-Back Wall Location Plan and Sections dated November 16, 2010 (Sheets EX21 1 of 2 and 2 of 2) by Omni-Means, Inc. (attached) and the Fulkerth Retaining Wall Plans prepared by Cornerstone Structural Engineering, Inc. (attached). RW1 (Retaining Wall 38E00006) (the northern wall) is approximately 252.5 feet long and will extend from Sta. 20+58.00 to Sta. 23+10.45 (“F” Line) and RW2 (Retaining Wall 38E0005) (the southern wall) is approximately 247.8 feet long and will extend from Sta. 21+28.01 to 23+75.79 (“F” Line). The center 200 feet of each wall will consist of a ground anchor wall and Standard Type 1 retaining walls are planned at both ends of each anchored wall. As such, the anchor walls will extend from Sta. 20+94 to Sta. 22+94 (“F” Line) for RW1 and Sta. 21+46 to 23+46 (“F” Line) for RW2. The anchored walls will support surcharge loading from the adjacent bridge abutment foundations which consist of spread footings.

The bottoms of the anchored walls are expected to extend approximately 1.5 feet below proposed sidewalk grade. The tops of the walls are planned to extend just above the current slope face. The current slope is paved and at a gradient of approximately 1½:1 (H:V). Additional information can be gleaned from the attached retaining wall plans by Cornerstone Structural Engineering.

1.3. PURPOSE AND SCOPE OF WORK

The purpose of this investigation was to evaluate the general soil conditions, and provide geotechnical recommendations and opinions to aid in wall design. The authorized scope of services consisted of the following:

- A geotechnical field exploration program included drilling two borings near the two proposed retaining walls;
- Geotechnical laboratory testing;
- Engineering analysis; and,
- Preparation of this written report.

This report provides the following:

- A description of the proposed project;
- A summary of the field exploration and laboratory testing programs;
- A description of the site surface and subsurface conditions encountered during the field investigation, including a Log of Test Borings sheets for the retaining walls;
- Comments on the regional geology and site engineering seismology, including liquefaction potential and seismically induced settlement;
- Comments on the general corrosion characteristics of the site soils; and
- Recommendations for design of planned retaining wall systems, including recommendations for a tie-back anchor system.

Appendix A presents the Log of Test Borings Drawings. Laboratory test results are presented in Appendix B. The results of laboratory tests from a prior study by Kleinfelder are presented in Appendix C. The recommended acceleration and displacement design response spectra are presented graphically and numerically in Appendix D. Supporting calculations for the liquefaction analyses conducted on the current exploration data and lateral surcharge pressures are presented in Appendix E. The results of slope stability analyses conducted for the proposed anchored retaining walls are presented in Appendix F. Appendix G presents the completed Caltrans Comment and Response Form for our September 4, 2014 Draft Foundation Report.

1.4. POLICY EXCEPTIONS

No known exceptions to Caltrans policy were made in the geotechnical evaluation for the foundations for this project.

2. FIELD AND LABORATORY PROGRAMS

2.1. FIELD INVESTIGATION AND TESTING

The field exploration for the tie-back anchor walls was performed in conjunction with the exploration program for the Fulkerth Interchange Project. The field exploration was conducted March 17 and 18, 2011. A site reconnaissance by a staff engineer and the drilling of two (2) test borings were completed on March 17 and 18, 2011. The borings were drilled with a CME 75 truck-mounted drill rig using hollow stem auger techniques. The borings depth ranged from approximately 36½ to 51½ feet below the existing ground surface. The approximate locations of the test borings are indicated on the Log of Test Borings drawing in Appendix A of this report.

The earth materials encountered in the borings were visually classified in the field and a continuous log was recorded. In-place samples of the soils encountered were collected from the borings at selected depths by driving a 2.5-inch I.D. split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic safety hammer free falling a distance of 30-inches. In addition, an ASTM D1586 standard penetrometer without liners (barrel I.D. of 1.5 inches) was driven 18-inches in the same manner. This latter sampling procedure generally conformed to the ASTM D1586 test procedure. Resistance to sampler penetration over the last 12-inches is reported on Log of Test Boring drawings. The penetration indices listed on the logs have not been corrected for the effects of overburden pressure, sampler size, rod length, or hammer efficiency. In addition, bulk samples were obtained from auger cuttings at selected borings.

Borings A-11-001 and A-11-002 are presented on the Log of Test Boring sheets in Appendix A. The As-Built Log of Test Borings sheet for the original Fulkerth Avenue Undercrossing is also included in Appendix A.

Penetration rates determined in general accordance with ASTM D1586 were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

2.2. LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples to evaluate pertinent engineering properties. The laboratory testing program was designed with emphasis on the evaluation of geotechnical properties of the soil conditions as they pertain to the proposed construction. The laboratory testing program for the project included performing the following tests:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Direct Shear (ASTM D3080)
- Grain Size Distribution (ASTM D422, without hydrometer)
- Amount of Soil Finer than 75µ (ASTM D1140)
- Resistance Value (California Test Method No. 301)
- Soluble Sulfates (California Test Method No.417)
- Soluble Chlorides (California Test Method No.422)
- Resistivity and pH (California Test Method No. 643)

The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 4.0 ("Corrosion Evaluation"). The other test results are provided in Appendix B. Note that direct shear test results are presented for ultimate strength which is defined at 20 percent strain and for peak strength which typically occurred between 5 and 10 percent strain.

2.3. PREVIOUS GEOTECHNICAL LABORATORY TESTING

A bulk sample was obtained by Kleinfelder in 2009 from the surface of existing embankment slopes near the abutment foundations. The sample was visually classified as silty sand and a direct shear test was performed on the remolded sample of the near surface soils obtained from the embankment. The results are presented in Appendix C.

3. SITE GEOLOGY AND SUBSURFACE CONDITIONS

3.1. SURFACE CONDITIONS AND TOPOGRAPHY

The natural terrain in the project area is relatively flat. SR99 is elevated with earth embankments and is generally about 25 feet in elevation above Fulkerth Avenue. The existing southbound on- and off-ramps are immediately west of the bridge abutments. The areas of the proposed SR99 southbound on- and off-ramps are undeveloped with a heavy growth of annual weeds and grasses. The existing parallel bridges (38-142R/L) are overcrossings, which are approximately 128 feet long and 53 feet wide. The slopes in front of the abutments are currently lined with concrete with a gradient of approximately 1½:1 (H:V). Fulkerth Avenue is a 4-lane asphalt concrete roadway throughout the project limits.

3.2. REGIONAL GEOLOGY

The project site lies in the central portion of the San Joaquin Valley and the Great Valley geomorphic province in California. This province was formed by the filling of a large structural trough or downwarp in the underlying bedrock. The trough is situated between the Sierra Nevada Range on the east and south and the Coast Range on the west. Both of these mountain ranges were initially formed by uplifts that occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during the Tertiary time, and is continuing today. The trough that underlies the valley is asymmetrical, with the greatest depths of sediments near the western margin. The sediments that fill the trough originated as erosion material from the adjacent mountains and foothills.

3.3. EARTH MATERIALS

At the location of the proposed project, the native sediments in the project area have been mapped by Wagner, Bortugno and McJunkin, 1991 (San Jose 2° geologic sheet) by the United States Geological Survey (USGS) as Modesto Formation sediments of the Pleistocene age (Qm). These sediments are described as typically consisting of fine to coarse-grained sediments deposited from streams emerging from the eastern highlands.

In general, the soils encountered in the borings and test pits consisted of silty sand (SM), poorly graded sand (SP), and sand with silt (SP-SM). A layer of sandy silt (ML) was encountered in a nearby test pit DRI-3 from approximately 2 to 5 feet below grade. In the two borings drilled behind the abutments (Borings A-11-001 and A-11-002), approximately 24 to 27 feet of compacted fill was encountered over the native materials. The fill soils below the level of the existing spread foundations appear to consist of alternating layers, 5 to 10 feet in thickness, of sands with 4 to 14 percent fines (passing the No. 200 sieve) and silty sands with 17 to 26 percent fines. The natural soils in these borings consisted of interbedded layers of sands and silty sands. The soils encountered at the boring locations were medium dense to very dense to the depths explored.

A more detailed description of the materials encountered in the test borings is noted on the Log of Test Borings in Appendix A.

3.4. GEOLOGIC HAZARDS

The following conclusions were made with respect to potential geologic hazards:

- Landslides are not anticipated due to the relatively flat nature of the site.
- Deep ground subsidence due to over drafting of groundwater is not evident in the area, and is not anticipated to affect the site.
- Hydrocompactive soils are not generally present in the area, and were not observed in the test borings.
- Soils at the site have a low expansion potential. Experience in the area and performance of existing structures in the area indicate low potential for heaving at the site.
- Other than the potential for slight to moderate ground motion, no seismically related hazards are anticipated to impact the site.

3.5. GROUNDWATER CONDITIONS

Groundwater was encountered at approximately 40½ feet below existing ground surface at boring A-11-001 (drilled within the existing fill embankment between and behind the southern overcrossing abutment), which is approximately 25 feet above the general grade of the area. This indicates groundwater was generally 15 to 16 feet below the natural ground surface or 15 to 16 feet below the bottom of the planned anchor wall or at approximate Elevation 80. This is somewhat consistent with other borings drilled for the Fulkerth Interchange Project. Anchors extending below ground water will require special drilling techniques to reduce the potential for caving. Groundwater conditions at the site may experience minor change at times in the future.

4.CORROSION EVALUATION

4.1. CORROSION SCREENING

Soil samples from borings A-11-001and A-11-002 were tested to evaluate the soluble sulfate content, soluble chloride content, Minimum resistivity and pH. Specific test results are presented in Table 4.1-1.

TABLE 4.1-1
CORROSION RELATED TESTING

Boring No.	Depth (ft)	Soluble Sulfate (mg/kg)	Soluble Chloride (mg/kg)	Minimum Resistivity (ohm-cm)	pH
A-11-001	17.5	--	--	3760	7.4
	22.5	6.9	39	--	
A-11-002	20	--	--	5950	7.4
	25	3.1	45	--	

Laboratory tests indicate the soluble sulfates, soluble chlorides, and resistivity are all outside the Caltrans threshold limits. Accordingly, the soils are not considered to be corrosive to buried metals and concrete in contact with the site soils.

5. SEISMIC RECOMMENDATIONS

5.1. LOCAL FAULTING

There are no known faults, which cut through the local soil at the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone, as defined by Special Publication 42 (revised 2007) published by the California Geologic Survey (CGS). Numerous faults and shear zones within the region could influence the project site. The more significant of these faults, with respect to the project site, are Segments 7 and 8 of the Great Valley Fault (17 miles southwest), the Ortigalita Fault (27 miles southwest), and the Foothills Fault System (27 miles east)

5.2. SEISMIC DESIGN CRITERIA

Seismic design parameters were developed in accordance with the Caltrans Seismic Design Criteria Version 1.7.

The project site is located in a region with the potential for slight to moderate seismic activity. The more significant faults that could influence the project site include Segment 7 of the Great Valley Fault (Fault ID No. 25) and the Santa Cruz Mountains Section of the San Andreas Fault (Fault ID No. 310). According to the Caltrans fault database, the Great Valley Fault is a reverse fault with a dip angle of 15 degrees towards the west and assigned Maximum Magnitude (M_{Max}) of 6.7; and the Santa Cruz Mountains Section of the San Andreas Fault is a right-lateral strike slip (RLSS) with a dip angle of 90 degrees and assigned Maximum Magnitude (M_{Max}) of 7.9. The characteristics of these two faults are summarized in Table 5.2-1.

Based on the subsurface data for the site, an evaluation of the shear wave velocity in the upper 30 meters (V_{s30}) is estimated to be 361 meters per second (m/s). Based on the subsurface data and per Figure B.12 of Caltrans SDC, the site can be classified as Soil Profile Type D. The site is not located within a California deep soil basin region, as defined by Caltrans, so $Z_{1.0}$=263 m and $Z_{2.5}$=2 km were used in the probabilistic analysis and deterministic analysis. Site characteristics and governing deterministic faults are summarized in Table 5.2-1 below.

TABLE 5.2-1
SITE CHARACTERISTICS AND GOVERNING DETERMINISTIC FAULTS PARAMETERS

Site Coordinates	Lat = 37.5072 deg, Long = -120.8778 deg
Shear Wave Velocity	361 m/s
Depth to V_s=1.0 km/s, $Z_{1.0}$	263 m
Depth to V_s=2.5 km/s, $Z_{2.5}$	2 km
Fault Name and ID Number	Great Valley fault (Segment 7), No. 25
Maximum Magnitude (M_{Max})	6.7
Fault Type	Reverse
Fault Dip	15 degrees
Dip Direction	West
Bottom of Rupture Plane	10 km
Top of Rupture Plane (Z_{tor})	7 km
R_{RUP}[1]	26.7 km
R_{jB}[2]	25.7 km
R_X[3]	21.6 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	1
Fault Name and ID Number	San Andreas fault (Santa Cruz Mountains section), No. 310
Maximum Magnitude (M_{Max})	7.9
Fault Type	Right Lateral Strike Slip (RLSS)
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	15 km
Top of Rupture Plane (Z_{tor})	0 km
R_{RUP}[1]	93.7 km
R_{jB}[2]	93.7 km
R_X[3]	93.7 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0

Notes:
[1]R_{RUP} = Closest distance from the site to the fault rupture plane.
[2]R_{JB} = Joyner-Boore distance; the shortest horizontal distance to the surface projection of the rupture area.
[3]R_X = Horizontal distance from the site to the fault trace or surface projection of the top of the rupture plane.

5.2.1 Deterministic Response Spectrum

The deterministic response spectrum was calculated using the Caltrans Deterministic Spreadsheet and checked using ARS Online as required by Caltrans. The deterministic response spectrum from the Minimum Spectrum for California governed.

5.2.2 Probabilistic Response Spectrum

The probabilistic response spectrum was developed using the ARS Online as suggested by Caltrans, for $V_{s30} > 300$ m/s.

5.2.3 Design Response Spectrum

The upper envelope of the deterministic and probabilistic spectral values determines the design response spectrum. The probabilistic response spectra was found to govern for all periods. The recommended acceleration and displacement design response spectra are presented graphically in Appendix D.

5.2.4 References

Caltrans, Caltrans ARS Online, http://dap3.dot.ca.gov/shake_stable/.

Caltrans. Geotechnical Services Manual, Version 1.0, August 2009.

Caltrans. Seismic Design Criteria, Appendix B Design Spectrum

Caltrans. Website http://dap3.dot.ca.gov/shake_stable/technical.php

5.3. LIQUEFACTION POTENTIAL AND DYNAMIC COMPACTION

In order for liquefaction of soils due to ground shaking to occur, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils are non-plastic,
- Ground motion is of sufficient intensity to act as a triggering mechanism.

Based on the relative density of the site soils, groundwater conditions encountered and the design PHGA of 0.28g, evaluation based on Youd et al (2001) indicates anticipated cyclic stress from a design event (default minimum response) is not likely sufficient to result in liquefaction or seismically induced settlement. The results of the liquefaction and seismic induced settlement analyses are presented in Appendix E.

Dynamic compaction is another type of seismically induced settlement that can occur in unsaturated loose granular material or uncompacted fill soils. The subsurface conditions encountered in the borings advanced at the site are generally not considered conducive to dynamic compaction. Based on methods by Tokimatsu and Seed (1987), approximately 0.1 inch of settlement due to dynamic compaction was calculated to potentially occur during a design earthquake.

6. RETAINING WALLS

6.1. GENERAL

Based on the field exploration, laboratory testing, and geotechnical analyses, the soils at the site are suitable for supporting the planned retaining walls RW1 and RW2. Engineering analyses were performed and recommendations are provided in Sections 6.2 and 6.3 for the anchored wall portions of RW1 and RW2 on the north and south side of Fulkerth Avenue. Recommendations for the Standard Type 1 portions of RW1 and RW2 are presented in Section 6.4.

6.2. ENGINEERING ANALYSES FOR ANCHORED WALLS

6.2.1. General

Engineering analyses presented in this section were performed to evaluate and provide recommendations for:

- Lateral earth pressure and lateral surcharge pressures due to the adjacent foundations
- Dynamic incremental earth pressure
- Ultimate frictional component between the wall and the retained earth
- Tie-back anchor frictional resistance
- Lateral wall pressure required to satisfy slope stability requirements.

Note that results of the analyses discussed herein indicate that the governing lateral earth pressure for design of the tie-back retaining wall is based on those required to satisfy slope stability requirements.

6.2.2. Wall Design Cases

The critical sections of retaining wall evaluated were determined to be where the back of wall is closest to the front of the abutment foundations on each side of Fulkerth Road. The critical sections are near the west end of the each retaining wall and are represented in Cross Sections A-A' (for RW1) and C-C' (for RW2) on Sheet 2/2 of EX21 (attached). At these locations, the back of retaining wall is noted as being 7 inches and 2.1 feet away from the 'as-built' location of the existing spread foundations. The location and other information regarding the existing abutment foundations were obtained from the as-built drawings dated May 1971. Table 6.2-1 presents the data used in our analyses for the tie-back retaining wall at the two locations.

TABLE 6.2-1
TIE-BACK WALL INFORMATION

Item Description	Section A-A' (RW1)	Section C-C' (RW2)
Distance Between Abutment Footing and Back of Wall (inches)	6	24
Bottom of Abutment Footing Elevation (feet) (1)	105.25 (1)	105.25 (1)
Abutment Foundation Bearing Pressure: • Service Limit (Assumed $\phi = 0.35$)(3)	3000	3000
Width of Abutment Footing (feet)	8	8
Top of Sidewalk Elevation at Base of Wall (feet)	96.2	96.3
Bottom of Wall Elevation (feet)	94.7	94.8
Top of Wall Elevation (feet)	108.9	108.0
Height of Pressure Surface (feet)(2)	10.5	10.5

Notes (1) Elevation shown is corrected based on difference between 1929 datum on as-built drawings relative to the NAVD 1988 datum used as the basis of current topographic information. The as-built elevation was increased by 2.25 feet accordingly.
(2) Refers to portion of the wall subject to surcharge load from the abutment footing. This is equal to change in elevation from bottom of footing to bottom of wall.
(3) The As-Built Foundation Plans dated May 1971 indicated the allowable bearing pressure used in design was 1.5 tons per square feet. The resistance factor presented ($\phi = 0.35$*)* is typical and assumed.

Lateral pressures for use in final design of the retaining wall will depend on the final configuration of the tie-back anchors. Analyses were conducted for various tieback configurations. Final design is based on three levels of tie-back anchors. As such, this report presents the engineering analyses and recommendations associated with a three tie-back anchor level design referred to as Case 1. The details of the configuration used in our analyses is described below and shown on Figure 1.

<u>*Case 1 – Three Tie-back Anchor Level Design*</u> – Three rows of tie-back anchors will be installed at 15 degrees from the horizontal at depths of 1.5 feet, 4.5 feet, and 8.25 feet below the bottom of the abutment foundations. Three levels of temporary excavations will be made to depths of 3 feet, 6 feet, and 10.5 feet below bottom of the existing abutment foundations to facilitate the installation of each level of tie-back anchors. Each excavation level will be made using the ABC slot cut excavation sequencing method as described below.

<u>A-B-C Slot Cut Method -</u> The A-B-C slot-cut excavation procedure is a top-down excavation and construction method for retaining walls. The method requires that two slot widths on each side of the current slot width being excavated are either yet to be made or have been completed with the tie-back anchors and applicable portions of the retaining wall. The A-B-C slot cut excavation sequencing method consists of the following steps.

1. For the first excavation level, slots with a designated width are layed out along the face of the proposed wall and designated as A, B, or C slots in sequence (i.e ..A-B-C-A-B-C… and so forth).
2. The 'A' slots are then excavated to the bottom of the first level and the portion of the tieback anchor wall within each of the 'A' slots for the first level are constructed.
3. After the wall has been completed in the 'A' slots for the first level, the 'B' slots are excavated for the first level and that portion of the tie-back anchor wall is constructed.
4. After the wall has been completed in the 'B' slots for the first level, the 'C' slots are excavated for the first level and that portion of the tie-back anchor wall is constructed.
5. Steps 1 through 4 are then completed for each subsequent excavation level until the wall is completed. The second and subsequent levels of excavation should not be started until the wall has been completely installed in the level above it.

6.2.3. <u>Geotechnical Parameters Used in Analyses and Design</u>

Design geotechnical parameters were based on site specific laboratory data and experience in the area. Consideration was also given to correlations with sample penetration rates. Table 6.2-2 provides a summary of geotechnical design parameters for the soils used for the anchored retaining wall. For permanent design considerations the ultimate (20 percent strain) shear strength parameters were used in our analyses. For temporary slope stability considerations, the peak shear strength parameters (which typically occurred at lower strains (5 to 10 percent strain) were used in our analyses.

TABLE 6.2-2
ANCHORED RETAINING WALL DESIGN VALUES

Material	Condition		γ (pcf)	φ (deg)	c (psf)
Existing Embankment Fill Below Abutment Footing	Temporary Excavations	Peak Shear Strength	118	34	250
	Permanent Design	Ultimate (20%strain) Shear Strength	118	32	160

6.2.4. <u>Lateral Earth Pressures and Surcharge Pressures</u>

This section presents the design criteria for the proposed tie-back retaining wall based on lateral earth pressure and lateral surcharge pressure demands. The dynamic incremental earth pressure is also evaluated in this section.

We evaluated the lateral earth pressures and the lateral surcharge pressure surcharge imposed by the abutment foundation for Case 1 described in Section 6.2.2 for both Sections A-A' and C-C'. Table 6.2-3 presents tie-back wall design criteria for lateral earth pressure based on anticipated soil conditions, the ultimate shear strength parameters, and tie-back wall configuration. The lateral pressure distribution associated with service loading for tie-back design should be based on Figure 5.5.5.7.1-1 of Caltrans Bridge Design Specifications (August 2004). The lateral earth pressure dynamic increment is based on one-third of the PHGA of 0.28g, which was determined using the Caltrans SDC 1.7 criteria. When considering seismic loading, the dynamic pressure increment is in addition to the static value.

TABLE 6.2-3
TIE-BACK WALL DESIGN PARAMETERS

Item Description	Case 1 (3 Anchor Wall)
Maximum Ordinate Of The Pressure Diagram[1], p_a, (unfactored)	730 psf
Dynamic Increment (unfactored)	4 psf/ft
Nominal Frictional Coefficient[2] (unfactored) (Between wall elements and retained soil)	0.62

Note[1]: For use with Figure 5-12.1 of Caltrans Bridge Design Specifications..
[2]: Considers shotcrete of rough soil cut which results in $\delta/\phi=1.0$

The lateral surcharge pressure on the tie-back wall due to the adjacent 8-foot wide abutment foundation, with a service load of 3000 pounds per square foot, is provided on Figure 2. The supporting calculation is provided in Appendix E.

6.2.5. Tie-Back Anchor Frictional Resistance

The bonded length will be the contractor's responsibility. The minimum unbounded zone should be at least 15 feet. Additionally, as noted in Caltrans Review Comments dated December 10, 2014 the unbonded zone should extend 15 feet beyond the back side of the existing bridge footing.

6.2.6. Slope Stability and Slot-Cut Analyses

6.2.6.1. Slope Stability Analysis

Slope stability analyses were performed to evaluate the stability of temporary excavations in order to develop recommended slot-cut widths for the different levels of excavation, and to assess the required lateral resistance (from a slope stability standpoint) required of the planned wall. Analyses of Sections A-A' and C-C' for wall design Case 1 (3 rows of tie-back anchors) were conducted for each anticipated level of excavation and for the final configuration of the walls. Peak shear strength parameters and reduced tieback anchor loads were used in the evaluation of the temporary staged excavations and ultimate strengths were used for evaluation of the final required lateral resistance for the permanent wall configuration. A minimum factor of safety (FS) of 1.5 for slope stability considerations was used for both the temporary and permanent conditions.

Analyses for the upper 3 feet of wall construction (1^{st} level of excavation) considered potential failure both in front of, and behind, the abutment footing. Analysis by GEO-SLOPE utilized methods of slices by Morgenstern-Price, Janbu, and Bishop, which all produced comparable results. Analyses presented herein are based on Bishop's method. The horizontal space between the front of the footing and the back of wall are relatively small for the sections analyzed (6 inches to 2 feet) so the potential effect of a tension crack between the back of wall and front of footing was neglected. For potential failure surfaces behind the footing, an 8-foot deep tension crack was included in the analyses.

For the 1^{st} and subsequent levels of excavation, the governing condition was a failure behind the footing. Analysis of the 2^{nd} and 3^{rd} levels of excavation (prior to installation of their corresponding anchors) considered anchor tension on the upper completed portion(s) of the anchored wall. The anchor loads in the temporary excavation analyses were lower than the final recommended anchor loads for permanent design which brings some conservatism to the evaluation. A potential failure surface could not be forced between the footing and the upper completed portion of the tie-back wall. Consequently, the stability of the lower 2^{nd} and 3^{rd} levels of excavation is governed by a potential failure behind the abutment footing.

The horizontal resistance load required for the completed wall (permanent condition) to satisfy a minimum FS of 1.5 for each design case was evaluated as part of this study. The results are presented in pounds per lineal foot of wall in Table 6.2-4 and therefore need to be multiplied by the anchor spacing set by the designer in determining the required resistance load for each anchor. Since the recommended value is the horizontal component, the actual resistance load in the inclined anchor tendons will be greater than the horizontal values presented in the Table 6.2-4.

Note that horizontal resistance loads required to satisfy the minimum FS of 1.5 for permanent slope stability of the completed wall exceeded the horizontal loads when considering staged earth pressures and the lateral surcharge pressure due to the abutment foundation.

Accordingly, the anchor loads presented in Table 6.2-4 should be used in design of the wall from a geotechnical standpoint.

The results of the stability analyses for each excavation level (with and without tie-back anchors) are presented in Appendix F. A summary of the FS for each analysis case is presented in Table F-1 in Appendix F.

TABLE 6.2-4
MINIMUM REQUIRED HORIZONTAL RESISTANCE LOAD FOR ANCHORS

Retaining Wall, Section and Case	Excavation/Anchor Level	Recommended Minimum Horizontal Resistance Load in Pounds per Lf of Wall
RW1, Section A-A', Case 1 (3 Anchors)	1st Cut (0 to 3 feet)	5450
	2nd Cut (3 to 6 feet)	4575
	3rd Cut (6 to 10.5 feet)	7200
RW2, Section C-C', Case 1 (3 Anchors)	1st Cut (0 to 3 feet)	4100
	2nd Cut (3 to 6 feet)	4550
	3rd Cut (6 to 10.5 feet)	5925

6.2.6.2. Slot-Cut Analyses

Conventional stability analysis uses two-dimensional models. Slot construction is a three-dimensional problem. Generally, slot dimensions are based on judgment or experience and not any rational analysis. The City of Los Angeles uses an analytical approach to slot construction, which utilizes side resistance on a sliding block with dimensions equal to the slot cut. The side resistance is a combination of adhesion and friction, using an at-rest pressure as the normal loading. Lambe and Whitman (1969) suggested a weighted safety factor approach as an approximate treatment of three-dimensional effects. This concept is most easily applied when the three-dimensional failure surface is known (e.g., existing landslide). In general terms, the safety factor at the edge of the slot cut should be nearly the same as before the cut is made. At the middle of the slot, the safety factor will be closer to that of an infinitely long cut. As consideration moves away from the edge of the cut toward the middle, the safety factor will transition.

The analyses performed initially considered the City of Los Angeles approach. However, the comparatively large side resistance on the block (to the normal 2-dimensional driving and resisting forces), resulted in unrealistic slot dimensions (greater than 100 feet). Consequently, final analysis considered a weighted average safety factor. As a conservative judgmental approach, the maximum edge safety factor was considered to transition to the minimum safety factor in a distance 2 feet for failure surfaces in front of the footing and 6 feet for a failure behind the footing.

Utilizing the slope stability analyses results presented in Table F-1 in Appendix F, maximum slot cut widths for each level of excavation were evaluated. The FS in the transition zone is considered to be the average of the minimum FS at the edge and the minimum FS at the middle of the slot. The minimum weighted FS is set to 1.5 and the maximum slot cut distance was determined. An example calculation is included in Appendix F. The results are presented in Table F-2. Recommended maximum slot cut widths are presented in Section 6.3.

6.3. CONCLUSIONS AND RECOMMENDATIONS FOR ANCHORED WALLS

Based on our engineering analyses it is our professional opinion that design and installation of the proposed tie-back anchor walls is feasible. Due to the presence of thick layers of sands with relatively low fines (4 to 14 percent) encountered immediately below the foundation level of the existing abutment foundations, it is recommended that the temporary excavation Case 1 (3 levels of excavations with 3 rows of tiebacks) be used to construct and support the proposed walls. Lateral design pressures on the tie-back walls should be based on those developed to satisfy a minimum FS of 1.5 for slope stability considerations. The minimum horizontal anchor loads to be used in design are summarized in Table 6.3-1. Tie-back anchor should be designed based on the parameters provided in Section 6.2.5. Recommended maximum slot cut widths presented in Table 6.3-1 are for use in a typical A-B-C Slot Cut method of progressive wall excavation as described in Section 6.2-2.

TABLE 6.3-1
MINIMUM REQUIRED HORIZONTAL RESISTANCE LOAD FOR ANCHORS AND MAXIMUM SLOT CUT WIDTHS FOR CASE 1

Section and Wall	Excavation/Anchor Level	Minimum Recommended Horizontal Anchor Load (lbs./lf)	Maximum Recommended Slot Cut Width (feet)
A-A' (RW1) (3 Anchor Rows)	1st Cut (0 to 3 feet)	5450	10
	2nd Cut (3 to 6 feet)	4575	10
	3rd Cut (6 to 10.5 feet)	7200	10
C-C' (RW2) (3 Anchor Rows)	1st Cut (0 to 3 feet)	4100	10
	2nd Cut (3 to 6 feet)	4550	10
	3rd Cut (6 to 10.5 feet)	5925	10

6.4. EARTHWORK

Earthwork recommendations are specifically addressed in the GDR for the project. In general, any required fill or backfill should be constructed in accordance with the latest revisions of Caltrans Standard Specifications (2010).

7. LIMITATIONS

Recommendations contained in this report are based on the field observations, subsurface explorations, laboratory tests, and present knowledge of the proposed construction, as described in this report. It is possible that soil conditions vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, Kleinfelder should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction changes from that described in this report, the recommendations should also be reviewed. Kleinfelder has not reviewed the final grading plans or foundation plans for the project.

Kleinfelder has strived to present the findings, conclusions and recommendations in this report in a manner consistent with the standards of care and skill ordinarily exercised by members of this profession practicing under similar conditions in the vicinity of the project site, and at the time the services were performed. No warranty, express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by Kleinfelder during project construction in order to evaluate compliance with the recommendations and/or to provide supplemental recommendations, as needed, if anticipated subsurface conditions are encountered.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year (without review) from the date of the report. Land use, site conditions or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party, and client agrees to defend, indemnify, and hold harmless Kleinfelder from any claim or liability associated with such unauthorized use or non-compliance.

The scope of the geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials. Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.

ATTACHMENTS

PRELIMINARY NOT FOR CONSTRUCTION

DIST	COUNTY	ROUTE	POST MILE TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	4.3	7	19

REGISTERED CIVIL ENGINEER DATE

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REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

CITY OF TURLOCK
156 SOUTH BROADWAY
TURLOCK, CA 95380

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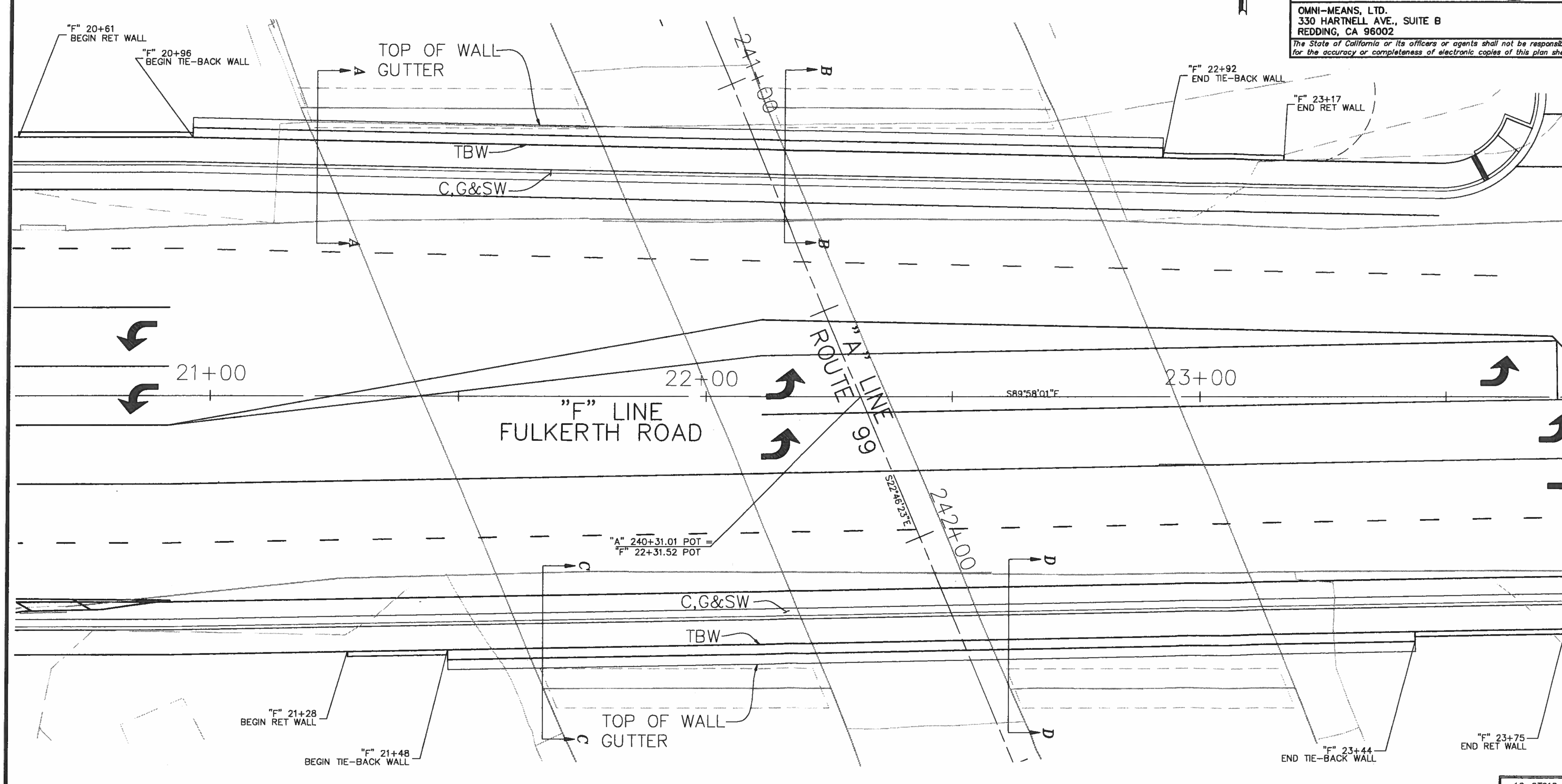

10-0T910

REVISIONS			
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1 inch = 50 ft.

