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Engineering & Capital Projects – Water & Wastewater (AEP Div.)
525 B Street, Suite 750
San Diego, CA 92101

March 6, 2024
NOVA Project No. 2023274.1

Subject: Geotechnical Addendum and Response to City Review Comments
Stormwater Diversion at Point Loma Wastewater Treatment Plant
1902 Gatchell Road, San Diego, CA
PRJ-1084313

Dear Ms. McMullen:

NOVA Services, Inc. (NOVA) prepared this geotechnical addendum to respond to the referenced review comments from the City of San Diego (City of San Diego, 2023) for the Stormwater Diversion project at the Point Loma Wastewater Treatment Plant. Kimley-Horn and Associates (KHA) is retained by the City of San Diego, and NOVA is retained by KHA as the subconsultant to provide geotechnical assistance and effectively take over as the geotechnical consultant of record for the project.

The following plates, figures, and appendices have been attached to this document.

Plate 1	Geotechnical Map
Plate 2	Geotechnical Map
Plate 3	Geologic Cross-Sections
Appendix A	References
Appendix B	Historical Imagery
Appendix C	Slope Stability Analyses
Appendix D	Critical Geotechnical Evaluations

INTRODUCTION

The Point Loma Water Treatment Plant (hereinafter the 'Plant') was constructed in the early 1960's to serve the metropolitan San Diego region. It is considered a critical public facility. The Plant is located adjacent to the Pacific Ocean in the southwest portion of Point Loma, San Diego. The western edge of the Plant is located at the top of sensitive coastal bluffs. Geotechnical Investigations have been performed at the Plant since the late 1950's. Grading of the Plant was conducted in the 1960's and consisted of constructing a relatively flat pad which required cutting into the hillside east of the Plant and filling the western portion of the site (Appendix B shows images of the current Plant as well as sequential aerial photographs of the Plant as it was constructed).

Over the last 60 years, erosion of some of the bluffs has affected portions of the facility. As a result, the bluffs at the backs of coves received shoreline protective devices and sea caves have been infilled. The current bluff area is composed of a combination of simple bluffs and modified landform bluffs (City of San Diego, 2004). Numerous geotechnical investigations and analyses have been performed to evaluate bluff retreat rates as they relate to coastal engineering. Appendix A presents a bibliography of these investigations and a figure depicting the approximate locations of relevant investigations performed at the Plant that were provided by the City of San Diego for our review. Appendix D presents the most relevant investigations (in their entirety) regarding bluff retreat and stability.

To satisfy the requirements of a MS4 Permit (Disposal of Stormwater Run Off), the Plant has proposed to install six wet wells at various locations within the western portion of the Plant to collect stormwater. The collected stormwater will be pumped to the eastern portion of the site via new, dedicated storm drain pipelines, so it can be treated and mixed with wastewater. The wet wells will not be occupied except for maintenance (i.e., less than 4,000 man-hours per year). The improvements are not considered crucial to the operation of the Plant. The facilities are in their proposed locations because stormwater flows downhill. The locations are at the lowest elevation within their particular drainage basins. They cannot be moved and still function as intended.

SCOPE OF WORK

A geotechnical investigation for the proposed facilities was prepared on September 29, 2020, by Atlas Engineering West Inc. (Atlas, 2020). This report was the subject of the Development Services Department-Geology (DSD-Geology) comments. The following document is a response to those comments.

To respond to the comments by DSD-Geology, we have performed the following tasks:

1. Review of published geologic maps and reports regarding Point Loma geology, seismology, coastal bluff formation and erosion, and predicted sea level rise.
2. Review of previous exploratory logs in the study area to create cross-sections.
3. Review of stereo aerial photographs and Google Earth images.
4. Observation and filming of storm conditions along the bluffs during an El Niño event at the height of a Perigean Tide.
5. Geologic mapping of the western portion of the Plant.
6. Preparation of four geologic cross-sections for use in evaluating slope stability, and the geologic map showing site geology, hybrid bluff top, modified bluff top, and 40-foot setback lines.
7. Evaluation of predicted sea level changes in combination with Perigean Tides (King Tides), El Niño storm surge, and wave set-up.

8. Geotechnical analyses to prepare responses to DSD-Geology comments.

It is our opinion, based on our analyses, that the proposed facilities will not be adversely impacted by fault ground rupture. The proposed facilities will not adversely affect slope or bluff stability. Coastal buff retreat will not likely impact the proposed facilities due to existing shoreline protection devices. Projected sea level rise may affect the life of the existing shoreline protection devices, but we assume that the devices will be enhanced over time to protect the essential facilities at the Plant which, in turn, will protect the proposed non-essential facilities.

RESPONSE TO DSD-GEOLOGY COMMENTS

The review comments and NOVA's responses are provided below.

<u>Comment 00018</u>: *The project's geotechnical consultant must submit a geotechnical addendum or update letter for the purpose of an environmental review that specifically addresses the proposed development plans and the following:*

<u>Response</u>: This letter serves as the requested geotechnical addendum for the purposes of a geotechnical review, with the referenced proposed development plans.

<u>Comment 00019</u>: *Per the City's Guidelines for Geotechnical Reports, the geotechnical investigation report must contain a geologic/geotechnical map that shows the distribution of fill and geologic units, location of exploratory excavations, and location of cross-sections. The map should be on the current topographic base that shows the proposed development.*

<u>Response</u>: See Plates 1 and 2.

<u>Comment 00020</u>: *Circumscribe the limits of anticipated remedial grading on the geologic/geotechnical map to delineate the proposed footprint of the project.*

<u>Response</u>: No remedial grading is anticipated or recommended. The footprints of the proposed facilities are shown on Plates 1 and 2.

<u>Comment 00021</u>: *The project is located in Geologic Hazard Category (GHC) 53 as shown on the City's Seismic Safety Study Geologic Hazard Maps and is characterized by sloping terrain, unfavorable geologic structure, and variable slope stability. The geotechnical consultant must indicate if the geologic structure at the site is favorable or unfavorable with respect to slope stability at the site.*

<u>Response</u>: The Point Loma Formation dips into slope (See Plates 1 and 2) and the overlying old paralic deposits are flat lying. The geologic structure is favorable at the site with respect to slope stability. Slope stability at the site is controlled by slope steepness and bluff erosion. It is our opinion that the proposed facilities will not affect bluff stability nor will bluff stability affect the facilities (See Plate 3 and Appendix C). The vault and wet well locations are in areas protected by existing shoreline protection devices or are sufficiently landward of a hybrid top of bluff (See Plates 1 and 2).

<u>Comment 00022</u>: *The project's geotechnical consultant must provide a professional opinion that the site will have a factor-of-safety of 1.5 or greater for both gross and surficial stability following project*

completion. If necessary, provide recommended mitigation measures to reduce the geotechnical impacts from slope instability to a level of insignificance.

Response: NOVA performed slope stability analyses of Geologic Cross-Sections A-A', B-B', C-C', and D-D' as shown on Plate 3. Analyses were performed using SLIDE2 V9.019 software to calculate factors of safety (FS) against slope failure using Spencer's method, which satisfies both force and moment equilibrium. Five different materials were defined to represent the subsurface conditions. The materials include existing fill, colluvium, old paralic deposits, Point Loma Formation, and previously placed rip rap. The properties of the materials selected for the analyses are listed in Table 1.

Table 1. Strength Parameters for Slope Stability Analyses

Material Name	Unit Weight (pcf)	Cohesion (psf)	Fricton Angle (deg)
Fill (Qf)	120	100	30
Rip Rap (Qr)	130	0	50
Colluvium (Qcol)	120	0	30
Old Paralic Deposits (Qop)	128	600	35
Point Loma Formation (Kp)	130	1500	45

The shear strength parameters were derived from previous laboratory test results obtained during previous geotechnical investigations and our experience with similar materials on previous projects in the site area. The groundwater level was taken at an elevation of 0 feet above mean seal level (MSL).

NOVA's slope stability analyses consisted of evaluating the static factors of safety of the existing and proposed configurations of Cross-Sections A-A', B-B', C-C', and D-D'. The pseudostatic factors of safety of the proposed slope configurations were also evaluated. A seismic coefficient (k_h) of 0.15 was used for the pseudostatic case. The results of the analyses are summarized in Table 2.

Table 2. Slope Stability Analyses Results

Geologic Cross-Section	Existing Condition	Proposed Condition	
	Static Factor of Safety	Static Factor of Safety	Pseudostatic Factor of Safety
A-A'	1.99	1.99	1.65
B-B'	3.08	3.04	2.25
C-C'	1.68	1.68	1.52
D-D'	1.56	1.56	1.29

Factors of safety of 1.5 and 1.25 or greater, respectively, are generally considered acceptable in geotechnical practice within the City of San Diego for static and pseudostatic conditions.

Comment 00023: *The project site is also located in GHC 12 as shown on the City's Seismic Safety Study Geologic Hazard Maps. GHC 12 is a fault buffer zone characterized by potentially active, inactive, or activity unknown faults with a low to moderate risk. Provide an explicit opinion whether or not an "active" or "potentially active" fault trace passes beneath the proposed construction. The opinion must be supported by adequate data.*

Response: See Plates 1 and 2. There are no active or potentially active faults beneath the proposed vault and wet well sites (City of San Diego, 2018, p.39). Alfred Carsola, a geological consultant, observed faults in the coastal bluffs during an investigation performed at the Plant prior to construction (Carsola, 1958, Appendix D). The bluffs had not been altered and the faults depicted on the City of San Diego's Geotechnical Hazards Maps were observed not to offset the 120,000- to 80,000-year-old Nestor Terrace. Carsola considered the faults to be pre-Pleistocene and likely pre-Quaternary. No geotechnical document prepared for the Plant has stated that faulting at the site is anything but pre-Pleistocene.

Regional mapping (Kennedy, 1975) shows the faults to be buried beneath the late Pleistocene old paralic deposits (f.k.a. Bay Point Formation). Similar faults in northern Point Loma are mapped as buried beneath the Quaternary very old paralic deposits (f.k.a. Lindavista Formation). The faults are antithetic to the current (Holocene) faults in the region and are relatively discontinuous.

The subject faults were exposed in areas that have now been filled or covered in shoreline protection devices. Artificial exposures such as trenches cannot be made due to the presence of underground utilities and other improvements. Geophysical and CPT soundings are also impossible due to utilities and improvements. Given the evidence provided here-in, and the fact that the planned construction does not consist of habitable structures, it is our opinion that site specific fault rupture investigations are unwarranted.

Comment 00024: *If faulting is discovered on the project site, it must be evaluated and analyzed for activity level. The project's geotechnical consultant must indicate whether or not the fault hazard presents a potentially significant geotechnical effect on the proposed development.*

Response: The fault hazard does not present a potentially significant geotechnical effect on the proposed development. See response to Comment 00023.

Comment 00025: *If a fault hazard is discovered, the consultant should indicate if project features will reduce the potential hazard to a level of insignificance or recommend additional mitigation measures.*

Response: There is no active faulting suspected beneath the proposed facilities or the Plant. Should an unknown, new fault rupture the proposed facilities, the facilities can be repaired or replaced without affecting the Plant operations, which reduces the hazard to insignificance.

Comment 00026: *If necessary, the project's professional geologist must recommend an appropriate structural setback if faults are determined to be a potentially significant geotechnical effect on the proposed development that cannot be mitigated by any other means.*

Response: No setback is recommended.

Comment 00027: *The project site is also located in GHC 43 and 44 as shown on the City's Seismic Safety Study Geologic Hazard Maps and is characterized by a generally unstable to moderately stable coastal bluff. The project's geotechnical consultant must provide a site-specific coastal bluff determination.*

Response: See Plates 1 and 2. The proposed facilities are spread over most of the western portion of the Plant. Therefore, a bluff top that transverses much of the western portion of the Plant was established. The bluffs at the Plant range from simple to modified landform bluffs (City of San Diego, 2004). The depicted bluff top can be considered a hybrid bluff top because of the varying conditions.

For simple bluffs, 1953 aerial photographs (Appendix B) were used to establish a bluff top based on an easily visible vegetation line. Field observations at the Plant show that coastal shrubs and grass will grow lushly on the gently sloping terraces underlain by old paralic deposits. Where the slope begins to steepen seaward of the bluff top, erosion eliminates soil that supports the vegetation. The simple bluff tops were drawn in accordance with Diagram III-1 of the referenced City of San Diego Coastal Bluffs and Beaches Guidelines (SD, 2004).

Modified landform bluffs consist of shoreline protection devices consisting of riprap and retaining walls. Aerial photographs (USDA, 1953) and field observations were used to establish the bluff top in accordance with Diagrams III-3 and III-4 (SD, 2004).

Sea caves and collapsed sea caves have been filled. Bluff tops were established from as-built plans and Diagram III-5 (SD, 2004).

Comment 00028: *In addition to the standard geologic information, the geologic/geotechnical map should show details of coastal landforms features pertinent to the study of coastal bluffs and clearly show the location of the coastal bluff edge that has been accurately located in accordance with the City's Coastal Bluffs and Beaches Guidelines.*

Response: See response to Comment 00027. Given the type of facilities being proposed and the heavily modified landforms they will be constructed on or near, the evaluation of bluff retreat using City of San Diego Coastal Bluffs and Beaches Guidelines (SD, 2004) seems inappropriate. It is our opinion that the top of the engineered protection be considered a modified bluff top in modified landform areas.

Comment 00029: *Three cross sections are typically required for coastal bluff sites. The cross sections are typically located at the property lines or at the limits of the proposed development and at least one intermediate cross section aligned orthogonal to the bluff edge. Show the distribution of geologic units, geologic structure, and coastal landforms features on the cross sections.*

Response: See Plate 3.

Comment 00030: *The project's geotechnical consultant must address the site-specific coastal bluff recession rates. Provide copies of the aerial photographs or historic maps used to determine coastal bluff recession rates. Clearly show where distances were measured on the aerial photographs or historic maps.*

Response: Group Delta Consultants, Inc. (GDC, 1988a, 1988b, 1995, Appendix D) have conducted three detailed evaluations of bluff retreat rates at three locations at the Plant. They arrived at 0.5 inches per year for headlands and between 2.5 and 4.0 inches per year for coves. We agree that these rates are appropriate for simple bluffs. The proposed facilities, however, are in areas protected by shoreline protection devices (Plates 1 and 2). While GDC indicated that protection would decrease retreat rates in the long term, they did not give a revised rate. Since the protection measures are based on a 75-year life expectancy, it follows that the retreat rate behind the protection measures would be negligible in 75 years. This would apply to our hybrid bluff top (SD, 2004) and modified bluff top (determined by the top of existing protection measures).

Comment 00031: *The project's geotechnical consultant should consider conducting a Sea-Level Rise analysis in accordance with Chapter 6 of the California Coastal Commission Sea Level Rise Policy Guidance, Interpretive Guidelines for Addressing Sea Level Rise in Local Coastal Programs and Coastal Development Permits, adopted August 12, 2015.*

Response: We have used the referenced guidelines (CCC, 2015) to evaluate potential sea level rise at the proposed facilities. We are using sea level predictions for areas south of Cape Mendicino, San Diego Tidal Guage (Table G-12, OPC, 2018, Low Risk Aversion). We are using Mean Sea Level, and the North American Vertical Datum of 1988 (NAVD 88) datum.

Table 3 - Predicted Sea Level Rise

Year	Sea Level Rise (feet)
2030	0.6
2050	1.2
2100	3.6

To evaluate the impact of sea level rise on the proposed facilities, we have adjusted the mean sea level with the following extreme still water level contributors.

Table 4 - Extreme Still Water Level Contributors

King Tides	5 feet
Storm Surge	2.5 feet (GDC, 1995)
Wave Set Up	2.0 feet (GDC, 1995)
Total	9.5 feet

Table 5 - Extreme Sea Level Rise (Predicted Sea Level +9.5 Feet)

Year	Sea Level Rise (feet)
2030	10.1
2050	10.7
2100	13.1

The highest water level ever recorded at Scripps Pier (before 2015) is 7.43 feet which occurred during an El Niño storm event (Caltrans, 2015). If the Extreme Water Level Contributors are added to the existing 2.5-foot Mean Sea Level (NAVD 88), this results in a 12-foot high Extreme Sea Level, well above the highest recorded Extreme Water Level actually measured. Considering that there are only three to four King Tides per year, it is our opinion that a 7.5-foot-high water level is more appropriate for modeling a most likely extreme high-water level for the site.

The predicted sea level rise ranges with most likely extreme-water levels (7.5-foot) are presented below.

Table 6 - Most Likely Extreme Sea Level Rise

Year	Sea Level Rise Elevations (feet) (NAVD 88)
2030	8.1
2050	8.7
2100	11.1

The most recent analysis of the shoreline protection devices (GDC, 1995) used 6.3-foot maximum still water level for design. This amount is based on highest high tide, 100-year storm surge, 1-foot-high wave setup, and 0.5-foot, long-term sea-level rise. The other structures utilized a similar figure (GDC, 1988a, 1988b).

Comment 00032: *The project's geotechnical consultant should provide an analysis of the potential effects on bluff stability of rising sea levels, using latest scientific data (SDMC 143.0143(B)) and an analysis of the potential effects of past and projected El Nino events on bluff stability (SDMC 143.0143(C)). The report must also provide an analysis of whether this section of coastline is under a process of retreat (SDMC 143.0143(D)) if the proposed setback from the buff edge is less than 40 feet.*

Response: The proposed facilities are protected by existing shoreline protection devices (Plates 1 and 2). As described above, the structures are designed to withstand bluff erosion through their design life (75 years). We have discussed why we believe that a bluff top consisting of the top of the coastal engineering structures should be considered the top-of-bluff when evaluating siting of the proposed wet wells.

Based on the documentation reviewed, it is apparent that the existing coastal structures were not designed to accommodate the Extreme Sea Level Rise or Most Likely Extreme Sea Level Rise predictions (See response to Comment 00031). We do not know how the structures will react to the predicted sea level rise. However, using extreme still water and most likely extreme still water values in addition to the 2100 SLR values, the shoreline protection devices will not be over topped.

The calculated extreme sea level rises will likely only affect the lower proposed PS2C wet well and vault. The effect will likely consist of frequent flooding (wave splash) resulting in higher usage of the pump. There will be more salt water pumped through that facility as well. The other wet wells are proposed to be constructed at elevations above the calculated extreme sea level in the year 2100.

While a portion of the Point Loma bluffs are retreating, the bluffs affecting the proposed facilities are not retreating if the existing protection remains in place. We assume that review of the existing shoreline protection devices' performance will be ongoing as the structures protect existing critical facilities adjacent to the proposed wet wells. Any alteration, replacement, or enhancement to the coastal structures to protect critical facilities will, in turn, protect the proposed wet wells.

CLOSURE

NOVA appreciates the opportunity to be of service to The City of San Diego in partnership with Kimley-Horn and Associates, Inc. Should you have any questions regarding this letter or other matters, please contact the undersigned at 858.292.7575 x 417.

Sincerely,
NOVA Services, Inc.

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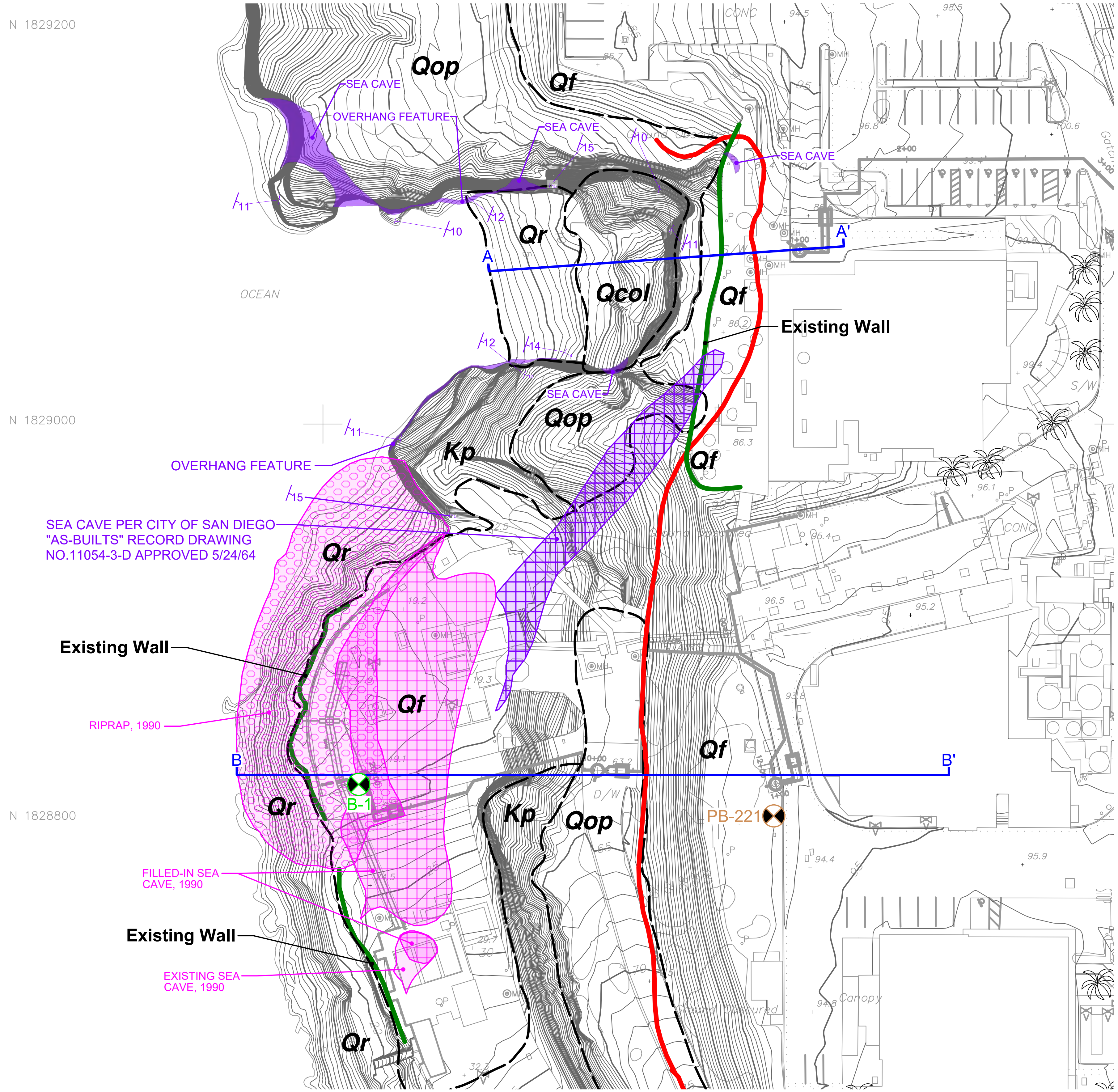
EXP. 9/30/25
GEOTECHNICAL
STATE OF CALIFORNIA

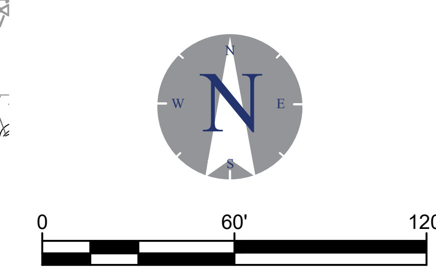
Attachments:	Plate 1	Geotechnical Map
	Plate 2	Geotechnical Map
	Plate 3	Geologic Cross-Sections
	Appendix A	References
	Appendix B	Historical Imagery
	Appendix C	Slope Stability Analyses
	Appendix D	Critical Geotechnical Evaluations

PLATES



KEY TO SYMBOLS

Qf	FILL
Qcol	COLLUVIUM
Qr	RIP RAP
Qop	OLD PARALIC DEPOSITS
Kp	POINT LOMA FORMATION
	GEOLOGIC CONTACT
	HYBRID BLUFF-TOP
	PROPOSED MODIFIED GROUND BLUFF-TOP
B B'	GEOLOGIC CROSS-SECTION
15	BEDDING ATTITUDE
PB-221	GEOTECHNICAL BORING (CWP Geosciences, 1994)
B-1	GEOTECHNICAL BORING (ATLAS, 2020)



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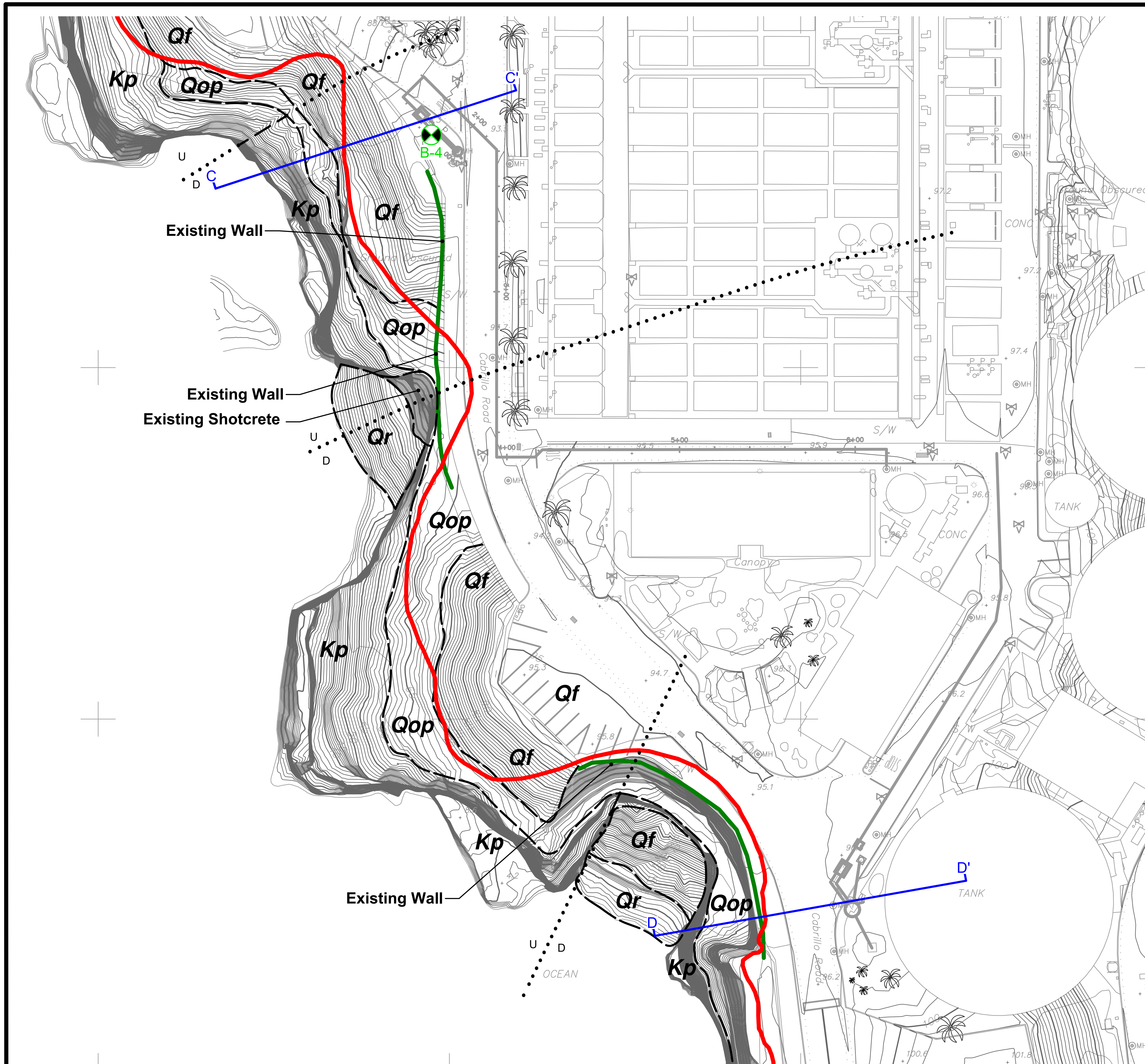
STORMWATER DIVERSIONS

POINT LOMA WASTEWATER TREATMENT PLANT
LANDS END ROAD, SAN DIEGO, CA 92106

PROJECT NO.:	2023274.1
DRAWN BY:	DJ/DB
REVIEWED BY:	LV/AN/GD
SCALE:	1"=60'
DRAWING TITLE:	

GEOTECHNICAL MAP

PLATE NO. 1 OF 3



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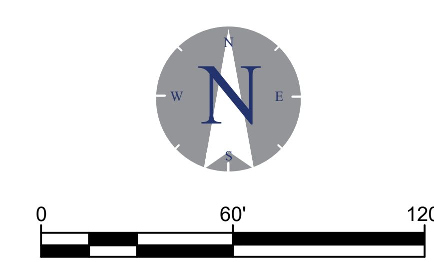
D D'	GEOLOGIC CROSS-SECTION
B-4	GEOTECHNICAL BORING (ATLAS, 2020)
	FAULT - DASHED WHERE INFERRED; DOTTED WHERE CONCEALED



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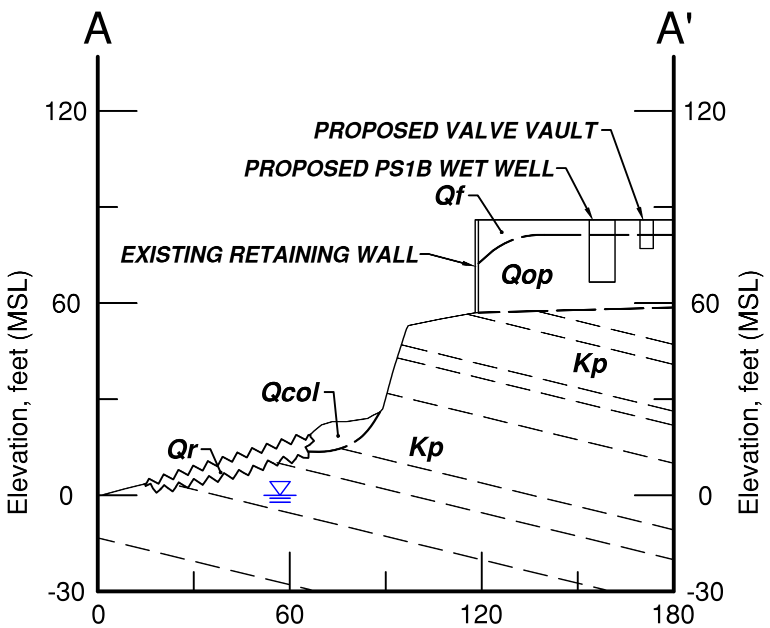
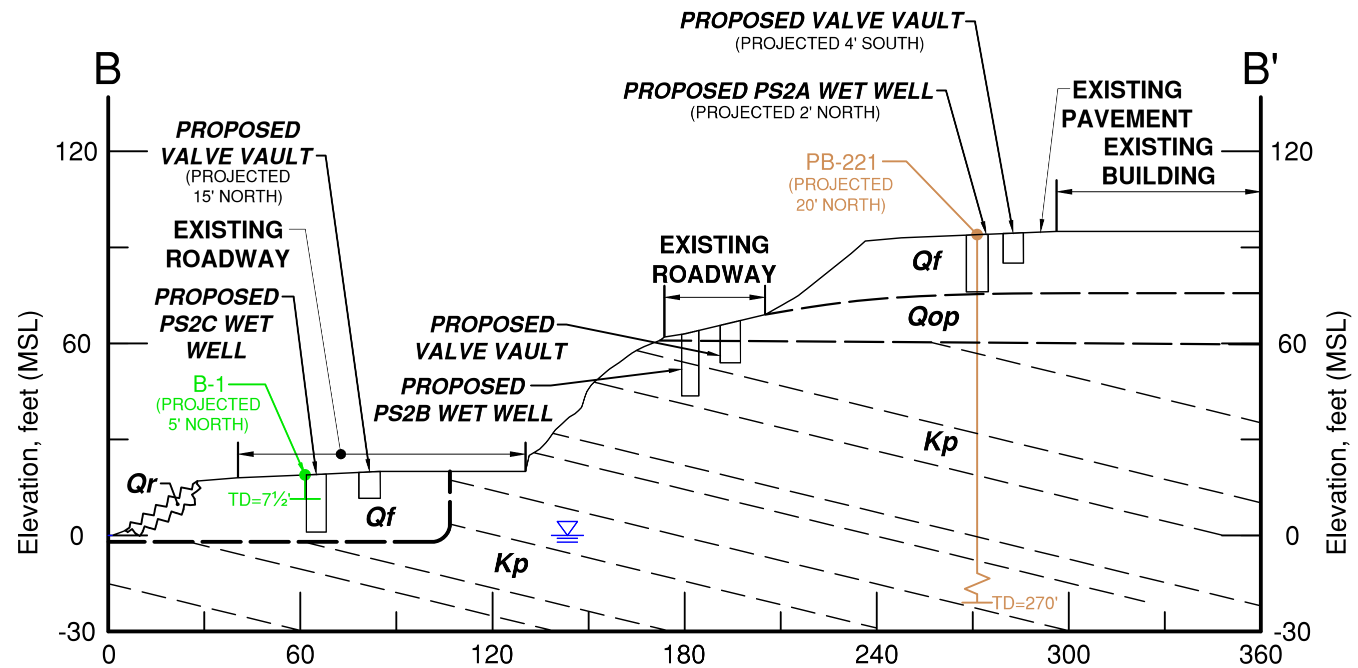
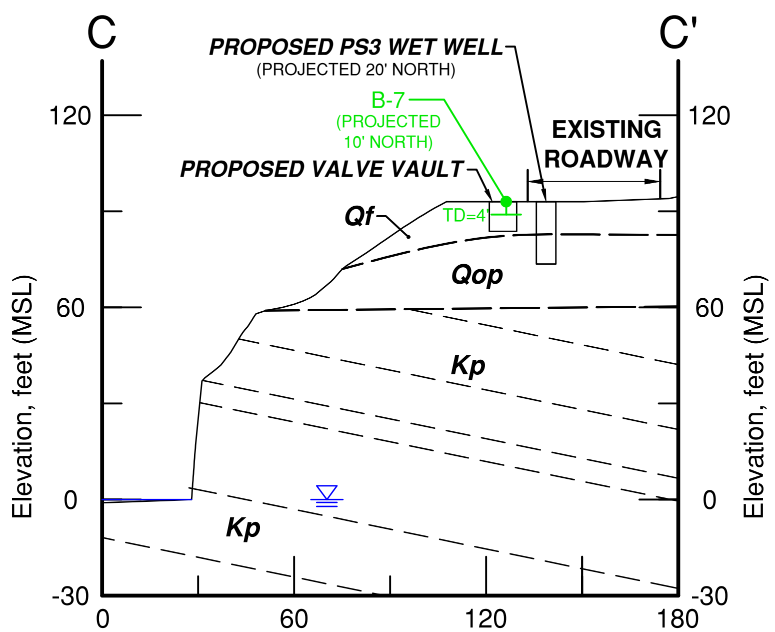
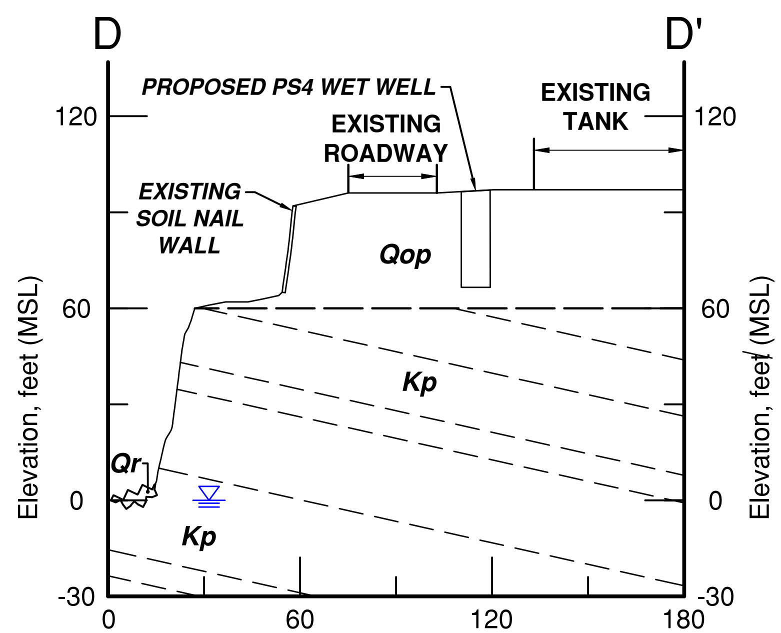
GEOTECHNICAL MAP

PLATE NO. 2 OF 3









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B-7	GEOTECHNICAL BORING (ATLAS, 2020)

0 60' 120'

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GEOLOGIC CROSS-SECTIONS

APPENDIX A
References

Atlas Engineering West Inc. (Atlas), 2020, Geotechnical Investigation, Point Loma Wastewater Treatment Plant Site Improvements, San Diego, California, Atlas No. 190329P4.3, September 29.

California Coastal Commission (CCC), 2015, Sea Level Rise Policy Guidance, Interpretive Guidelines for Addressing Sea Level Rise in Local Coastal Programs and Coastal Development Permits, Unanimously Adopted August 12.

———, 2018, Appendix G, Sea Level Rise Projections for 12 California Tide Gauges, Final Adopted Science Update, Pages 291 through 304, November 7.

California Coastal Records Project (CCRP), Website https://www.californiacoastline.org/? accessed February 2024, 1972, *Aerial Photographic Survey of the California Coastline, Oblique Aerials, Images 7242028 through 7242030.*

———, 1979, Aerial Photographic Survey of the California Coastline, Oblique Aerials, Images 7956064_13.42.00 through 79556066_13.42.00, May 3.

———, 1987, Aerial Photographic Survey of the California Coastline, Oblique Aerials, Images 8701122 through 8701125, June.

———, 2002, Aerial Photographic Survey of the California Coastline, Oblique Aerials, Images 9788_13.53.38, 9789_13.53.42, 9790_13.53.46, 9791_13.53.50, 9792_13.53.54, 9793_13.54.03, 9794_13.54.05, 9795_13.54.07, 9796_13.54.11, 9797_13.54.18, 9798_13.54.24, October 30.

———, 2013, Aerial Photographic Survey of the California Coastline, Oblique Aerials, Images 201312701_14.34.08, 201312702_14.34.09, 201312703_14.34.11, 201312704_14.34.12, 201312705_14.34.13, 201312706_14.34.15, 201312707_14.34.16, 201312708_14.34.17, 201312709_14.34.19, 201312710_14.34.20, 201312711_14.34.22, 201312712_14.34.23, September 29.

California Division of Mines and Geology, Kennedy, M.P., and Peterson, G.L., (Kennedy), 1975, Bulletin B-200.Sect B, Geology of the San Diego Metropolitan Area, California, Section B., 1:24,000.

Caltrans, 2015, North Coast Corridor Public Works Plan/Transportation and Resource Enhancement Program, District 11 and San Diego Association of Governments, Draft Amendment #1, December.

Carsola, Alfred J., Geological Consultant (Carsola), 1958, Engineering Geology of an Area on the West Side of Pt. Loma Peninsula, San Diego, California, September 29.

CWP Geosciences, 1994, Point Loma Tunnel Outfall, Field Test Boring Log PB-221 and PB-223, Drawing Nos. 50-C-567 through -570 and 50-C-575 through -576, CIP No. 40920.1, May 25.

City of San Diego, 2004, San Diego Municipal Code, Land Development Code, Coastal Bluffs and Beaches Guidelines, Amended from Resolution Number R-293254, August.

———, 2018, Guidelines for Geotechnical Reports.

———, 2023, Project Issues Report, PRJ-1084313, 1902 Gatchell Rd, San Diego, CA, Discretionary Project, November 2.

Geotechnical Consultants, Inc. (GC), 1980, Geotechnical Reconnaissance, Proposed Point Loma Wastewater Treatment Plant Expansion, Accelerated Projects, City of San Diego, California, March 6.

———, 1994, Geotechnical Investigation for Proposed Sludge Pumping Facilities, Point Loma Wastewater Treatment Plant, City of San Diego, California, May 24.

Group Delta Consultants, Inc. (GDC), 1988a, Shoreline Protection Project, Point Loma Treatment Plant, San Diego, California, Resolution No. R-269683, Project No. 1089-ES01, May 6.

———, 1988b, Geotechnical Evaluation and Bluff Retreat Study, Point Loma Treatment Plant, Access Road, San Diego, California, Project No. 1106-SI01, October 26.

———, 1995, Final Preliminary Report, North Shoreline Protection Improvements, Point Loma Wastewater Treatment Plant, San Diego, California, CIP No. 46-175.0, Project No. 1601, July 3.

———, 1996, Limited Geotechnical Investigation for Navy Property Improvements, Point Loma Wastewater Treatment Plant, San Diego, California, CIP No. 46-175.0, Project No. 1601, February 15.

HDR, Inc. (HDR), 2022, *Point Loma Wastewater Treatment Plant Storm Water Diversion Project, 50% Submittal, City of San Diego, California, WBS: B-20001*, dated September 29 & 30.

Kennedy, M.P. and Tan, S.S., 2008, *Geologic Map of the San Diego 30' x 60' Quadrangle, California, California Geological Survey, Regional Geologic Map No. 3*, Scale 1:100,000.

Kimley-Horn and Associates, Inc. (KHA), 2023, *H226033 Stormwater Diversions at Point Loma WWTP (B-20001) 22CK11*, *Geotechnical Assistance Proposal,* October 18.

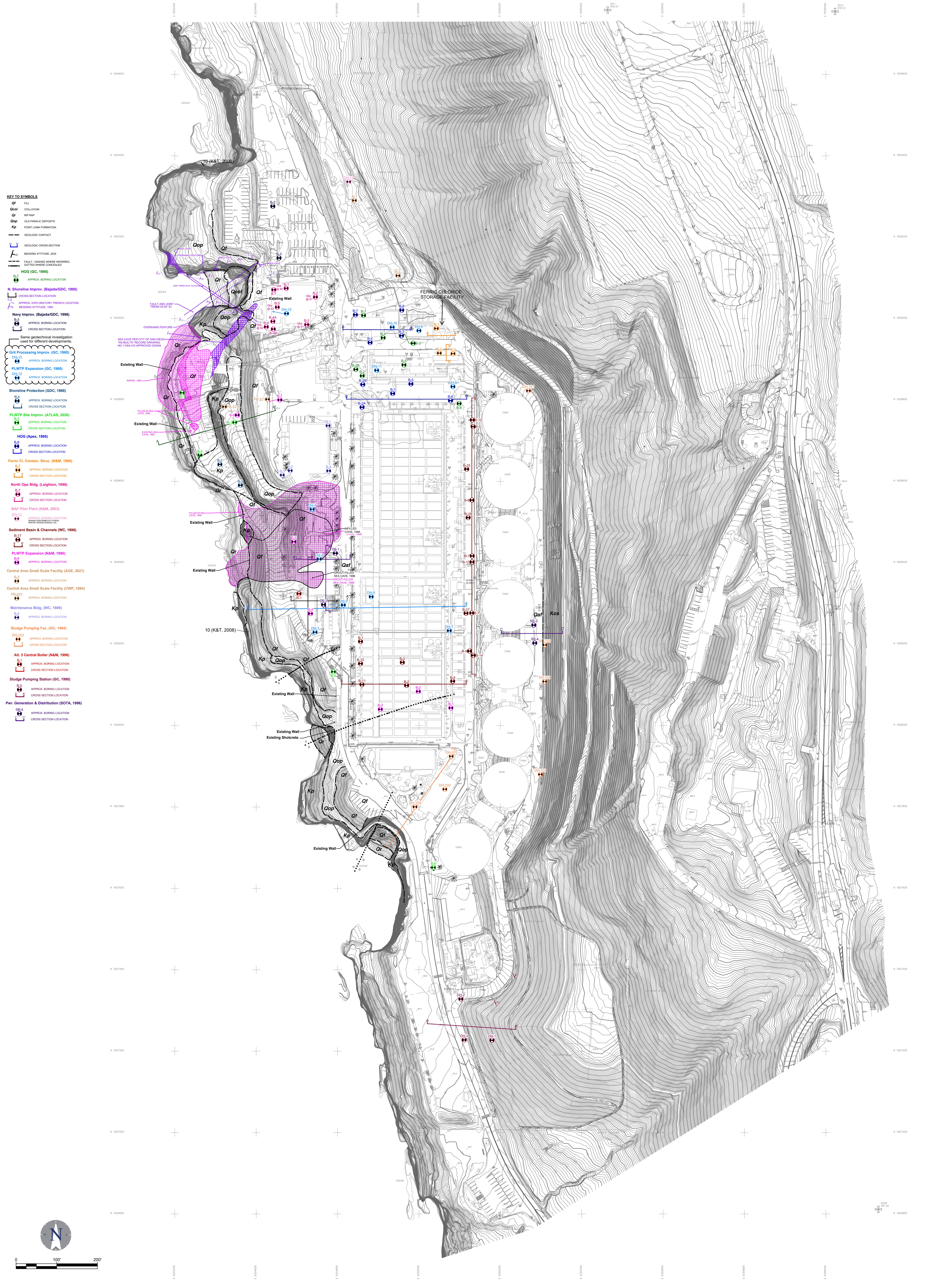
Leighton and Associates, Inc. (LA), 1996, *Geotechnical Investigation, North Operations Building, Point Loma Wastewater Treatment Plant, San Diego, California, Project No. 4950061-001,* January 18.

Milow, E. Dean, 1959, Engineering Geology Aspects of Point Loma, California, undated.

Ninyo & Moore (NM), 1990, *Geotechnical Investigation, Point Loma Waste Water Treatment Plant Expansion, 1902 Gatchell Road, San Diego, California, Project No. 101721-01,* August 30.

United States Department of Agriculture (USDA), 1953, Aerial Photograph, Point Loma Peninsula, Flight AXN, Line 7M, Frames 178 and 179, May 2.

Woodward-Clyde Consultants (WC), 1986, Geotechnical Investigation for Proposed Sedimentation Basin and Influent Channels, Point Loma Wastewater Treatment Facility, San Diego, California, Project No. 56107E-SI01, May 7.


KEY TO SYMBOLS
Qf FILL
Qcol COLLUVIUM
Qr RIP RAP
Qop OLD PARALIC DEPOSITS
Kp POINT LOMA FORMATION
GEOLOGIC CONTACT
GEOLOGIC CROSS-SECTION
BEDDING ATTITUDE, 2024
FAULT - DASHED WHERE INFERRED; DOTTED WHERE CONCEALED
HOG (GC, 1995)
APPROX. BORING LOCATION
N. Shoreline Improv. (Bajada/GDC, 1995)
CROSS-SECTION LOCATION
APPROX. EXPLORATORY TRENCH LOCATION
BEDDING ATTITUDE, 1995
Navy Improv. (Bajada/GDC, 1996)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
Same geotechnical investigation used for different developments.
Grit Processing Improv. (GC, 1980)
APPROX. BORING LOCATION
PLWTP Expansion (GC, 1980)
APPROX. BORING LOCATION
Shoreline Protection (GDC, 1988)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
PLWTP Site Improv. (ATLAS, 2020)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
HOG (Apex, 1995)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
Ferric Cl. Contain. Struc. (N&M, 1996)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
North Ops Bldg. (Leighton, 1996)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
BAF Pilor Plant (N&M, 2003)
APPROX. BORING LOCATION
*BORING PERFORMED BY OTHERS.
*REPORT MISSING BORING LOG.
Sediment Basin & Channels (WC, 1986)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
PLWTP Expansion (N&M, 1990)
APPROX. BORING LOCATION
Central Area Small Scale Facility (AGE, 2021)
APPROX. BORING LOCATION
Central Area Small Scale Facility (CWP, 1994)
APPROX. BORING LOCATION
Maintenance Bldg. (WC, 1989)
APPROX. BORING LOCATION
Sludge Pumping Fac. (GC, 1994)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
Alt. 3 Central Boiler (N&M, 1996)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
Sludge Pumping Station (GC, 1990)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
Pwr. Generation & Distribution (SOTA, 1996)
APPROX. BORING LOCATION
CROSS SECTION LOCATION
FERRIC CHLORIDE STORAGE FACILITY
10 (K&T, 2008)
Existing Wall
Existing Shotcrete
INFILLED COVE, 1996
GROUT-FILLED SEA CAVE, 1996
Qaf
Kcs
TANK
N
0
100'
200'

APPENDIX B

Historical Imagery

HISTORICAL AERIAL IMAGERY

Point Loma Wastewater Treatment Plant (PLWTP)
Stormwater Diversion

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NOVA Project No. 2023274.1

NOVA Services

4373 Viewridge Avenue
Suite B
San Diego, California 92123
858.292.7575

944 Calle Amanecer
Suite F
San Clemente, CA 92673
949.388.7710

usa-nova.com

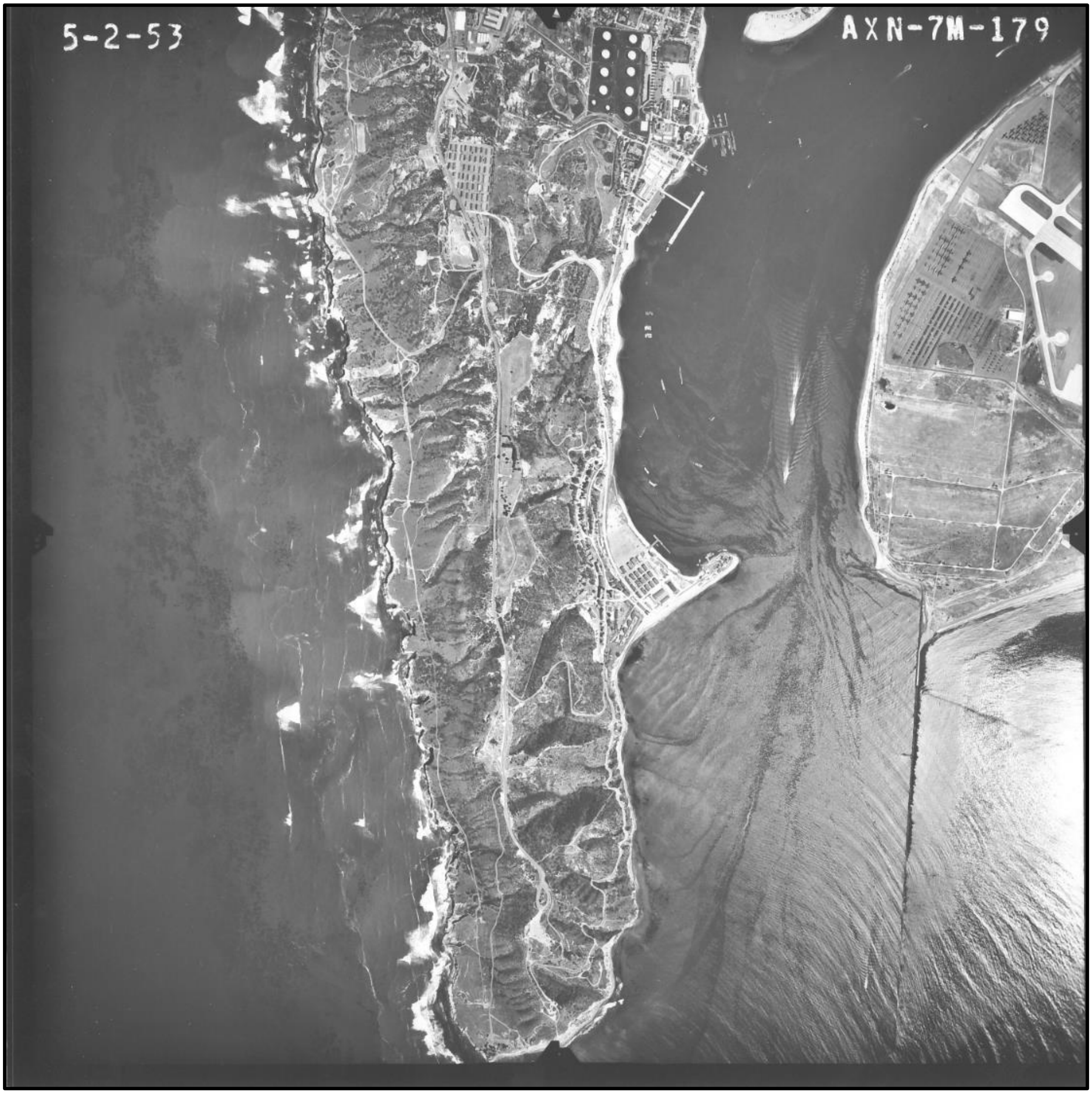


Figure B-1.1. Aerial, May 1953

Figure B-2.1. Oblique Aerial, 1972 (N32 40.53, W117 15.19)

Figure B-2.2. Oblique Aerial, 1972 (N32 40.75, W117 15.26)

Figure B-2.3. Oblique Aerial, 1972 (N32 40.93, W117 15.32)

Figure B-3.1. Oblique Aerial, May 1979 (N32 40.60, W117 15.21)

Figure B-3.2. Oblique Aerial, May 1979 (N32 40.75, W117 15.26)


Figure B-3.3. Oblique Aerial, May 1979 (N32 40.92, W117 15.31)

Figure B-4.1. Oblique Aerial, June 1987 (N32 40.52, W117 15.19)

Figure B-4.2. Oblique Aerial, June 1987 (N32 40.65, W117 15.22)

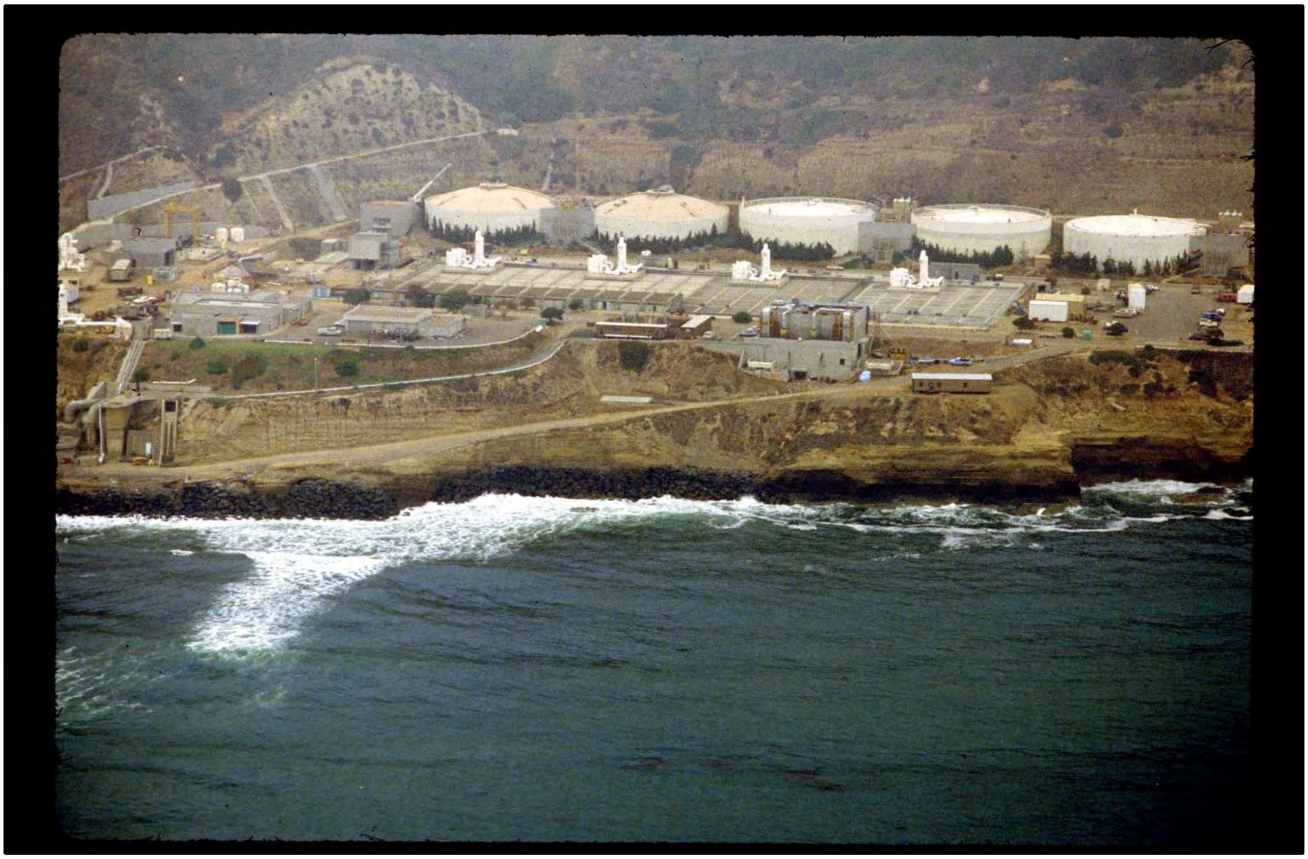


Figure B-4.3. Oblique Aerial, June 1987 (N32 40.77, W117 15.27)

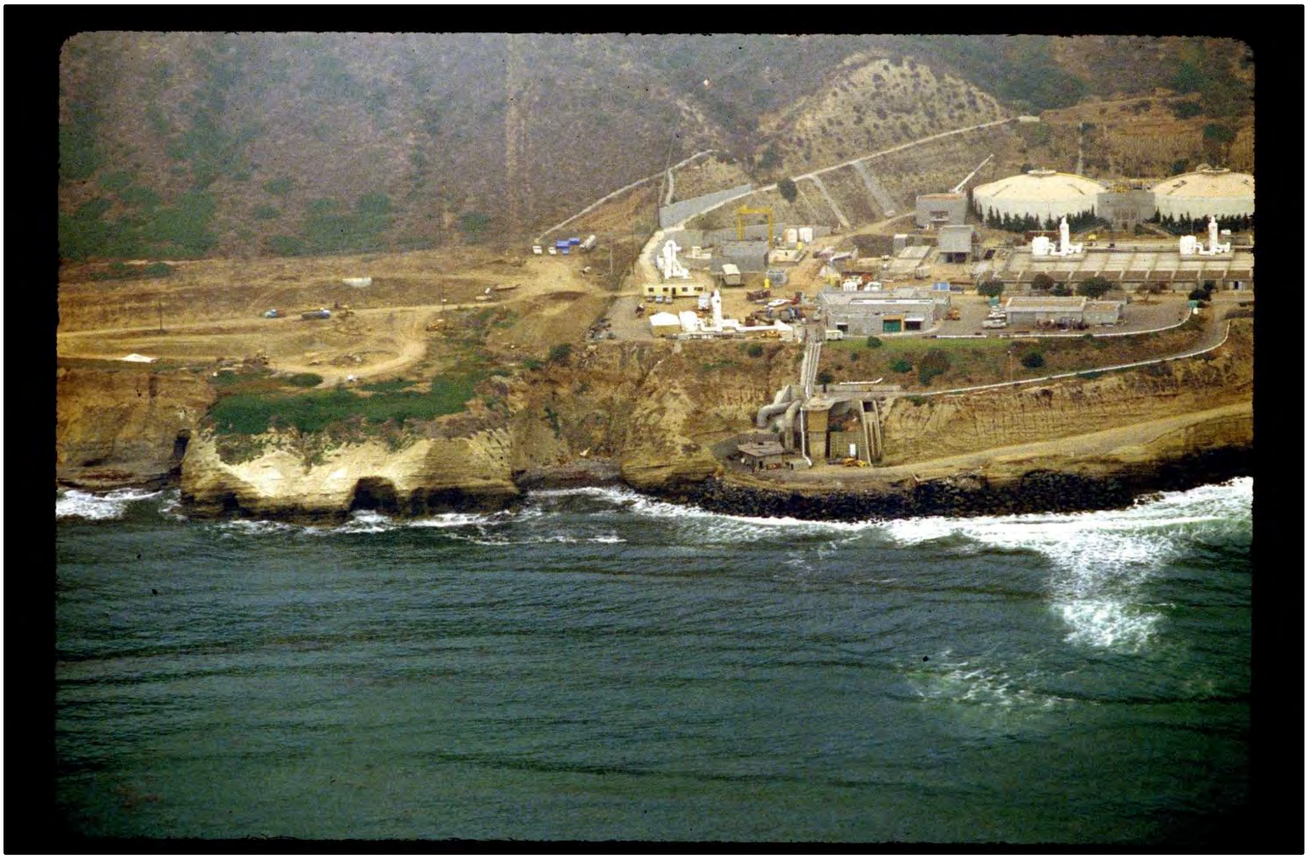


Figure B-4.4. Oblique Aerial, June 1987 (N32 40.89, W117 15.30)

Figure B-5.1. Oblique Aerial, October 2002 (N32 40.56, W117 15.20)

Figure B-5.2. Oblique Aerial, October 2002 (N32 40.63, W117 15.22)

Figure B-5.3. Oblique Aerial, October 2002 (N32 40.71, W117 15.24)

Figure B-5.4. Oblique Aerial, October 2002 (N32 40.75, W117 15.26)

Figure B-5.5. Oblique Aerial, October 2002 (N32 40.77, W117 15.27)

Figure B-5.6. Oblique Aerial, October 2002 (N32 40.79, W117 15.27)

Figure B-5.7. Oblique Aerial, October 2002 (N32 40.87, W117 15.30)

Figure B-5.8. Oblique Aerial, October 2002 (N32 40.91, W117 15.31)

Figure B-5.9. Oblique Aerial, October 2002 (N32 40.95, W117 15.32)

Figure B-5.10. Oblique Aerial, October 2002 (N32 40.99, W117 15.34)

Figure B-5.11. Oblique Aerial, October 2002 (N32 41.03, W117 15.35)

Figure B-6.1. Oblique Aerial, September 2013 (N32 40.55, W117 15.07)

Figure B-6.2. Oblique Aerial, September 2013 (N32 40.58, W117 15.08)

Figure B-6.3. Oblique Aerial, September 2013 (N32 40.64, W117 15.10)

Figure B-6.4. Oblique Aerial, September 2013 (N32 40.67, W117 15.11)

Figure B-6.5. Oblique Aerial, September 2013 (N32 40.73, W117 15.12)

Figure B-6.6. Oblique Aerial, September 2013 (N32 40.76, W117 15.13)

Figure B-6.7. Oblique Aerial, September 2013 (N32 40.79, W117 15.14)

Figure B-6.8. Oblique Aerial, September 2013 (N32 40.85, W117 15.16)

Figure B-6.9. Oblique Aerial, September 2013 (N32 40.87, W117 15.17)

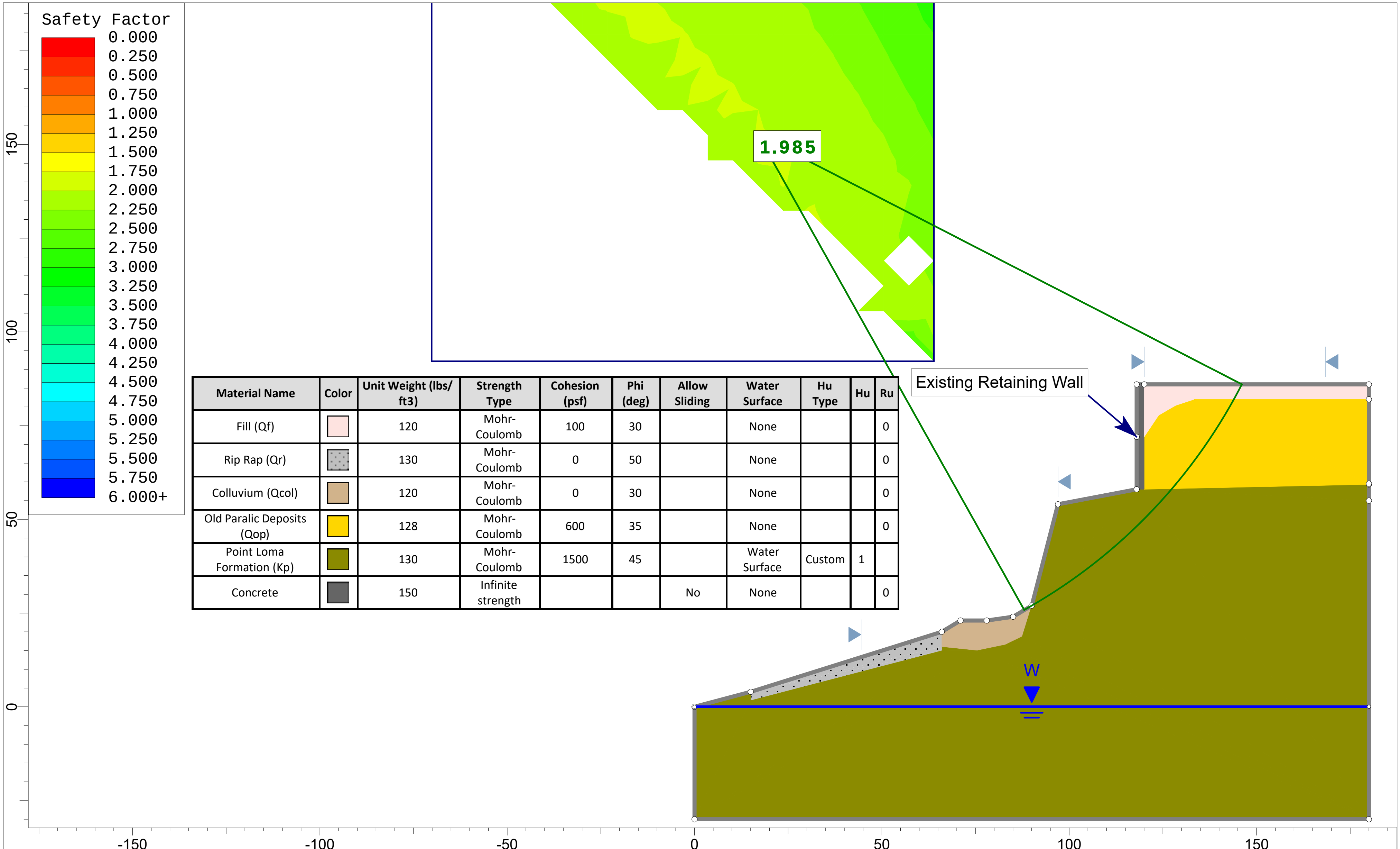
Figure B-6.10. Oblique Aerial, September 2013 (N32 40.90, W117 15.18)