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Redding, CA 96002
(530) 242-1700
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WALNUT CREEK
VISALIA
JOB NO. 45-7328-39

TIE-BACK WALL LOCATION

ROUTE 99 / FULKERTH ROAD
CITY OF TURLOCK

SCALE	1" = 10'
DESIGNED	RAW
DRAWN	WSB
CHECKED	KEM
FILE NAME	164EX021
DATE	11/16/10

SHEET No.
EX21
1 OF 2

PRELIMINARY
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CONSTRUCTION

DIST	COUNTY	ROUTE	POST MILE TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	4.3	7	19

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CIVIL
STATE OF CALIFORNIA

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156 SOUTH BROADWAY
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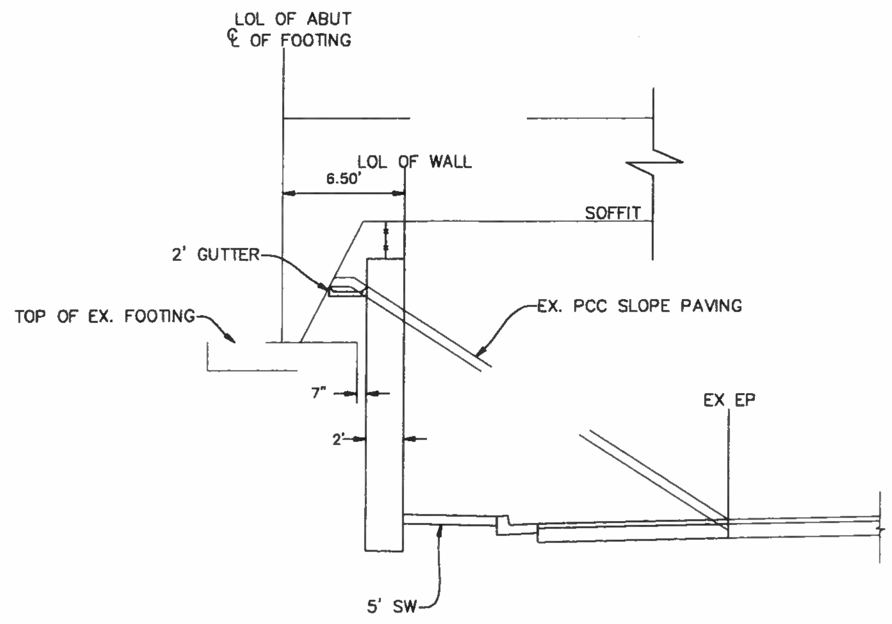
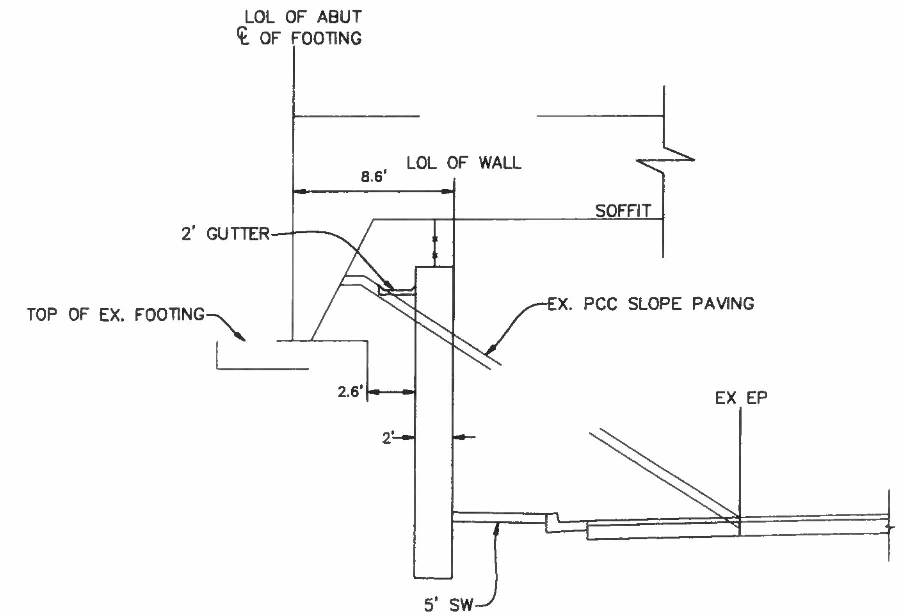
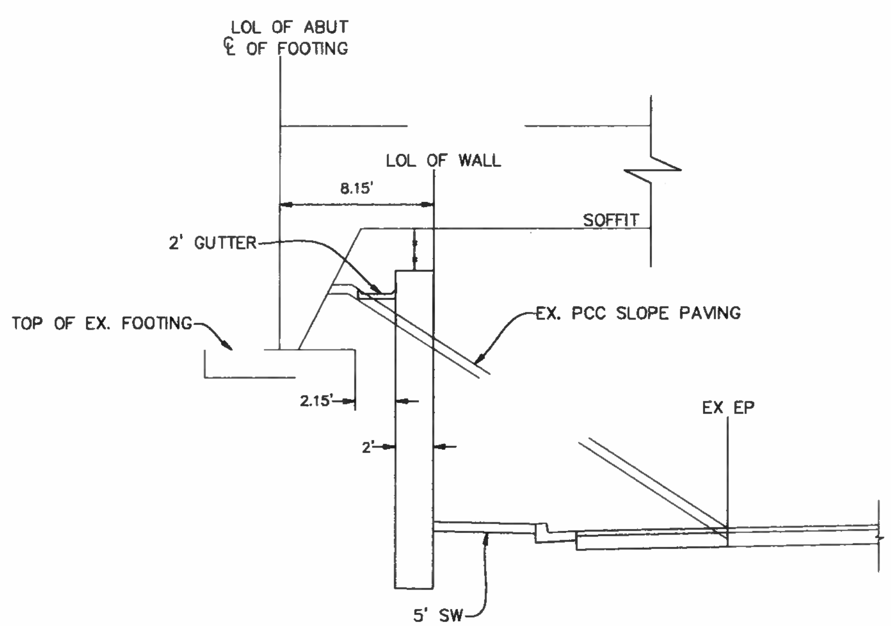


SECTION A-A



SECTION B-B



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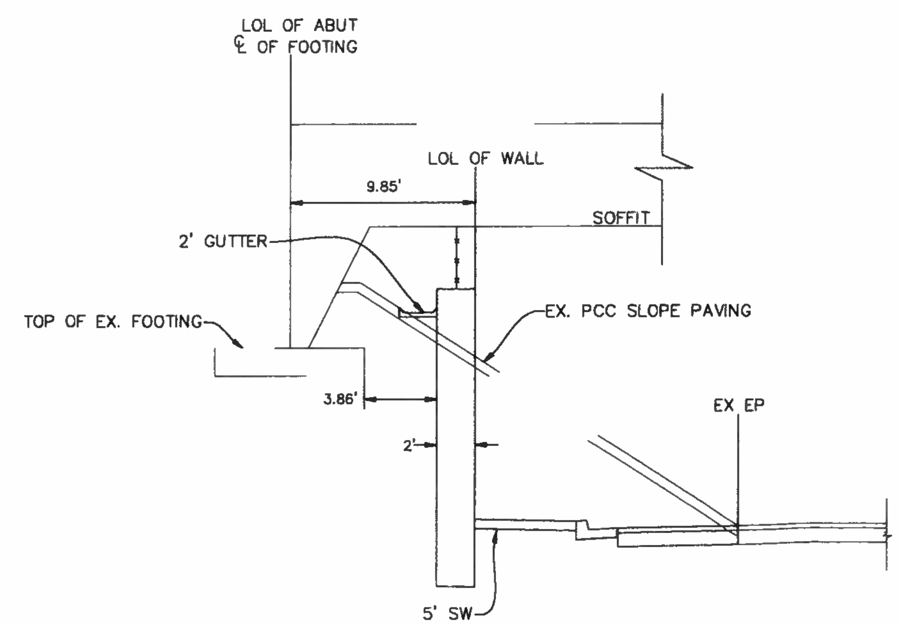

SECTION D-D

REVISIONS			
NO.	DESCRIPTION	DATE	BY

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VISALIA
JOB NO. 45-7328-39

TIE-BACK WALL LOCATION

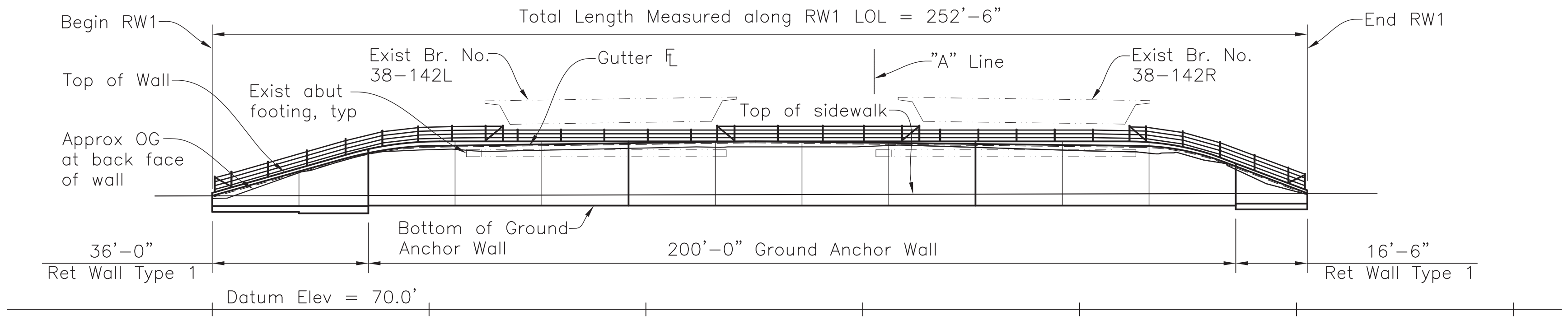
ROUTE 99 / FULKERTH ROAD
CITY OF TURLOCK

SCALE	NTS
DESIGNED	RAW
DRAWN	WSB
CHECKED	KEM
FILE NAME	164EX021
DATE	11/16/10

10-0T910

SHEET No. EX21

2 OF 2



DEVELOPED RW1 ELEVATION

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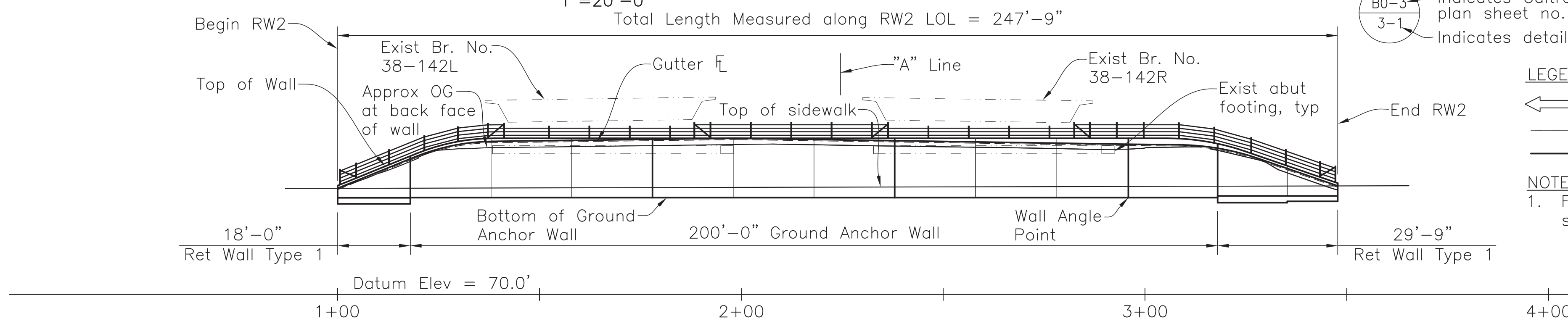

MIRRORED DEVELOPED RW2 ELEVATION

1"=20'-0"

CALTRANS STANDARD PLANS DATED 2010

A10A	ABBREVIATIONS (SHEET 1 OF 2)
A10B	ABBREVIATIONS (SHEET 2 OF 2)
A10C	LINES AND SYMBOLS (SHEET 1 OF 3)
A10D	LINES AND SYMBOLS (SHEET 2 OF 3)
A10E	LINES AND SYMBOLS (SHEET 3 OF 3)
A62B	LIMITS OF PAYMENT FOR EXCAVATION AND BACKFILL
B0-1	BRIDGE DETAILS
B0-3	BRIDGE DETAILS
RSP B3-1A	RETAINING WALL TYPE 1 (CASE 1)
RSP B3-5	RETAINING WALL DETAILS No. 1
B3-6	RETAINING WALL DETAILS No. 2
RSP B11-47	CABLE RAILING

B0-3 / 3-1 — Indicates Caltrans standard plan sheet no. / Indicates detail no.

LEGEND:

- Indicates direction of traffic
- Indicates existing structure
- Indicates new construction

NOTES:

1. For General Notes and Quantities see FOUNDATION PLAN sheet.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	R4.1/R4.9	1	X

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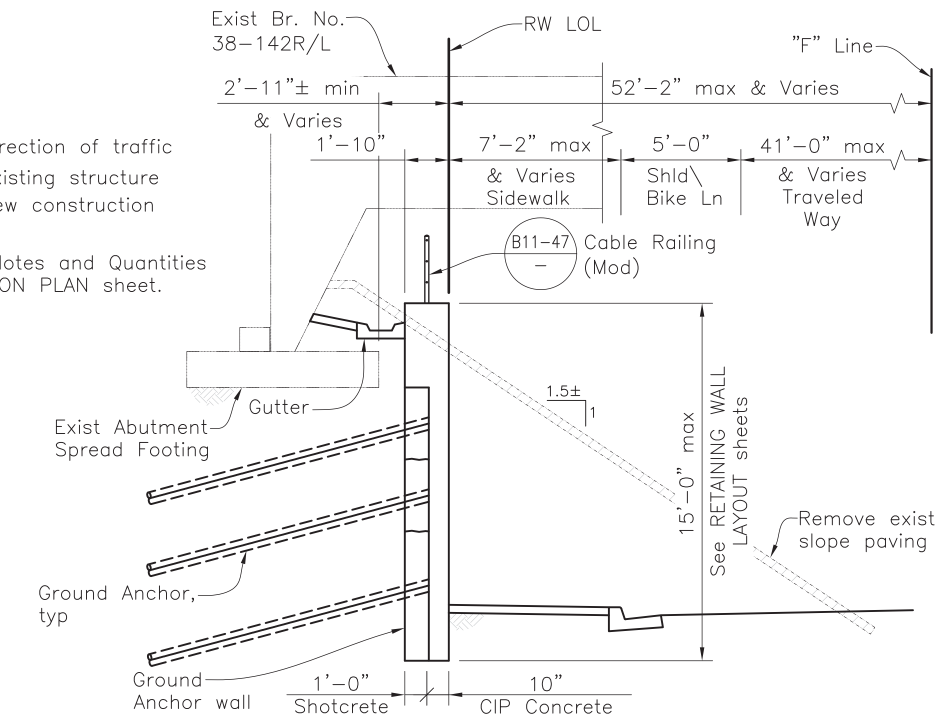
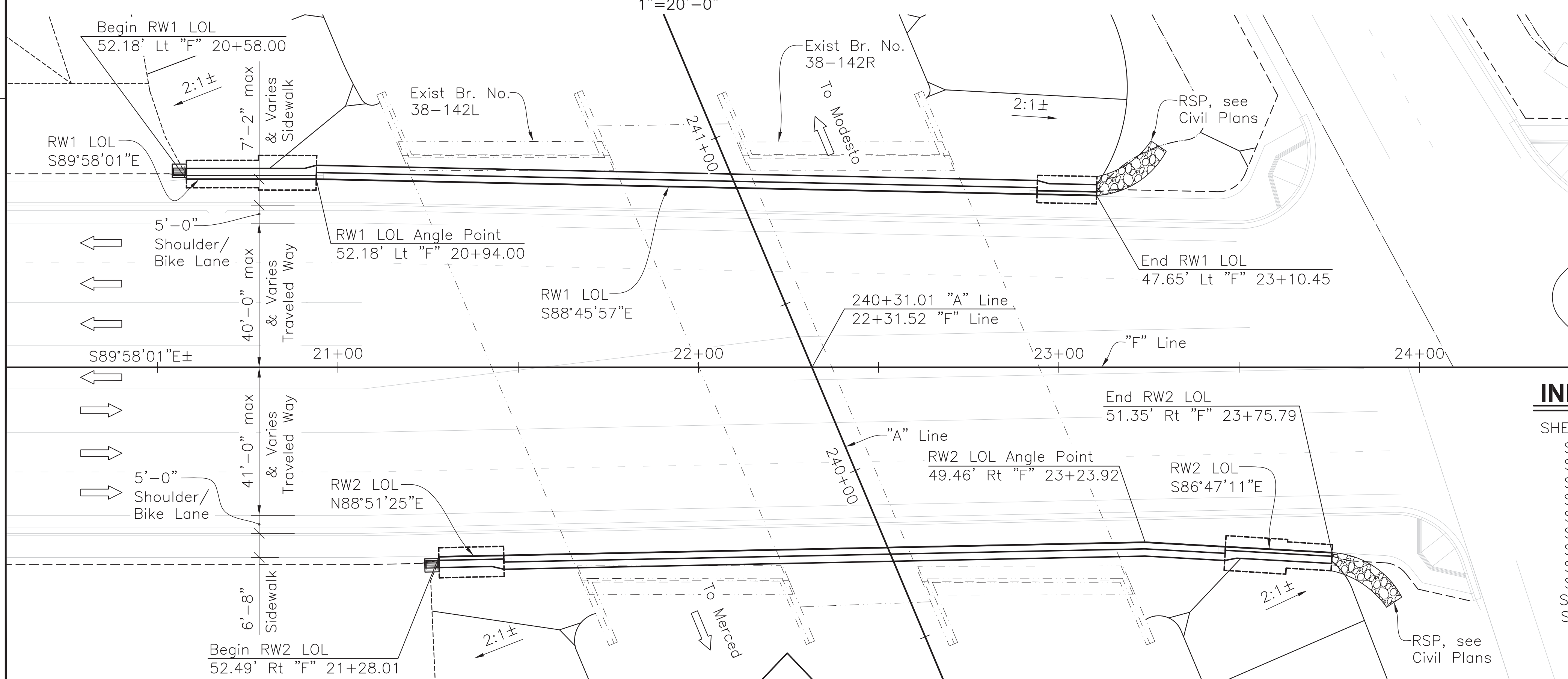
TURLOCK, CA 95830



TYPICAL GROUND ANCHOR WALL SECTION

No Scale

60% SUBMITTAL PRELIMINARY, NOT FOR CONSTRUCTION

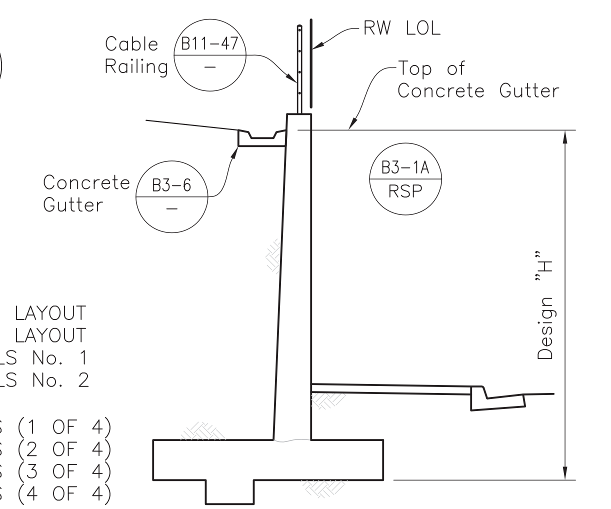


PLAN

1"=20'-0"

INDEX TO PLANS

SHEET NO.	TITLE
S-1	GENERAL PLAN
S-2	FOUNDATION PLAN
S-3	RETAINING WALL No. 1 LAYOUT
S-4	RETAINING WALL No. 2 LAYOUT
S-5	RETAINING WALL DETAILS No. 1
S-6	RETAINING WALL DETAILS No. 2
S-7	DRAINAGE DETAILS
S-8	LOG OF TEST BORINGS (1 OF 4)
S-9	LOG OF TEST BORINGS (2 OF 4)
S-10	LOG OF TEST BORINGS (3 OF 4)
S-11	LOG OF TEST BORINGS (4 OF 4)



TYPICAL TYPE 1 RETAINING WALL SECTION

No Scale

ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

PLAN CHECK SET - NOT FOR CONSTRUCTION

DESIGN OVERSIGHT

SIGN OFF DATE

	BY	CHECKED		BY	CHECKED
DESIGN	Chris Ingle		LRFD	LIVE LOADING: "LOAD & RESISTANCE FACTOR DESIGN", & "HL93 W/ "LOW BOY" & PERMIT DESIGN VEHICLE"	
DETAILS	Scott Hamm		LAYOUT	Mark Weaver	
QUANTITIES	Chris Ingle		SPECIFICATIONS	Mark Weaver	PLANS AND SPECS COMPARED

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Mark Weaver — PROJECT ENGINEER

BRIDGE NO. 38-142 R/L

POST MILE 4.55

FULKERTH RETAINING WALLS

GENERAL PLAN

DESIGN GENERAL PLAN SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: –

PROJECT NUMBER & PHASE: –

CONTRACT NO.: 10-0T9100

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES

SHEET 1 OF 11

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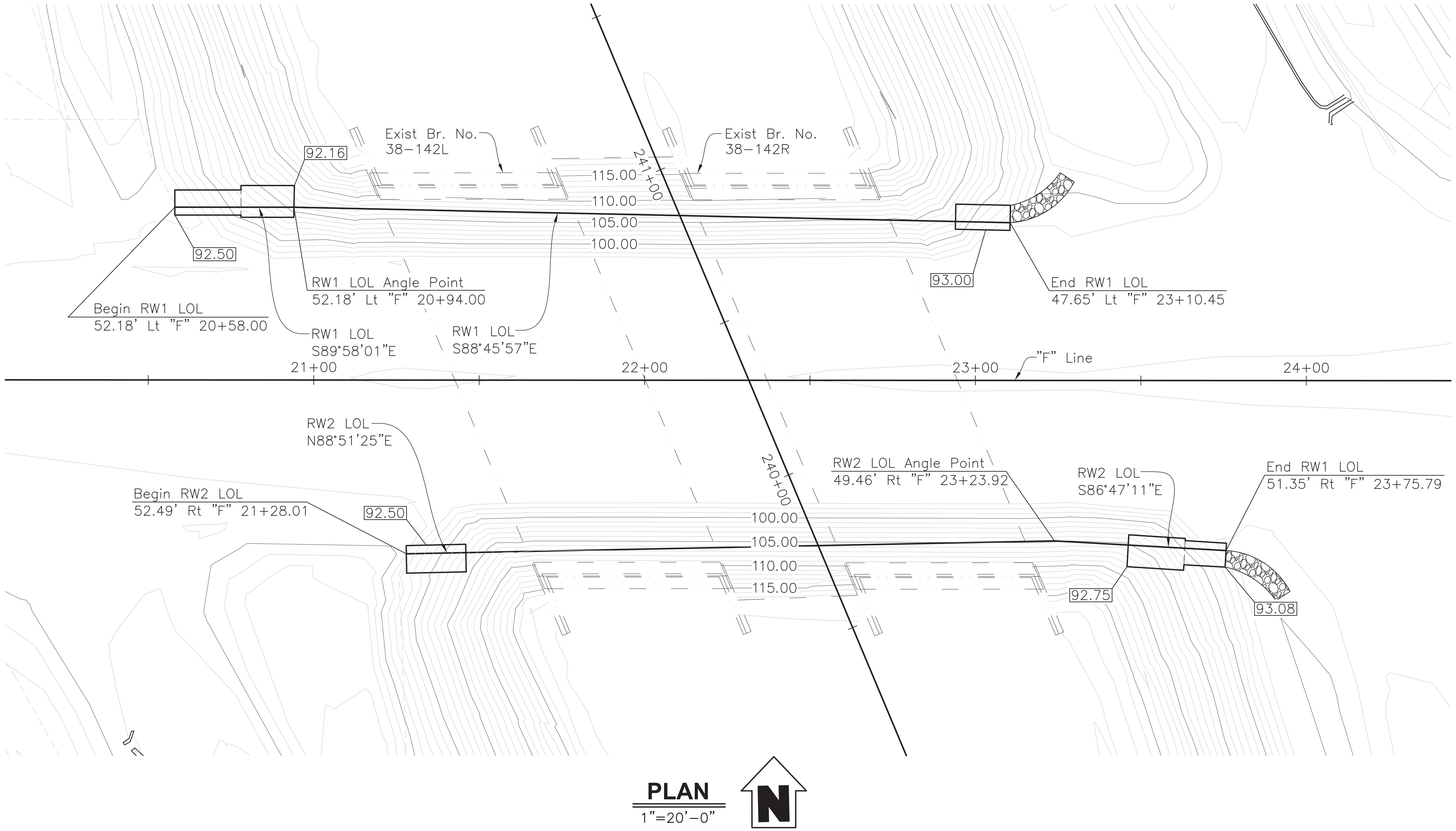
TURLOCK, CA 95830

QUANTITIES

STRUCTURE EXCAVATION (RETAINING WALL)	292	CY
STRUCTURE EXCAVATION (GROUND ANCHOR WALL)	574	CY
STRUCTURE BACKFILL (RETAINING WALL)	185	CY
STRUCTURE BACKFILL (GROUND ANCHOR WALL)	30	CY
GROUND ANCHOR	116	EA
STRUCTURE CONCRETE, RETAINING WALL	298	CY
SHOTCRETE	177	CY
BAR REINFORCING STEEL (RETAINING WALL)	100289	LB
MINOR CONCRETE (GUTTER)	506	LF
CABLE RAILING	507	LF
REMOVE SLOPE PAVING	7668	CY



PLAN
1"=20'-0"

GENERAL NOTES
LOAD AND RESISTANCE FACTOR DESIGN

DESIGN: AASHTO LRFD Bridge Design Specifications, 2012 (Sixth Edition) with California Amendments (AASHTO-CA BDS-6)

WALL DESIGN LOADING: Lateral Earth load based on Caltrans Memo to Designers 5-12 "Earth Retaining Systems Using Ground Anchors"

SOIL PARAMETERS: Geotechnical Design Report by Kleinfelder, dated September 2, 2011
ϕ = 32°
γ = 118 pcf
c = 160 psf

SEISMIC: k_h = 0.278
k_v = 0.000

REINFORCED CONCRETE: f'c = 3600 psi
fy = 60 ksi

REINFORCED SHOTCRETE: f'c = 4000 psi
f'ci = 3600 ksi
fy = 60 ksi

GROUND ANCHORS: See RETAINING WALL DETAILS No. 2 sheet.
LL = Anchor lock-off load = 0.67 (FDL)
Unbonded Length = 15'-0"

Ground Anchor Factored Test Load (FTL)			
Lift	FTL	FDL	LL
First	80	80	54
Second	90	90	60
Third	80	80	54

LEGEND:
92.50 Indicates bottom of footing elevation

60% SUBMITTAL PRELIMINARY, NOT FOR CONSTRUCTION

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

PLAN CHECK SET - NOT FOR CONSTRUCTION

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GEOTECHNICAL PROFESSIONAL | APPROVAL DATE

DESIGN OVERSIGHT
SIGN OFF DATE

SCALE: AS NOTED	VERT.DATUM NAVD 88	HORZ.DATUM NAD 83 Epoch 2007.00
PHOTOGRAMMETRY AS OF: N/A	ALIGNMENT TIES N/A	
SURVEYED BY B. Howard	DRAFTED BY J. Wunschel	
FIELD CHECKED BY T. Eckerman	CHECKED BY R. Blais	

DESIGN	BY Chris Ingle	CHECKED
DETAILS	BY Scott Hamm	CHECKED
QUANTITIES	BY Chris Ingle	CHECKED

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

Mark Weaver
PROJECT ENGINEER

BRIDGE NO. 38-142 R/L
POST MILE 4.55

FULKERTH RETAINING WALLS
FOUNDATION PLAN

FOUNDATION PLAN SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: -
PROJECT NUMBER & PHASE: -
CONTRACT NO.: 10-0T9100

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REVISION DATES | SHEET 2 | OF 11

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	R4.1/R4.9	3	X

REGISTERED STRUCTURAL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER TODD M. GOOLKASIAN No. S3543 EXP. 3/31/15 STRUCTURAL STATE OF CALIFORNIA

CORNERSTONE STRUCTURAL ENGINEERING GROUP
986 W. Alluvial Ave, Suite 201
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(559)320-3200

CITY OF TURLOCK
156 SOUTH BROADWAY
TURLOCK, CA 95830

CONSTRUCTION SEQUENCE

Stage 1 – Excavate and construct ground anchor wall for segment 1 of Lift 1
Stage 2 – Excavate and construct ground anchor wall for segment 2 of Lift 1
Stage 3 – Excavate and construct ground anchor wall for segment 3 of Lift 1
Stage 4 – Repeat Stages 1 through 3 for Lift 2 along full length of wall
Stage 5 – Repeat Stages 1 through 3 for Lift 3 along full length of wall
Stage 6 – Construct cast-in-place concrete facing in front of ground anchor wall along full length of wall
Stage 7 – Excavate and construct Type 1 retaining walls, see B3-1A RSP

NOTE:
Excavation for a lower lift of ground anchors must not be started until the wall has been completely installed in the lift above.

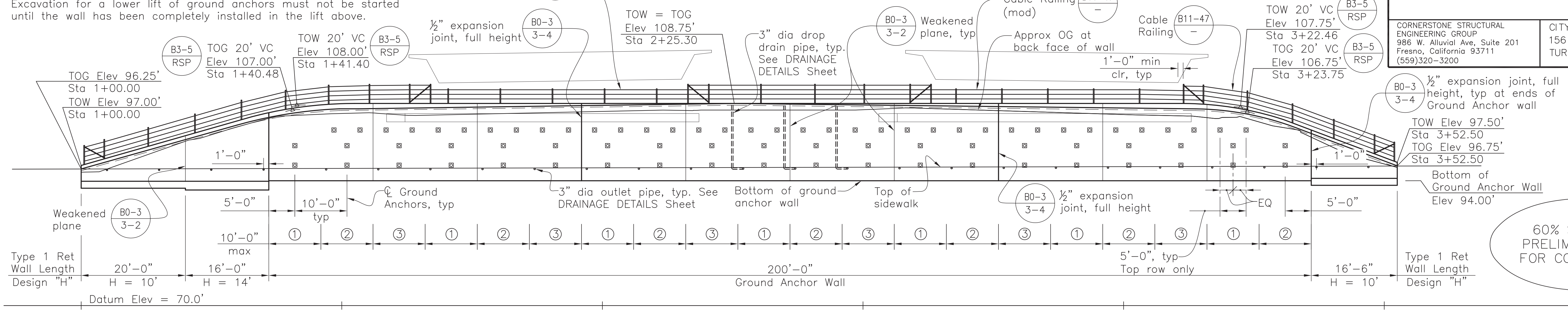
NOTES:

1. For "Section A–A", see RETAINING WALL DETAILS No. 1 sheet.
2. For Type 1 Retaining wall drainage, see DRAINAGE DETAILS sheet

LEGEND:

- ⊠ Indicates Ground Anchor locations
- – · – Indicates existing structure
- —— Indicates new construction
- TOW Indicates top of wall
- TOG Indicates top of gutter

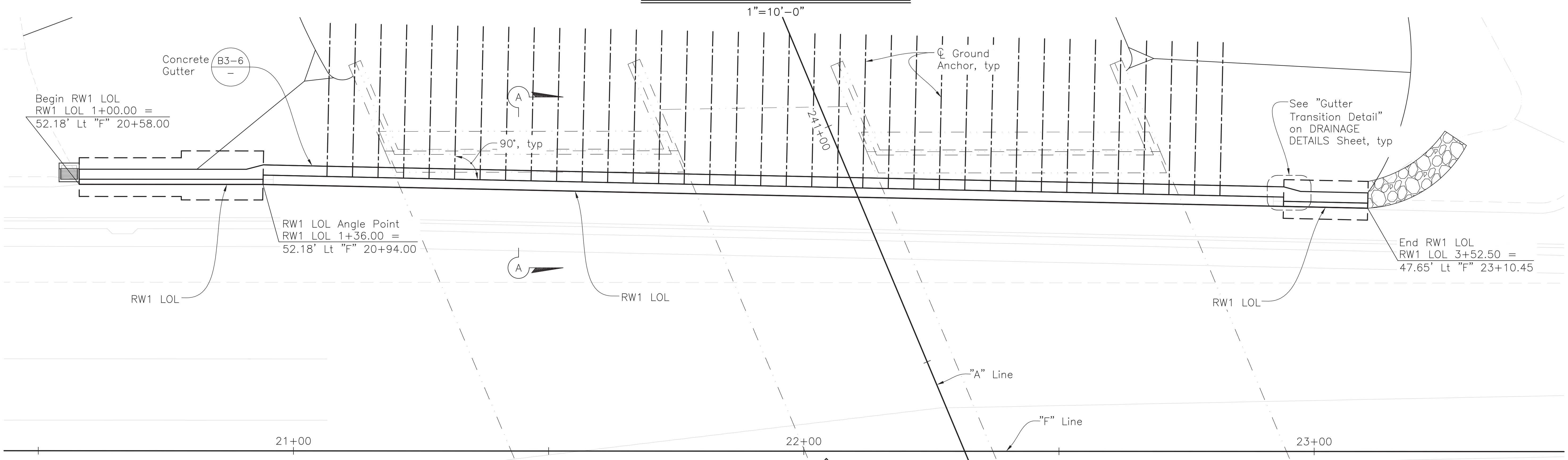
NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.



60% SUBMITTAL PRELIMINARY, NOT FOR CONSTRUCTION

DEVELOPED RW1 ELEVATION

1"=10'-0"



PLAN

1"=10'-0"

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	BY		CHECKED
DESIGN	BY	Chris Ingle	CHECKED
DETAILS	BY	Scott Hamm	CHECKED
QUANTITIES	BY	Chris Ingle	CHECKED

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Mark Weaver
PROJECT ENGINEER

BRIDGE NO. 38-142 R/L
POST MILES 4.55

FULKERTH RETAINING WALLS

RETAINING WALL No. 1 LAYOUT

DESIGN DETAIL SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: –
PROJECT NUMBER & PHASE: –
CONTRACT NO.: 10-0T9100

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SHEET 3 OF 11

FILE => W:\2014\2014003 Fulkerth Tieback Wall PS&E\2014003S3 – Retaining Wall No.1 Layout.dwg

USERNAME => shamm DATE PLOTTED => 5/8/2014 TIME PLOTTED => 3:32:54 PM

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	R4.1/R4.9	4	X

REGISTERED STRUCTURAL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER TODD M. GOOLKASIAN No. S3543 EXP. 3/31/15 STRUCTURAL STATE OF CALIFORNIA

CORNERSTONE STRUCTURAL ENGINEERING GROUP
986 W. Alluvial Ave, Suite 201
Fresno, California 93711
(559)320-3200

CITY OF TURLOCK
156 SOUTH BROADWAY
TURLOCK, CA 95830

CONSTRUCTION SEQUENCE

Stage 1 – Excavate and construct ground anchor wall for segment ① of Lift 1
Stage 2 – Excavate and construct ground anchor wall for segment ② of Lift 1
Stage 3 – Excavate and construct ground anchor wall for segment ③ of Lift 1
Stage 4 – Repeat Stages 1 through 3 for Lift 2 along full length of wall
Stage 5 – Repeat Stages 1 through 3 for Lift 3 along full length of wall
Stage 6 – Construct cast-in-place concrete facing in front of ground anchor wall along full length of wall
Stage 7 – Excavate and construct Type 1 retaining walls, see B3-1A/RSP

NOTE:
Excavation for a lower lift of ground anchors must not be started until the wall has been completely installed in the lift above.

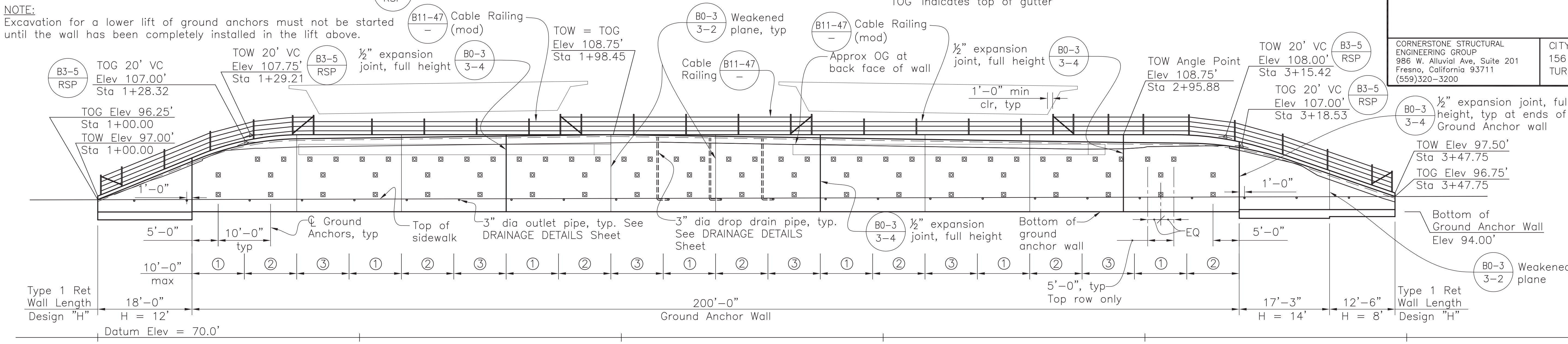
NOTES:

1. For "Section A–A", see RETAINING WALL DETAILS No. 1 sheet.
2. For Type 1 Retaining wall drainage, see DRAINAGE DETAILS sheet.

LEGEND:

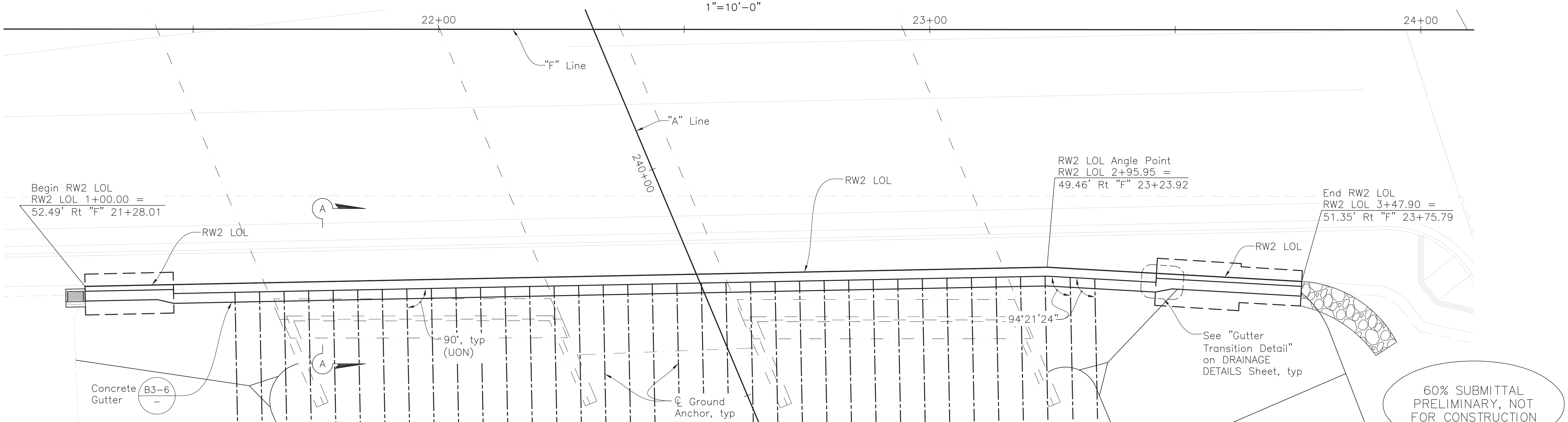
- ⊠ Indicates Ground Anchor locations
- – · – Indicates existing structure
- —— Indicates new construction
- TOW Indicates top of wall
- TOG Indicates top of gutter

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.



MIRRORED DEVELOPED RW2 ELEVATION

1"=10'-0"



PLAN

1"=10'-0"

N

60% SUBMITTAL PRELIMINARY, NOT FOR CONSTRUCTION

PLAN CHECK SET - NOT FOR CONSTRUCTION

ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

	BY	CHECKED
DESIGN	Chris Ingle	
DETAILS	Scott Hamm	
QUANTITIES	Chris Ingle	

DESIGN OVERSIGHT

SIGN OFF DATE

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Mark Weaver
PROJECT ENGINEER

BRIDGE NO. 38-142 R/L
POST MILES 4.55

FULKERTH RETAINING WALLS
RETAINING WALL No. 2 LAYOUT

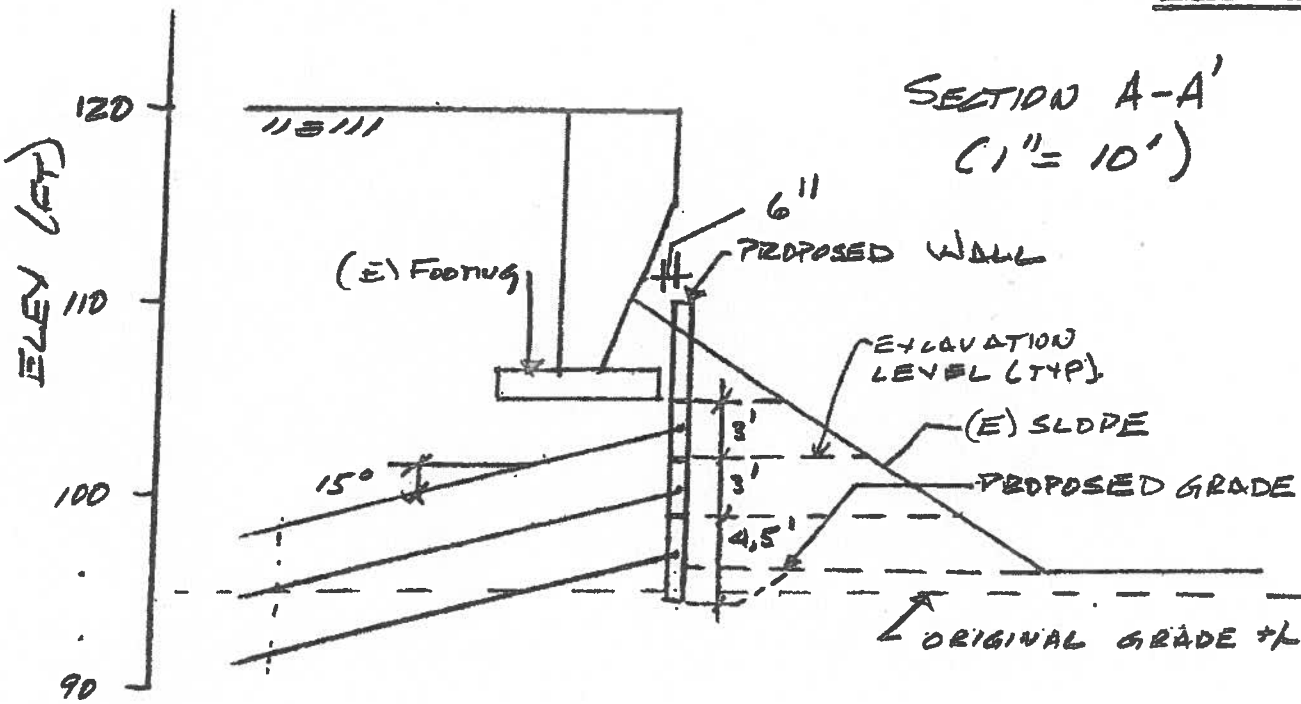
FIGURES

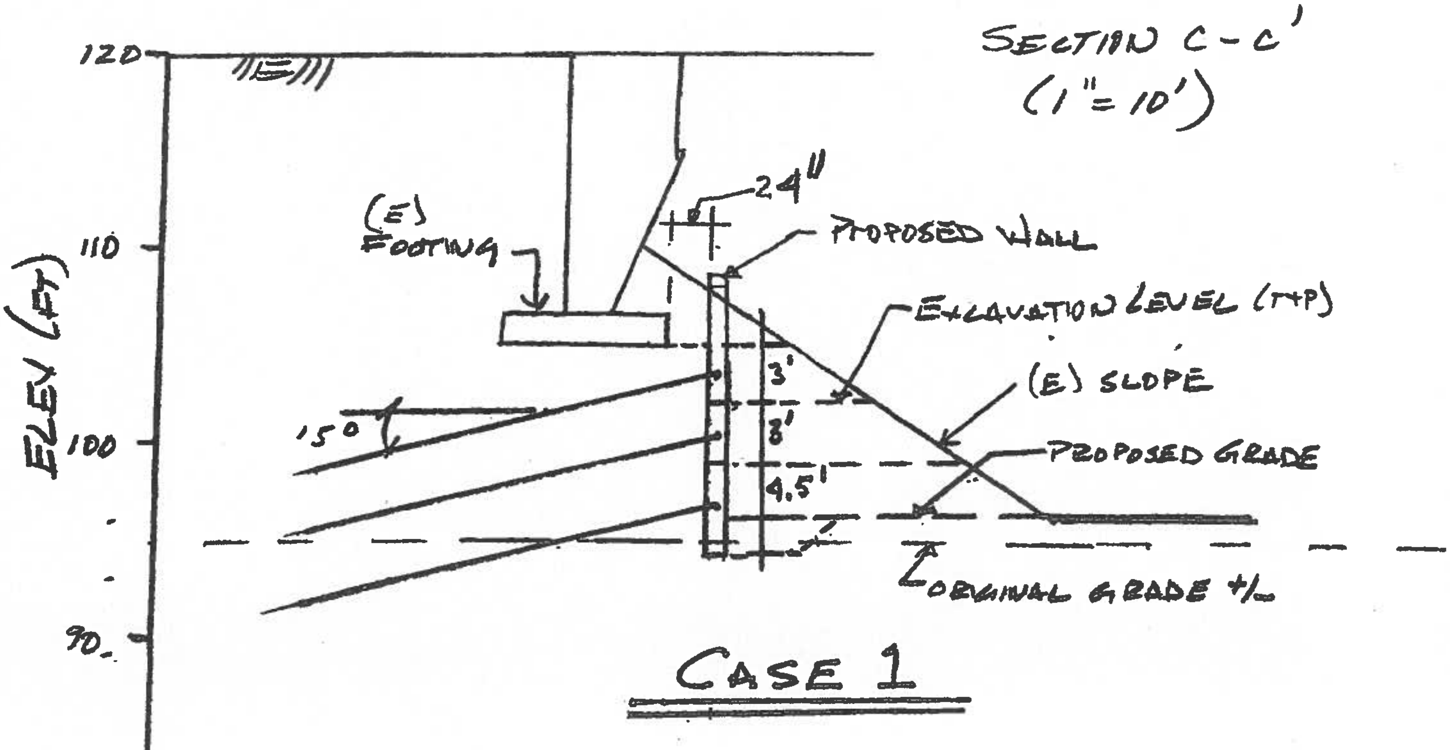
KLEINFELDER
Bright People. Right Solutions.

Sheet ____ of ____

PROJECT Fulkerth / SZ99 PROJECT NO. 98834/GEO2
SUBJECT Tieback Ret. Wall BY JJK DATE 12/28/
REVIEWED BY ____ DATE ____

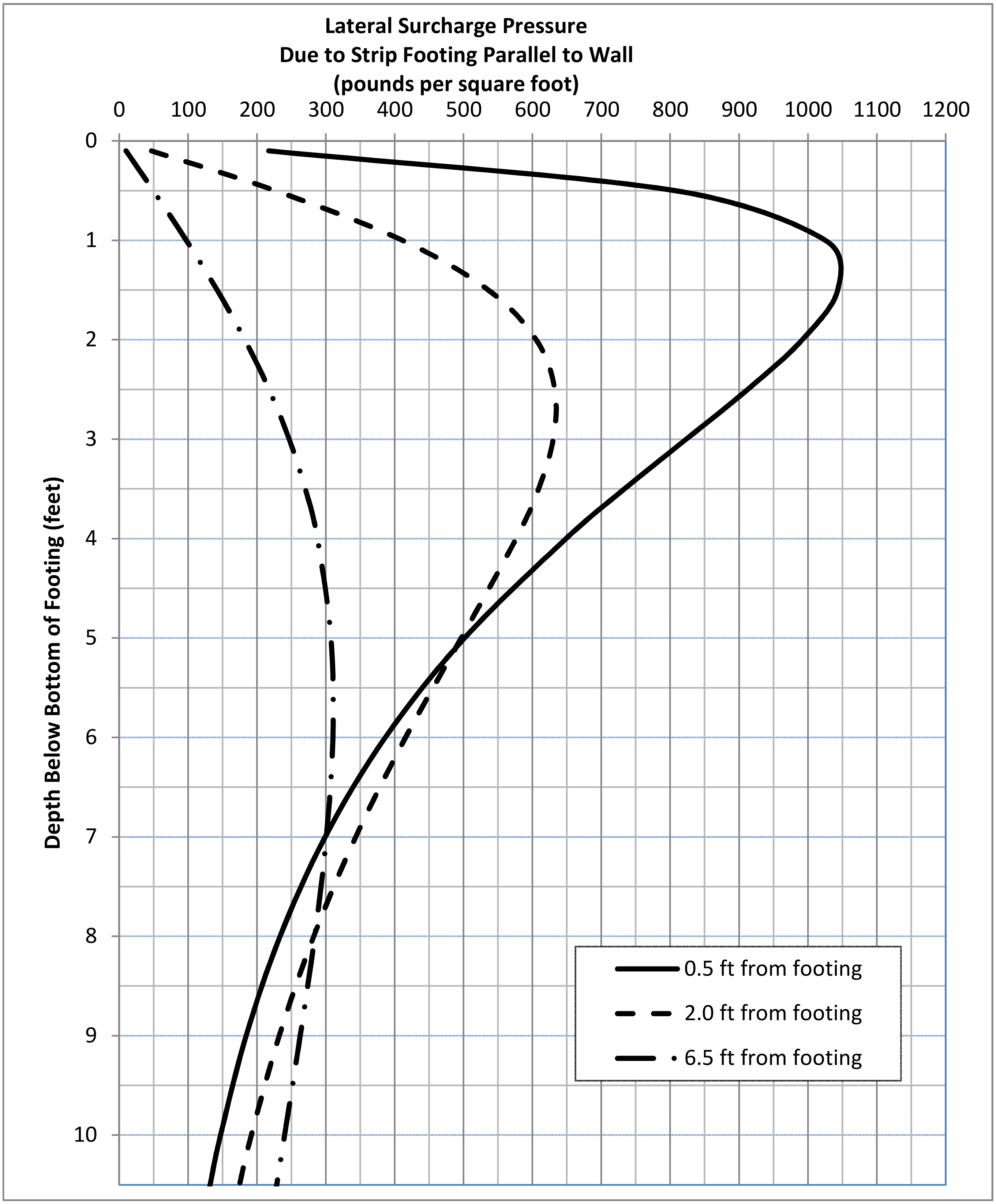
CASE 1





CASE 1

FIGURE 1



Note: Strip footing is 8 feet wide with a contact pressure of 3000 lbs/sf. (from as-built drawings)

Figure 2

September 2014

KLEINFELDER
Bright People. Right Solutions.

APPENDIX A

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO	TOTAL SHEETS
10	STA	99	R4.1/R4.9	213	216

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER Justin J. Kempton No. 2385 Exp. 9/30/15 GEOTECHNICAL STATE OF CALIFORNIA

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

Prepared for:
OMNI-MEANS, LTD
330 HARTNELL AVE., SUITE B
REDDING, CALIFORNIA 96002

Prepared by:
KLEINFELDER
5125 N. GATES AVE., SUITE 102
FRESNO, CALIFORNIA 93722

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

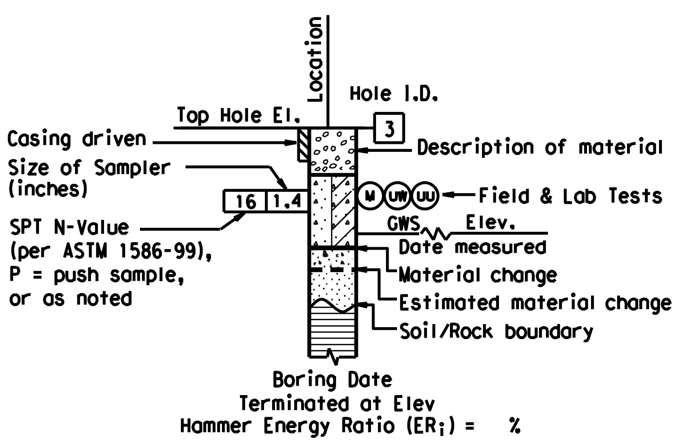
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring (hollow or solid stem bucket)
Size	R RW RC P	Rotary drilled boring (conventional) Rotary drilled with self-casing wire-line Rotary core with continuously-sampled, self-casing wire-line Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778)
⌜ ⌝	O	Other (note on LOTB)

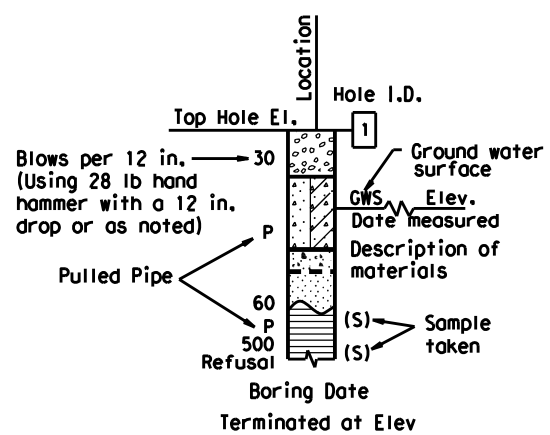
Note: Size in inches.

CONSISTENCY OF COHESIVE SOILS

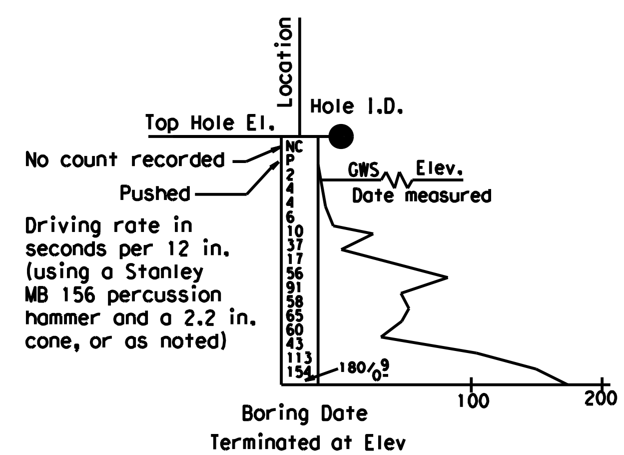
Description	Shear Strength (tsf)	Pocket Penetrometer Measurement, PP, (tsf)	Torvane Measurement, TV, (tsf)	Vane Shear Measurement, VS, (tsf)
Very Soft	Less than 0.12	Less than 0.25	Less than 0.12	Less than 0.12
Soft	0.12 - 0.25	0.25 - 0.5	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.5	0.5 - 1	0.25 - 0.5	0.25 - 0.5
Stiff	0.5 - 1	1 - 2	0.5 - 1	0.5 - 1
Very Stiff	1 - 2	2 - 4	1 - 2	1 - 2
Hard	Greater than 2	Greater than 4	Greater than 2	Greater than 2



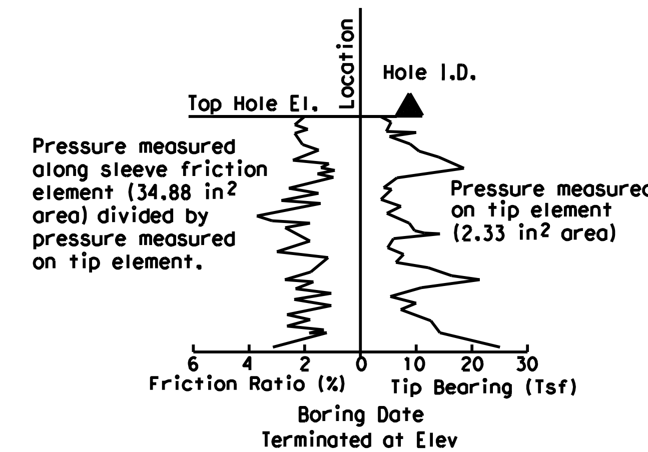
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly-graded GRAVEL Poorly-graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly-graded GRAVEL with CLAY (or SILTY CLAY) Poorly-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly-graded SAND with SILT Poorly-graded SAND with SILT and GRAVEL		
SP-SC	Poorly-graded SAND with CLAY (or SILTY CLAY) Poorly-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

- C — Consolidation (ASTM D 2435)
- CL — Collapse Potential (ASTM D 5333)
- CP — Compaction Curve (CTM 216)
- CR — Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- CU — Consolidated Undrained Triaxial (ASTM D 4767)
- DS — Direct Shear (ASTM D 3080)
- EI — Expansion Index (ASTM D 4829)
- M — Moisture Content (ASTM D 2216)
- OC — Organic Content-% (ASTM D 2974)
- P — Permeability (CTM 220)
- PA — Particle Size Analysis (ASTM D 422)
- PI — Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- PL — Point Load Index (ASTM D 5731)
- PM — Pressure Meter
- R — R-Value (CTM 301)
- SE — Sand Equivalent (CTM 217)
- SG — Specific Gravity (AASHTO T 100)
- SL — Shrinkage Limit (ASTM D 427)
- SW — Swell Potential (ASTM D 4546)
- UC — Unconfined Compression-Soil (ASTM D 2166); Unconfined Compression-Rock (ASTM D 2938)
- UU — Unconsolidated Undrained Triaxial (ASTM D 2850)
- UW — Unit Weight (ASTM D 4767)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	R4.1/R4.9	214	216

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — Justin J. Kempton — No. 2385 — Exp. 9/30/15 — GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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Prepared for:
OMNI-MEANS, LTD
330 HARTNELL AVE., SUITE B
REDDING, CALIFORNIA 96002

Prepared by:
KLEINFELDER
5125 N. GATES AVE., SUITE 102
FRESNO, CALIFORNIA 93722

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

MOISTURE

Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% - 10%
Little	15% - 25%
Some	30% - 45%
Mostly	50% - 100%

PARTICLE SIZE

Description		Size (in.)
Boulder		Greater than 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		Less than 1/300

TIME PLOTTED => $TIME
DATE PLOTTED => $DATE
USERNAME => $USER

X
DESIGN OVERSIGHT
X
SIGN OFF DATE

DRAWN BY: D. FAHRNEY
CHECKED BY: M. BELTRAN
M. SHUBERT
FIELD INVESTIGATION BY:
DATE: X

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

X
PROJECT ENGINEER

BRIDGE NO. 38-142 R/L
POST MILES R4.55

SR99 AND FULKERTH ROAD INTERCHANGE
LOG OF TEST BORINGS (2 of 3)

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: X
PROJECT NUMBER & PHASE: X
CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES X

SHEET 10 OF 12

FILE => $REQUEST

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	STA	99	R4.1/R4.9	215	216

GEOTECHNICAL PROFESSIONAL DATE

PLANS APPROVAL DATE

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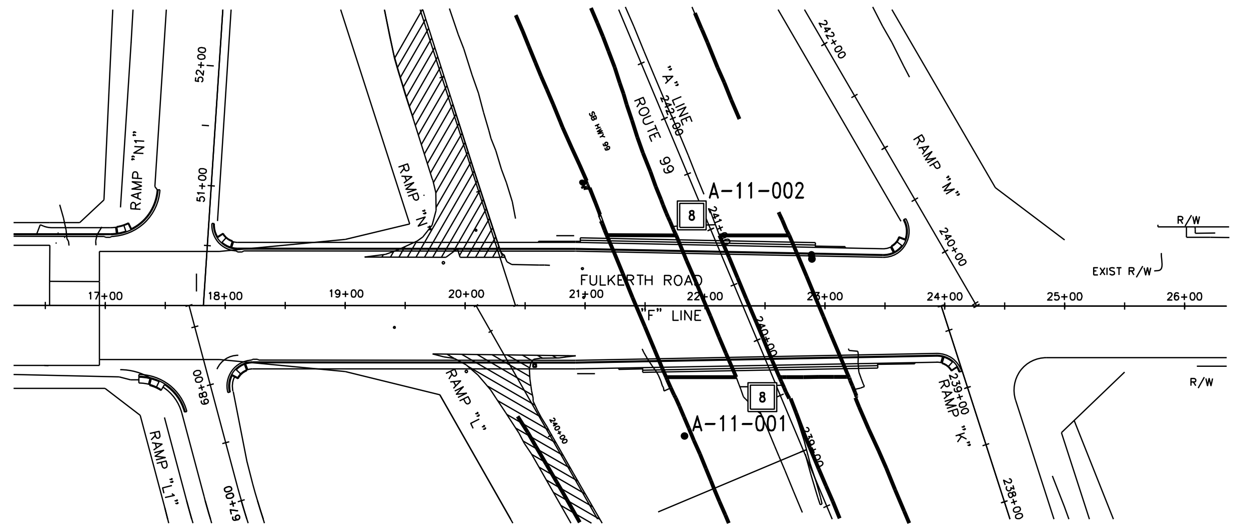
REGISTERED PROFESSIONAL ENGINEER Justin J. Kempton No. 2385 Exp. 9/30/15 GEOTECHNICAL STATE OF CALIFORNIA

Prepared for:
OMNI-MEANS, LTD
330 HARTNELL AVE., SUITE B
REDDING, CALIFORNIA 96002

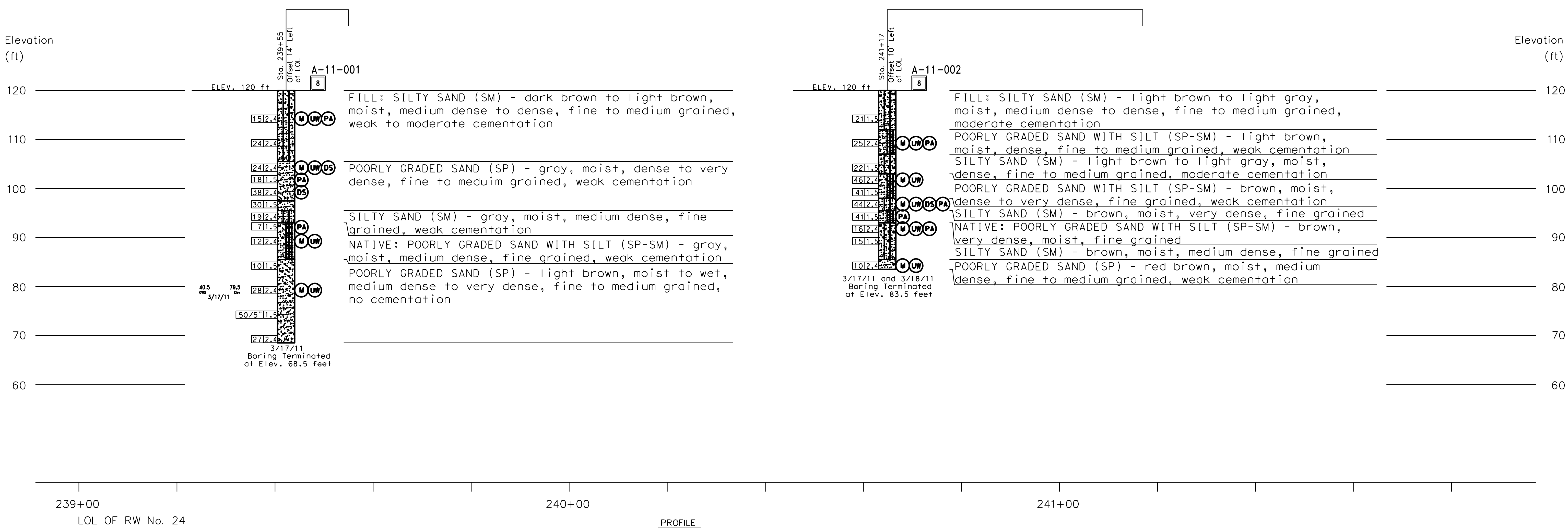
Prepared by:
KLEINFELDER
5125 N. GATES AVE., SUITE 102
FRESNO, CALIFORNIA 93722

NOTES:

1. B.M. R.P. No. 3 Elev. 290.85
   $1\frac{1}{4}$" I.P. w/ H&T, dn 0.6'
   34" Lt 22+25.00 "C"
2. Groundwater was encountered within the depths of exploration at 40.5 feet below existing grade in boring A-11-001.
3. Hammer type - Automatic safety hammer with a 140 lb safety drop hammer dropping 30 inches.



PLAN
1" = 80'



PROFILE
Vertical 1" = 10'
Horizontal 1" = 10'

X DESIGN OVERSIGHT	DRAWN BY	D. FAHRNEY	M. SHUBERT FIELD INVESTIGATION BY:
X SIGN OFF DATE	CHECKED BY	M. BELTRAN	DATE: X

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

X PROJECT ENGINEER

BRIDGE NO. 38-142 R/L
POST MILES R4.55

SR99 AND FULKERTH ROAD INTERCHANGE
LOG OF TEST BORINGS (3 of 3)

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: X
PROJECT NUMBER & PHASE: X
CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES X

SHEET 11 OF 12

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

DIVISION OF ENGINEERING SERVICES - GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	POST MILES-TOTAL PROJECT	Sheet No.	Total Sheets
10	STA	99	R4.1/R4.9	216	216

REGISTERED GETECHNICAL ENGINEER DATE

REGISTERED PROFESSIONAL ENGINEER Justin J. Kempton No. 2385 Exp. 9/30/15 GEOTECHNICAL STATE OF CALIFORNIA

LOG OF TEST BORINGS

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

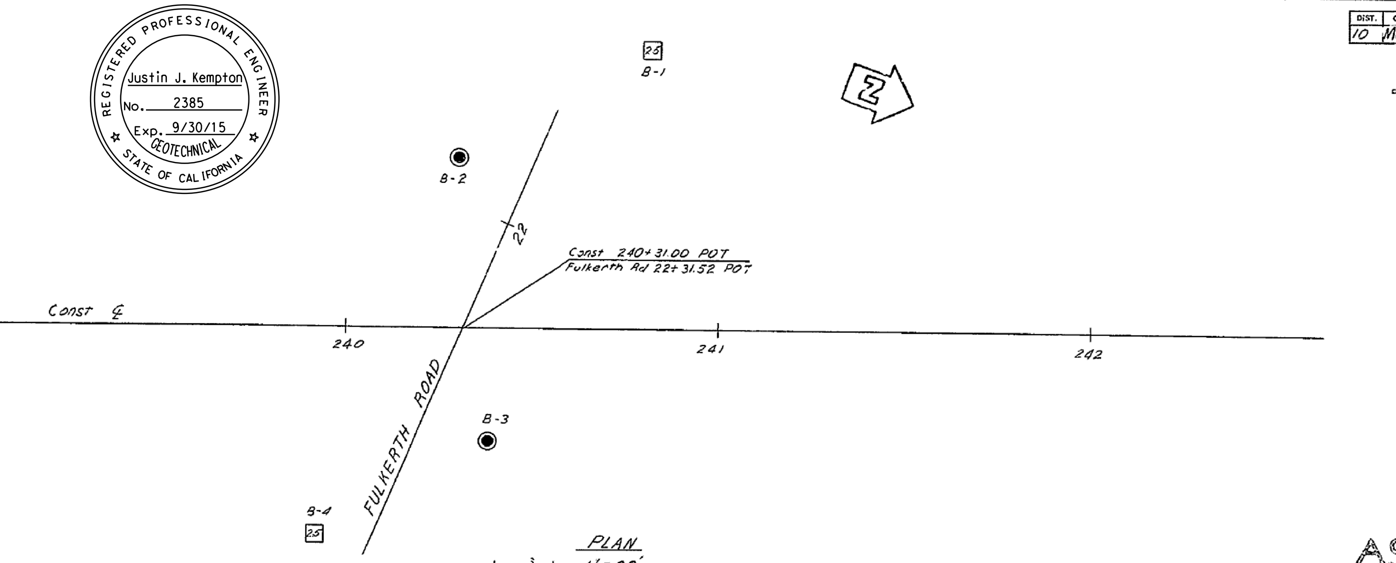
UNIT: X
PROJECT NUMBER & PHASE: X

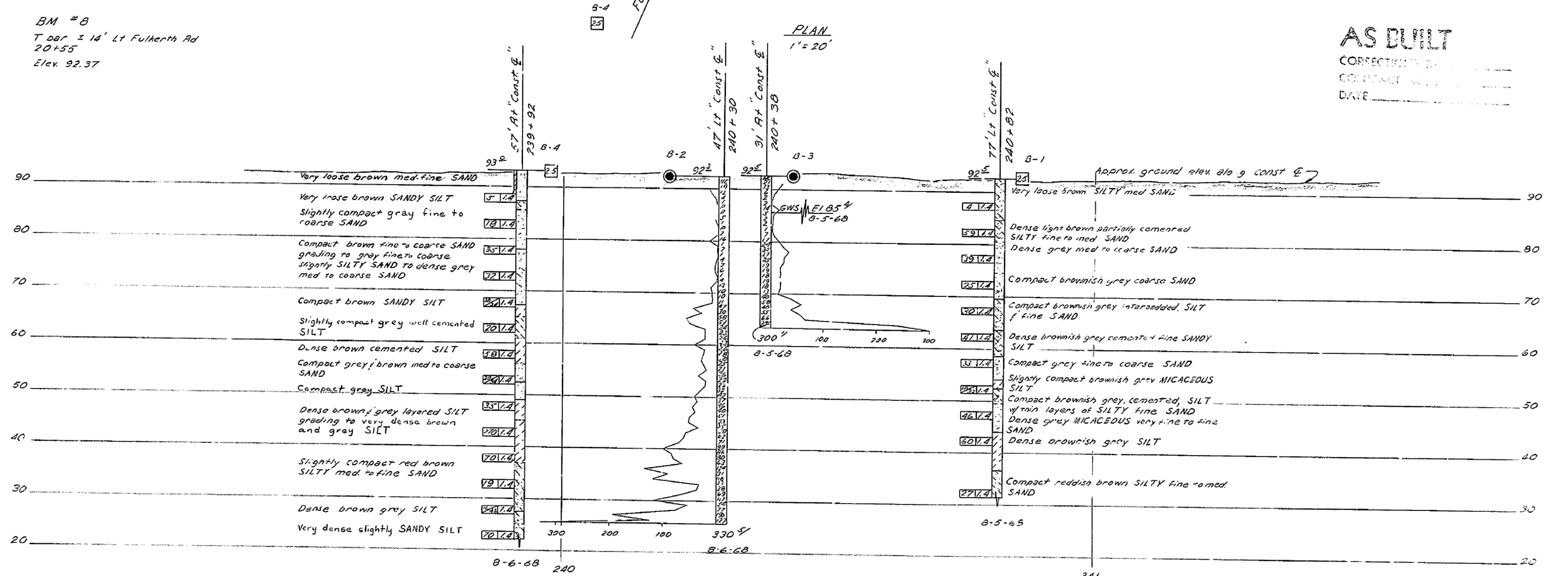
Revisions made to this Log of Test Borings from the original ____ Log of Test Borings are the addition of the following table and notes:

BRIDGE No.	Sheet	of
~~38-142 R/L~~	~~11~~	~~12~~ 239

DIST.	COUNTY	ROUTE	POST MILES—TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
10	Mer.Sta	99	R3.2 / R7.3	278	309

DATE APPROVED May 17 1971





AS BUILT
CORRECTIONS BY
CONTRACT NO.
DATE

AS BUILT PLANS
Contract No. 10-030104
Date Completed 05-71
Document No. 00001989

LEGEND OF BORING OPERATIONS

PENETROMETER
2½" CONE PENETROMETER
SAMPLER BORING (DRY)
ROTARY BORING (WET)
AUGER BORING (DRY)
JET BORING
CORE BORING
TEST PIT

1" SOIL TUBE
ROTARY BORING
PENETRATION BORING

LEGEND OF EARTH MATERIALS

GRAVEL
SAND
SILT
CLAY
SANDY CLAY OR CLAYEY SAND
SANDY SILT OR SILTY SAND
SILTY CLAY OR CLAYEY SILT
PEAT AND/OR ORGANIC MATTER
FILL MATERIAL
IGNEOUS ROCK
SEDIMENTARY ROCK
METAMORPHIC ROCK

CLASSIFICATION OF MATERIAL BASED ON STANDARD GRADE SIZE LIMITS

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

FIELD STUDY BY W. WILHELMS
DRAWN BY R. WILHELMS 10-8-68
CHECKED BY R. WILHELMS

278

BRIDGE DEPARTMENT
ENGINEERING GEOLOGY SECTION

STATE OF CALIFORNIA
TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

FULKERTH ROAD UNDERCROSSING

LOG OF TEST BORINGS

BRIDGE NO.	POST MILE	DRAWING NO.	SHEET	OF
38-142 R/L			9	9

WO 030101
CU 10203
PR38142-2

Disregard prints bearing earlier revision dates →

REVISION DATES (PRELIMINARY STAGE ONLY): 9/10/68, 4/2/70

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF TRANSPORTATION.

DATE 8-26-74 SIGNATURE TITLE

SCALE 1" = 50' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1" = 10' or 1" = 100'
SCALE 1" = 20'
SCALE 1" = 40'

KLEINFELDER
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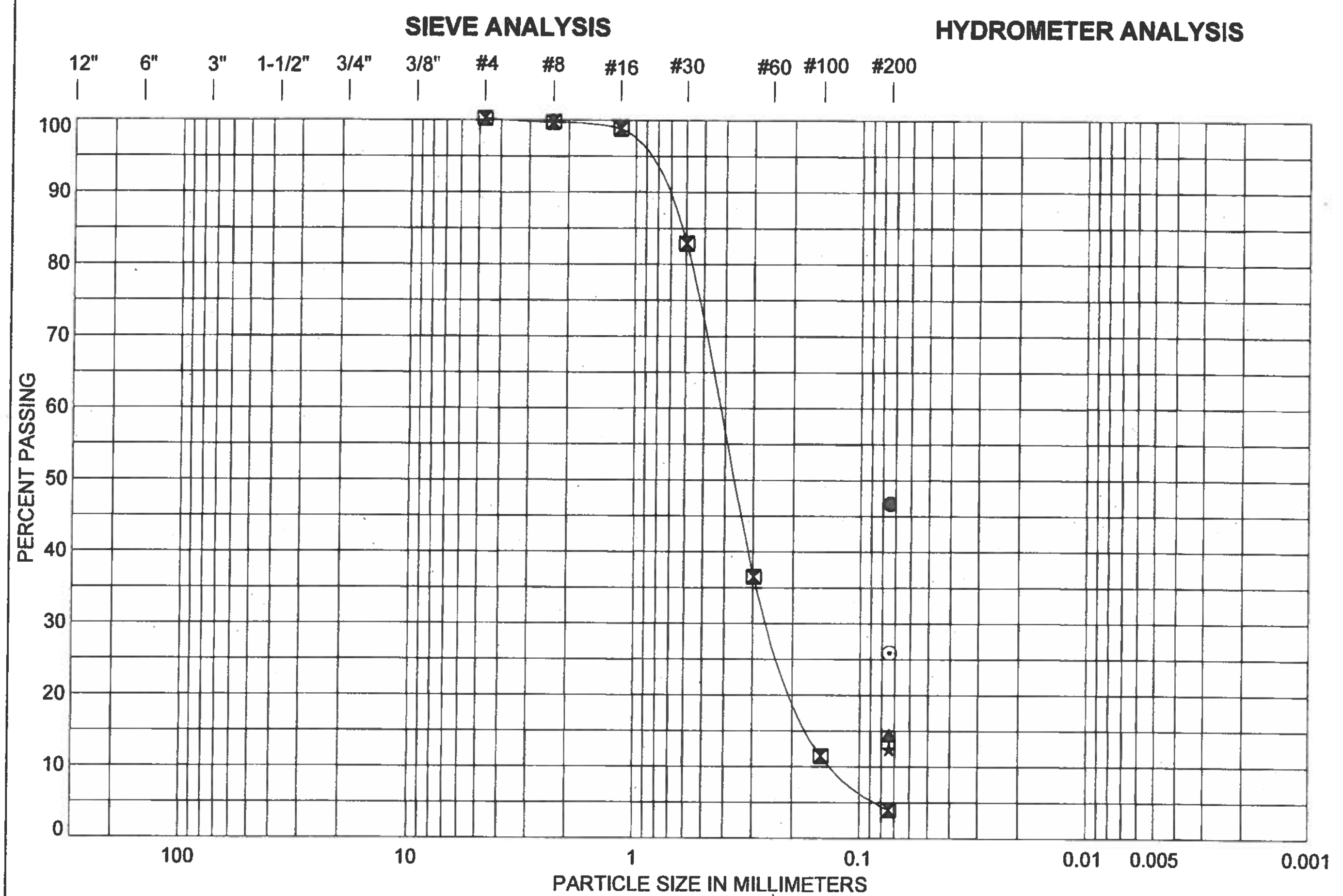
APPENDIX B



COBBLE	GRAVEL		SAND			SILT	CLAY
	coarse	fine	coarse	medium	fine		

LEGEND:	SOURCE	DEPTH (ft)	DESCRIPTION
●	A-11-001	6.0	Silty SAND (SM)
⊠	A-11-001	17.5	Poorly Graded SAND (SP)
▲	A-11-001	27.5	Poorly Graded SAND with silt (SP-SM)
★	A-11-002	11.0	Poorly Graded SAND with silt (SP-SM)
⊙	A-11-002	23.5	Silty SAND (SM)

KLEINFELDER
Bright People. Right Solutions.