Figure B-6.11. Oblique Aerial, September 2013 (N32 40.96, W117 15.19)

Figure B-6.12. Oblique Aerial, September 2013 (N32 40.99, W117 15.20)

APPENDIX C

Slope Stability Analyses

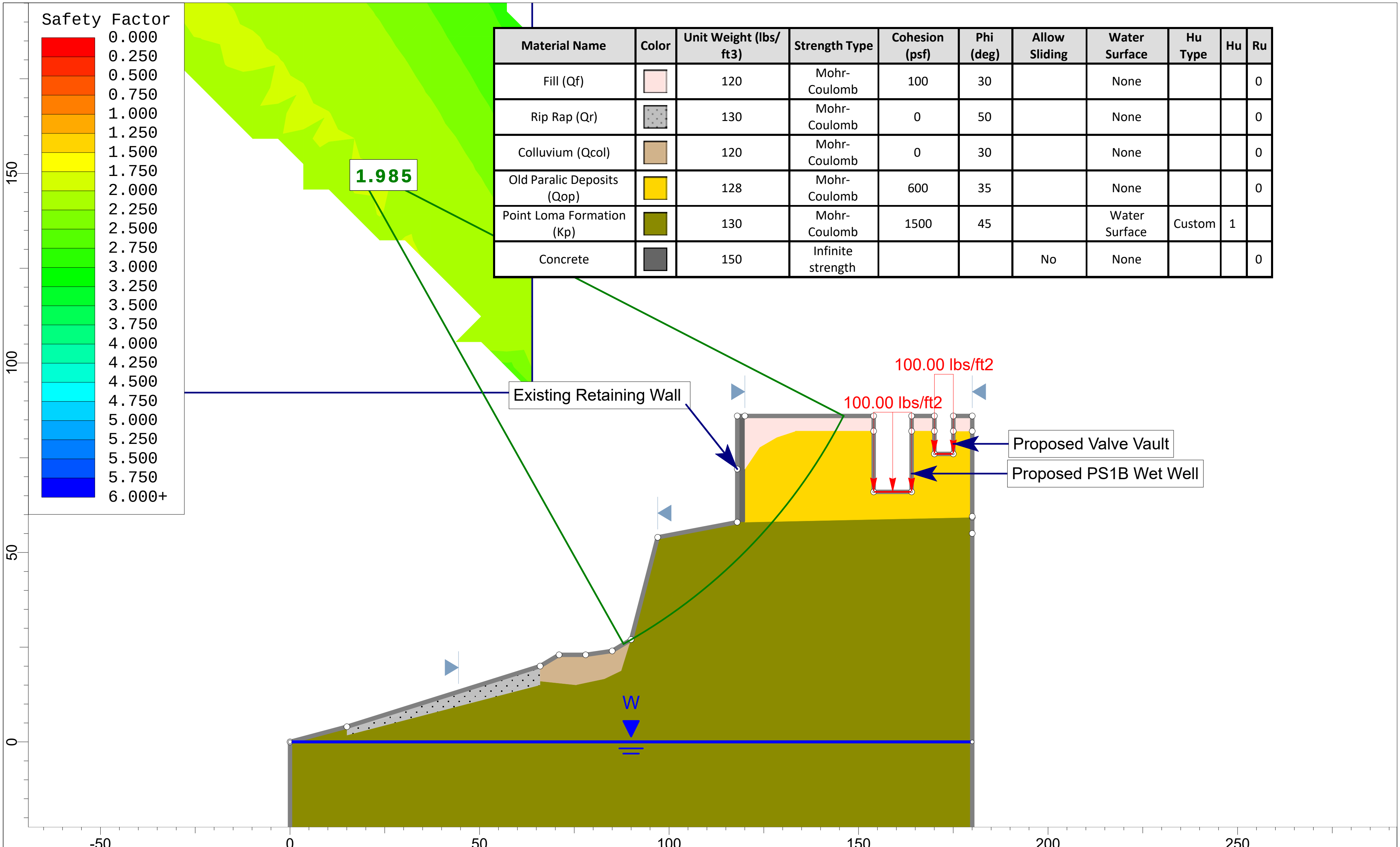


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30		None			0
Rip Rap (Qr)		130	Mohr-Coulomb	0	50		None			0
Colluvium (Qcol)		120	Mohr-Coulomb	0	30		None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35		None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45		Water Surface	Custom	1	
Concrete		150	Infinite strength			No	None			0

NOVA Services

Cross Section A-A': Existing Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.1



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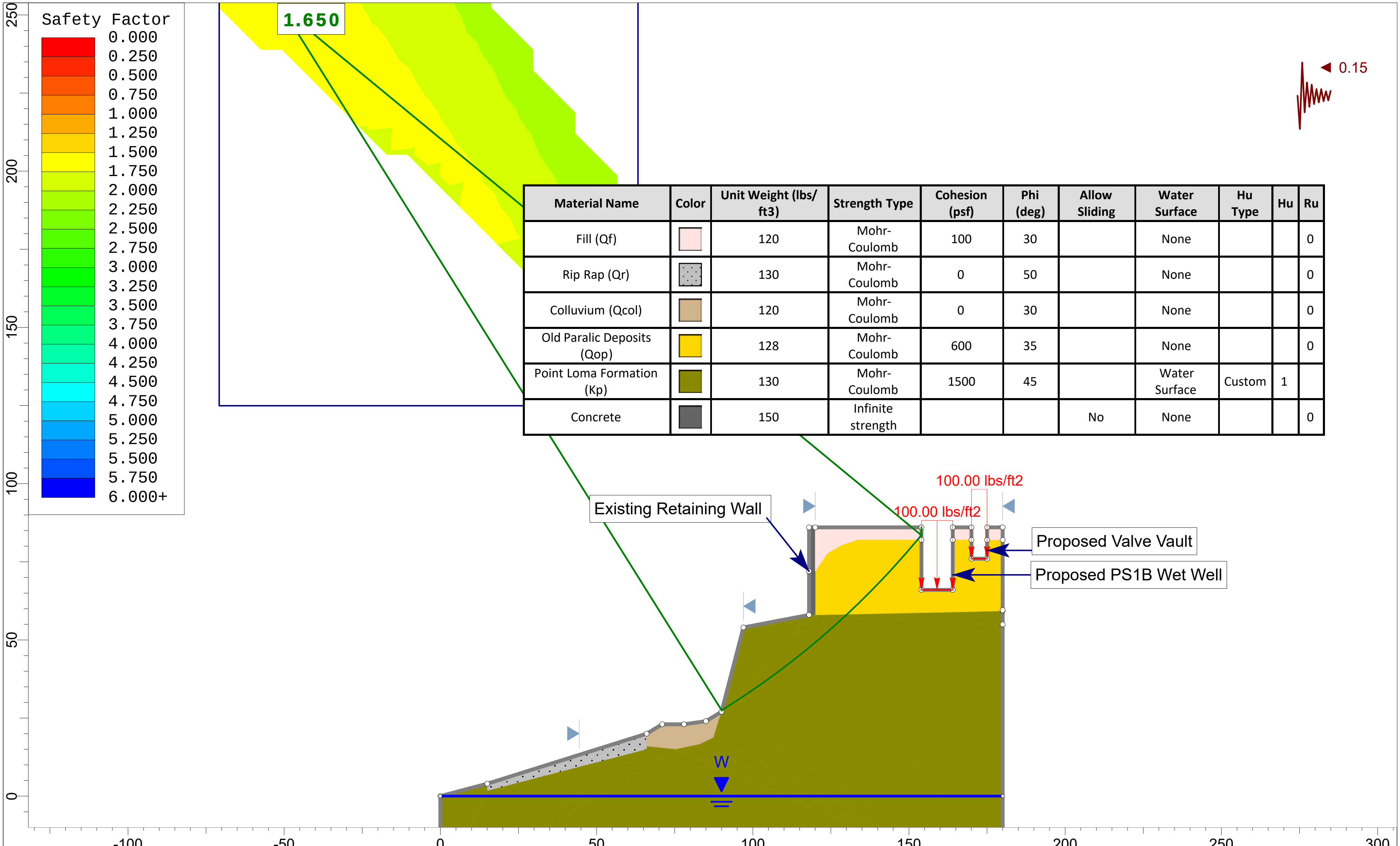
Fill (Qf)		120	Mohr-Coulomb	100	30		None			0
Rip Rap (Qr)		130	Mohr-Coulomb	0	50		None			0
Colluvium (Qcol)		120	Mohr-Coulomb	0	30		None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35		None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45		Water Surface	Custom	1	
Concrete		150	Infinite strength			No	None			0

NOVA Services

Cross Section A-A': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30		None			0
Rip Rap (Qr)		130	Mohr-Coulomb	0	50		None			0
Colluvium (Qcol)		120	Mohr-Coulomb	0	30		None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35		None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45		Water Surface	Custom	1	
Concrete		150	Infinite strength			No	None			0

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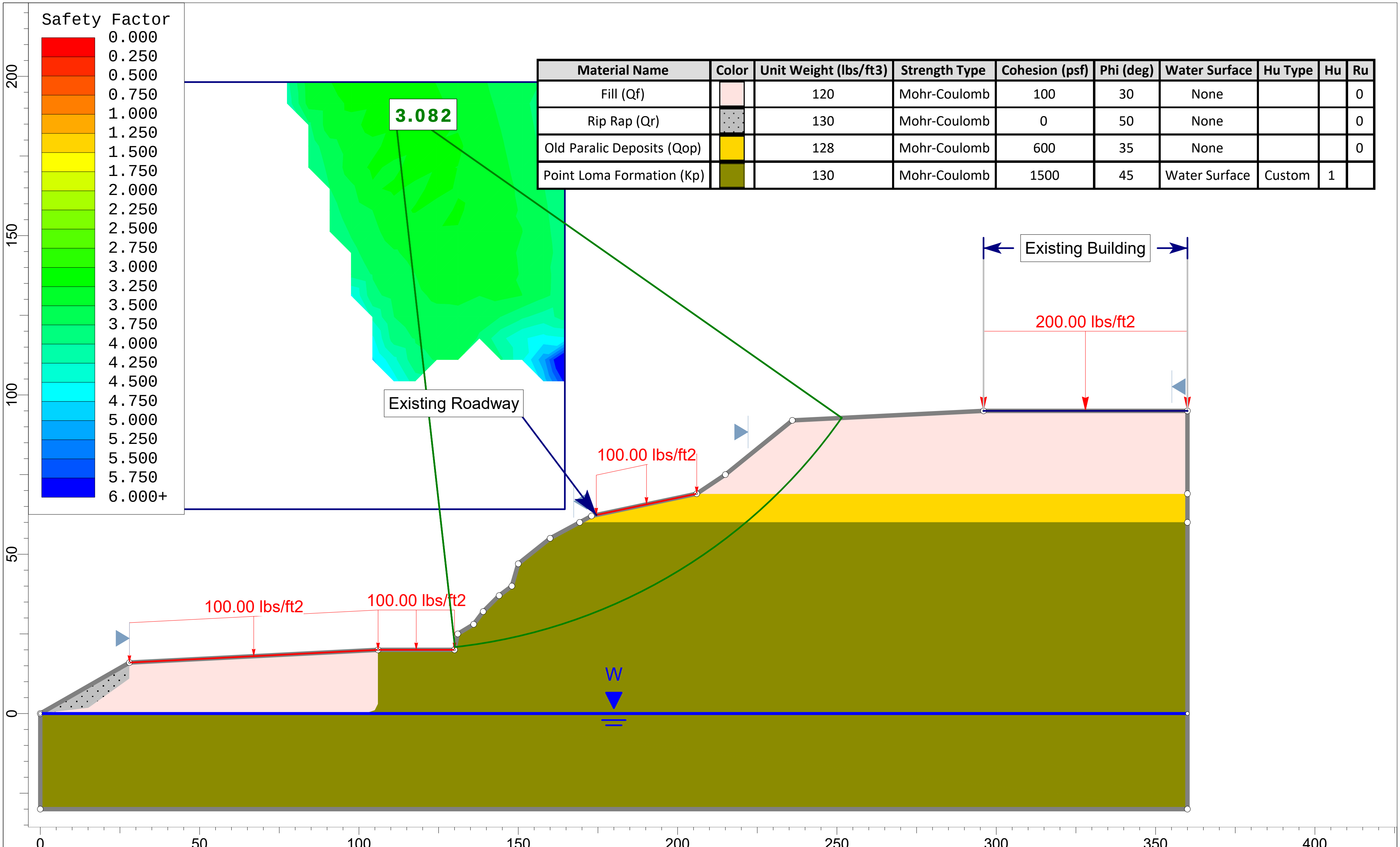
NOVA Services

Cross Section A-A': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Pseudostatic Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30	None			0
Rip Rap (Qr)		130	Mohr-Coulomb	0	50	None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35	None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45	Water Surface	Custom	1	

NOVA Services

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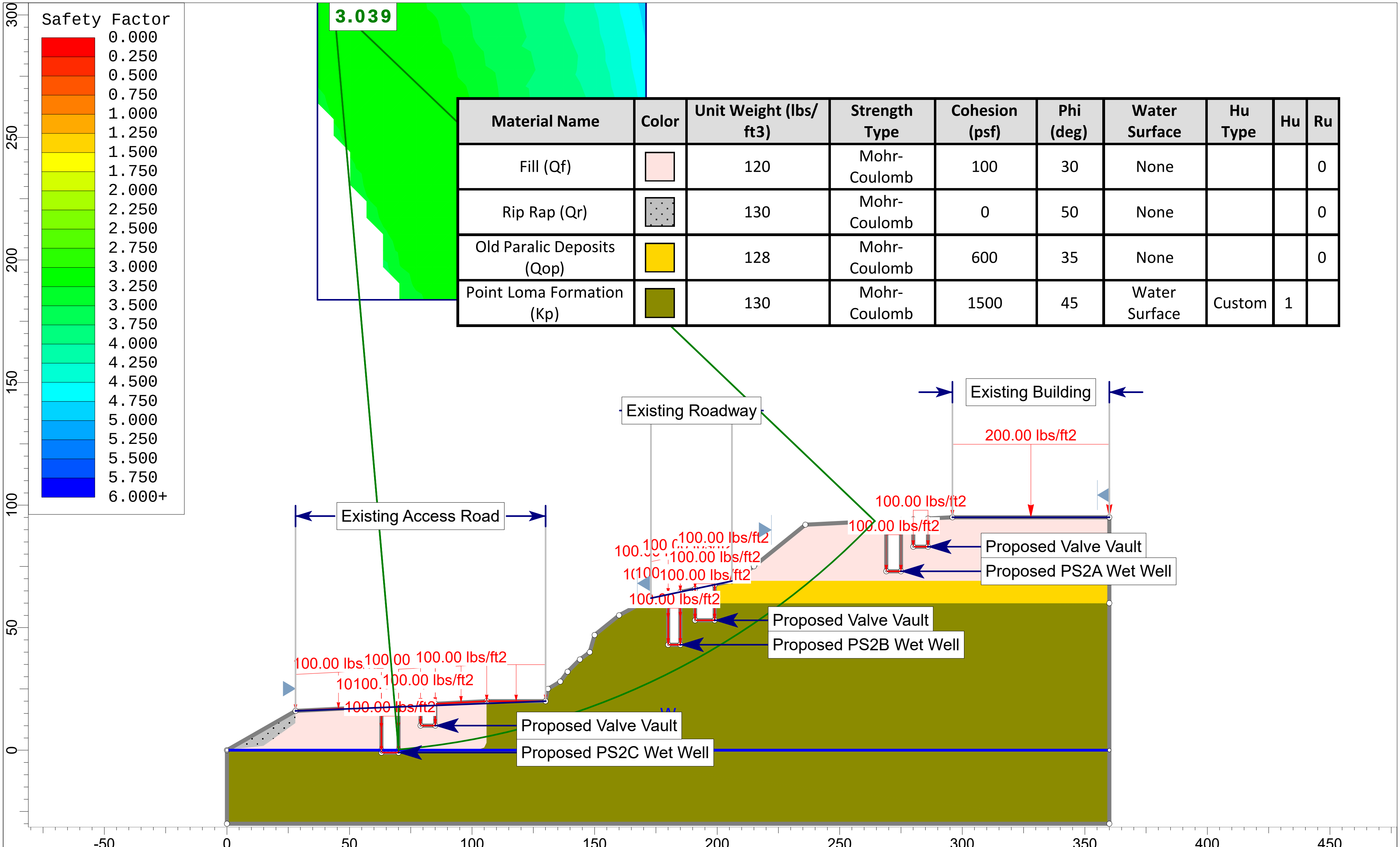
Cross Section B-B': Existing Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.4



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30	None			0
Rip Rap (Qr)		130	Mohr-Coulomb	0	50	None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35	None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45	Water Surface	Custom	1	

NOVA Services

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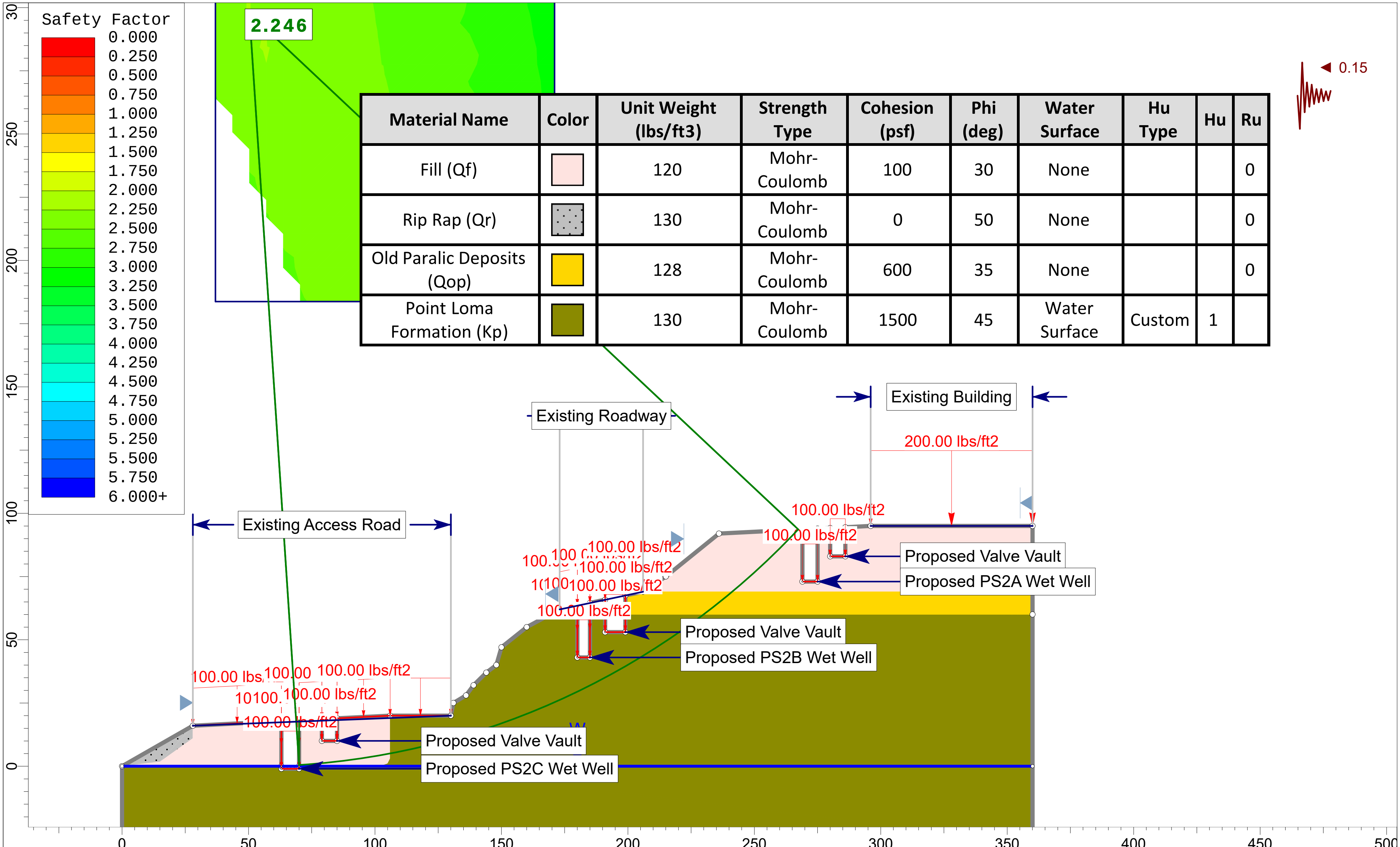
Cross Section B-B': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.5



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30	None			0
Rip Rap (Qr)		130	Mohr-Coulomb	0	50	None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35	None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45	Water Surface	Custom	1	

NOVA Services

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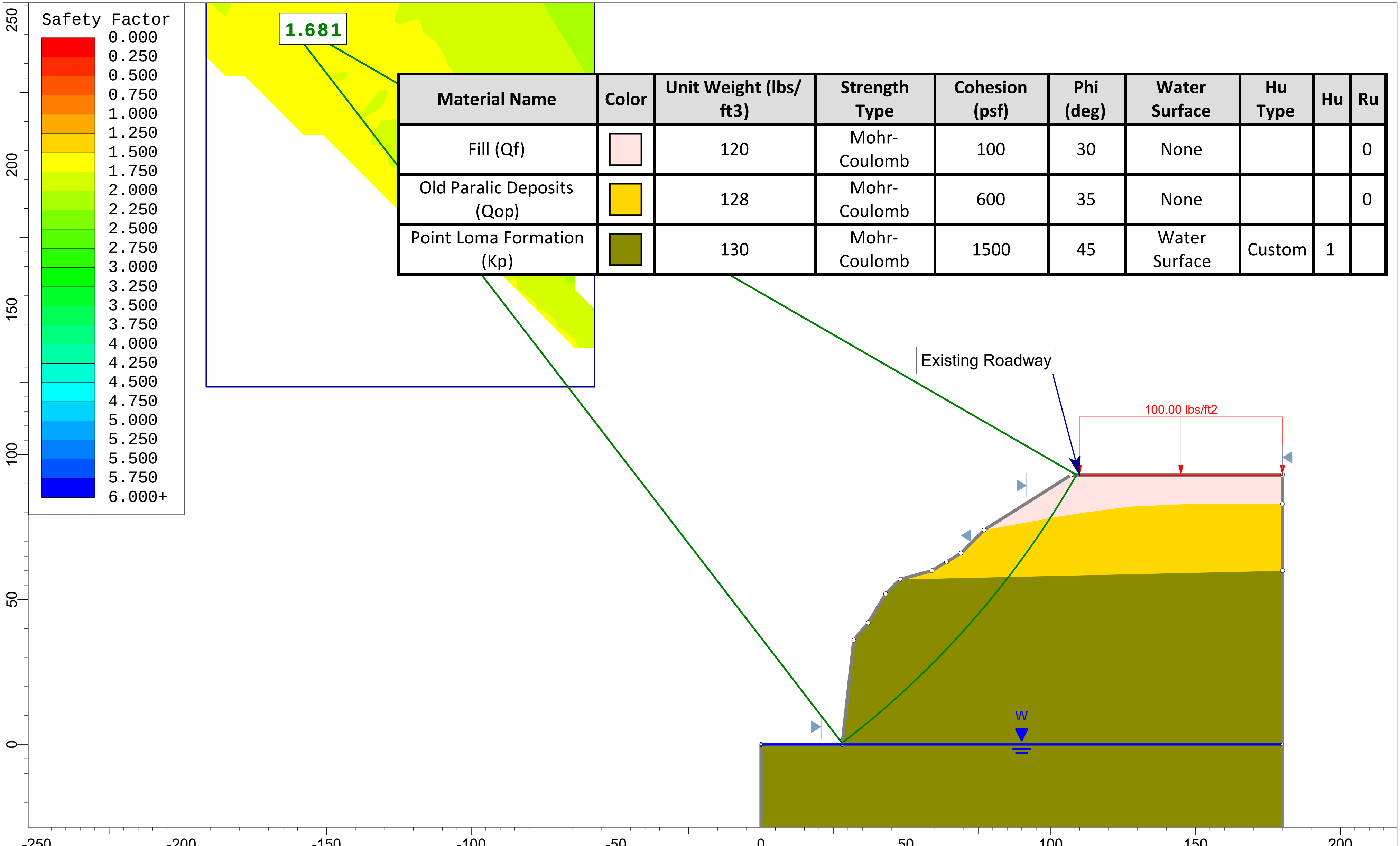
Cross Section B-B': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Pseudostatic Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.6



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30	None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35	None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45	Water Surface	Custom	1	

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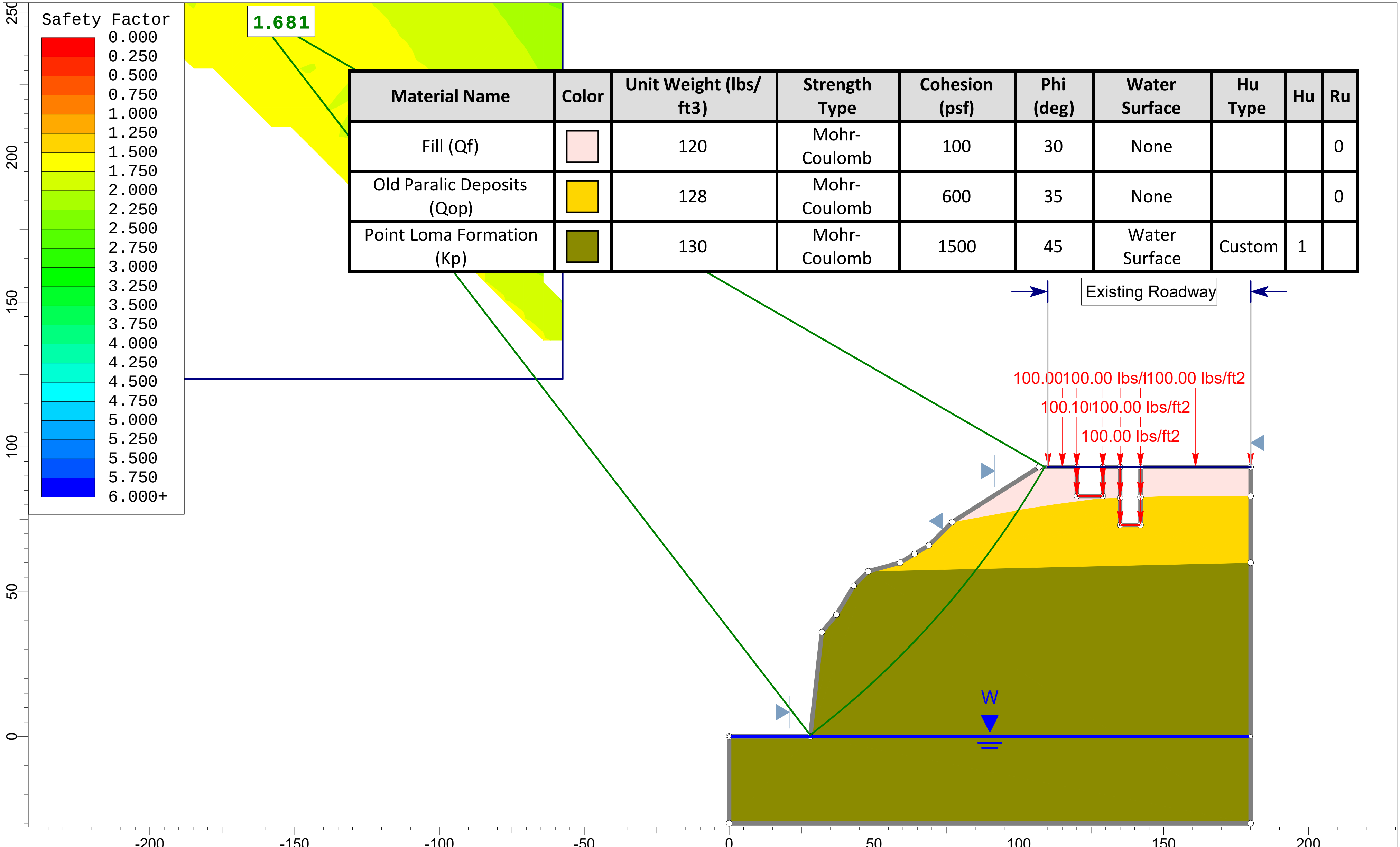
NOVA Services

Cross Section C-C': Existing Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.7



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30	None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35	None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45	Water Surface	Custom	1	

NOVA Services

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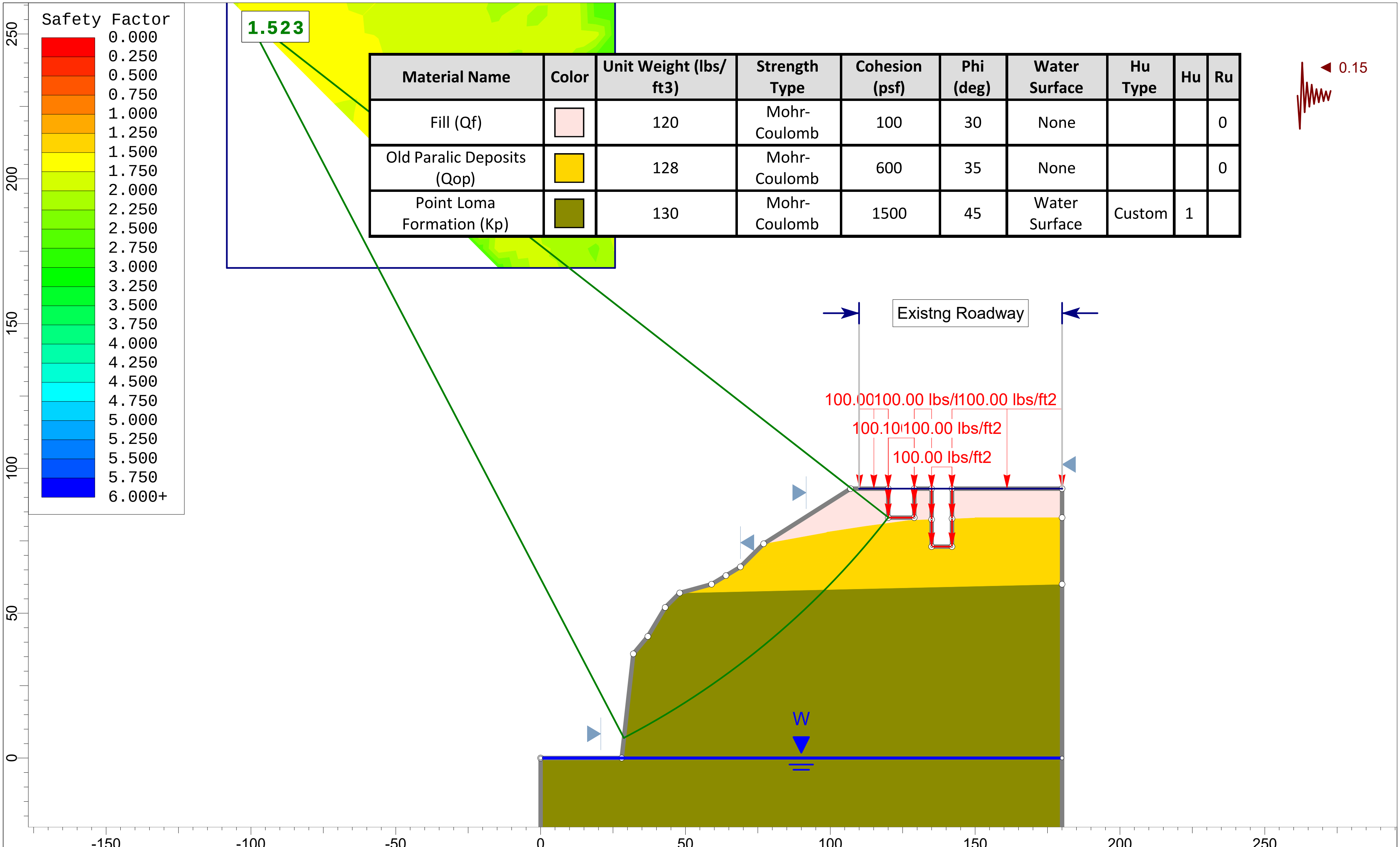
Cross Section C-C': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.8



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Fill (Qf)		120	Mohr-Coulomb	100	30	None			0
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35	None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45	Water Surface	Custom	1	

NOVA Services

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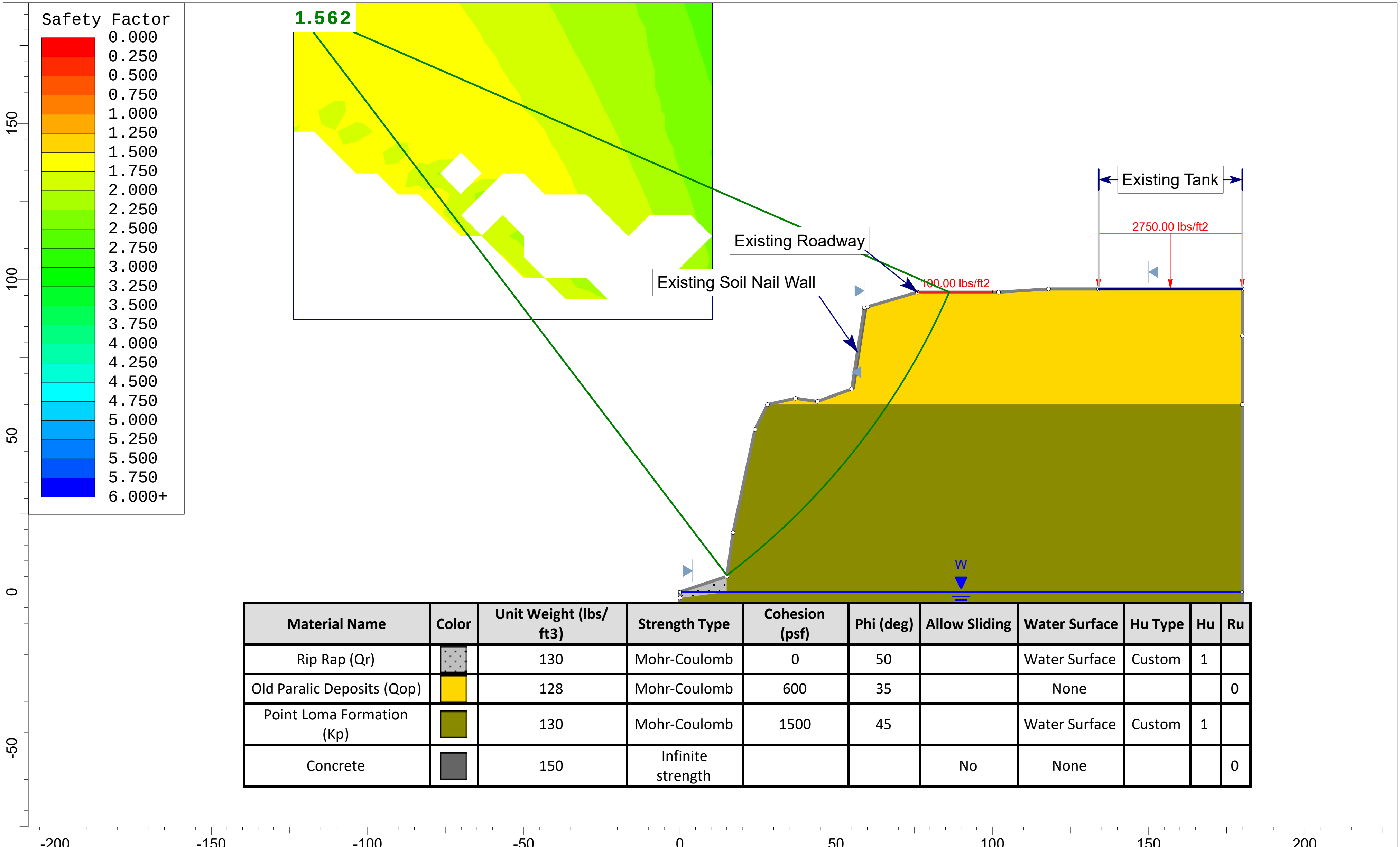
Cross Section C-C': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Pseudostatic Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.9



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu	Ru
Rip Rap (Qr)		130	Mohr-Coulomb	0	50		Water Surface	Custom	1	
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35		None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45		Water Surface	Custom	1	
Concrete		150	Infinite strength			No	None			0

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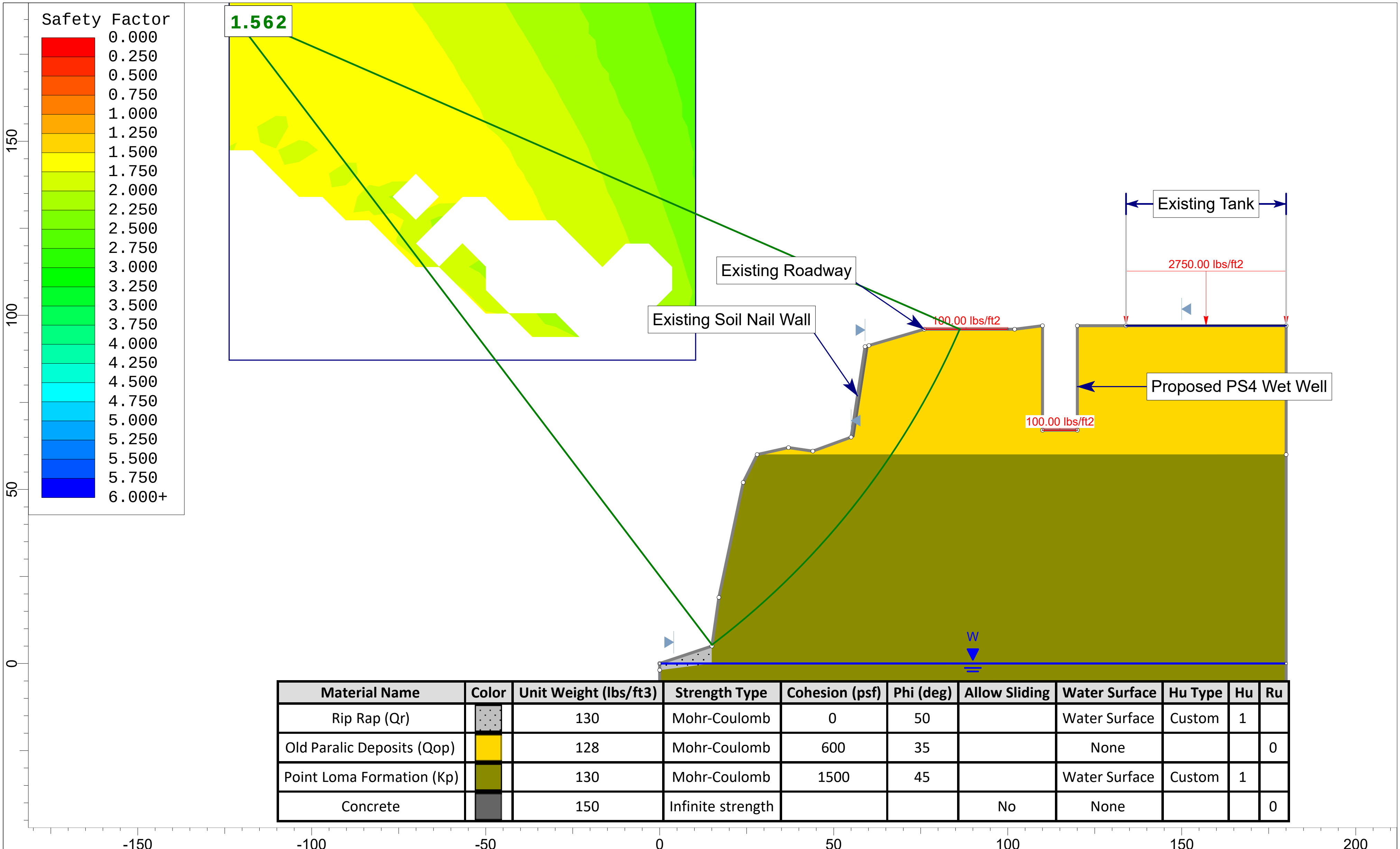
NOVA Services

Cross Section D-D': Existing Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.10



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu	Ru
Rip Rap (Qr)		130	Mohr-Coulomb	0	50		Water Surface	Custom	1	
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35		None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45		Water Surface	Custom	1	
Concrete		150	Infinite strength			No	None			0

NOVA Services

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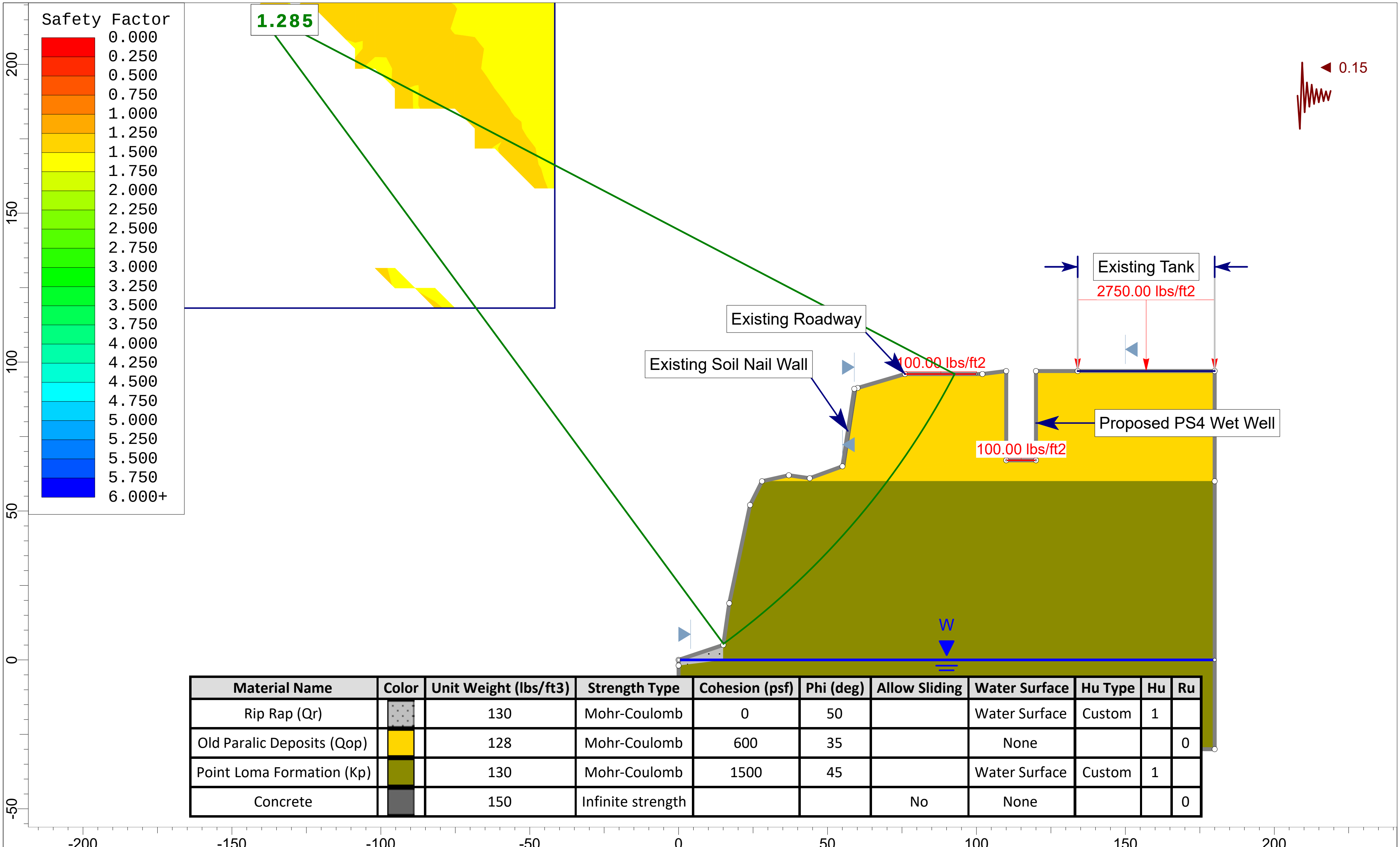
Cross Section D-D': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Static Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.11



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu	Ru
Rip Rap (Qr)		130	Mohr-Coulomb	0	50		Water Surface	Custom	1	
Old Paralic Deposits (Qop)		128	Mohr-Coulomb	600	35		None			0
Point Loma Formation (Kp)		130	Mohr-Coulomb	1500	45		Water Surface	Custom	1	
Concrete		150	Infinite strength			No	None			0

SLIDEINTERPRET 9.019

NOVA Services

Cross Section D-D': Proposed Condition

Project: Stormwater Diversion at PLWTP	Project No.: 2023274.1
Analysis Method: Spencer's Method - Pseudostatic Analysis	Date: March 2024
Analysis By: GCD	Figure No.: C.12

APPENDIX D

Critical Geotechnical Evaluations

CRITICAL GEOTECHNICAL EVALUATIONS

Point Loma Wastewater Treatment Plant (PLWTP)
Stormwater Diversion

TABLE OF CONTENTS

NOVA Project No. 2023274.1

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Suite F
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949.388.7710

usa-nova.com

Q.1

Engineering Geology of an Area on the west side of Pt. Loma Peninsula, San Diego, California

29 September 1958

Alfred J. Carsola
Alfred J. Carsola
Geological Consultant

29 September 1958

Mr. Philip H. Benton
Benton Engineering
4153 Adams Avenue
San Diego 16, California

Dear Mr. Benton:

Submitted herewith are the original and four copies of the report on the engineering geology of an area on the west side of the Point Loma peninsula. The study described in the report was completed with our verbal instructions and your letter confirming our contract.

Respectfully submitted,

Alfred J. Carsola

Alfred J. Carsola
Geological Consultant
3569 Addison Street
San Diego 6, California

G-766a

INTRODUCTION

This report describes the geology of a proposed site for 3 concrete digestors on Pt. Loma, San Diego (Fig. 1). A complete geological survey of the area would normally include more data than that included in this report, but the nature of the problem requires that only the geologic features affecting foundation engineering be examined, and thus the report deals principally with these features.

The area of interest is bounded on the east by Woodward Road, and Gatchell Road and on the west by the sea. Northern and southern boundaries are shown in Fig. 1. Owing to the steepness and instability of the sea cliffs below an elevation of about 60 feet above sea level and to the depth of the water at the foot of the cliffs at other than extremely low tide, geologic surveying along the present wavecut beach is extremely difficult and hazardous, if not impassible. Even at low tide waves break against the foot of the cliff and the combination of breaking waves and backwash renders exploration extremely difficult. A reconnaissance of the rocks exposed along the lower sea cliff has been made, but without the use of conventional geologic surveying instruments, which an earlier attempt at exploration showed are difficult to use, easily damaged, and unreliable under the circumstances. Detailed study of that portion of the cliffs above 60 feet has been accomplished without difficulty. At the same time observation from the upper cliffs is sufficiently good to permit mapping of the larger faults exposed in the lower cliffs.

Dip and strike of bedding and faults was measured with the Brunton compass. Locations were plotted on U. S. G. S. map of the Point Loma Quadrangle, 7.5 minute series (Topographic), scale 1:24000 and on a special U. S. Navy Electronics

Laboratory (N. E. L.) map, scale 1:7200. The former map shows topography with a 25-foot contour interval, while the latter shows works of man, such as buildings, roads, etc. not shown on the U. S. G. S. map. Figure 1, except for the inset, was drawn from an overlay on the N. E. L. map.

GEOMORPHOLOGY

The most conspicuous topographic features of Pt. Loma are the uplifted marine terraces, for which the San Diego area is geologically famous. These terraces have been described by Ellis and Lee (1919), Hanna (1926), and Hertlein and Grant (1944). The older terraces occur at the higher levels. Thus, of the three easily recognized terraces on Pt. Loma, the oldest and highest is the San Diego Mesa, also called the Linda Vista Terrace and the Linda Vista Mesa.

Over most of San Diego and on Pt. Loma this terrace occurs at an elevation of approximately 300 feet. The broad flat top of Pt. Loma is underlain by this terrace, which is covered by a reddish-brown concretionary formation ranging from silt to conglomerate. Ranging in age from Upper Pliocene to Lower Pleistocene, this formation is known as the Sweitzer formation.

The next youngest terrace is conspicuously developed near an elevation of 100 feet. It slopes westward from an elevation of about 150 feet to an elevation near 100 feet in the northern part, but closer to 75 feet in the southern part of the area investigated. The buildings on the west side of Pt. Loma have in most cases been constructed on this terrace. This terrace, henceforth referred to as the 100-foot terrace, and the next younger, the 60-foot terrace are both shown in Photograph 2.

The youngest uplifted terrace is exposed where small youthful canyons intersect the steep bluffs overlooking the sea. This terrace is found at an elevation of approximately 60 feet above sea level. This terrace is covered by an unconsolidated to slightly consolidated sedimentary deposit consisting principally of mixtures of sand and silt, and hereafter referred to as the "Terrace" formation. This terrace slopes westward from a locus of points buried by the Terrace formation, but probably located near the foot of the bluffs east of Woodward Road.

At the foot of the seacliff is an irregular terrace presently being cut at the present stand of sea level. It represents a graphic picture of how the 60-foot, 100-foot, and the Linda Vista terraces each appeared during some still-stand of the sea during the Pleistocene. Since then a combination of lowering of sea level and upwarping of the coast has left these terraces high above sea level, covered in part by deposits laid down by waves, streams and the wind.

The sea cliff is being rapidly eroded by waves, which erode the softer rocks at sea level, leaving the overlying rocks unsupported so that they ultimately slump and fall to the foot of the cliffs. The Cretaceous rocks forming the lower bluffs on Point Loma are extensively faulted and jointed, offering zones of weakness to the sea which cuts rapidly into these fractures forming sea arches, sea caves (Photograph 1), and small indentations in the coastline. As shown in Photograph #1, cutting of the caves leaves the slides and roofs unsupported, leading to slumping. Not only does the geologic structure in the Cretaceous rocks composing the lower cliffs control the location of sea caves and coastal embayments, but it also determines the site of the small canyons which indent the coastline. In nearly every case a fault or large joint can be seen in the exposed cliffs below the lower terminus of these canyons.

Thus the structure and relative resistance to erosion of the Cretaceous rocks below the 60-foot terrace is not limited to the control of the configuration of the coastline, but extends for some distance inland.

STRATIGRAPHY

Rocks of two formations outcrop in the area investigated. The cliffs below the 60-foot elevation are composed of Cretaceous sandstones, shales and mudstones, known as the Chico formation. On the 60-foot terrace a sedimentary unit composed principally of silty sands and sands has been mapped. This formation, of Pleistocene age, is unnamed, but will be referred to as the Terrace formation in this report.

The Chico formation. Two mappable members of the Chico formation can be recognized in the lower sea cliffs.

The lower member consists principally of gray mudstone and shale containing many small pieces and thin layers of lignite interbedded with sandstone beds rarely more than one foot thick. This member is generally highly jointed and easily eroded and where it is exposed to wave erosion, the cliffs are being rapidly cut back.

The upper member consists of heavy beds of sandstones interbedded with thin layers of siltstone and shale. This member is relatively resistant to erosion so that where it is exposed to the waves, the cliffs are being slowly worn back. Thus, this member generally underlies the headlands. It is everywhere present as the rock just underlying the 60-foot terrace.

The Terrace formation. This formation, as stated earlier, overlies the 60-foot terrace in the manner shown in Figure 2, and Photographs 2, 3 and 4. It immediately underlies the 100-foot terrace in the area understudy.

In general, this formation is indistinctly bedded, although locally bedding appears more distinct than is verified by close examination (Photograph #5). For the most part this formation consists of varying combinations of silt and sand. Locally, fragments of sandstone and siltstone, apparen[illegible]om erosion of the Chico formation, form distinct horisons. So[illegible]ratics are 8 to 10 inches in maximum diameter. Apparently some of th[illegible]strated by the two testholes drilled by Benton Engineering and ha[illegible]rted on their summary sheet. These are only eroded fragments, not [illegible]es.

At the pump-house about 1/2 miles north [illegible]rn limit of the area, a basal conglomerate about one foot thick is the [illegible]ber of the Terrace formation. This conglomerate is composed not only [illegible]rived from the Chico formation, but includes pebbles and cobbles of [illegible]orphic and igneous rocks such as quartzite and granodiorite.

In the first canyon south of the rifle range [illegible]errace is exposed where the lower cliffs intersect the canyon. A few fe[illegible]ebbly bed containing numerous shells of molluscs overlies the trunca[illegible]beds (Photograph #6). This bed is the basal member of the Terrace fo[illegible]hat location. This appears to be the locality mentioned by Hertlein and Gra[illegible]n page 70 of their report, in which they have identified the shells as Pleis[illegible]ge.

The Terrace formation does not slump rea[illegible]s exposed in road cuts and gully walls, it maintains steep slopes (Phot[illegible]6, 7). However, this formation is fine-grained and uncemented, and [illegible]ily eroded by running water. The fluted, badlands type of gullying seen in Ph[illegible]is typical of erosion by running water in soft, unconsolidated material. [illegible]uths of the canyons, where an abrupt increase in gradient is provided by the [illegible]erosion of the Terrace formation is very rapid. Inland from the cliffs [illegible]ate of erosion results in thicker exposures of the Terrace formation.

STRUCTURE

The Chico formation is conspicuously jointed and faulted (Fig. 1, Photographs 1, 8, 9). The joints were not closely studied, but two nearly vertical sets, almost at right angles to each other, are present.

Similarly, there also appear to be two sets of faults, one striking north-south and the other striking approximately east-west (Fig. 1). All the faults seen are vertical or nearly so. Displacements range from a few inches, such as shown along the small fault in Photograph 8, to 15 feet along the second fault north of the southern boundary of the area investigated.

None of the faults seen penetrates above the 60-foot terrace into the Terrace formation. These are old faults which had ceased to be active when the 60-foot terrace was cut. Their age is not known, except that it is post-Cretaceous and pre-late Pleistocene.

It is highly probable that the faults are inactive. However, as described in the section on geomorphology, these fractures influence both cliff and stream erosion, and siting of structures should avoid at least the principal faults. The problem is minimized with increasing distance from the cliffs, but should always be considered.

The Chico formation strikes approximately parallel to the coast, generally a few degrees west of north. Dips are toward the northeast, usually between 5° and 10°. It is important to note that, along the west side of Pt. Loma, the Cretaceous rocks, which form the base of the cliffs, are dipping inland away from the sea. This is a more favorable situation, from an engineering standpoint, than if the rocks dipped seaward, for in the latter case, slumping and landslides would be a major problem, especially during prolonged periods of rainy weather.

Attitudes of the Terrace formation are difficult to measure, due principally to the fact that it is so indistinctly bedded. It is also crossbedded in places, which means that dip is variable, and no one measurement can be considered typical. In general, it appears to dip westward at angles no greater than 3°, or to be horizontal. Thus, slumping and landsliding of the Terrace formation is insignificant.

CONCLUSIONS AND RECOMMENDATIONS

The following observations are significant in consideration of an engineering project site in the area surveyed on West Point Loma:

(1) A steep cliff at the seaward edge of the proposed site is being rapidly cut back by the sea. This erosion is most effective at the immediate cliff edge.

(2) The Cretaceous Chico formation, which outcrops along the lower, steeper cliffs is faulted and jointed, but the faults are old and inactive. They influence the avenue of cliff erosion and stream cutting in the canyons which intersect the cliffs.

(3) The faults referred to above do not penetrate into the material overlying the Chico formation.

(4) The Chico formation dips inland away from the coast, which minimizes the possibility of large landslides, slumps or rockfalls.

(5) The unconsolidated mixture of sand and clay (Terrace formation) which immediately underlies the proposed site inland from the cliffs is easily eroded by running water, but does not readily slump.

(6) The Terrace formation thickens inland (eastward) from the cliffs.

Based on the above observations, the following recommendations are made:

(1) The area is suitable for large engineering structures, provided that they are constructed at least 200 feet away from the upper cliff edge and the small canyons which incise the area.

(2) While not imperative, it is advisable to avoid the projections of the principal faults shown in the cliffs.

(3) Owing to the fact that the Terrace formation thickens toward the east, it is advisable to locate engineering structures as close to the eastern boundary of the area investigated as possible.

REFERENCES

Ellis, A. J. and Lee, C. H., 1919, "Geology and ground waters of the western part of San Diego County, California", U. S. Geological Survey Water Supply Paper 446, pp. 1-321.

Hanna, M. A., 1926, "Geology of the La Jolla Quadrangle," University of California Publications, Bulletin of the Department of Geological Sciences, vol. 16, no. 7, pp. 187-246.

Hertlein, L. A. and Grant, U. S., IV, 1944, "The geology and paleontology of the marine Pliocene of San Diego, California," Part 1, Geology, Memoirs of the San Diego Society of Natural History, Vol. II, 72 pp.

Photograph #1.

Sea cave resulting from wave erosion in zone of weakness provided by fault shown by line. Note slumping of strata on the sides and in the roof of the cave.

Photograph #2.

View of coastline showing present wave cut terrace exposed at low tide, 60-foot terrace and 100-foot terrace. The Chico formation (Cretaceous) forms the steep cliff below the 60-foot terrace, while the Terrace formation (Pleistocene) has been deposited on the 60-foot terrace (indicated by dashed line). Photograph taken facing north from a headland seaward of rifle range.

Photograph #3.

Photograph of contact between Terrace formation (above dashed line) and Chico formation. Note that the mouth of the small stream valley is truncated by the cliffs and that the stream has cut a foot or two into the Chico formation.

Photograph #4.

Photograph taken facing south from the point where photographs 2 and 3 were taken. Dashed line indicates contact between Terrace and Chico formations.

Photograph #5.

Nearly horizontal bedding in Terrace material indicated by erosional benches parallel to the bedding. The flat terraces are underlain by more resistant material than that composing the slopes.

Photograph #6.

Contact between Chico formation (below dashed lines) and basal, shelly Terrace formation (above dashed line) exposed at the mouth of a canyon located in Figure 1.

Photograph #7.

View showing badlands gullying typically developed in Terrace formation. Note gently sloping 100-foot terrace just above the valley in which the photograph was taken.

Photograph #8.

Small fault in Chico formation. Photograph taken from an indentation in the cliff produced by selective wave erosion following the line of weakness provided by the fault. Right side of fault has moved upward about one foot relative to left side, as shown by displacement of sandstone beds in the upper part of the photograph.

Photograph #9.

The sea cliff bordering the area investigated. Note the fault (indicated by arrow) and the dashed line indicating the contact between the Terrace and Chico formations. Note that the fault does not extend into the Terrace formation, but is truncated by the unconformity between the latter and the Chico formation.

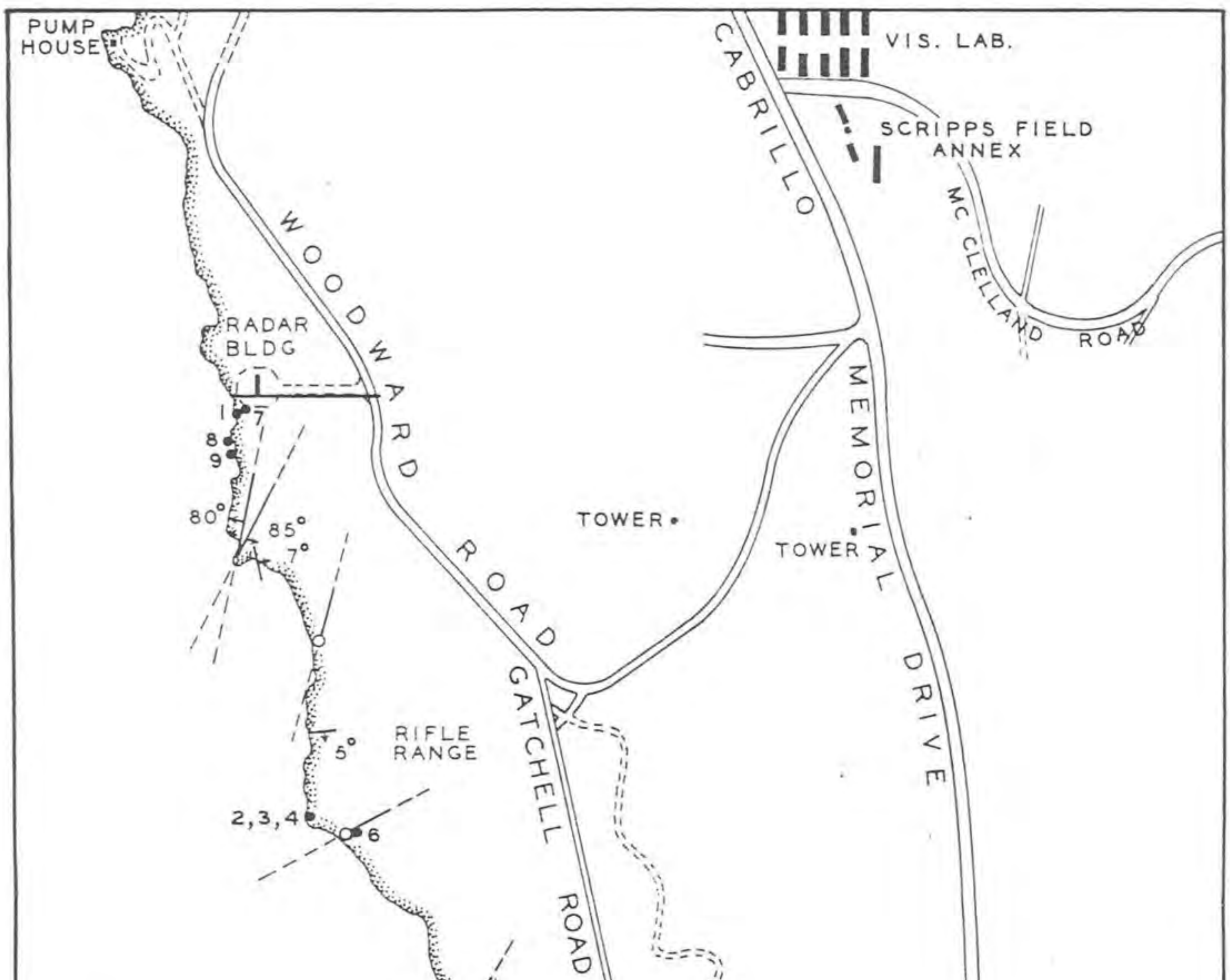

Figure 1

Map of Pt. Loma peninsula showing location of area investigated. The actual area investigated, bounded by the coastline, Woodward and Gatchell Roads, and the heavy east-west lines connecting them, is shown in the main figure, which is an expansion of the obliquely lined area shown in the inset.