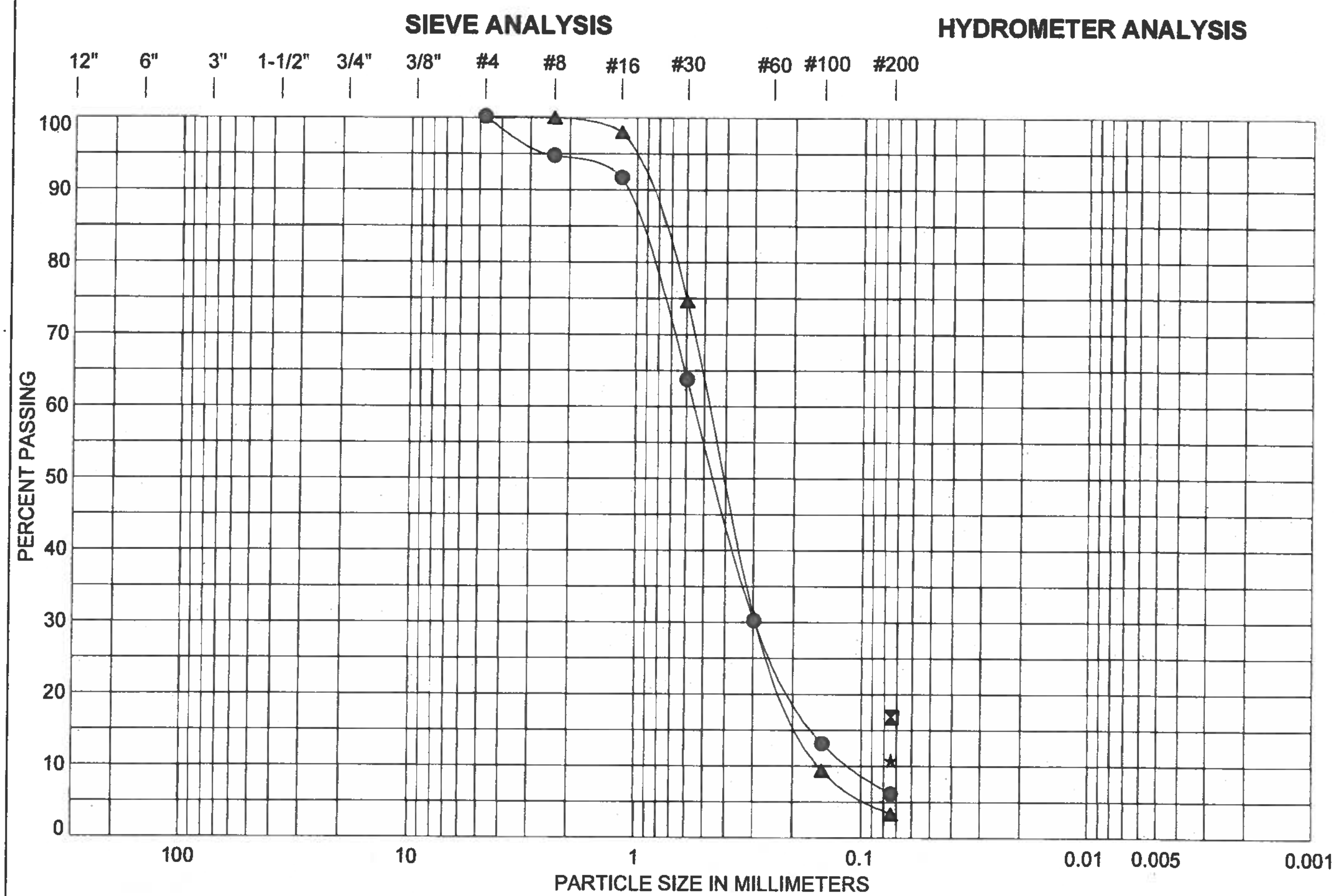
Drafted By:
Date: 5/11/2011

Project No.: 98834
File Number:

GRAIN SIZE ANALYSIS
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-1

KA_GRAIN_SIZE KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11



COBBLE	GRAVEL		SAND			SILT	CLAY
	coarse	fine	coarse	medium	fine		

LEGEND:	SOURCE	DEPTH (ft)	DESCRIPTION
●	A-11-002	25.0	Poorly Graded SAND with silt (SP-SM)
⊠	A-11-002	28.5	Silty SAND (SM)
▲	B-1	0.0	Poorly Graded SAND (SP)
★	B-3	10.0	Poorly Graded SAND with silt (SP-SM)

KA_GRAIN_SIZE KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11

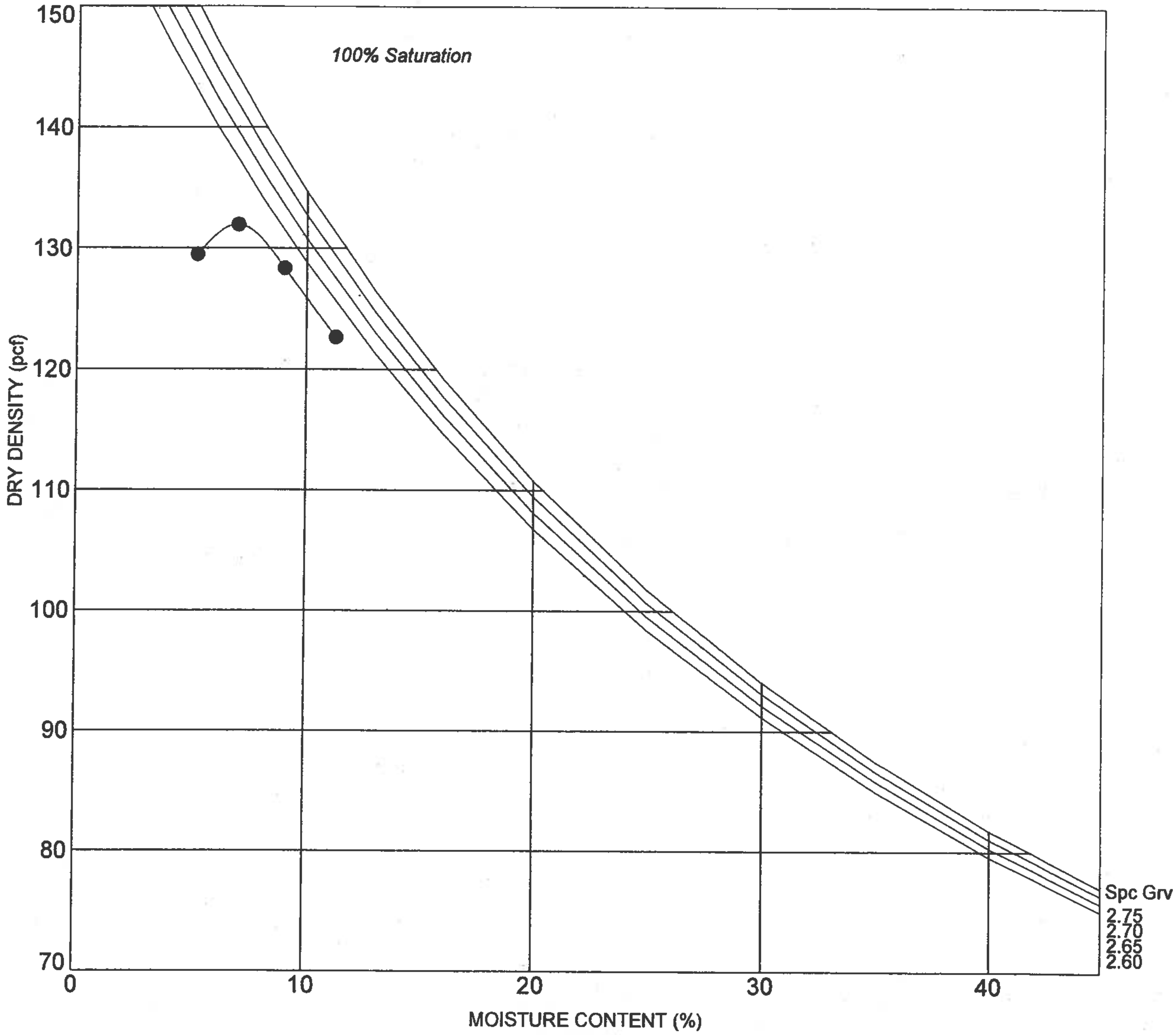
KLEINFELDER
Bright People. Right Solutions.

Drafted By:
Date: 5/11/2011

Project No.: 98834
File Number:

GRAIN SIZE ANALYSIS
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-2



LEGEND:	SOURCE	DEPTH (ft)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	TEST METHOD	DESCRIPTION
●	B-1	0.0 - 5.0	6.5	132.0	ASTM D1557 Method A	Poorly Graded SAND (SP)

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Date: 5/11/2011

Project No.: 98834
File Number:

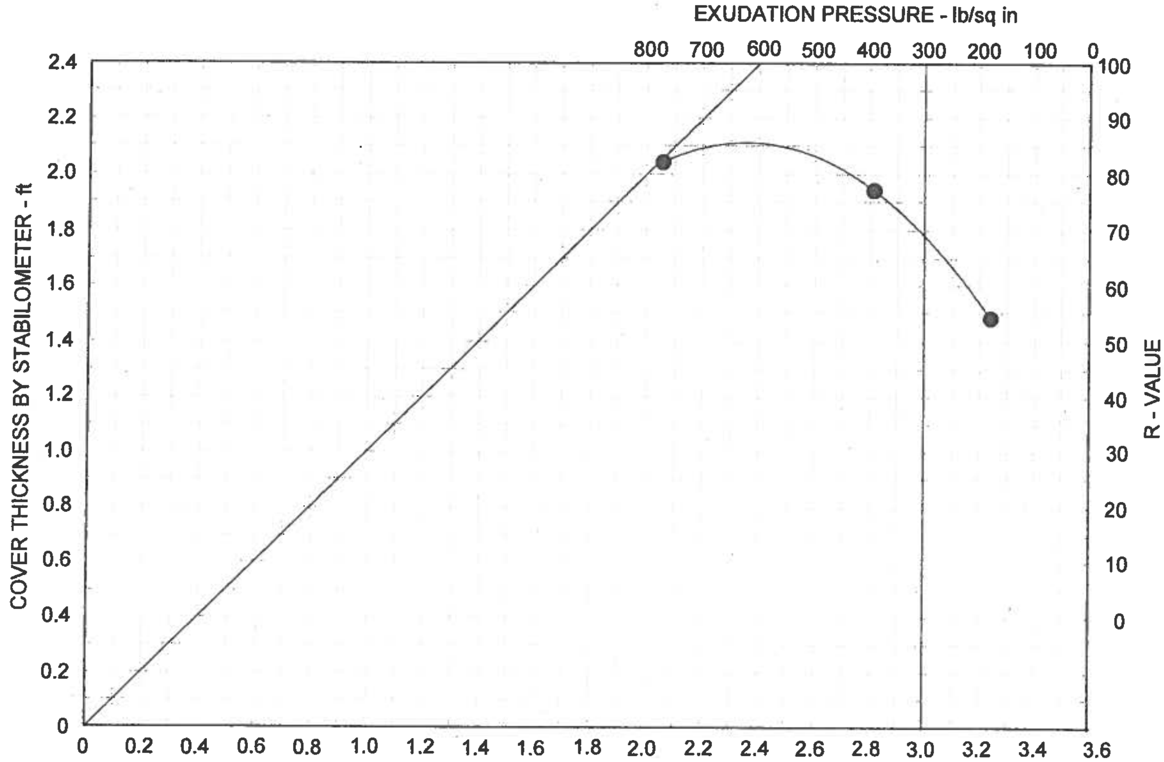
COMPACTION CURVE
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-3

KA-COMPACTION KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11

SAMPLE LOCATION: B-1 @ 0 - 5 feet

SAMPLE DESCRIPTION: Poorly Graded SAND (SP)



SPECIMEN	A	B	C
EXUDATION PRESSURE, lb/sq in	772	394	182
EXPANSION PRESSURE, lb/sq ft	0	0	0
RESISTANCE VALUE, R	82	77	54
MOISTURE AT TEST, %	7	8	10
DRY DENSITY AT TEST, lb/cu ft	129.4	128.2	126.6
R-VALUE AT 300 lb/sq in EXUDATION PRESSURE	69		

KA_R_VALUE KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11

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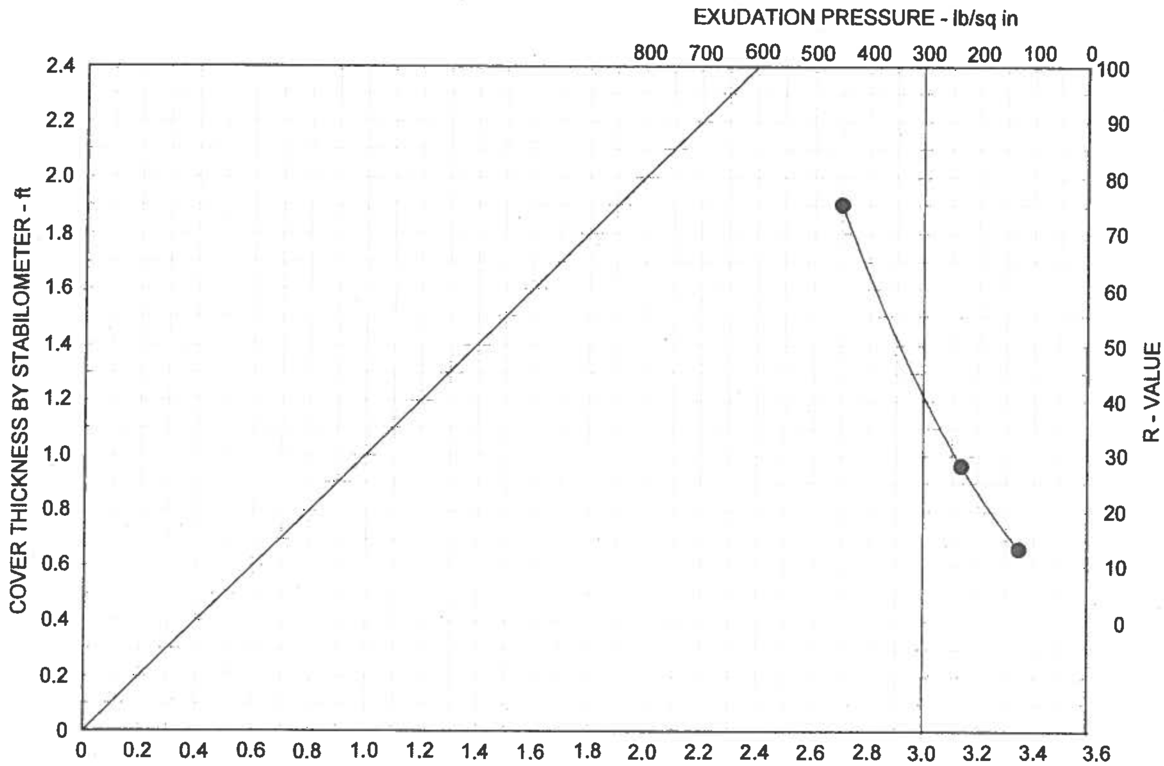
RESISTANCE VALUE
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-4

SAMPLE LOCATION: B-2 @ 0 - 5 feet

SAMPLE DESCRIPTION: Poorly Graded SAND (SP)



SPECIMEN	A	B	C
EXUDATION PRESSURE, lb/sq in	451	234	129
EXPANSION PRESSURE, lb/sq ft	0	0	0
RESISTANCE VALUE, R	75	28	13
MOISTURE AT TEST, %	9	11	12
DRY DENSITY AT TEST, lb/cu ft	126.7	122.8	120
R-VALUE AT 300 lb/sq in EXUDATION PRESSURE	**40**		

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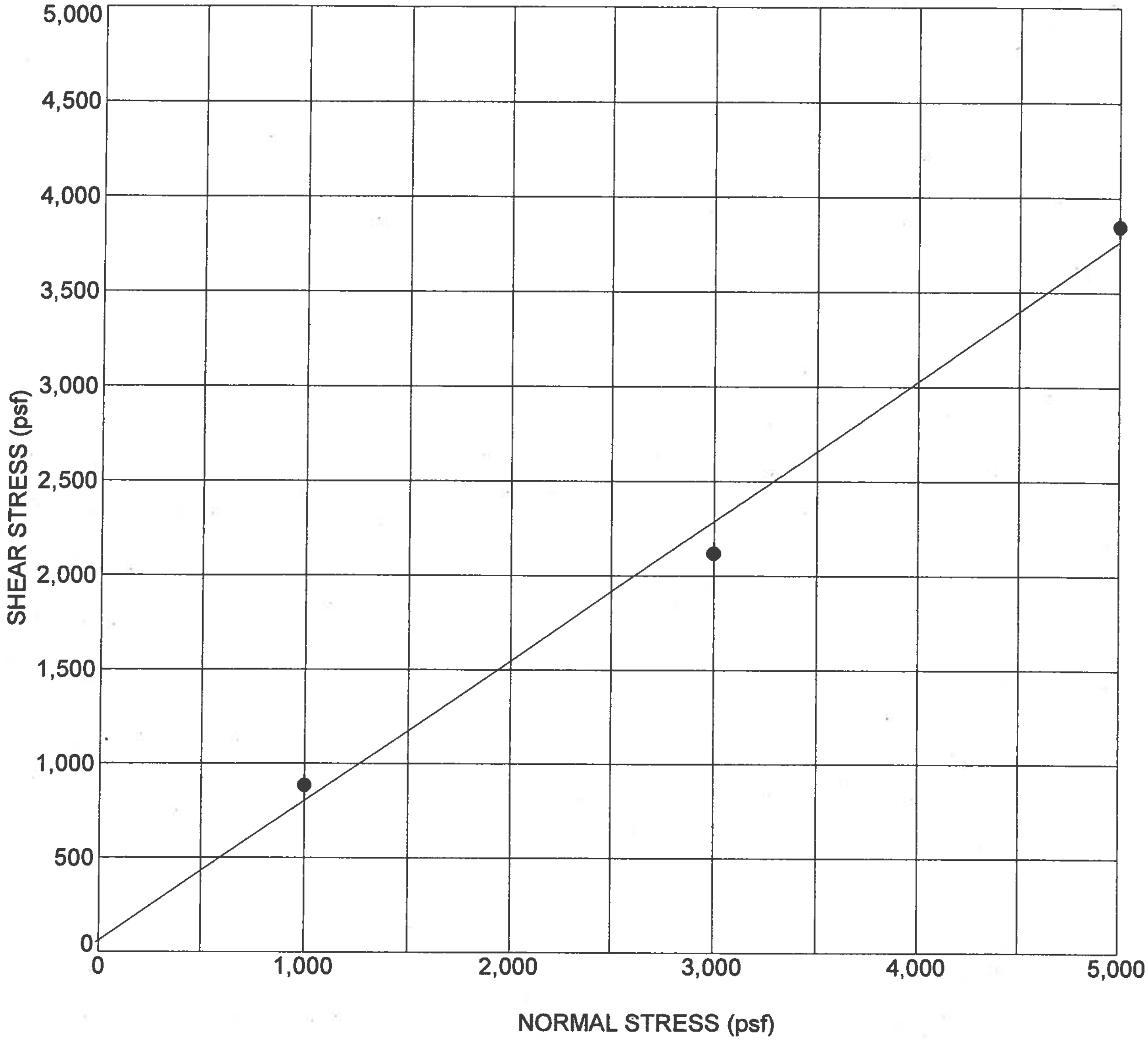
File Number:

RESISTANCE VALUE
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-5



SOURCE: A-11-001
DEPTH: 16 ft (PEAK STRENGTH)
SOIL DESCRIPTION: Poorly Graded SAND (SP)

FRICTION ANGLE = 37 deg
COHESION = 60 psf

FINAL DRY DENSITY (pcf)	116.6	116.5	108.0
INITIAL WATER CONTENT (%)	6.9	6.9	6.9
FINAL WATER CONTENT (%)	18.4	20.1	19.1
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	885	2119	3847

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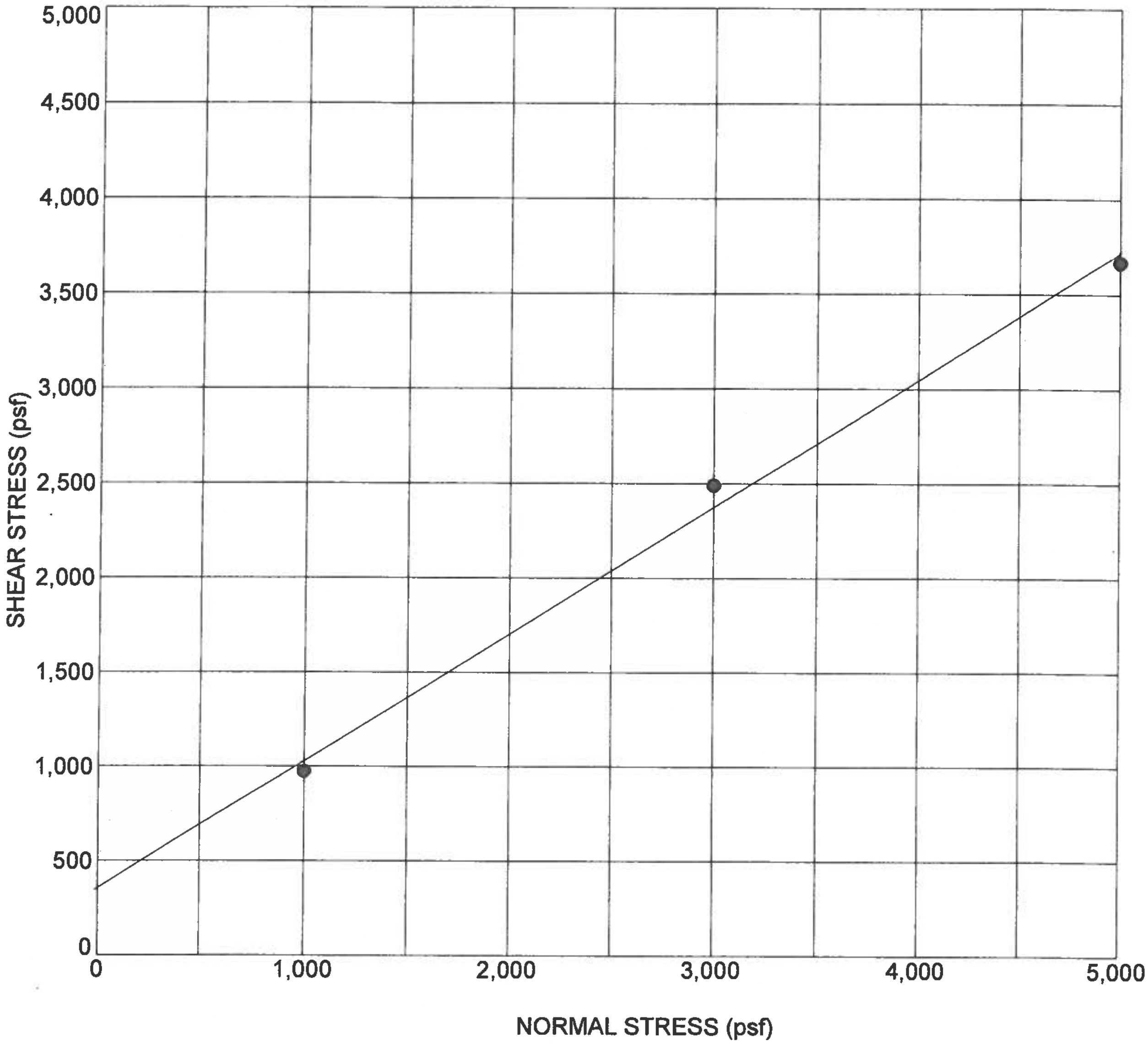
DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-6

KA_DIRECT_SHEAR KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11



SOURCE: A-11-001
DEPTH: 21 ft (PEAK STRENGTH)
SOIL DESCRIPTION: Poorly Graded SAND (SP)

FRICTION ANGLE = 34 deg
COHESION = 355 psf

FINAL DRY DENSITY (pcf)	114.3	114.5	114.5
INITIAL WATER CONTENT (%)	8.7	8.7	8.7
FINAL WATER CONTENT (%)	18.7	18.2	16.1
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	974	2490	3668

KA_DIRECT_SHEAR KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11

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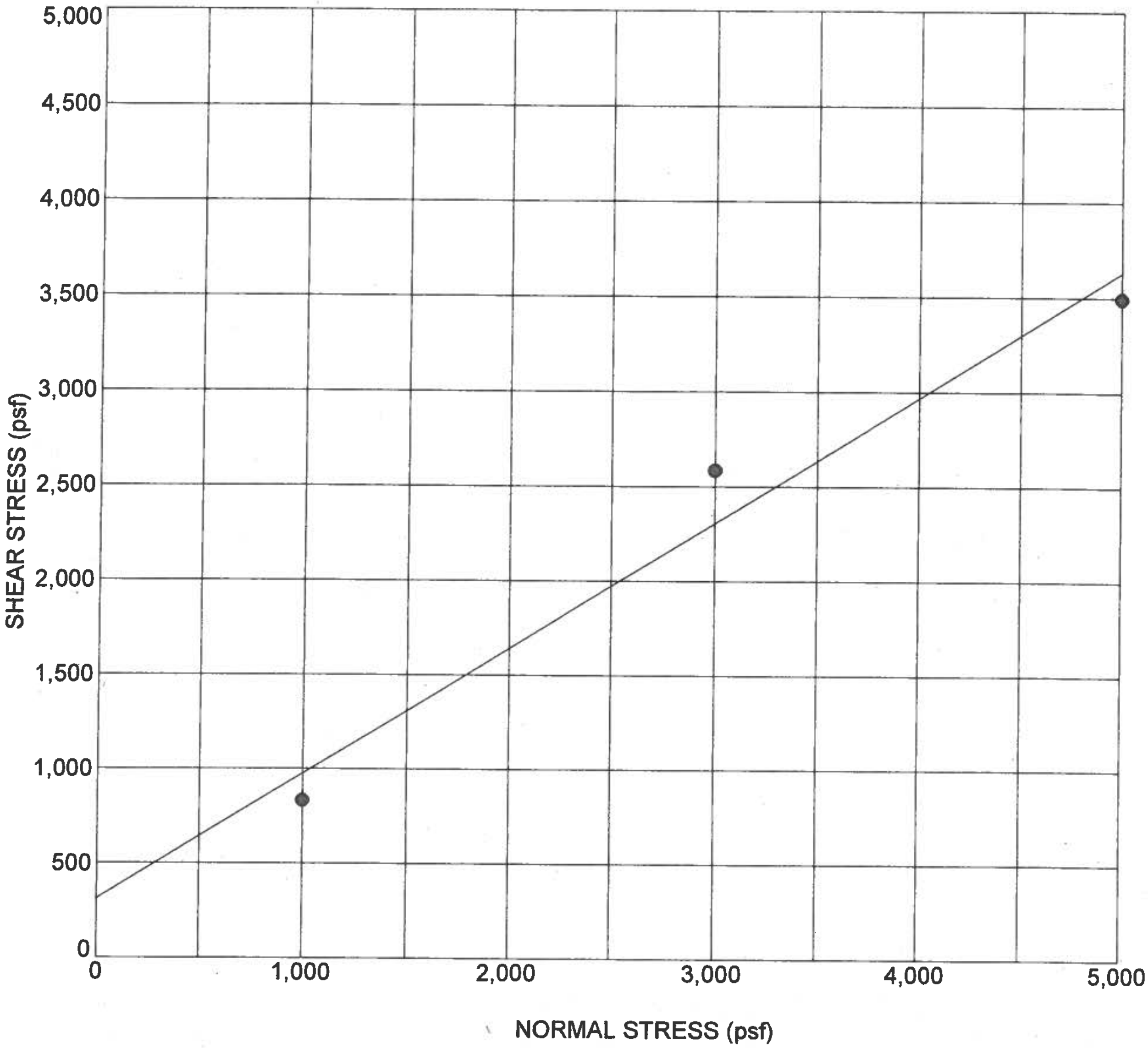
File Number:

DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-7



SOURCE: A-11-002
DEPTH: 23.5 ft (PEAK STRENGTH)
SOIL DESCRIPTION: Silty SAND (SM)

FRICTION ANGLE = 34 deg
COHESION = 310 psf

FINAL DRY DENSITY (pcf)	120.7	121.8	121.7
INITIAL WATER CONTENT (%)	8.6	8.6	8.6
FINAL WATER CONTENT (%)	13.8	12.8	11.9
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	833	2585	3494

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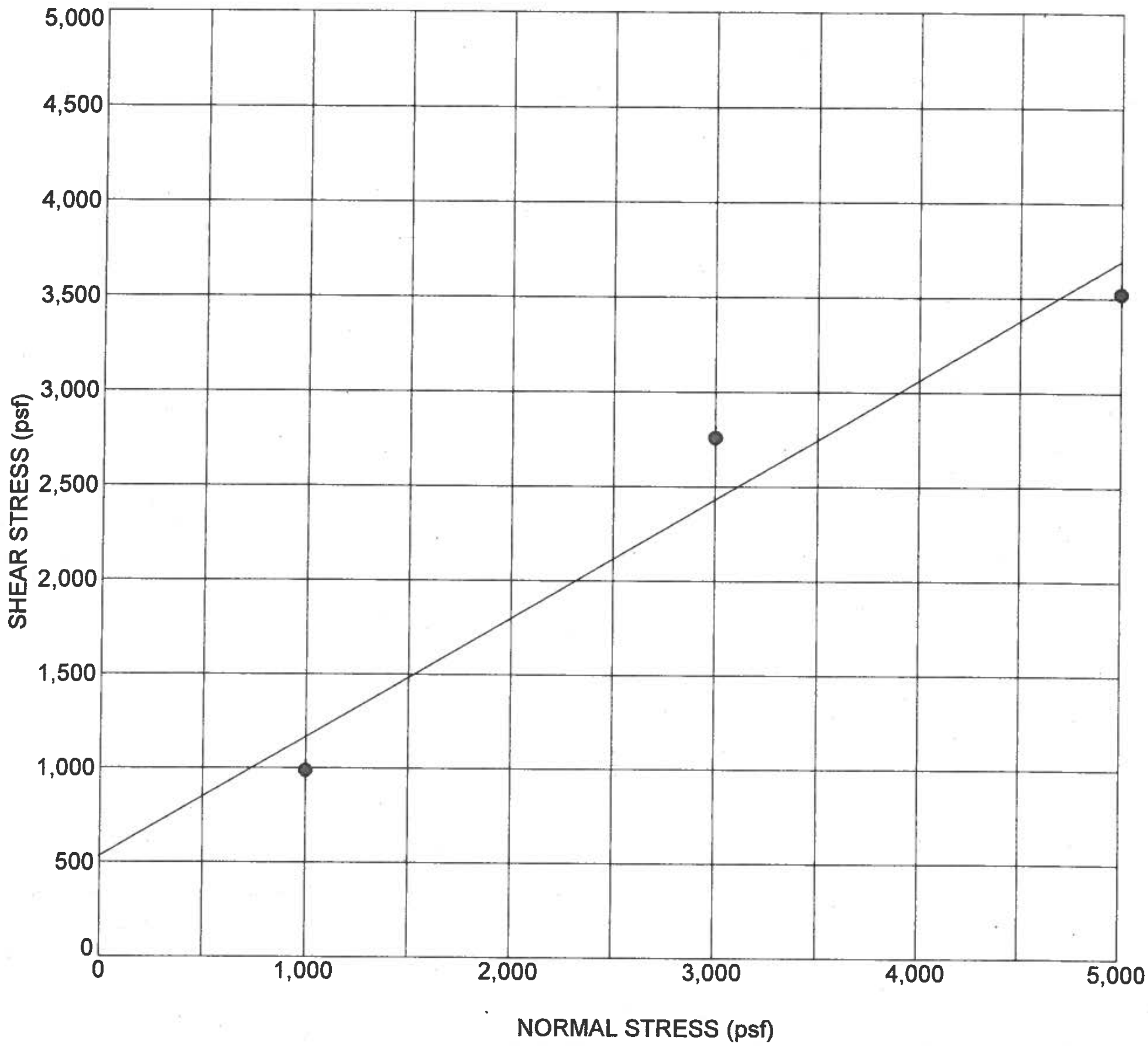
DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-8



SOURCE: B-1
DEPTH: 0 to 5 ft (REMOLDED PEAK STRENGTH)
SOIL DESCRIPTION: Poorly Graded SAND (SP)

FRICTION ANGLE = 32 deg
COHESION = 520 psf

FINAL DRY DENSITY (pcf)	118.8	118.8	118.7
INITIAL WATER CONTENT (%)	6.5	6.5	6.5
FINAL WATER CONTENT (%)	13.4	13.1	13.8
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	988	2759	3523

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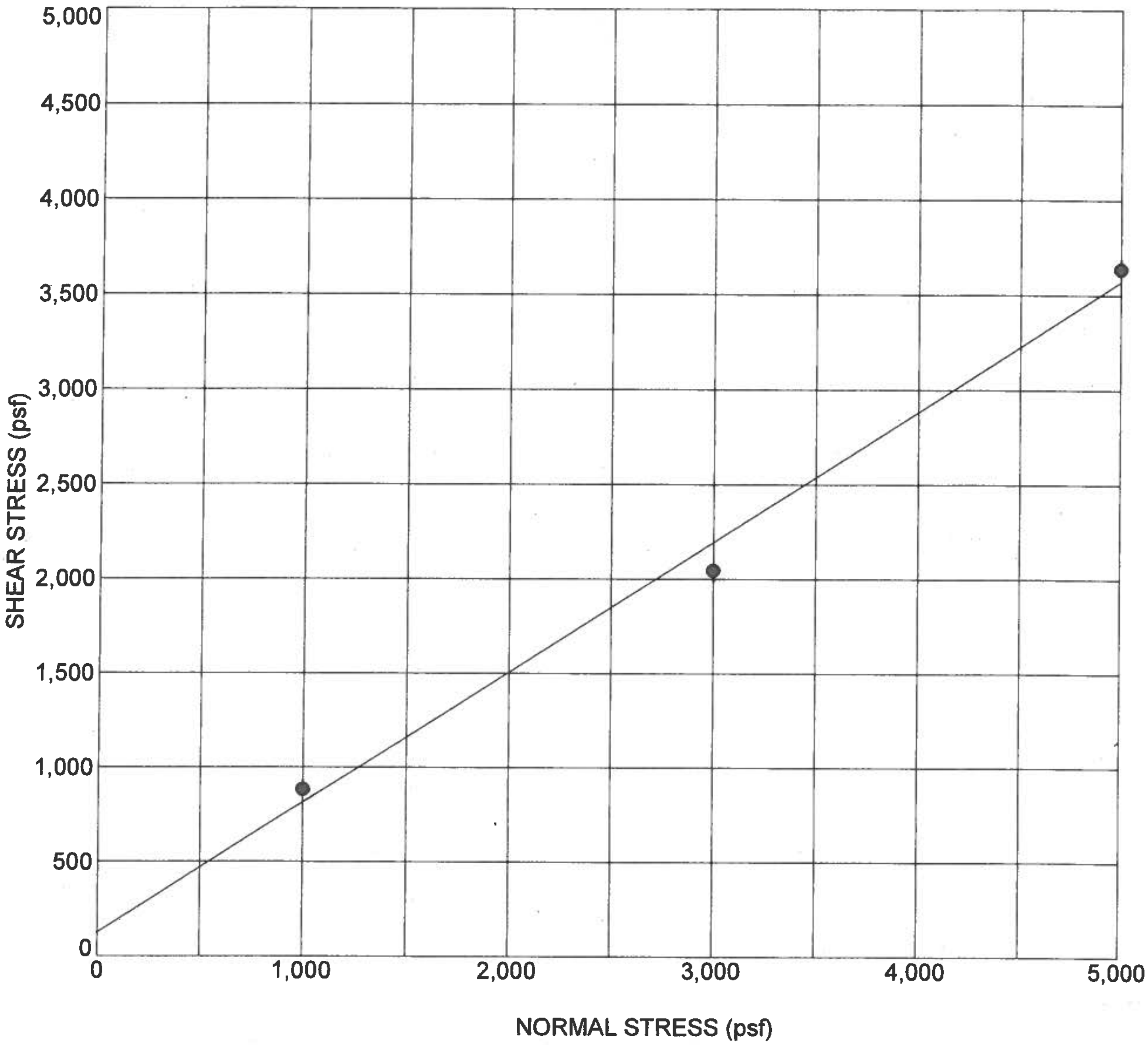
DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-9



SOURCE: A-11-001
DEPTH: 16 ft (ULTIMATE STRENGTH)
SOIL DESCRIPTION: Poorly Graded SAND (SP)

FRICTION ANGLE = 35 deg
COHESION = 125 psf

FINAL DRY DENSITY (pcf)	116.6	116.5	108.0
INITIAL WATER CONTENT (%)	6.9	6.9	6.9
FINAL WATER CONTENT (%)	18.4	20.1	19.1
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	883.6	2044.8	3637.7

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DIRECT SHEAR TEST
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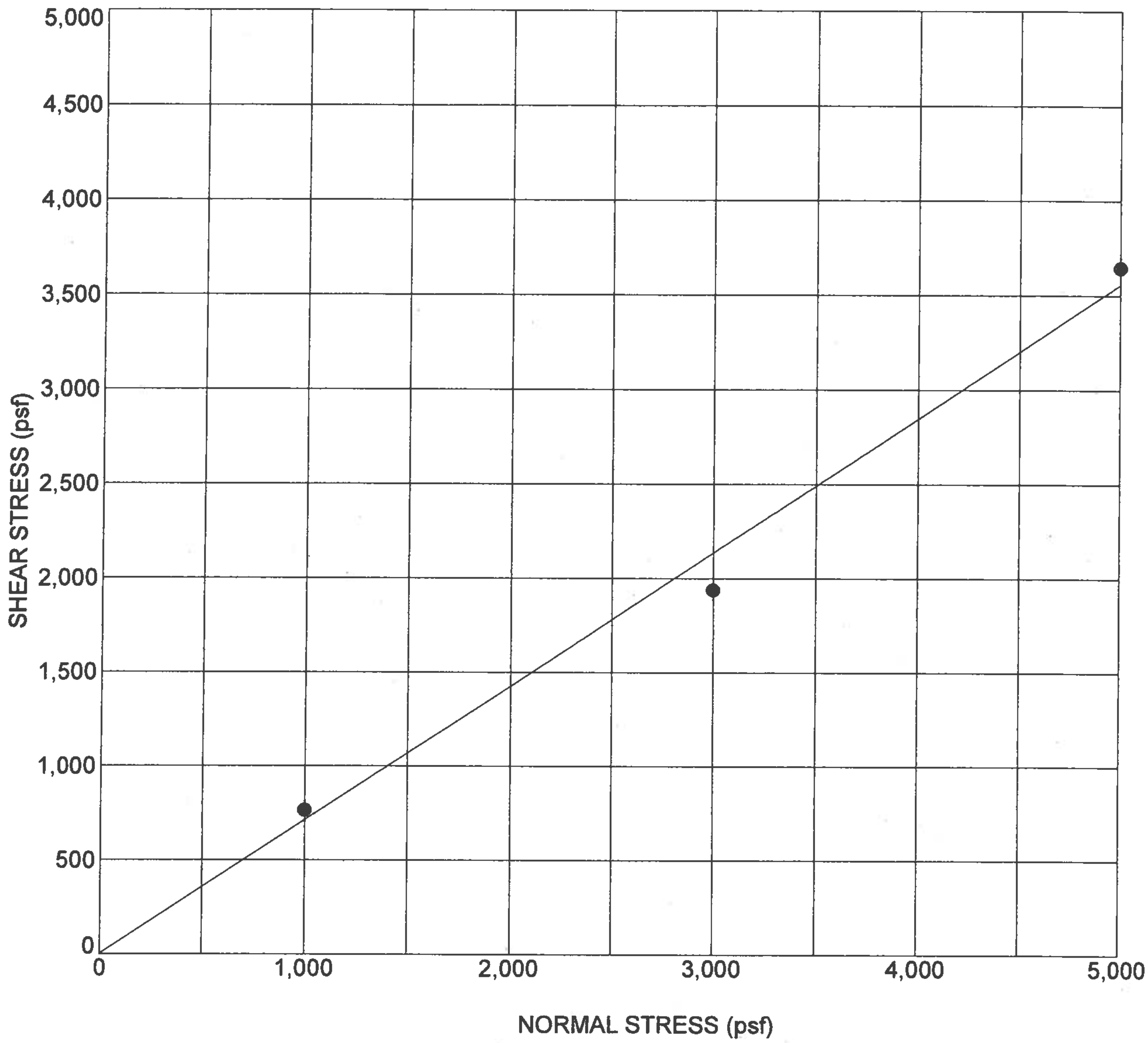
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-10

Drafted By:
Date: 5/11/2011
Project No.: 98834
File Number:



SOURCE: A-11-001
DEPTH: 21 ft (ULTIMATE STRENGTH)
SOIL DESCRIPTION: Poorly Graded SAND (SP)

FRICTION ANGLE = 35 deg
COHESION = 0 psf

FINAL DRY DENSITY (pcf)	114.3	114.5	114.5
INITIAL WATER CONTENT (%)	8.7	8.7	8.7
FINAL WATER CONTENT (%)	18.7	18.2	16.1
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	765.3	1938.1	3646.1

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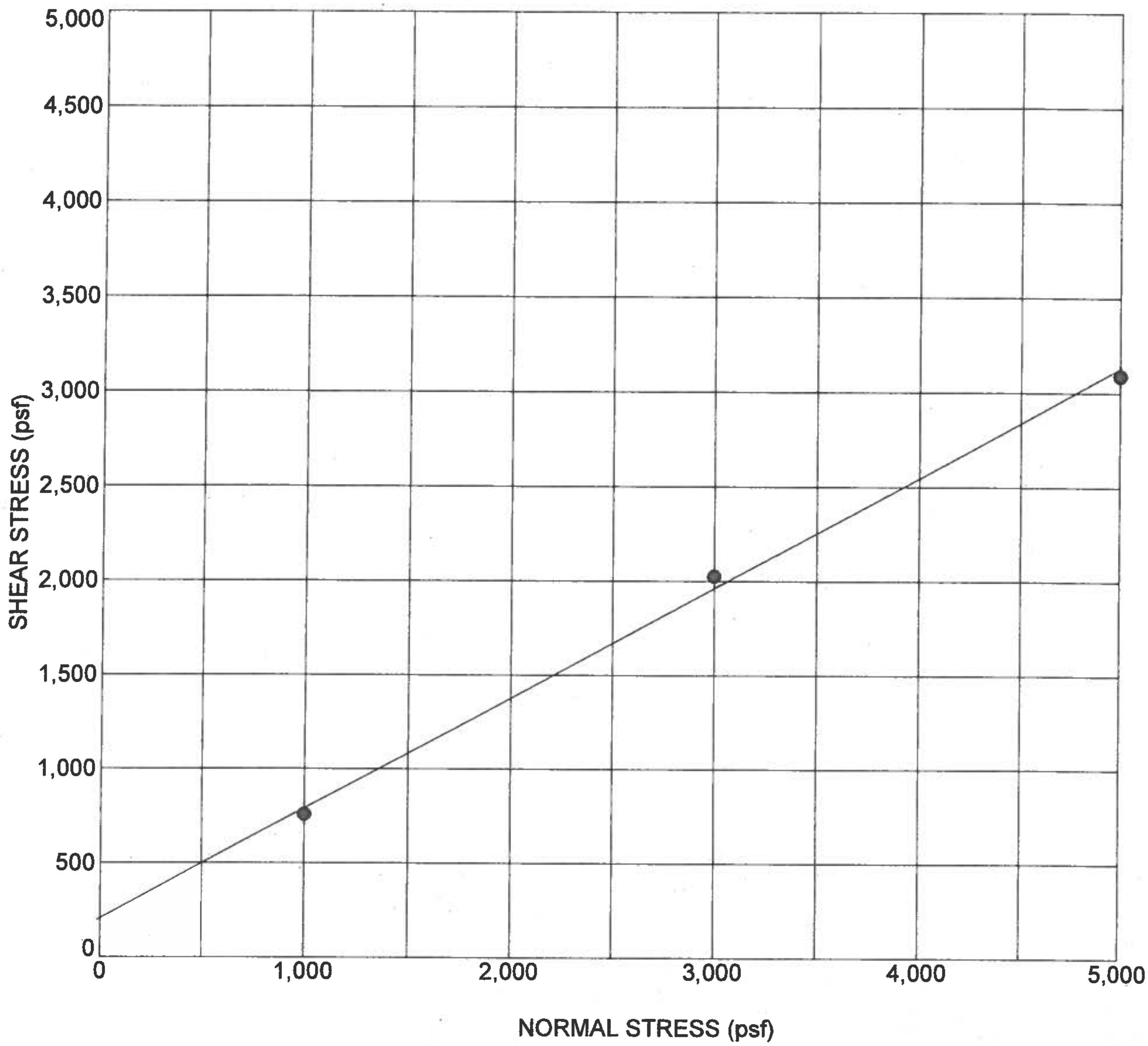
DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-11

KA_DIRECT_SHEAR KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11



SOURCE: A-11-002
DEPTH: 23.5 ft (ULTIMATE STRENGTH)
SOIL DESCRIPTION: Silty SAND (SM)

FRICTION ANGLE = 30 deg
COHESION = 210 psf

FINAL DRY DENSITY (pcf)	120.7	121.8	121.7
INITIAL WATER CONTENT (%)	8.6	8.6	8.6
FINAL WATER CONTENT (%)	13.8	12.8	11.9
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	758.7	2026	3089.9

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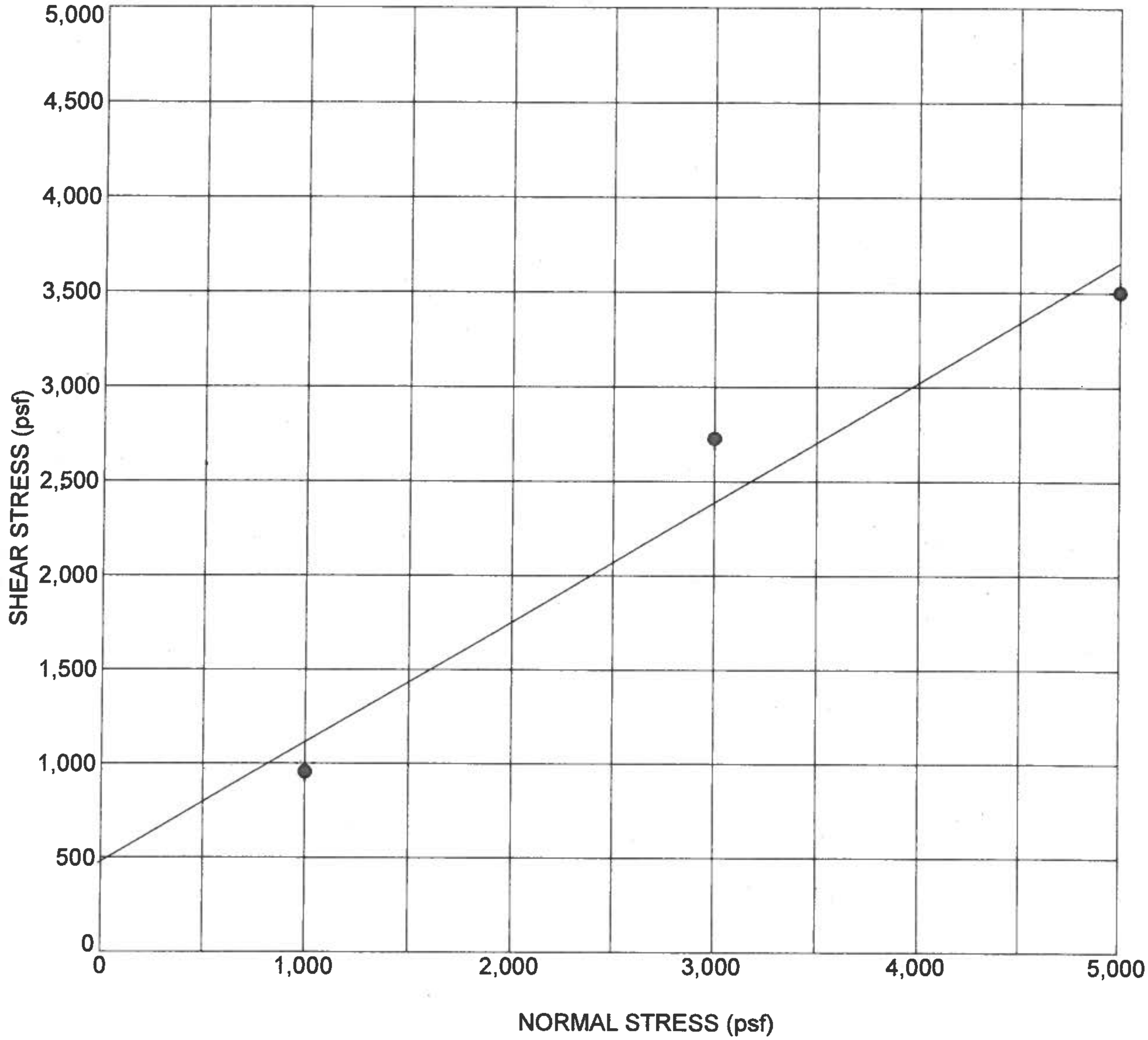
Date: 5/11/2011
Project No.: 98834
File Number:

DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
SR99 & FULKERTH RD INTERCHANGE
TURLOCK, CALIFORNIA

PLATE
B-12



SOURCE: B-1
DEPTH: 0 to 5 ft (REMOLDED ULTIMATE STRENGTH)
SOIL DESCRIPTION: Poorly Graded SAND (SP)

FRICTION ANGLE = 32 deg
COHESION = 485 psf

FINAL DRY DENSITY (pcf)	118.8	118.8	118.7
INITIAL WATER CONTENT (%)	6.5	6.5	6.5
FINAL WATER CONTENT (%)	13.4	13.1	13.8
NORMAL STRESS (psf)	1000	3000	5000
MAXIMUM SHEAR (psf)	955.6	2726.5	3501.8

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File Number:

DIRECT SHEAR TEST
GEOTECHNICAL DESIGN REPORT
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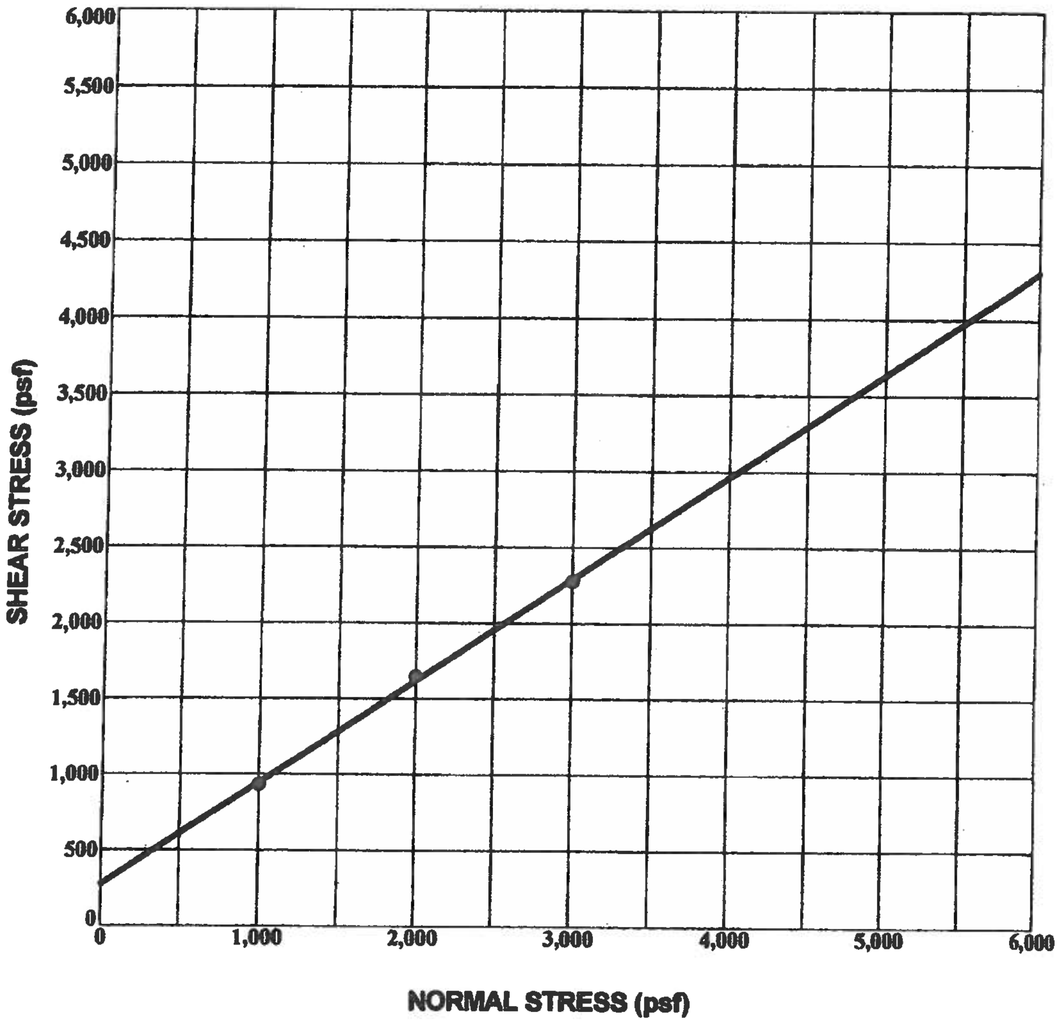
TURLOCK, CALIFORNIA

PLATE
B-13

KA_DIRECT_SHEAR KA CORPORATE STD.GDT KLEINFELDER_GINT_LIBRARY_R2B.GLB 98834.GPJ 5/11/11

APPENDIX C

DIRECT SHEAR



Source: TP-1
Depth: 1.0 ft
Test Type: Consolidated - Drained
Soil Description: Silty Sand (SM)

Friction Angle = 34 deg
Cohesion = 275 psf

Dry Density (pcf)	108.9	109.0	109.8
Initial Water Content (%)	9.0	9.0	9.0
Final Water Content (%)	19.1	18.8	18.0
Normal Stress (psf)	1000	2000	3000
Shear Stress(psf)	932	1648	2276

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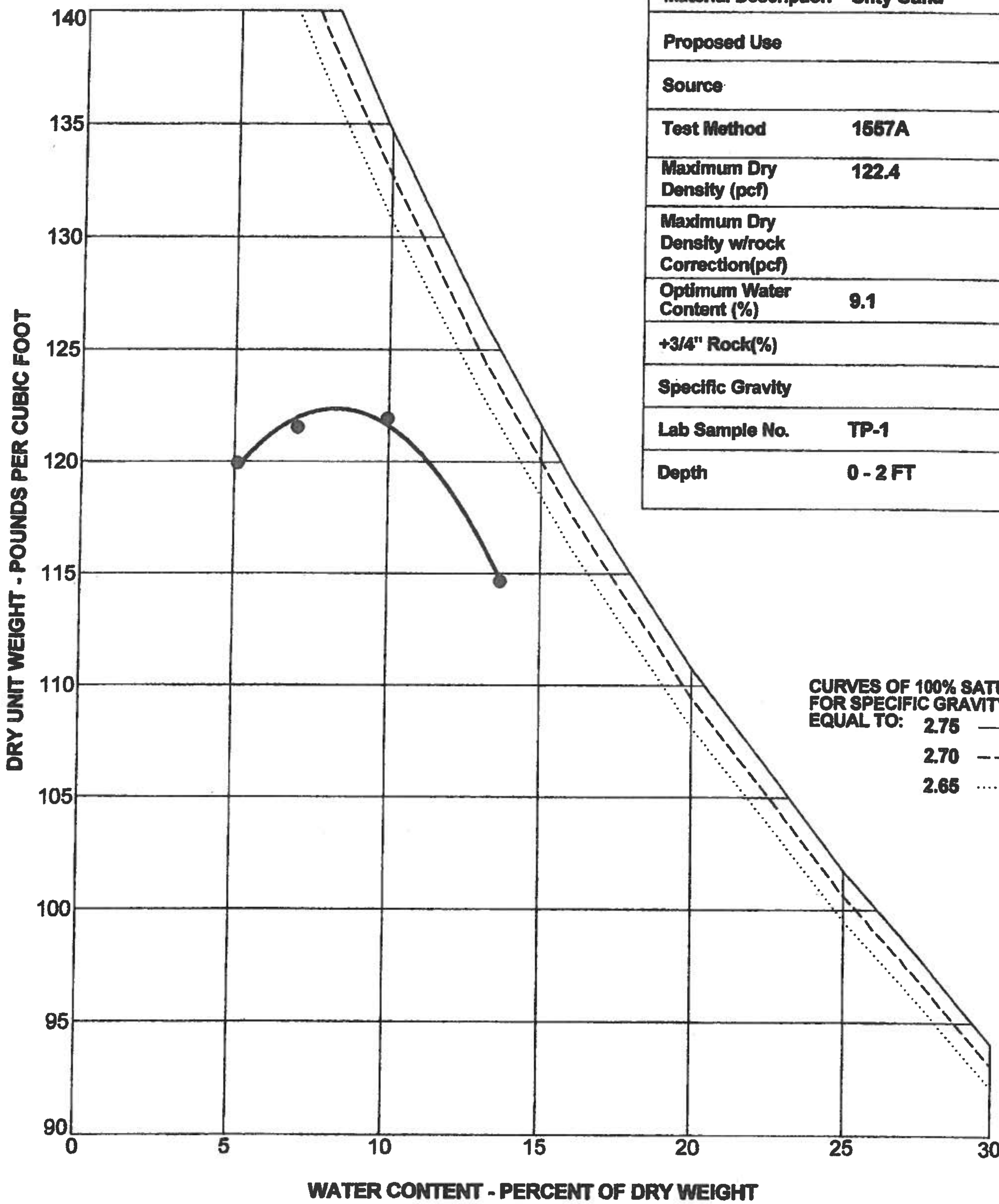
PROJECT NO. 98834

DIRECT SHEAR TEST
SR99/Fulkerth Road Interchange Project
Fulkerth Road & SR99
TURLOCK, CA

PLATE
A-1



SUMMARY OF TEST RESULTS	
Material Description	Silty Sand
Proposed Use	
Source	
Test Method	1557A
Maximum Dry Density (pcf)	122.4
Maximum Dry Density w/rock Correction(pcf)	
Optimum Water Content (%)	9.1
+3/4" Rock(%)	
Specific Gravity	
Lab Sample No.	TP-1
Depth	0 - 2 FT

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PROJECT NO. 98834

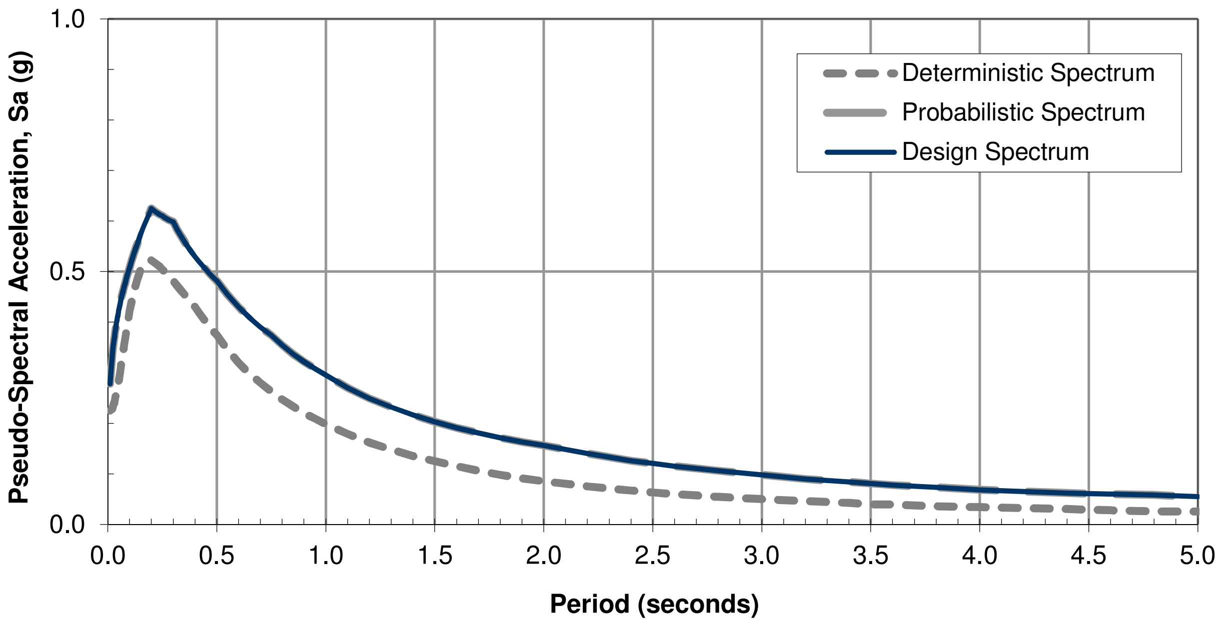
MOISTURE DENSITY RELATIONSHIP
SR99/Fulkerth Road Interchange Project
Fulkerth Road & SR99
TURLOCK, CA

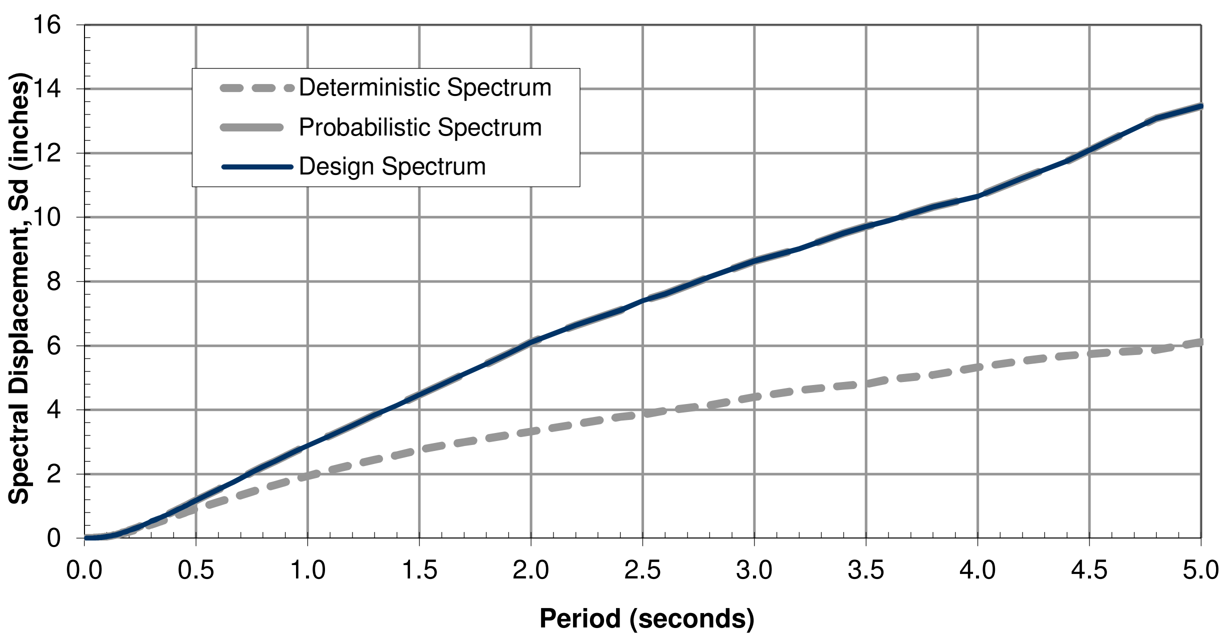
PLATE A-2

APPENDIX D

SITE DATA

Latitude (degrees):	37.5072	Shear Wave Velocity, V_{s30}:	361 m/s
Longitude (degrees):	-120.8778	Depth to Vs = 1.0 km/s, $Z_{1.0}$:	263 m
		Depth to Vs = 2.5 km/s, $Z_{2.5}$:	2 km





KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO 98834	**2009 CALTRANS SDC** **ARS CURVES**	**FIGURE**
	DRAWN: 5/2/11	GEOTECHNICAL DESIGN REPORT SR99 AND FULKERTH ROAD INTERCHANGE TURLOCK, CALIFORNIA	**D-1**
	DRAWN BY: MB		
	CHECKED BY:		
	FILE NAME: Design ARS.xls		

DESIGN ARS CURVE ORDINATES

Period (s)	Sa (g)	Sd (inches)	Period (s)	Sa (g)	Sd (inches)
0.010	0.278	0.000	0.360	0.554	0.703
0.020	0.333	0.001	0.380	0.541	0.765
0.022	0.341	0.002	0.400	0.529	0.828
0.025	0.353	0.002	0.420	0.519	0.896
0.029	0.367	0.003	0.440	0.509	0.964
0.030	0.370	0.003	0.450	0.504	0.999
0.032	0.376	0.004	0.460	0.499	1.033
0.035	0.385	0.005	0.480	0.490	1.105
0.036	0.388	0.005	0.500	0.482	1.179
0.040	0.399	0.006	0.550	0.454	1.344
0.042	0.404	0.007	0.600	0.430	1.515
0.044	0.409	0.008	0.650	0.409	1.691
0.045	0.411	0.008	0.667	0.402	1.750
0.046	0.414	0.009	0.700	0.390	1.870
0.048	0.418	0.009	0.750	0.374	2.059
0.050	0.423	0.010	0.800	0.355	2.224
0.055	0.434	0.013	0.850	0.337	2.383
0.060	0.444	0.016	0.900	0.322	2.553
0.065	0.453	0.019	0.950	0.308	2.721
0.067	0.457	0.020	1.000	0.295	2.887
0.070	0.462	0.022	1.100	0.270	3.198
0.075	0.470	0.026	1.200	0.249	3.509
0.080	0.478	0.030	1.300	0.232	3.838
0.085	0.486	0.034	1.400	0.216	4.144
0.090	0.493	0.039	1.500	0.203	4.470
0.095	0.500	0.044	1.600	0.191	4.786
0.100	0.507	0.050	1.700	0.181	5.120
0.110	0.522	0.062	1.800	0.171	5.423
0.120	0.536	0.076	1.900	0.163	5.759
0.130	0.549	0.091	2.000	0.156	6.107
0.133	0.552	0.096	2.200	0.140	6.632
0.140	0.561	0.108	2.400	0.126	7.103
0.150	0.573	0.126	2.500	0.121	7.402
0.160	0.584	0.146	2.600	0.115	7.609
0.170	0.595	0.168	2.800	0.106	8.134
0.180	0.605	0.192	3.000	0.098	8.633
0.190	0.615	0.217	3.200	0.090	9.020
0.200	0.625	0.245	3.400	0.084	9.504
0.220	0.618	0.293	3.500	0.081	9.712
0.240	0.612	0.345	3.600	0.078	9.894
0.250	0.610	0.373	3.800	0.073	10.317
0.260	0.607	0.402	4.000	0.068	10.649
0.280	0.602	0.462	4.200	0.065	11.222
0.290	0.600	0.494	4.400	0.062	11.748
0.300	0.598	0.527	4.600	0.060	12.426
0.320	0.582	0.583	4.800	0.058	13.079
0.340	0.567	0.642	5.000	0.055	13.458

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PROJECT NO	98834
DRAWN:	5/2/11
DRAWN BY:	MB
CHECKED BY:	
FILE NAME:	Design ARS.xls

2009 CALTRANS SDC
ARS CURVES

GEOTECHNICAL DESIGN REPORT
SR99 AND FULKERTH ROAD INTERCHANGE
TURLOCK, CALIFORNIA

FIGURE
D-2

APPENDIX E

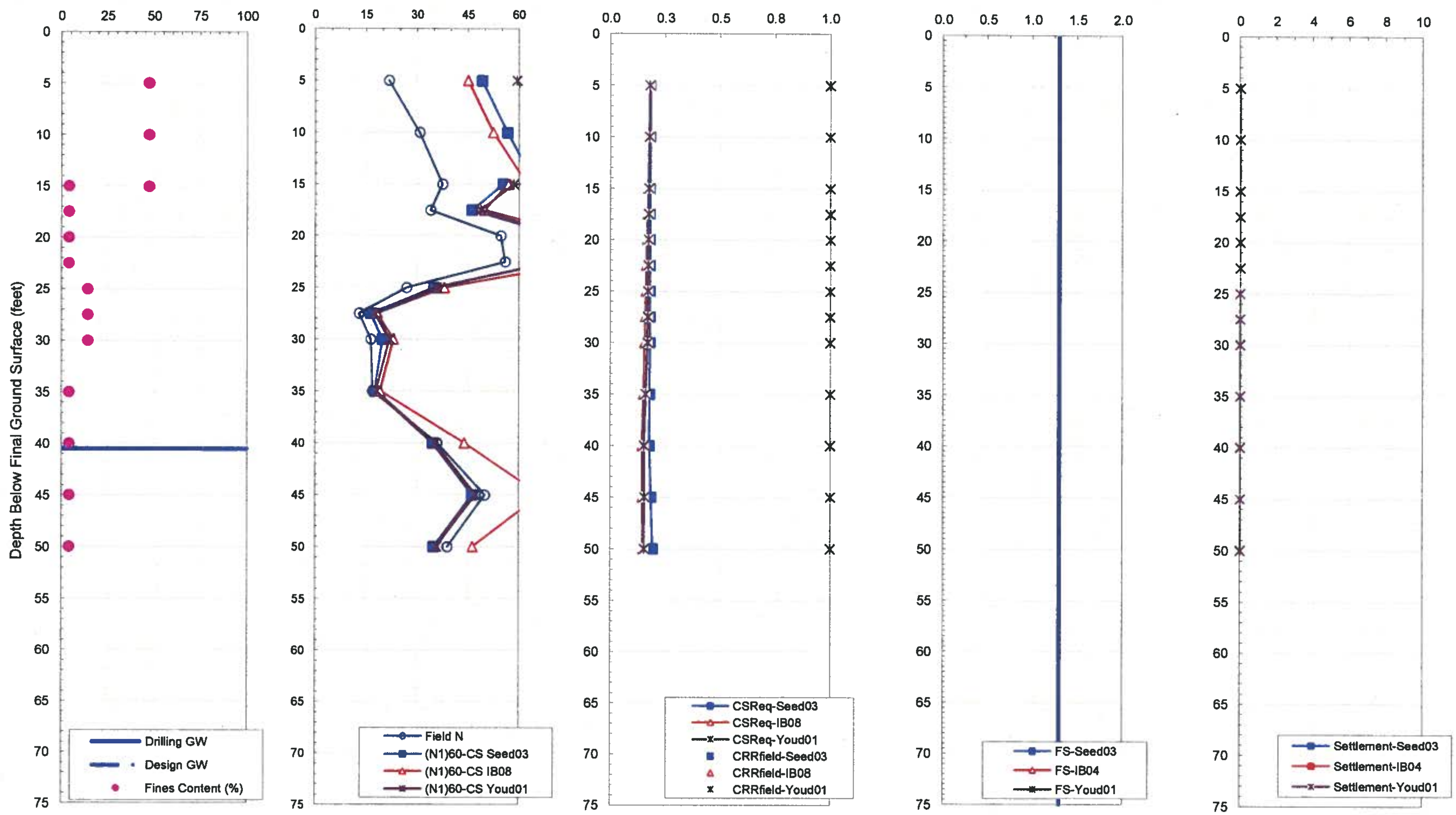
Boring ID: **A-11-001**

M_w= 6.7
PGA= 0.28g

Groundwater Depth During Drilling (ft) = **40.5 ft**
Design Groundwater Depth (ft) = **40.5 ft**

Existing Ground Elevation = **0.0 ft**
Final Ground Elevation = **0.0 ft**

Ana. by: M. BELTRAN
Checked by: J. KEMPTON



1. $(N_1)_{60\text{-}CS}$ capped at 60; 2. CRRfield = 1 for non-liquefiable soils; 3. FS is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils including soils above G.W.T.

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Merced, California 95341
t: 209 384 7552 f: 209 384 8218

Project Name: SR 99 & Fulkerth Ave.
Project No.: 98834
Project Location: TURLOCK, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

PLATE E-1

Lateral Surcharge Pressure due to Strip Footing Surcharge per

References:

Project Name:	Fulkerth Retaining Wall
Project Number	98834 / GEO2

Calculation

				Init.
Program By:	J Kempton	Date:	5/10/2011	
Checked By		Date:		NA

Assumptions

1 Uniform soil profile

2 No Hydrostatic pressure

Input Parameters

Retained Earth Conditions

		Symbol	Units	Value			
1	***Friction Angle***			32			
2	Height of Wall (pressure surface)	h	ft	10.5			
3	Strip Footing pressure	*p*	psf	3000			
4	Width of Strip Footing	B	ft	8			
5	Distance from pressure surface to near edge of strip footing	b	ft	0.5	2	6.5	6.5
				0.5 ft from footing	2.0 ft from footing	6.5 ft from footing	

SOIL SOLUTION

= Elastic Solution x R

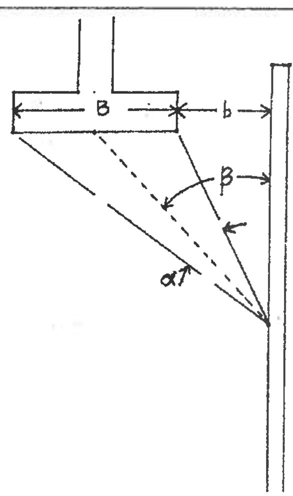
R=[tan^2(45-Phi/2)]/.5

R = 0.615

For b=	0.5	2	6.5	6.5
Vertical distance from load, z	Horizontal Surcharge at Depth z, Δ_{ph}	Horizontal Surcharge at Depth z, Δ_{ph}	Horizontal Surcharge at Depth z, Δ_{ph}	Horizontal Surcharge at Depth z, Δ_{ph}
ft				
0.1	217	47	10	10
0.5	807	227	50	50
1	1025	411	97	97
1.5	1043	536	142	142
2	992	605	182	182
2.5	913	632	218	218
3	823	630	247	247
3.5	734	609	270	270
4	649	576	288	288
5	502	497	308	308
6	386	416	310	310
7	299	344	301	301
8	234	282	284	284
9	184	232	263	263
10	147	192	240	240
10.5	132	175	229	229
11	119	159	217	217

ELASTIC SOLUTION

= (*p*/*π*) * [α-sin(α)*cos(2β)]



For b=	0.5	2	6.5	6.5
Vertical distance from load, z	Horizontal Surcharge at Depth z, Δ_{ph}	Horizontal Surcharge at Depth z, Δ_{ph}	Horizontal Surcharge at Depth z, Δ_{ph}	Horizontal Surcharge at Depth z, Δ_{ph}
ft	psf	psf	psf	psf
0.1	*353*	*76*	*16*	*16*
0.5	*1313*	*369*	*81*	*81*
1	*1669*	*669*	*159*	*159*
1.5	*1698*	*872*	*231*	*231*
2	*1614*	*985*	*297*	*297*
2.5	*1485*	*1029*	*354*	*354*
3	*1340*	*1025*	*402*	*402*
3.5	*1194*	*991*	*440*	*440*
4	*1056*	*938*	*468*	*468*
5	*816*	*808*	*500*	*500*
6	*629*	*677*	*505*	*505*
7	*487*	*559*	*490*	*490*
8	*380*	*460*	*462*	*462*
9	*300*	*378*	*428*	*428*
10	*239*	*312*	*391*	*391*
10.5	*215*	*284*	*372*	*372*
11	*193*	*259*	*354*	*354*

APPENDIX F

Table F-1 : Summary of Slope Stability Analyses

Section	Case Number and Description	Condition[1]	Required Horizontal Load in lbs./lf at anchor locations for Minimum FS =1.5				Factor of Safety [2]								
							Initial Site Cut to expose Footing	Cut 1 - Front	Cut 1 - Back	Back of Footing					
			Anchor 1	Anchor 2	Anchor 3	Total				Cut 1 with Anchor 1 installed	Cut 2 with Anchor 1 installed	Cut 2 with Anchors 1 and 2 installed	Cut 3 with Anchors 1 and 2 installed	Final Cut with all Anchors installed	Final Configuration with Toe backfill and Sidewalk in place
A-A'	**Case 1:** 3 Excavation Levels = 3 ft., 3 ft., and 4.5 ft.; 3 Rows of Anchors at 1.5 ft., 4.5 ft. & 8.25 ft. below bottom of footing	Temporary Excavation	4450	3750	5900	14100	1.6	5.8	1.6	1.8	1.4	1.9	1.1	1.5	>1.5
		Final	**5429**	**4575**	**7198**	**17202**	**Not applicable**							**1.5**	**>1.5**
A-A'	**Case 2:** Excavation Levels = 4 ft. and 6.5 ft.; 2 Rows of Anchors at 2 ft. & 7.25 ft. below bottom of footing	Temporary Excavation	5720	8400	NA	14120	1.6	5.4	1.5	1.9	0.9	1.5	NA	1.5	>1.5
		Final	**6921**	**10164**	**NA**	**17085**	**Not applicable**							**1.5**	**>1.5**
C-C'	**Case 1:** 3 Excavation Levels = 3 ft., 3 ft., and 4.5 ft.; 3 Rows of Anchors at 1.5 ft., 4.5 ft. & 8.25 ft. below bottom of footing	Temporary Excavation	3250	3600	4700	11550	1.6	3.2	1.6	1.8	1.5	1.9	1.1	1.5	>1.5
		Final	**4095**	**4536**	**5922**	**14553**	**Not applicable**							**1.5**	**>1.5**
C-C'	**Case 2:** Excavation Levels = 4 ft. and 6.5 ft.; 2 Rows of Anchors at 2 ft. & 7.25 ft. below bottom of footing	Temporary Excavation	4470	7075	NA	11545	1.6	2.5	1.6	1.9	1.0	1.5	NA	1.5	>1.5
		Final	**5588**	**8844**	**NA**	**14431**	**Not applicable**							**1.5**	**>1.5**

Notes: (1) - Analyses of Temproary Excavations based on Peak Shear Strength Parameters; Analyses for Final Conditoin used Ultimate Strength Parameters; See Text.

(2) - Cuts 1, 2 and 3 refer to the temporary excavation levels for tie-back anchors to be installed prior to excavating the next level.