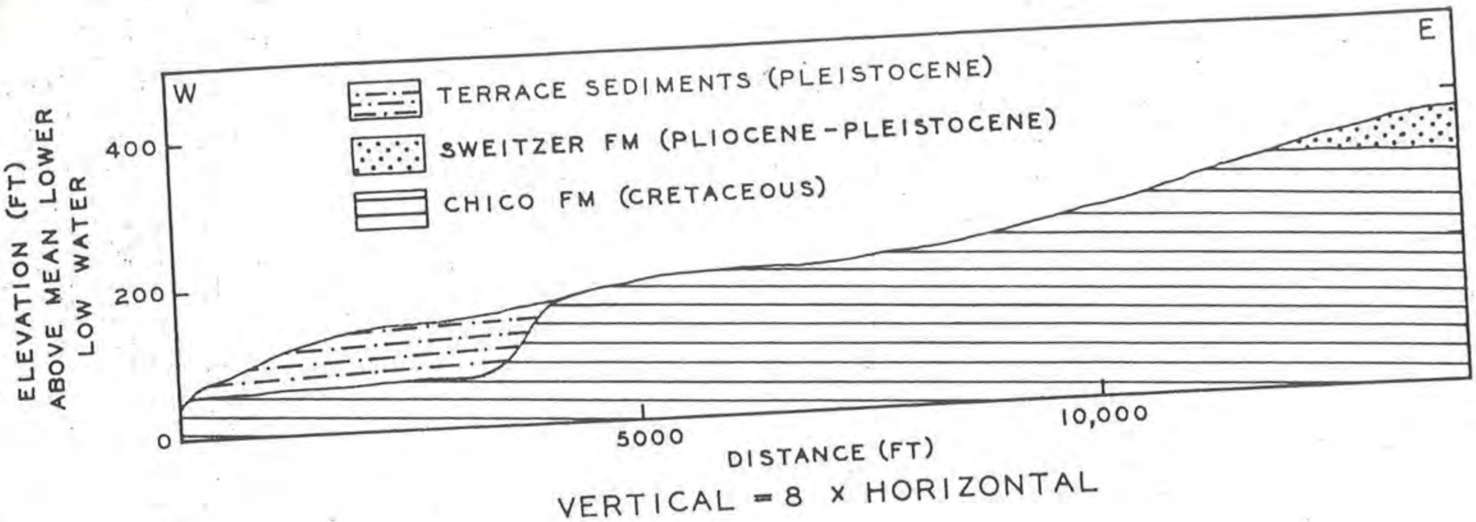

Figure 2

Generalized profile along an east-west line about halfway between the Rifle Range and the Radar Building, and extending from the foot of the cliffs to the top of Pt. Loma peninsula. Note that both the Terrace formation and the Sweitzer formation (which does not appear in the area investigated) have been deposited on terraces, and that the Chico formation dips eastward, that is, away from the seacliffs.

ENGINEERING GEOLOGY ASPECTS OF POINT LOMA, CALIFORNIA

by

E. Dean Milow

1959

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CONCLUSIONS AND RECOMMENDATIONS

Geologic stability of area - With only one exception, evidence shows that Point Loma is relatively stable and free from present day deformational activity or other relatively rapid geologic processes. The exception is a large active slide on the east side of Point Loma, west of Rosecrans Blvd. (See geologic map.) The slide area is outlined by the tensional cracks in the surficial material in the proximity of the failure planes and the two main pressure ridges occurring near the toe of the slide with concomitant slight upward bulging of Rosecrans Blvd. (See Plate I.) This is a rotational slide moving in a northeasterly direction. The separation at the head of the slide occurs along two intersecting faults trending northwest and slightly east of north, respectively. Along the sides of the upper part of the slide the rock mass is parting along two fault zones trending northeast. Toward the eastern margin, the slide becomes larger and compound with the failure planes branching off onto other intersecting and parallel faults. The movement in the resulting larger slide area is represented by at least two separate blocks coincident with the two main pressure ridges along the toe of the slide. The change of direction and position of the tension cracks in the vicinity of these other faults near the lower reaches of the slide show the complex nature of this part of the unstable mass. It should be understood that Recent deformational activity or movement along these faults is not the cause of this slide, but only that this slide is taking advantage of these rock fractures in the area as separation or failure planes. The whole slide is more or less coincident with a previously formed synclinal structure in the layered rocks. The direction of inclination of these beds form an additional control on the limits and movement of the slide . The greatest movement of the slide is following this structural trough with the base of the rock mass separating along the inclined well developed bedding planes in the upper part of the siltstone member. The base of the slide in the longitudinal direction can be generally considered as a broad curving surface, concave upward, with the headward portion rotating down and the toe area rotating up.

To stabilize this active slide, two controls will be necessary. One is to redistribute the mass of the slide by removing appreciable amounts of material near the head of the slide proper and at the head of the subsidary branching portions in its lower part and placing this material near the toe as a buttress fill. This fill would be placed below and east of Rosecrans Blvd. The other factor to control is the seepage of water into these failure planes which acts as a lubricant and initiates or perpetuates the movement. Even though the annual precipitation in the area is quite low, the constant irrigation of the expanding National Cemetary above this area provides more than adequate surface runoff to charge the ground water of the area. In fact, this is the most likely cause to the renewed movement of the slide in the last few years. Corrective drainage can be accomplished by lining the present surface drainage areas above the slide with an impervious cap such as asphaltum particularly over the surface near its head and upper lateral margins and carrying this drainage well away from the slide area to prevent percolating into the ground and recharge of the ground water. As can be realized, the cost of such a project would be considerable and therefore, the proposed pipeline route along Rosecrans Blvd. east of this slide should be discarded for other possibilities with less costs involved.

The older rocks comprising the Point have been through considerable stress and strain or deformation in the geologic past and these rocks have been ruptured resulting in fractures in the rock extending to considerable depth. Differential movement has taken place along some of these fractures in the past placing rocks of a different nature in junctaposition. These latter type of fractures are called faults. Some of the movements along these faults resulted in a greater magnitude of stratigraphic displacement than others while no appreciable displacement is evident along other fractures and these are called joints.

The expression of these fractures, especially the faults of significant stratigraphic separation, are shown on the geologic map where the trace of these fractures intersect the land surface and are extended on the map only as far as they could be recognized and followed by exposure of the fractures in rock outcrops or where they affected formational boundaries or key beds. It should be realized that in most cases these faults have greater extent than shown on the map and continue in their trending direction even though the trace is not shown in the older rocks or the faults are hidden beneath younger materials not affected by the faulting.

The rocks of the area contain many more fractures, both faults and joints, than are shown on the geologic map or in the cross-sections. These have little to no affect on the distribution and boundaries of the different rocks but are quite frequent over most of the area. For short distances they were observed to occur 10 to 20 feet apart or less. In other outcrops these fractures occur some 25 to 50 feet apart or with quite an irregular frequency. At least six sets of faults and joints can be recognized in the area distinguished by the attitude of the fault plane. Of the six sets of faults, only 3 of these contain a few faults that can be considered of past major consequence and exhibit stratigraphic offset of a magnitude of 100 feet or more. These three systems which contain faults that appreciably affect the distribution of rock types trend N5-15E and dip mostly to the south with a few dipping north; and N60-70E, dipping mostly south but also contain a few dipping north; and N60-85W, all observed dipping south. The other three sets containing faults of minor concequence trend N20W to nearly north-south and dip mostly southerly; N30-40E, dipping north; and N20-40E, dipping south.

As previously stated, Point Loma can be considered as relatively stable in its present stage of geologic history. The evidence that supports this is the relationship of geologic structures such as faults, joints and folds to the rocks affected by these structures and the geologic age of these rocks. Of particular importance is the Late Pleistocene terrace materials (Pltm) as this formation is not affected by these structures. This shows that the last deformational activity (faulting, folding, etc.) pre-dates the deposition of these sedimentary materials. It should be mentioned that these Late Pleistocene materials cap the majority of the area underlain by the Swietzer and Bay Point formation but for convenience of illustration the distribution of the Late Pleistocene terrace materials are shown on the geologic maps only where this material overlaps these two older Pleistocene formations and covers the Cretaceous sandstone and siltstone members (see geologic cross-sections for a clearer, more accurate picture of the occurrence of these materials). Because of the Late Pleistocene capping, the trace of faults cutting through the Sweitzer formation are not shown continuously in all cases. Since some of the faults affect the distribution, position and attitude of the Sweitzer formation from its original nearly horizontal occurrence at a stratigraphic position above 300 feet elevation to a position as low as 200 feet elevation or less and inclined locally 20-30 degrees or more, it is evident that some deformation post-dates the deposition of this formation. The relative geologic age of the Sweitzer formation is presently considered to be Pleistocene, probably Early Pleistocene. Considerable time must have existed after this Pleistocene faulting and before the deposition of Late Pleistocene materials to account for the deposition of the Bay Point formation and the extensive modification of the surface expression of this faulting and the formation of the incipient present topography and land forms covered by the Late Pleistocene terrace materials (see Plate II, fig. I and Plate III.) In addition, the Late Pleistocene materials contain marine fossils at elevations as high as 325 feet showing that the sea level has since lowered to its present position; a well developed soil profile exists on these terrace materials commonly to depths of 5 feet or more; and subsequent erosion has incised and modified the original distribution of these materials: all of which takes considerable time. These data suggest the age for the accumulation of the Late Pleistocene terrace materials at a magnitude of 10-20,000 years; therefore, the last deformational activity in the vicinity of the Point occurred somewhat earlier than this time. Consequently, one can state that the area under immediate consideration is geologically stable within the probability limits of at least 10-20,000 years.

Since only some of the multitude of faults existing in the older Cretaceous units affect the Sweitzer formation; the observable fact that jointing in the older rocks does not extend up into the Sweitzer; a much greater stratigraphic separation exists between the same horizon in the Cretaceous rocks than between those of the Sweitzer formation on either side of certain faults (see geologic cross-sections) and that opposite directions of differential stratigraphic separation is evident along some faults between horizons in the Cretaceous rocks and those in the Sweitzer formation (see Plate II, fig. 2 and cross-sections): much of the present occurrence of different Cretaceous rocks in junctaposition along faults can be attributed to a deformational episode that took place earlier than the Pleistocene (pre-one million years ago). Empirical evidence also suggests this pre-Pleistocene deformation to be due to compressional forces or couple forces resulting in compressional effects while the Pleistocene deformation affecting the Sweitzer formation was due to tension, hence relaxing of the earlier crustal unrest of the area. This plus the feature of a much greater apparent displacement along the faults and greater overall activity attributed to the earlier deformation than during the Pleistocene further strengthens the conclusion of present relative stability of the crust in the immediate area.

Ground water - It should be expected that portions of the drift, depending on route selected, will initially encounter appreciable amounts of ground water during construction. This will be especially true near source areas and when passing through materials which readily allow the through percolation of ground water. Since there is no surface expression of the local ground water table being much above the present sea level, the only concern in the tunnel will be where a sufficient flow of water from the surface down to the ground water table exists. These conditions can be forseen only in the vicinity of the residential areas to the north or adjacent to the National Cemetery. The main available routes for this water is along the fractures in the rock and possibly along certain layers in the Cretaceous sandstone member, especially near its base, which are porous and permeable enough to allow percolation of water. Appreciable flow of water initially encountered along any fracture in the excavation of the drift for the tunnel would be expected to continually decrease after the initial draining of the potential ground water and the modification effects of the tunnel itself on the distribution and movement of ground water in the vicinity of the tunnel. The effects and areas of concern will be covered in the discussion of the various proposed routes.

Discussion of the geology of the proposed tunnel routes - This discussion will cover the various proposed tunnel routes in order from north to south. The northern most tunnel, entering near Talbot St. and running southwest to the west side of the Point, would pass through the rock of the Cretaceous sandstone member most of the way at the proposed elevation of the tunnel (see reconnaissance geologic map and cross-section). These well indurated, silty medium-grained massive sandstones with occassional thin siltstone interbeds at intervals would be more difficult to quarry than other rocks of the area but will undoubtedly stand unsupported for greater distances and produce a better shear stress balance of arching of the overburden than other rocks comprising the area. The one exception would be within fault zones or where several sets of fractures intersect in the vicinity of the drift causing slivers or blocks of this rock to be unsupported upon excavation. As seen by the cross-section and map this route would intersect at least two of the major faults in the area plus other minor ones shown on the map and pass through at least two junctions of different materials, i.e., from sandstone into siltstone and back into the sandstone. Because of the proximity of this tunnel to a man-introduced source of percolating water from the surface, tunnel seepage will probably be encountered especially in areas of frequent fractures. Based on observed rates of surface seepage of ground water in other areas of western San Diego County covered by residential areas, an estimate of probable maximum rate of inflow along this route would be about ½-1 gal/day/ lineal foot of tunnel, subject to variability due to frequency of fractures along the route or intersection of aquifer beds in the sandstone member. The other proposed route from the Talbot St. entrance passing nearly south to the vicinity of NEL would remain parallel to and near the fracture zones for considerable distances as well as the crossing of several interfaces between the siltstone and sandstone units in fault contact. This would invite more strucural design, construction and maintanence problems due to the lack of strength of the intra-fault and fracture breccia blocks and gouge as well as the greater possibility of caving on a larger scale because of the proximity, frequency and attitude of the fractures to the tunnel.

The tunnels entering the Point in the vicinity of the San Diego City Boundary would cross the same faults as the northern most Talbot St. route but these tunnels would pass through different materials for most of their distance, These tunnels would encounter the sandstone member to the fault in the vicinity of Catalina Blvd. and then pass through the siltstone member the rest of the distance, even though the the tunnels cross another fault of considerable stratigraphic displacement. This Cretaceous siltstone member is of well laminated beds of indurated siltstone and sometimes claystones separated by thin sandstone laminae and occassional thicker sandstone interbeds. This unit for the most part has well developed bedding planes allowing the material to readily part along the laminations. The softer nature of this rock aided by the combined effect of intersecting bedding planes and fractures will permit much easier and rapid quarrying of the rock for a tunnel. However, these characteristics will also cause it to stand unsupported for much shorter distances and time. Also, "stoping upward" of the tunnel roof can be expected to be common in places and little assistance of support by the arching effect can be expected in the immediate vicinity of the tunnel walls and roof. Because these tunnels transect the structural trend of the attitude of the rocks and other structures such as faults which have changed the attitude of the rocks; the total amount and especially the direction of "lining pressure" on the tunnel and pipe will be quite variable along the course of the tunnels.

The two proposed tunnels entering the Point in the vicinity of McClellan Rd. and connecting with the tunnel parallel to the west side of the Point to the sewage treatment site have the best possibility of initially remaining parallel to the geologic structure of the area. With only slight modification of the position of the entrance of these tunnels away from the significant faults in the area (see geologic map and cross-sections) these tunnels can be aligned to pass through areas containing the least amount of faulting of considerable magnitude and pass through a nearly continuous section of the siltstone member. "Lining pressures" will be more consistant over greater distances in these tunnels because of the greater consistancy of the attitudes of strata along the route. The concentration of pressure will be along the southeast side of the tunnel in the beginning portions of the tunnel gradually shifting to lateral pressures on the roof of the tunnel as it crosses the gentle syncline in the vicinity of the crest and continuing to the west side of the Point. Even though joints and faults will be intersecting these tunnels, none will cause the junction of units of different kinds of rocks along the route nor drastically change the attitude of the strata at the proposed elevation. In total footage of tunneling as well as expected relative lack of encountered structural problems, these tunnels would undoubtedly be the lowest in costs.

The tunnel route paralleling the west side of the Point to the vicinity of the proposed treatment site appears to be quite favorable with some exceptions. The northern part of this route will pass through the siltstone member and for most of the way these beds have a dip component towards the east creating stability to the surface and overburden along the route as well as creating fairly constant concentration of "lining pressures" toward the west side of the tunnel. The one exception is near the middle of the proposed route where the tunnel will cross a fault zone of considerable width which has placed the sandstone member down in the path of the tunnel route south of the fault zone. Field evidence certainly suggests that several separate major faults are converging to this local from the east side of the Point and it may be the combined effect of all these faults existing along this area on the west side that accounts for the considerable stratigraphic displacement of several hundred feet. Nevertheless, any movement of this magnitude is expected to be dissipated along several fractures in a zone rather than along one fracture. One should, therefore, anticipate the rock along this zone to have little strength because of the frequency of fractures and brecciated, pulverized rock as well as the problems of support of the pipe line across such a zone compounded by having different kinds of rocks on either side of the zone. The base of the sandstone member encountered south of this fault zone is quite conglomeritic at this southerly occurrence and contains pebbles, cobbles and boulders up to five feet or more in dimensions of harder metamorphic and igneous rocks which may cause more difficulty in the excavation of the tunnel in this area (see cross-section B - B'). As previously mentioned, a source of percolating ground water exists in the vicinity of the National Cemetery. The amount of inflow can be expected to be several times the magnitude of the estimate for the residential area as most of the canyons near the Cemetery were observed daily to contain great quantities of runoff forming a substantial reservoir and hydrostatic head to the ground water.

What ever the final decision may be on the selected tunnel route based on the total factors involved, a general rule to follow to alliviate the local geologic factor is to select and/or modify the tunnel route which will best parallel the geologic structure, i.e., trends in folds, faults and continuity of rock types.

Discussion of the geology along the proposed open trench routes and pumping stations - Because the Late Pleistocene terrace materials are somewhat irregular in thickness and cover an irregular surface on the older rocks the trenches along both sides of the Point may encounter exposures of the older rocks along the trench. This will be especially true where the trench line approximates the mapped contact between these Late Pleistocene materials and the older rocks. Because of the generally poor compaction of these Late Pleistocene materials relative to the older rocks differential compaction and setting would occur where the pipe line transverses these different materials. It is recommended that the bed of the pipe crossing the Late Pleistocene materials be treated or modified to prevent this possible future readjustment.

If the final pumping station sites have been located on areas underlain by the Late Pleistocene terrace materials or especially man-made fill, proper testing of this material should be conducted to determine suitability of foundation footing. If considerable vibrations in pumping structures are anticipated and it is at all economically feasable, the foundations or the pumping station itself should be placed in the older Cretaceous rocks to prevent future structural damage.

fig. 1 : West shoulder of Rosecrans Blvd. in vicinity of slide showing some of the pressure ridges forming along the toe of the slide.

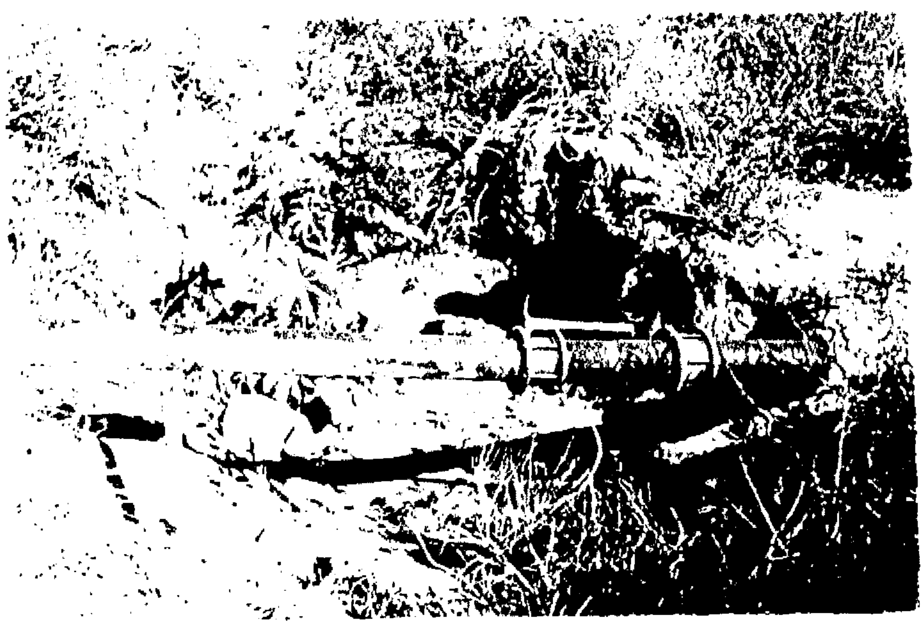
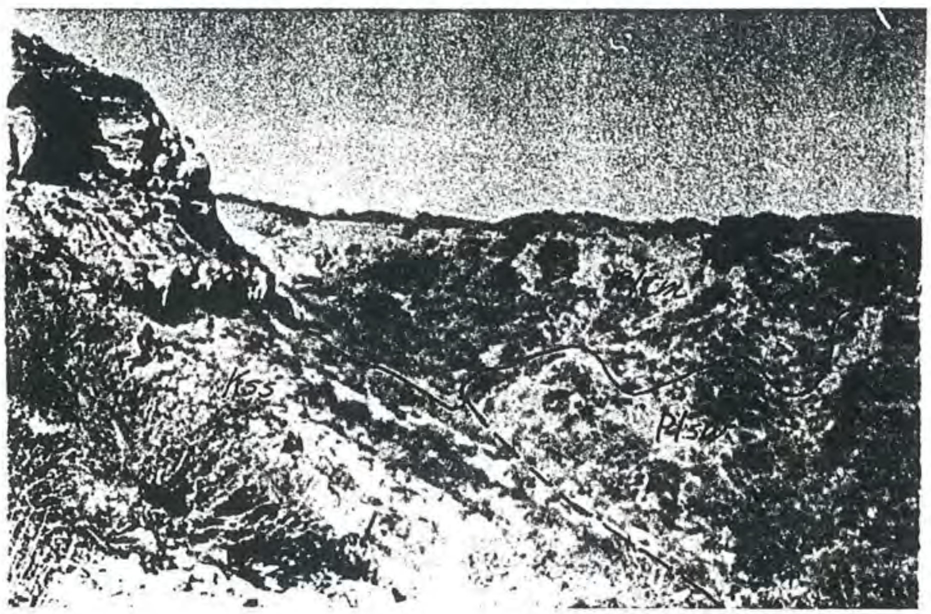


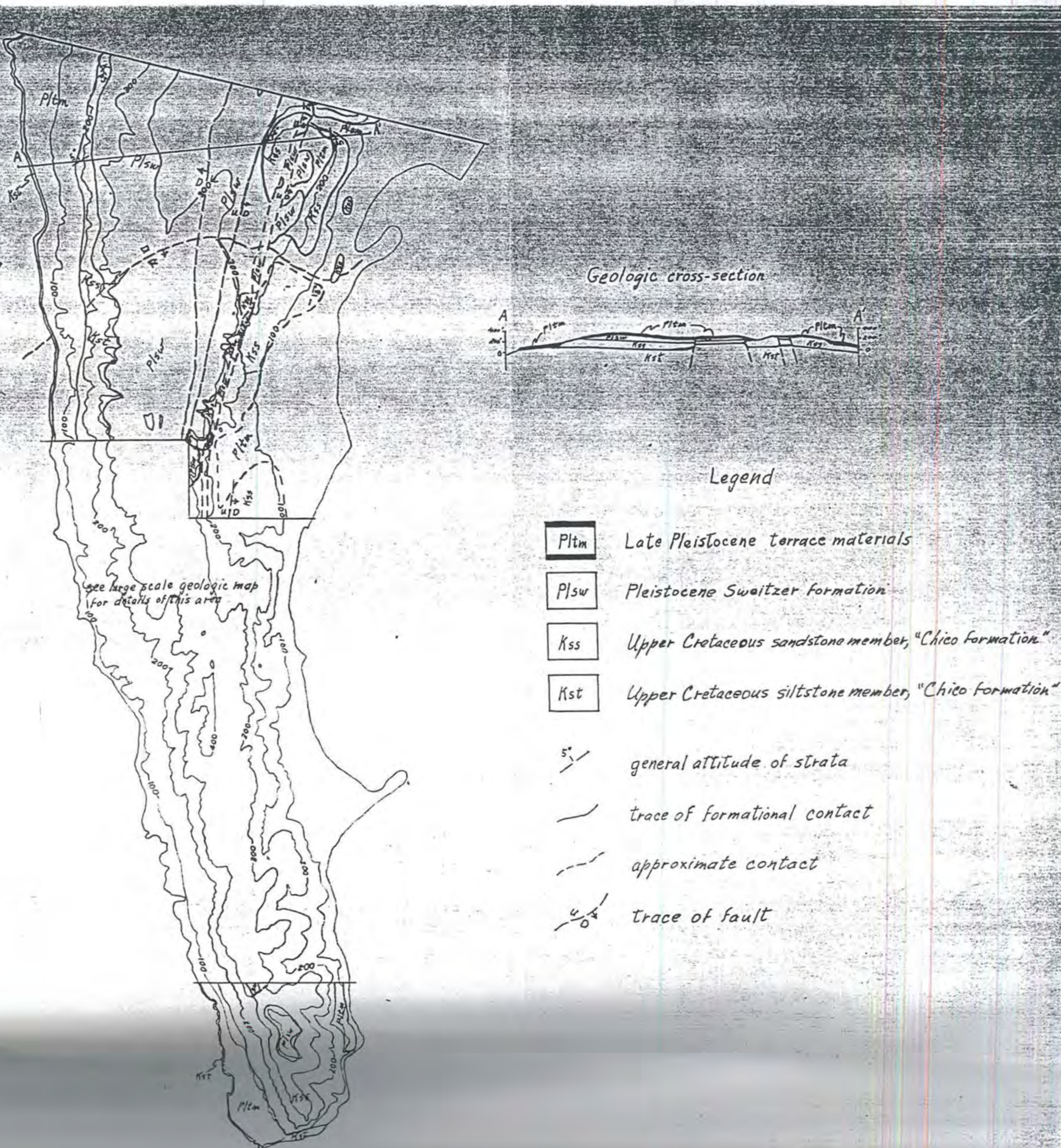
fig. 2 : Large tensional crack in surficial material near head of slide showing effect of movement over last few years. Note some of the extensions added to pipeline crossing slide failure zone.

fig. 1 : Fault southwest of McClellan Road displacing the Cretaceous sandstone member (Kss) and the Sweitzer formation (Plsw) but not the Late Pleistocene terrace materials (Pltm). Note the surface on which the Pltm sets. This surface truncates the faulted Plsw showing that considerable erosion preceded the deposition of the Pltm.

fig. 2 : Road cut in McClellan Road showing change in relative movements along fault. Compressional pre-Sweitzer faulting, displacement of bed "A" in Kss up on left side of fault; tensional post-Sweitzer faulting, Plsw faulted down on left side of fault. Note depositional occurrence of Pltm along eroded fault zone subsequent to faulting and covering part of fault.

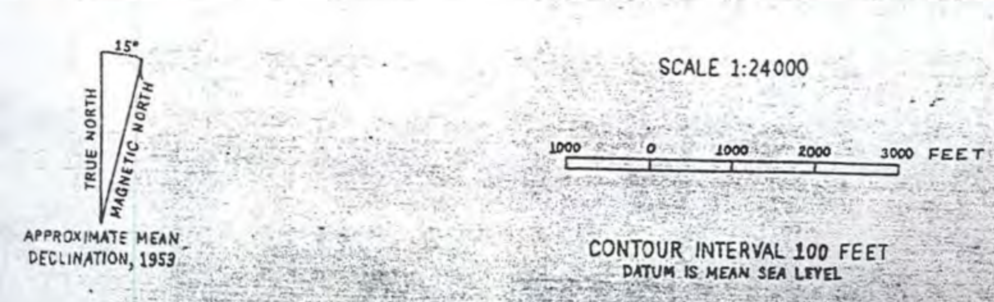


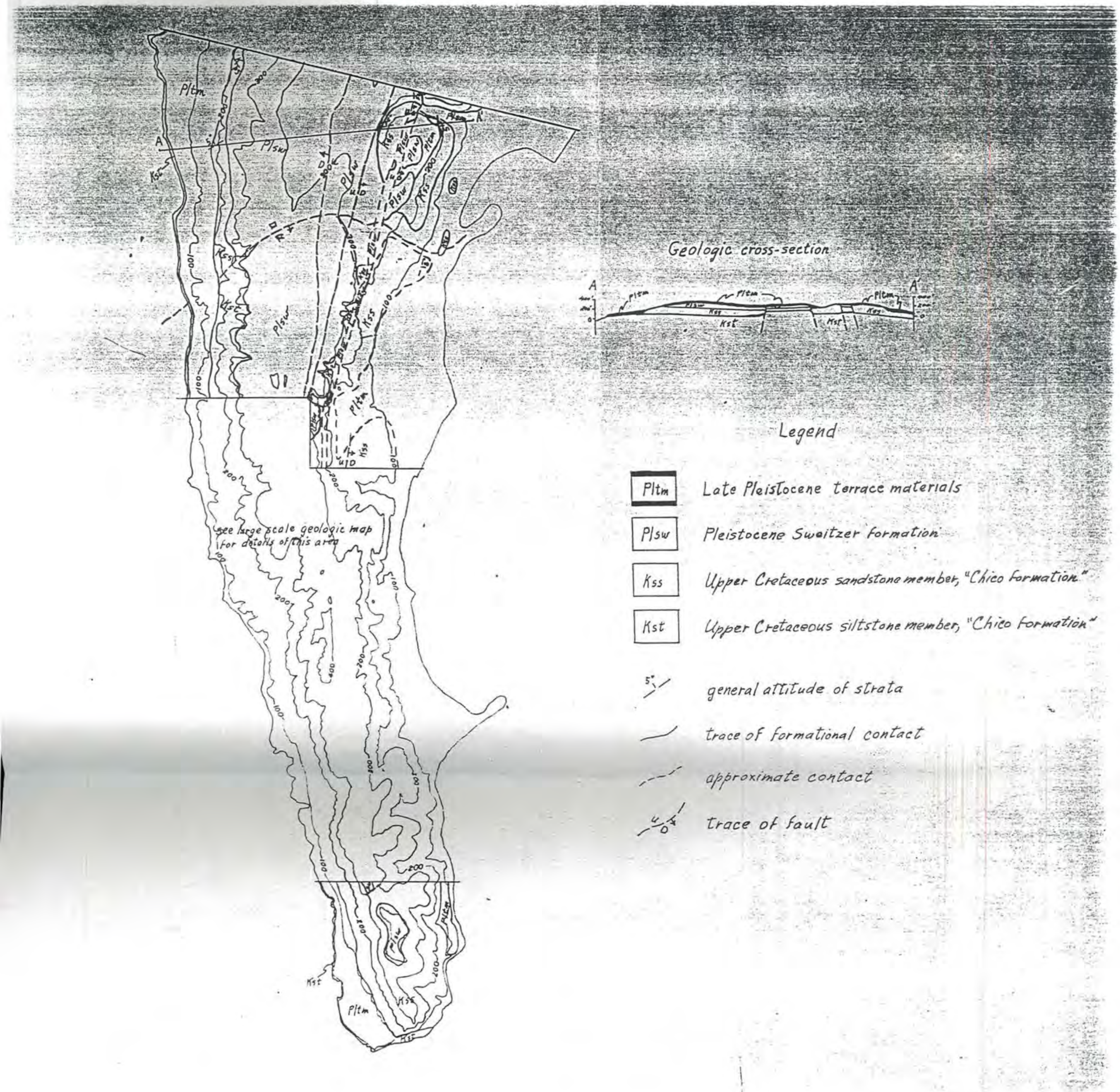
Head of tributary canyon above and west of McClellan Road - Rosecrans Blvd. intersection showing Sweitzer formation (Plsw) in fault contact with the Cretaceous sandstone member (Kss) and unfaulted, overlying fossiliferous Late Pleistocene terrace materials (Pltm) following older topographic surface and forming thick accumulations in depressions of this old surface. Note that the distribution of the Pltm has also been modified and incised by more recent erosion.



Reconnaissance geologic map of a portion of Point Loma, California

by E. Dean Milow





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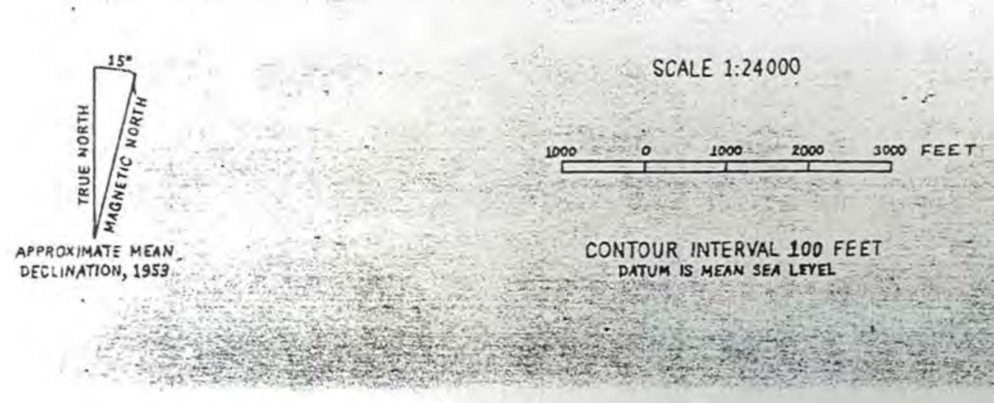
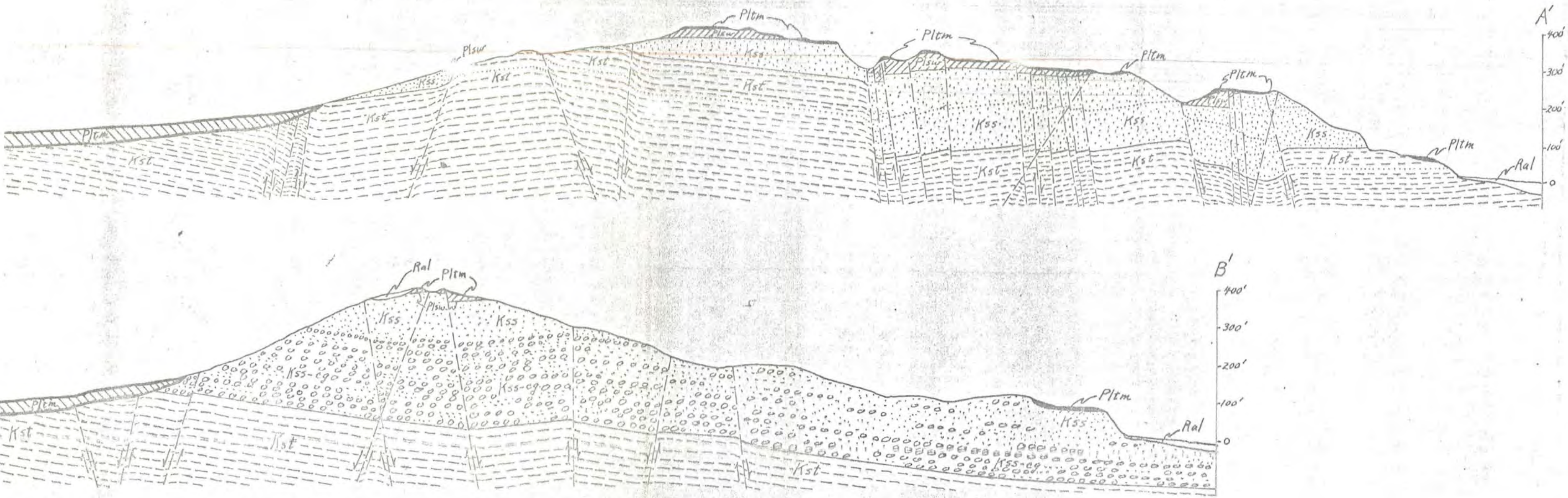


Cross-sections of lines A-A' and B-B' on
Geologic map of a portion of Point Loma, California
by E. Dean Milow

scale 1:3000

Ral	Recent alluvium and man-made fill
Pltm	Late Pleistocene terrace materials
Plsw	Pleistocene Sweitzer formation
Kss / Kss-cg	Upper Cretaceous sandstone member, "Chico Formation"
Kst	Upper Cretaceous siltstone member, "Chico Formation"
	Fault plane



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	Fault plane

Terra Costa Walt.
858-573-6900 Crampton.

SHORELINE PROTECTION PROJECT
POINT LOMA TREATMENT PLANT
SAN DIEGO, CALIFORNIA

Prepared for
CITY OF SAN DIEGO
Water Utilities Department

RESOLUTION NO. R-269683

Project No. 1089-ES01
May 6, 1988

pton

GROUP DELTA CONSULTANTS, INC.

ACKNOWLEDGEMENTS

The authors would like to thank Pountney & Associates, Inc. for their preparation of the topographic base maps, assistance in developing the offshore bathymetry, and for their contributions to the alternate concept designs. We also wish to thank Mr. Gerry Bader and the other members of URS Corporation who participated in the identification and evaluation of alternate design concepts for coastal protection, and for their review of the coastal design criteria.

We are also grateful to Ms. Debra Frischer for her coordination with the various regulatory agencies and for solicitation of their initial comments regarding preliminary design concepts. Ms. Frischer has also suggested preliminary environmental constraints which were used in our evaluation of alternate design concepts to ensure compliance with general coastal policies.

Special thanks go to Mr. Walt Waldorf of Scripps Institution of Oceanography for his contributions to the wave data presented in this report, including offshore bathymetry from data provided by the National Oceanographic and Atmospheric Administration (NOAA) and for his transformation of deep ocean wave spectra to local deep water wave spectra, and for his computer modeling refracting various local deep water waves onto the NOAA nearshore bathymetry.

Special thanks also go to Mr. Gerald Kuhn of Scripps Institution of Oceanography for his invaluable assistance and critique of our methodologies for developing rate of bluff retreat. Mr. Kuhn also provided several photographs, maps and other records from his personal library to assist us in our historical correlations of bluff retreat rates. We also wish to thank Mr. Kuhn for allowing us to reproduce, herein, a figure previously published by Emery and Kuhn in the Geological Society of America, July 1982.

We wish to also express our thanks to the various personnel in the Water Utilities Department with the City of San Diego who assisted us on this project and provided their past experience with erosion processes in the Plant area, and their observations regarding performance of the existing coastal works in the Plant vicinity.

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Project No. 1089-ES01
May 6, 1988

CITY OF SAN DIEGO
Water Utilities Department
Executive Complex
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Suite 400
San Diego, California 92101

Attention: Mr. Michael D. Havrilla
Assistant Civil Engineer

SHORELINE PROTECTION PROJECT
POINT LOMA TREATMENT PLANT
SAN DIEGO, CALIFORNIA

RESOLUTION NO.: R-269683

Gentlemen:

In accordance with our Consultant Agreement, we have completed Phase 1 studies associated with the preparation of construction documents, environmental documents, and permit acquisition for certain shoreline and upper bluff stabilization measures considered necessary to protect existing improvements and maintain access at the Point Loma Treatment Plant in San Diego, California.

The accompanying report presents the results of the various engineering support studies, site information and constraints, and alternate concept designs for shoreline and upper cliff stabilization at the Point Loma Treatment Plant.

If you have any questions or require additional information, please give us a call.

GROUP DELTA CONSULTANTS, INC.

Very truly yours,

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

Braven R. Smillie
R.G. 402, C.E.G. 207

WFC/BRS/jg

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SHORELINE PROTECTION PROJECT
POINT LOMA TREATMENT PLANT
SAN DIEGO, CALIFORNIA

RESOLUTION NO. R-269683

1.0 INTRODUCTION

The City of San Diego owns and operates the metropolitan sewerage system which currently provides service to a population of approximately 1.5 million customers in San Diego and 16 surrounding municipalities and sewerage districts. In 1963, with the opening of the Point Loma Wastewater Treatment Plant, the City now processes approximately 165 million gallons of sewage on a daily basis, and discharges the treated effluent into the ocean through a 9-foot-diameter pipe extending approximately 2.5 miles offshore to a water depth of 200 feet.

The Point Loma Treatment Plant is situated on approximately 37 acres of land, with approximately 2,150 lineal feet of ocean frontage and is located approximately 3/4 miles northerly of the Point Loma Lighthouse. Coastal bluffs in this area rise to over 90 feet above sea level, and many of the improvements associated with the wastewater treatment plant extend relatively close to the bluffs.

Erosion and cliff retreat are ongoing processes along the San Diego coastline. At the Point Loma Treatment Plant, limited amounts of rock slope protection have been placed during, and several times since, construction to help control erosion. By 1984, stone revetments had been placed at the base of approximately 50 percent of the bluffs supporting the wastewater treatment plant.

Erosion continues to encroach upon improvements at the plant and the City of San Diego now desires to upgrade coastal protection in the vicinity of the treatment plant to mitigate further erosion in this area. Additionally, an undesirable amount of wave runup currently disrupts the facilities in the vicinity of the outlet structure during periods of high tide and high surf. Mitigation of wave runup in this area is desirable. Several new retaining walls are also desired landward of the hydro access road, and southerly of the gas utilization facility, to improve slope stability and maintain usable space in these areas.

The City is very interested in arresting future shoreline erosion and, to this extent, wants coastal protective works at all locations within the plant limits where continued erosion within the next 75 years could affect improvements on site. The City has no desire to reclaim additional land; however, upper bluff stabilization is considered necessary to stabilize the near-vertical and, in some areas, over-vertical upper portions of the bluffs within the plant area. As a minimum, the Armco Binwall in the vicinity of the Administration Building will require rehabilitation or replacement, and the lower hydro access road will require stabilization to preserve access down to the lower hydro electric powerhouse.

Group Delta Consultants, Inc. (GDC) was retained to provide construction documents, environmental documents, and permit acquisition for the coastal protective works ultimately proposed for shoreline stabilization in the vicinity of the Plant. This report presents the results of GDC's various engineering support studies including site information and constraints, and concept designs for proposed stabilization of the coastal bluffs in the vicinity of the Point Loma Treatment Plant.

2.0 SCOPE OF WORK

The scope of work has been performed in general accordance with the Consultant Agreement with the City of San Diego, Document No. R-269683, filed on November 9, 1987 with the Office of the City Clerk, San Diego, California.

Specifically, the scope of work includes that effort necessary for providing construction documents, environmental documents and permit acquisition for certain shoreline and upper bluff stabilization measures considered necessary to protect existing improvements and maintain access at the Point Loma Treatment Plant. This document describes the results of the various engineering support studies, site information and constraints, and alternate concept designs for shoreline and upper bluff stabilization at the Plant. The specific tasks to be accomplished during this phase of work include:

- DEVELOP SITE INFORMATION AND CONSTRAINTS
  - Data Collection From Various Agencies;
  - Field Surveying & Photogrammetry;
  - Bathymetry;
  - Utility Research & Plotting on Base Map;
  - Field Editing;
  - Preparation of Base Map;
  - Geotechnical Investigation;
  - Estimate Rate of Bluff Retreat;
  - Develop Design Waves; and
  - Initial Environmental Constraints;
- PRELIMINARY STUDIES
  - Identification & Evaluation of Alternate Design Concepts;
  - Concept Designs & Cost Estimates;
  - Environmental Constraints; and
  - Regulatory Constraints.

This report includes the technical background for alternative design concepts considered feasible, and includes preliminary cost estimates associated with each alternate. This report further addresses:

- The effectiveness of proposed coastal protection works;
- The necessity for structures along the plant limits;
- The economy of patented wall alternatives, especially landward of the hydro access road;

- Probable post-construction maintenance requirements;
- Degree of physical impacts, beneficial or adverse, on abutting property;
- Impact on existing infrastructure; and
- Compliance with City of San Diego standards and Army Corps of Engineers standards.

3.0 ADDITIONAL WORK AFTER APPROVAL OF THIS DOCUMENT

The preferred design concepts will be presented to the other permitting agencies (Coastal Commission, Army Corps of Engineers, etc.) and to agencies with interests/concerns in regard to the project. Comments will be reviewed and utilized as appropriate in modifying this document for use in the initial review process stages.

Environmental Scoping

The City Environmental Quality Division (E.Q.D.) of the Planning Department has directed that no Initial Environmental Assessment (AEIS) will be required, as this report sufficiently addresses the preferred project to prepare the Scoping Letter for preparation of the Draft Environmental Impact Report (DEIR). The City E.Q.D. will prepare the Scoping Letter after meeting with the consultants and reviewing this document. The City E.Q.D. will also prepare and send the Notice of Preparation.

Initiate Permit/Preliminary Review Process

With the completion of the Scoping Letter and Notice of Preparation for the DEIR by the City E.Q.D., and the completion of the final preparation of the Preliminary Plans and Cost Estimates, the permit process with the City, Army Corps of Engineers and California Coastal Commission will be initiated. During this time, the DEIR will also be prepared and submitted to the City. The DEIR review and hearing process will bring additional comments and may require some modifications to the design proposals (as effective engineering allows). Any changes will be further reviewed with the City, Coastal Commission staff, and the Corps prior to the preparation of the Final EIR and the submittal of permit application and documents to the Coastal Commission. Modifications will be made to already submitted applications and plans as required with other agencies. Application for Coastal Commission Permit will not be accepted until all local agency discretionary permits have been approved and the EIR is certified.

Follow-up will occur during the permit processing period with all agencies requiring permits, so that no unnecessary delays will occur because of any missing materials, information or questions.

Hearings will be attended by the key members of the consulting team. It is anticipated that there will be at least one Coastal Commission Hearing (possibly two). Coastal Commission approval will take from 4 to 8 weeks, once the filing is considered complete (once all other agencies' discretionary permits have been approved and received).

Project approval at the City can occur with the certification of the Final EIR. EIR review and certification, from submittal of the DEIR through to publishing of the Final EIR, can take as long as 6 months or more.

The Army Corps of Engineers will require a Section 404 Permit and a Section 10 Permit, along with at least the EA. Processing time at the Corps is approximately 100 days (or 3.5 to 4 months).

The need for other agency permits will not be known until the specific project design proposals have been brought to those agencies for review. Additional permit acquisitions are unlikely.

<u>Environmental Documents</u>
A Draft Environmental Impact Report (DEIR) shall be prepared in compliance with the California Environmental Quality Act (CEQA). Preliminary environmental evaluations for both the City of San Diego and the Corps of Engineers will be prepared to make a determination of significance. We assume that the Corps of Engineers

will utilize the CEQA EIR as an environmental assessment to determine their permit requirements. We have assumed that the Corps will not require a joint NEPA/CEQA EIS/EIR.

<u>30%, 90% and Final Submittal</u>
After preliminary design alternatives have been selected and the environmental process completed, we shall prepare preliminary plans, outline specifications and cost estimates for the 30 percent submittal. To the extent possible, environmental concerns and the various regulatory agency concerns will be incorporated into these design documents. After City, Corps of Engineers, and Coastal Commission review, 90% and, ultimately, final construction documents will be prepared.

4.0 FIELD STUDIES

Field studies for this phase of work were conducted during the period between November 1987 and March 1988. Field studies included a detailed geologic mapping of the site, the field work associated with geotechnical studies for design of both tied-back walls and conventional gravity walls, site survey work associated with preparation of the topographic map and offshore bathymetry, and a detailed assessment of potential site constraints that could impact proposed improvements. A detailed description of the geotechnical investigation is included in Appendix A. We have completed topographic base maps at a scale of 1 inch equals 20 feet, and have also developed reduced copies of this base map at a scale of 1 inch equals 40 feet and 1 inch equals 100 feet. Reproducible or blueline copies of any of these topographic base maps can be provided upon request.

The existing rock revetment was inventoried at 8 locations to determine existing riprap gradation. Five specific gravity tests were also performed on representative samples of the metavolcanic rock. The mean and standard deviation of specific gravity were found to be 2.84 and 0.03, respectively (bulk density of 177 pcf). The approximate locations selected for rock sampling are shown on the Site Plan and Geologic Map, Figure 1. Results of measured gradations are shown on Figures 2 and 3. As indicated on the figures, the rock revetment in the vicinity of the outlet structure is generally smaller than 1/4 to 1/2-ton in size, while the rock downslope of the lower hydro access road is generally 2 to 6-ton in size.

5.0 GENERAL SITE CONDITIONS

5.1 Existing Improvements

As can be seen on the frontispiece, virtually all of the level ground within the Point Loma Wastewater Treatment Facility is currently being utilized for the processing and treatment of sewage. Access roads into the main facility and down to the lower hydro electric powerhouse front the existing coastal bluffs and, in several locations, have been encroached upon by continuing coastal erosion. Virtually the entire site has been graded since construction of the facility in 1963, resulting in essentially no natural open-space areas on site.

During and after initial construction of the facilities in 1963, riprap was placed over approximately 40 percent of the shoreline in front of the plant facilities in order to reduce erosion in this area. In general, the existing revetment is comprised of good quality, angular, metamorphic quarry rock, with no concrete rubble or other debris common to much of the coastal protection found throughout Southern California.

5.2 Topography and Bathymetry

Point Loma is a long promontory extending approximately 6 miles southward from the low land adjacent the San Diego River. Parts of its western shoreline are bordered by a narrow terrace with a top elevation ranging from 25 to 95 feet above sea level. The shoreline of Point Loma is irregular due to differences in geologic structure and in rock hardness. Wave erosion has etched out less resistant rock masses, resulting in shallow pocket coves between rocky headlands. Small pocket beaches have sporadically formed in areas where sufficient sand is available.

Offshore, the sea floor is comprised of the sedimentary rocks of the Point Loma Formation. Isolated, erosion-resistant stacks exist seaward of the intertidal zone, resulting in isolated topographic highs that cross a ledgy shelf surface. Seaward ledges become progressively deeper, interspersed with surge channels typically approaching the shoreline along trends of the major geologic joint sets which control the erosion resistance of this formational unit.

Offshore bathymetry was measured at point locations, extending from the base of the existing rock revetments, out to a distance of approximately 400 feet offshore. Average sea-floor elevations were recorded acknowledging that a blocky and fractured sea-floor surface exists throughout this portion of the coastline. Bathymetric spot elevations are shown on the Site Plan and Geologic Map, Figure 1.

5.3 Geology

Three geologic formations, two natural surficial deposits and two types of man-placed earth materials are present at the Point Loma Treatment Plant. The areal distribution of these units is shown in Figure 1 (Site Plan and Geologic Map). The geologic formations are the Point Loma and Cabrillo Formations of Cretaceous age, and the Bay Point Formation of Quaternary age. The natural surficial deposits are shingle beaches and slump zones along the sea cliff. The man-placed earth materials are comprised of artificial fill and rock revetments. The following sections describe these units in order from oldest to youngest.

Point Loma Formation: The Point Loma Formation is an approximately 900-foot-thick (Kennedy, 1975) sedimentary layer that discontinuously crops out in coastal areas of northern Baja California and as far north as Carlsbad. At the site, it forms the lower, more resistant parts of the sea cliff up to elevations of 54 to 60 feet, and it dips into the sea cliff at about 8 to 12 degrees. The Point Loma Formation extends seaward, comprising the shore platform, and extends inland beneath the coastal terrace. The term "shore platform" is described in the Glossary and in Appendix C.

The Point Loma Formation consists of well-indurated marine sediments deposited by an offshore and deep-water submarine fan. Offshore deposits are represented by the thin-bedded siltstone and fine sandstone exposed in the upper part of the sea cliff. Deep-water deposits are represented by the erosion-resistant thick-bedded mudstone and sandstone exposed at the base of the cliffs. The Point Loma Formation ranges in age from approximately 70 to 80 million years within Rosario Group rocks of the Upper Cretaceous Period.

Cabrillo Formation: The Cabrillo Formation is a 560± foot thick sedimentary deposit that discontinuously crops out in coastal San Diego County from the southern tip of Point Loma to Carlsbad. At the site, it forms the slopes east of the coastal terrace on which the Plant is situated. The formation consists of moderately-indurated, massive marine sandstone and conglomerate deposited in the nearshore area of a submarine fan. The Cabrillo Formation conformably overlies the Point Loma Formation. The age of the Cabrillo Formation ranges from approximately 66 to 70 million years within the Rosario Group rocks of the Upper Cretaceous Period.

Bay Point Formation: The Bay Point Formation, deposited on the coastal terrace on which the Plant is built, ranges up to approximately 35 feet in thickness and forms the upper part of the sea cliff above elevations of 54 to 60 feet. The cliff-forming section of Bay Point Formation is approximately 20-feet thick and is comprised of locally derived marine sandstone. The upper section is likely to be non-marine sandstones, which form moderate slopes up to the 90 to 95 foot elevation of the coastal terrace.

The Bay Point Formation is comparatively restricted in age to 120,000 years plus or minus several thousand years. It was deposited on an ancient wave-cut platform formed during the last interglacial period when worldwide sea level was approximately 20 feet higher. Geologic evidence indicates that, since deposition of the Bay Point Formation, Point Loma has been uplifted approximately 40 feet at a rate of about 0.4-inches per 100 years.

Beach Deposits: The cove near the north boundary of the site contains a pocket beach consisting of gravel and cobbles. This coarse-grained beach, known as a shingle beach, sits on the shore platform and is estimated to be 5 to 10-feet in thickness.

Slumps: Slump-fall materials are located in the coves near the northern property limits downslope of the hydro access road and near the southerly visitor parking lot. They consist of Bay Point Formation soils that have fallen from the upper bluffs to the back of the coves, forming moderately-steep slopes of loose material.

Artificial Fill: Artificial fill, placed by man as opposed to filling by sedimentary deposition, is exposed continuously along the top of the sea cliff. These fills were placed to expand the useable flat area of the coastal terrace. The fills are generally 5 to 15-feet thick, but are up to 25-feet thick behind the existing binwall and up to 90-feet thick in the major filled cove beneath the existing gas utilization facility.

Rock Revetments: Rock revetments extend along portions of the base of the shoreline and sea cliff. These materials consist predominantly of angular, metamorphic quarry rock, which measures up to 5½ feet in maximum dimension, that was placed as shore protection.

5.4 Wave Climate

Determining the wave potential at a given coastal location requires a number of critical assumptions regarding the budget of deep water waves, the sheltering effect of offshore islands, and the refraction of waves in water of variable depths. Waves that break along the San Diego County shoreline generally range in height from 2 to 5 feet; however, large waves ranging from 6 to 10 feet in height are not uncommon. Such large waves can be expected to arrive at almost any time during the year and to continue for 3 to 4 days at a time. These high-wave episodes are frequently unaccompanied by strong winds. Breakers with estimated heights of 15 to 20 feet have been observed off the coastline within the study area.

The shoreline from La Jolla to Point Loma is exposed to wave action, unaffected by island interference, through three relatively well-defined corridors of wave approach. Waves with periods of 8 seconds or shorter have an unobstructed approach from the northwest between Santa Rosa Island and San Nicholas Island, from the west between Tanner Bank and Cortez Bank and between Tanner Bank and San Clemente Island, and from the south and southwest between Cortez Bank and Los Coronados Islands. Short period waves also approach this shore segment from the northwest between Santa Cruz Island and the mainland with a limited fetch of 130 to 140 nautical miles.

The study area is somewhat sheltered from deep-water waves with periods longer than 10 to 12 seconds. Large deep-water waves are refracted around the channel islands and shoal waters off the California coast and ultimately reach the coastline as somewhat smaller and directionally modified local deep-water waves. Historical extreme-wave events have been summarized in Appendix B and mathematically refracted through the continental shelf region to produce local deep-water waves that approach the site. These local waves were then analyzed approaching Point Loma using seafloor bathymetry available through NOAA and using a refraction computer model at Scripps Institution of Oceanography. A description of this technique is also included in Appendix B.

5.5 Winds

Sea breezes attaining velocities of 10 to 20 miles per hour blow landward across the shoreline nearly every afternoon. Reciprocal land breezes at night have much lower velocities. Storms moving in from the Pacific Ocean occasionally bring somewhat stronger winds to the San Diego area, but their duration is relatively short. Tropical cyclones from the south reach San Diego on rare occasions. Winds along the coastline within the study area come predominantly from the west, northwest, and southwest, respectively; average wind velocities are less than 10 miles per hour. Extreme sustained wind speeds approaching 50 knots are expected off the Southern California coast below 35 degrees latitude statistically once in 100 years (NOAA, 1980).

5.6 Rainfall

Semi-arid conditions prevail in the San Diego area. Most rainfall occurs between November and March. Average annual rainfall ranges from approximately 9 to 10 inches. The maximum recorded rainfall is approximately 26 inches (1883-84).

5.7 Tides

The tides along the Pacific coast have a semidiurnal inequality. The lowest tide each year is about -1.7 feet (MLLW Datum). The highest tide is about 7.3 feet, MLLW Datum (+4.42 feet MSL Datum).

5.8 Storm Surge and Wave Setup

Extreme storm surges are presented as a function of return period at selected California tide stations in the 1980 NOAA study with those for La Jolla shown below:

Return Period Years	Storm Surge Feet
5	2.0
10	2.1
25	2.2
50	2.3
100	2.4

Storm surge when combined with tidal variations results in a statistical extreme water elevation composed of astronomical water surface and storm surge as follows (NOAA, 1980):

Return Period Years	Extreme Water Elevation Feet (MSL Datum)
5	4.4
10	4.5
25	4.6
50	4.7
100	4.8

Wave setup results from the superelevation of the water surface over the normal surge elevation due to onshore mass transport of the water by wave action alone. Calculations for the design conditions herein indicate a wave setup ranging from 1.0 to 2.0 feet.

6.0 SHORELINE EROSION

6.1 Lower Bluff Erosion

The Point Loma Formation is exposed along the entire base of the sea cliffs in the study area; it is vertical to near-vertical in most areas and is 54 to 60 feet high. Erosion at the base of the cliff, up to approximately elevation +10 feet (MSL), is due predominantly to direct wave impact acting upon small joints and fissures in the massive rock unit and by water-hammer effects. Much of the Point Loma Formation is quite intact and appears to have experienced little erosion in the last 50 years. In other areas, where fractures and joints in the rock are more prevalent, erosion is occurring more rapidly. Where shear zones are present, surge channels and caves have developed.

6.2 Upper Bluff Erosion

The upper bluffs are comprised of the Point Loma Formation above approximate elevation +10 to +20 feet (MSL) and the overlying Bay Point Formation beginning at elevations ranging from approximately 54 to 60 feet. The Bay Point sands form the upper portion of the bluffs and are approximately 35 feet in thickness. These sands are subject to several different forms of erosion as a result of the following actions:

- Wave spray and wave splash;
- Undermining of the basement rock and caving of the resulting oversteepened slopes; and
- Wind, rain, irrigation, and uncontrolled surface runoff.

The upper bluffs, which support little or no vegetation, are exposed to the elements throughout most of the site. Wave spray and splash often reach these unprotected sands, causing saturation of the outer layer and subsequent sloughing of oversteepened slopes.

In areas where the Point Loma Formation is experiencing erosion at the base of the sea cliff, the overlying upper bluffs become undermined and subsequently fail through loss of vertical support. This results in oversteepened slopes that stand nearly vertically. The Point Loma Formation and the lower cliff-forming section of marine Bay Point Formation form the existing sea cliff. The upper slope-forming section of Bay Point Formation stands vertically, until the pore-water tension within the soil has had a chance to dissipate; then sloughing occurs. The slopes are relatively stable when they attain inclinations of about 1 to 1.

Wind, rain, irrigation, and uncontrolled surface runoff contribute to minor erosion of the upper cliff face, especially on the more exposed, oversteepened portions of the friable sands. A considerable amount of rilling has occurred along portions of the upper cliffs as a result of these actions.

6.3 Rate of Bluff Retreat

When studying and reporting sea cliff erosion and retreat, care must be taken to distinguish between cliff retreat rates based on: (a) bluff or cliff top retreat, (b) shoreline or cliff base retreat, and (c) averages between the top and bottom at various locations along the cliff. The degree of erosion can vary significantly from spot to spot on a sea cliff, and is influenced by many independent and dependent variables (that is, lithology, joints or fracture characteristics, beach configuration, offshore bottom conditions, climate, impacting wave configuration and energy, and human effects). Because erosion does not necessarily act uniformly over a sea cliff, nor necessarily at a uniform rate, the lack of clarification of the basis for the qualitative erosion rate values can lead to confusing and misleading results.

Kennedy (1973) provides a good general discussion of the erosion processes and forces acting on the Point Loma peninsula. Based on a comparison of old and new photographs, Kennedy reported that 75 percent of the sea cliff area has undergone no appreciable erosion during the last 75 years; only 20 percent of the cliff has undergone very rapid retreat of 10 feet in the last 75 years (0.13 feet per year), with nearly 5 feet of retreat occurring in the late 1940's. Kennedy's reported average erosion rate was 3 feet in 75 years (0.04 feet per year).

6.3.1 Site-Specific Bluff Retreat Rate

In order to evaluate the rate of bluff retreat in the vicinity of the Point Loma Treatment Plant, a review was made of the following data:

- Stereographic aerial photographs from 1939 to the present;
- Pertinent Historical Society photographs and supporting data (some of which were taken as early as the 1800s);

- Topographic maps and supporting field notes dating back to 1859;
- Applicable geologic and geotechnical literature;
- Historical storm data; and
- Wave climate.

A detailed geologic site reconnaissance was then performed to map sediments exposed in the bluffs in order to develop an understanding of the soil characteristics and strength of individual stratigraphic units, the bedding attitudes, faulting, joint and fracture patterns, and to look for evidence of perched groundwater seepages. An inventory was also made of adjacent and nearby bluffs in order that they could be compared to the site-specific stratigraphy, structure, slope geometry and stage of development.

After evaluating the data collected, geologists and oceanographers most conversant with the Point Loma shoreline processes were contacted at the following agencies:

- The Army Corps of Engineers;
- The United States Geological Survey;
- Scripps Institution of Oceanography;
- San Diego County; and
- The State of California.

Based on a review of the available data, our geologic inventory of the site vicinity, and discussions with other experts, we have developed a design rate of bluff retreat and a 50 and 75-year bluff retreat line. A detailed description of the methods used for determining the bluff retreat rate is included in Appendix C. The 50 and 75-year no project bluff retreat line is shown on the Site Plan and Geologic Map, Figure 1.

6.3.1.1 Effect of Existing Stone Revetment

The presence of a stone revetment at the base of the coastal bluffs mitigates direct wave impact onto the bluffs. Although direct wave impact is reduced, wave runup and the attendant splash is likely increased, which contributes to both lower bluff and upper bluff erosion. Since construction of the Plant in 1963, there have been several periods of riprap placement with the net effect of further reducing coastal erosion. We have reviewed available stereographic aerial photographs from the following dates:

Stereographic Aerial Photographic Coverage

Date of Photograph	Photographic Scale
1987	1:12,000 (photographically enlarged to 1"=200')
1986	1:12,000
1985	1:40,000
1982	1:24,000
1981	1:24,000 (photographically enlarged to 1"=200')
1978	1:40,000 (photographically enlarged to 1"=200')
1972	1:20,000 (photographically enlarged to 1"=200')
1964	1:24,000 (photographically enlarged to 1"=200')
1960	1:24,000 (photographically enlarged to 1"=200')
1953	1:24,000 (photographically enlarged to 1"=200')
1950	1:24,000 (photographically enlarged to 1"=200')
1949	1:20,000
1939	1:24,000 (photographically enlarged to 1"=200')

Since erosion rate was, in part, evaluated by the cliff retreat rate based on successive photo evaluation and historical storm data (Table B-2, Appendix B) the effect of existing stone revetments on bluff retreat rate was evaluated based on the annualized reduction in retreat rate after placement of rock when compared to the pre-1963 data. This information has also been incorporated into the evaluation of our 50 and 75-year bluff retreat lines shown on Figure 1.

7.0 DESIGN CONSIDERATIONS

In order to determine an appropriate shoreline cliff stabilization program, especially in an area of extreme environmental sensitivity, it is important to address the problems of visual aesthetics; current uses of the area; the present hazards associated with already unstable oversteepened slopes; the potential for future erosion; and the impact of minimal or no stabilization in certain areas. Relevant sections of the California Coastal Act require that coastal protection be limited to only those areas where continued erosion will impact existing improvements. To satisfy this concern, no significant erosion control measures were considered along areas of the coastline that are not already experiencing cliff retreat or in areas where erosion is not expected to endanger existing improvements within the plant facility. Proposed improvements have been limited to only those considered necessary to assure the long-term and uninterrupted access of the main access road and lower hydro access road, both of which front the upper bluffs throughout the Plant limits. Of the 2,150 feet of total shoreline frontage within the Plant facility, proposed improvements to coastal protection are only being considered along approximately 775 lineal feet.

The main erosion problems at the base of the sea cliff are associated with the direct impact of waves and/or wave runup in the areas where joints and fractures are present in the Point Loma Formation. Various types of seawalls, grouting and surface treatments were considered to protect these areas. To date, coastal protection (which currently exists along 1,025± feet of the coastline) is limited to the use of rock revetments at the base of near-vertical coastal bluffs. By the time rock revetments had been placed at the base of these bluffs, lower bluff erosion had advanced to the point where portions of the bluffs within the

Plant area were near vertical and, in some cases, over vertical, as in the vicinity of the Armco Binwall adjacent the Administration Building.