Table F-2 : Slot Cut Evalaution Analyses

Section	Cut	Governing Slide Surface Discription	Transition Zone	FS_{MAX}	FS_{MIN}	$FS_{TRANSITION}$	Weighted $FS_{DESIRED}$	Calculated Slot Width for ABC Slot Cut Method for Weighted FS = 1.5
	(feet)		(feet)	(at edge of slot)	(at center of slot)			(feet)
A1	First, 3 ft	Back of Footing	6	**1.6**	**1.6**	1.6	1.5	FS > 1.5
	Second, 3 ft	Back of Footing	6	**1.8**	**1.4**	1.6	1.5	31
	Third, 4.5 ft	Back of Footing	6	**1.9**	**1.1**	1.5	1.5	11
A2	First, 4 ft	Back of Footing	6	**1.6**	**1.5**	1.5	1.5	FS > 1.5
	Second, 6.5 ft	Back of Footing	6	**1.9**	**0.9**	1.4	1.5	10
C1	First, 3 ft	Back of Footing	6	**1.6**	**1.6**	1.6	1.5	FS > 1.5
	Second, 3 ft	Back of Footing	6	**1.8**	**1.5**	1.7	1.5	FS > 1.5
	Third, 4.5 ft	Back of Footing	6	**1.9**	**1.1**	1.5	1.5	12
C3	First, 4 ft	Back of Footing	6	**1.6**	**1.6**	1.6	1.5	FS > 1.5
	Second, 6.5 ft	Back of Footing	6	**1.9**	**1.0**	1.5	1.5	10

KLEINFELDER
Bright People. Right Solutions.

SHEET 1 OF 1

PROJECT NO. 98834 / GEO 2

PROJECT Fulkerth | REVIEWED BY | DATE

SUBJECT Example Slot Cut Width Analyses | BY JK | DATE 1/17/11

SECTION A-A'; CASE 1; CUT 3

Failure Behind Footing : $FS_{Max} = 1.86$
$FS_{Min} = 1.08$

Failure ahead of footing not possible

Transition zone $FS = \left(\frac{1.86+1.08}{2}\right) = 1.47$

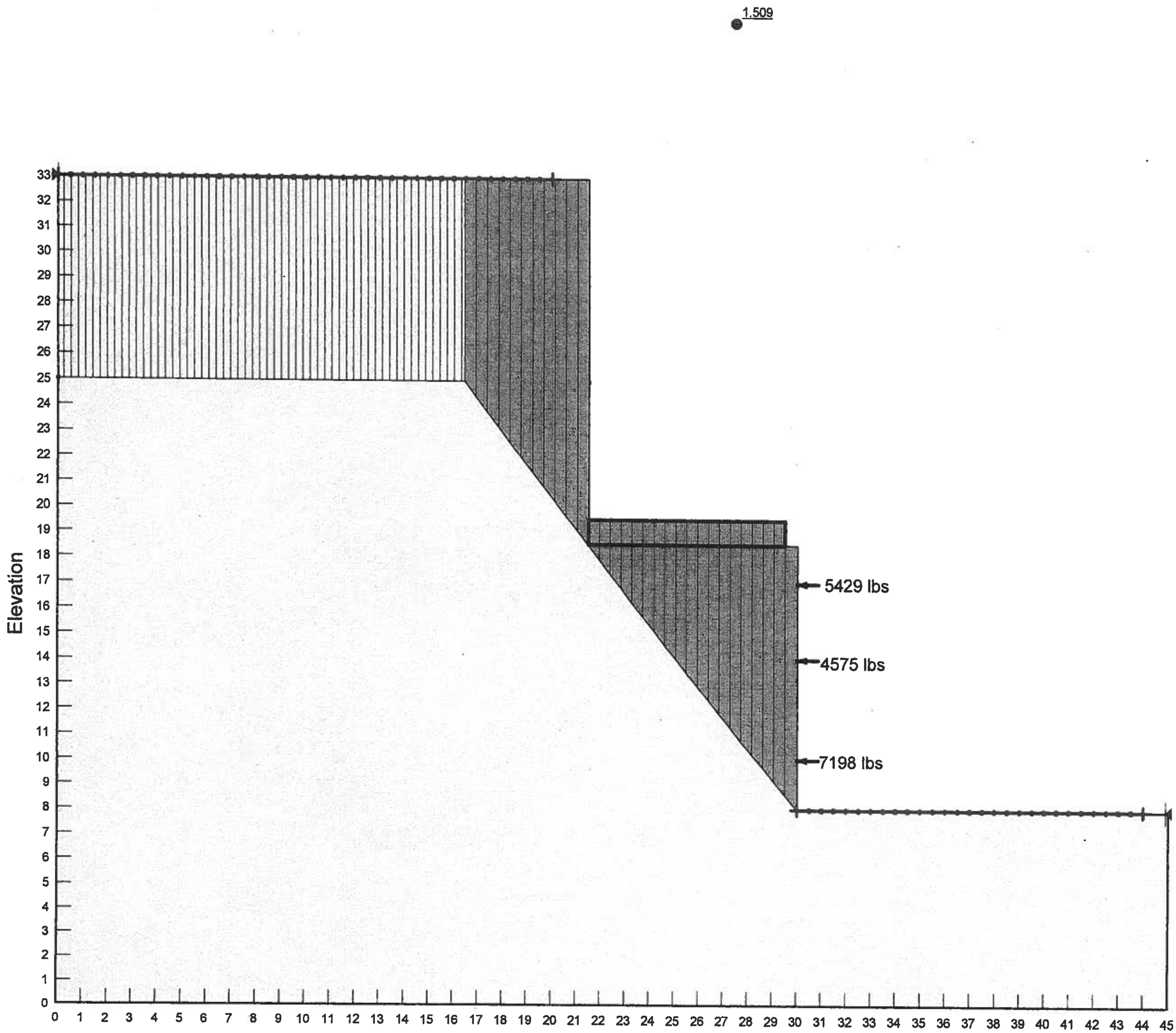
Transition zone = 6' (assumed)

Weighted FS = 1.5 for slotcut width, d

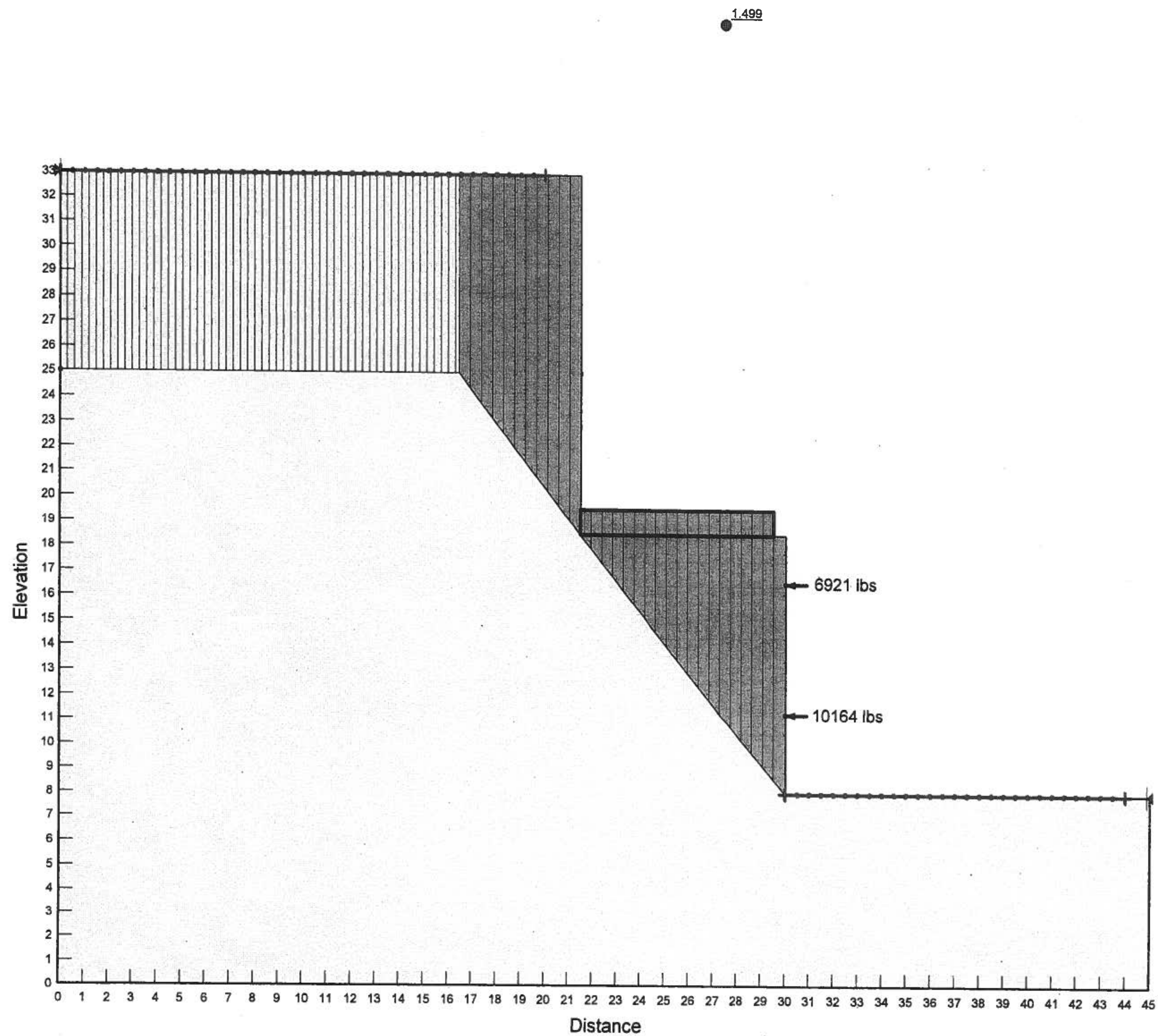
Solve for d $1.5(d) = 1.47 \cdot 2 \cdot 6' + 1.08 \cdot (d - 2 \cdot 6)$

$d = 11.1$ Say 11' max

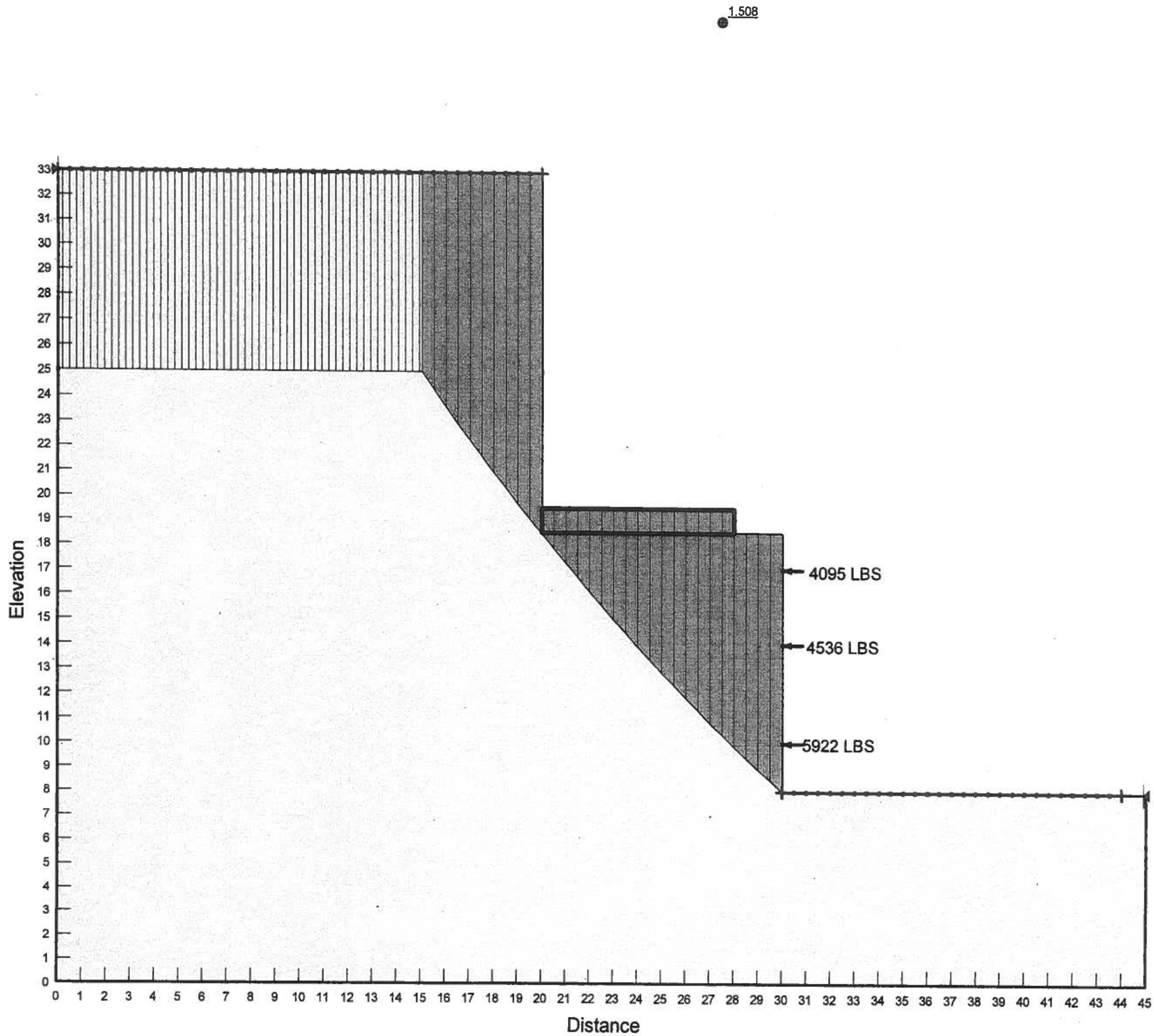
Sample Slot Cut Calculation



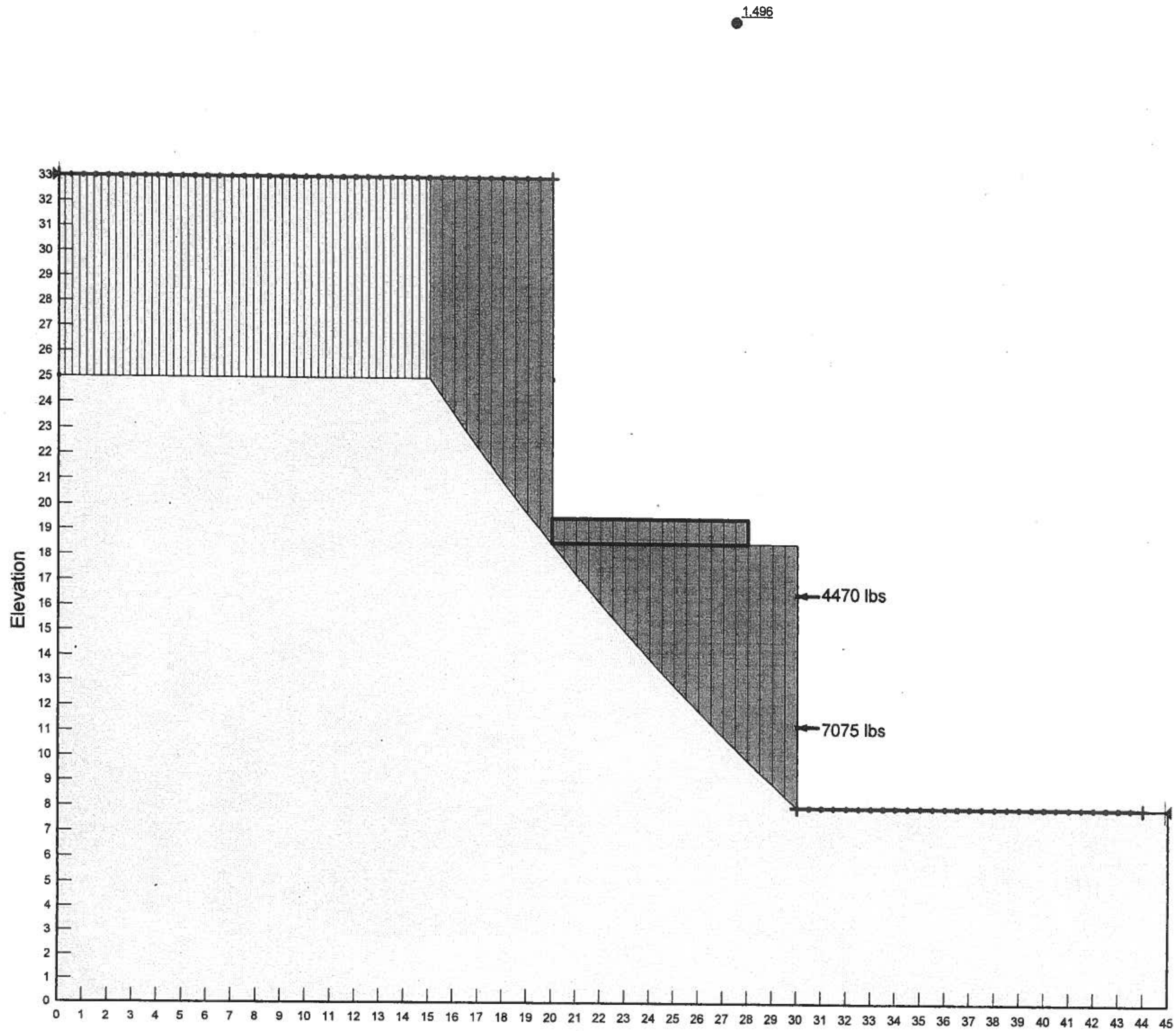
Section A-A' Case 1 - Cut 3 w/ TB (FINAL)



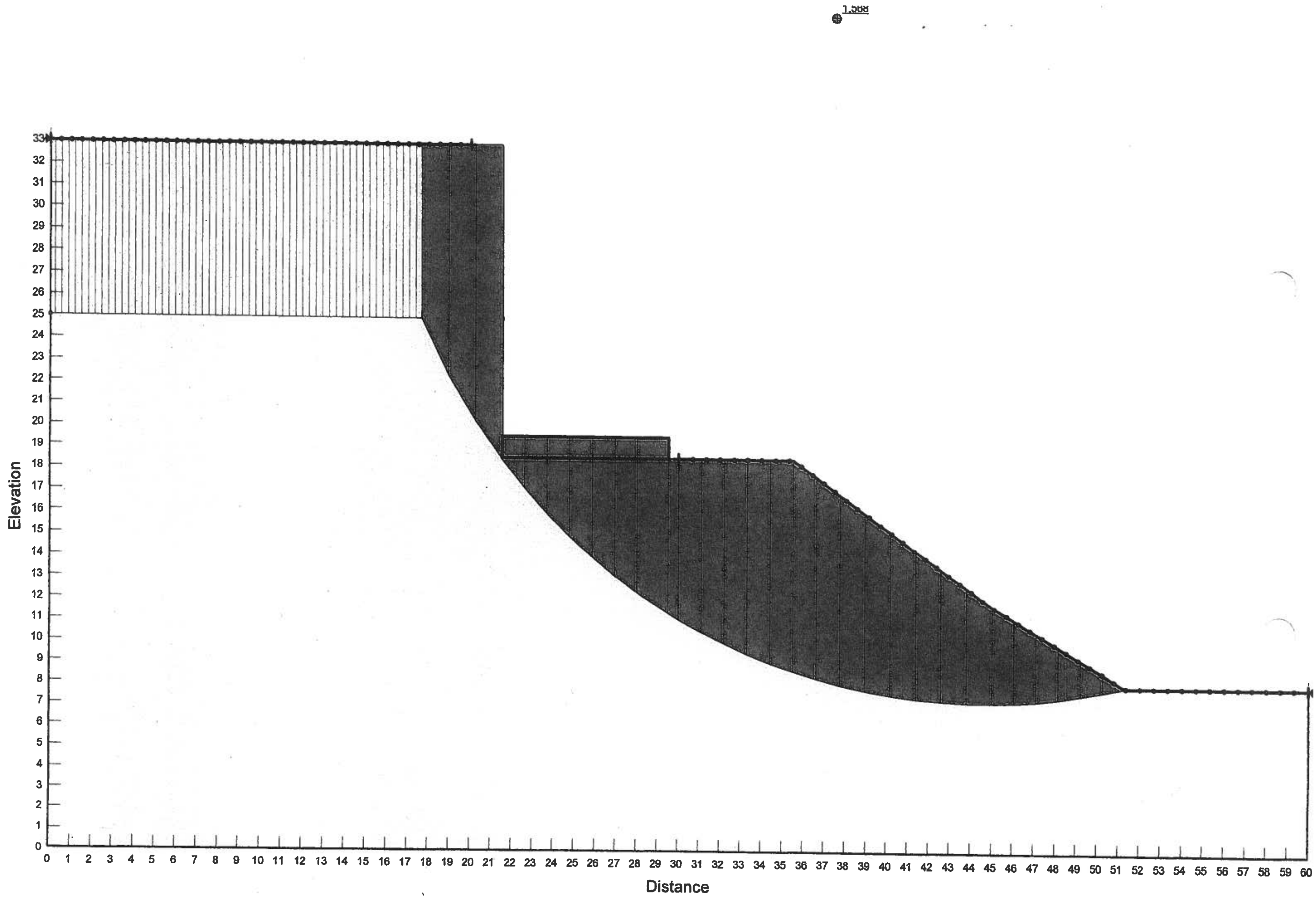
Section A-A' Case 2 - Cut 2 w/ TB (FINAL)



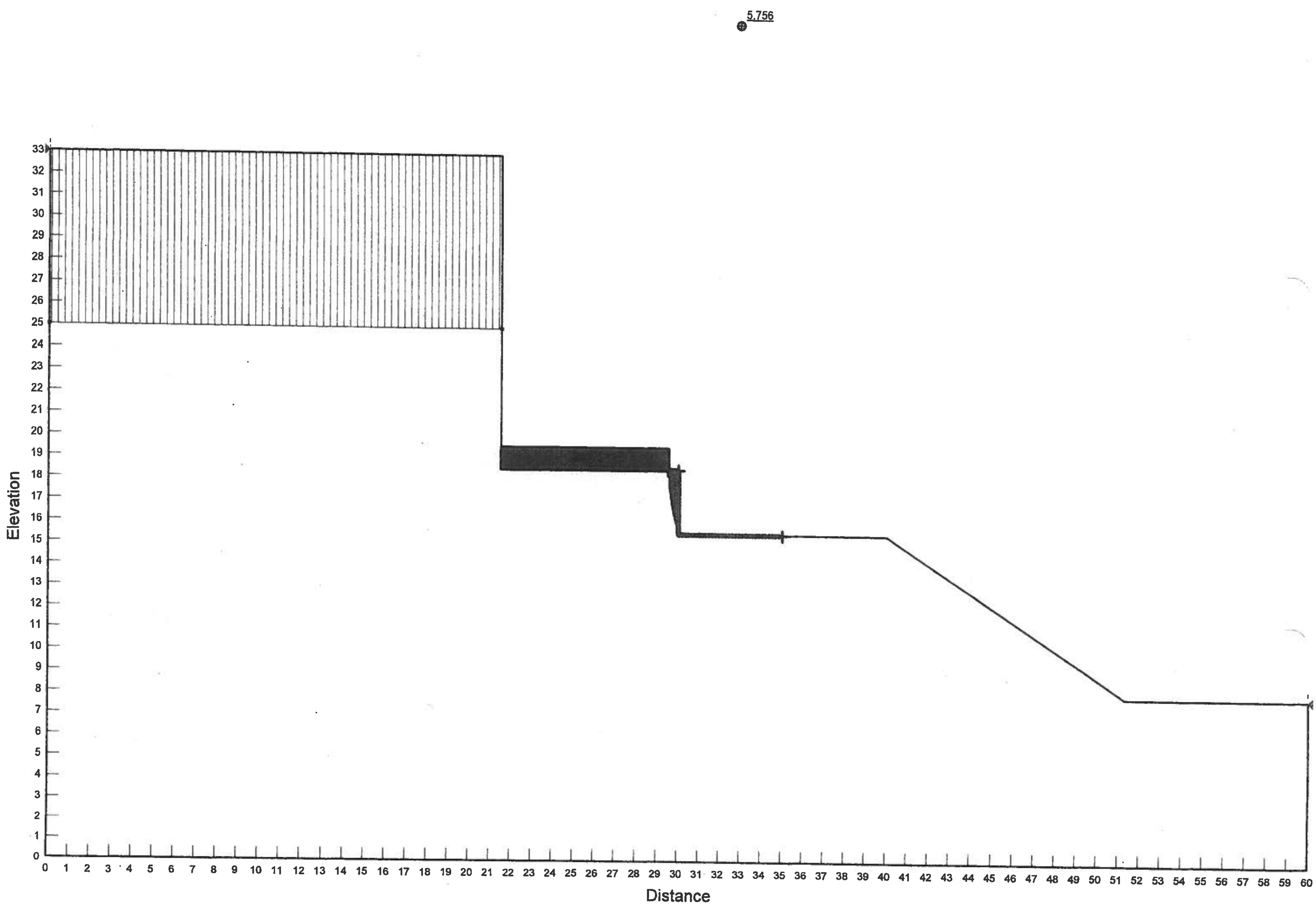
Section C-C' Case 1 - Cut 3 w/ TB (FINAL)



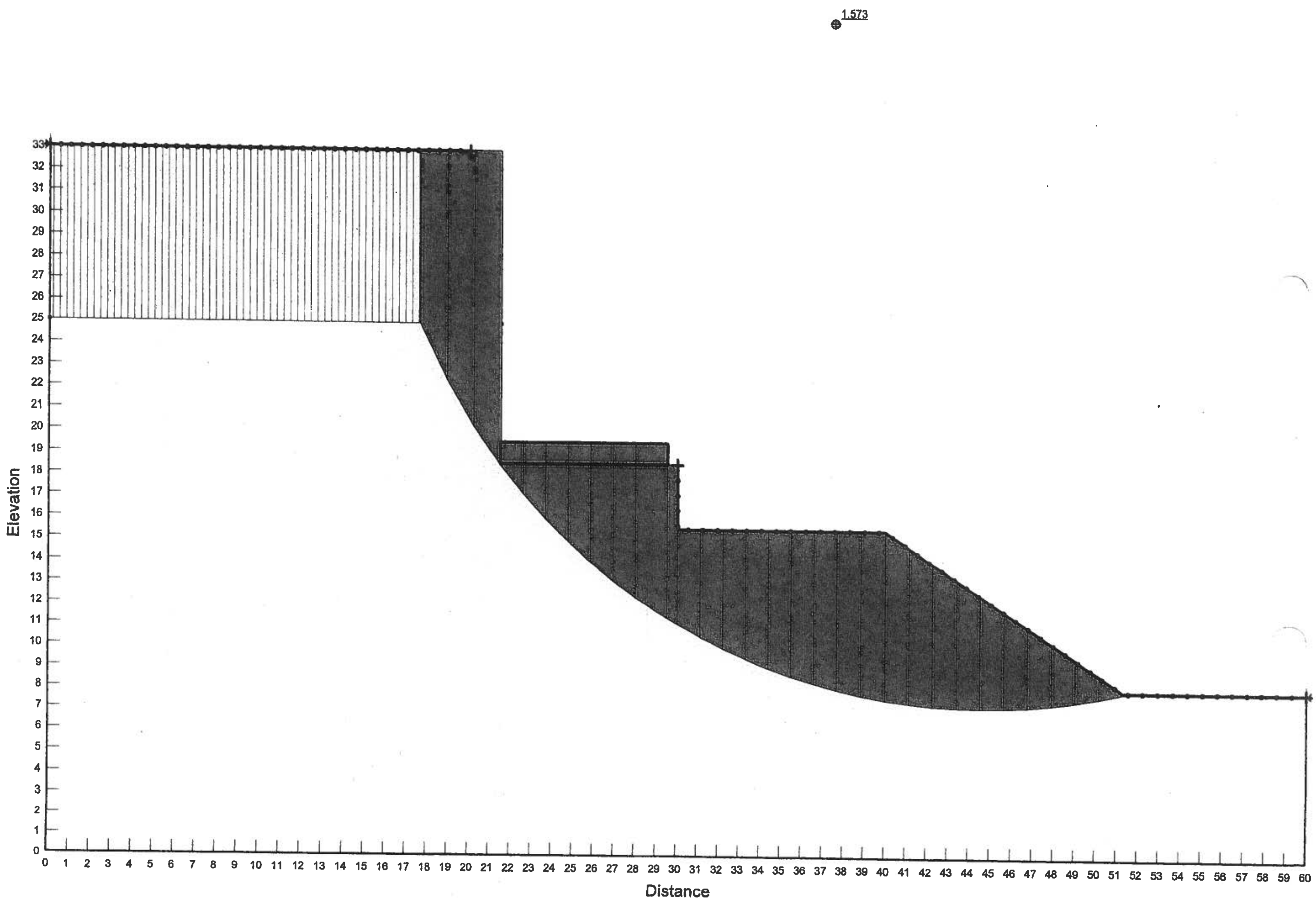
Section C-C' Case 2 - Cut 2 w/ TB (FINAL)