Due to the localized near-vertical nature of the upper bluffs which encroach onto the existing access road and lower hydro access road, consideration has been given to the construction of vertical tied-back walls, designed to conform to, and visually blend into, the existing near-vertical bluffs.

7.1 Flexible vs. Rigid Structures

All of the existing coastal protection at the Point Loma Treatment Plant consists of rock revetment, which is considered to be a flexible structure. Moreover, in front of the outlet structure and downslope of the gas utilization facility, the existing exposed ground surface is comprised of rock (and soil fill) and, as such, is capable of consolidating somewhat. When considering new coastal protection structures in these areas, it is necessary that these structures be designed to accommodate differential movements when supported on foundation elements that are themselves flexible. For this reason, composite structures must be compatible and, when used with rock, the appurtenant structures must also be as flexible as the revetment foundation. It should be pointed out that the rock revetment system is probably the most durable and economical form of shoreline protection and it is used throughout the world. One of its few design disadvantages is in its foundation performance for composite systems. As such, composite systems utilizing rock revetments necessitate the use of a semi-flexible wall, such as Reinforced Earth*, a cribwall, or articulating gravity wall.

Along all but the northerly portion of the Plant limits, where the coastal bluffs are set back somewhat from the shoreline, special consideration can be given to economical forms of lower shoreline

* Patented Product

protection, combined with upper bluff stabilization techniques, using either rigid structures such as concrete walls or tied-back walls, or semi-rigid structures such as binwalls, cribwalls or Reinforced Earth.

The use of conventional cantilevered concrete seawalls in any of the locations currently proposed for coastal stabilization requires removal of all existing rock down to the underlying bedrock in order to provide adequate foundation support for conventional cantilevered concrete seawalls.

7.2 Revetment Design

A stable riprap design section requires consideration of such factors as the maximum anticipated deep-water design wave height and wave period that could be expected to occur over the life of the structure. Upon reaching the coastline, the design wave reaches a depth of water so shallow that the waves collapse or break. This depth is equal to about 1.3 times the wave height. During periods of extreme high tide, small swells of approximately 2 to 4 feet in height may actually maintain most of their wave energy and break directly on the structure. During periods of heavy storms, where deep-water wave heights are tens of feet high, these waves break quite a distance offshore, reform as smaller waves, and eventually impart a portion of the original wave energy onto the shore protection structure.

Wave characteristics are normally determined for deep water and then propagated shoreward to the structure (Appendix B). Deep water significant wave height and significant wave period may be determined if wind speed, wind direction and fetch length are known. This information, with water level data, is used with refraction analyses to determine wave conditions at the site. Wave conditions at a site depend critically on the water level and the corresponding sea-floor elevation at the base of the structure. Consequently, knowledge of sea-floor bathymetry and the design still water level (SWL) must be established to evaluate the wave forces on a coastal structure.

7.3 Tied-Back Walls

To arrest and control future erosion of the coastal bluffs, tied-back walls have been considered below the lower hydro access road where bluff erosion has encroached onto the roadway; below the Armco Binwall where erosion has begun to undermine the Binwall; and southerly of the visitor parking lot where erosion of the upper bluff is beginning to encroach on the access road.

The construction of tied-back walls involves surface preparation of the bluff face, installation of steel anchors and reinforcement, and application of a structural surface, such as shotcrete, to the bluff surface. The resulting wall can then conform to the irregular surface of the existing bluff. Pigments can be used in the shotcrete mix to produce an earthen tone comparable to the natural bluff face. Specific structural design requirements depend on the geotechnical constraints specific to the site (Appendix A).

7.4 Resurfacing of Armco Binwall

An existing Armco Binwall* exists along the edge of the bluff in the vicinity of the Administration Building. The binwall is constructed of galvanized steel bins coated with an asphalt concrete film to retard corrosion. At present, portions of the outer skin have corroded to the point where the soil backfill within the bin is now exposed. As part of our field studies, the rear portion of the binwall was unearthed at approximately mid-length of the wall and the rear stringers and stretchers exposed in the excavation. After examining the internal elements of the wall, it was apparent that corrosion had removed the galvanizing and caused loss of base metal in the possible range of up to 25 percent. After reviewing the long-term corrosion potential of the steel Binwall with Mr. Joe Mataich, a corrosion engineer with the Water Utilities Department, it was concluded that the entire

*Patented Product

binwall would likely disintegrate within the next 50 years. As such, the gravity structure will likely fail in the near future, requiring a total rehabilitation or total replacement of the wall.

In order to rehabilitate the existing binwall, consideration must be given to future loss of the existing gravity mass, resulting in load transfer to the new structure, which is equivalent to the lateral earth pressure resulting from the in-situ soil being restrained by the new surface treatment. Design details are provided in the Geotechnical Investigation, Appendix A.

7.5 Reinforced Earth Walls

A Reinforced Earth wall has been selected for protection of the fill slope below the gas utilization facility. This type of retaining structure is economical, very flexible, and can easily be articulated along the coastline to decrease the visual impact. Since the precast concrete facing elements serve only to retain soil, numerous surface textures and panel geometries are available. The current concept utilizes a 20-foot-high wall embedded 6 to 9 feet below the existing rock surface with a top-of-wall elevation of +38 feet. Soil backfill will be placed behind the wall extending up to the hydro access road at an inclination of 2 to 1. A technical description of the Reinforced Earth wall concept is presented in Appendix D.

7.6 Geotechnical Constraints

Site-specific geotechnical studies were performed to address the geotechnical requirements for design of tied-back walls and gravity walls. The geotechnical factors influencing the design of a tied-back wall system includes the stability of the slope, the strength of the anchor zone, the anchor system, and the forces acting on the tied-back wall system. In design of a gravity structure, geotechnical considerations include the bearing capacity and associated settlement, and earth pressures acting on the gravity structure. A detailed description of the geotechnical design aspects effecting the proposed structures is presented in Appendix A.

8.0 PRELIMINARY ENVIRONMENTAL CONSTRAINTS

A number of government agencies will be involved in the review process of the Point Loma Sewage Treatment Plant Shoreline Protection project. They have either direct or indirect environmental concerns with regard to the work proposed, as well as some having permitting authority over the project.

Several phone conversations and meetings have occurred with key agency personnel to review the preliminary proposals with them and to get their input and concerns. Preliminary discussions were also used to help determine which permits would be required and what types of reviews might occur.

The agencies that have been contacted in preliminary discussions include the following:

- U.S. Army Corps of Engineers;
- U.S. Park Service;
- National Marine Fishery Service;
- State Department of Fish and Game;
- U.S. Fish and Wildlife Service;
- Environmental Protection Agency;
- California Coastal Commission; and
- City of San Diego - Environmental Quality Division of the Planning Department.

Additional agencies were contacted, but stated they had no jurisdiction or concerns in regard to the project.

8.1 Initial Environmental/Agency Concerns

Due to weather problems, an on-site biological review of the project (both terrestrial and marine) has not yet been completed;

therefore, the concerns and constraints in the following discussion are from the various agencies, from preliminary discussions and/or their preliminary review of the project's approach.

U.S. Army Corps of Engineers: The U.S. Army Corps of Engineers expressed two preliminary concerns in regard to the project:

1. That there be no major impacts to the biological community, especially the marine community; and
2. That the protective devices be designed to the minimum amount necessary.

U.S. Park Service: The U.S. Park Service contacts expressed the following concerns in regard to the proposed project:

1. Ocean water quality;
2. Air quality;
3. Short-range construction impacts, including additional deterioration and/or collapse of portions of Cabrillo Road through Park Service land due to increased construction-related traffic;
4. Prevention of construction debris from going into the ocean or onto park land; and
5. Cultural/archaeological survey availability.

State of California Department of Fish and Game: The Department of Fish and Game contact expressed preliminary concerns in the areas of:

1. Impacts on tidal and subtidal areas where rock revetments will (or may) be placed; and
2. What will happen around the edges of the protective devices.

National Marine Fishery Service: The contact with the National Marine Fishery Service had no immediate concerns with the project as proposed, however, would further review the specifics on any designs that encroached into intertidal or subtidal areas.

Environmental Protection Agency (EPA): The contact with the EPA indicated that the project should comply with the Army Corps of Engineers 404 guidelines.

U.S. Fish and Wildlife Service: The contact with the U.S. Fish and Wildlife Service indicated that his involvement was in reviewing the Army Corps of Engineers permits and endangered species concerns.

California Coastal Commission: Coastal Commission contacts in San Diego expressed the following preliminary concerns in regard to the proposed designs for the shoreline protection project:

1. To prove need for each protection location (permits for protective devices are usually only issued when a primary structure is in danger of being damaged or destroyed);
2. Impact on benthic fauna;
3. Examination of existing drainage structures and drainage flow on site to prevent future erosion problems from occurring;
4. Whether or not the sewage treatment plant is in the appropriate location;
5. Whether the hydro electric powerhouse has to be where it is located;
6. Whether the access road can be relocated;
7. That the least amount necessary of protective devices be allowed or proposed;
8. That alternative solutions be shown and discussed;
9. That minimal alterations of landforms occur (e.g., filling in caves rather than using wall device);
10. What the impact of proposed walls will have on shoreline processes and what further impact they will have around ends of walls;
11. Whether vegetative slopes are presently irrigated; and
12. Whether riprap will effect tide pools or tidal land.

8.2 Agency Permits Required

It has been determined (to date) that three permits will need to be acquired:

1. California Coastal Commission Development Permit;
2. U.S. Army Corps of Engineers 404 Permit; and
3. U.S. Army Corps of Engineers 10 Permit.

Prior to submittal of the formal application to the Coastal Commission for permit processing, the final EIR must be approved by the San Diego City Council. The time constraints through the agencies include approximately 100 days for the Army Corps of Engineers process, approximately 6 months for the EIR process and approximately 2 months through the Coastal Commission process.

9.0 CONSTRUCTION CONSTRAINTS

The Point Loma Treatment Plant is a continuously-operating facility having need for continual plant access. Staging areas will be made available to the contractor immediately inside the Plant boundary on approximately 1 acre of land adjacent to, and westerly of, Cabrillo Road. A staging area, approximately 35 feet by 110 feet in plan dimension, will also be available just northerly of Section 2 on the lower hydro access road. The contractor will be required to cooperate with Plant personnel and maintain access down to the lower hydro electric powerhouse, at least on an intermittent basis.

It should be noted that construction of this project will require substantial truck traffic, carrying construction materials and equipment over Cabrillo Road. This impact will be addressed within the EIR.

10.0 PROPOSED SHORELINE STABILIZATION

The preferred shoreline and upper bluff stabilization approach within the Point Loma Treatment Plant boundaries limits proposed improvements to only those areas where continued erosion within the next 75 years will likely impact existing improvements. Specific wall types and dimensions described herein have been selected based on estimates of future erosion rates, the cost and effectiveness of proposed coastal works, and the long-range needs of the Plant. Erosion rates have been evaluated from a geologic inventory of erosion processes along the bluffs, a review of available historical aerial photographs further corroborating erosion processes, and discussions with Plant personnel eliciting their past experience with erosion processes in the Plant area.

Coastal and nearshore improvements are proposed in six areas within the Plant boundaries, as described below, generally extending from north to south. Alternate design concepts considered appropriate for each location are also described herein.

<u>Section 1 - Vicinity of Outlet Structure:</u> In the vicinity of the outlet structure, an undesirable amount of wave runup currently disrupts facilities during periods of high tide and high surf. Additionally, the protective stone in front of the outlet structure is generally comprised of a well-graded quarry stone, much of which is less than quarter-ton in size. The slope of this revetment currently ranges from 2:1 to 3:1 (horizontal to vertical) which exacerbates wave runup due to the effect of ramping. The smaller stones are also hurled toward the powerhouse and outlet structure during periods of increased wave activity.

A more substantial protective rock revetment is currently proposed in front of the outlet structure, having a crown elevation of approximately 17 feet (MSL datum). The revetment would extend a total of approximately 230 feet in length and would incorporate the existing rock in this area as core stone. The revetment would consist of 8-ton armor stone constructed at an inclination of 1.7:1.0. The approximate geometry of this revetment is illustrated on Figure 4.

An 8-inch-diameter reinforced concrete drain pipe will also be installed crossing under the existing asphalt concrete roadway, accommodating nuisance drainage waters which currently discharge through two 8-inch clay drain pipes mid-way up the concrete spillway.

Section 2 - Lower Hydro Access Road Where Upper Bluff Erosion Has Encroached Upon Roadway: The bluffs within the lower portion of the hydro access road have locally eroded to the extent that portions of the roadway surface have been lost, and slight overhangs currently exist in some areas. Continued erosion in this area will likely eliminate future access to the hydro electric powerhouse from the hydro access road. An 85± foot long vertical tied-back wall is proposed in this area, with a 3½ foot high parapet. The wall would extend from the top of the bluff down to the existing rock, approximately 30± feet below. No additional bluff reclamation is proposed. The wall is proposed to conform to the existing near-vertical bluff, provide a barrier wall where it currently encroaches upon the roadway, and be designed to visually blend into the existing coastal bluffs. The approximate geometry of this tied-back wall is illustrated in Figures 5 and 6.

As an alternate to the vertical tied-back wall, consideration has also been given to the placement of additional rock over a length of approximately 100 feet, extending up to an elevation of approximately 25 feet. Although slightly less expensive, this option still results in 5± feet of additional potential future encroachment into the already narrow lower hydro access roadway.

A small sea cave (collapsed at the rear and partially filled-in by riprap) exists just northerly of this section. The approximate limits of the filled-in collapse and the existing "arch" which remains at the front of the cave are shown on the Site Plan and Geologic Map (Figure 1). The sea cave has a current unsupported height of approximately 12 feet above the base of the cave, with an estimated volumetric dimension of 75 cubic yards. The roof-arch at the front of this sea cave is estimated to be approximately 3 to 5 feet in thickness and would likely collapse if driven over by heavy equipment.

Based on examination of the interior of the cave, it appears that the roof rock previously collapsed at the rear of the cave and that an attempt was made to backfill through the collapse with rock and concrete. It is currently proposed to form the front face of this cave and entirely grout-in the cave to eliminate the potential for future collapse.

<u>Section 3 - Upper Portion of Hydro Access Road in Vicinity of Existing Fill Slope:</u> A relatively large sea cave existed at one time beneath the gas utilization facility, which has since been filled in. Where the original mouth of the sea cave necked down to the face of the bluff, it was approximately 100 feet in width. This area has since been filled in and an earthen fill slope now exists in this area, protected by a relatively substantial rock revetment. Continued wave runup in this area has eroded the base of the fill slope and continued erosion will likely undermine the fill slope and the hydro access road in this area.

A 106± foot long, 20 foot high, Reinforced Earth wall is proposed in this area. This wall would have a base elevation of 18 feet and the lower 6 to 9 feet obscured by the existing stone revetment. As such, only 11 to 14 feet of this wall would be visible. A wave deflector would be incorporated into the top of the wall and the wall would have a top-of-wall elevation of +38 feet. The slope above the wall would be regraded at an inclination of 2:1 and relandscaped. A small amount of additional 8-ton rock would also be placed in front of the northerly end of the wall, where local depressions currently exist in the present rock revetment. The approximate geometry of this wall is illustrated on Figure 7.

<u>Section 4 - Vicinity of Armco Binwall:</u> An Armco Binwall currently exists along the edge of the bluff in the vicinity of the Administration Building. The Binwall is constructed of steel bins, portions of which have corroded to the point where soil is now

exposed on the front face of the binwall. Moreover, lower bluff erosion has advanced to the point where the Binwall is locally undermined, suggesting that continuing erosion will eventually cause the entire Binwall to collapse, resulting in loss of the upper access road. As such, a structural tied-back vertical concrete facing is proposed to resurface the entire exposed face of the Armco Binwall.

A vertical tied-back wall is also proposed along the base of the bluff where localized erosion is undercutting the Binwall. This wall will be approximately 60 feet in length and similar in appearance to the tied-back wall proposed along the lower hydro access road (Section 2). This wall would also be protected with approximately 70 feet of additional 8-ton rock revetment which would augment existing rock at the base of the Binwall. Existing rock locally extends up to elevation 22 feet. The proposed additional rock would extend up to, and develop a more uniform crown elevation of +25 feet. The approximate geometry of the binwall facing and lower tied-back wall is illustrated in Figures 8 and 9.

As an alternate to rehabilitation of the existing binwall and/or placement of the vertical tied-back wall below the base of the binwall, consideration has been given to the entire removal of the binwall and replacement with a Reinforced Earth wall. This has several advantages in that the Reinforced Earth wall alternate is more economical than rehabilitation of the existing binwall and, because it is a vertical structure, an identical top-of-wall location results in 5½ feet of additional sacrificial bluff at the base of the wall. After placement of the proposed additional rock up to a crown elevation of +25 feet at the base of this wall, 75-year estimates of bluff retreat in this area are approximately 5 feet.

Replacement of the entire binwall has a singular disadvantage in that a major construction excavation will be required in the vicinity of the Administration Building, requiring temporary relocation of utilities within and adjacent to the existing roadway and possibly temporary relocation of the trailer nearest the binwall. As of this writing, relocation costs have not been determined for this alternate.

Section 5 - Southerly of Southerly Visitor Parking Lot: Coastal erosion in this area is encroaching upon the existing access road to the Plant and will eventually require realignment of the road if coastal protection is not provided. The remnants of a sea cave exist immediately southerly of the visitor parking lot where coastal erosion is most prevalent. Approximately 60 feet of additional 8-ton rock would be placed in this area to augment existing rock and further mitigate localized lower bluff erosion in this area.

The upper portion of the bluff exposes younger, more erodible formational soils underlying from 4 to 6 feet of fill soils, presumably placed during construction of the roadway. Approximately 120± feet of a vertical tied-back wall with a short parapet is proposed along the upper bluffs in this area, extending down to the lower Cretaceous-age formational soils (approximate elevation 60 feet), which are considerably more erosion-resistant. This wall would also be similar in appearance to the wall at Section 2, and the lower wall at the base of the Armco Binwall (Section 4).

Section 6 - Hydro Access Road and GUF Access Road Retaining Walls: Retaining walls are proposed landward of the hydro access road and easterly of the GUF access road immediately southerly of the gas utilization facility. Natural slopes in these areas are locally steeper than 1:1, and continually ravel, reportedly requiring maintenance of adjacent roadways. Additional usable land is desired in these areas and retaining walls will allow for development of additional level ground at the base of the walls. Flexible cribwalls are currently proposed for this area, a detail of which is illustrated on Figure 10.

A 2-inch-thick asphaltic concrete overlay is also proposed to upgrade the existing lower hydro access road. The travelway will also be increased to a 24-foot paved roadway with 5-foot shoulders. Additionally, it is proposed to temporarily remove the engineering trailer from its pad and build-up the pad to correct drainage in this area. The existing concrete cribwall located between the outlet structure and the emergency spillway is also in need of repair and will be rehabilitated as a part of this project.

At the request of City staff, we will include the walls as a deductive alternative for contract bidding purposes in the event that insufficient funds exist for construction of all or part of these walls. In order to maintain 24 feet of usable roadway on the lower hydro access road, the deductive alternate will exclude a 100± foot long wall in the vicinity of Section 2 in order to maintain the necessary minimum roadway width in this area where coastal erosion has encroached upon the roadway. In other words, whether or not the deductive alternate is constructed, the 100± foot long wall in the vicinity of Section 2 will still be necessary.

11.0 DESIGN CRITERIA

11.1 Stone Revetments

Deep-water significant wave heights and wave periods were transformed to local deep-water wave heights and directions for 13 historical storm events considered representative of worst-case conditions for the San Diego County coastline. Refraction diagrams were then developed to model the effect of near-shore bathymetry on the wave height and distribution of wave energy for a series of deep-water wave conditions. In virtually all instances, design wave heights were controlled by sea-floor bathymetry at the base of structures which limited the maximum design wave that could develop in front of structures. Sea-floor bathymetry in front of proposed structures ranges from an elevation of -4 feet to -6.0 feet (MSL datum). The average near-shore slope extending out a distance of 400± feet was on the order of 50:1. In all instances, sea-floor materials were comprised of the sedimentary rocks of the Point Loma Formation, judged to be erosion resistant. For design purposes, we have assumed no additional erosion of these formational materials at the base of structures.

A maximum design still water level of 7.3 feet (MSL) was selected for design which includes both the highest high yearly tide, combined with a statistical 100-year storm surge, 2 feet of wave setup, and 1/2 foot of additional height to account for long-term rise in sea level. This results in a maximum design breaker height of 13 feet, and a required minimum stone size of 8 tons (U.S. Army Corps of Engineers Shore Protection Manual - 1977 Edition, Chapter 7). Runup calculations indicate that design storm waves with periods on the order of 20 seconds may result in as much as 15 feet of wave runup above the design still water level reaching elevations of over 20 feet.

11.2 Tied-Back Walls

Surface preparation of the bluff would consist of the removal of small protrusions which would be difficult to install reinforcement around. Slots would be cut in the bluff surface at the vertical edges of the concrete in order to key the concrete into the bluff. Pockets of rock would be cut from the bluff face at the anchor locations so as to provide additional concrete thickness to assure adequate tie-back anchorage strength.

A matrix of holes on approximately a 12± foot square grid would be drilled to a depth of about 60± feet in the bluff face. High strength steel, PVC encased anchor bars would be installed in these holes and grouted into stable areas of the formation well shoreward of the bluff face.

Epoxy-coated reinforcing steel would be attached to the bluff surface. Two layers of steel would be used at anchor locations. Colored shotcrete, a form of pneumatically installed concrete, would be blown on the bluff surface to fully encase the reinforcing steel in a minimum of 8 inches of concrete. After the concrete has cured, the tie backs will be tensioned to prevent development of active pressures in any fracture rock behind the wall.

The wall would extend a minimum of 5 feet below the level of rock revetment to provide adequate erosion resistance from wave runup at the rock/wall interface. The top of the wall will be extended 3½ feet above the adjacent roadway elevation to provide a traffic barrier (and to comply with OSHA requirements). This parapet extension will be constructed of formed concrete.

11.3 Resurfacing of Armco Binwall

Rehabilitation of the binwall would consist of the addition of a new 10-inch colored structural concrete surface on the binwall's exterior face. The new wall would be stabilized with tie backs placed completely through the existing binwall and anchored into the underlying formational soils. The tie backs would be placed at 10-foot centers, midway between the existing binwall columns and spacer plates. At the binwall's greatest height, three levels of tie backs would be required.

Construction would commence with cleaning the front wall stringers of all loose corroded material. Holes would be cut in the front face of the binwall and pipe casings driven through the fill and rear binwall stringers. Holes for the anchors would then be drilled through the pipe casing and into the formational soils. High strength steel anchors, encased in PVC sheathing, would be grouted into the rock.

The new structural concrete face would be placed directly against the exterior of the front face stringers. A ribbed concrete cross section would result from the filling between the flutes of the binwall stringers. This concrete section would form a 10-foot-wide beam spanning vertically between tie backs. The completed tied-back wall would act independently of the binwall, allowing it to eventually disintegrate without adversely affecting the performance of the new wall.

11.4 Gravity Walls

Gravity walls proposed on site include cribwalls and Reinforced Earth walls, both patented products. Both of these gravity walls are considered to be semi-flexible structures capable of accommodating substantial differential settlements. Specific geotechnical guidelines for both products are included in Appendix A. In general, these wall types, when constructed in conformance with manufacturer's recommendations, perform well even when subjected to substantial differential settlement (see Appendix D).

12.0 ESTIMATED TOTAL PROJECT COST

Estimated costs for construction of proposed improvements described herein were developed in dollars per facial square foot for total in-place cost for all wall elements; in dollars per cubic yard for in-place cost of all armor stone, assuming a conversion of 1.5 tons per cubic yard; and in dollars per square foot for an asphalt concrete overlay. The cost estimates include installation of all elements of a proposed concept, plus any required grading and/or landscaping. It should be noted that these costs are based on manufacturer's suggested prices and present contractor's average installation cost. These costs could vary somewhat depending on availability, suppliers, and bidding costs. A breakdown of total estimated project costs is provided in Table 1. Unit costs used in our estimate of construction costs are listed below:

ITEM	UNIT COST
8-ton Riprap	$35/ton ($52.50/cu. yd.)
Tied-Back Wall	$75/facial sq. ft.
Reinforced Earth wall	$50/facial sq. ft.
Cribwall:	$33/facial sq. ft. (as proposed)
1 Cell	$18/facial sq. ft.
2 Cell	$25/facial sq. ft.
3 Cell	$40/facial sq. ft.
Asphalt Concrete	$1/sq. ft.
Grout	$100/cu. yd.

REFERENCES

BOOKS, ARTICLES, AND REPORTS

Abbott, P.L., Ed., 1982, "Geologic studies in San Diego," prepared for San Diego Association of Geologists Field Trips, April 1982, 160 pp.

Benton Engineering, Inc., 1960, "Soil and foundation investigation, sewer treatment plant site, City of San Diego, California."

Bird, E.C.F., 1985, "Coastline changes - a global review," John Wiliey and Sons, 219 pp.

Bowden, K.F., 1983, "Physical oceanography of coastal waters," Halsted Press, 302 pp.

Burdette, E.L., and W.W. Howard, 1982, "Intercomparison of directional wave spectra from an NDBO discus-hulled buoy and a wavestaff array," Proceedings, Oceans '82 Conf., p. 1282-1287.

Ellis, M.Y., Ed., 1978, "Coastal mapping handbook," U.S. Government Printing Office, Washington, 113 pp., Appendices.

Emery, K.O., and Kuhn, G.G., 1982, "Sea cliffs: their processes, profiles, and classifications," Geological Society of America Bulletin, v.93, pp. 644-654.

Emery, K.O., and Kuhn, G.G., 1980, "Erosion of rock shores at La Jolla, California," Marine Geology, 37, pp. 197-208.

Fulton, K., 1981, "A manual for researching historical coastal erosion," Report No. T-CSGCP-003, University of California, Santa Cruz, Writing Program, published by California Sea Grant College Program, 56 pp.

Fulton-Bennet, K., and Griggs, G.B., undated, "Coastal protection structures and their effectiveness," jointly published by California Department of Boating and Waterways, and the Marine Sciences Institute, 48 pp.

Gastil, G., and Higley, R., 1977, "Guide to San Diego area stratigraphy," prepared for AAPG-SEG Petroleum Exploration School Field Trip, December 10, 1977, Published by The Edwin C. Allison Center, 62 pp.

REFERENCES (continued)

BOOKS, ARTICLES, AND REPORTS (continued)

Group Delta Consultants, 1987, "Proposal for design services, shoreline protection project, Point Loma Treatment Plant, City of San Diego, California," Proposal No. 1089, prepared for City of San Diego, Water Utilities Department, 31 pp., Exhibits A-D.

Haggerty, B., 1984, "Weather of the future," San Diego Magazine, March 1984, pp. 124-126 and 264-267.

Hileman, J.A., 1979, Seismicity of the San Diego region in Abbott, P.L., and Elliott, W.J. (eds), Earthquakes and other perils, San Diego region, p. 11-20.

Inman, D.L., R.T. Guza, D.W. Skelly, and T.E. White, 1986, "Southern California coastal processes data summary," Ref. No. CCSTWS 86-1, Feb 1986, U.S. Army Corps of Engineers, 572 pp.

Jennings, C.W., 1975, Fault map of California: California Division of Mines and Geology Geologic Data Map No. 1.

Kennedy, M.P., Clark, S.H., Greene, H.G., and Legg, M.R., 1980, Recency and character of faulting offshore metropolitan San Diego, California: California Division of Mines and Geology Map Sheet 42.

Kennedy, M.P., 1975, "Geology of the San Diego metropolitan area, California," California Division of Mines and Geology, Bulletin 200, 56 p., map scale 1"=2000'.

Kennedy, M.P., 1973, Seacliff erosion at Sunset Cliffs, San Diego: Geology, v. 26, no. 2.

Kuhn, G.G., 1980, "Greatly accelerated man-induced coastal erosion and and new sources of beach sand, San Onofre State Park and Camp Pendleton, northern San Diego County, California," Shore and Beach, October 1980, pp. 9-13.

Kuhn, G.G., and Shepard, F.P., 1984, "Sea cliffs, beaches, and coastal valleys of San Diego County, California," University of California Press, 193 pp.

REFERENCES (continued)

BOOKS, ARTICLES, AND REPORTS (continued)

Kuhn, G.G., and Shepard, F.P., 1983, "Beach processes and sea cliff erosion in San Diego County, California," in: CRC Handbook of Coastal Processes and Erosion (Chapter 13), Paul D. Komar, ed., CRC Press, Inc., Boca Raton, FL, pp. 267-284.

Kuhn, G.G., and Shepard, F.P., 1983, "Importance of phreatic volcanism in producing abnormal weather conditions," Shore and Beach, October 1983, pp. 19-29.

Kuhn, G.G., and Shepard, F.P., 1980, "Coastal erosion in San Diego County, California," Proceedings of the Conference: "Coastal Zone '80," American Society of Civil Engineers, pp. 1899-1918.

Kuhn, G.G., and Shepard, F.P., 1981, "Should Southern California build defenses against violent storms resulting in lowland flooding as discovered in records of past century," Shore and Beach, Vol. 49, No. 4, pp. 3-10.

Kuhn, G.G., and Shepard, F.P., 1979, "Accelerated beach-cliff erosion related to unusual storms in Southern California," California Geology, March 1979, pp. 58-59.

Le Mehaute, B., and J.D. Wang, 1982, "Wave spectrum changes on a sloped beach," Jour. Water, Port, Coastal and Ocean Div., Amer. Soc. Civil Eng., v. 108, p. 33-47.

Lee, Louis, Pinckney, C.J., and Bemis, C., 1976, "Sea cliff base erosion," American Society of Civil Engineers, National Water Resources and Ocean Engineering Convention, Preprint 2708, 14 pp.

Legg, Mark, and Agnew, D.L., 1979, The 1862 earthquake in San Diego in Abbott, P.L., and Elliott, W.J. (eds), Earthquakes and other perils, San Diego region, p. 139-142.

Lemonick, M.D., 1987, "Shrinking shores," Time Magazine, 10 August 1987, pp. 38-47.

Marine Advisers, 1961, "A statistical survey of ocean wave characteristics in Southern California waters," prepared for the U.S. Army Corps of Engineers, Los Angeles District, Marine Advisers, Inc., La Jolla, CA, 30 pp.

REFERENCES (continued)

BOOKS, ARTICLES, AND REPORTS (continued)

May, K.D., 1987, "Historical chronology of extreme storm and wave events in San Diego County," prepared for Moffatt and Nichol, Engineers, 37 pp.

McGrath, J., Ed., 1985, "California's battered coast - proceedings from a conference on coastal erosion," San Diego, February 6-8, 1985, Sponsored by California Coastal Commission, 403 pp.

Munk, W.H., G.H. Miller, F.E. Snodgrass, and N.F. Barber, 1963, "Directional recording of swell from distant storms," Phil. Trans. Roy Soc. London, A, v. 55, n. 1062, p. 505-584.

Munk, W.H., and M.A. Traylor, 1947, "Refraction of ocean waves, a process linking underwater topography to beach erosion," Jour. Geol., v. 55, n. 1, p. 1-26.

Naval Facilities Engineering Command, 1982, "Coastal protection, design manual 26.2," 314 pp., references, glossary, symbols.

Norris, R.M., 1968, Sea cliff retreat near Santa Barbara, California: California Division of Mines and Geology, Mineral Information Service, v. 21, No. 6, p. 87-91.

North, W.J., 1954, Size distribution, erosive activities, and gross matebolic efficiency of the marine intertidal snarls, Littorina planaxis and L. Scutulata: Biol. Bull. 106, p. 185-197.

Pawka, S.S., and R.T. Guza, 1983, "Coast of California waves study - site selection," SIO Ref. 83-12, 51 pp., Scripps Inst. of Oceanogr., La Jolla, Calif.

Pease, R.C., 1979, Scarp degradation and fault history south of Carson City, Nevada: University of Nevada, Reno, Masters Thesis, 90 p.

Richter, C.F., 1958, Elementary seismology: W.H. Freeman, San Francisco, 768 p.

Ross, A., and Dowlen, R.J., Eds., 1973, "Studies on the geology and geologic hazards of the greater San Diego area, California," San Diego Association of Geologists, 152 pp.

REFERENCES (continued)

BOOKS, ARTICLES, AND REPORTS (continued)

Sanders, N.K., 1968, The development of the Tasmanian shore platforms, University of Tasmania, Ph.D. Thesis.

Schnabel, P.B., and Seed, H.B., 1973, Accelerations in rock for earthquakes in the western United States: Bulletin of the Seismological Society of America, v. 63, no. 2, p. 501-516.

Seed, Bolton, and Whitman, Robert V., 1970, "Design for earth retaining structures for dynamic loads," in Lateral stresses in the ground and design of earth-retaining structures, American Society of Civil Engineers, New York, p. 103-147.

Seymour, R.J., R.R. Strange, III, D.R. Cayan, and R.A. Nathan, 1984, "Influence of El Niños on California's wave climate," Proc. 19th Coastal Eng. Conf., Amer. Soc. Civil Eng., p. 577-592.

Seymour, R.J., and M.H. Sessions, 1976, "A regional network for coastal engineering data," Proc. 15th Coastal Eng. Conf., Amer. Soc. Civil Eng., p. 60-71.

Shepard, F.P., and Kuhn, G.G., 1983, "History of sea arches and remnant stacks of La Jolla, California, and their bearing on similar features elsewhere," Marine Geology, 51, pp. 139-161.

Shepard, F.P., and Grant, IV, U.S., 1947, "Wave erosion along the Southern California coast," Geological Society of America Bulletin, v.58, pp. 919-926.

Slemmons, D.B., 1977, State-of-the-art for assessing earthquake hazards in the United States, Report 6, Faults and earthquake magnitudes: U.S. Army Engineer Waterways Experiment Station Miscellaneous Paper S-73-1, 128 p.

Southern California Chapter American Public Works Association, and Southern California District Associated General Contractors of California, Joint Cooperative Committee, 1985, Standard Specifications for Public Works Construction.

State of California, 1984, Standard Specifications, State of California, Department of Transportation, p. 72-1 to 72-10.

REFERENCES (continued)

BOOKS, ARTICLES, AND REPORTS (continued)

Trenhaile, Alan S., 1987, "The geomorphology of rock coasts," Clarendon Press, Oxford, 384 pp.

U.S. Army Corps of Engineers, 1987, "Coast of California, storm and tidal waves study - coastal cliff sediments, San Diego region," 21 sectins, plates.

U.S. Army Corps of Engineers, 1986, "Coast of California, storm and tidal waves study - Southern California coastal photography and beach profile index," 66 pp., plates.

U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - geotechnical data inventory, Southern California," 3 sections, references, plates.

U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - hydraulic data inventory," 35 pp.

U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - hydrologic data inventory, Southern California coastal zone," 102 pp., appendices.

U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - littoral zone sediments, San Diego region," 7 sections, appendices.

U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - shoreline movement data report, Portugese Point to Mexican border (1852-1982)," 48 pp., appendix.

U.S. Army Corps of Engineers, 1984, "Coast of California, storm and tidal waves study - geomorphology framework report, Dana Point to the Mexican border," 9 sections, references, appendices.

U.S. Army Corps of Engineers, 1977, Geologic investigation of Segment A at Sunset Cliffs, San Diego, California.

U.S. Army Corps of Engineers, 1977, Shore Protection Manual, U.S. Army Coastal Engineering Research Center, Fort Belvoir, Virginia, v. I, II, and III.

REFERENCES (continued)

BOOKS, ARTICLES, AND REPORTS (continued)

U.S. Army Corps of Engineers, 1970, Specifications for Shore Protection for Ocean Beach Area, Sunset Cliffs Segment B, San Diego, California, U.S. Army Engineer District, Los Angeles, California.

U.S. Army Corps of Engineers, 1965, Special study of City of San Diego (Sunset Cliffs): Beach Erosion Control Report on Coast of Southern California, Contract No. W04193ENG5196, U.S. Army Engineer District, Los Angeles, California.

U.S. Corps of Engineers, Los Angeles District, March 1, 1960, "Beach erosion control report, A cooperative study of San Diego County, California," Appendix D, Plate 8.

Wallace, R.E., 1977, Profiles and ages of young fault scarps, north-central Nevada: Geological Society of America Bulletin, v.88, p.1267-1281.

Woodward-Clyde Consultants, August 1981, Design memorandum, geotechnical and civil engineering design services for Sunset Cliffs shoreline and upper cliff stabilitation project, San Diego, California.

REFERENCES (continued)

VERTICAL AERIAL PHOTOGRAPHS

Nov/Dec, 1987 -	Pountney & Associates/B&W
February, 1985 -	USGS-EROS/Color
March, 1982 -	NOAA/Color
November, 1981 -	USGS-EROS/B&W
April, 1978 -	California Department of Water Resources/B&W
November, 1976 -	California Department of Water Resources/B&W
September, 1972 -	USGS-EROS/B&W
October, 1966 -	USGS-EROS/B&W
August, 1966 -	NOAA/B&W
September, 1964 -	U.S. Department of Agriculture/B&W
September, 1960 -	NOAA/B&W
May, 1953 -	U.S. Department of Agriculture/B&W
November, 1950 -	NOAA/B&W
1949 -	U.S. Department of Agriculture/B&W
1939 -	USGS - NCIC

REFERENCES (continued)

MAPS AND CHARTS

1967, (photo revised 1975), "Point Loma, California," USGS Topographic Quadrangle, scale 1:24,000.

November 1, 1960 (plans approved), November 3, 1961 (as-built), Plans for the Construction of Cabrillo Road and Site Grading for Sewage Treatment Plant, 17 sheets, City of San Diego Drawing Nos. 9074-1D to 9074-17D.

February 10, 1960, Untitled topographic map, No. 186-1689 (18-76), City Engineers Office, City of San Diego, scale 1":200'.

1953, "Point Loma, California," USGS Topographic Quadrangle, scale 1:24,000.

1942 ed., (surveyed 1938 - 1939), "Point Loma, California," USGS Topographic Quadrangle, scale 1:31,680.

June 12, 1941, "Desired improvements, San Diego Harbor, California," scale 1:15,000.

1916 and 1917, "California, San Diego Bay, Point Loma to San Diego," U.S. Coast and Geologic Survey, scale 1:10,000.

July 1, 1911, "Entrance to San Diego Harbor, California," U.S. Army Corps of Engineers, scale 1:1000±.

04-01-1910, "Submarine cable route, Fort Rosecrans, San Diego Harbor, California," U.S. Army Signal Office, scale 1:10,000.

1904, "San Diego Quadrangle," U.S.G.S. Topographic Quadrangle, scale 1:62,500.

July 1902, "San Diego Bay, California," U.S. Coast and Geodetic Survey, scale 1:40,000.

May 9, 1902, "Entrance to San Diego Harbor, California," U.S. Army Corps of Engineers, scale 1:10,000.

March 13, 1901, "Entrance to San Diego Harbor, California," U.S. Army Corps of Engineers, scale 1:10,000.

REFERENCES (continued)

MAPS AND CHARTS (continued)

1859, "San Diego Bay, California," U.S. Coast Survey, scale 1:40,000.

July 1858, "Plat of the Pueblo Lands of San Diego," U.S. Surveyor General, scale 1":80 chains.

1856, "Hydrography of San Diego Bay and vicinity," U.S. Coast Survey, scale 1:10,000.

1853, "San Diego entrance and approaches," U.S. Coast Survey, scale 1:20,000.

1852, "Map of False Bay near San Diego, California," U.S. Coast Survey, scale 1:10,000.

1851, "Map of San Diego Bay, California, from Point Loma to New Town," U.S. Coast Survey, scale 1:10,000.

1851, "San Diego Harbor, California," U.S. Coast Survey, scale 1:10,000.

Undated (prior to 1848), untitled map of San Diego area - Spanish, scale in varas.

GLOSSARY OF TERMS

BLOCKFALL: Rapid decent of a large angular rock fragment derived from breaking of the parent rock mass, usually along joints.

BLUFF: The rising ground bordering the sea which may include a sea cliff, but is characterized by an upper, moderately-sloping, section ending at a coastal terrace.

BLUFF TOP: The boundary between the bluff and the coastal terrace.

BLUFF-TOP RETREAT: Landward migration over time of the bluff top caused by marine erosion on the sea cliff and subaerial erosion of the bluff.

CAUSTIC: In refraction of waves, the name given to the curve to which adjacent wave rays refracted by a bottom whose contour lines are curved, are tangents. The occurrence of a caustic always marks a region of crossed wave rays and high wave convergence.

CLAPOTIS: Nonbreaking waves.

CLIFF-PLATFORM JUNCTION: The location at the base of the sea cliff where the near-horizontal shore platform meets the near-vertical sea cliff.

COASTAL TERRACE: Any long, narrow, relatively level surface bounded along the shoreward edge by a sea cliff and along the landward edge by ascending slopes.

DIURNAL: Having a period or cycle of approximately 1 tidal day.

EROSION: The mechanical destruction of the land or sea floor and the removal of rock and soil by running water, waves and currents, moving ice, wind, and gravity. It includes the processes of weathering, solution, corrosion, and transportation.

FETCH: The horizontal distance (in the direction of the wind) over which a wind generates seas.

FORESHORE ZONE: A part of the shore lying between the upper limit of wave wash at high tide and the low water mark. The foreshore is usually traversed by the uprush and backrush of waves; however, the foreshore is typically absent at the site.

GEOMORPHOLOGY: That branch of both physiography and geology which deals with the form of the earth, the general configuration of its surface, and the changes that take place in the evolution of landform.

HEADLAND (HEAD): A high steep-faced promontory extending into the sea.

INSHORE ZONE: A zone of variable width extending from the low water line at the shore to the seaward edge of the breaker zone.

NEAP TIDE: A tide occurring near the time of quadrature of the moon with the sun. The neap tidal range is usually 10 to 30 percent less than the mean tidal range.

NEARSHORE ZONE: An indefinite zone extending seaward from the shoreline well beyond the breaker zone.

REFRACTION: The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than the part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours.

SEA CLIFF: A more or less continuous line of seaward-facing high, steep rock faces or precipices that are caused by marine and subaerial erosion.

SEMIDIURNAL TIDE: A tide with two high and low waters in a tidal day.

SHORE: The narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a beach.

SHORE PLATFORM: The horizontal or gently seaward sloping surface produced along a shore by wave erosion and other subaqueous erosion processes. Synonym: wave-cut platform.

STANDING WAVES: Nonbreaking waves.

SUBAERIAL EROSION: Erosion that occurs on the land surface due to removal of surface material by wind, water, and gravity in its broadest sense. This also includes the weathering process which produces more erodible material. Contrasted with marine erosion.

WASTING: The gradual destruction or wearing away of a landform surface by wind, gravity, and rill wash, but excluding subaqueous erosion.

WAVE SPECTRUM: A graph showing the distribution of wave energy as a function of wave frequency. The spectrum may be based on observations and/or theoretical considerations. Several forms of graphic display are widely used. See Appendix B.

GLOSSARY OF TERMS (continued)

WAVE HEIGHT: The vertical distance between a crest and the preceding trough.

WAVE LENGTH: The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

WAVE RAY (ORTHOGONAL): On a wave-refraction diagram, a line drawn perpendicularly to the wave crests.

WAVE SETUP: Superelevation of the still water surface over normal surge elevation due to onshore mass transport of water by wave action alone.

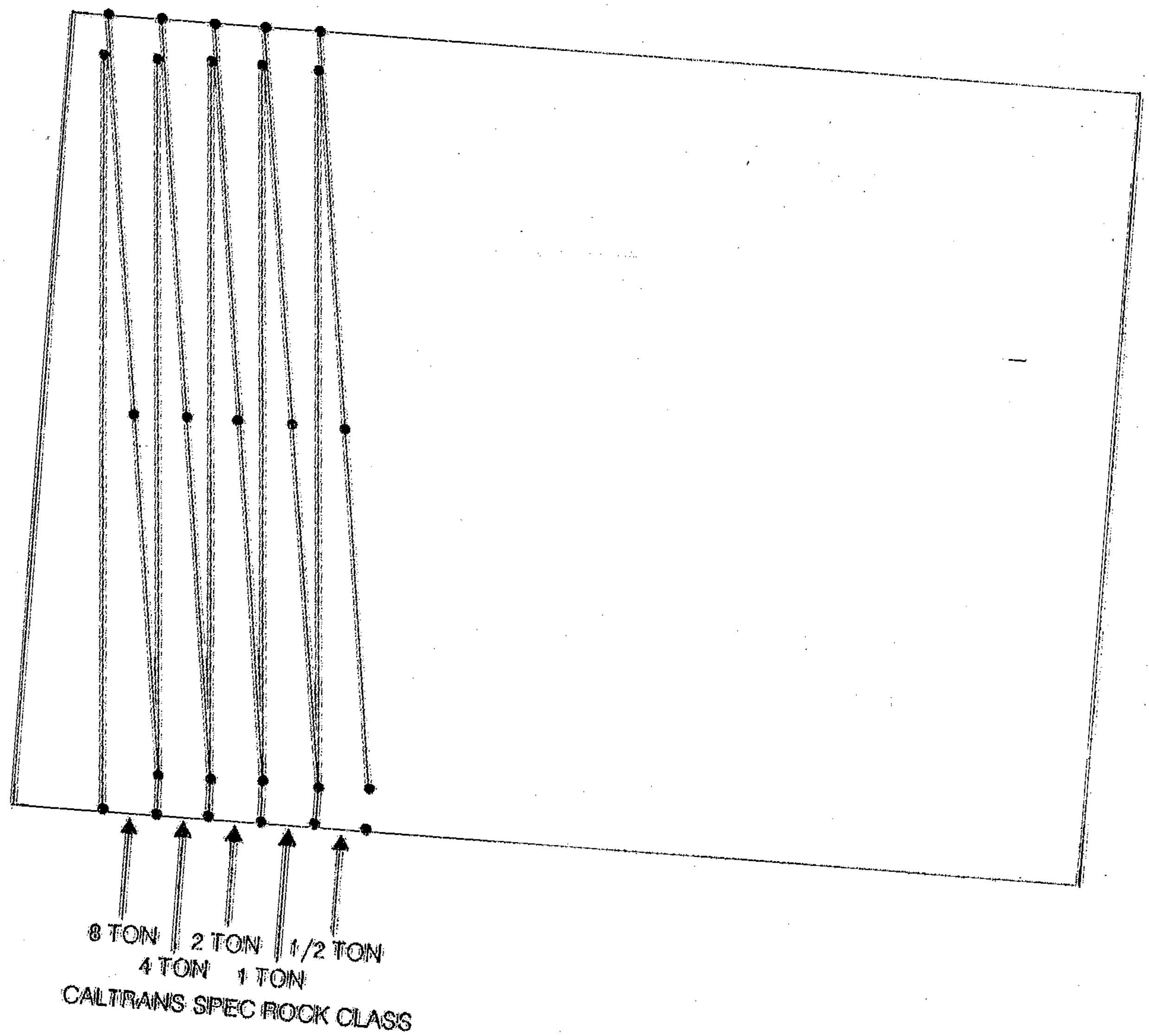
WEARING: The gradual destruction of a landform surface by movement of loose rock fragments or particles driven by wind, waves, running water or ice that causes rubbing, grinding, knocking, scraping, and bumping against the landform surface.

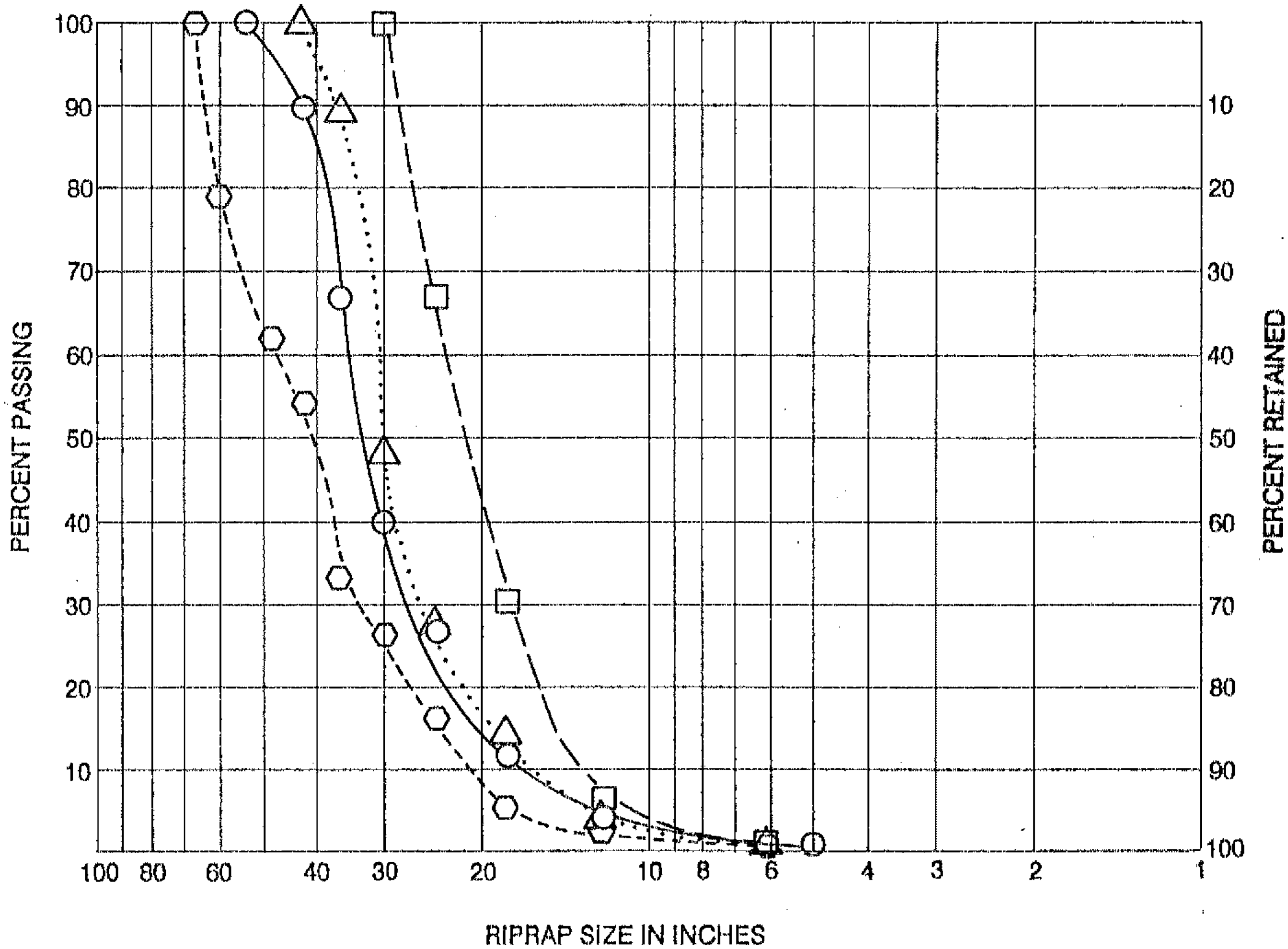
WEATHERING: The physical disintegration and chemical decomposition of rock that produces an in-situ mantle of softer material that is more easily eroded.

TABLE 1

ESTIMATED PROJECT COSTS

Section 1	
7,400 tons rock @ $35/ton	$ 259,000
Section 2	
3,570 sq. ft. tied-back wall @ $75/sq. ft.	267,750
75 cu. yd. grout @ $100/cu. yd.	7,500
Section 3	
2,300 sq. ft. Reinforced Earth Wall @ $50/sq. ft.	115,000
Construction Backcut & Subgrade Prep	25,000
600 tons rock @ $35/ton	21,000
Section 4	
5,440 sq. ft. binwall rehabilitation @ $75/sq. ft.	408,000
3,000 sq. ft. lower tied-back wall @ $75/sq. ft.	225,000
1,650 tons rock @ $35/ton	57,750
Section 5	
4,680 sq. ft. tied-back wall @ $75/sq. ft.	351,000
1,200 tons rock @ $35/ton	42,000
Section 6	
14,800 sq. ft. cribwall @ $33/sq. ft.	488,400
22,000 sq. ft. asphaltic conrete @ $1/sq. ft.	22,000
Regrade Engineering Trailer Pad	15,000
TOTAL CONSTRUCTION COST	$ 2,304,400
15% Contingency	345,660
Design Fees	331,000
10% City Administration	298,106
	$ 3,279,166


8 TON
4 TON
2 TON
1 TON
1/2 TON
CALTRANS SPEC ROCK CLASS

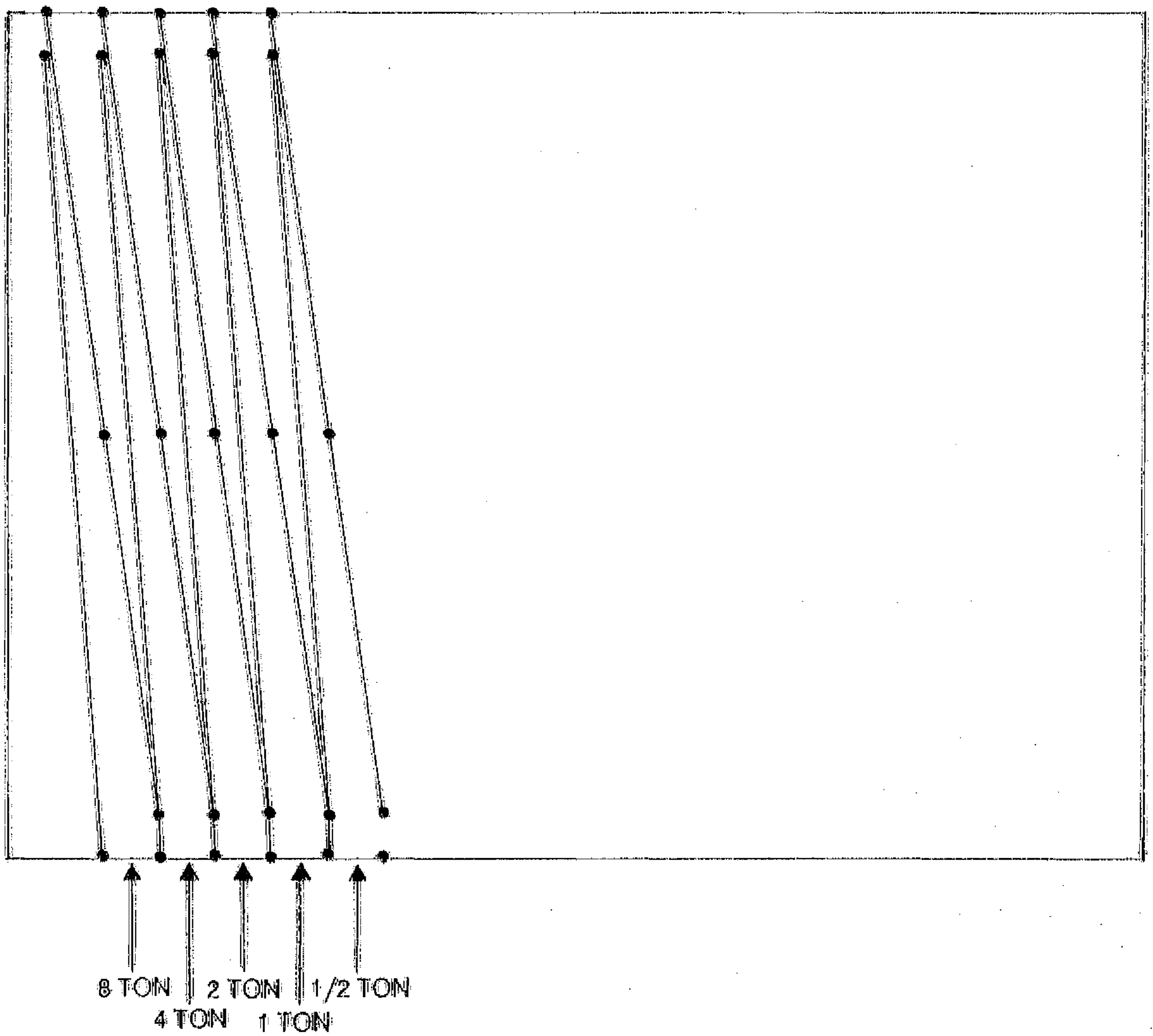


SAMPLE	SYMBOL	LOCATION
1	—○—	VICINITY OF OUTLET STRUCTURE
2	– –□– –	VICINITY OF OUTLET STRUCTURE
3	· · ·△· · ·	VICINITY OF OUTLET STRUCTURE
4	- - - ⬡ - - -	VICINITY OF OUTLET STRUCTURE

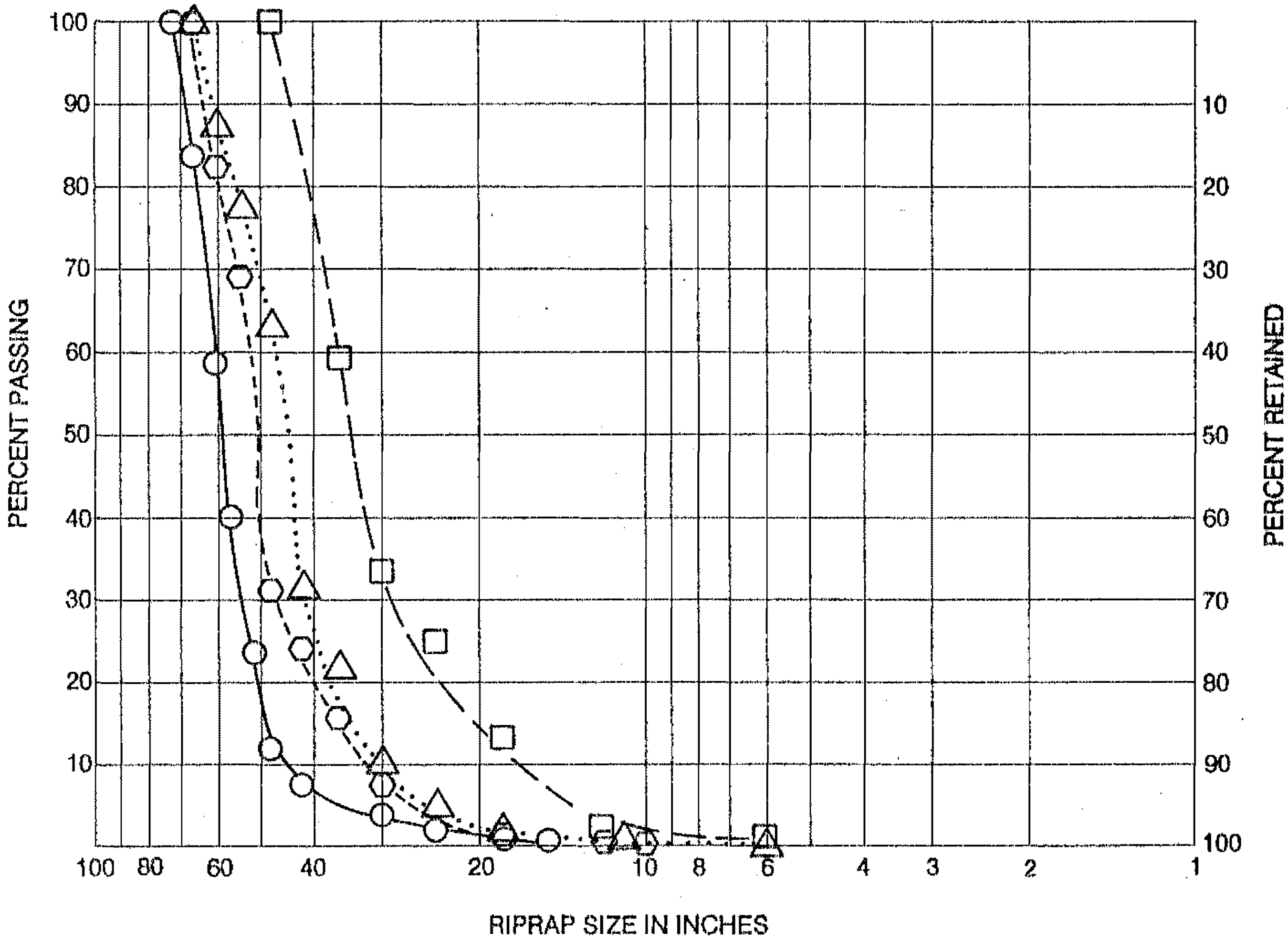
RIP RAP SIZE DISTRIBUTION

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 2

GROUP DELTA CONSULTANTS, INC.



CALTRANS SPEC ROCK CLASS



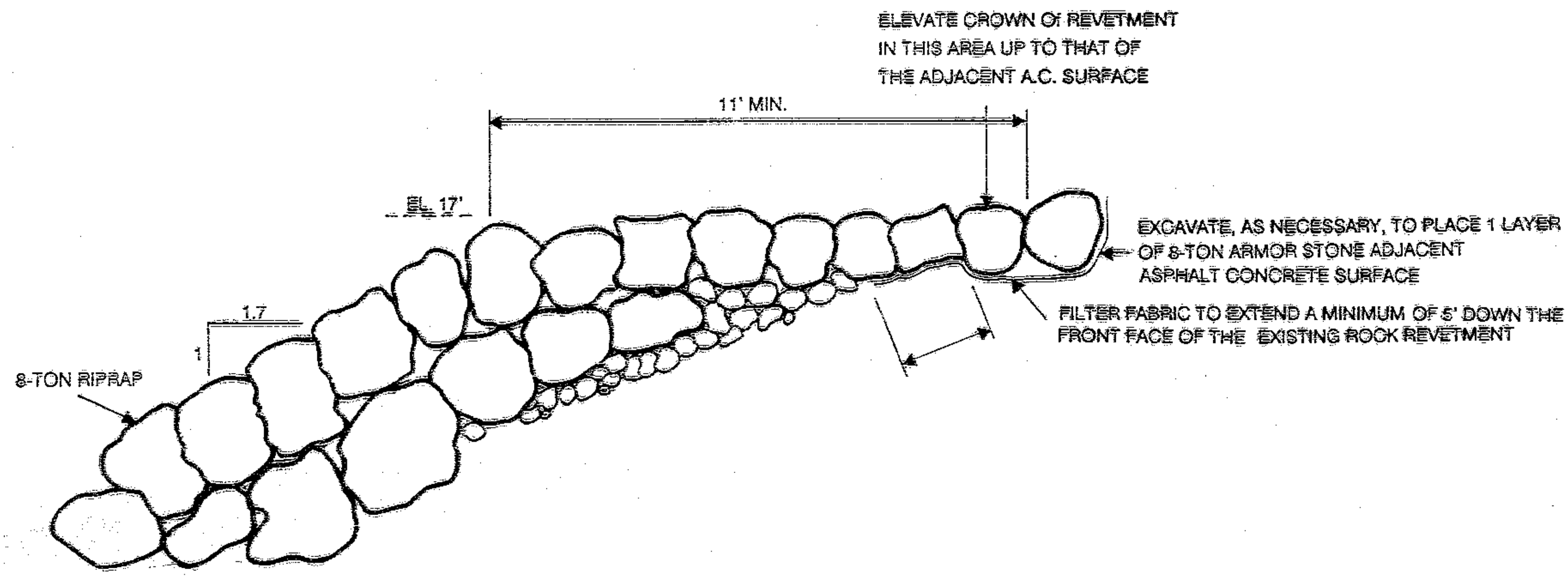
SAMPLE	SYMBOL	LOCATION
5	—○—	DOWNSLOPE OF HYDRO ACCESS ROAD
6	– –□– –	DOWNSLOPE OF HYDRO ACCESS ROAD
7	···△···	DOWNSLOPE OF HYDRO ACCESS ROAD
8	----⬡----	DOWNSLOPE OF HYDRO ACCESS ROAD

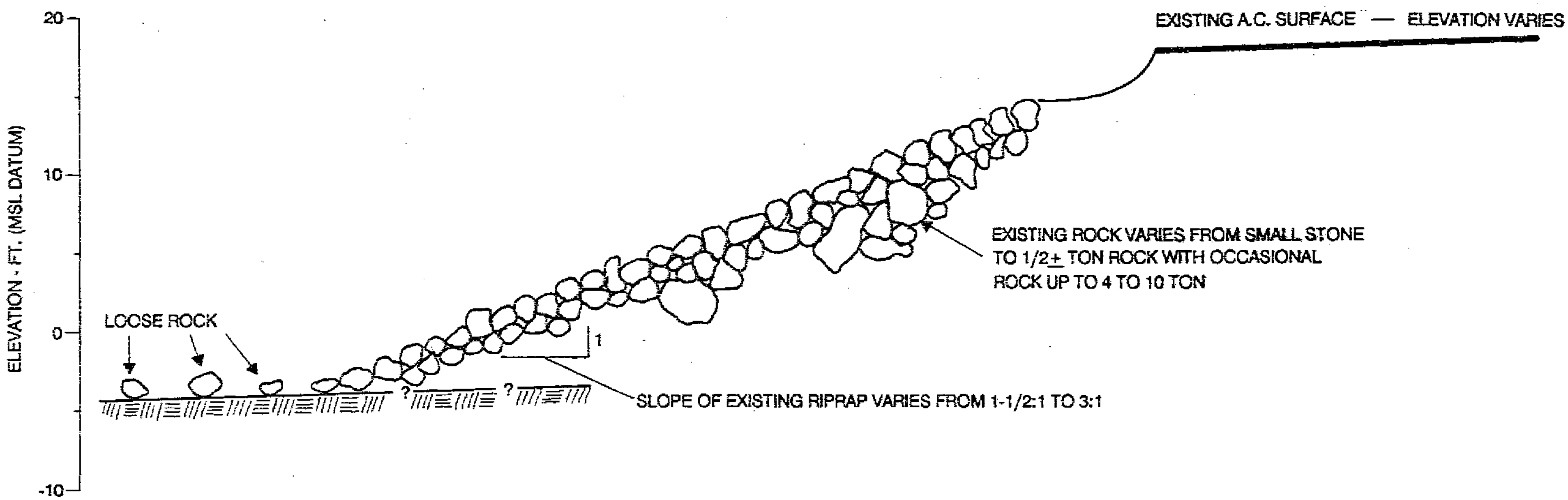
RIP RAP SIZE DISTRIBUTION

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 3

GROUP DELTA CONSULTANTS, INC.

RESHAPE EXISTING ROCK, AS NECESSARY, TO PROVIDE A ROCK CORE AT A SLOPE OF 1.7:1 UP TO EL. 14 FT. PLACE 2 LAYERS OF 8-TON ARMOR STONE (11' MINIMUM) ON FRONT FACE OF CORE STONE AS SHOWN. MINIMUM CROWN WIDTH TO BE 11' OR LONGER, AS NECESSARY, TO BALANCE VOLUME OF CORE STONE.

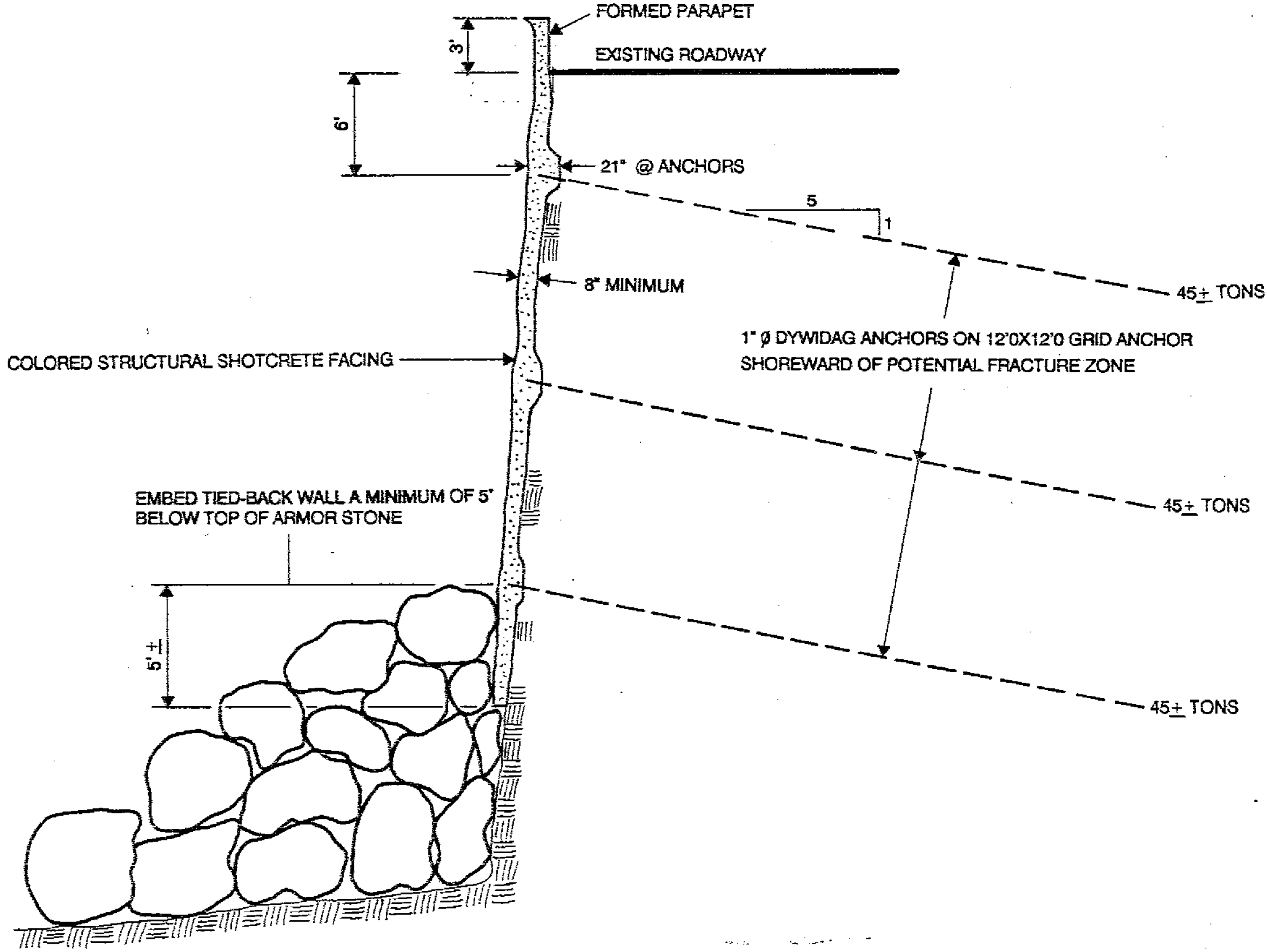




STONE REVETMENT ADJACENT OUTLET STRUCTURE

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 4

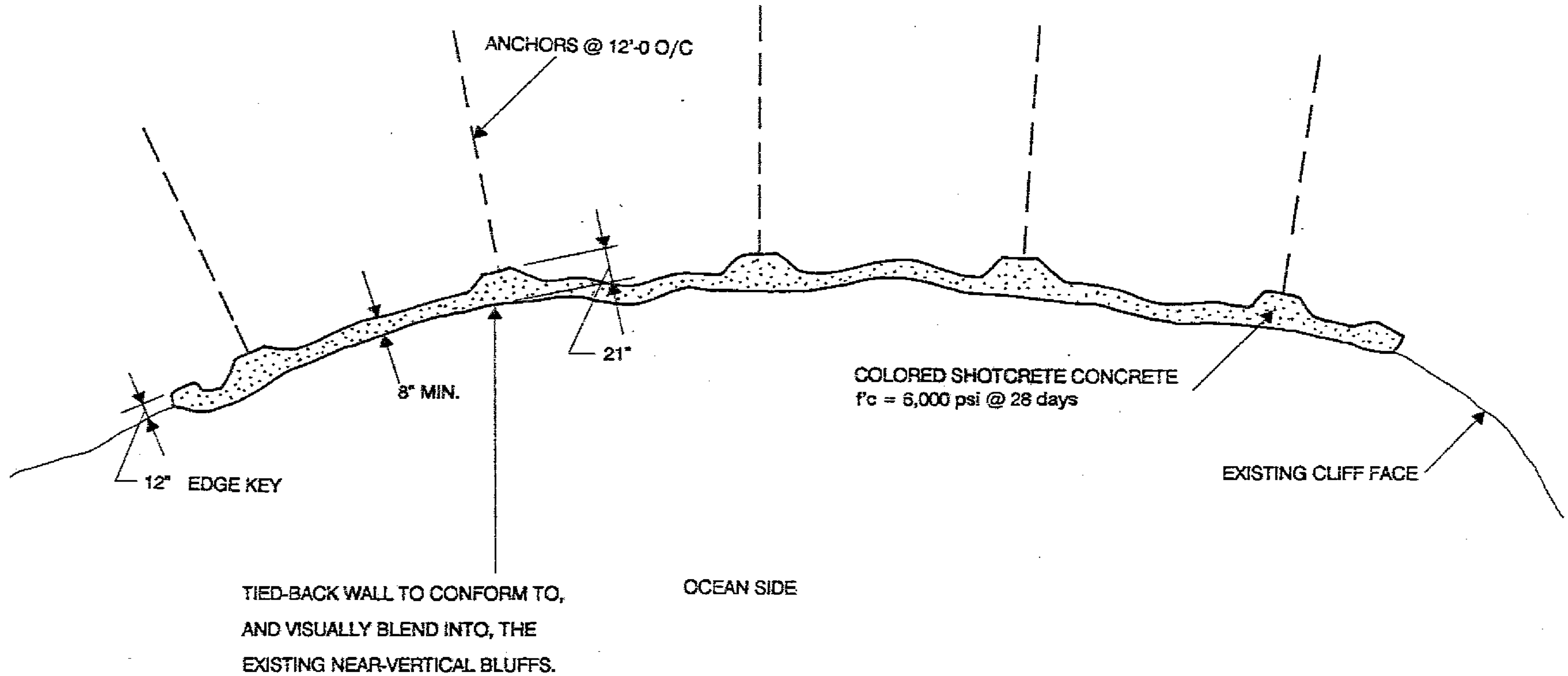
GROUP DELTA CONSULTANTS, INC.



TIED BACK WALL TYPICAL SECTION

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 5

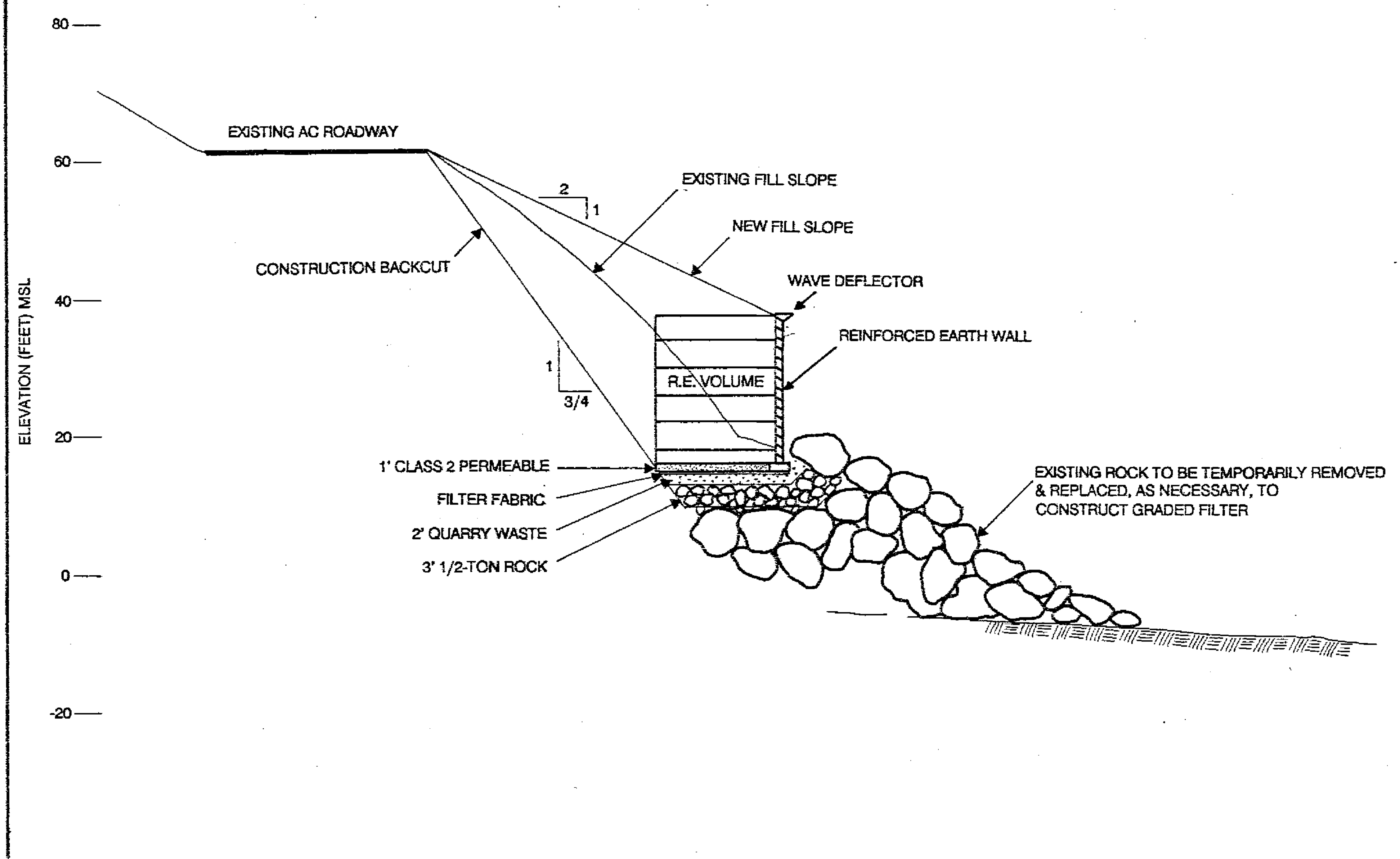
GROUP DELTA CONSULTANTS, INC.



TIED BACK WALL TYPICAL PLAN VIEW

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 6

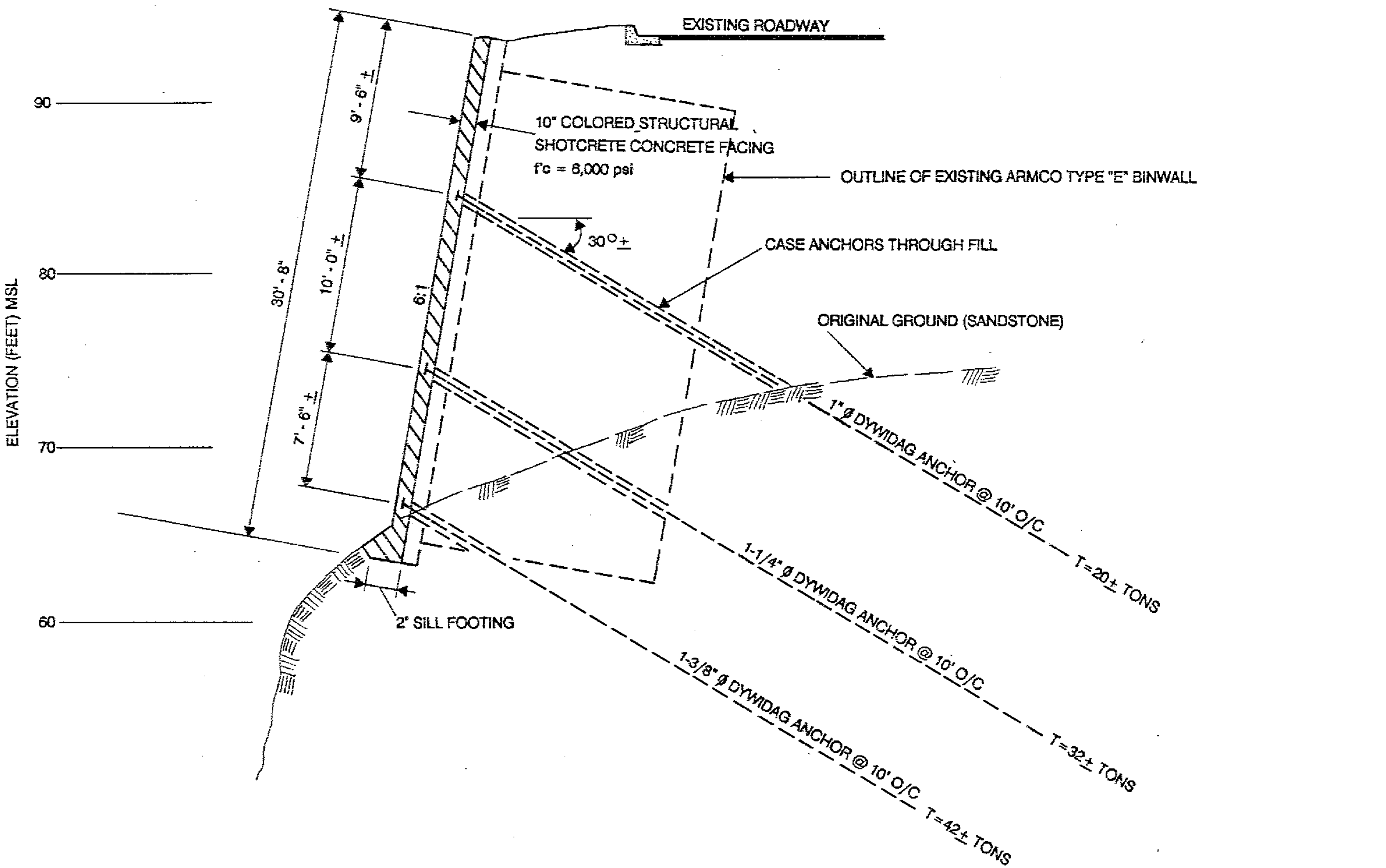
GROUP DELTA CONSULTANTS, INC.



REINFORCED EARTH WALL DOWNSLOPE OF GAS UTILIZATION FACILITY

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 7

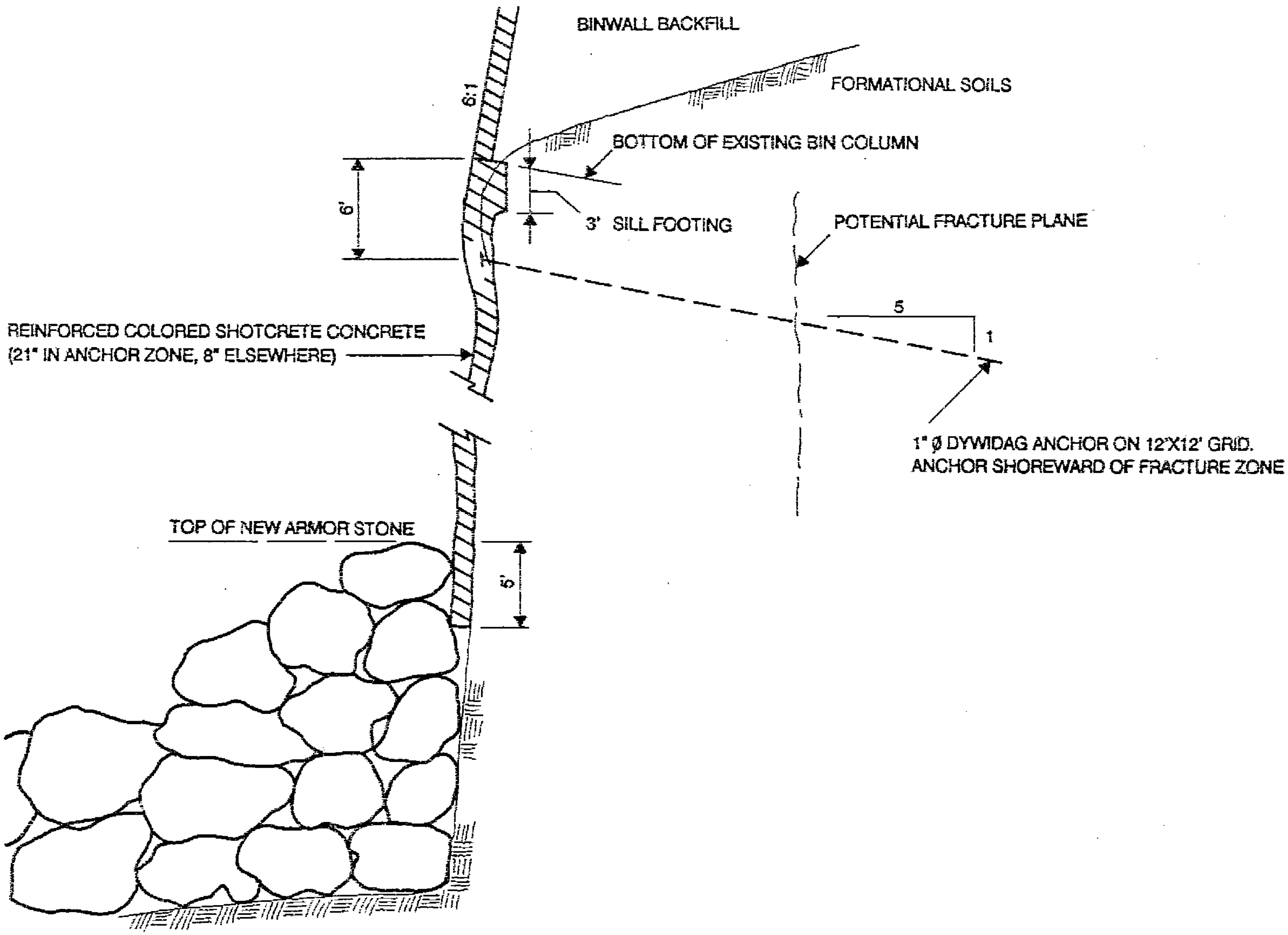
GROUP DELTA CONSULTANTS, INC.



BINWALL FACING

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 8

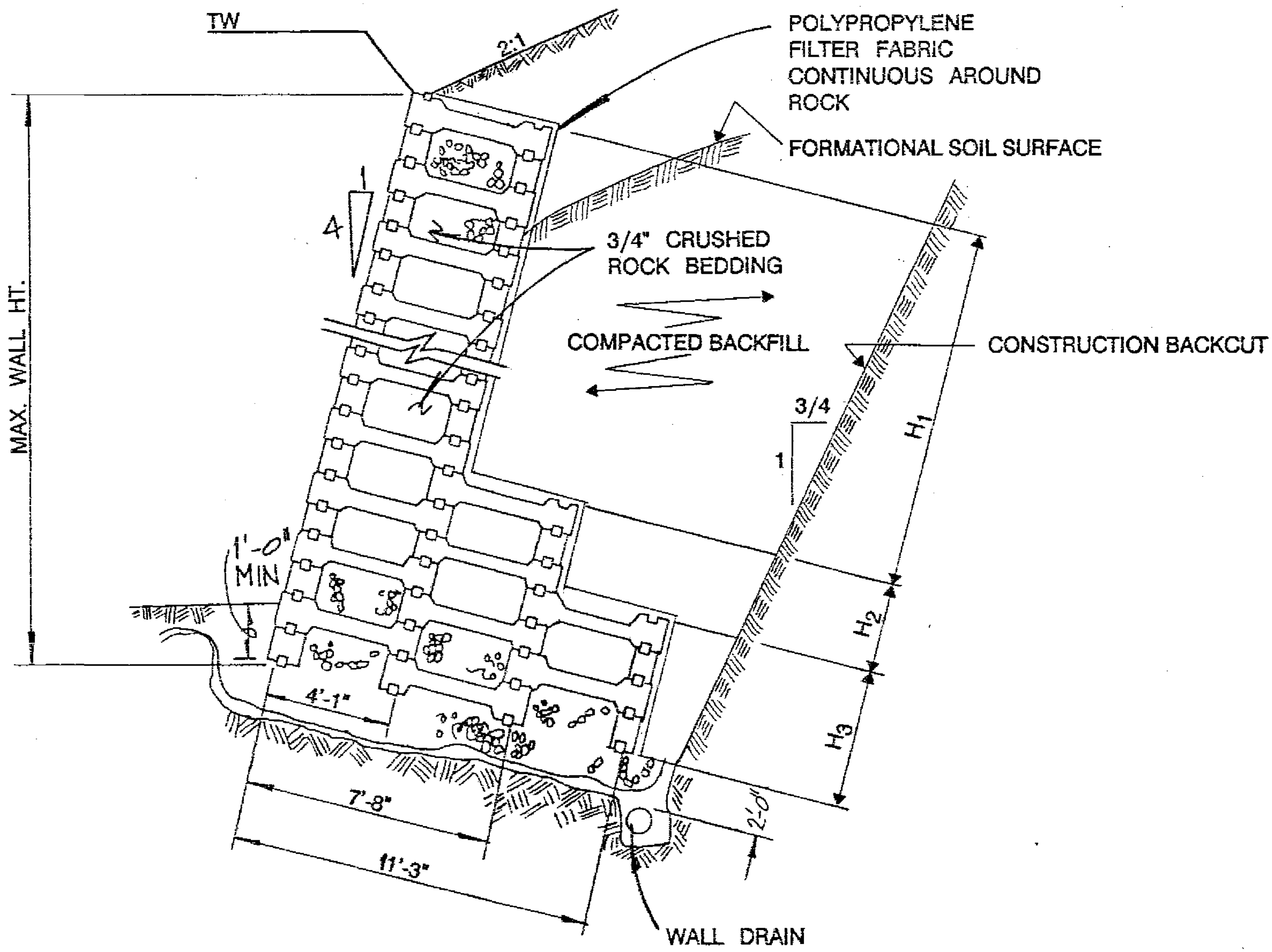
GROUP DELTA CONSULTANTS, INC.



TIED BACK WALL DETAIL BELOW BINWALL

Project No. 1089-EC01	POINT LOMA TREATMENT PLANT	Figure 9

GROUP DELTA CONSULTANTS, INC.



TYPICAL CRIBWALL SECTION

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure 10

GROUP DELTA CONSULTANTS, INC.

APPENDIX A

GEOTECHNICAL INVESTIGATION FOR UPGRADING OF SHORELINE PROTECTION POINT LOMA TREATMENT PLANT SAN DIEGO, CALIFORNIA

APPENDIX A

GEOTECHNICAL INVESTIGATION
POINT LOMA TREATMENT PLANT
SAN DIEGO, CALIFORNIA

TABLE OF CONTENTS

APPENDIX A

GEOTECHNICAL INVESTIGATION FOR UPGRADING OF SHORELINE PROTECTION POINT LOMA TREATMENT PLANT SAN DIEGO, CALIFORNIA

1.0 PROJECT DESCRIPTION

The upgrading of the Point Loma Treatment Plant shore protection consists of the stabilization of existing coastal bluffs and the mitigation of wave runup in the vicinity of the outlet structure. The proposed shoreline protection considers a combination of tied-back walls, Reinforced Earth walls, cribwalls, and rock revetments. Specifically, the project comprises six critical sites designated Sections 1 through 6. They are:

- Section 1 - The vicinity of the outlet structure;
- Section 2 - The lower hydro-access road corresponding to a localized bluff erosion area;
- Section 3 - The upper portion of the hydro-access road in the vicinity of the existing fill slope;
- Section 4 - The Armco Binwall;
- Section 5 - The bluff area south of the southerly visitor parking lot; and
- Section 6 - The hydro-access road and GUF access road.

A brief description of the proposed upgrading of the shoreline protection for each of these sites is presented below.

<u>Vicinity of Outlet Structure</u> - The proposed shoreline protection for this area consists of the placement of additional protective rock in front of the outlet structure, to a crown elevation of approximately 17 feet. The revetment would consist of 8-ton armor stone constructed at an inclination of 1.7:1.

<u>Lower Hydro-Access Road in the Vicinity of Upper Bluff Erosion</u> - The proposed improvement in this area consists of an 85± foot long vertical tied-back wall with an approximately 3½-foot-high

parapet. The wall is planned to conform to the existing near-vertical bluff and is to provide a barrier wall where erosion has encroached into the roadway.

Upper Portion of Hydro-Access Road in Vicinity of Existing Fill Slope - The proposed improvement in this area consists of an approximately 100-foot-long Reinforced Earth wall which will incorporate a wave deflector. The wall is to have a top-of-wall elevation of approximately 38 feet. It is anticipated that the Reinforced Earth wall will also support a fill slope which is inclined approximately 2:1 (horizontal to vertical). Limited additional rock revetment is also required at the base of the wall to provide additional protection.

Armco Binwall - The proposed improvements in the vicinity of the Armco Binwall consist of:

- Resurfacing the binwall with a tied-back structural concrete facing. The purpose of this tied-back facing is to provide stability, assuming future total deterioration of the existing binwall;
- A proposed vertical tied-back wall along the base of the bluff where localized erosion is undercutting the binwall. This wall would be approximately 60 feet in length and similar in appearance to the tied-back wall proposed along the lower hydro-access road; and
- Additional rock revetment beneath the tied-back wall to provide additional protection. The lateral extent of this additional rock revetment is on the order of 70 feet.

Bluff Area South of Southerly Visitor Parking Lot - The proposed improvements in this area consist of approximately 120 feet of vertical tied-back walls along the upper portion of the bluff, with approximately 60 feet of additional rock revetment.

Hydro-Access Road and GUF Access Road Retaining Walls - Retaining walls are proposed landward of the hydro-access road and easterly of the GUF-access road immediately south of the gas utilization facility. These walls will result in additional stability and create additional usable land in these areas. The inclusion of these walls within the proposed project will be dependent upon the availability of funds.

Other minor improvements, including limited grading, additional storm drains and resurfacing of the lower hydro access road, are also proposed.

2.0 SCOPE OF WORK

The purpose of this geotechnical investigation was to provide information pertaining to the subsurface soil and geologic conditions within the project site, and to provide geotechnical input to the design of the various earth retaining systems. Specifically, our scope of work consisted of:

- Drilling and sampling of four exploratory soil borings;
- Performing laboratory tests to classify and evaluate the strength characteristics of the subsurface materials;
- Engineering analyses; and
- Presenting the results of our investigation and recommendations in a report.

3.0 GEOTECHNICAL SITE CHARACTERIZATION

To characterize the subsurface conditions at the project site, we reviewed reports from previous geotechnical studies, and performed a field investigation and laboratory testing.

3.1 Review of Previous Studies

Two documents were reviewed as part of our geotechnical investigation. These were:

- "Soil and Foundation Investigation, Sewer Treatment Plant Site, City of San Diego, California," prepared by Benton Engineering, Inc., dated February 29, 1960; and

- Construction drawings entitled "Construction of Cabrillo Road and Site Grading for Sewage Treatment Plant," prepared by the City of San Diego Engineering Department, dated November 1, 1960.

Summary of Benton Engineering Report - Information obtained from Benton Engineering's report and the grading plans was used to help characterize the site. Benton Engineering's report was used to qualify site geology and to assess the nature of fill materials within the reclaimed sea cove areas. Their report discussed the suitability of cut material as a source for filled ground. Strength characteristics were assessed for recompacted bulk samples of near-surface silty sands. Materials were recompacted to 90 percent relative compaction (resulting in a dry density of approximately 110 pcf) as determined by ASTM D-698 and saturated prior to direct shear tests. Results of Benton Engineering's tests indicated that the saturated compacted soil had an approximate cohesion of 175 psf and an angle of friction of 28 degrees. On the basis of these tests and their computations, Benton Engineering recommended that fill slopes be constructed no steeper than 1.5 to 1 (horizontal to vertical) for slopes having a height not exceeding 60 feet. For slopes with heights in excess of 60 feet, they recommended slope inclinations no steeper than 2 to 1 (horizontal to vertical).

3.2 Field Investigation

Field exploratory work for the geotechnical investigation was conducted between November 1987 and February 1988. The investigation included a geologic reconnaissance of the site and the drilling of four borings at the approximate locations shown on the Site Plan and Geologic Map, Figure 1. On the basis of the geologic site reconnaissance, we determined that four borings would be sufficient to describe the geotechnical characteristics of the formational materials located on site.

The borings were advanced to approximate elevations of -4 feet (MSL datum) to +28.5 feet, using a Mobile B-61 truck-mounted drill rig with hollow-stem augers. Refusal was encountered between elevations of 16 and 28.5 feet in Borings B-1 and B-4, respectively.

The test borings were generally sampled at frequent intervals to the bottom of the boring. Samples were obtained either from a 3-inch O.D., $2^{3/8}$-inch I.D. California ring sampler, or a 3-inch O.D., $2^{3/8}$-inch I.D. Moss sampler. The California ring sampler was advanced approximately 18 inches, when possible, by driving a 140-pound hammer falling 30 inches; samples obtained from the Moss sampler were generally from a 5-foot interval, and obtained by pushing the sampler ahead of the auger as the auger is advanced into the soil. Field logs of the materials encountered in the test borings were prepared by an engineer and were based on a visual examination of the sampled soils, drill cuttings, and the action of the drilling equipment. Samples obtained from the borings were sealed to preserve in-situ moisture and transported to the laboratory for additional inspection and testing.

A Key to Boring Logs is presented as Figure A-1. Final logs of test borings are presented as Figures A-2 through A-5. The descriptions on the logs are based on field logs, sample inspection and laboratory test results.

3.3 Laboratory Testing

Representative samples of the soils observed during our field exploration were tested to verify field classifications and provide strength characteristics. Moisture content and dry density tests were performed to assist in classification of the formational soils. Soil strengths were evaluated by consideration of the dry density-moisture content of the various material types, the penetration resistance of the sampler, and the geologic characteristics of the various materials. In addition, drained direct shear tests were also performed on 5 samples considered representative of the various material types encountered within the formation to aid in evaluating the strength of the materials. A grain size distribution test was performed on material obtained from the binwall.

Results of the moisture content and dry density tests are presented with the penetration resistance of the sampler at the corresponding sample locations on the logs of the test borings. Results of the direct shear tests are shown on Figures A-6 through A-10. The grain size distribution test is presented as Figure A-11.

4.0 GENERAL SITE CONDITIONS

4.1 Section 1 - Vicinity of Outlet Structure

This area was reclaimed during initial construction in 1963 and is illustrated on Cross Sections A - A' through C - C', Figures A-12 and A-13. The subsurface conditions generally consist of a fill material overlying an eroded shelf of Point Loma Formation. Existing surface elevations at the site vary between 17 and 22 feet. In general, the site is overlain by an asphalt pavement of undetermined thickness. The depth and nature of fill material is unknown. Comparisons between a 1960 grading plan and the current topography indicate that the fill depth ranges from 15 to 25 feet. The site is bound on the west by a rock revetment. The revetment extends seaward ranging from 50 to 90 feet from the shoreline. The thickness of the rock revetment is estimated to vary from zero to 20± feet. The revetment is generally comprised of 1/4 to 1-ton stone.

4.2 Section 2 - Lower Hydro-Access Road

This area consists of a bluff of formational materials which is bounded on the east by a cut slope, and on the west by a rock revetment, as illustrated on Cross Sections D - D' and E - E', Figures A-13 and A-14. The bluff, generally comprised of the Point Loma Formation, has near-vertical slopes that range from 15 to 30 feet above the top of the revetment.

A roadway has been cut and benched into the bluff. The roadway consists of approximately 6 inches of asphalt concrete pavement which is underlain by approximately 3 feet of fill materials. Roadway elevations in this area vary from 38 feet to 48 feet.

The cut slope to the east has inclinations on the order of 1/4:1.0 to 1/2:1.0 and varies between approximately 20 and 30 feet in height. The cut slope is comprised of the Point Loma and Bay Point Formations, and unclassified fill.

A rock revetment extends seaward ranging from 20 to 50 feet from the bluff face. The top of the revetment varies between elevation 15 and 20 feet. The thickness of the revetment is estimated to vary from zero to 25± feet. The revetment is generally comprised of 1 to 6-ton stone.

4.3 Section 3 - Lower Hydro-Access Road

This area consists of at least two fills which are buttressed by a rock embankment (Benton, 1960), as illustrated in Cross Sections F - F' and G - G', Figures A-14 and A-15. For purposes of discussion, we have designated these two fills as the "roadway fill" and the "sea cave fill".

The buttress consists of a base layer of filter rock, overlain by core stone with a minimum weight of 1 ton. This is in turn overlain by a cap stone which has a 7-ton minimum weight. The filter material consisted of aggregate sand. The seaward design slope was 2:1 (horizontal to vertical) slope; the landward design slope face was 1½:1 (horizontal to vertical).Review of 1960 grading plans indicates that the rock buttress extends from elevation -10 to elevation +25 feet.

The "roadway fill" extends from the existing rock buttress to a top-of-slope elevation varying between +54 and +64 feet. The extent and nature of the fill material is unknown. The roadway fill appears to have been constructed at slopes on the order of 1.4:1 (horizontal to vertical). However, wave action has locally eroded the slope to an inclination on the order of 1:1 and steeper. Information was not available to determine the extent of benching of the roadway fill into the sea cave fill.

The "sea cave" fill extends from approximate elevation -5 feet to approximately +96 feet. Review of Benton Engineering's report indicates that the material used for this fill was obtained from the areas excavated during grading of the main Treatment Plant site. These materials are comprised of the Cabrillo and Point Loma Formations. Documentation pertaining to the exact composition and properties of this fill material were not available.

4.4 Site 4 - Armco Binwall

The area consists of a filled-in depression which is retained by an Armco Binwall. The binwall is founded on formational materials, as illustrated on Cross Sections H - H' and I - I', Figures A-15 and A-16. The base of the binwall, at its deepest section, is at an approximate elevation of 60 feet; the top of the binwall is at an approximate elevation of +95 feet. The binwall is approximately 15 feet in width and the face is battered at a 1:6 (horizontal to vertical) slope. The fill materials behind the binwall range in thicknesses from 25 feet behind the face of the binwall, to zero (0) where the fill daylights approximately 200 feet landward from the top of the binwall. The nature of the fill material as encountered by our boring consisted of damp sands with gravel, gravel with scrap steel, and silty sand with gravel (Boring B-4, Figure A-5). Grain size information (Figure A-11) from a sample obtained from the binwall and at a depth of 5 feet, indicates the material contained within the binwall consists of silty sands with gravel. Observations of an excavation within the binwall indicates that the gravel content within the binwall is greater than that indicated by the grain size distribution test. In addition, cobbles were observed within the excavation. Approximately 10 feet of Bay Point Formation was encountered above the Point Loma Formation in the vicinity of Boring B-4.

At the base of the bluff, a rock revetment exists which extends approximately 40 to 60 feet seaward. The top of the revetment at the bluff face is at approximate elevation 22 feet. The revetment is comprised of armor stone varying in weight between 6 tons and 10 tons.

4.5 Section 5 - Bluffs South of Southerly Visitor Parking Lot

This area consists of a fill overlying formational materials, as illustrated on Cross Section J - J', Figure A-16. The surface of the fill has an approximate elevation of +96 feet, and the thickness of the fill near the bluff edge ranges from 5 to 12 feet. The fill extends easterly approximately 140 feet where it daylights with the ground surface.

At the base of the bluff, a rock revetment exists which extends approximately 60 to 70 feet seaward. The top of the revetment at the bluff face is at an approximate elevation of +20 feet. The surface of the rock revetment is comprised of armor stone varying in weight between 1 and 2 tons.

4.6 Section 6 - Hydro-Access Road and GUF Access Road

Site conditions in this area are described in paragraphs 4.2 and 4.3.

5.0 FAULTING AND SEISMICITY

The Southern California region is subject to significant hazards from moderate to large earthquakes. For example, approximately 32 damaging earthquakes of estimated magnitude (M) 6 or greater have occurred within 100 miles of San Diego County since about 1800 (Hileman, 1979, Figure 1). All of these earthquakes were at least

60 miles from the site; however, a smaller earthquake in 1862 appears likely to have occurred on a local fault, either onshore or offshore, south or west of San Diego (Legg and Agnew, 1979, p. 42). Based on correlation of the reported effects with the scales in Richter (1958, p. 353), this earthquake is likely to have been a moderate earthquake having a Magnitude of approximately 5½ (M 5½).

The lack of a record of large earthquakes near the site is not a reliable indicator of the actual earthquake hazard because large earthquakes from an individual fault may be separated by hundreds or thousands of years. In order to provide a useful estimate of the earthquake hazard, we have reviewed the record of damaging earthquakes (M 6 or greater), identified the active faults located within 60 miles of the site, and evaluated the record of geologic displacements and smaller earthquakes (less than M 6) for other faults that could be nearby earthquake sources. Our results are summarized in Table A-1.

Three active fault zones pass within 60 miles of the site. The three active fault zones are, in order of proximity to the site, the Coronado Banks, Elsinore, and San Clemente. Four other active fault zones may also cause significant ground motion at the site. The slip type, length and estimated earthquake magnitudes associated with these faults are presented in Table A-1.

The nearest fault zone is the Coronado Banks fault zone, located offshore as close as 7 miles southwesterly of the site (Kennedy and others, 1980, Plate 1). It is reported to run from a point about 20 miles southwest of the City of San Clemente to beyond the Mexican border. The total length of the fault is approximately 50 miles and it is likely to be a strike-slip fault. We have used one-half of its total length as a length of surface rupture to estimate the maximum credible earthquake of M 6-3/4, using the length-magnitude relationship of Slemmons (1977, Figure 27). The Coronado Banks fault zone is near an area where the epicenters of numerous microearthquakes (M 2.0 to 3.4) have been plotted. This fault is similar to the Elsinore fault in that no large earthquakes are confirmed to have originated from it during this century. In our opinion, the Coronado Banks fault zone may be associated with a M 6 earthquake during a typical 100-year period.

Four other possible sources of earthquakes are included in Table A-1, although the reality of the earthquake hazard from them is controversial within the engineering community. These are the Rose Canyon (with two possible types of slip), Vallecitos, San Diego Trough, and Calabasas fault zones.

The nearest of these possible sources of earthquakes is the Rose Canyon fault zone. The Rose Canyon fault zone has generally been considered to be inactive; however, some microearthquakes (M less than 4.0) have epicenters near one possible alignment of the fault. A series of these microearthquakes occurred in June 1985. Moreover, we understand that some evidence of displacement of geologic deposits that may be Holocene in age (up to 11,000 years old) has been discovered near downtown San Diego. One useful guideline for considering a fault to be active for engineering design purposes is whether Holocene-aged deposits are displaced by the fault. Consequently, it may be advisable to consider the hypothetical earthquake hazard from the Rose Canyon fault zone.

The seismic hazard at the site is ground shaking from moderate to large earthquakes that may occur at distances of 7 miles or greater. No active faults are known to be located closer to the site or to pass through the site; therefore, ground rupture is not a hazard. Similarly, the favorable geologic conditions at the site make other seismic hazards negligible. These negligible hazards include earthquake-induced lurching, shallow ground rupture, liquefaction, settlement, and tsunami. Although tsunamis are more of a potential hazard than the other items listed, the largest tsunami recorded in San Diego since 1906 was less than 5-feet high in tidal range. Considering the 20-foot elevation of the lowest facilities at the site, and the favorable nearshore sea-bottom conditions, tsunami of this size would not be expected to adversely affect the plant.

A useful index of the hazard from ground shaking is the peak horizontal acceleration that may be expected at the site from the maximum probable earthquake occurring on a fault at the point of least distance to the site. This approach is conservative because the maximum probable earthquake is likely to occur on some other segment of the fault instead of at the point of closest approach to the site. The least distance to the site for each active fault, and other potential sources of earthquakes, is shown in Table A-1.

The maximum peak horizontal acceleration expected from the active faults is 0.28 g from the Coronado Banks fault zone. The other faults are also potential sources of significant acceleration. In particular, the 0.04 g acceleration from a large earthquake on the San Andreas fault may have a duration three to five times that of the strong ground motion from the Coronado Banks fault zone. Ground shaking from the San Andreas fault may also contain a high proportion of long-period ground motion.

6.0 DESIGN CONSIDERATIONS

6.1 Design Parameters

A critical factor in any geotechnical evaluation is the selection of design parameters. Our selection of design parameters was based upon our site reconnaissance, field investigation, laboratory testing, correlations in published literature, and parametric evaluations of existing conditions. Design parameters were selected for the unclassified fills and the formational materials. Parameters utilized in our evaluation are summarized in Table A-2. Information pertaining to the existing fills was unavailable at the time of our investigation. As such, parameters for the fill soils were based on typical conservative values for compacted soils obtained from the literature. The parameters chosen for the formational materials represent a lower estimate of anticipated strengths.

6.2 Tied-Back Walls

The geotechnical factors influencing the design of a tied-back wall system includes the stability of the slope, the strength of the anchor zone, the type of anchor system, and the forces acting on the tied-back wall system. The results of our analyses indicate that the sea cliffs are generally stable with respect to deep-seated, rotational-type failures. However, the bluffs are susceptible to local instabilities associated with undermining and erosion due to wave action. The undermining appears to produce a series of block-type failures (blockfall) resulting in bluff retreat. Results of our investigation also indicate that the formational materials are highly jointed in localized areas. The joint pattern is both perpendicular and parallel to the cliff face. Formational materials encountered in our borings, in general, are suitable for supporting the proposed anchor systems.

6.3 Gravity Structures

The gravity structures associated with this project consist of cribwalls and Reinforced Earth walls. Geotechnical considerations associated with the design of gravity structures include bearing capacity, settlement, earth pressures acting on the gravity structure, and other external forces such as those due to waves. Present plans indicate that, in general, the gravity-type structures will be founded on undocumented fill materials. We anticipate that these materials will provide adequate bearing support; however, we anticipate that the compressibility of these materials will be variable. As such, the gravity wall systems need to be flexible in order to accommodate the anticipated moderate differential settlements. Recommendations for bearing capacity, earth pressures, and estimates of differential settlements are provided in Section 7.3.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 General

Based on geotechnical considerations, the structures currently proposed for shoreline protection at the Point Loma Treatment Plant may be constructed within the materials encountered at the site, provided the recommendations in this report are incorporated into the design of the structures.

7.2 Tied-Back Walls

Two different applications of tied-back walls are anticipated for the shore protection on this project. One system consists of a tied-back wall for the purpose of restraining the fill materials within the existing Armco Binwall. The second system consists of a tied-back wall used to stabilize currently unprotected sea cliffs.

7.2.1 General Anchor Design

We anticipate that anchor design will consist of bar tendons grouted into an inclined anchor hole and post-tensioned against the face of the wall. Either friction anchors or belled anchors could be used for the tie backs. However, for the conditions at this site, it has been our experience that friction anchors involve fewer installation problems and provide more uniform support than belled anchors. Therefore, we recommend straight, friction anchors.

Allowable anchor capacity is based upon the surface area of bonded anchor. We recommend a minimum anchor diameter of 12 inches. Allowable anchor loads for both types of tie backs can be calculated from the following equation:

$$T_{all} = 1.65\ L \left(\frac{d}{12}\right) (0.032z + 0.7)$$

Where: L = effective length of anchor measured in feet.
d = anchor diameter measured in inches (12 inches minimum).
z = depth of anchor (below the ground surface), measured in feet.
T_{all} = allowable anchor capacity in kips.

For example, the computed allowable anchor capacity for an anchor with 50 feet of effective embedment, a diameter of 12 inches, and approximately 10 feet below the ground surface is 82.5 kips. This equation utilizes a factor of safety of 2.0 against ultimate pullout resistance.

All tied-back walls should be constructed and proof-tested in the field according to specifications which will be provided in Phase 2 Project Documents.

7.2.2 <u>Tied-Back Walls Supporting Armco Binwalls</u>

Tied-back anchors used to support the existing Armco Binwall should be designed so that the effective embedment length for the anchors is calculated from that portion of the anchor embedded into formational materials. The approximate depth of the formational soils are illustrated on Cross Section H - H', Figure A-15. The load carrying portion of the anchor should not extend into the existing fills. Tied-back anchors should be designed on a 20 to 30 degree downward slope measured from the horizontal at the face of the binwall.

The tied-back wall should be designed to accommodate earth pressures and vehicle loads. We recommend that the tie-back wall be designed for an active earth pressure equivalent to a fluid pressure of 28 pounds per cubic foot. This assumes that the soil behind the retaining wall will consist of compacted granular soils. Recommended earth pressures do not include hydrostatic loading. If drainage is not provided or maintained immediately behind the wall, hydrostatic forces need to be included in the design.

In addition, to simulate the loading due to the periodic movement of heavy truck traffic, an additional uniform pressure of 60 psf should be assumed to act on the entire wall. This value assumes that the vehicle will be a minimum of 10 feet from the tied-back wall and will be limited to a tandem axle truck with 18,000 pound maximum axle load weight. Walls subjected to surcharge loads, applied at a distance behind the wall equal to the wall height, should be designed for an additional uniform pressure equal to, or less than, 0.3 times the surcharge load.

7.2.3 Tie Backs for General Bluff Stabilization

Tie backs used to restrain walls for general bluff stabilization should be designed to resist a uniform lateral pressure of 7H, where H is the total vertical height of the sea cliff, which will likely be in excess of the total height of tied-back wall. All anchors should be embedded in formational materials and the bond length of the anchor measured from a point a minimum of 30 feet from the bluff face. Tie backs should be constructed at a 10± degree downward angle measured from horizontal at the face of the bluff.

7.3 Gravity Retaining Walls

Allowable Bearing Pressures - We recommend that gravity-retaining wall structures be designed for an allowable bearing pressure of 3,000 pounds per square foot. We estimate that differential settlements for the cribwalls will be on the order of 1/2 to 1 inch between structural components of the system. Differential settlements for the Reinforced Earth structure are expected to be less than 1/2-foot locally.

<u>Active Earth Pressures</u> - Construction of the proposed gravity-type structures within Sections 3 and 6 are anticipated to consist of cribwalls and Reinforced Earth retaining walls. We recommend that these structures be designed to resist the load imposed by an active earth pressure equivalent to a fluid weighing 35 pounds per cubic foot. This assumes that soil behind the retaining wall will consist of compacted granular soils. It also assumes level backfill conditions and that no surcharge loads exist. For walls where backfill slopes are anticipated to be constructed at inclinations as steep as 2 to 1, the walls should be designed for an active equivalent fluid pressure of 40 pounds per cubic foot. Recommended earth pressures do not include hydrostatic loading. If drainage is not provided or maintained immediately behind the wall, hydrostatic forces need to be included in the design. Walls subject to surcharge loads, applied at a distance behind the wall equal to, or less than, the wall height, should be designed for an additional uniform pressure equal to 0.3 times the surcharge load.

<u>Lateral Resistance</u> - To provide resistance for design lateral loads, we recommend that passive pressure be assumed equivalent to a fluid pressure of 350 pcf for footings and shear keys poured neat against sides of excavations. This value assumes a horizontal surface for the soil mass extending at least 10 feet from the base of the wall or 3 times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of materials in areas susceptible to erosion should not be included in design for passive resistance to lateral loads. If friction is to be used to resist lateral loads, we recommend a coefficient of friction of 0.35 between the soil and the base of the footing. If it is desired to combine friction and passive resistance in design, we recommend using a reduced friction coefficient of 0.25.

<u>Construction Considerations</u> - Construction of the proposed gravity structures within Sections 3 and 6 will necessitate the steepening of existing slopes. Review of the 1960 grading plans and current topography indicates that several of the temporary construction cuts will be made in undocumented fill. Geometric constraints due to the existing roadways, buildings, and proposed retaining

structure alignments will necessitate relatively steep and high cut slopes. Within Section 3, the inclination of the temporary construction cut slope is estimated to be on the order of 3/4 to 1 (horizontal to vertical) and is anticipated to have a height of up to 50 feet. Within Section 6, the cut slope inclination is estimated to be approximately 1 to 1 (horizontal to vertical) and have maximum cut heights on the order of 35 feet.

Estimates of slope stability for the proposed construction cut slopes indicate that these slopes may be marginally stable and local failures may occur. The length of time the slope is exposed, weather conditions, and construction procedures all could contribute to the potential instability of the slope. As such, consideration should be given to monitoring slopes during the construction excavations.

7.4 Seismic Design Considerations

Dynamic lateral forces are imposed upon retaining structures during seismic shaking. Although it may not be mandatory to include seismic loading in the sizing of structures and/or anchor assemblies, consideration should be given to mitigating a potential failure from overstressing foundation components during a design earthquake such as the maximum probable earthquake. Seismic loading on any earth retaining structure on site should be computed based on the following formula:

$$P_S = 45gH$$

Where P_S = Uniformly distributed seismic pressure in psf
g = The peak horizontal acceleration for the design earthquake; expressed in terms of percent g (i.e. 0.1, 0.2, etc.)
H = The height of the gravity wall in feet, or the total vertical height of the sea cliff, which will likely be higher than the tied-back wall.

For example, at Section 2, for a 30-foot-high tied-back wall where the ground surface is at elevation 44 feet and the shore platform is likely at elevation -4 feet (H = 48 feet), a design horizontal acceleration of 0.1g would result in a seismically-induced lateral pressure of 216 psf. This uniform pressure is added to the static wall pressures recommended in Sections 7.2 and 7.3 of this report.

REFERENCES

Benton Engineering, Inc., 1960, "Soil and foundation investigation, sewer treatment plant site, City of San Diego, California."

Hileman, J.A., 1979, Seismicity of the San Diego region in Abbott, P.L., and Elliott, W.J. (eds), Earthquakes and other perils, San Diego region, p. 11-20.

Kennedy, M.P., Clark, S.H., Greene, H.G., and Legg, M.R., 1980, Recency and character of faulting offshore metropolitan San Diego, California: California Division of Mines and Geology Map Sheet 42.

Legg, Mark, and Agnew, D.L., 1979, The 1862 earthquake in San Diego in Abbott, P.L., and Elliott, W.J. (eds), Earthquakes and other perils, San Diego region, p. 139-142.

Richter, C.F., 1958, Elementary seismology: W.H. Freeman, San Francisco, 768 p.

Seed, Bolton, and Whitman, Robert V., 1970, "Design for earth retaining structures for dynamic loads," in Lateral stresses in the ground and design of earth-retaining structures, American Society of Civil Engineers, New York, p. 103-147.

Slemmons, D.B., 1977, State-of-the-art for assessing earthquake hazards in the United States, Report 6, Faults and earthquake magnitudes: U.S. Army Engineer Waterways Experiment Station Miscellaneous Paper S-73-1, 128 p.

November 1, 1960 (plans approved), November 3, 1961 (as-built), Plans for the Construction of Cabrillo Road and Site Grading for Sewage Treatment Plant, 17 sheets, City of San Diego Drawing Nos. 9074-1D to 9074-17D.

TABLE A-1

EARTHQUAKE DESIGN PARAMETERS FOR
ACTIVE FAULTS WITHIN 60 MILES OF THE SITE
AND SELECTED OTHER FAULTS

ACTIVE FAULT OR FAULT ZONE	SLIP TYPE	FAULT LENGTH (miles)	MAXIMUM CREDIBLE (MC) EARTHQUAKE[1] (M)	MAXIMUM PROBABLE(MP) EARTHQUAKE[2] (M)	LEAST DISTANCE TO SITE[3] (miles)	PEAK HORIZONTAL ACCELERATION FOR MP[4] (g)
Coronado Banks	Strike	50	6-3/4	6	7	0.28
San Clemente	Strike	100	7-1/4	6-1/2	38	0.07
Elsinore	Strike	200	7-1/2	6-3/4	47	0.06
San Jacinto	Strike	150	7-1/2	7	68	0.04
San Andreas	Strike	500+	8-1/4	7-3/4	96	0.04
Agua Blanca	Strike	105	7-1/4	6-3/4	62	0.04
San Miguel	Strike	92	7-1/4	6-3/4	70	0.03
<u>OTHER FAULTS</u>						
Rose Canyon	Reverse	14	7	6	6	0.32
Rose Canyon	Strike	66	7	6	6	0.32
San Diego Trough	Oblique ?	62	7	6	17	0.13
Vallecitos	Strike	40	6-3/4	6	33	0.04
Calabasas	Oblique ?	18	6-3/4	6	46	0.03

1 Estimated to be the maximum earthquake capable of occurring. Derived by using one-half of the fault length as the length of surface rupture in an earthquake along the main fault in Figure 27 of Slemmons, 1977. Adjustments of up to 1/4-magnitude unit have been made based on consistency with the geologic expression of fault displacement.

2 Estimated to be the maximum earthquake likely to occur during a typical 100-year interval. Magnitude values are a judgement based on regional seismicity, earthquake activity near the fault, and geologic expression of fault displacement.

3 Measured from Jennings (1975), and Kennedy and others (1980).

4 Estimated from Schnabel and Seed (1973).

TABLE A-2

GEOTECHNICAL PARAMETERS

Material Type	Strength Characteristics C (psf)	ϕ (deg)	Total Unit Weight γt (pcf)	Qualitative Compressi-bility
Fill near outlet structure	200	31	120	Variable, moderate to high
Fill near sea cave	200	31	120	Moderate
Fill near binwall	200	31	120	Moderate
Bay Point Foprmation	200	34	125	Low
Point Loma Formation				
Peak	1,800	36	130	Low
Residual	1,100	33	130	Low

KEY TO EXCAVATION LOGS

LOGGED BY:	DATE DRILLED:	BORING ELEVATION:	BORING NUMBER:
DRILL RIG:	BORING DIAMETER:	HAMMER WT.: DROP:	

DEPTH (feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	DESCRIPTION	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
5			14	▽	Medium dense, moist, brown SILTY FINE SAND (SM)			

Unified Soil Classification

Water Table Measured On Date Indicated

Number of Blows Required to Advance Sampler One Foot

Sample Type:

- **B** **Plastic Bag**
- **CA** **California Drive with Rings**
- **CO** **Moss Sampler**

Sample Location

Depth Below Surface Elevation

Indicates Samples Tested for Other Properties:

DS **Direct Shear**

NOTES ON FIELD INVESTIGATION

1. Borings were advanced using a truck-mounted Mobile B-61 drill rig with an 8-inch hollow-stem auger.

2. California and Moss samplers were used to obtain soil samples. The California samplers were driven into the soil at the bottom of the borings with a 140-pound hammer falling 30 inches. When the samplers were withdrawn from the boring, the samplers were removed, visually classified, sealed in plastic containers, and taken to the laboratory for testing.

   The California sampler is an 18-inch-long, 2-1/2-inch inside diameter, 3-inch outside diameter, thick-walled sampler. The sampler is lined with 18 2-3/8-inch inside diameter brass rings. Relatively undisturbed, intact soil samples are retained in the brass rings.

   The Moss sampler is a continuous, thick-walled sampler with 2-3/8 inside diameter brass rings. The sampler is pushed into the soil ahead of the auger as the auger is advanced into the soil. Continuous, relatively undisturbed samples up to 5 feet in length may be obtained.