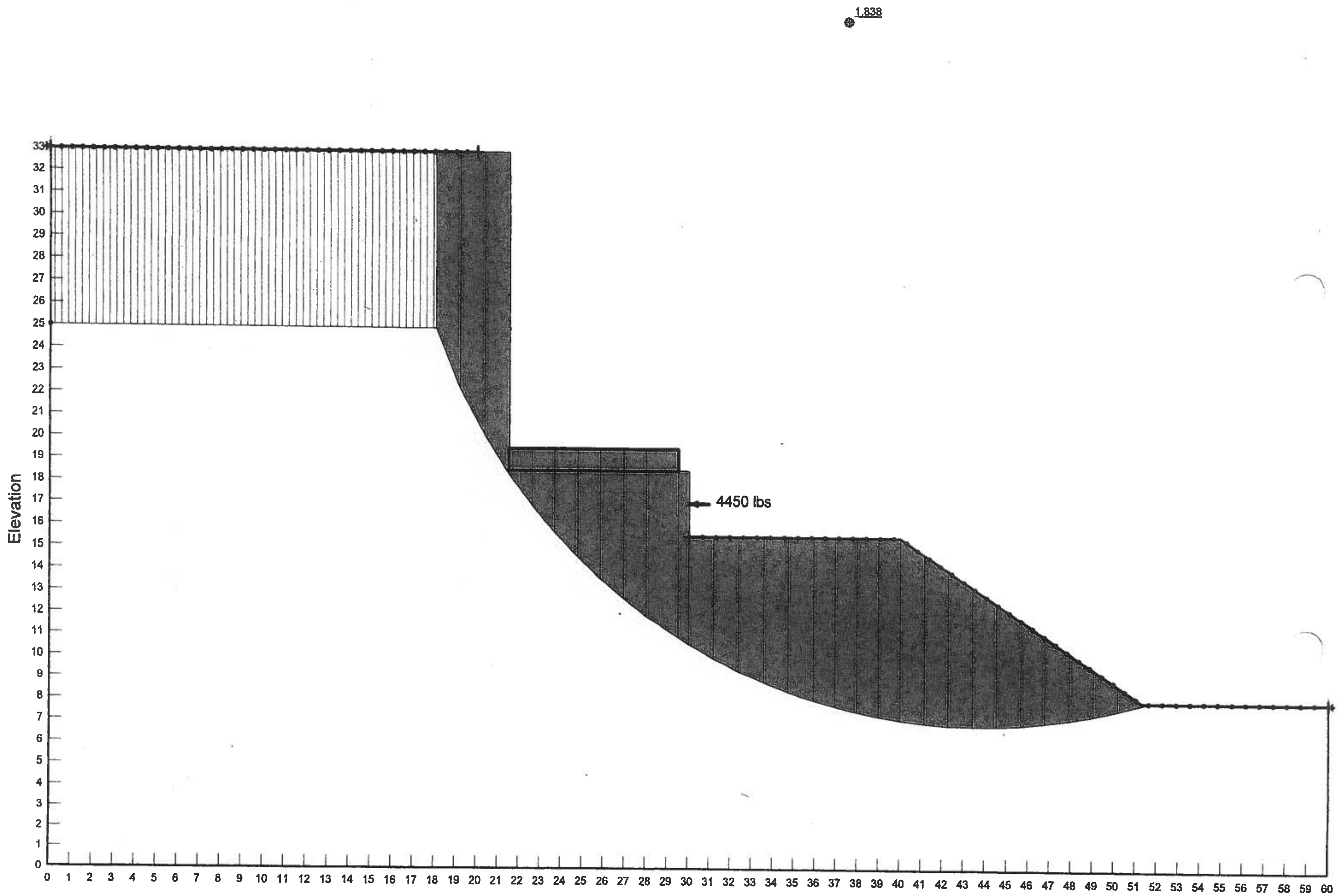
Initial Excavation



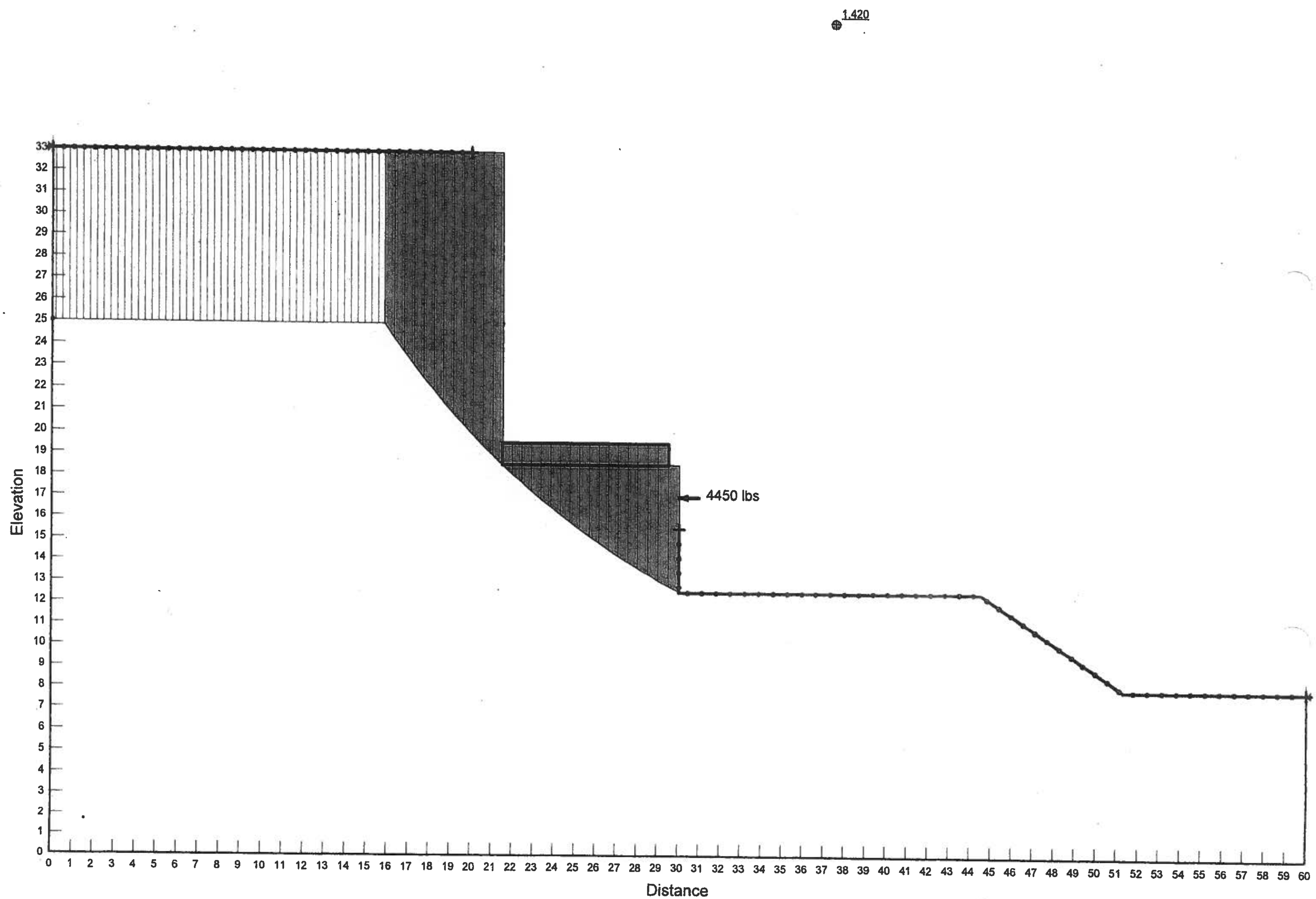
Section A-A' Case 1 - Cut 1



Section A-A' Case 1 - Cut 1



Section A-A' Case 1 - Cut 1 w/ TB



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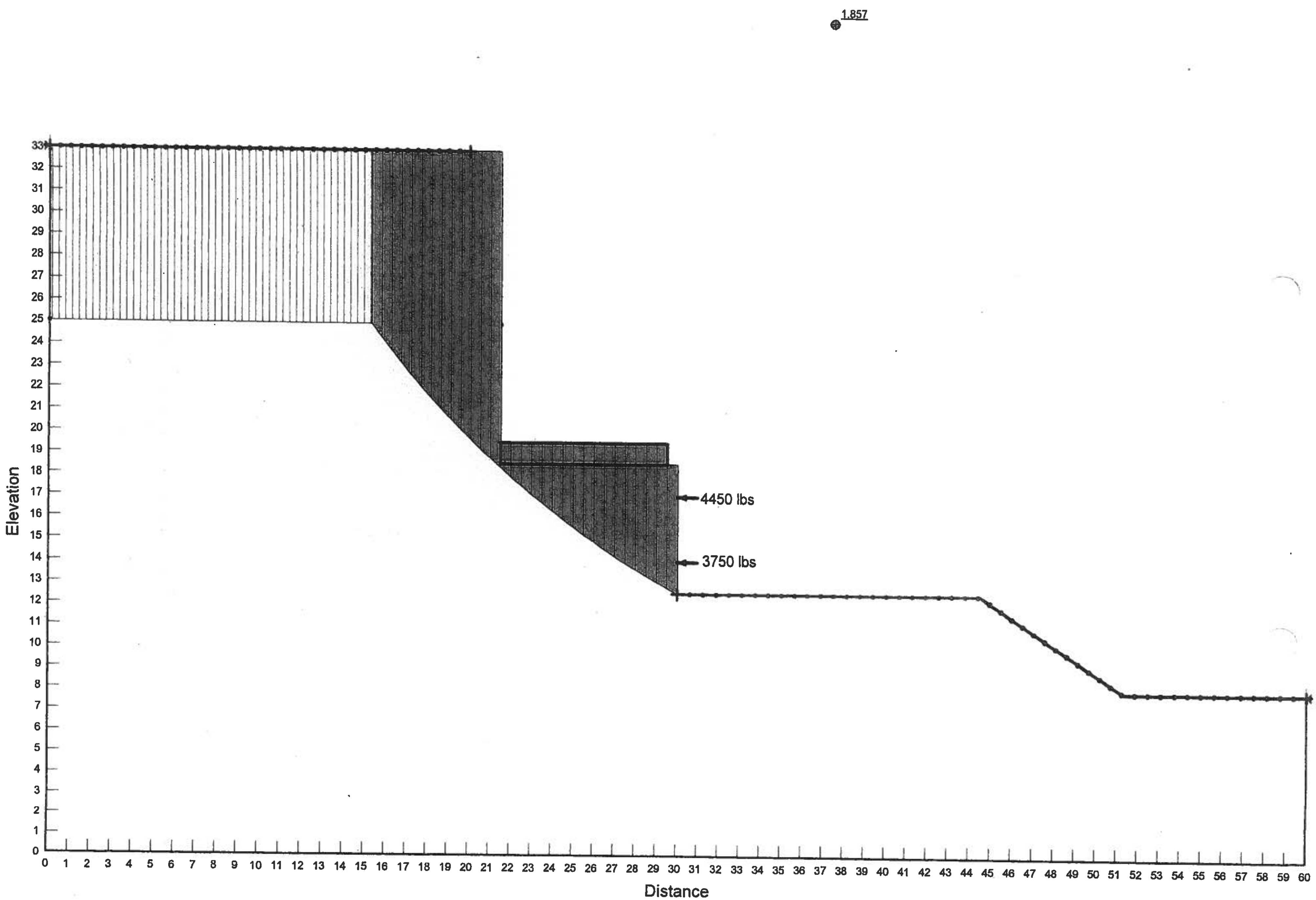
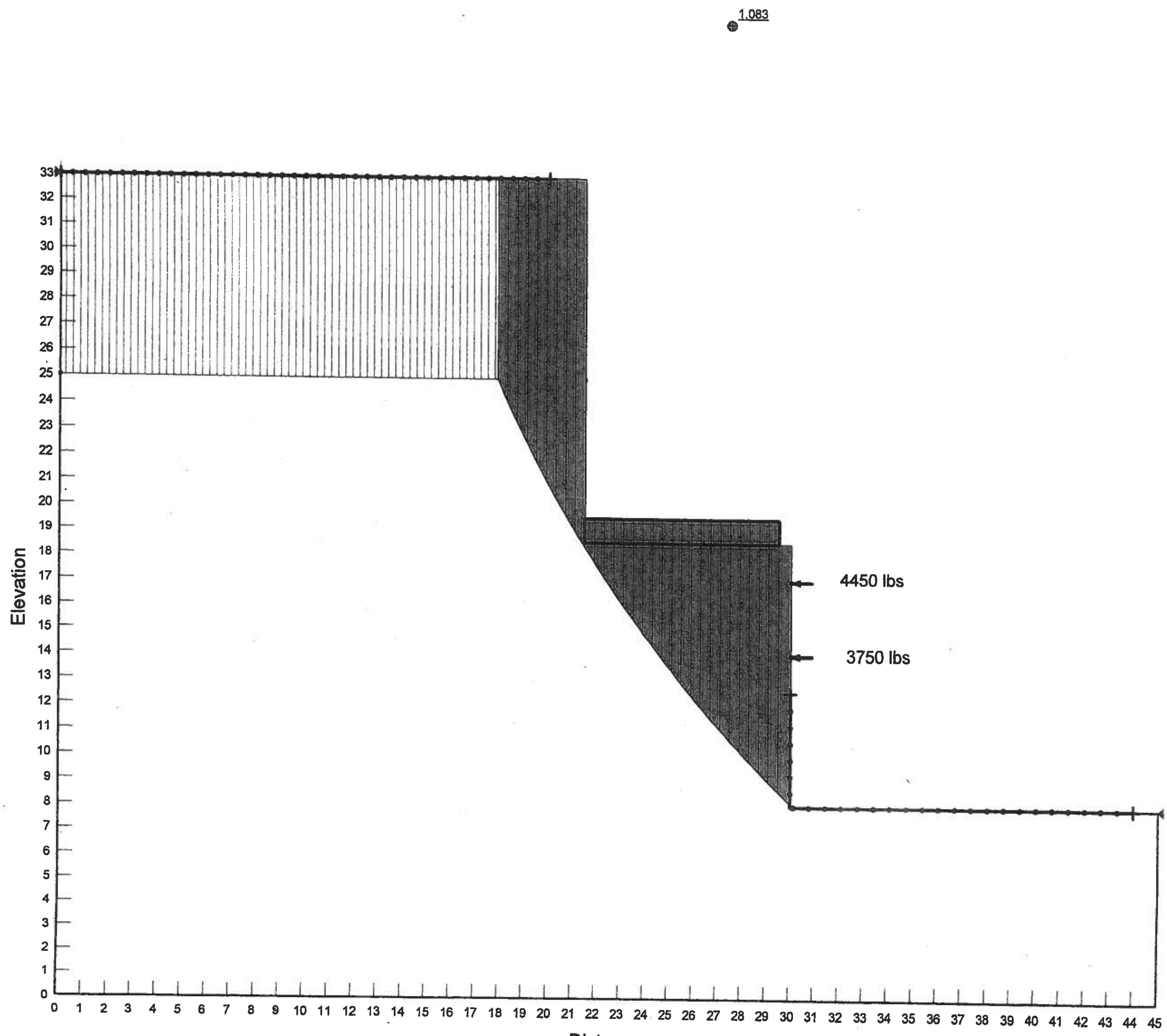
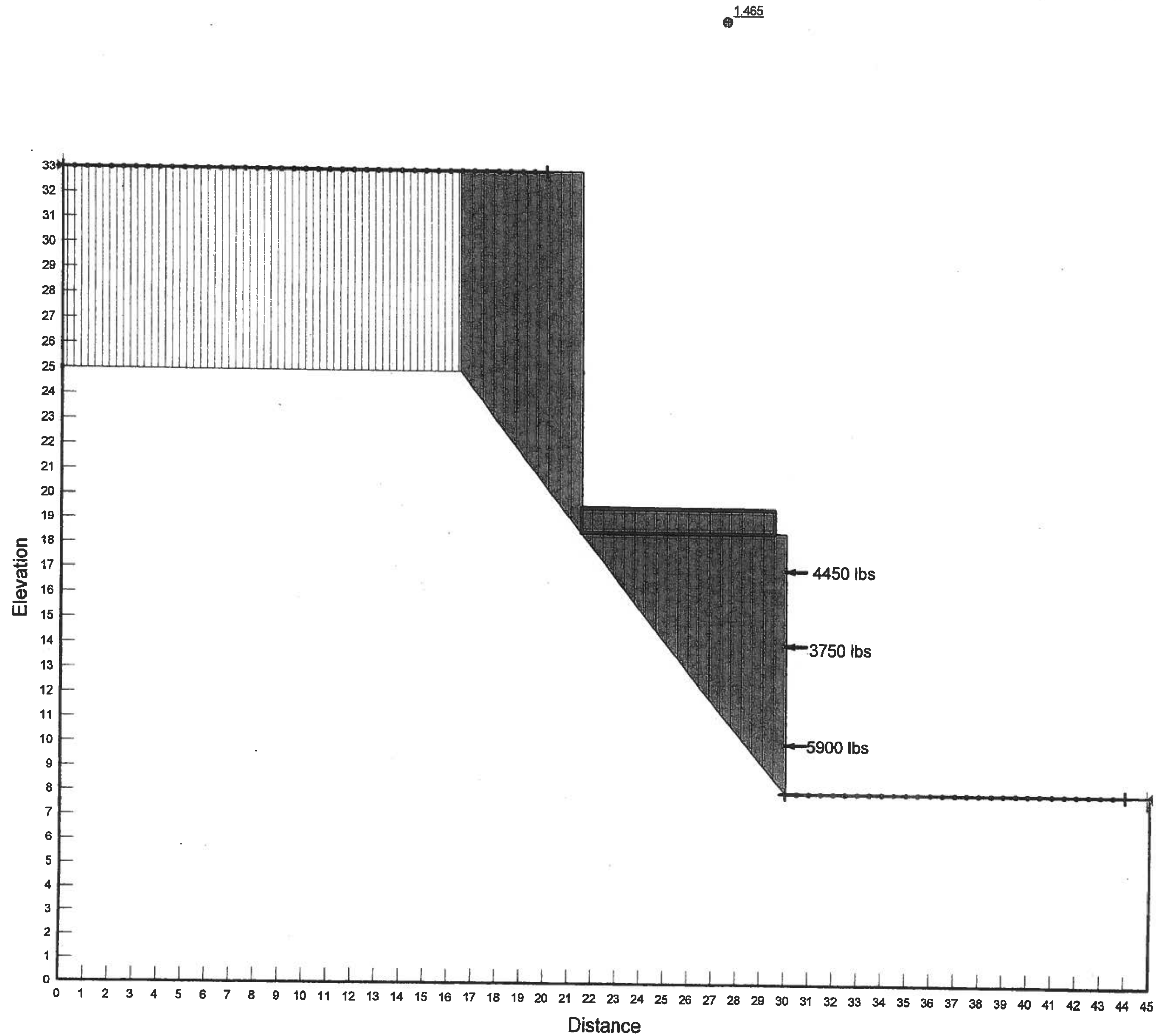
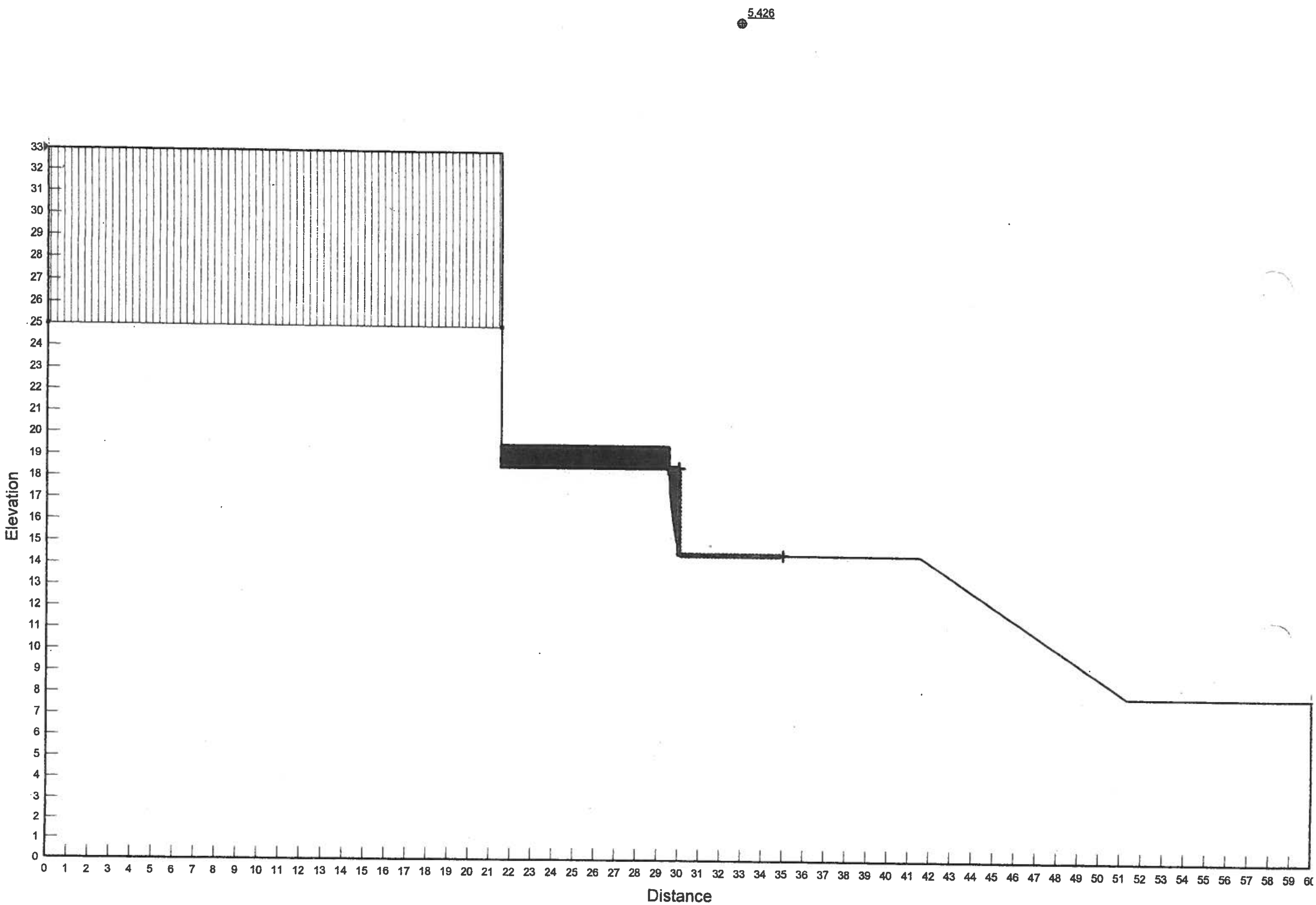


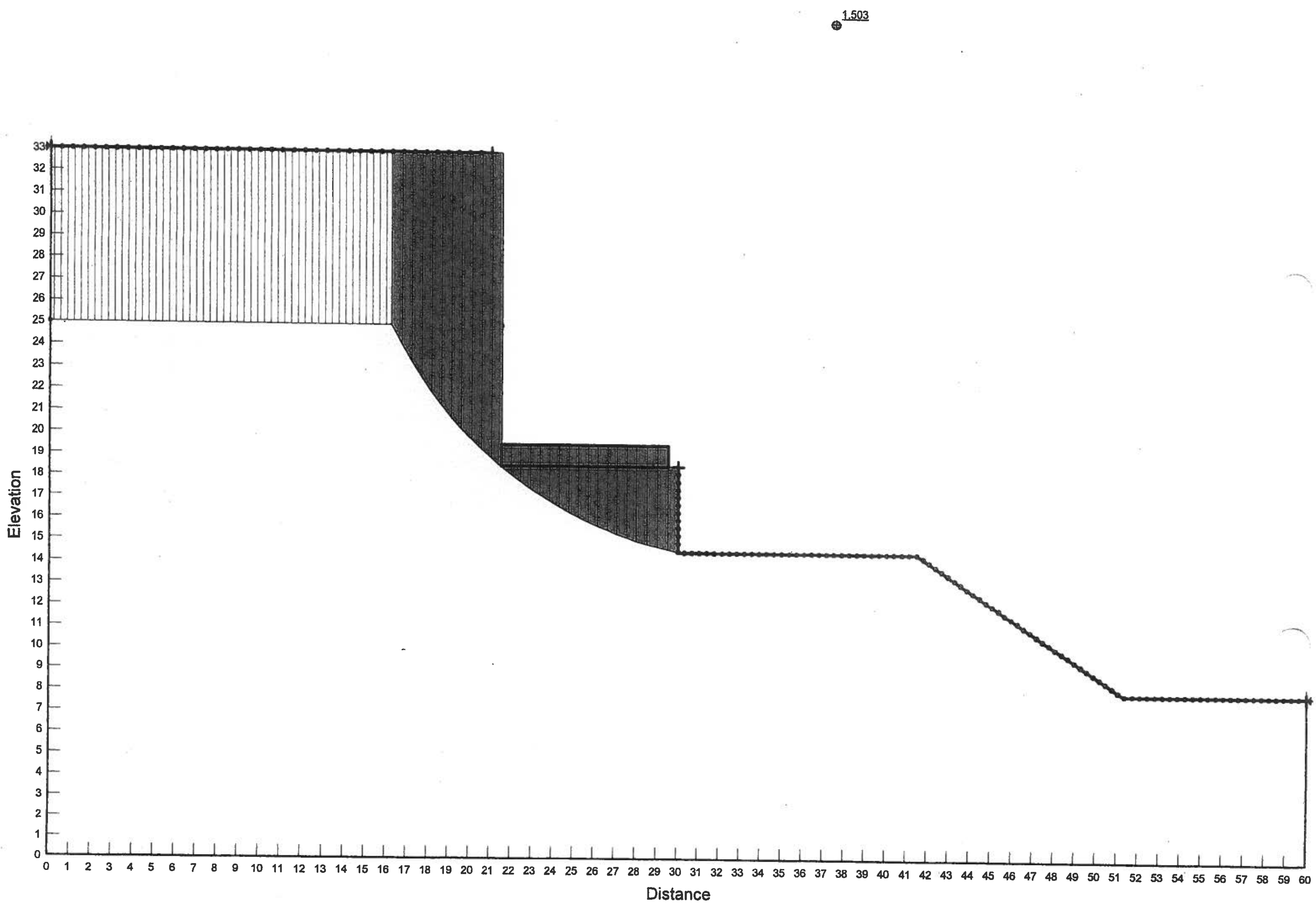
Section A-A' Case 1 - Cut 2 w/ TB


1.083
4450 lbs
3750 lbs
Elevation
Distance

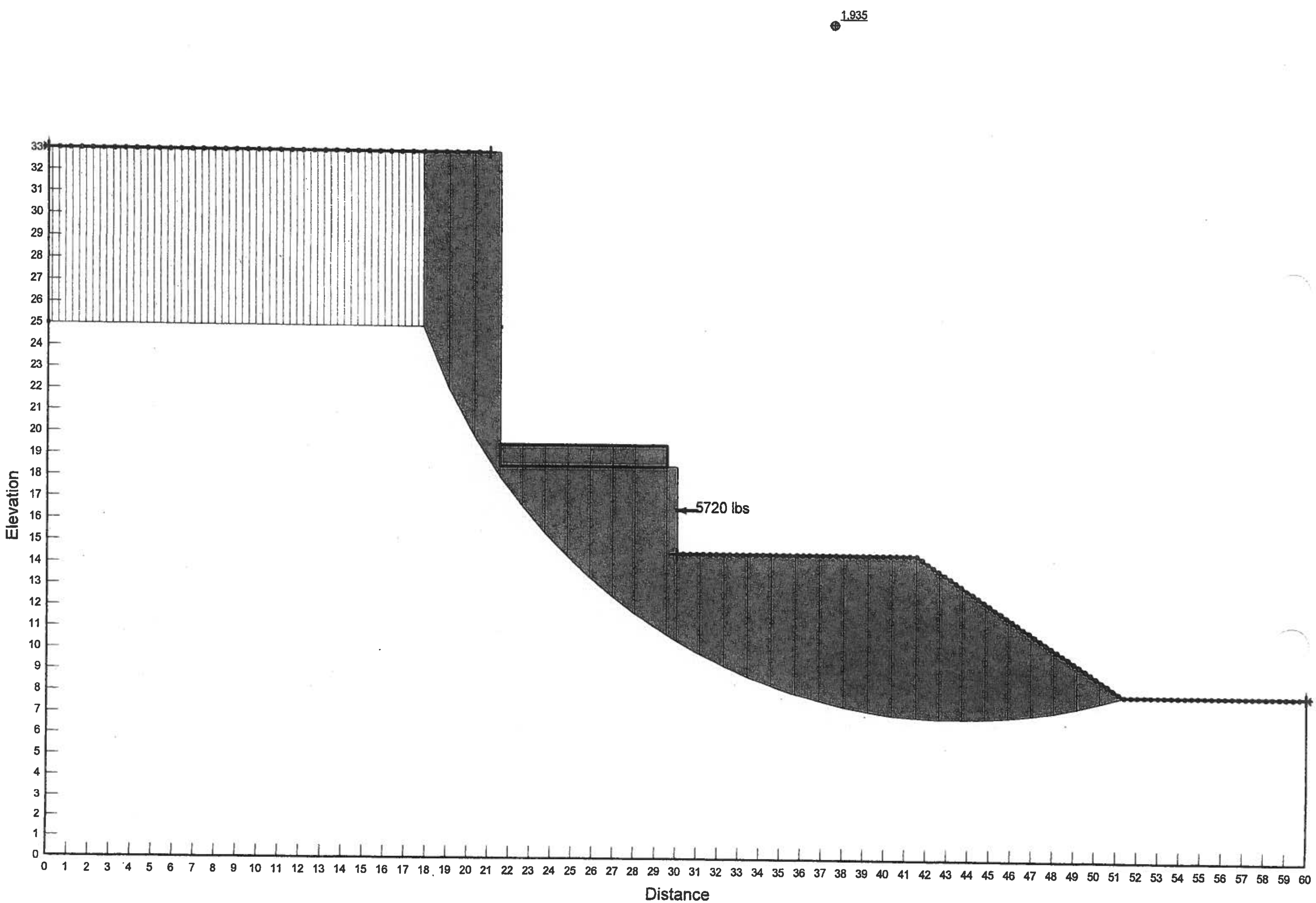

1.465
4450 lbs
3750 lbs
5900 lbs
Elevation
Distance
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33



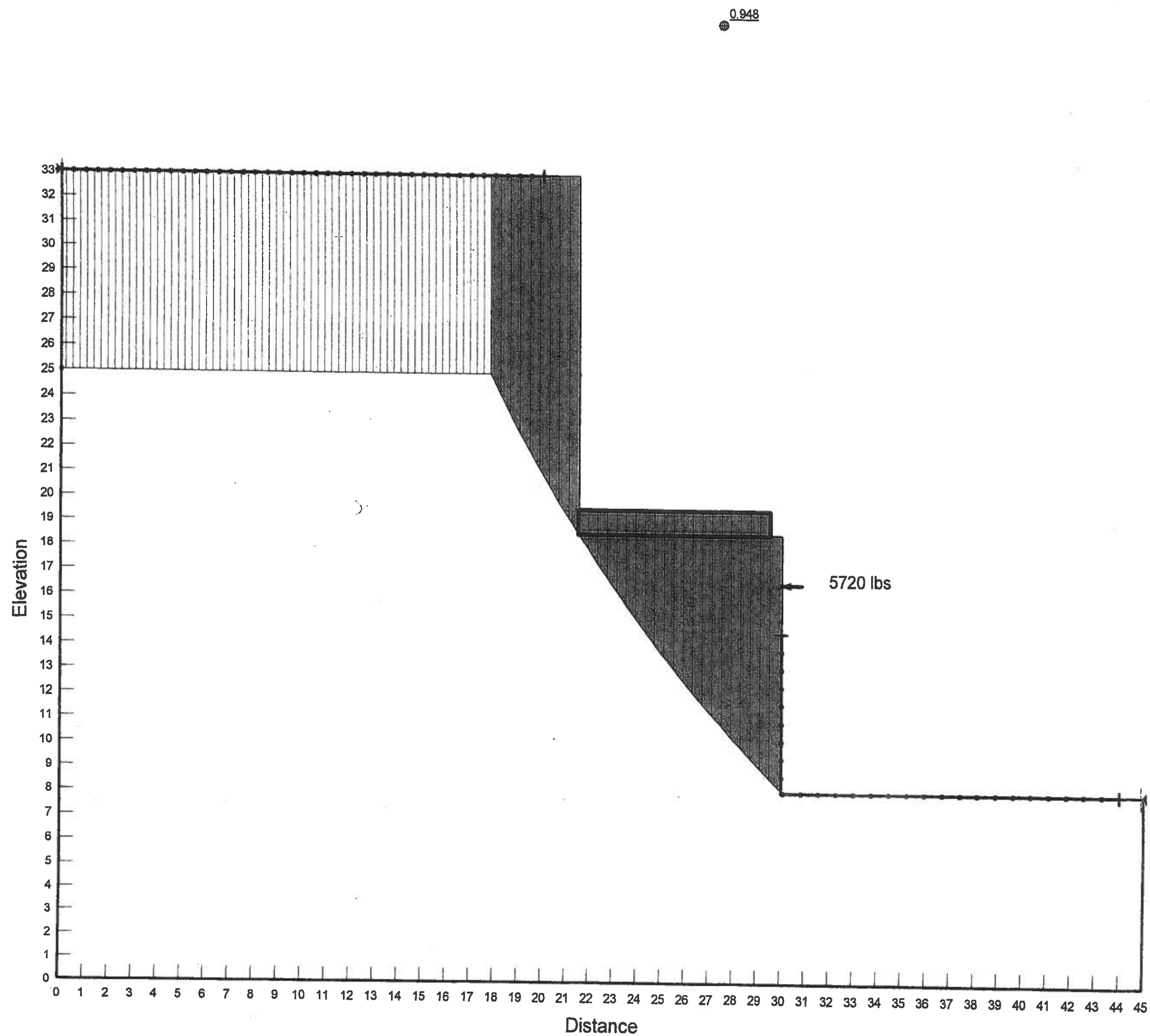
Section A-A' Case 2 - Cut 1



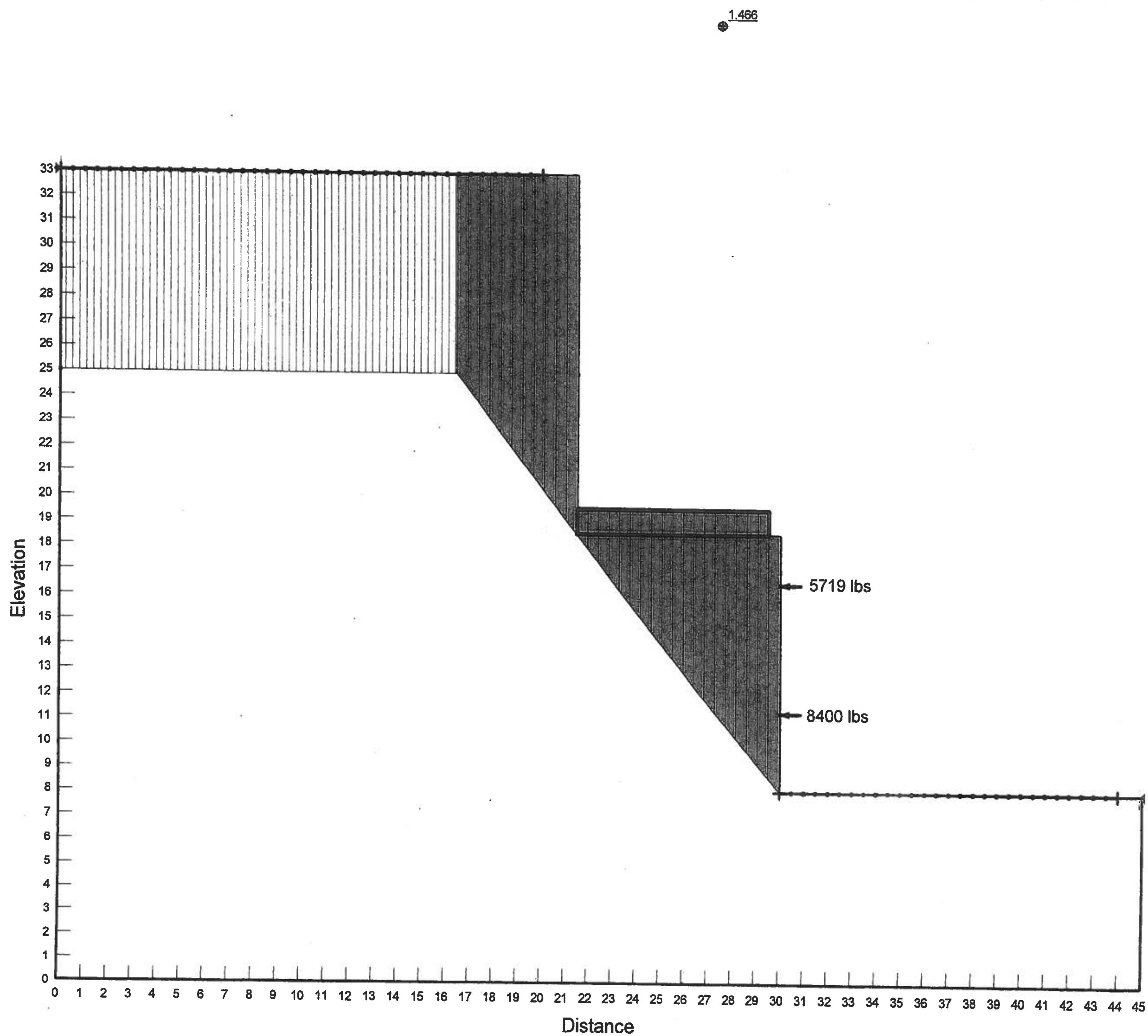
Section A-A' Case 2 - Cut 1



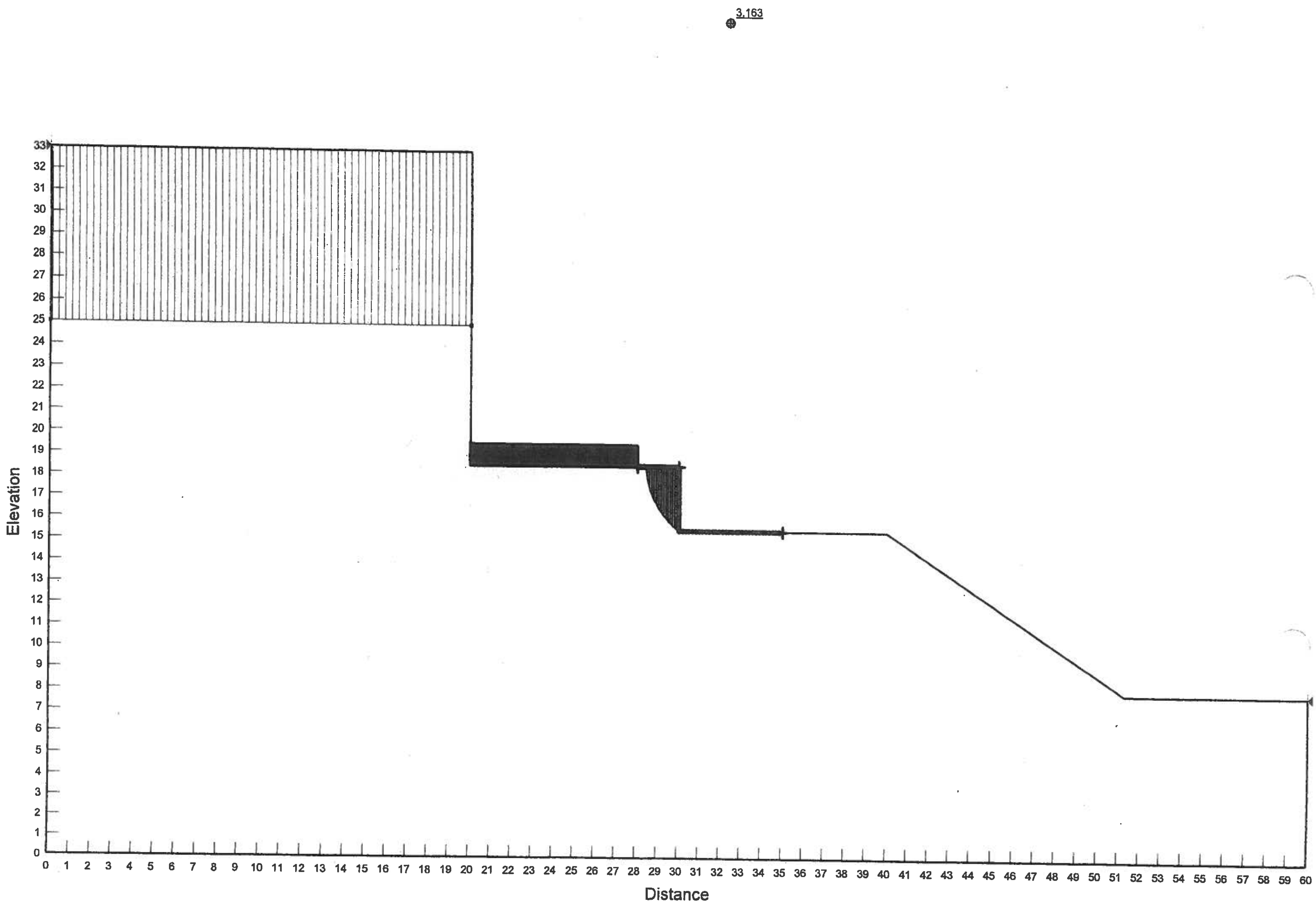
Section A-A' Case 2 - Cut 1 w/ TB



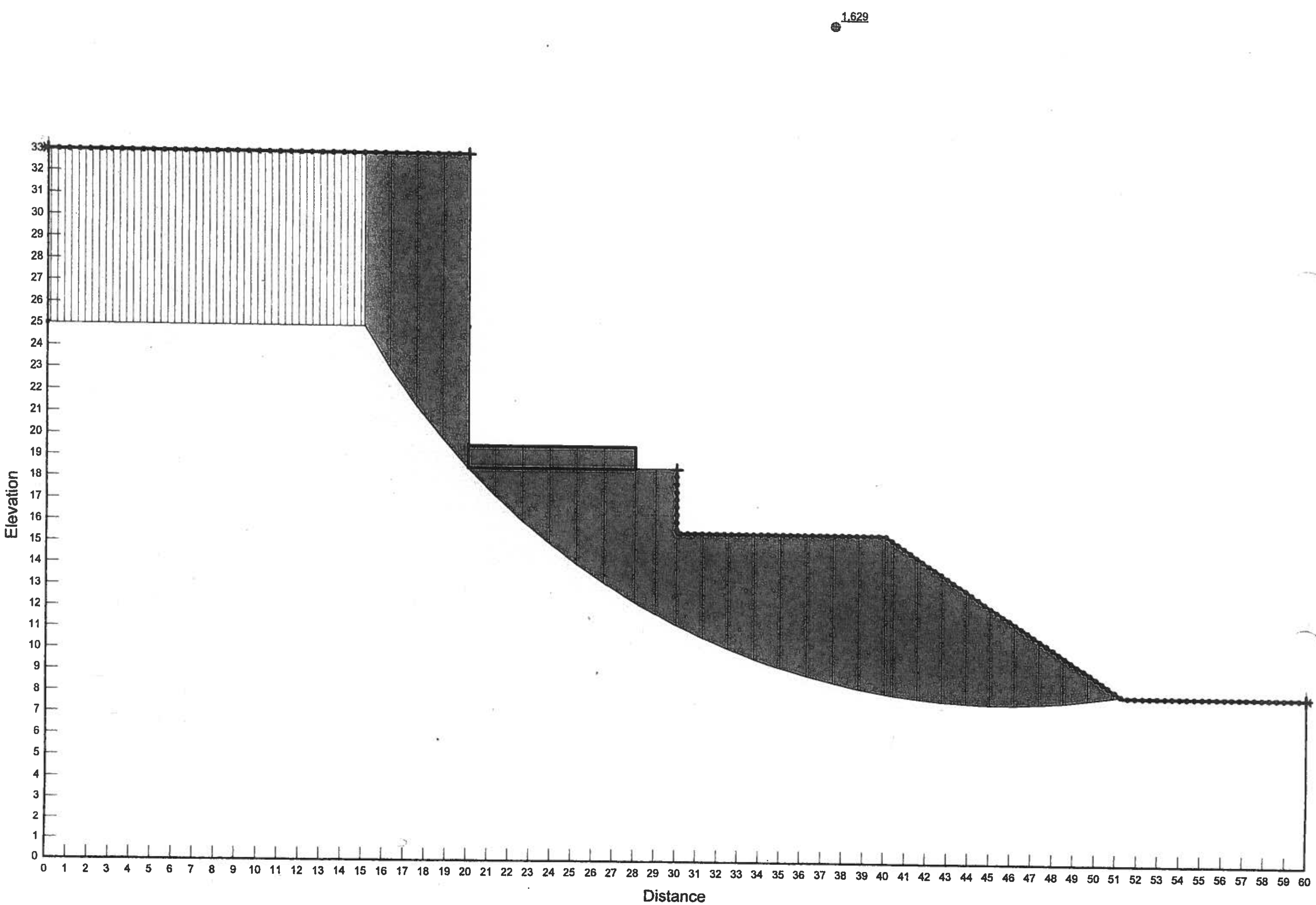
0.948
5720 lbs
Elevation
Distance



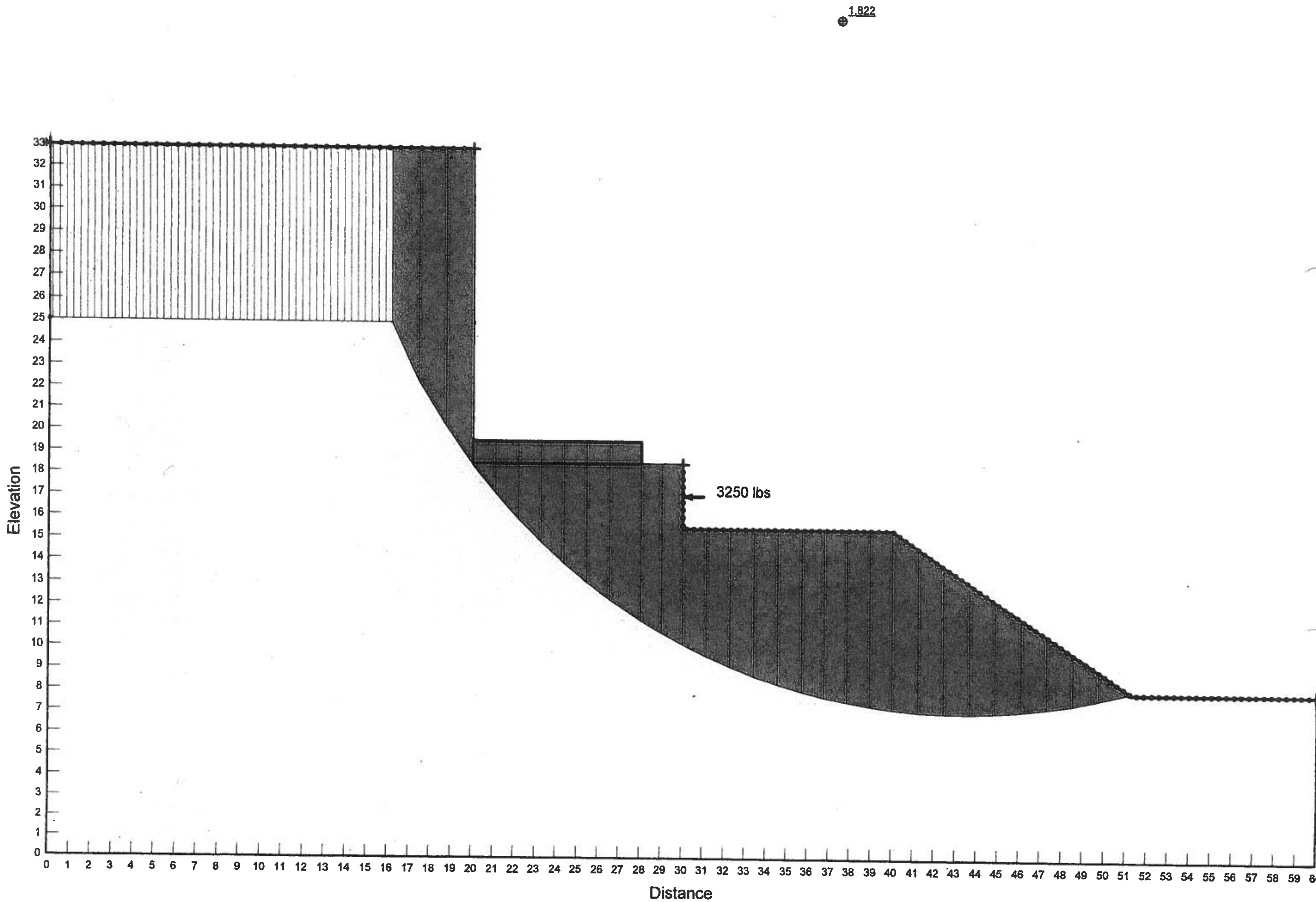
1.466
5719 lbs
8400 lbs
Elevation
Distance