3. Free groundwater was encountered in some borings as shown on the logs.

4. Classifications are based upon the Unified Soil Classification System and include color, moisture and consistency. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 1

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG

LOGGED BY: MWE	DATE DRILLED: 12-2-87	BORING ELEVATION: 96.6 feet (MSLD)	BORING NUMBER:
DRILL RIG: Mobile B-61	BORING DIAMETER: 8-inch	HAMMER WT.: 140 lbs. DROP: 30 in.	B - 1

DEPTH (Feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	DESCRIPTION	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
					Dry to damp, light brown SILTY SAND (SM), with gravel			
	1	CA	62		FILL	4.8	112.4	
	2	SK						
5	3	CA	66		Dense, damp, reddish-brown CLAYEY SAND (SC), with cobbles			
					BAY POINT FORMATION			
					Hard, moist to damp, reddish-brown SANDY CLAY (CL), with gravel			
10	4	CA	93					
					Very dense, dry to damp, light yellowish-brown SILTY SAND (SM) (partially cemented)			
15	5	CA	50/ 2 =					
					gravel lens			
20	6	CA	120		Dense, damp, reddish-brown CLAYEY SAND (SC-CL)			
25	7	CA	36			8.3 8.9	122.8 120.8	DS DS
30	8	CA	46					
35	9	CA	50/ 3=					
					Hard, damp, olive-gray FINE SANDY CLAY (CL), with interbeds of light tan, SILTY FINE SAND (SM-ML), with gypsum filled joints (1/16 inch to 1/4 inch wide)			
40	10	CA	100/ 6=		Laminar bedding - estimate 4" average bed thickness			
					POINT LOMA FORMATION			
45	11	CO						
	12	CO						
50								

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 2 a

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

B O R I N G   L O G

LOGGED BY: MWE	DATE DRILLED: 12-2-87	BORING ELEVATION: 96.6 feet (MSLD)	BORING NUMBER:
DRILL RIG: Mobile B-61	BORING DIAMETER: 8 inch	HAMMER WT.: 140 lbs. DROP: 30 in.	B - 1

DEPTH (feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	DESCRIPTION	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
					Hard, damp, very dark gray SILTY CLAY (CL-CH) (massive appearance with few thin interbeds and tight sparse jointing)			
	13	CO						
					POINT LOMA FORMATION			
55								
	14	CO						
60	15	CA	100/3"					
	16	CO						
	17	CO						
65	18	CO						
	19	CO						
70	20	CA	100+			17.3	113.6	DS
						16.1	112.8	DS
						15.8	114.7	DS
75	21	CA	100+			14.1	107.2	DS
	22	CO				14.5	110.4	DS
	23	CO						
80								
					BOTTOM OF BORING at 80½ feet - Refusal No groundwater encountered at time of drilling			
85								
90								
95								
100								

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 2 b

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG

LOGGED BY: MWE	DATE DRILLED: 12-3-87	BORING ELEVATION: 38.2 feet (MSLD)	BORING NUMBER:
DRILL RIG: Mobile B-61	BORING DIAMETER: 8 inch	HAMMER WT.: 140 lbs. DROP: 30 in.	B - 2

DEPTH (feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	DESCRIPTION	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
					A.C. pavement - approximately 6 inches thick			
					Damp, brown-gray SILTY SAND (SM), with gravel			
	1	CO						
5								
	2	CO			FILL			
					Hard, damp, olive-gray SILTY CLAY (CL), with interbeds of very dense, damp, light tan SILTY FINE SAND (SM)			
	3	CO						
10								
	4	CO			POINT LOMA FORMATION			
	5	CO						
15					— water seepage			
	6	CO						
	7	CO						
	8	CO						
20					— lime cemented layers			
	9	CO						
	10	CO			— lime cemented layers			
25								
	11	CO						
	12	CO						
30								
	13	CO						
	14	CO						
35								
	15	CO						
					— water seepage			
	16	CO						
40								
					BOTTOM OF BORING at 40 feet			
45								
50								

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 3

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

B O R I N G   L O G

LOGGED BY: MWE	DATE DRILLED: 12-4-87	BORING ELEVATION: 45.9 feet (MSLD)	BORING NUMBER:
DRILL RIG: Mobile B-61	BORING DIAMETER: 8 inch	HAMMER WT.: 140 lbs. DROP: 30 in.	B - 3

DEPTH (feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	D E S C R I P T I O N	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
					A.C. pavement - approximately 6 inches thick			
					Brown SILTY SAND (SM) with gravel base FILL			
					Hard, damp, gray SILTY CLAYSTONE (CL), interbedded with very dense, damp, brown SILTY to CLAYEY SAND (SM-SC)			
5	1	CO						
	2	CO			POINT LOMA FORMATION			
10	3	CO						
					cemented zone			
	4	CO						
15	5	CO						
	6	CO						
20	7	CO						
	8	CO			cemented zone			
25	9	CO						
	10	CO						
30	11	CO			cemented zone			
	12	CO						
35	13	CO						
	14	CO						
40	15	CO						
	16	CO						
45	17	CO						
					water seepage			
	18	CO						
50					BOTTOM OF BORING at 49 feet			

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 4

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

B O R I N G   L O G

LOGGED BY: BRS/WEG	DATE DRILLED: 12/7/87	BORING ELEVATION: 94.5 feet (MSLD)	BORING NUMBER:
DRILL RIG: Mobile B-61	BORING DIAMETER: 8 inch	HAMMER WT.: 140 lbs. DROP: 30 in.	B - 4

DEPTH (feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	D E S C R I P T I O N	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
					A.C. pavement - approximately 8-inches thick over 4-inch aggregate base			
					Damp, gray-brown SAND (SP), with gravel			
					FILL			
5					Damp, red-brown SILTY SAND (SM)			
					Damp, dark gray SANDY GRAVEL (GP), with pieces of scrap steel (1 inch average angular crushed rock)			
10								
15								
					Very moist, brown SILTY SAND (SM), with gravel			
20					water seepage			
	1	B						
25	2	CA	44					
					Medium dense, damp to moist, mottled red, brown and gray CLAYEY SAND (SC)			
	3	B						
30					BAY POINT FORMATION			
35	4	CA	55/6"		Hard, damp, olive-gray SILTY CLAY (CL), with interbeds of light tan SILTY FINE SAND (SM-ML)	16.9 17.2 17.7 17.9	112.3 107.3 107.8 104.8	DS DS DS DS
	5	CO						
40					gypsum and lime in seams and joints			
	6	CO			POINT LOMA FORMATION			
	7	CO						
45	8	CO						
	9	CO						
50								

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 5 a

B O R I N G   L O G

LOGGED BY: BRS/WEG	DATE DRILLED: 12/7/87	BORING ELEVATION: 94.5 feet (MSLD)	BORING NUMBER:
DRILL RIG: Mobile B-61	BORING DIAMETER: 8 inch	HAMMER WT.: 140 lbs. DROP: 30 in.	B - 4

DEPTH (feet)	SAMPLE NO.	TYPE	BLOWS/FOOT	GROUND-WATER	DESCRIPTION	MOISTURE CONTENT %	DRY DENSITY pcf	OTHER TESTS
	10	CO			Hard, damp, olive-gray SILTY CLAY (CL), with interbeds of light tan SILTY FINE SAND (SM-ML)			
	11	CO			POINT LOMA FORMATION			
55	12	CO						
	13	CO			cemented zone			
60	14	CO						
	15	CO			cemented zone			
65								
					BOTTOM OF BORING at 66 feet - Refusal to drill on hard claystone			
70								
75								
80								
85								
90								
95								
100								

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE NO.: A - 5 b

GROUP DELTA CONSULTANTS, INC.
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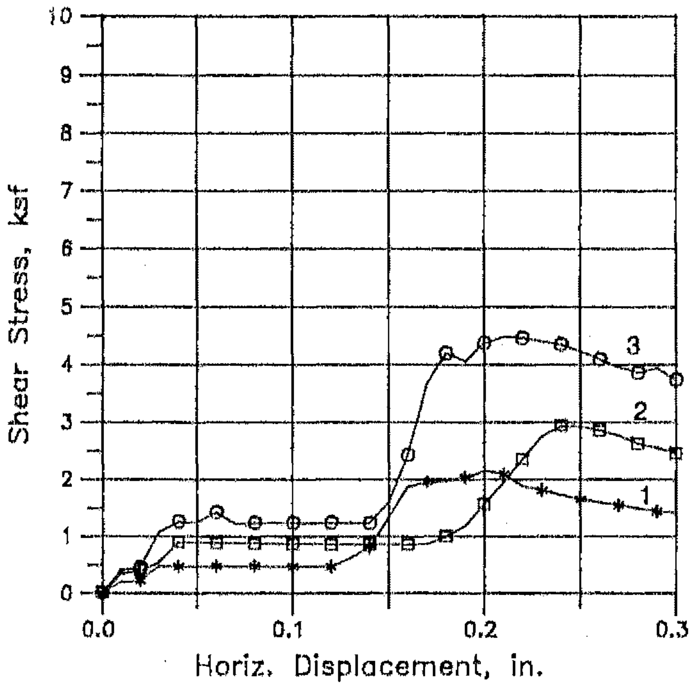
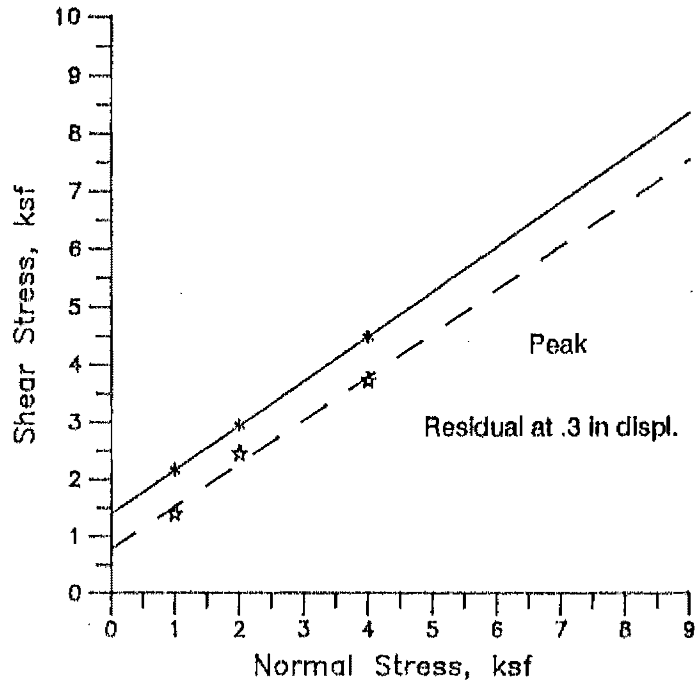




NOTE: OBSERVED HORIZONTAL DISPLACEMENT INDICATES PROBABLE SAMPLE DISTURBANCE IN THE OUTER ZONE OF SAMPLE

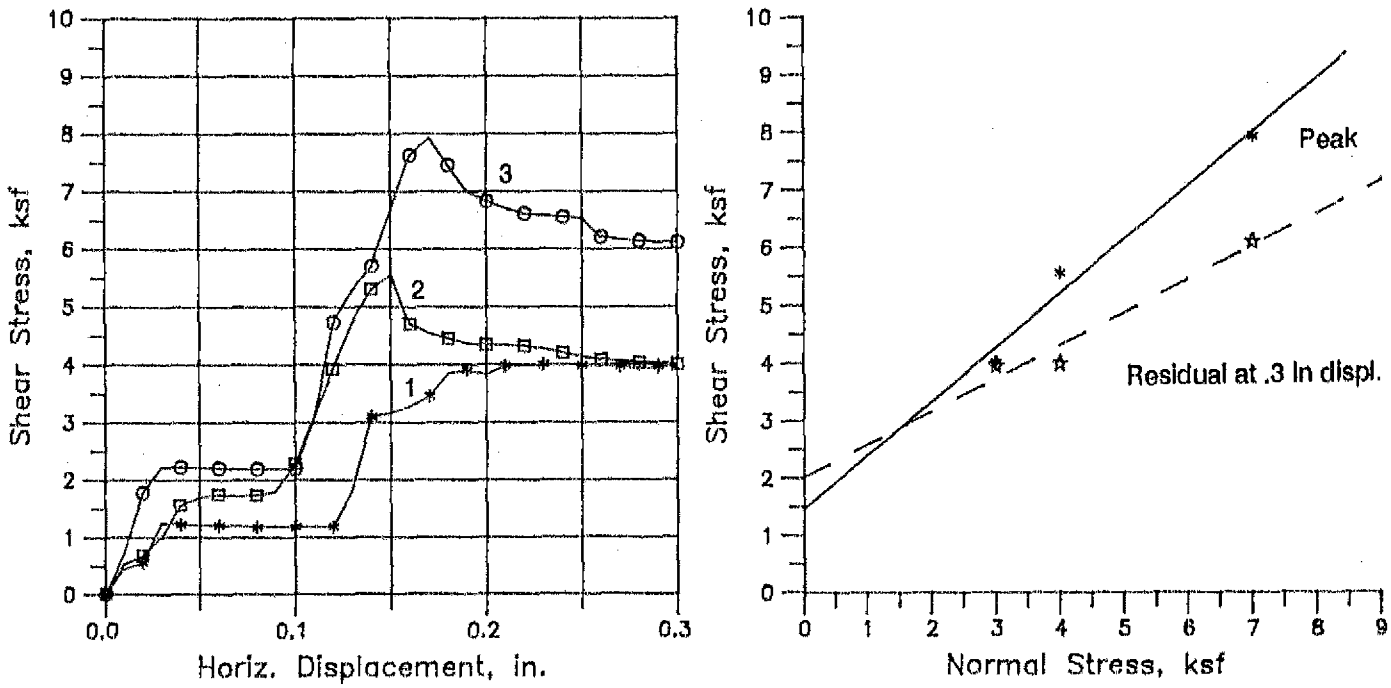
SAMPLE DATA					
Sample Number B1-7					
Sample/Classification BROWN PARTIALLY-CEMENTED SILTY SAND (Qbp)					
Specimen Number	1	2	3		
Height, inches	1.000	1.000	1.000		
Diameter, inches	2.375	2.375	2.375		
Initial Dry Density, pcf	118.3	122.8	120.8		
Initial Moisture Content, %	8.4	8.3	9.0		
Initial Saturation, %	56	63	65		
Final Dry Density, pcf					
Final Moisture Content, %					
Final Saturation, %					
Normal Stress, psf	1000	2000	4000		

TEST DATA	
Type of Test: CONSOLIDATED DRAINED	
Angle of Friction, Effective Ø' = 34° @ peak	
Cohesion, Effective C' = 7.4 ksf @ peak	Rate of Shear, in/min 0.024

DIRECT SHEAR

PROJECT: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE: A-6

GROUP DELTA CONSULTANTS, INC.



NOTE: OBSERVED HORIZONTAL DISPLACEMENT INDICATES PROBABLE SAMPLE DISTURBANCE IN THE OUTER ZONE OF SAMPLE

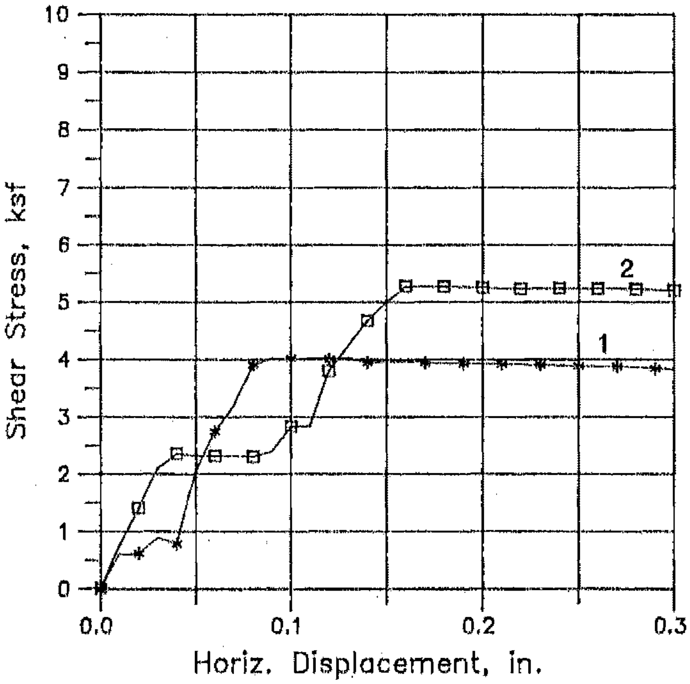
SAMPLE DATA					
Sample Number B1-20					
Sample/Classification BROWN SANDSTONE (Kp)					
Specimen Number	1	2	3		
Height, inches	1.000	1.000	1.000		
Diameter, inches	2.375	2.375	2.375		
Initial Dry Density, pcf	113.6	112.8	114.7		
Initial Moisture Content, %	17.3	16.1	15.8		
Initial Saturation, %	100	92	95		
Final Dry Density, pcf					
Final Moisture Content, %					
Final Saturation, %					
Normal Stress, psf	3000	4000	7000		

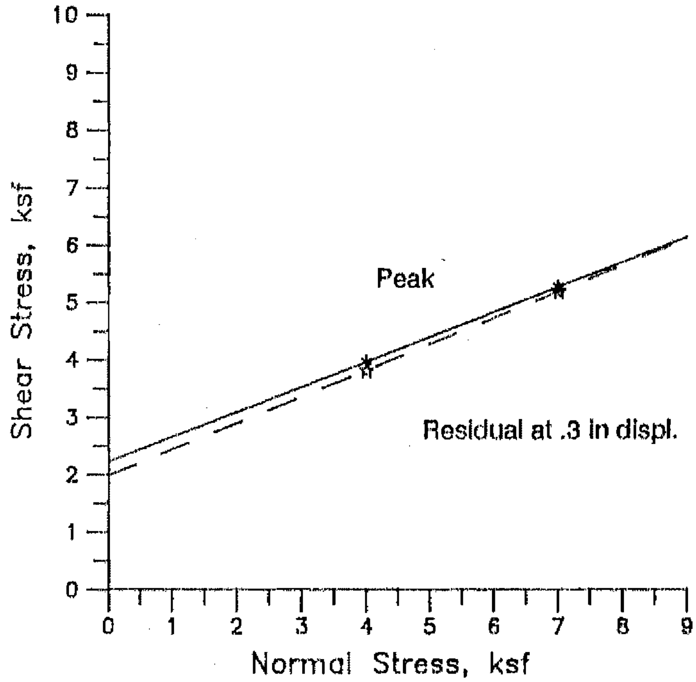
TEST DATA	
Type of Test: CONSOLIDATED DRAINED	
Angle of Friction, Effective Ø' = 40° @ peak	
Cohesion, Effective C' = 1.4 ksf @ peak	Rate of Shear, in/min 0.024

DIRECT SHEAR

PROJECT: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE: A-7

GROUP DELTA CONSULTANTS, INC.





NOTE: OBSERVED HORIZONTAL DISPLACEMENT INDICATES PROBABLE SAMPLE DISTURBANCE IN THE OUTER ZONE OF SAMPLE

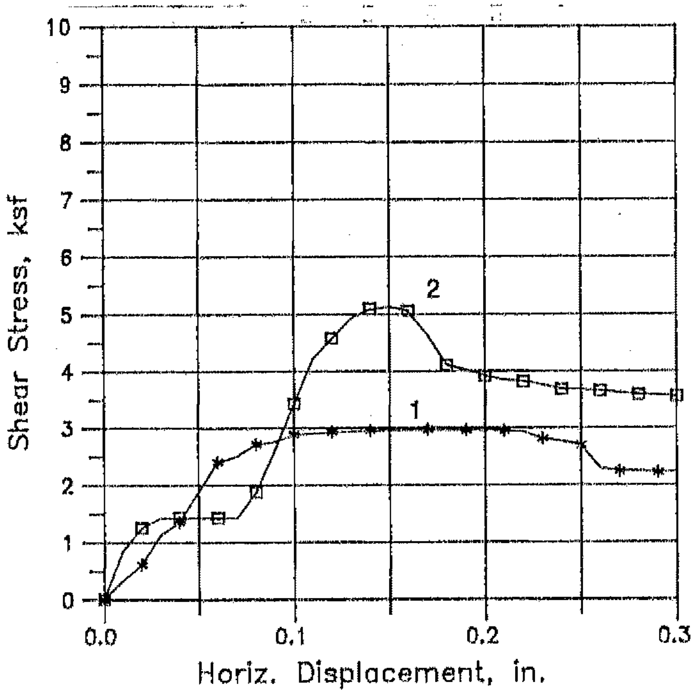
SAMPLE DATA					
Sample Number B1-21					
Sample/Classification GRAY MUDSTONE (Kp)					
Specimen Number	1	2			
Height, inches	1.000	1.000			
Diameter, inches	2.375	2.375			
Initial Dry Density, pcf	110.4	107.2			
Initial Moisture Content, %	14.5	14.1			
Initial Saturation, %	77	69			
Final Dry Density, pcf					
Final Moisture Content, %					
Final Saturation, %					
Normal Stress, psf	4000	7000			

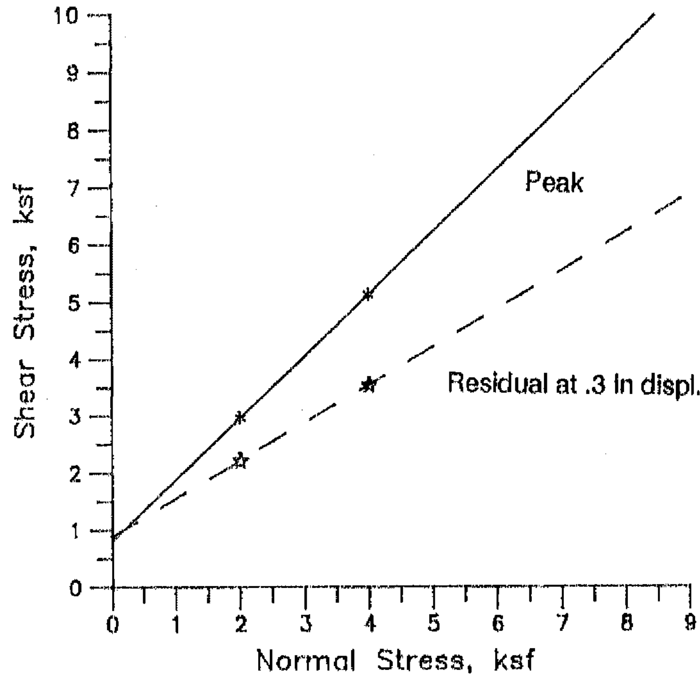
TEST DATA	
Type of Test: CONSOLIDATED DRAINED	
Angle of Friction, Effective Ø' = 21° @ peak	
Cohesion, Effective C' = 2.25 ksf @ peak	Rate of Shear, in/min 0.024

DIRECT SHEAR

PROJECT:	POINT LOMA TREATMENT PLANT	FIGURE: A-8

GROUP DELTA CONSULTANTS, INC.





NOTE: OBSERVED HORIZONTAL DISPLACEMENT INDICATES PROBABLE SAMPLE DISTURBANCE IN THE OUTER ZONE OF SAMPLE

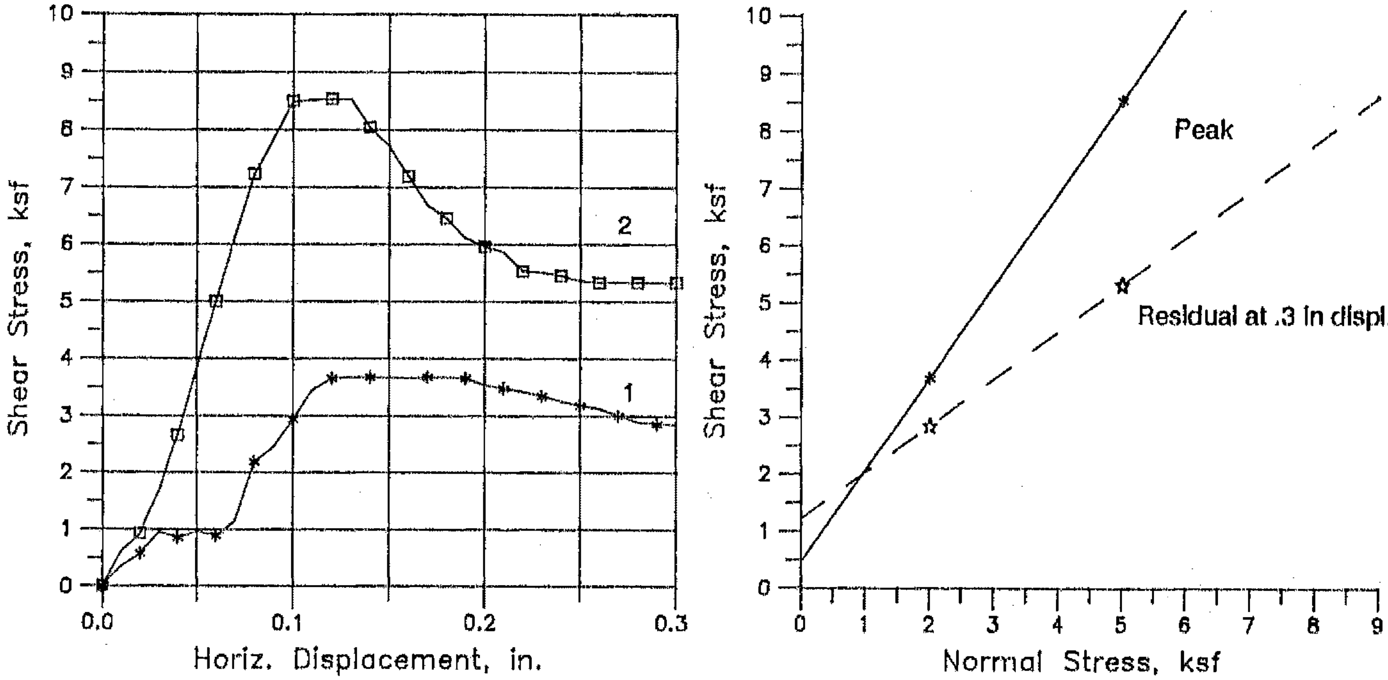
SAMPLE DATA					
Sample Number B4-4 SANDSTONE INTERBED					
Sample/Classification BROWN SANDSTONE (Kp)					
Specimen Number	1	2			
Height, inches	1.000	1.000			
Diameter, inches	2.375	2.375			
Initial Dry Density, pcf	107.8	104.8			
Initial Moisture Content, %	17.7	17.9			
Initial Saturation, %	88	82			
Final Dry Density, pcf					
Final Moisture Content, %					
Final Saturation, %					
Normal Stress, psf	2000	4000			

TEST DATA	
Type of Test: CONSOLIDATED DRAINED	
Angle of Friction, Effective ϕ' = 44° @ peak	
Cohesion, Effective C' = 0.8 ksf @ peak	Rate of Shear, in/min 0.024

DIRECT SHEAR

PROJECT: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE: A-9

GROUP DELTA CONSULTANTS, INC.



NOTE: OBSERVED HORIZONTAL DISPLACEMENT INDICATES PROBABLE SAMPLE DISTURBANCE IN THE OUTER ZONE OF SAMPLE

SAMPLE DATA					
Sample Number B4-4	MUDSTONE INTERBED				
Sample/Classification	GRAY MUDSTONE INTERBEDDED WITH BROWN SANDSTONE (Kp)				
Specimen Number	1	2			
Height, inches	1.000	1.000			
Diameter, inches	2.375	2.375			
Initial Dry Density, pcf	112.3	107.3			
Initial Moisture Content, %	16.9	17.2			
Initial Saturation, %	95	84			
Final Dry Density, pcf					
Final Moisture Content, %					
Final Saturation, %					
Normal Stress, psf	2000	5000			

TEST DATA	
Type of Test: CONSOLIDATED DRAINED	
Angle of Friction, Effective Ø' = 55° @ peak	
Cohesion, Effective C' = 0.5 ksf @ peak	Rate of Shear, in/min 0.024

DIRECT SHEAR

PROJECT: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE: A-10

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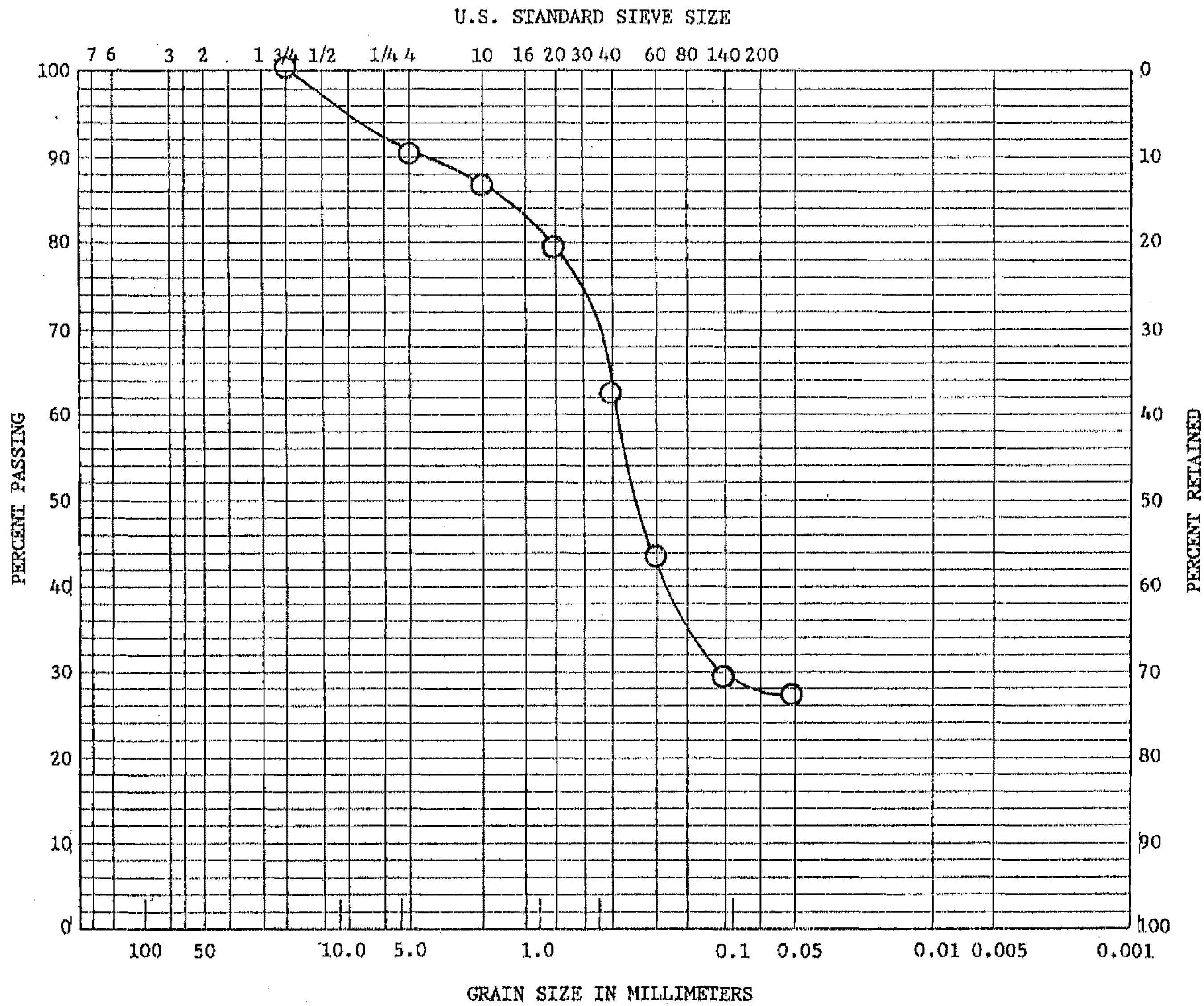
COBBLES	GRAVEL		SAND			SILT and CLAY
	Coarse	Fine	Coarse	Medium	Fine	

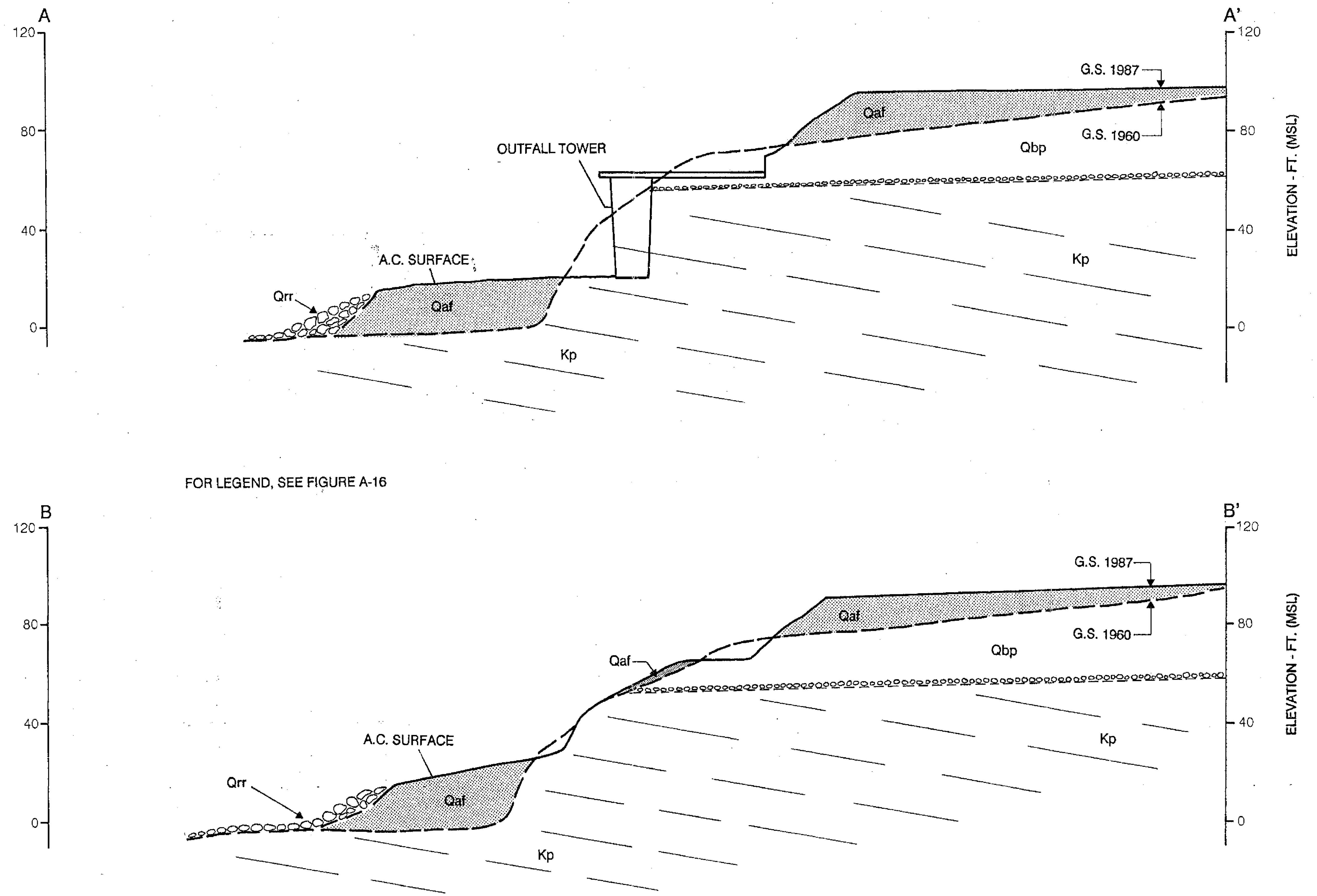


SAMPLE	DEPTH	SYMBOL	CLASSIFICATION	LL	PI
BINWALL BACKFILL	5'	—○—	LIGHT BROWN, SILTY SAND (SM)	23	7

GRAIN SIZE DISTRIBUTION

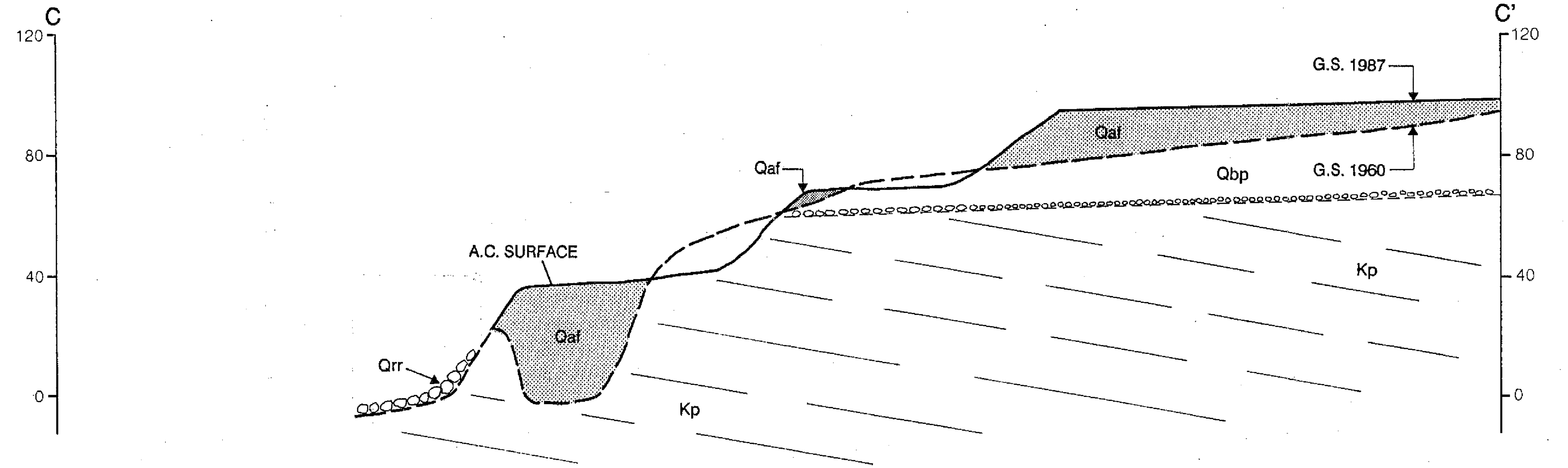
PROJECT: 1089-ES01	POINT LOMA TREATMENT PLANT	FIGURE: A-11

GROUP DELTA CONSULTANTS, INC.

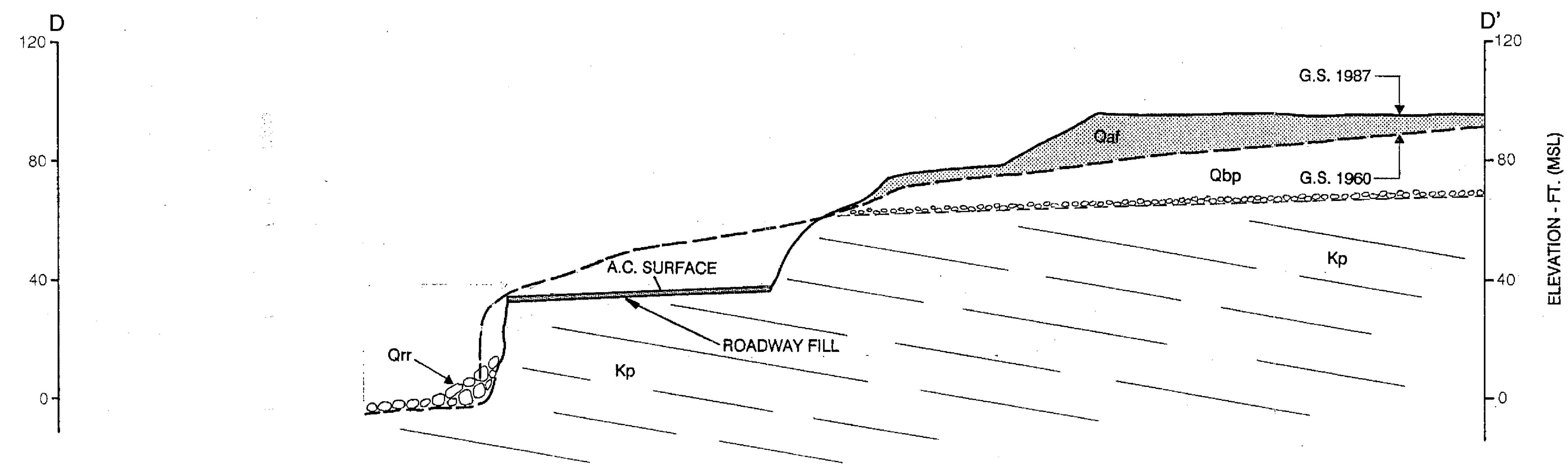


Project		Project No.	Figure
POINT LOMA TREATMENT PLANT	**GENERALIZED GEOLOGIC CROSS SECTIONS A - A' & B - B'**	1089-ES01	A-12

GROUP DELTA CONSULTANTS, INC.

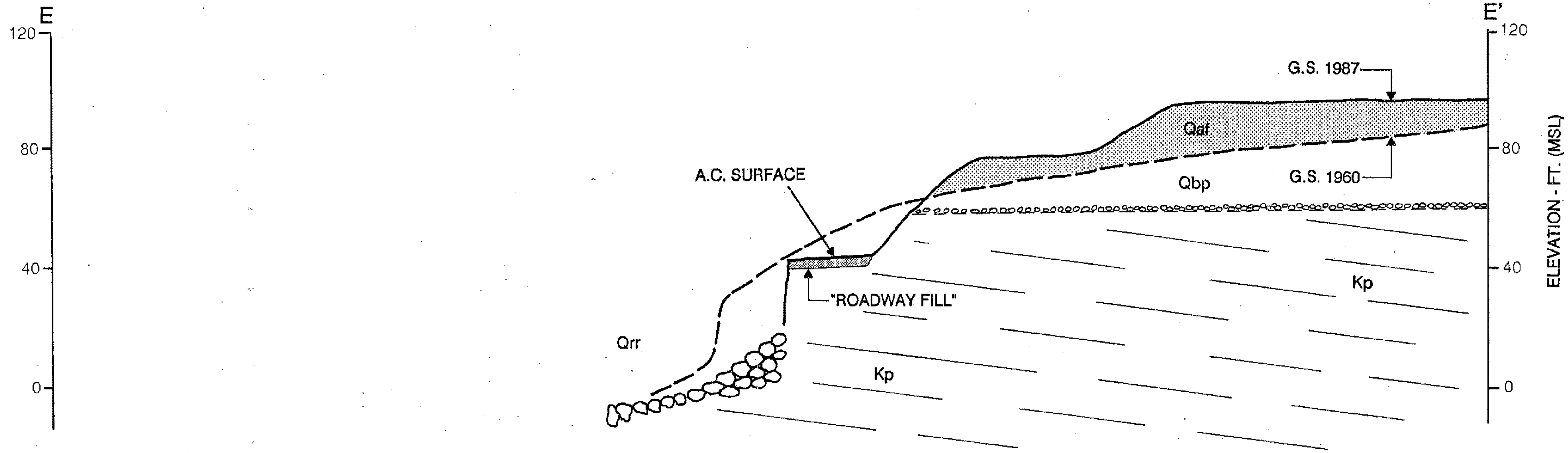


FOR LEGEND, SEE FIGURE A-16

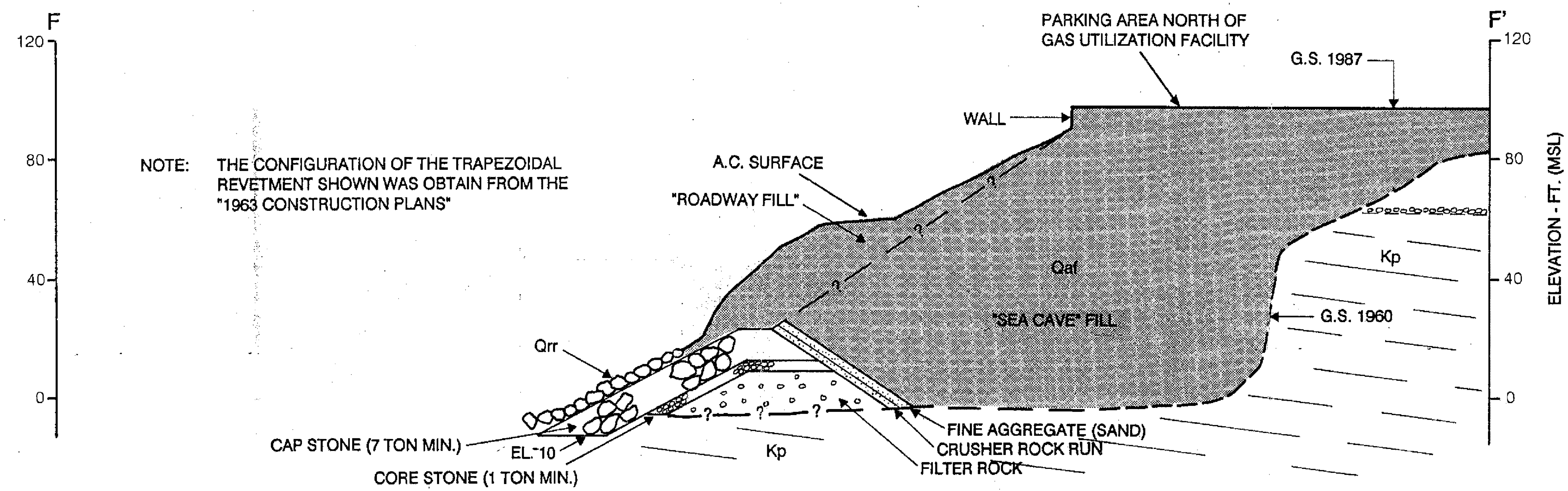


Project		Project No.	Figure
POINT LOMA TREATMENT PLANT	**GENERALIZED GEOLOGIC CROSS SECTIONS C - C' & D - D'**	1089-ES01	A-13

GROUP DELTA CONSULTANTS, INC.



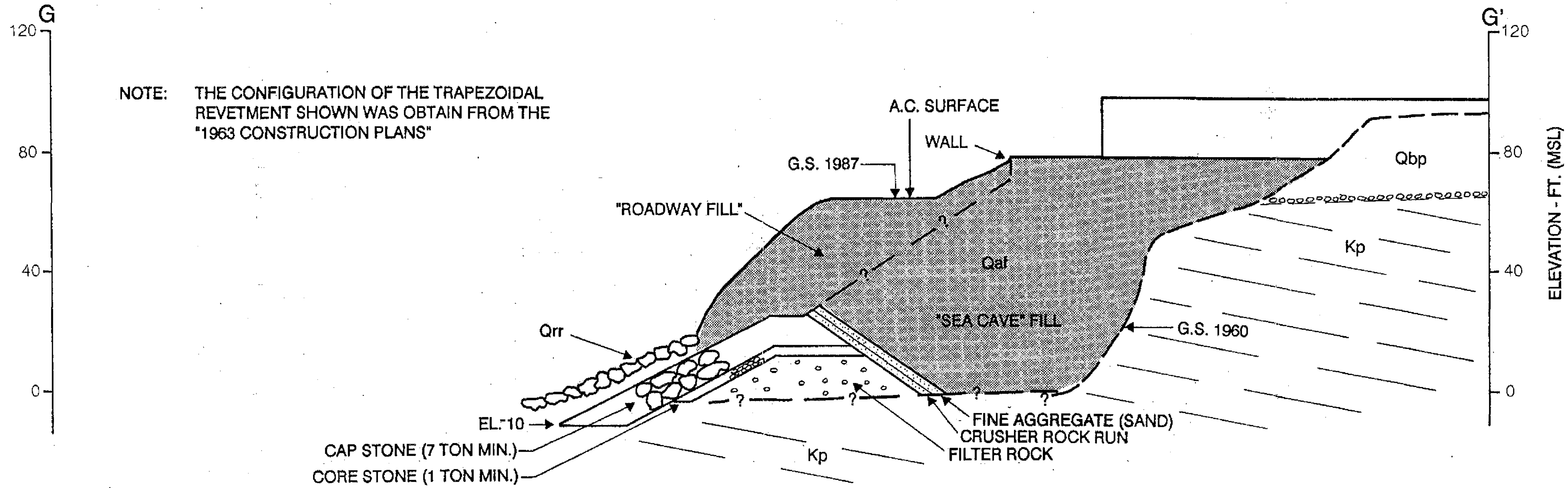
FOR LEGEND, SEE FIGURE A-16



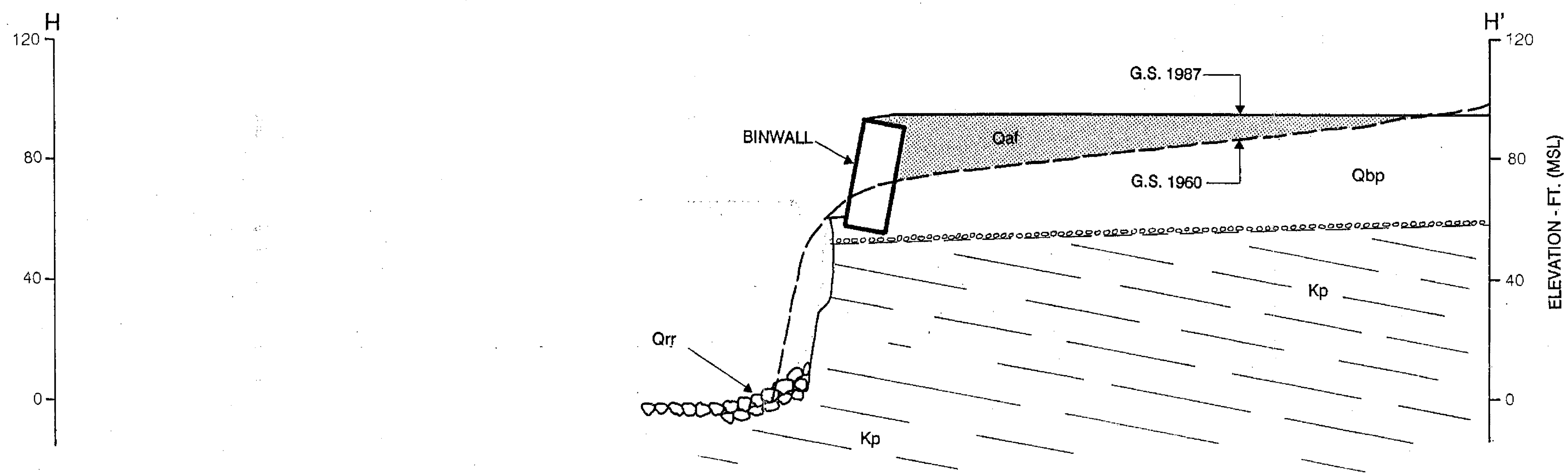
Project		Project No.	Figure
POINT LOMA TREATMENT PLANT	**GENERALIZED GEOLOGIC CROSS SECTIONS E - E' & F - F'**	1089-ES01	A-14

GROUP DELTA CONSULTANTS, INC.

NOTE: THE CONFIGURATION OF THE TRAPEZOIDAL REVETMENT SHOWN WAS OBTAIN FROM THE "1963 CONSTRUCTION PLANS"

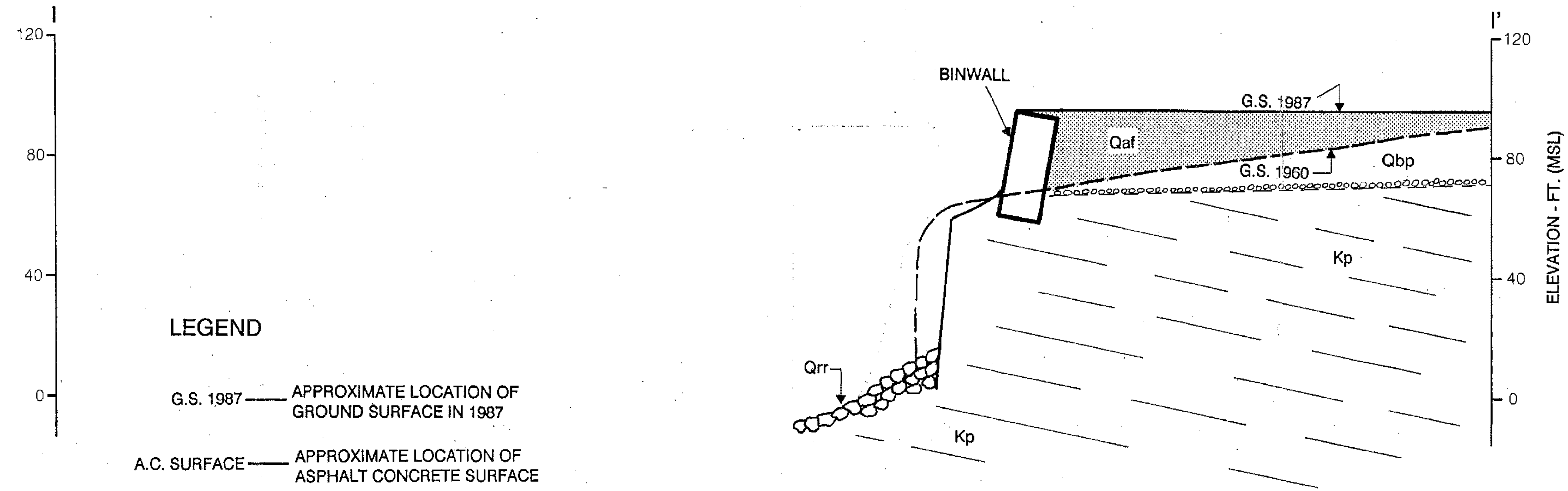


FOR LEGEND, SEE FIGURE A-16



Project		Project No.	Figure
POINT LOMA TREATMENT PLANT	**GENERALIZED GEOLOGIC CROSS SECTIONS G - G' & H - H'**	1089-ES01	A-15

GROUP DELTA CONSULTANTS, INC.



LEGEND

G.S. 1987 — APPROXIMATE LOCATION OF GROUND SURFACE IN 1987

A.C. SURFACE — APPROXIMATE LOCATION OF ASPHALT CONCRETE SURFACE

Qrr — RIPRAP REVETMENT

Qaf — ARTIFICIAL FILL

Qbp — BAY POINT FORMATION

Kp — POINT LOMA FORMATION

Project		Project No.	Figure
POINT LOMA TREATMENT PLANT	**GENERALIZED GEOLOGIC CROSS SECTIONS I - I' & J - J'**	1089-ES01	A-16

GROUP DELTA CONSULTANTS, INC.

APPENDIX B

WAVE DATA
POINT LOMA, CALIFORNIA

APPENDIX B

WAVE DATA
POINT LOMA, CALIFORNIA

TABLE OF CONTENTS

APPENDIX B

WAVE DATA
POINT LOMA, CALIFORNIA

1.0 WAVE GENERATION

Ocean waves off the coast of southern California fall into three main categories:

1. Northern hemisphere swell consisting of waves generated in the northern hemisphere, which arrive in the southern California waters after leaving the generating area;

2. Southern hemisphere swell consisting of similar waves generated south of the equator; and

3. Sea consisting of waves generated within the local area (Munk and Traylor, 1947).

1.1 Northern Hemisphere Swell

Winds which produce northern hemisphere swell are usually associated with one of the following meteorological situations (Marine Advisers, 1961):

1. Japanese-Aleutian storms which move from west to east in relatively high latitudes, often stagnating in the Gulf of Alaska. Occasionally, especially during winter and spring, this storm track shifts southward and the maximum wave heights occur at central or southern California latitudes. These extratropical cyclones are the most important source of severe waves reaching the California coast.

2. Hawaiian storms which move from west to east in mid-latitudes.

3. Tropical hurricanes which commonly develop off the west coast of Mexico. The resulting swell rarely exceeds 2 m, but a strong tropical storm will occasionally move far enough north to cause destructive high waves. The storm of September 1939, which passed directly over southern California causing very high waves locally, is an example.

1.2 Southern Hemisphere Swell

Munk, et al., (1963) point out three major source areas: The Ross Sea, the New Zealand-Australia-Antarctic sector, and the Indian Ocean. These southern ocean source areas are significantly blocked by island chains in the south Pacific Ocean. The South Pacific is of such a large area that waves from several southern storms commonly reach southern California simultaneously. Southern swell is most important during the southern winter (April through September).

1.3 Sea

Sea is the term applied to short, steep waves which are still in or near the area in which they are generated. Wind conditions which generate sea vary greatly as one moves offshore from the southern California coast, changing from relatively mild winds over the inner channels to strong, gusty winds outside the islands.

2.0 WAVE CLIMATE

There are two sources of wave data available for the study region: 1) in-situ wave measurements, and (2) hindcast predictions of the wave field, inferred from the wind field.

2.1 Measurements of In-Situ

2.1.1 Offshore Wave Data

There are relatively few long-term measurements of the deep ocean (unaffected by the Channel Islands and/or coastal bathymetry) wave field for the southern California region. The principal source of long-term in-situ measurements is the Coastal Data Information Program (Seymour and Sessions, 1976). Instrumented sites in the program include nondirectional buoys at Begg Rock and Point Arguello, both of which are almost fully exposed to deep ocean waves.

A directional NOAA buoy was deployed due south of San Nicholas Island and west of San Clemente Island in April 1984 for a period of 18 months. This site is open to virtually all important wave directions. However, the absolute accuracy of moored directional buoys is not well known, and studies suggest an accuracy of 10 degrees (Burdette and Howard, 1982). A 10 degree difference in deep water direction can sharply alter the expected coastal response (see refraction diagrams). Wave transformation through the Channel Islands is so sensitive to the details of the deep water directional spectrum that both very accurate instrumentation and high resolution estimator techniques are required if the deep ocean data is to be quantitatively related to specific coastal response.

2.1.2 Nearshore Wave Data

Waves originating outside the Channel Islands are highly modified upon reaching the coastline (Inman, et al., 1986). Northern swell is significantly sheltered by Point Conception and the Channel Islands. The Tanner and Cortez Banks also significantly alter deep-water waves as they approach the coastline. Figure B-1, taken from the Beach Erosion Control Report, A Cooperative Study of San Diego County, California,

Appendix IV, Phase 2, dated March 1, 1960, prepared by the Los Angeles District Corps of Engineers, illustrates the effect of island sheltering.

The nearshore bathymetry off the southern California coast is very complex. Refractive effects associated with shoals and banks in the island vicinity yield a rather different sheltering picture than obtained by simply assuming geometric shadows behind islands and banks. Visual observations in winter suggest that most of the energy approaches from directions 260 to 280 degrees which are geometrically shadowed by San Clemente Island and the Tanner and Cortez Banks.

In-situ wave measurements for Point Loma do not exist. The California Coastal Data Collection Program (Seymour and Sessions, 1976) has recorded wave height measurements at Imperial Beach (summarized in Figure B-2) and at the Mission Bay entrance channel (Table B-1, and Figures B-3 and B-4). This program was initiated in 1978 and has continued to the present with minor interruptions due to instrumentation failure. During this period, the winter waves of January through March 1983 were by far the most energetic (Seymour et al., 1984).

2.2 Wave Hindcasts

The most extensive deep-water ocean data is not directly measured, but is inferred from the wind field, which is usually inferred from maps of barometric pressure. It is clear that the quality of these wave hindcasts is limited in accuracy by the quality of the initial barometric pressure fields, and the subsequent models for the wind field and wave generation-propagation.

Marine Advisers (1961) has generated hindcast estimates for a station approximately 65 nautical miles southwest of San Clemente Island. Due to the relatively sparse nature of the available weather information, a considerable degree of subjectivity was involved in deducing large-scale wind fields. Assumptions used in this study lead to considerable computational simplification, but is also very nonphysical. The Marine Advisers (1961) data should be acknowledged as qualitative in nature, and is included here only for completeness and perspective. Their data is compactly expressed in wave roses (Figure B-5). The radiating bars represent direction classifications, and the concentric circles which intersect them form a frequency scale which is in percentage of the average total number of hours in a year (8766 hrs/yr). For example, the longest bar in the upper wave rose represents all northern hemisphere swell approaching from 300 to 310 degrees. The inner segment, out to the numeral 1, gives the frequency of waves from that direction in the 0.1 to 0.9 foot height group. It measures approximately 6.9 percent, which indicates that waves of this classification can be expected 0.069 x 8766 = 605 hours per year. Note that maximum south swell heights arriving from the southern hemisphere are only about 25 percent as large as north swell heights from the northern hemisphere. The Marine Advisers (1961) hindcasts are based on singular wave models rather than spectral wave models.

Seymour et al. (1984) has generated storm wave hindcast estimates for the period 1900 - 1984 using a single methodology which is spectral throughout. The hindcast location is near 35°N, north of Point Conception and the Channel Islands. Only waves with deep-water approach directions between SW and WNW were considered due to the fact that waves approaching more obliquely would be diminished considerably by refraction as they approached the shoreline. Further, the waves were ranked by their power (energy multiplied by period). This resulted in a list of 59 storms in which the resulting offshore significant wave height exceeded 3 m, all having periods equal to or exceeding 12 seconds. The tropical cyclone of September 1939, a major wave event in southern California, was added for a total of 60 storms. These are listed in Table B-2.

A second series was obtained by considering only the very largest events. The threshold significant wave height was raised to 6 m (20 feet). The second series contains only 18 storms because of its higher limit value, as shown in Table B-3. As indicated in

Seymour's paper "It should be clearly recognized that the possible quality of hindcast decreases with the age of the data, particularly prior to the 1950's. It is likely that some major storms in the early years were excluded because there was insufficient pressure field resolution and accuracy to estimate the real wind speeds. This is particularly true for small, intense storms like tropical cyclones. It is almost impossible to hindcast these storms prior to the availability of satellite imagery. However, since no series of this length had previously been published, and since the work used a consistent methodology throughout, we felt that they would make a valuable contribution to our knowledge of the wave climate off California."

It is interesting to note that during the 84-year hindcast that there were seven storms during the winter of 1982-83, with significant wave heights above 4.9 m. Note also the very long periods found in these storms. All seven storms had maximum periods of 14 to 25 seconds. This suggests that the 1982-83 waves were among the most energetic in this century. Using historical records, Seymour et al. (1984) demonstrated a strong correlation between moderate and strong El Niño events and large wave events in California. Furthermore, they suggest that pronounced warming of the surface waters along the California coast during a strong El Niño condition allows tropical cyclones to penetrate further northward than in non-El Niño years. These tropical cyclones are of such small spatial extent that they are not described in detail sufficient for hindcasts on pre-satellite weather maps.

3.0 WAVE REFRACTION ANALYSIS AT POINT LOMA

3.1 Technique

The sheltering of Point Loma by the Channel Islands and the Cortez and Tanner Banks prevents deep-water wave hindcasts from being used directly as input for a wave refraction model. To get a more realistic view of LOCAL deep-water wave characteristics, each storm event was presented by a directional wave spectrum which was transformed to a local deep-water spectrum as described by Le Mehaute and Wang (1982). Figures B-6, B-7 and B-8 show sheltered significant wave height as a function of direction. Local deep-water refers to the first deep water encountered offshore from a particular site. It is generally inside the islands and, therefore, potentially sheltered from deep ocean waves. Figure B-9 shows the bathymetry used for this refraction (obtained from NOAA data).

Each storm was modeled as a directional spectrum at a given frequency and with an energy density which fell off like cosine squared from the peak direction. The spectra were assigned a width of 15 degrees at the point at which the energy density was half the peak value, or 15 degrees, full width, half maximum (FWHM). The true shapes of deep ocean wave spectra are not well known due to the limited resolving power of modern day measuring techniques. Hence, the choice of a spectrum's shape and width is somewhat arbitrary at best. Fortunately, the spectral transformation techniques are not overly sensitive to these factors in most cases. Attachment A shows the spectral transformations used to get local directions for each event.

The deep ocean wave spectra were transformed to local deep-water wave spectra in 100 m of water offshore from Point Loma. The local deep-water significant wave heights and peak energy directions were then derived from the local spectra. From the 18 extreme events exceeding 6 m significant wave height (Table B-3), 10 events were chosen to have the largest local deep-water wave heights for a particular period and local direction. The remaining 8 events had similar periods and directions, but lower wave heights than one of the 10 selected for refraction. An additional three events were analyzed where severe storm damage was reported in southern California. These storms occurred on February 6, 1969, January 16, 1978 and January 27, 1983. The transformed local deep-water significant wave heights and periods for these 13 events are presented in Table B-4.

Wave rays were run at a 200 m spacing from local deep water. They were stopped when the wave height calculated along a ray exceeded 1.28 times the depth (breaking wave ratio). Note that some breaking wave heights were rather large and associated with areas of strong ray convergence. If these rays were permitted to continue, they would have formed caustics. The shortcomings of neglecting diffraction are thus evident and the breaker heights should probably be viewed as overestimates in these areas. Refraction results are presented in Attachment B for each of the 13 wave events.

APPENDIX B
WAVE DATA
POINT LOMA, CALIFORNIA

REFERENCES

Burdette, E.L., and W.W. Howard, 1982, "Intercomparison of directional wave spectra from an NDBO discus-hulled buoy and a wavestaff array," Proceedings, Oceans '82 Conf., p. 1282-1287.

Inman, D.L., R.T. Guza, D.W. Skelly, and T.E. White, 1986, "Southern California coastal processes data summary," Ref. No. CCSTWS 86-1, Feb 1986, U.S. Army Corps of Engineers, 572 pp.

Le Mehaute, B., and J.D. Wang, 1982, "Wave spectrum changes on a sloped beach," Jour. Water, Port, Coastal and Ocean Div., Amer. Soc. Civil Eng., v. 108, p. 33-47.

Marine Advisers, 1961, "A statistical survey of ocean wave characteristics in southern California waters," prepared for the U.S. Army Corps of Engineers, Los Angeles District, Marine Advisers, Inc., La Jolla, CA, 30 pp.

Munk, W.H., and M.A. Traylor, 1947, "Refraction of ocean waves, a process linking underwater topography to beach erosion," Jour. Geol., v. 55, n. 1, p. 1-26.

Munk, W.H., G.H. Miller, F.E. Snodgrass, and N.F. Barber, 1963, "Directional recording of swell from distant storms," Phil. Trans. Roy Soc. London, A, v. 55, n. 1062, p. 505-584.

Pawka, S.S., and R.T. Guza, 1983, "Coast of California waves study - site selection," SIO Ref. 83-12, 51 pp., Scripps Inst. of Oceanogr., La Jolla, Calif.

Seymour, R.J., and M.H. Sessions, 1976, "A regional network for coastal engineering data," Proc. 15th Coastal Eng. Conf., Amer. Soc. Civil Eng., p. 60-71.

Seymour, R.J., R.R. Strange, III, D.R. Cayan, and R.A. Nathan, 1984, "Influence of El Niños on California's wave climate," Proc. 19th Coastal Eng. Conf., Amer. Soc. Civil Eng., p. 577-592.

U.S. Corps of Engineers, Los Angeles District, March 1, 1960, "Beach erosion control report, A cooperative study of San Diego County, California," Appendix D, Plate 8.

TABLE B-1

Significant height return periods based on Mission Bay entrance channel data (Seymour, 1982).

SIGNIFICANT HEIGHTS FOR VARIOUS RETURN PERIODS
SEA-DOMINATED OBSERVATIONS

SIG. HT. (CM)	0.99 CONFIDENCE LIMITS UPPER LIM.	LOWER LIM.	RETURN PERIOD (YR)
322.7	381.3	303.1	1
374.4	455.3	344.1	5
395.8	486.7	360.9	10
416.8	517.7	377.2	20
428.9	535.7	386.6	30
443.9	558.3	398.2	50
455.6	575.8	407.1	75
463.9	588.4	413.4	100

****

SIGNIFICANT HEIGHTS FOR VARIOUS RETURN PERIODS
SWELL-DOMINATED OBSERVATIONS

SIG. HT. (CM)	0.99 CONFIDENCE LIMITS UPPER LIM.	LOWER LIM.	RETURN PERIOD (YR)
344.0	393.4	328.8	1
390.8	454.7	367.4	5
409.9	480.1	383.0	10
428.5	504.9	398.0	20
439.1	519.3	406.6	30
452.3	537.0	417.2	50
462.5	550.7	425.4	75
469.7	560.6	431.2	100

TABLE B-2

Hindcast (1900-84) Waves Exceeding 3 m Height Near 35° N
(Seymour et. al., 1984)

EXTREME WAVE EPISODES EXCEEDING 3 M. (BASIC SERIES)
1900 - 1984

DATE	SIG. HT. (m)	MAX. PERIOD	DIRECTION
13 MAR 05	8.8	15	247
17 NOV 05	3.3	17	286
31 DEC 07	5.3	16	282
12 MAR 12	3.2	12	220
26 JAN 14	5.8	13	223
03 FEB 15	7.5	14	235
01 JAN 18	3.7	16	280
12 FEB 19	5.3	12	299
20 DEC 20	4.7	13	301
15 OCT 23	3.7	16	296
01 FEB 26	6.9	15	257
03 JAN 27	5.8	20	287
06 NOV 28	4.0	17	294
01 JAN 31	3.9	16	276
28 DEC 31	7.4	18	288
19 DEC 35	4.7	16	267
13 DEC 37	4.5	16	272
06 JAN 39	7.9	19	285
25 SEP 39	4.5	15	205
24 JAN 40	4.3	16	267
25 DEC 40	5.7	16	270
20 OCT 41	3.3	17	294
30 DEC 45	3.9	19	285
13 FEB 47	3.9	16	265
04 NOV 48	4.7	18	300
15 NOV 53	5.7	17	269
15 JAN 58	3.1	22	280
26 JAN 58	6.8	14	259
05 APR 58	7.7	18	289
16 FEB 59	5.1	14	244
09 FEB 60	8.1	19	295
22 DEC 60	3.4	17	276
31 JAN 63	4.2	16	260
10 FEB 63	5.9	15	256
19 NOV 65	4.0	15	277
07 DEC 67	4.0	15	298
06 FEB 69	4.7	13	222
04 DEC 69	3.6	17	278
06 DEC 69	4.9	22	274
14 DEC 69	5.7	17	290
19 DEC 69	4.7	18	281
26 DEC 72	4.1	15	289
21 FEB 77	5.2	18	280
29 OCT 77	5.5	20	299
16 JAN 78	6.0	13	240
01 JAN 80	4.7	20	272
17 FEB 80	6.1	18	249
22 JAN 81	4.3	20	258
28 JAN 81	7.0	17	262
13 NOV 81	4.9	18	284
01 DEC 82	6.4	14	295
18 DEC 82	6.4	20	288
25 JAN 83	6.1	17	278
27 JAN 83	7.3	22	279
10 FEB 83	6.7	25	281
13 FEB 83	4.9	17	268
01 MAR 83	8.2	20	258
14 NOV 83	5.0	17	290
03 DEC 83	7.0	17	285
25 FEB 84	6.4	17	300

TABLE B-3

Hindcast (1900-84) Waves Exceeding 6 m Height Near 35° N
(Seymour et. al., 1984)

EXTREME WAVE EPISODES EXCEEDING 6 m
1900 - 1984

DATE	SIG. HT. (m)	MAX. PERIOD	DIRECTION
13 MAR 05	8.8	15	247
03 FEB 15	7.5	14	235
01 FEB 26	6.9	15	257
28 DEC 31	7.4	18	288
06 JAN 39	7.9	19	285
26 JAN 58	6.8	14	259
05 APR 58	7.7	18	289
09 FEB 60	8.1	19	295
17 FEB 80	6.1	18	249
28 JAN 81	7.0	17	262
01 DEC 82	6.4	14	295
18 DEC 82	6.4	20	288
25 JAN 83	6.1	17	278
27 JAN 83	7.3	22	279
10 FEB 83	6.7	25	281
01 MAR 83	8.2	20	258
03 DEC 83	7.0	17	285
25 FEB 84	6.4	17	300

TABLE B-4

Transformed Local Deep-Water Wave
Significant Wave Height and Period

13 sec waves	Wave Height/m	
210	4.7	05 Feb 1969
240	6.0	16 Jan 1978
14 sec waves		
240	8.3	13 Mar 1905
245	5.6	01 Feb 1926
265	5.6	26 Jan 1958
290	2.9	01 Dec 1982
17 sec waves		
245	5.8	17 Feb 1980
265	4.2	28 Jan 1981
290	4.0	03 Dec 1983
20 sec waves		
255	4.5	01 Mar 1983
265	2.7	10 Feb 1983
285	2.6	18 Dec 1982

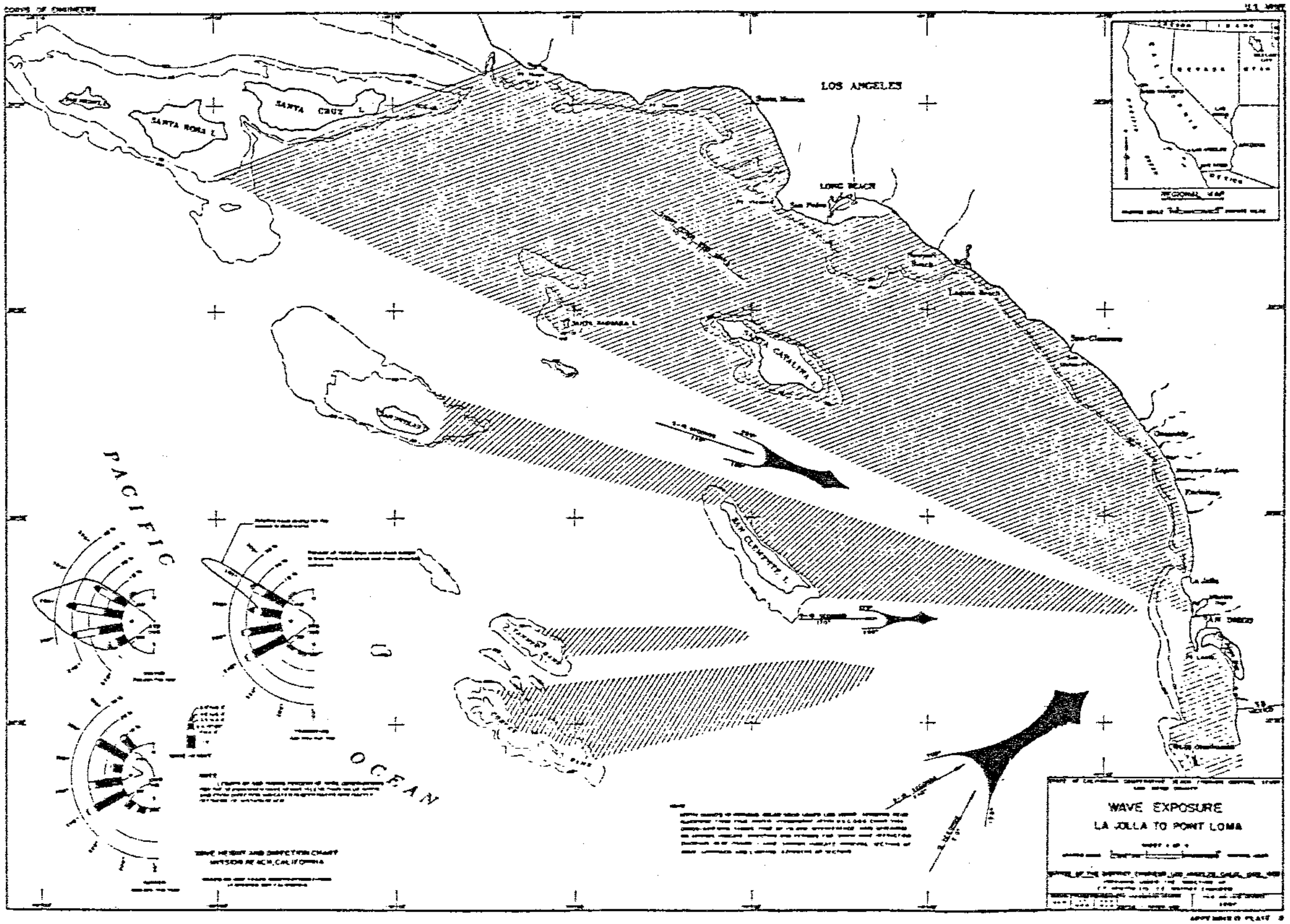
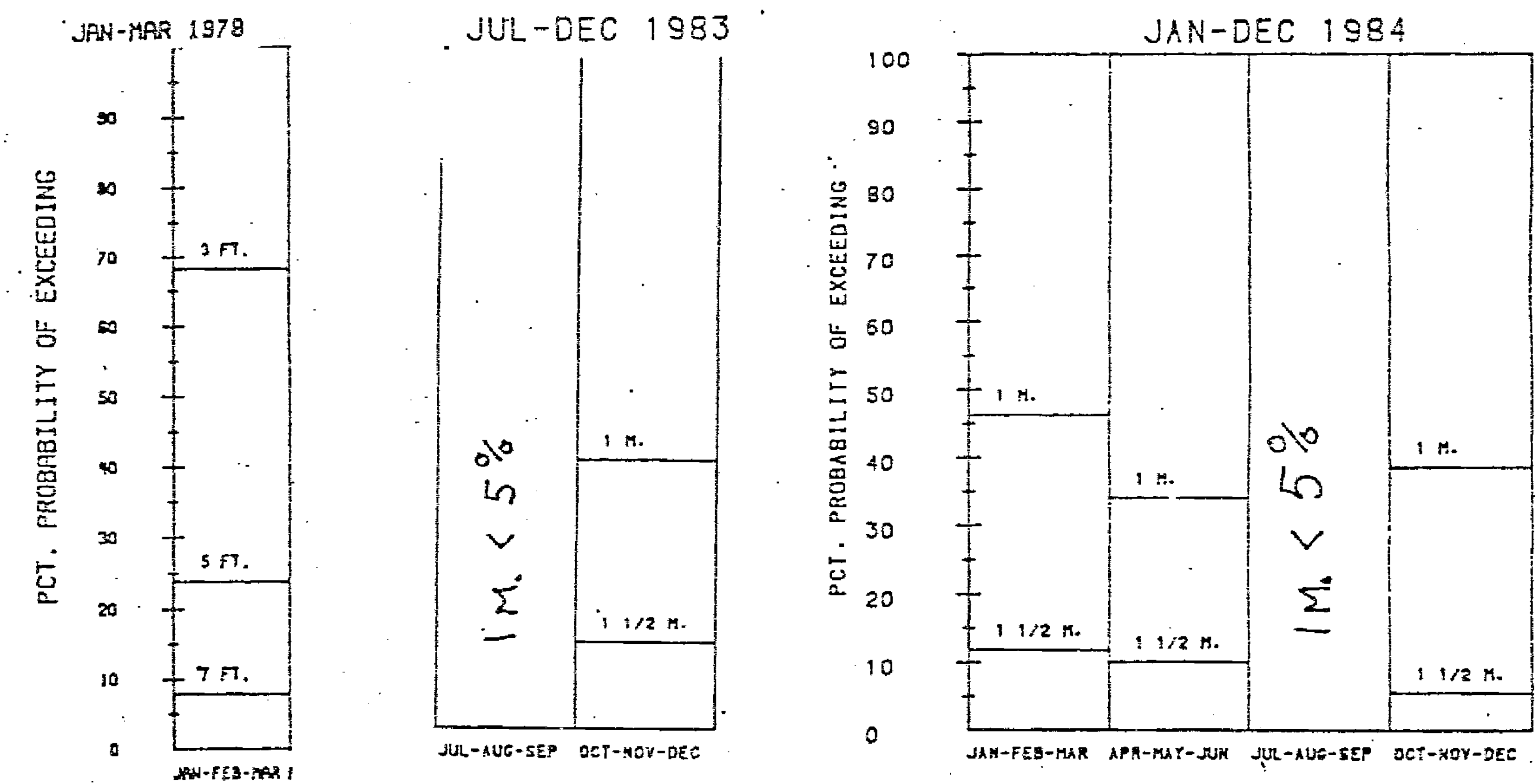

LOS ANGELES
LONG BEACH
SANTA CRUZ I.
SANTA ROSA I.
SANTA CATALINA I.
SAN CLEMENTE I.
SAN DIEGO
PACIFIC
OCEAN
WAVE EXPOSURE
LA JOLLA TO POINT LOMA
REGIONAL MAP

Figure B-1

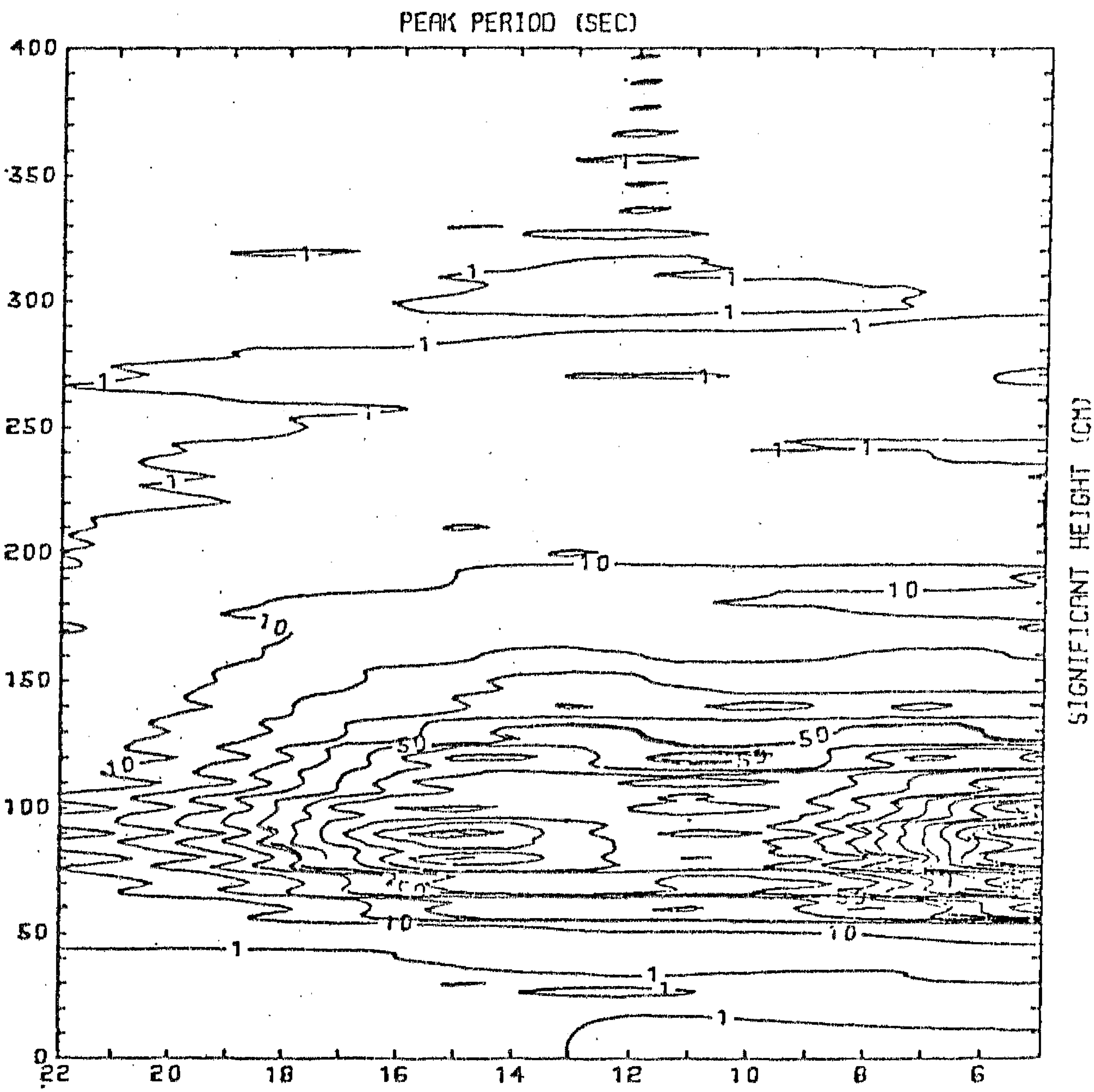
IMPERIAL BEACH



CDIP seasonal probabilities of exceeding various wave heights at Imperial Beach.

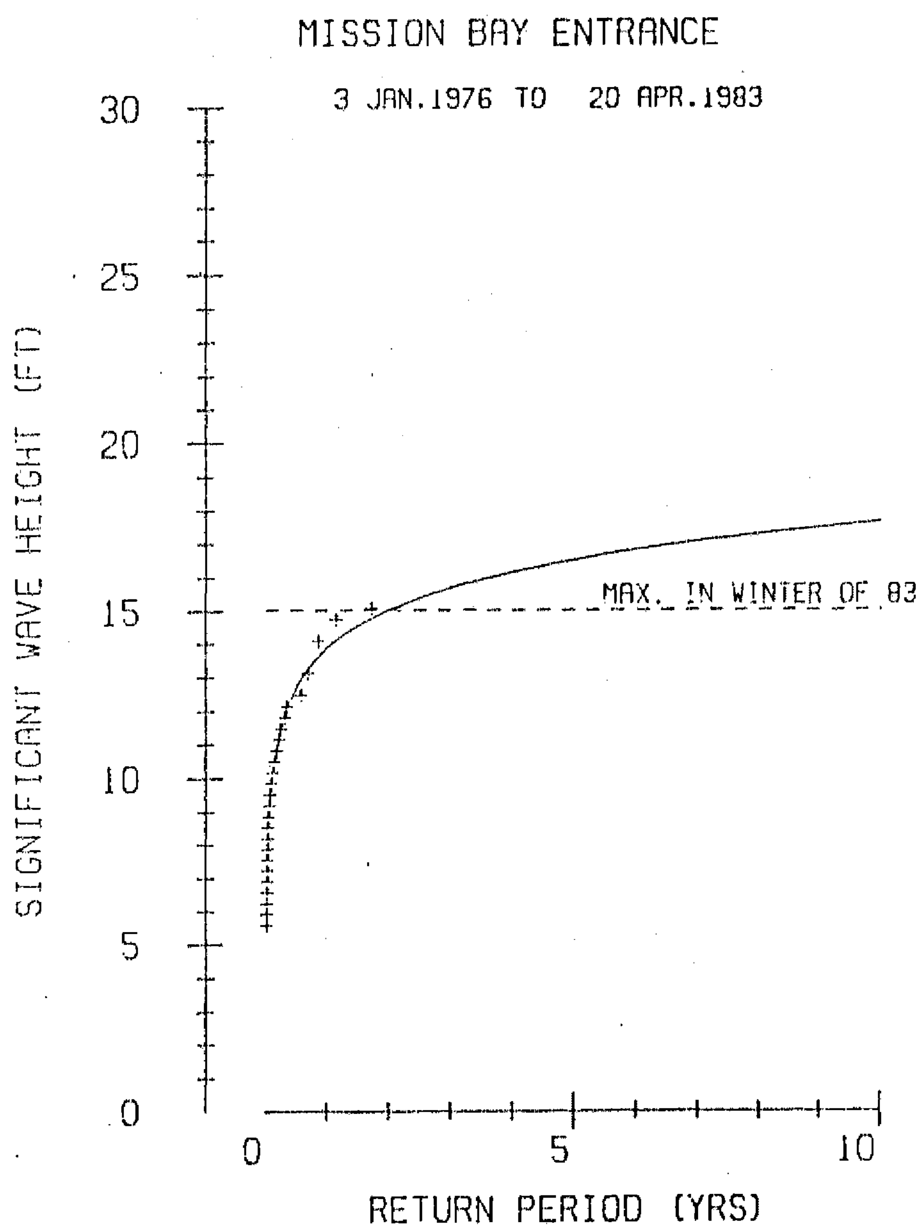
Figure B-2

MISSION BAY ENTRANCE
JOINT PERIOD AND HEIGHT
DISTRIBUTION CONTOURS



Contours of joint period and height distributions at Mission Bay entrance channel, 5591 samples (Seymour, 1982).

Figure B-3



Significant height return period at Mission Bay entrance channel (Seymour, 1983).

Figure B-4

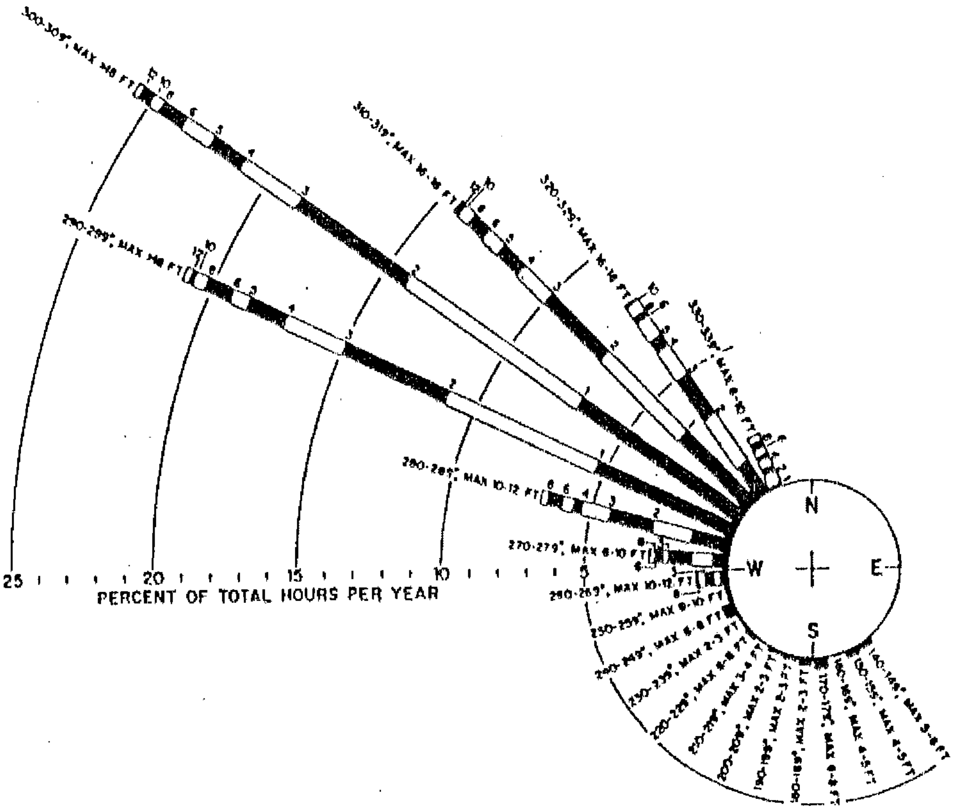
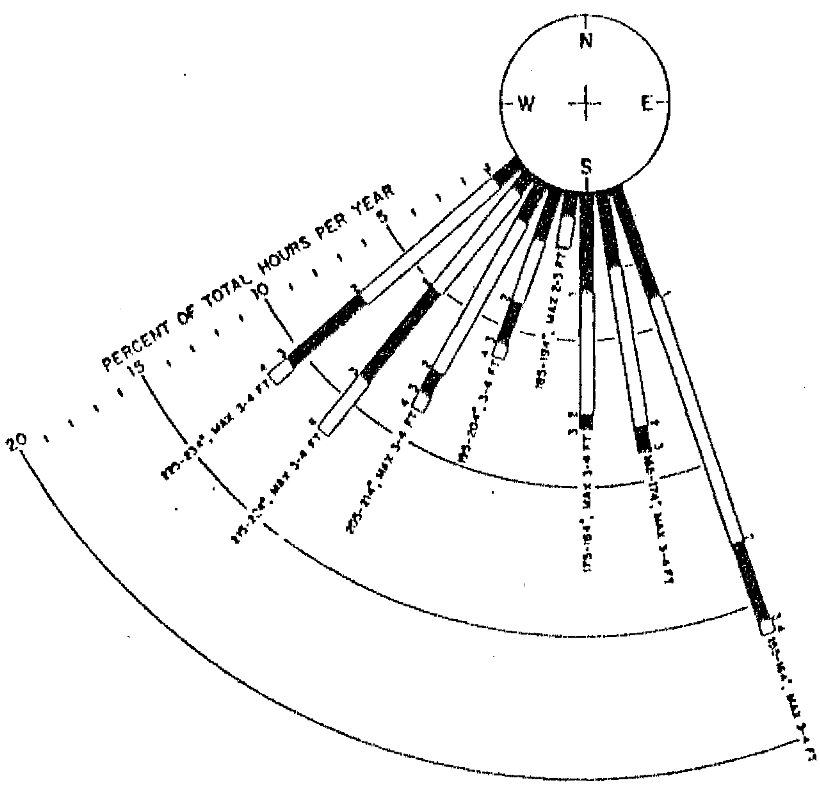
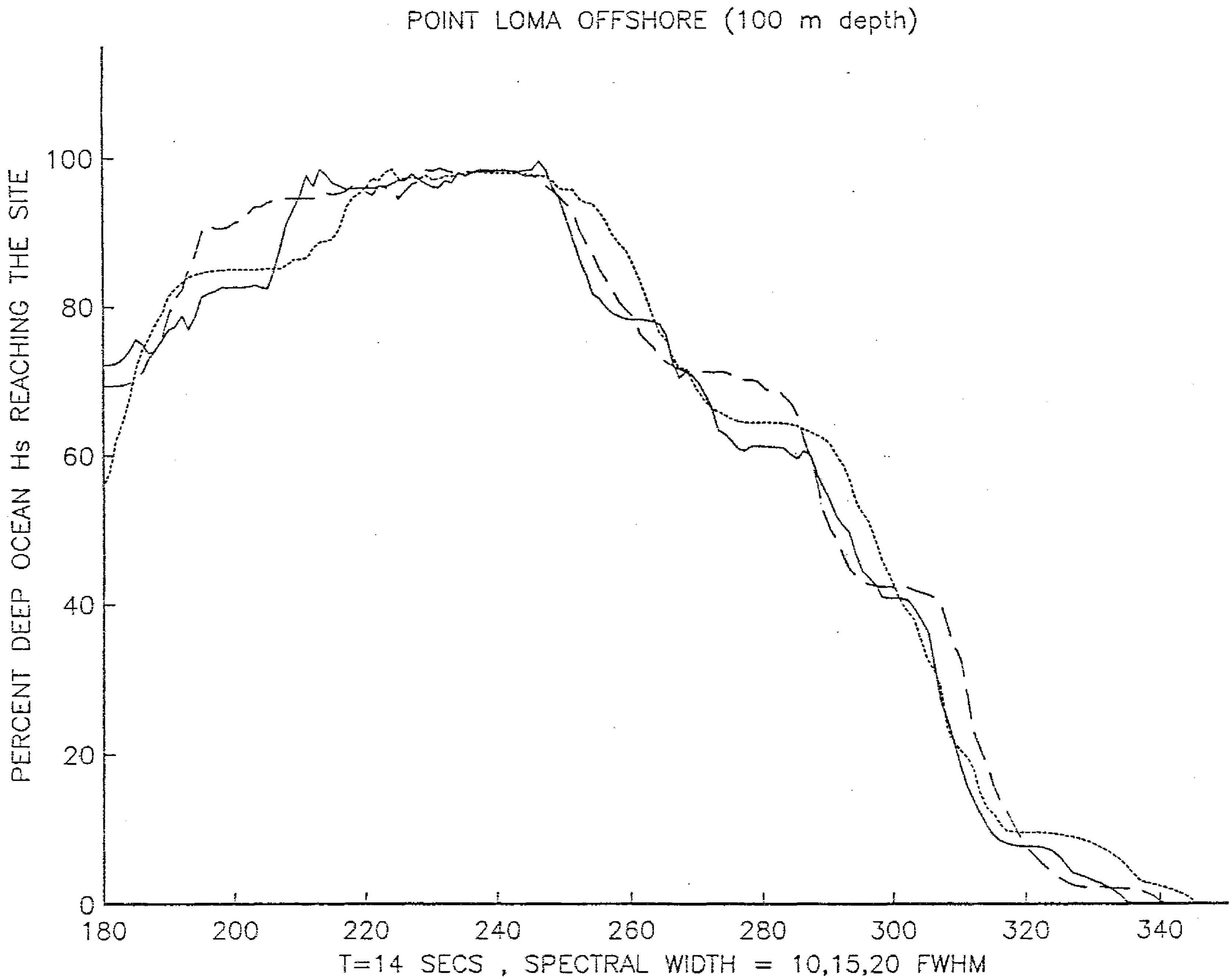


Figure 3.2.2-7 Wave rose, Station A, annual average (1956-58) northern hemisphere swell (Marine Advisers, 1961).



Wave rose, Station A, annual average (1948-50) southern hemisphere swell (Marine Advisers, 1961).

Figure B-5



POINT LOMA OFFSHORE (100 m depth)
PERCENT DEEP OCEAN Hs REACHING THE SITE
100
80
60
40
20
0
180
200
220
240
260
280
300
320
340
T=14 SECS , SPECTRAL WIDTH = 10,15,20 FWHM

Figure B-6

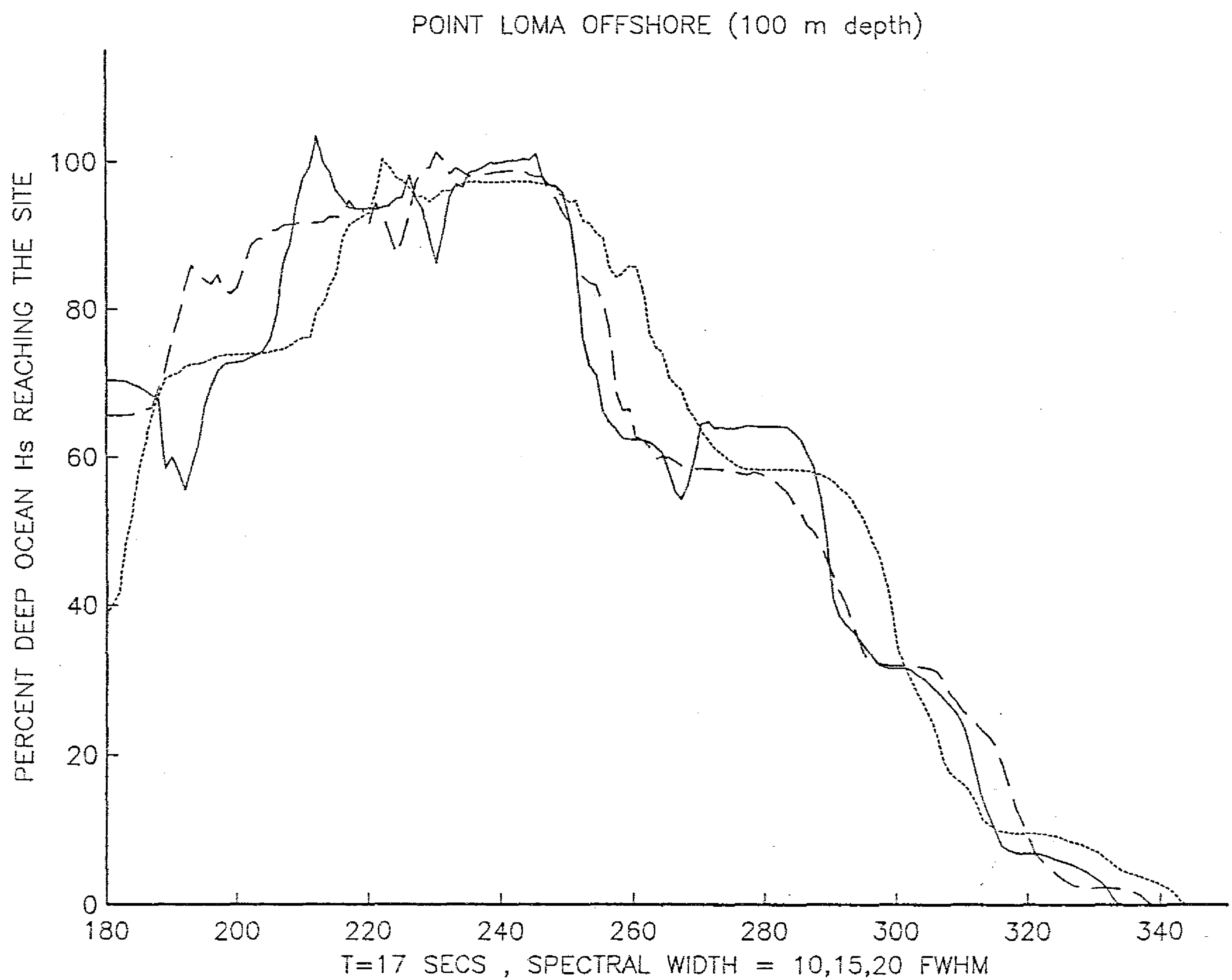

POINT LOMA OFFSHORE (100 m depth)
PERCENT DEEP OCEAN Hs REACHING THE SITE
0
20
40
60
80
100
180
200
220
240
260
280
300
320
340
T=17 SECS , SPECTRAL WIDTH = 10,15,20 FWHM

Figure B-7

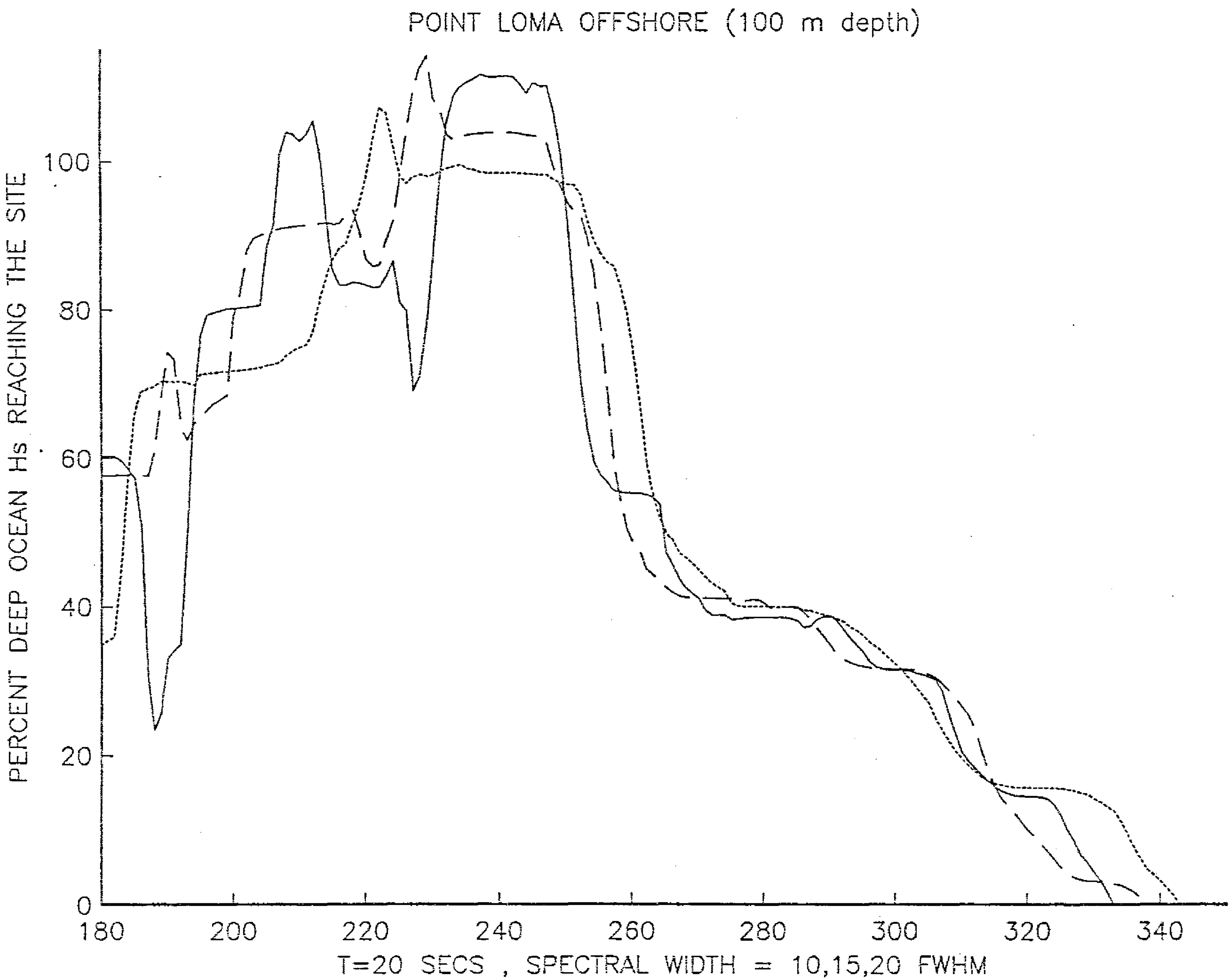
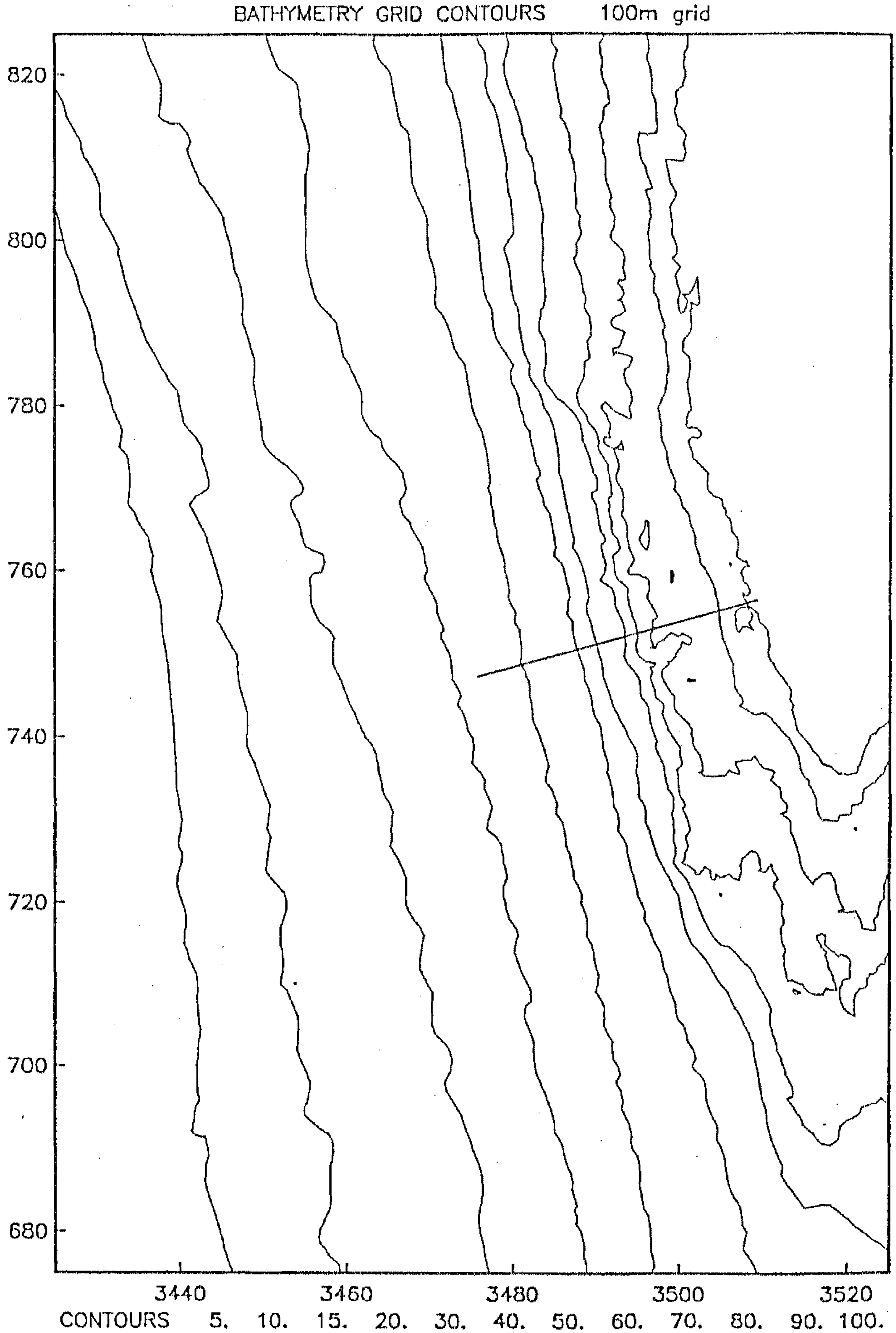

POINT LOMA OFFSHORE (100 m depth)
PERCENT DEEP OCEAN Hs REACHING THE SITE
0
20
40
60
80
100
180
200
220
240
260
280
300
320
340
T=20 SECS , SPECTRAL WIDTH = 10,15,20 FWHM

Figure B-8

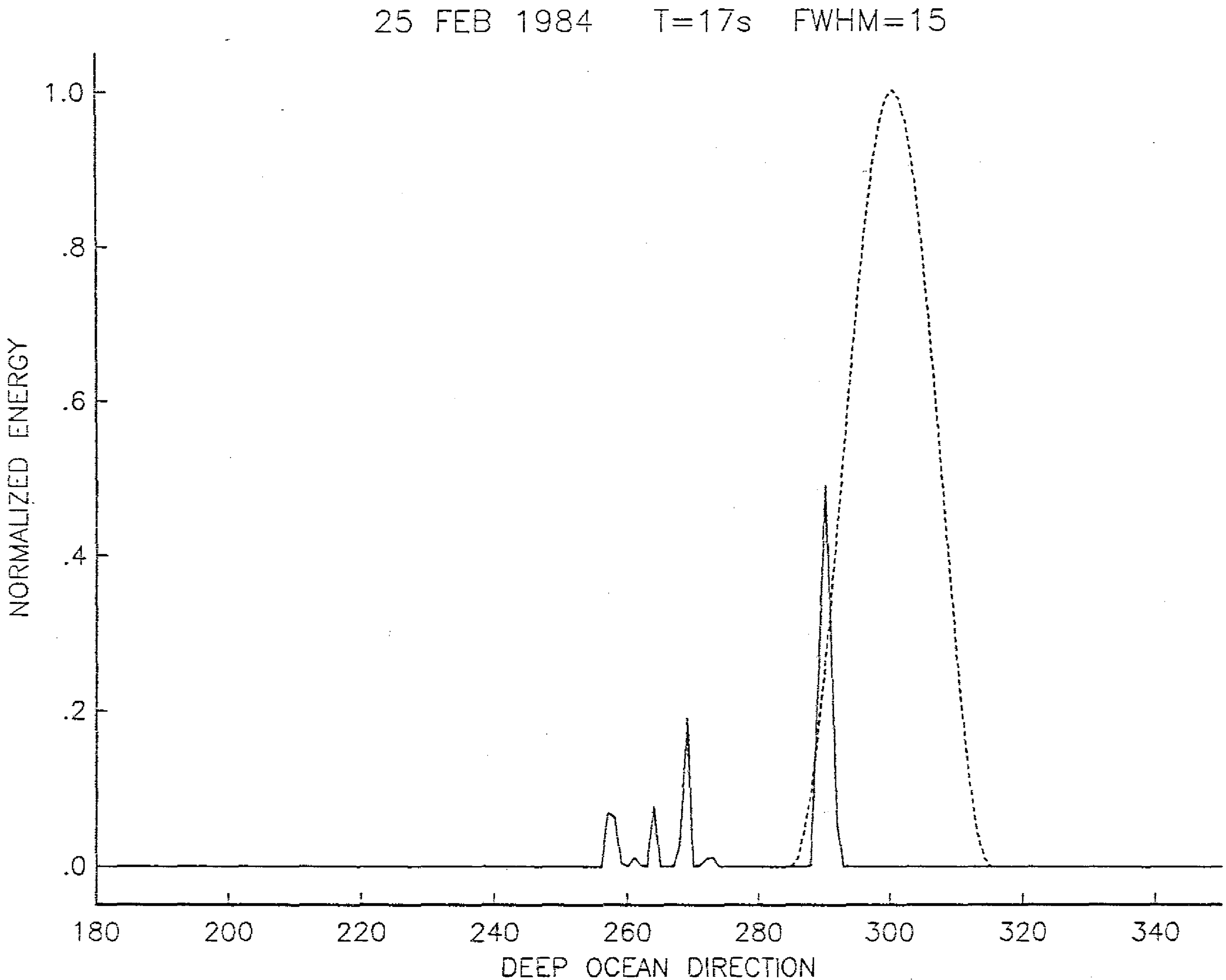


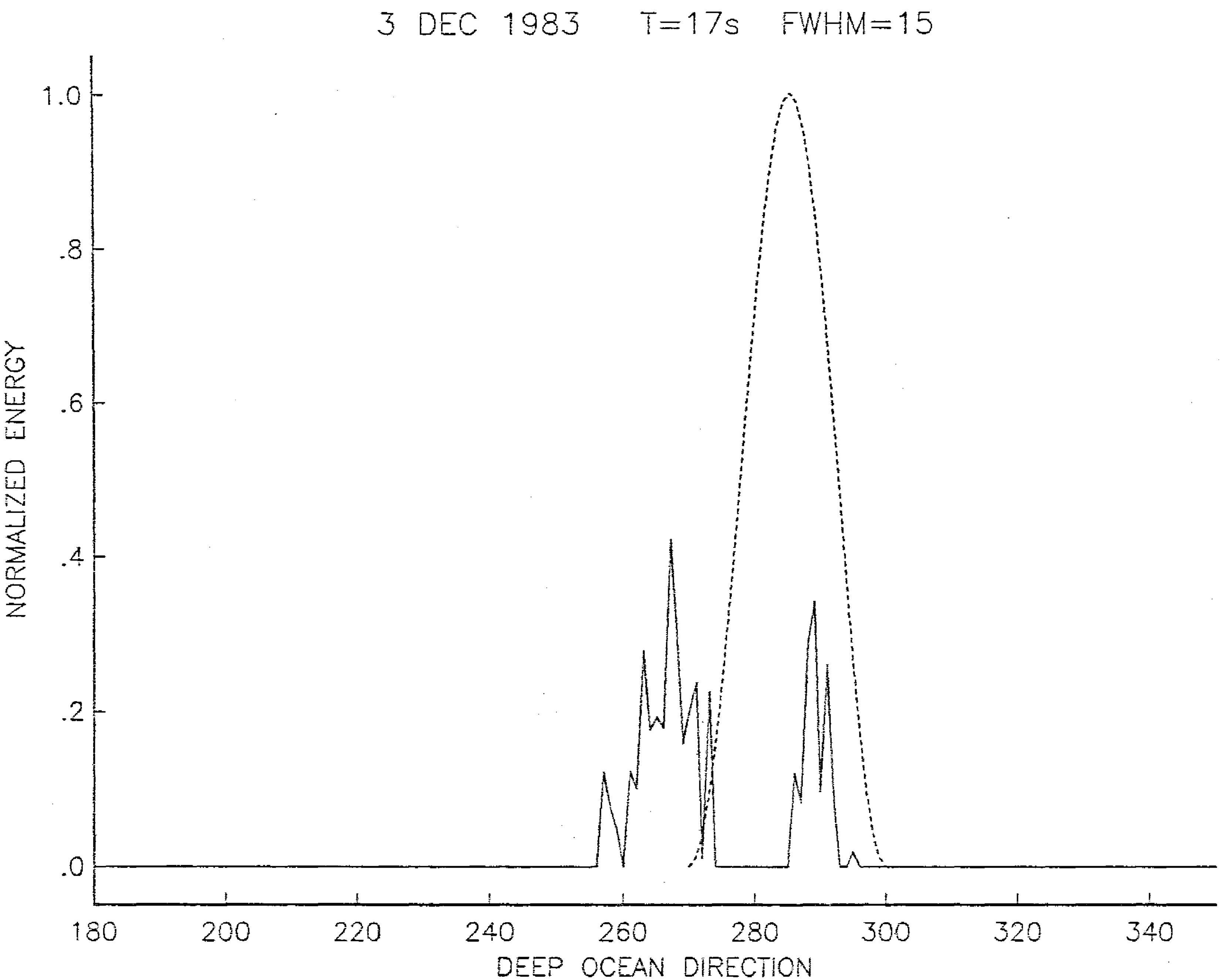
Bathymetry of the Point Loma area. Solid line shows sewage outfall orientation.

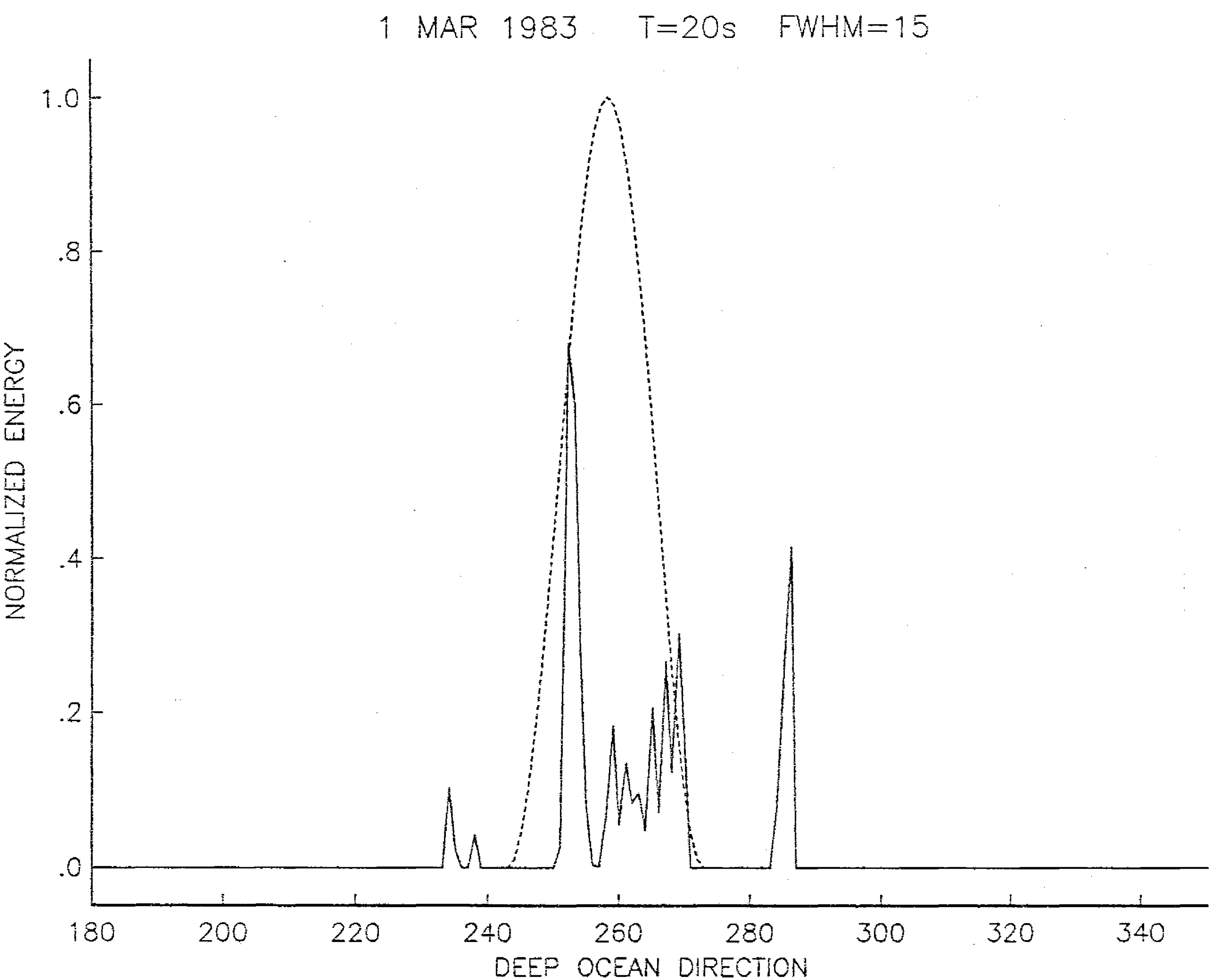
Figure B-9

Attachment A

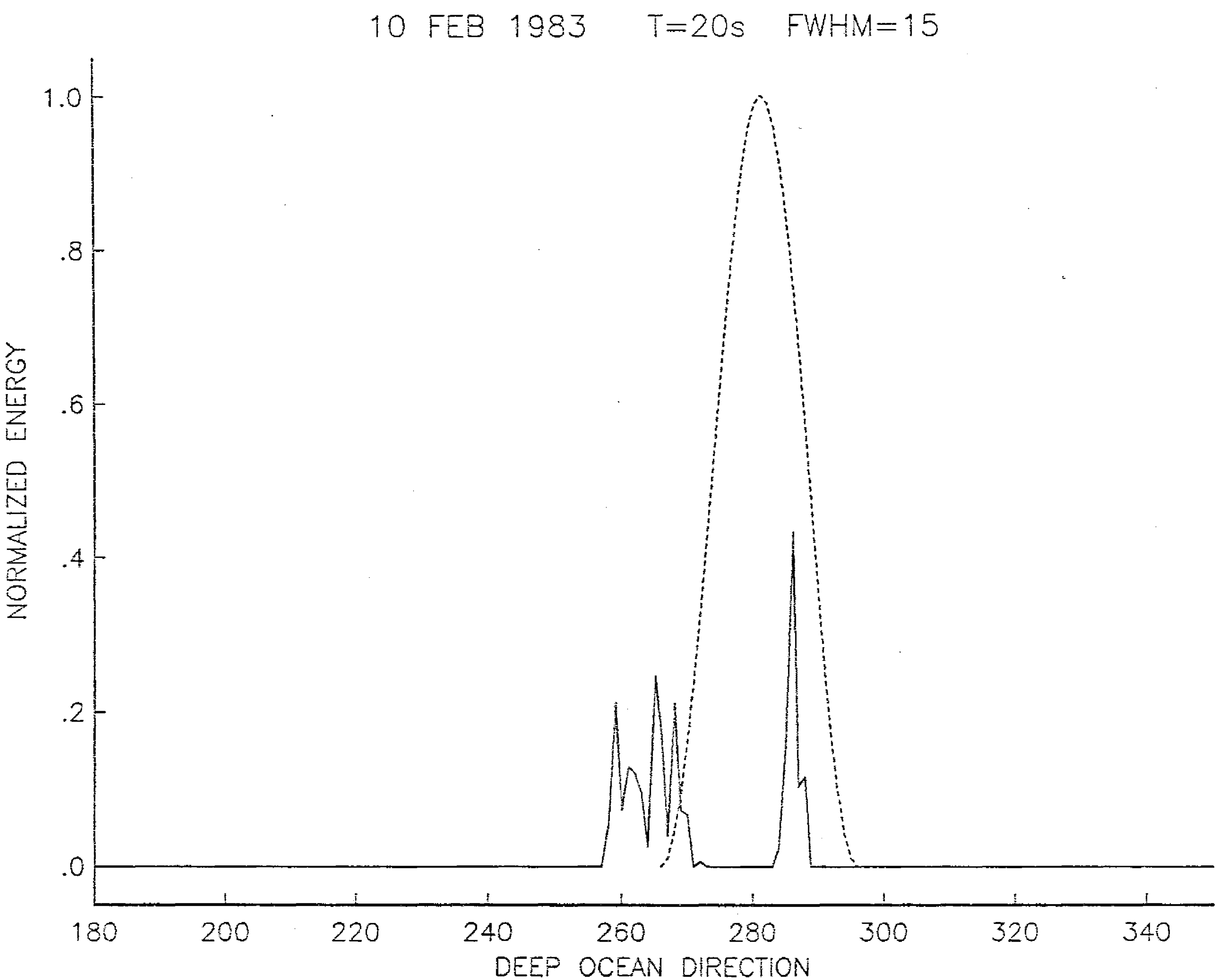
Spectral Transformations Used to Get Local Directions