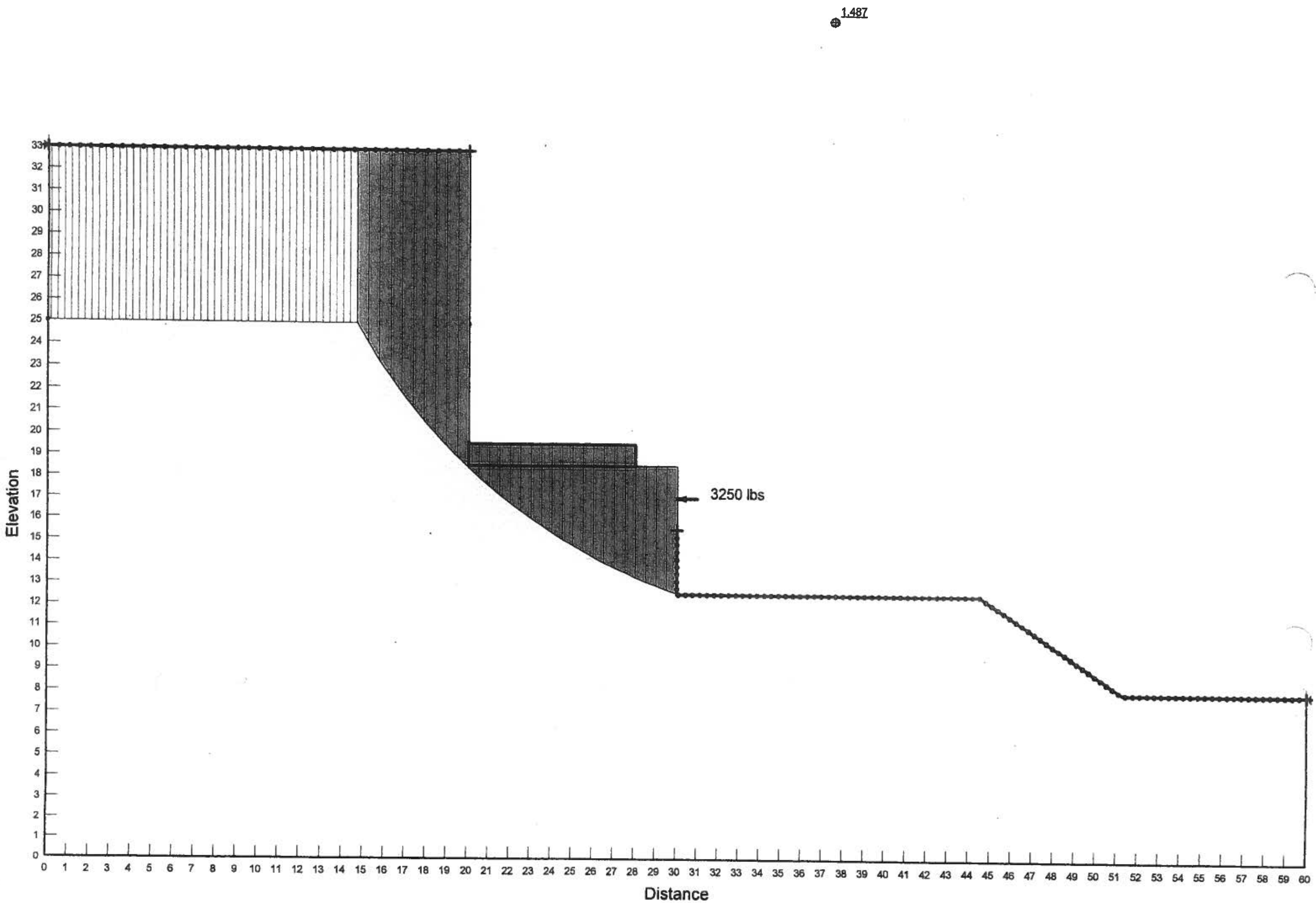
Section C-C' Case 1 - Cut 1



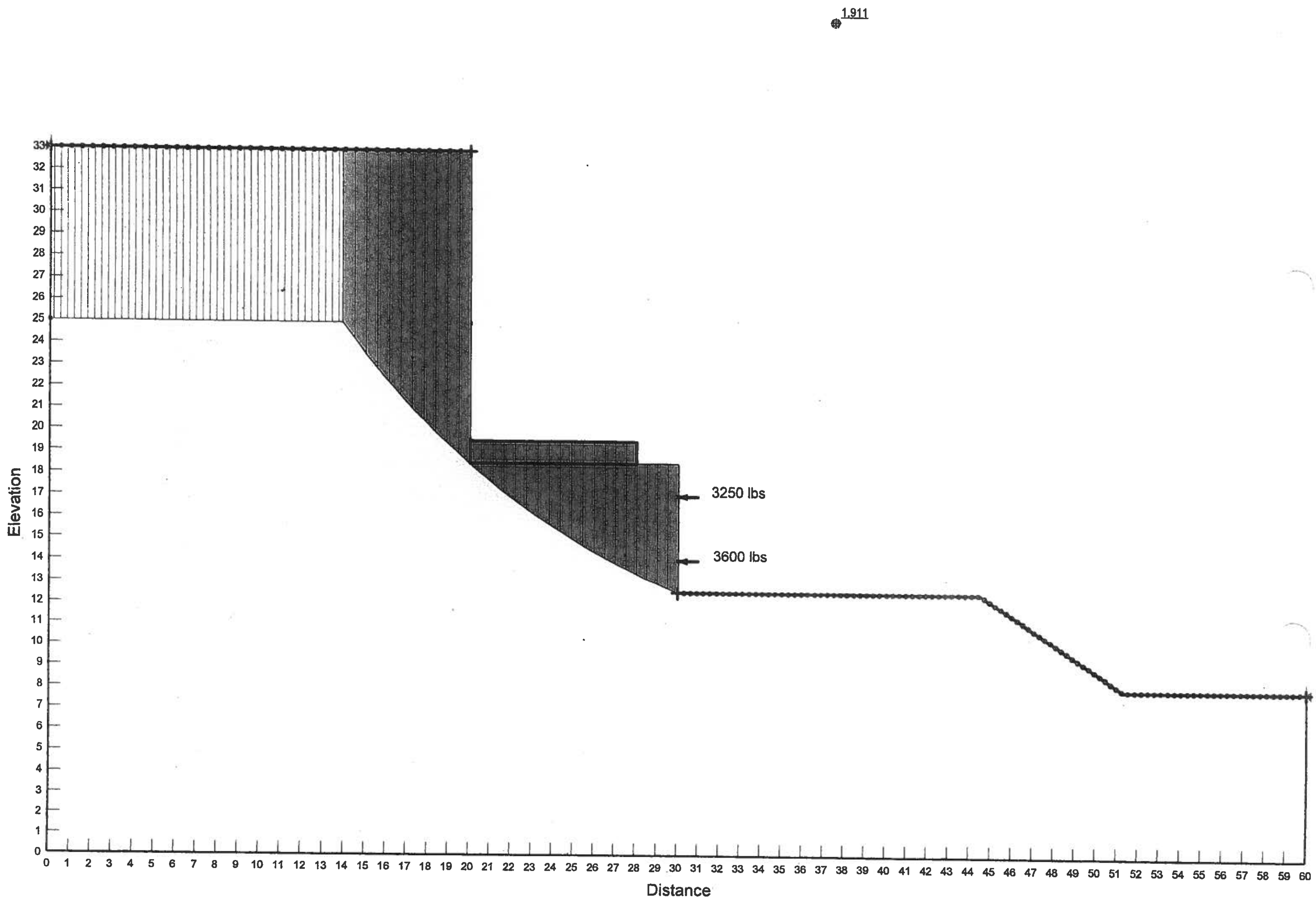
Section C-C' Case 1 - Cut 1



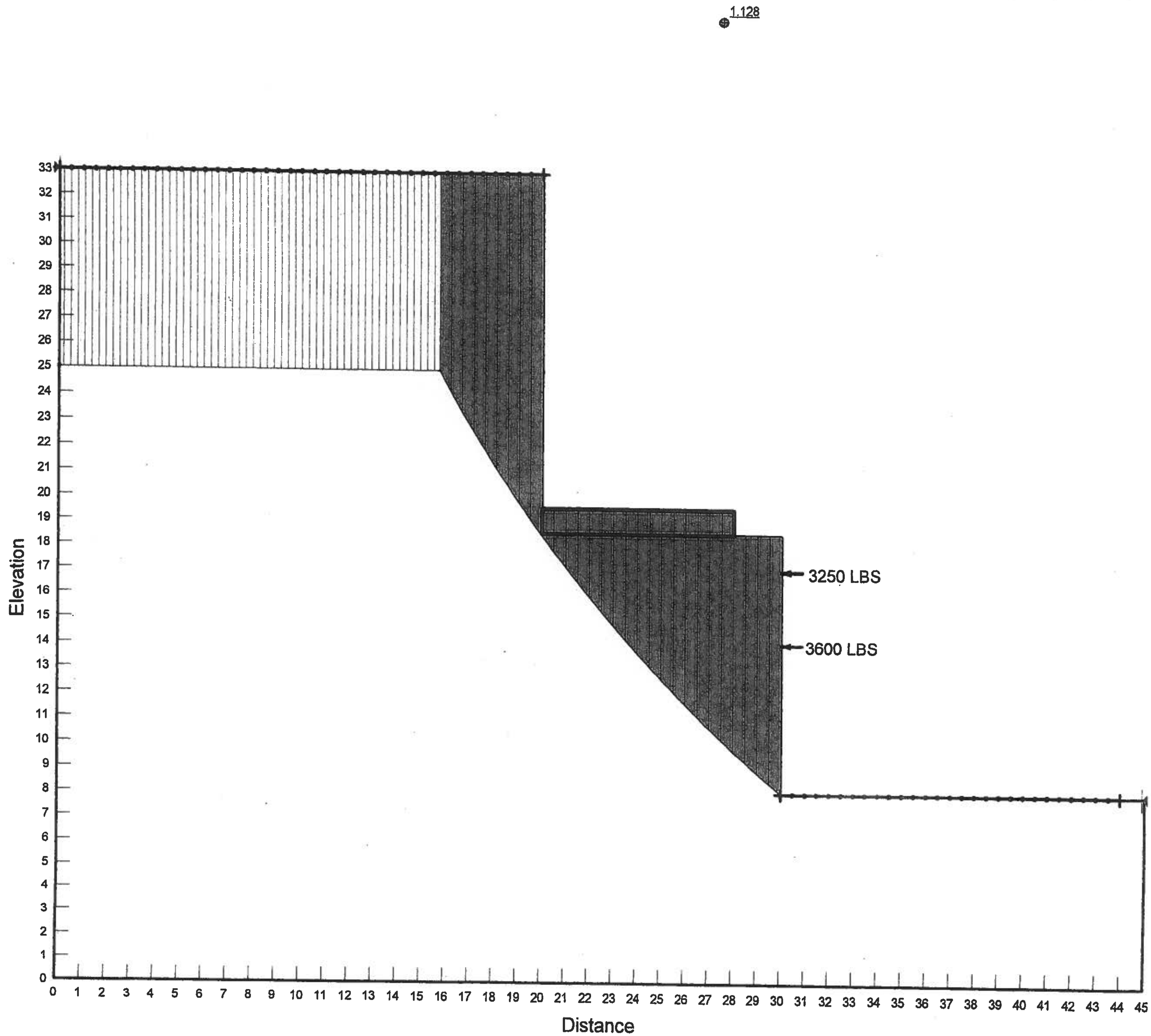
Section C-C' Case 1 - Cut 1 w/ TB

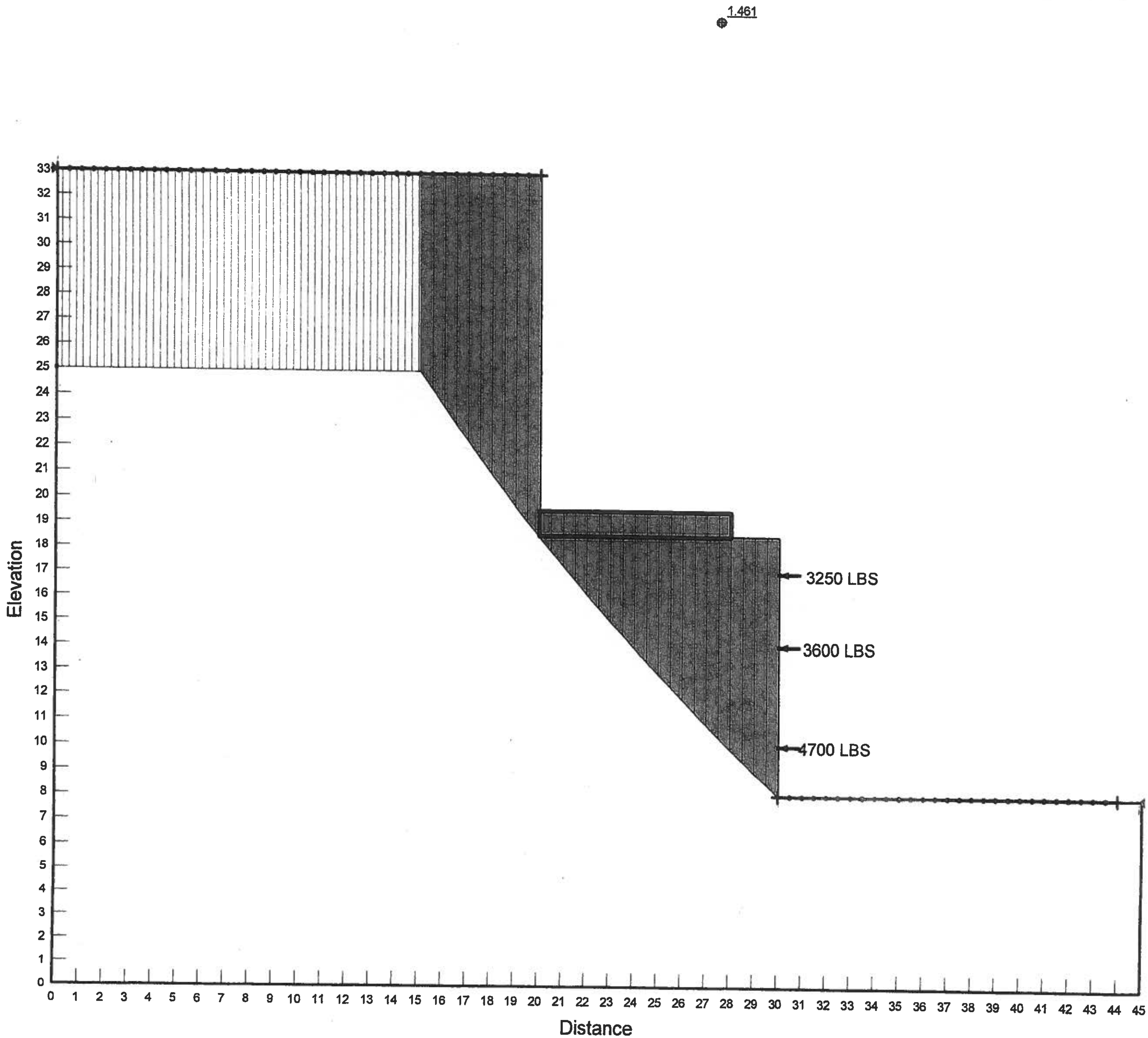


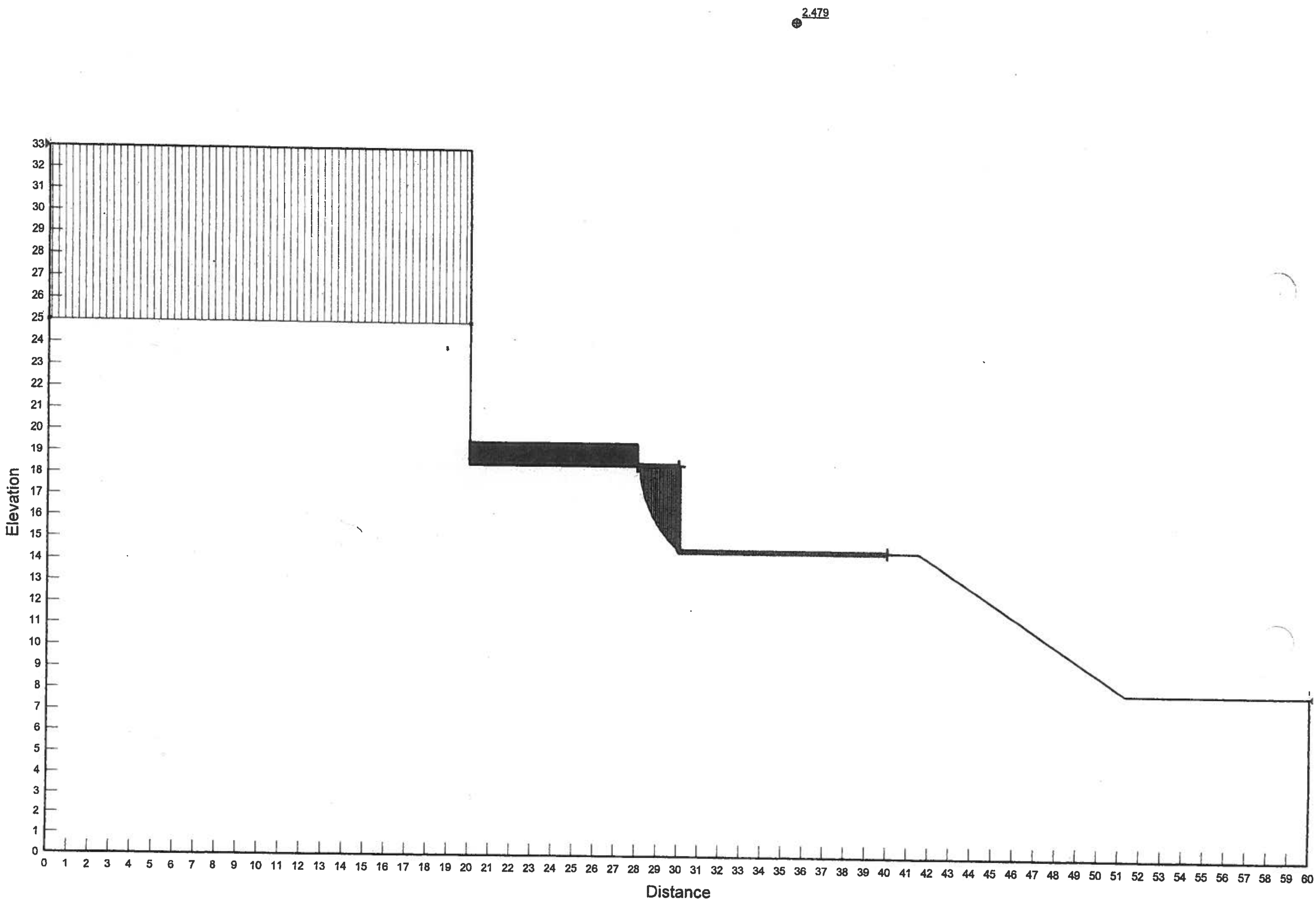
Section C-C' Case 1 - Cut 2



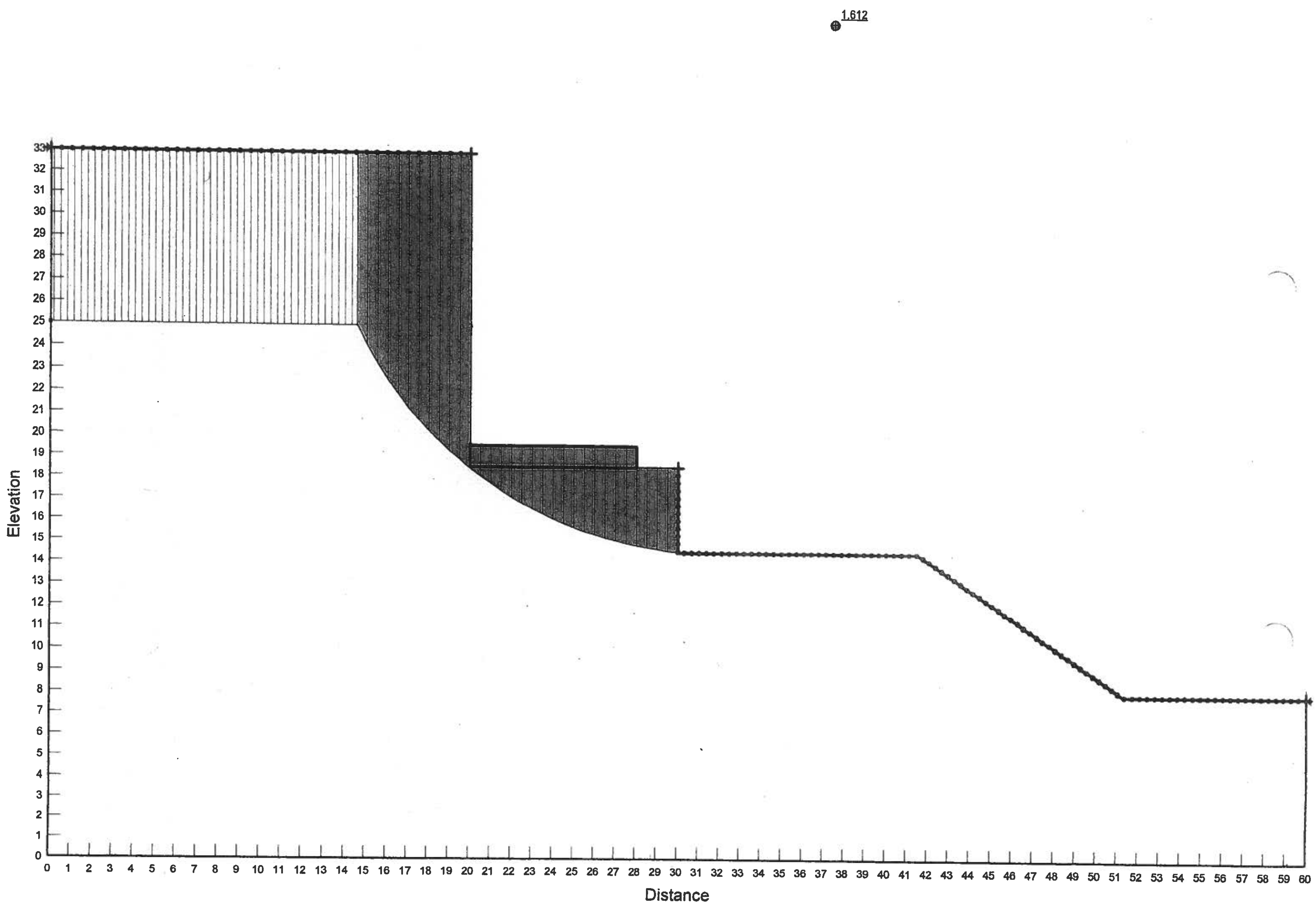
Section C-C' Case 1 - Cut 2 w/ TB


1.128
3250 LBS
3600 LBS
Elevation
Distance

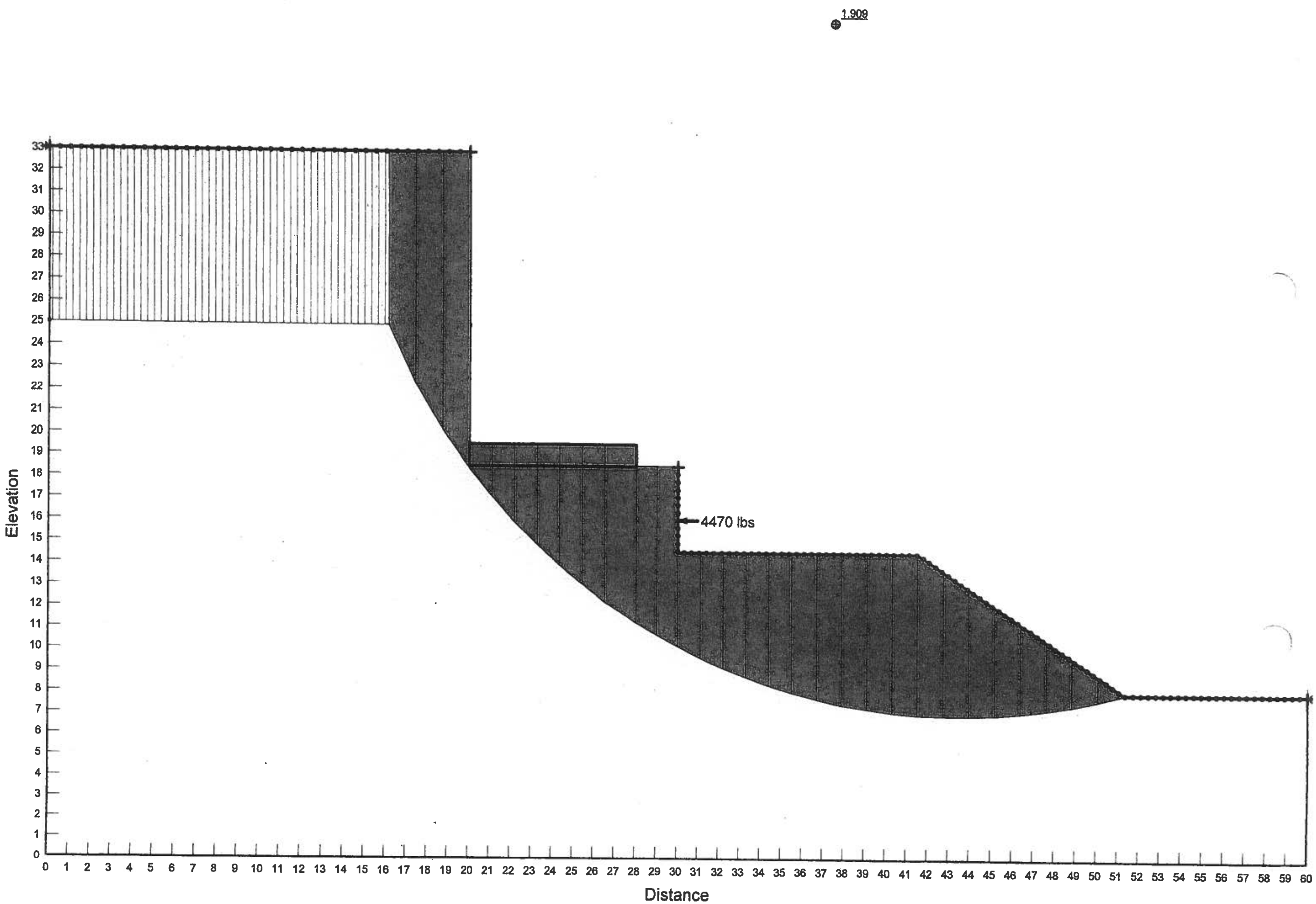

1.461
Elevation
Distance
3250 LBS
3600 LBS
4700 LBS



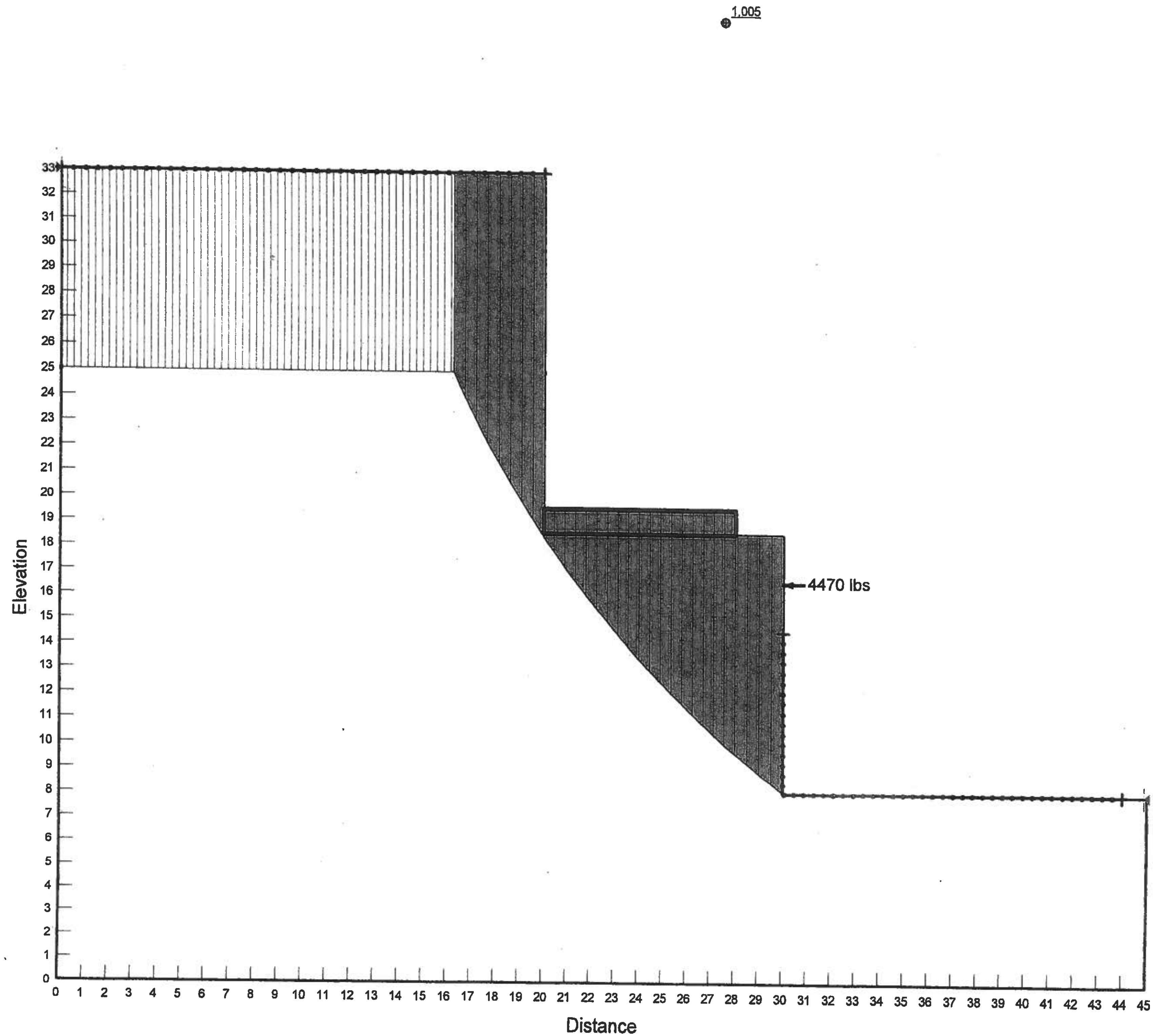
Section C-C' Case 2 - Cut 1

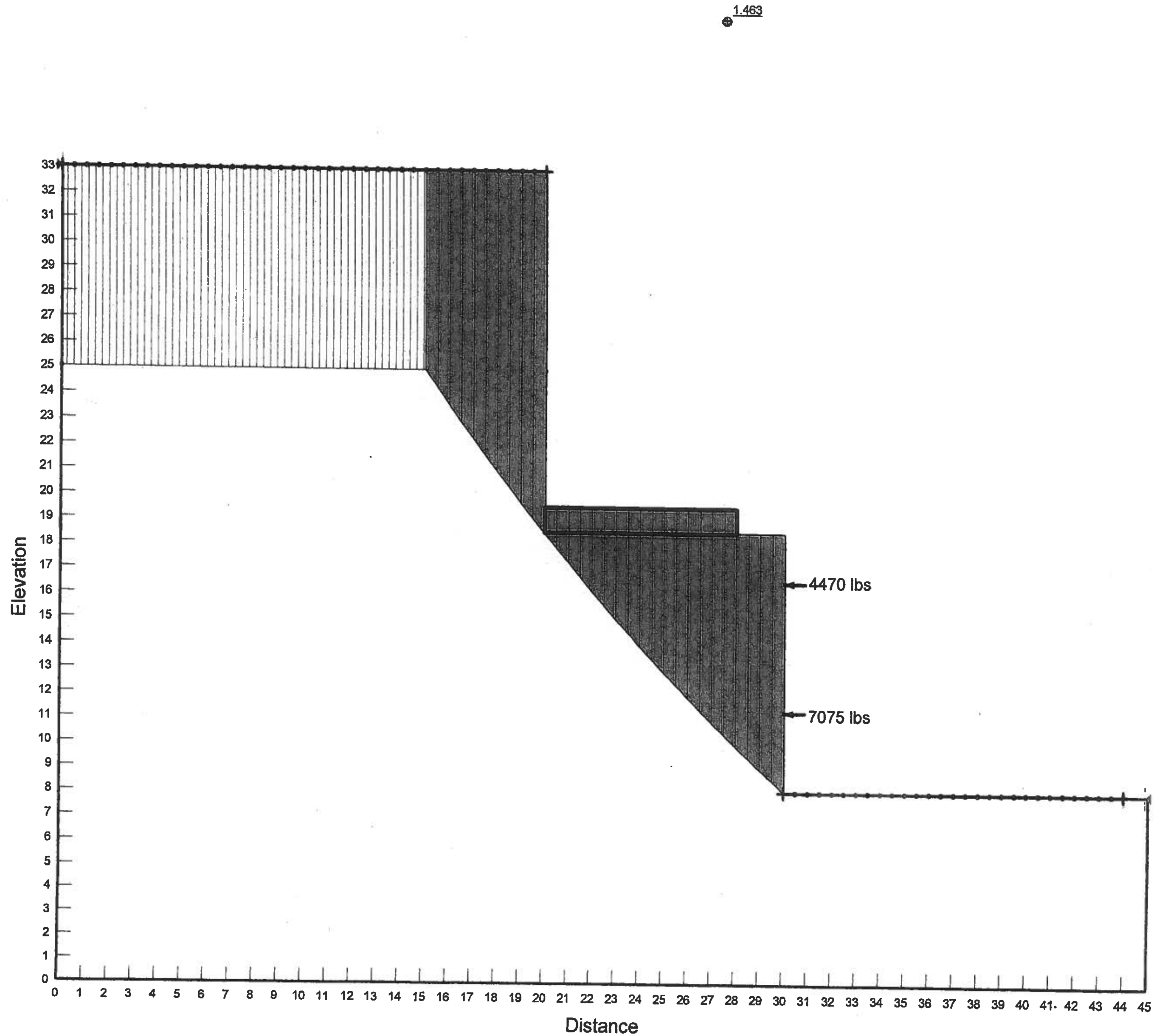


Section C-C' Case 2 - Cut 1



Section C-C' Case 2 - Cut 1 w/ TB


1.005
4470 lbs
Elevation
Distance



1.463
Elevation
Distance
4470 lbs
7075 lbs

APPENDIX G

Office of Special Funded Projects
Comment & Response Form
(Revised 01/15/13)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: _10_ EA: _0T910______ **EFIS__1000000396________** **Project Name:Fulkerth Rd tie-back wall_______** **OSFP Liaison:_Mason Leung_** **Phone:** _(209) 948-7073__ **e-mail:________**	__ **PSR/PDS (Review No.__** __ **APS/PSR (Review No.___)** __ **APS/PR (Review No.___)** __ **Type Selection** __ **65% PS&E Unchecked Details** _X_ **PS&E (Review No. 1__)** __ **Construction** __ **Other:_________**	**Reviewer Name: _John Huang_ _____** **Functional Unit: _METS/GS** **Cost Center: _59-3657** **Phone Number:227-1037 _e-mail: _______** **Date of Review: _12/10/14_** **Structure Name*: Fulkerth Rd tie-back wall___** (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
__Omni - Means, Ltd_____	_____		_______	_______

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	Foundation Plan	The minimum unbonded length needs to be at least 15 ft beyond the backside edge of existing spread footing	Noted. SD to respond.	
2	FR	15, table 6-.2-1	When it is LRFD, the bearing pressures needs to be presented according to LFRD requirement, please coordinate with SD to clarify/revise	Based on the value presented and the date (1971) of the as-built foundations plans, The allowable bearing pressure referenced is likely ASD. As such it is provide in the updated FR as the Service Limit bearing pressure.	
3	FR	18	When it is LRFD, the lateral pressures needs to be presented according to LFRD requirement, please coordinate with SD to clarify/revise, please refer to the GDR table 7.4-2	The lateral pressure in the referenced table is unfactored and the report will be updated accordingly.	
4	FR	18	Please clarify how the 4 ksf/ft for seismic incremental was developed, it seems to be small.	The seismic increment was determined by calculating the difference between the Total Seismic+ Static Force (obtained from Mononobe-Okabe with a K_h= 1/3 PGA = 0.09g) and the static Rankine Active Lateral Earth Pressure.	
5	FR		(C2) Approved subject to OSFP Verification	Noted.	
6	Note:		FR by Kleinfelder dated 9/4/14		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(for Reviewer's use)

Submittal Data (Reviewer to complete)

Dist-EA:_______ Reviewer:_______ Str Name*:_______

Date of Review:_______ Functional Unit:_______ Br No*._____ *=if applicable

7					
8					
9					
10					
11					
12					
13					
14					
15			Grading: Cs for DBB, Ls for Local Assistance		
16					
17					
18					
19					
20					