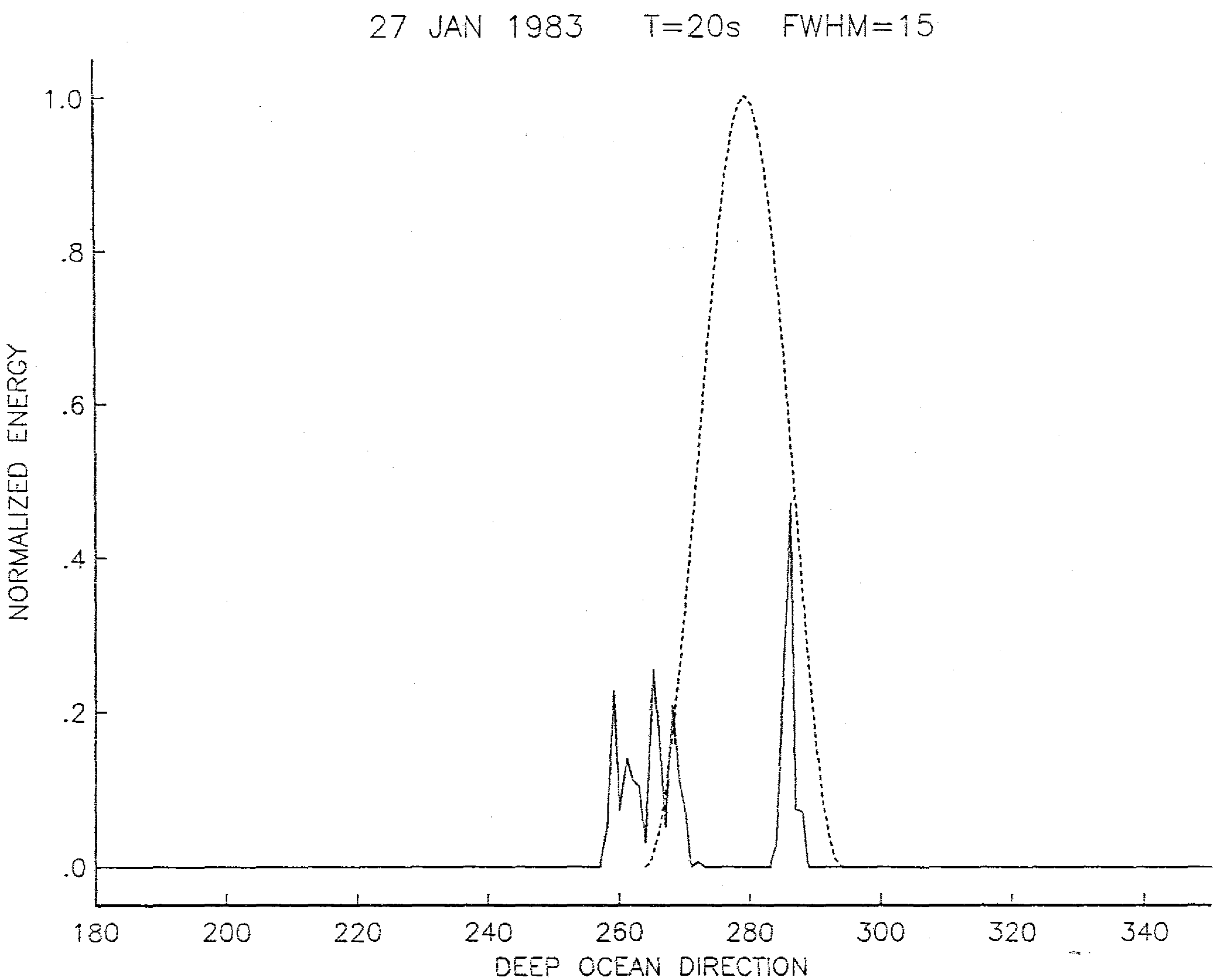

25 FEB 1984 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

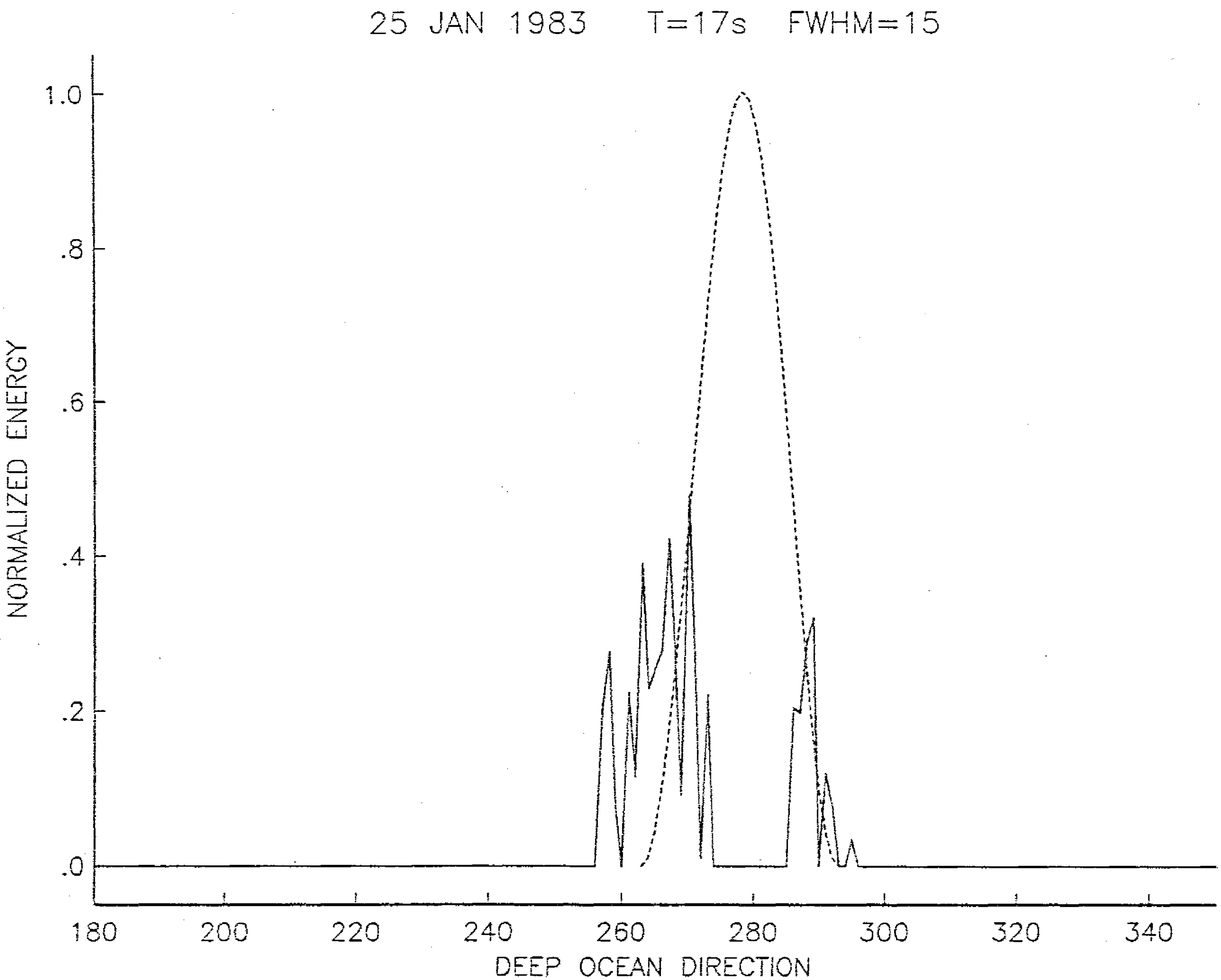

3 DEC 1983 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

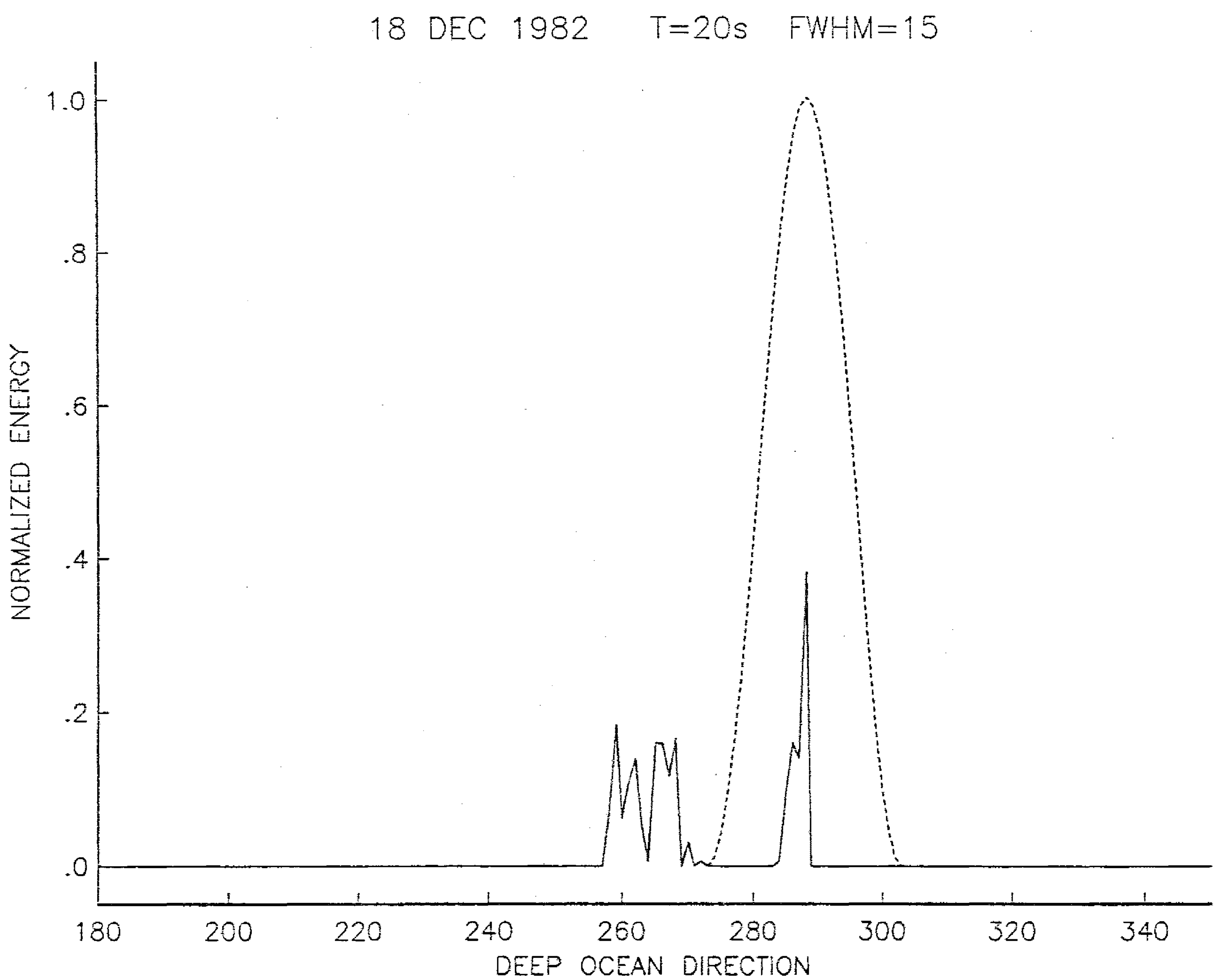


1 MAR 1983 T=20s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

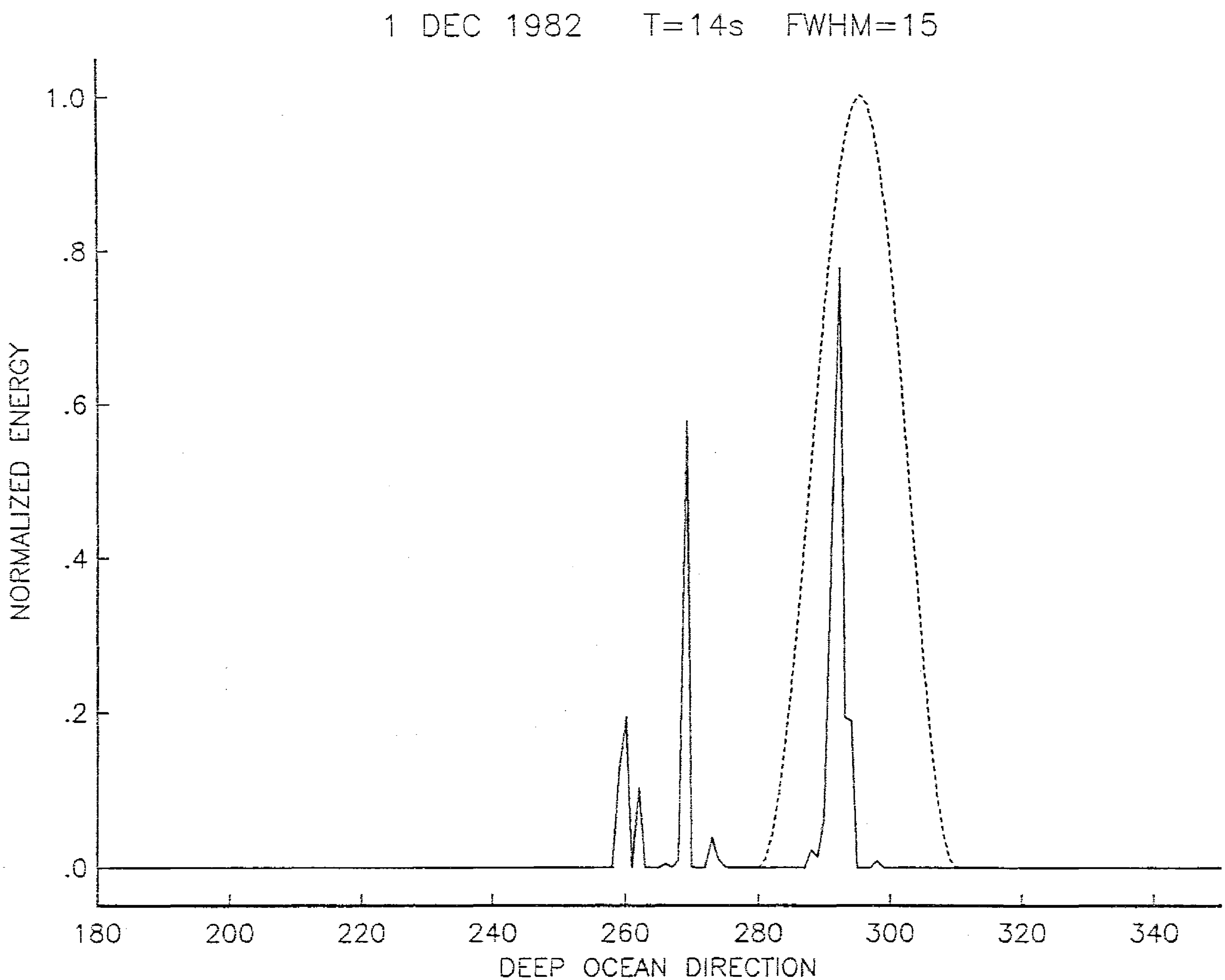

10 FEB 1983 T=20s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

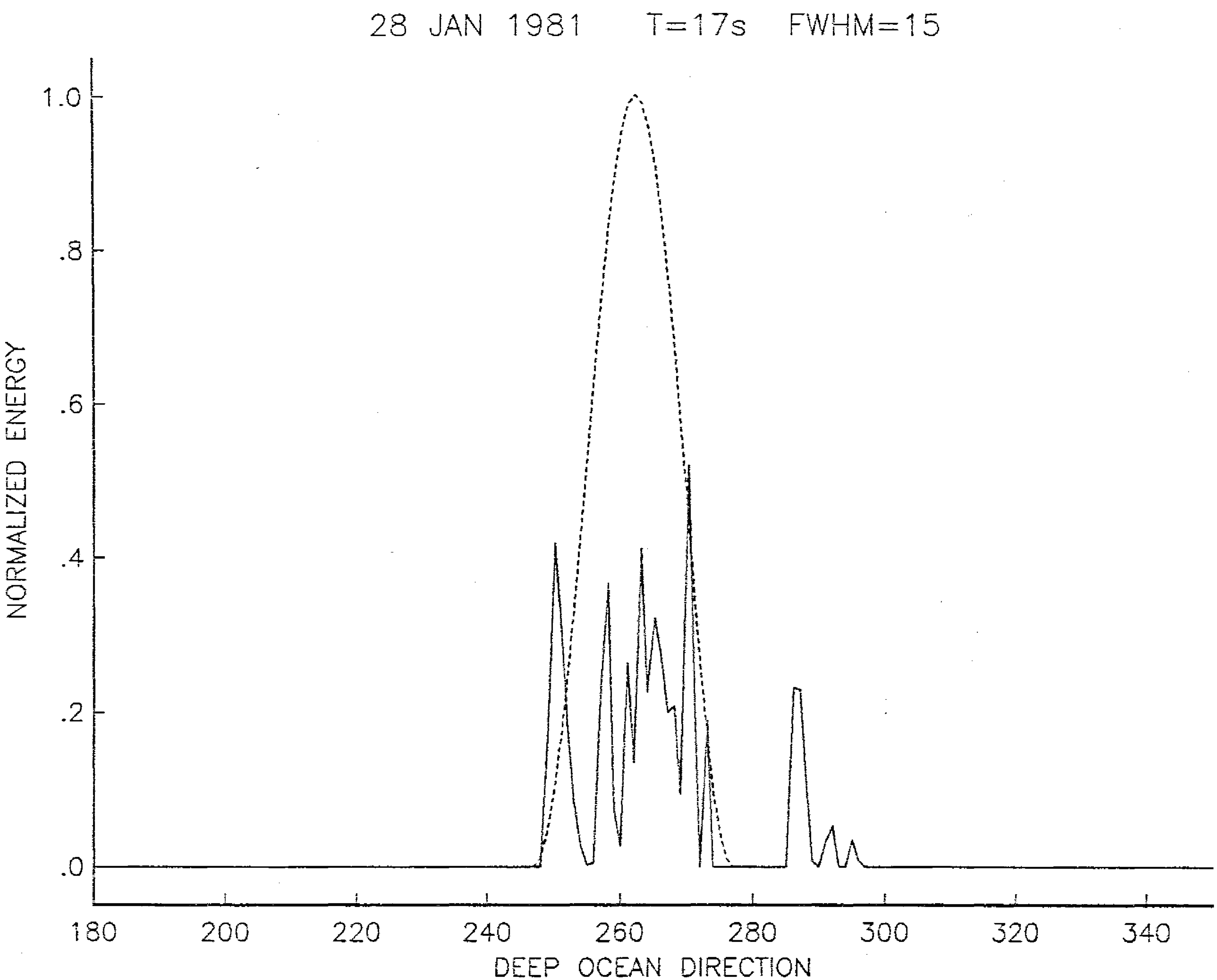

27 JAN 1983 T=20s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION


25 JAN 1983 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

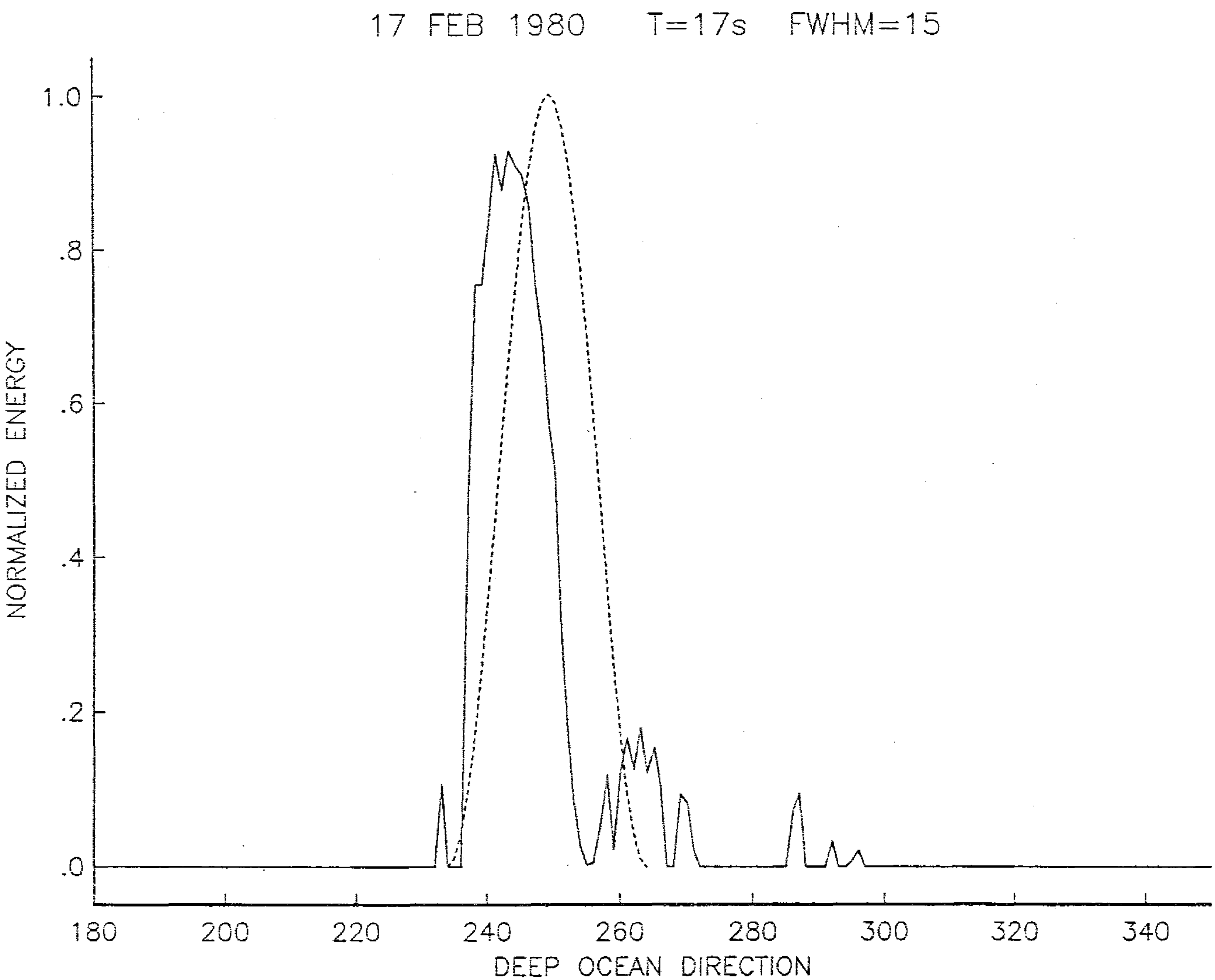


18 DEC 1982 T=20s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

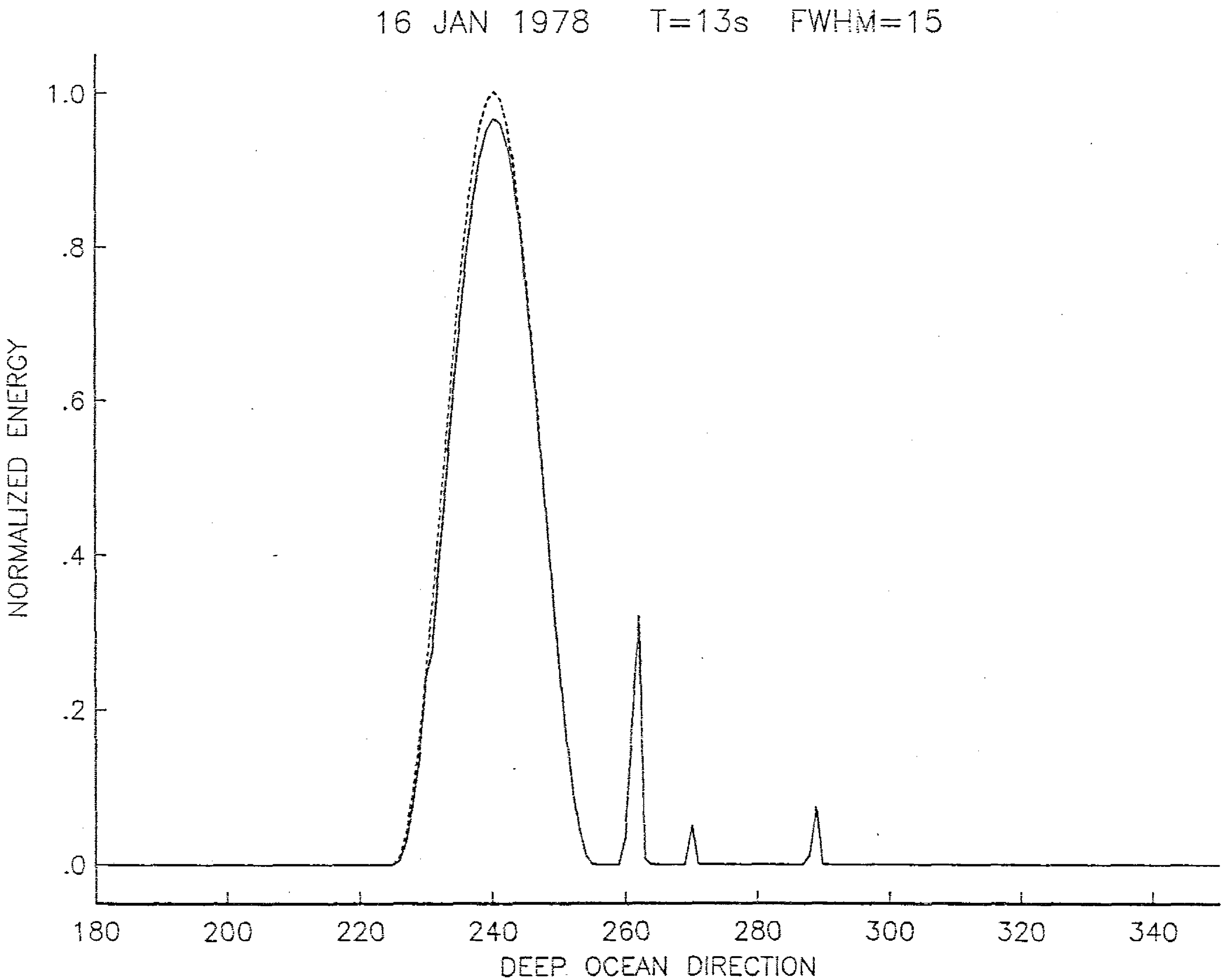

1 DEC 1982 T=14s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

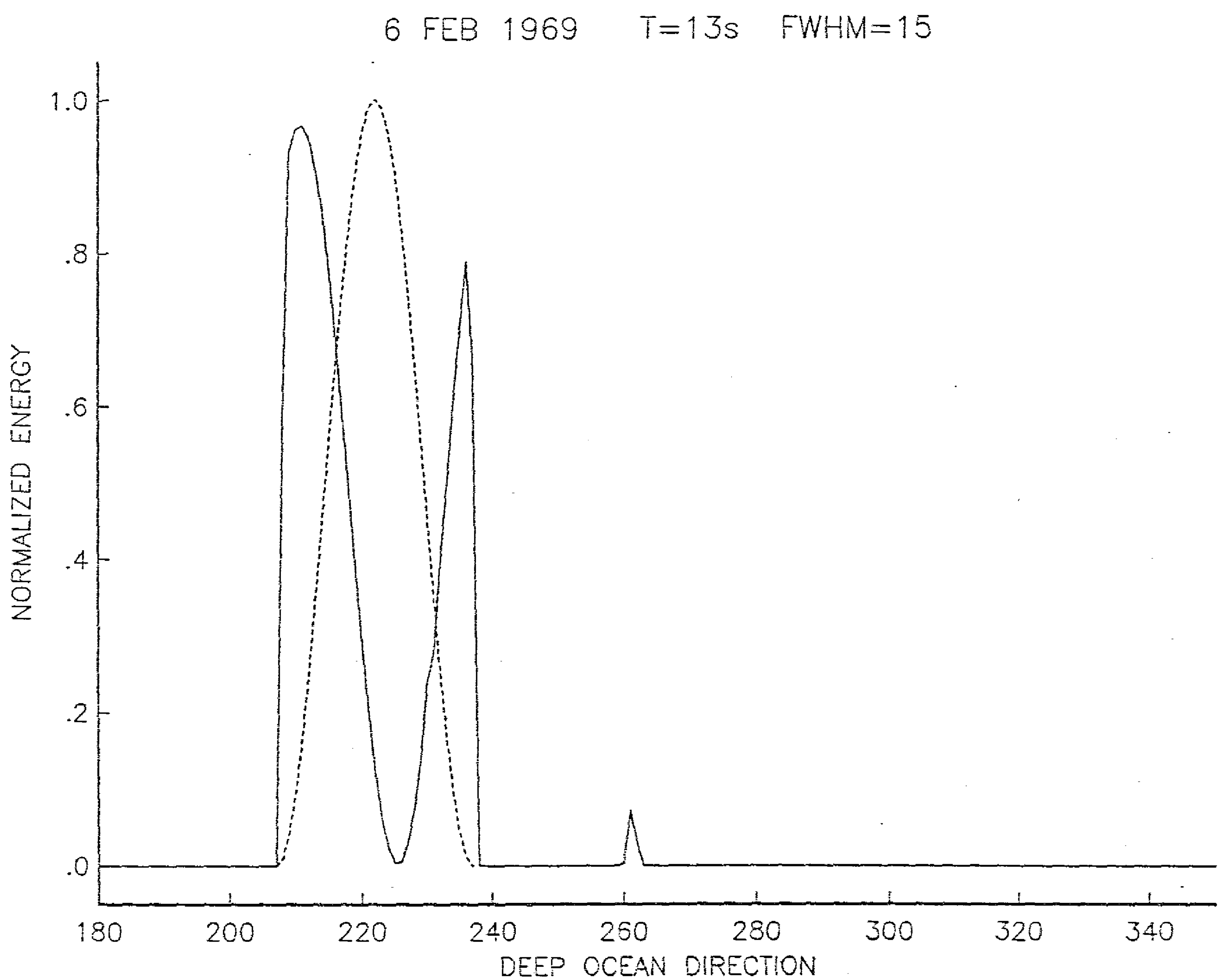


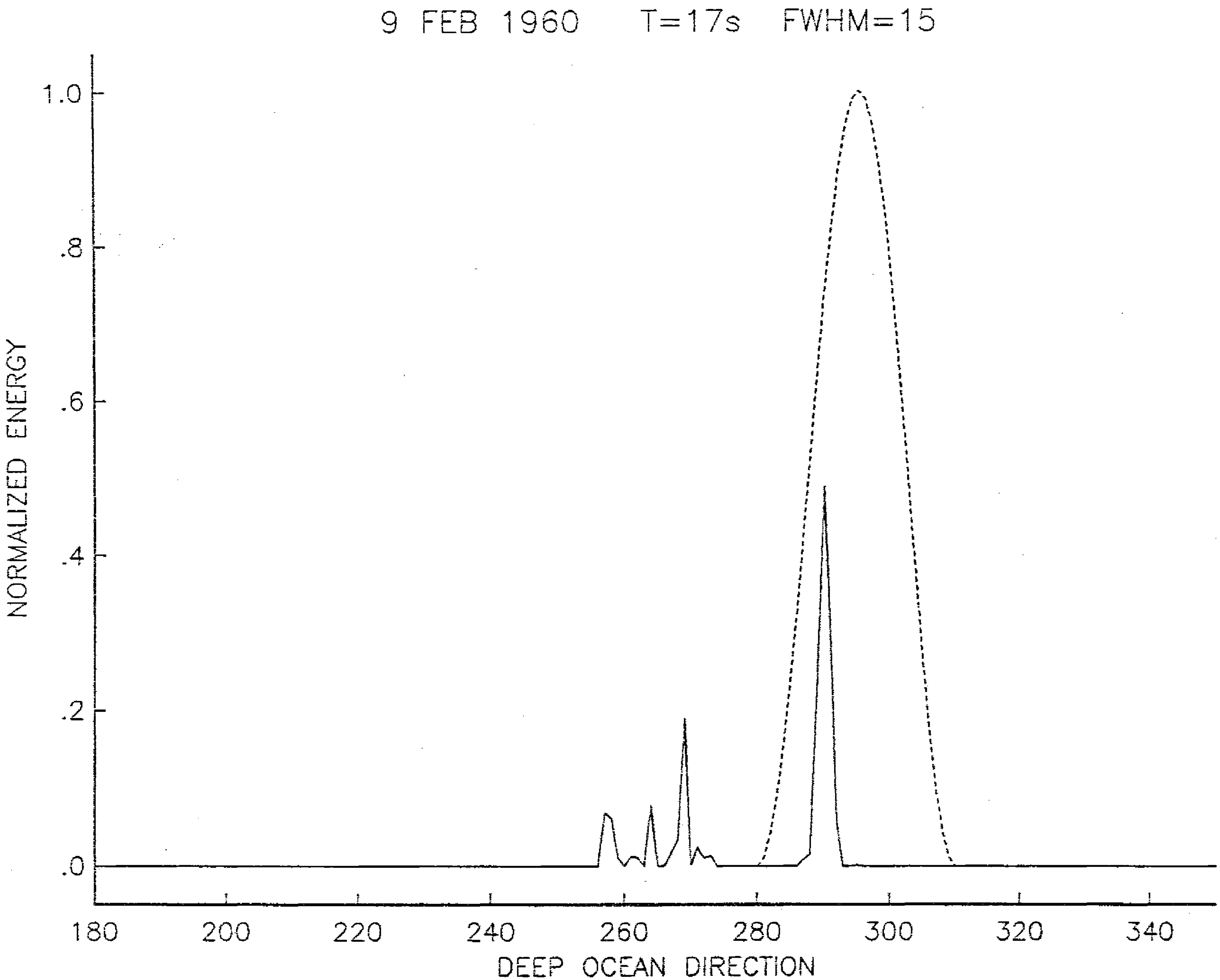
28 JAN 1981 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION



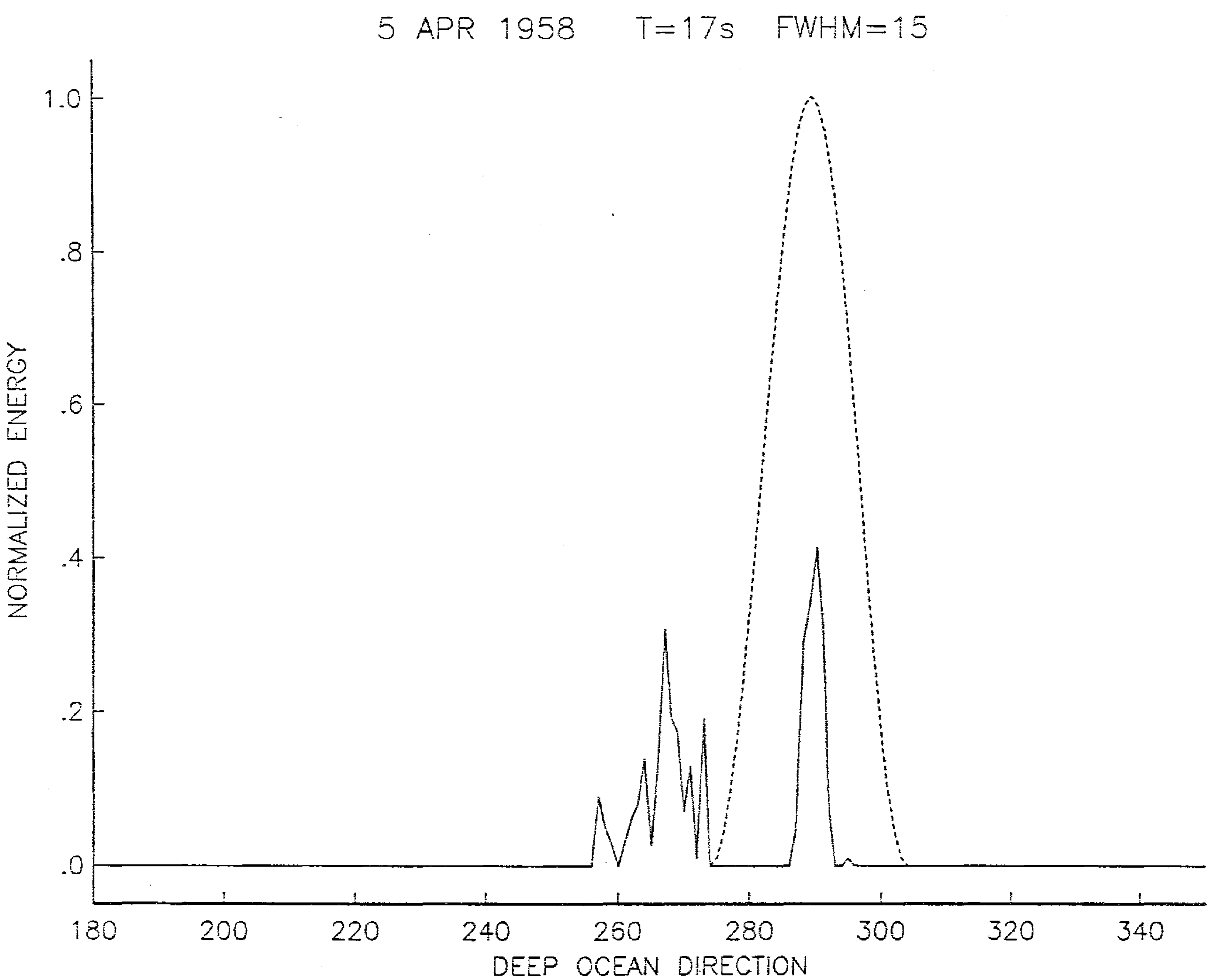
17 FEB 1980 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

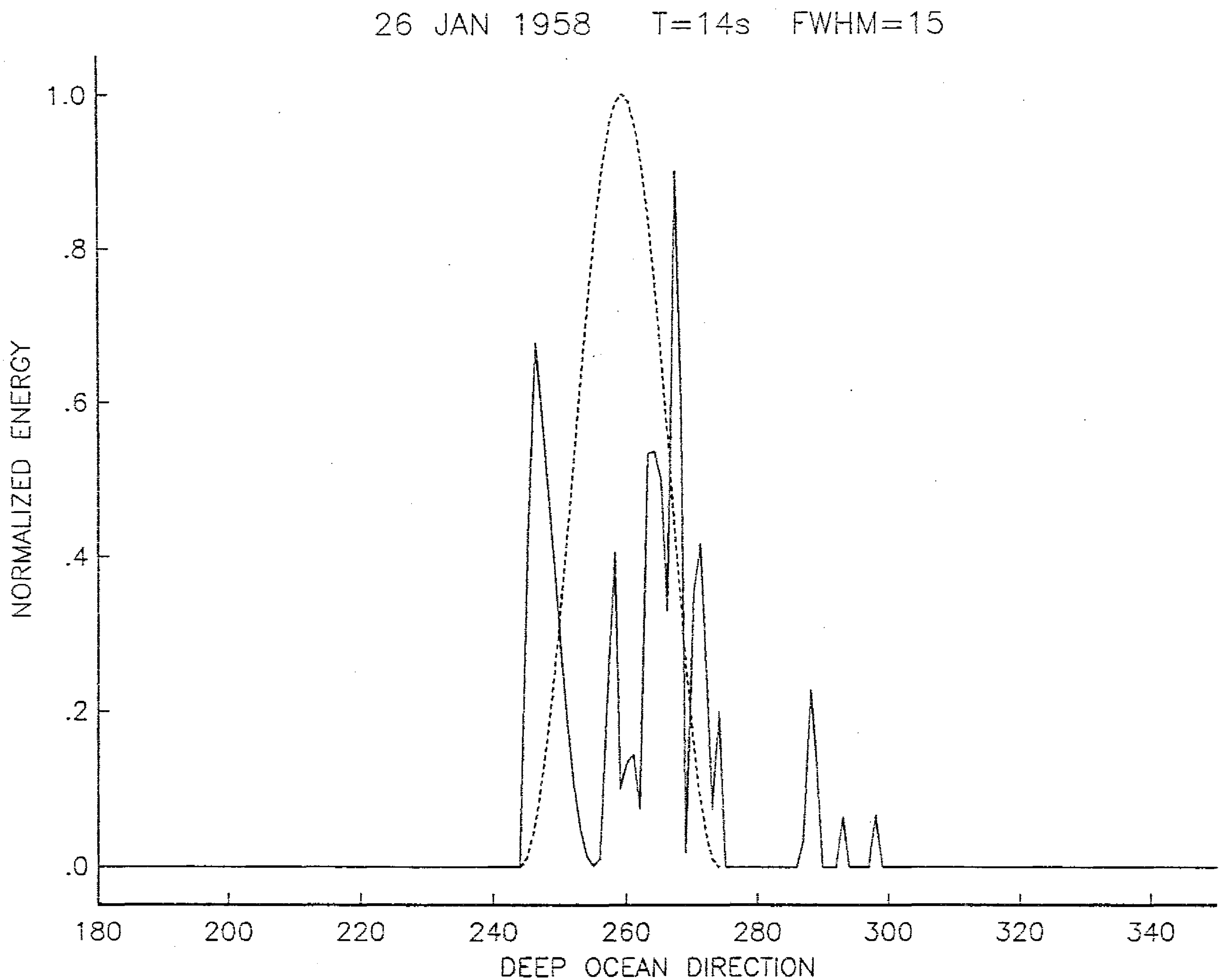

16 JAN 1978 T=13s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

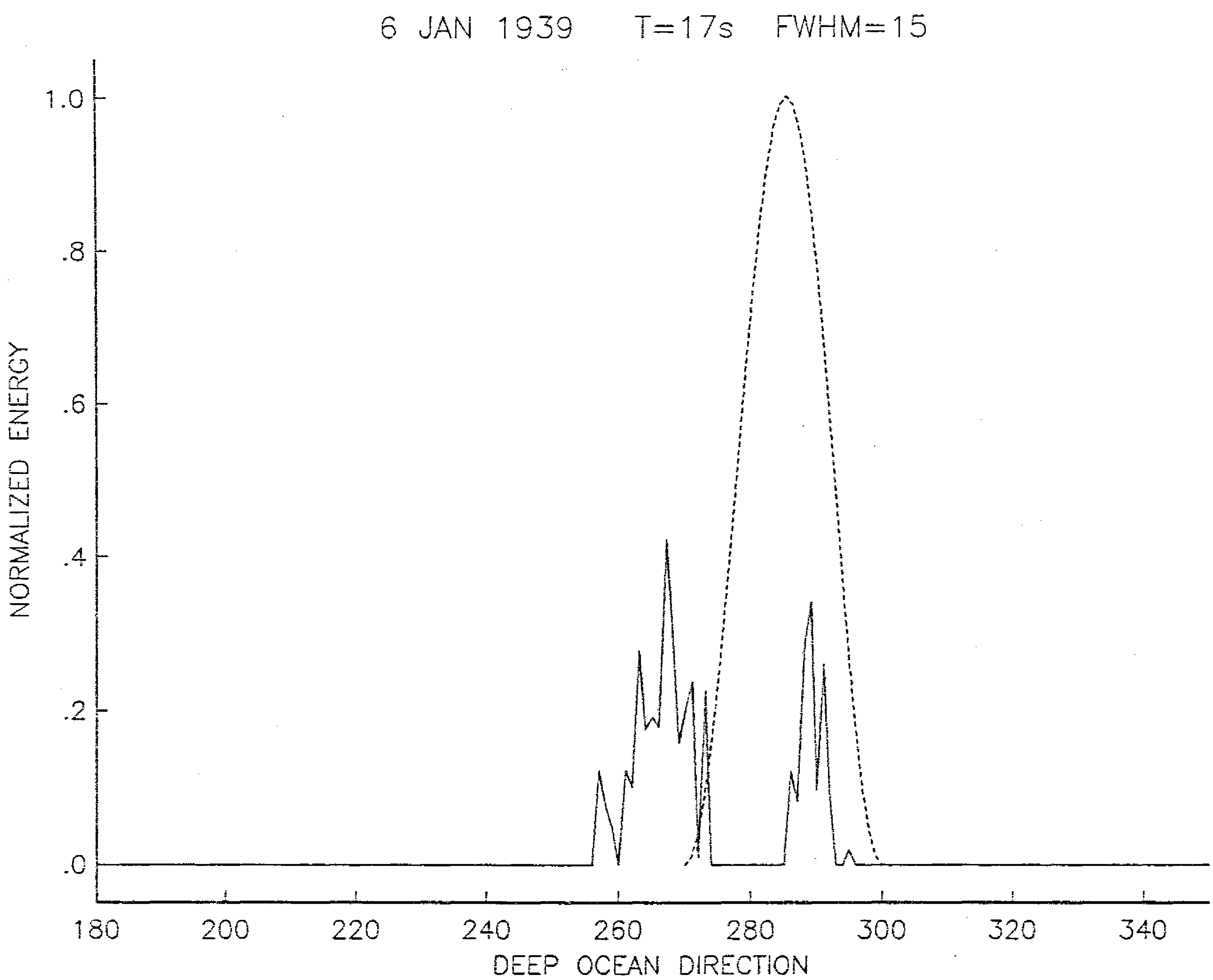

6 FEB 1969 T=13s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

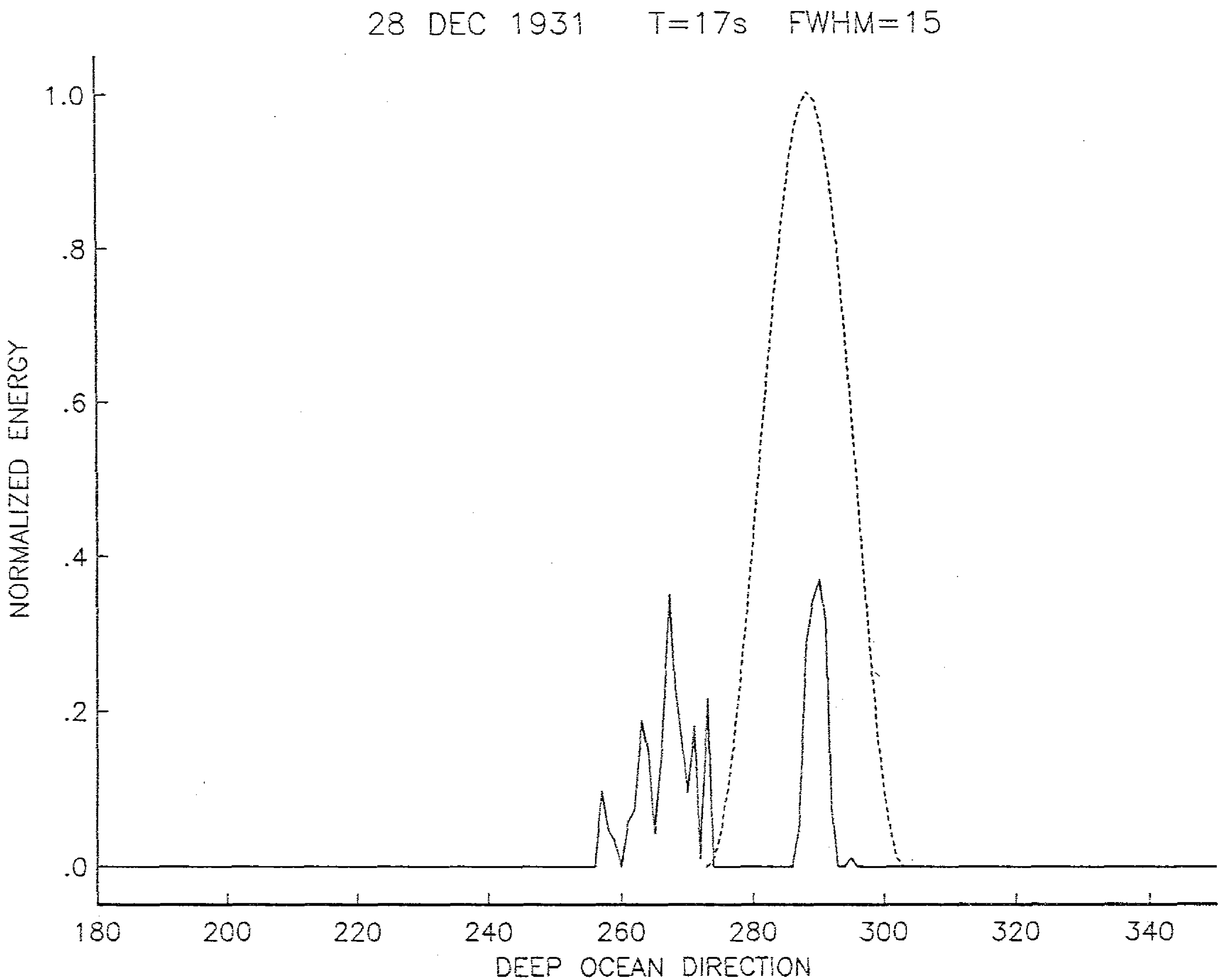


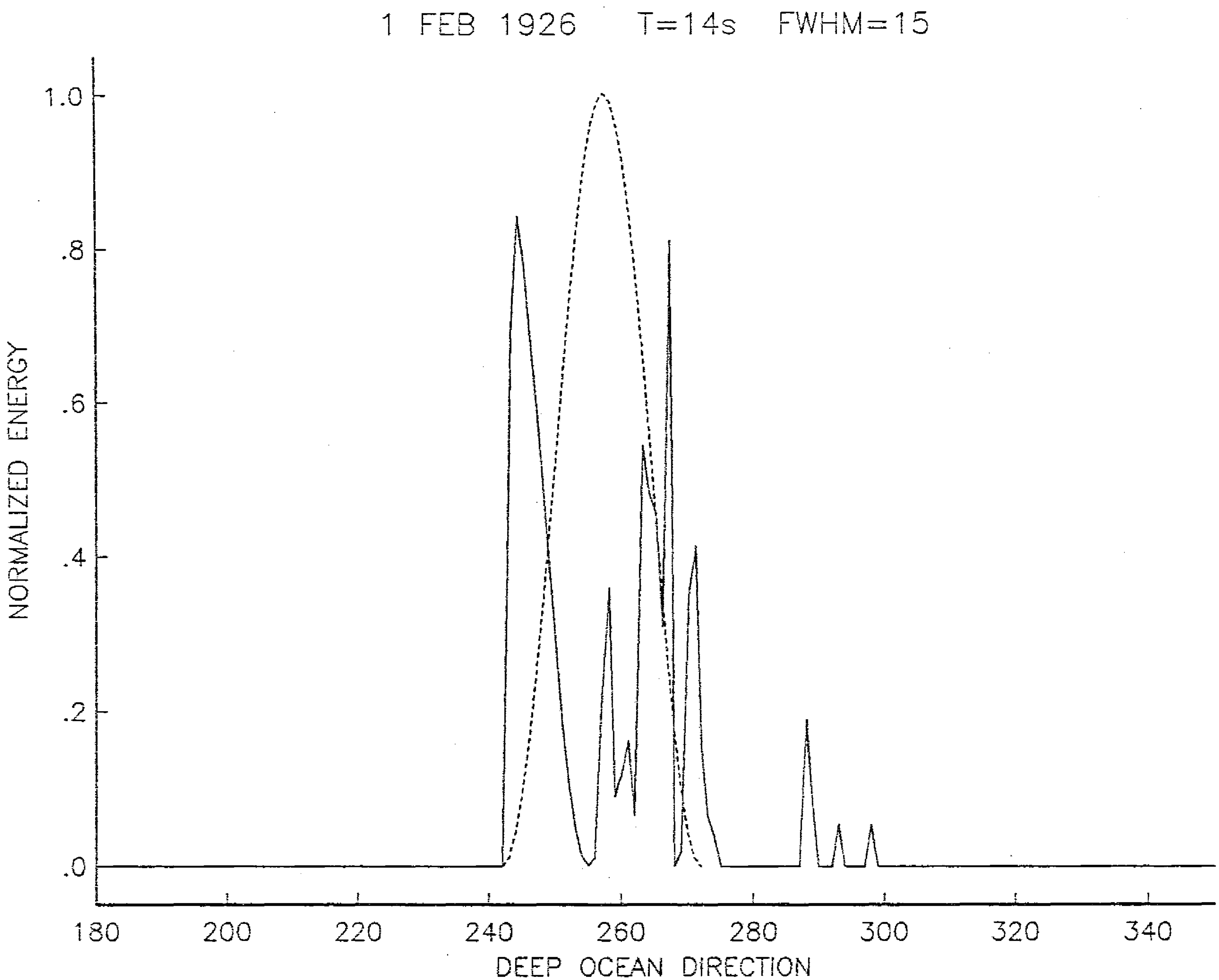
9 FEB 1960 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION


5 APR 1958 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

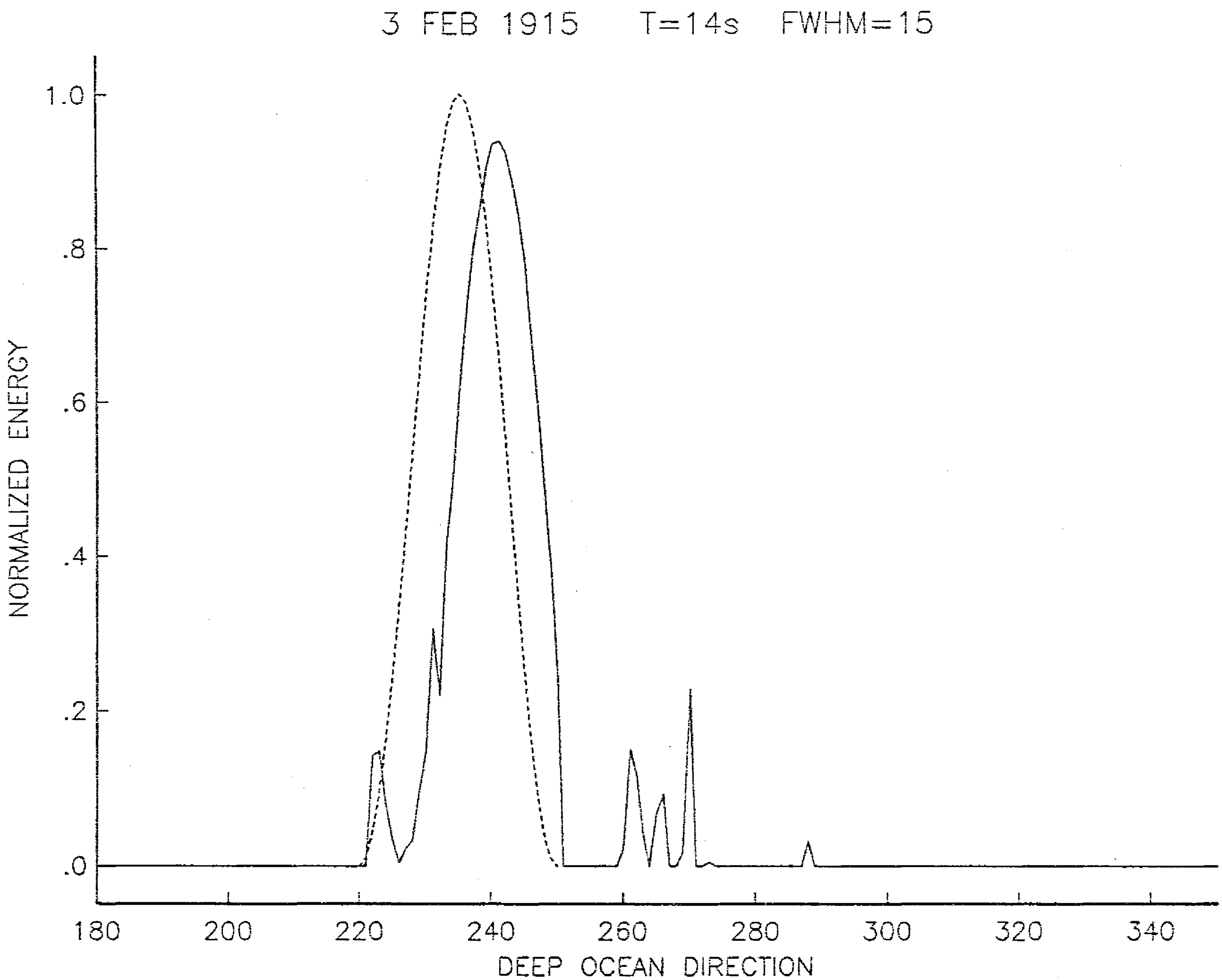

26 JAN 1958 T=14s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

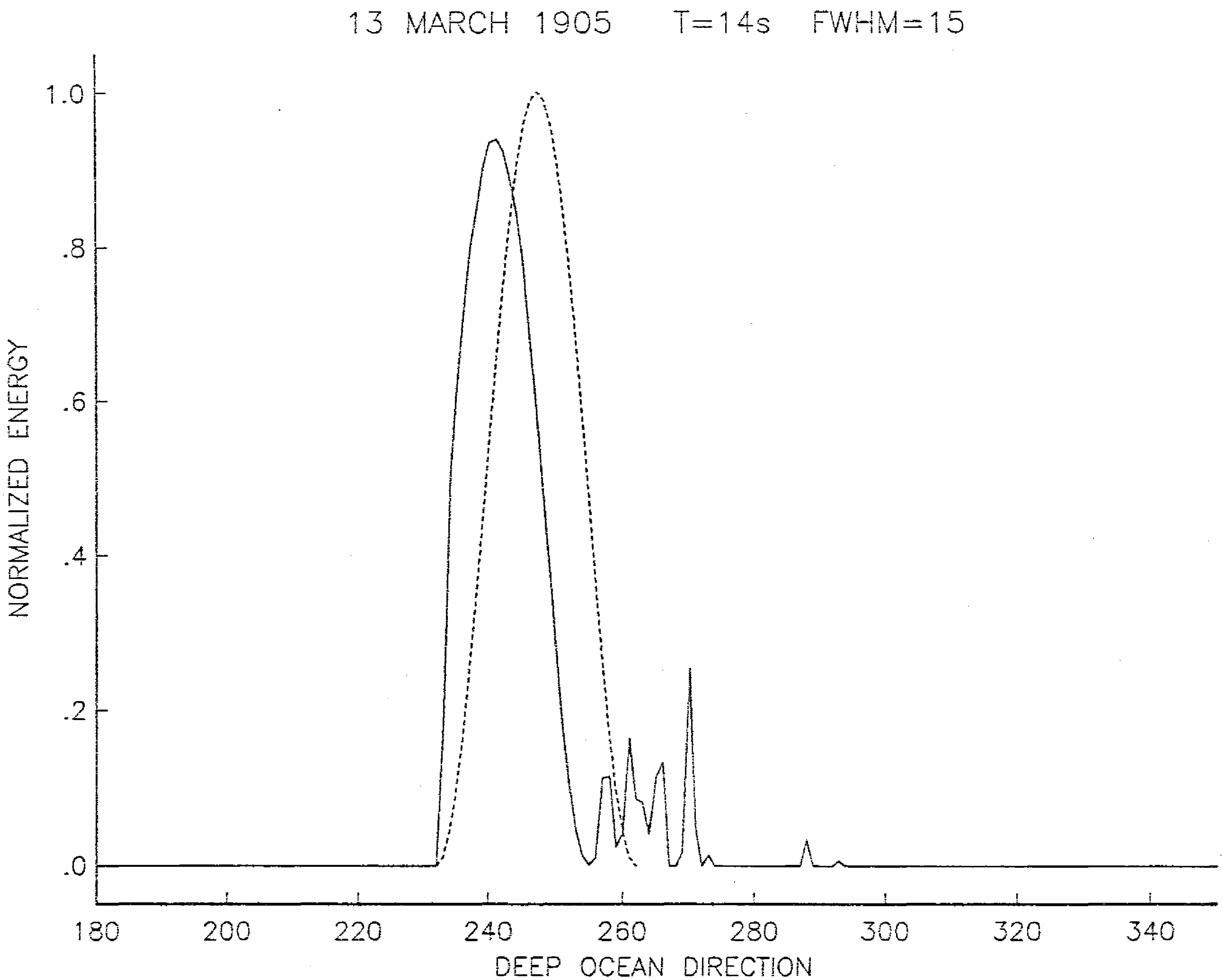

6 JAN 1939 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION


28 DEC 1931 T=17s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION



1 FEB 1926 T=14s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION


3 FEB 1915 T=14s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION


13 MARCH 1905 T=14s FWHM=15
NORMALIZED ENERGY
1.0
.8
.6
.4
.2
.0
180
200
220
240
260
280
300
320
340
DEEP OCEAN DIRECTION

Attachment B

Point Loma Wave Refraction

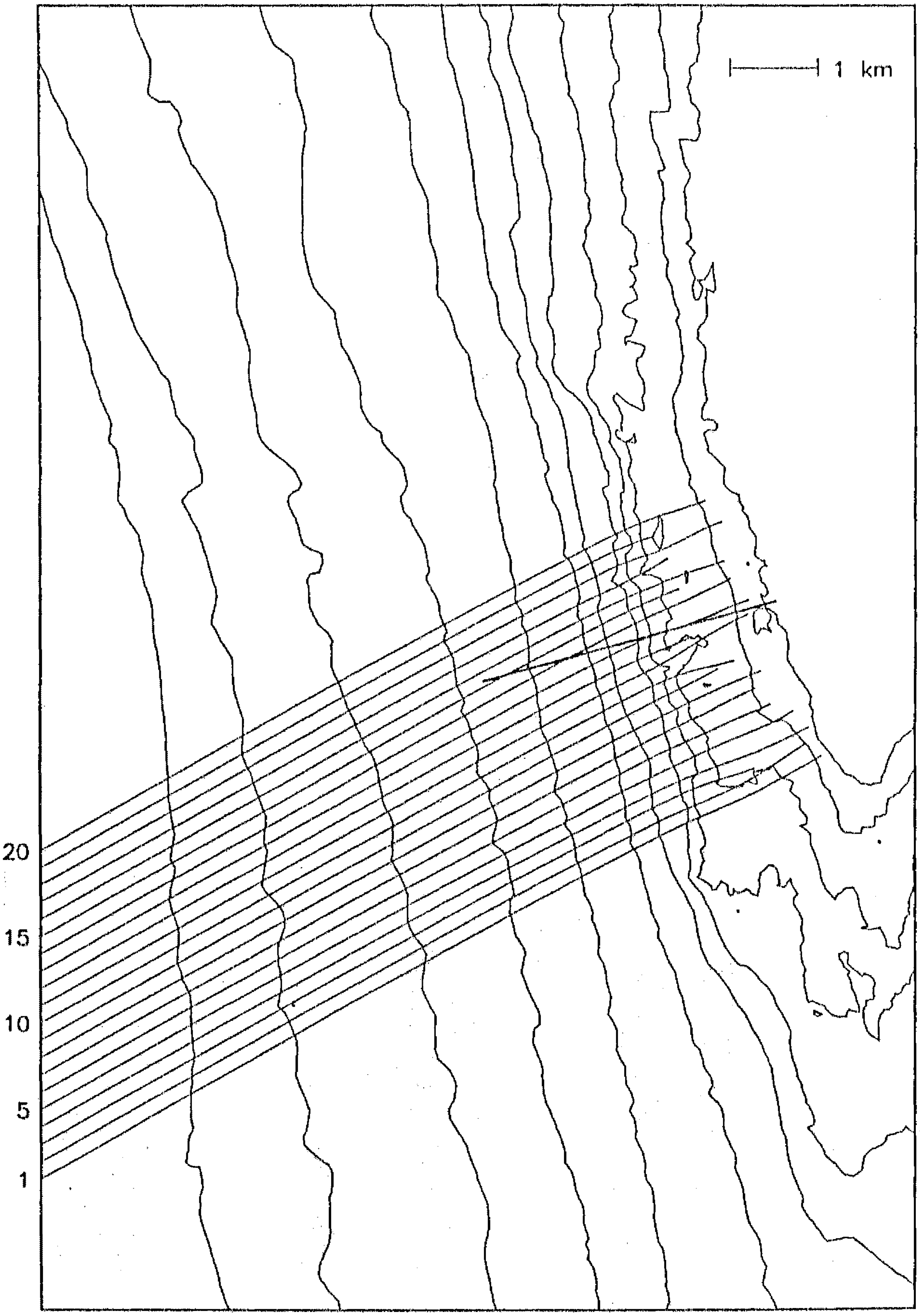
POINT LOMA WAVE REFRACTION

PERIOD = 13.0 s DIRECTION = 210.0° H_o = 4.7 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 40 9	117 14 50	226.5	4.3	3.4
2	32 40 6	117 14 55	228.1	6.5	5.3
3	32 40 8	117 15 6	236.0	6.4	5.0
4	32 40 14	117 15 11	235.3	7.3	5.7
5	32 40 23	117 15 12	235.8	6.8	5.3
6	32 40 39	117 15 0	230.8	3.9	3.0
7	32 40 23	117 15 35	233.1	13.1	10.3
8	32 40 52	117 15 1	245.4	2.2	1.7
9	32 40 34	117 15 47	226.4	13.6	10.7
10	32 40 56	117 15 20	236.2	5.4	4.3
11	32 41 1	117 15 27	234.7	8.0	6.3
12	32 41 8	117 15 25	244.7	5.4	4.2
13	32 41 11	117 15 48	217.6	11.9	9.3
14	32 41 2	117 16 2	236.6	14.9	11.6
15	32 41 21	117 15 35	236.9	8.7	6.8
16	32 41 32	117 15 41	242.7	9.4	7.4
17	32 41 40	117 15 35	241.1	5.0	3.9
18	32 41 49	117 15 27	263.8	3.5	2.7
19	32 42 7	117 15 36	242.5	6.0	4.7
20	32 42 2	117 15 49	242.0	10.6	8.3

POINT LOMA WAVE REFRACTION

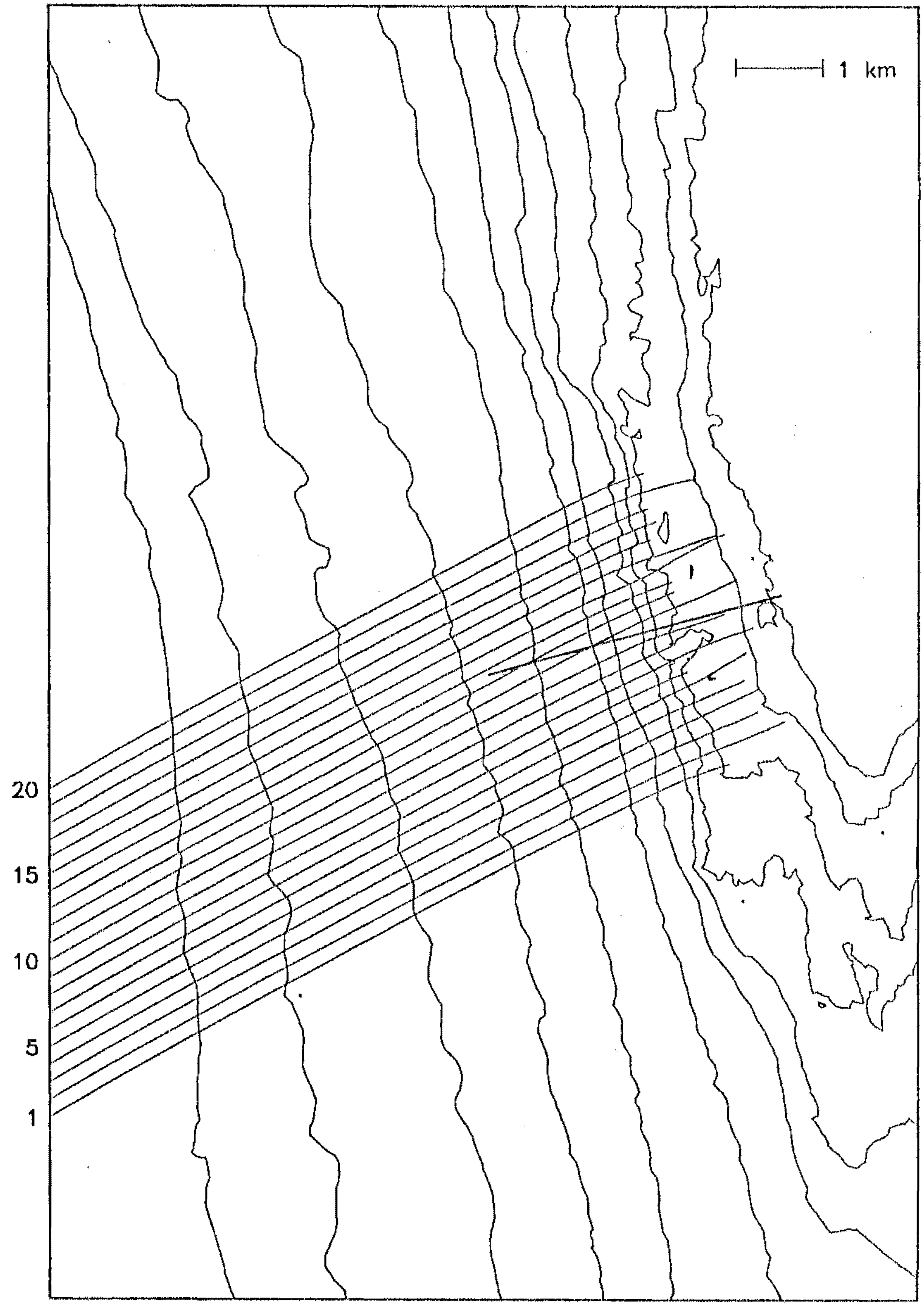


PERIOD = 13.0 s DIRECTION = 240.0° H_o = 6.0 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 49	117 14 46	240.4	7.3	5.7
2	32 39 54	117 14 53	234.9	9.1	7.1
3	32 39 45	117 15 16	249.7	13.7	10.7
4	32 40 0	117 14 51	243.5	5.5	4.6
5	32 40 6	117 14 58	235.6	7.4	5.8
6	32 40 8	117 15 8	246.2	7.6	5.9
7	32 40 14	117 15 15	245.3	8.7	6.8
8	32 40 20	117 15 12	249.0	7.0	5.4
9	32 40 21	117 15 31	239.8	13.5	10.6
10	32 40 24	117 15 23	254.2	11.3	8.8
11	32 40 41	117 15 17	249.0	5.9	4.5
12	32 40 33	117 15 42	242.7	14.4	11.2
13	32 40 47	117 15 16	250.2	5.6	4.4
14	32 40 55	117 15 23	244.0	7.7	6.1
15	32 40 51	117 15 46	249.4	10.2	7.9
16	32 41 1	117 15 26	251.2	7.7	6.0
17	32 41 2	117 15 51	238.1	12.2	9.5
18	32 41 16	117 15 27	246.9	5.9	4.6
19	32 41 10	117 15 57	253.2	9.0	7.1
20	32 41 24	117 15 34	249.7	7.1	5.6

POINT LOMA WAVE REFRACTION

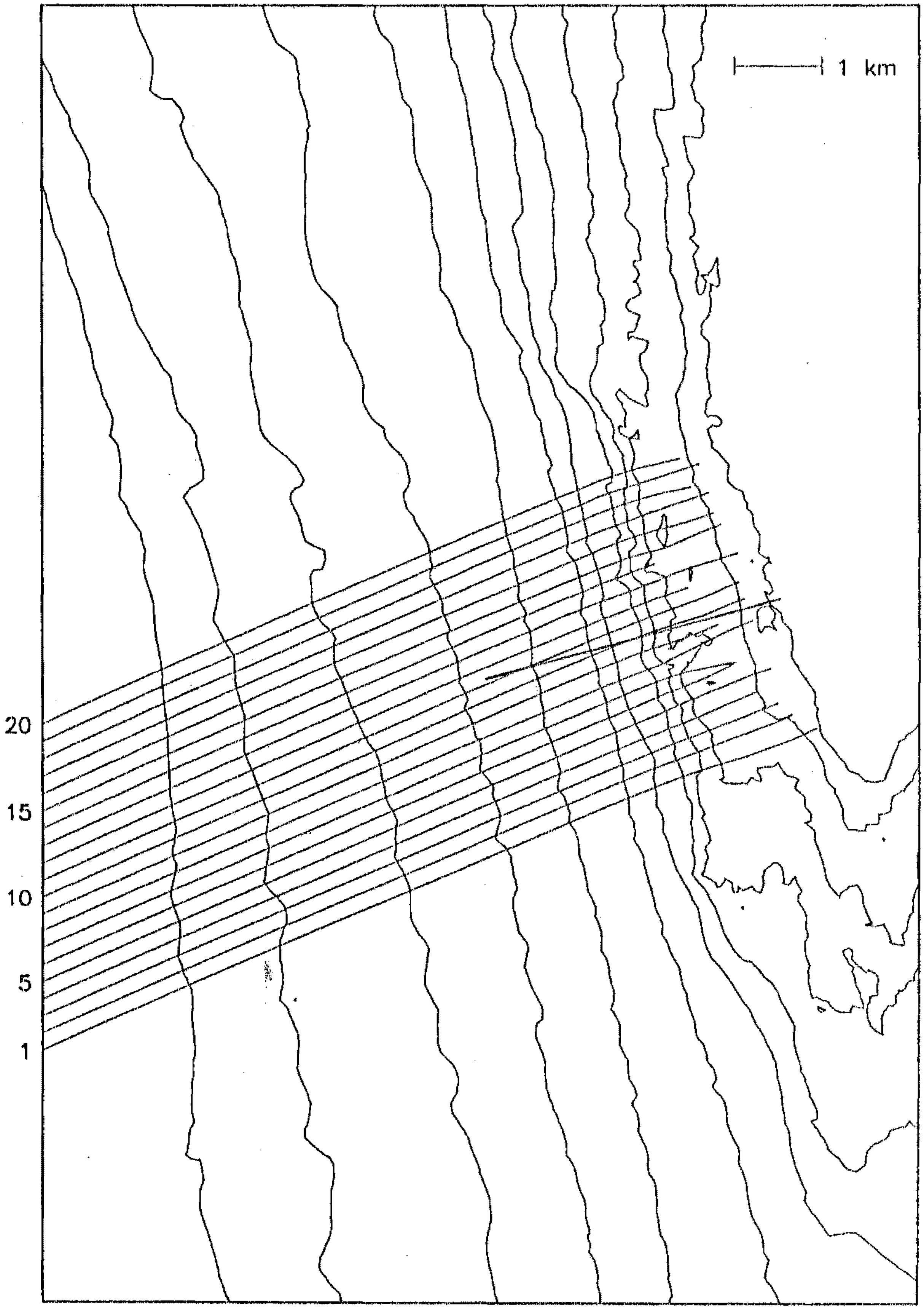


PERIOD = 14.0 s   DIRECTION = 240.0° H_o = 8.3 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 43	117 15 29	255.0	13.1	10.3
2	32 39 59	117 15 4	242.3	10.4	8.1
3	32 40 2	117 15 16	247.8	11.1	8.9
4	32 40 11	117 15 15	245.2	9.6	7.5
5	32 40 14	117 15 21	246.3	11.7	9.2
6	32 40 25	117 15 20	239.0	10.3	8.1
7	32 40 17	117 15 46	250.9	12.1	9.4
8	32 40 34	117 15 17	250.7	6.5	5.1
9	32 40 28	117 15 51	242.7	14.5	11.3
10	32 40 39	117 15 29	250.3	12.2	9.5
11	32 40 52	117 15 24	244.1	9.6	7.4
12	32 40 48	117 15 51	246.9	12.7	10.0
13	32 40 52	117 15 52	250.5	13.8	10.8
14	32 41 9	117 15 28	247.1	6.9	5.4
15	32 41 9	117 15 32	252.5	9.2	7.2
16	32 41 7	117 16 1	252.0	12.7	9.9
17	32 41 14	117 15 58	251.9	10.7	8.2
18	32 41 19	117 16 2	247.7	12.7	9.9
19	32 41 30	117 15 39	258.6	9.0	7.0
20	32 41 32	117 16 3	252.1	11.2	8.7

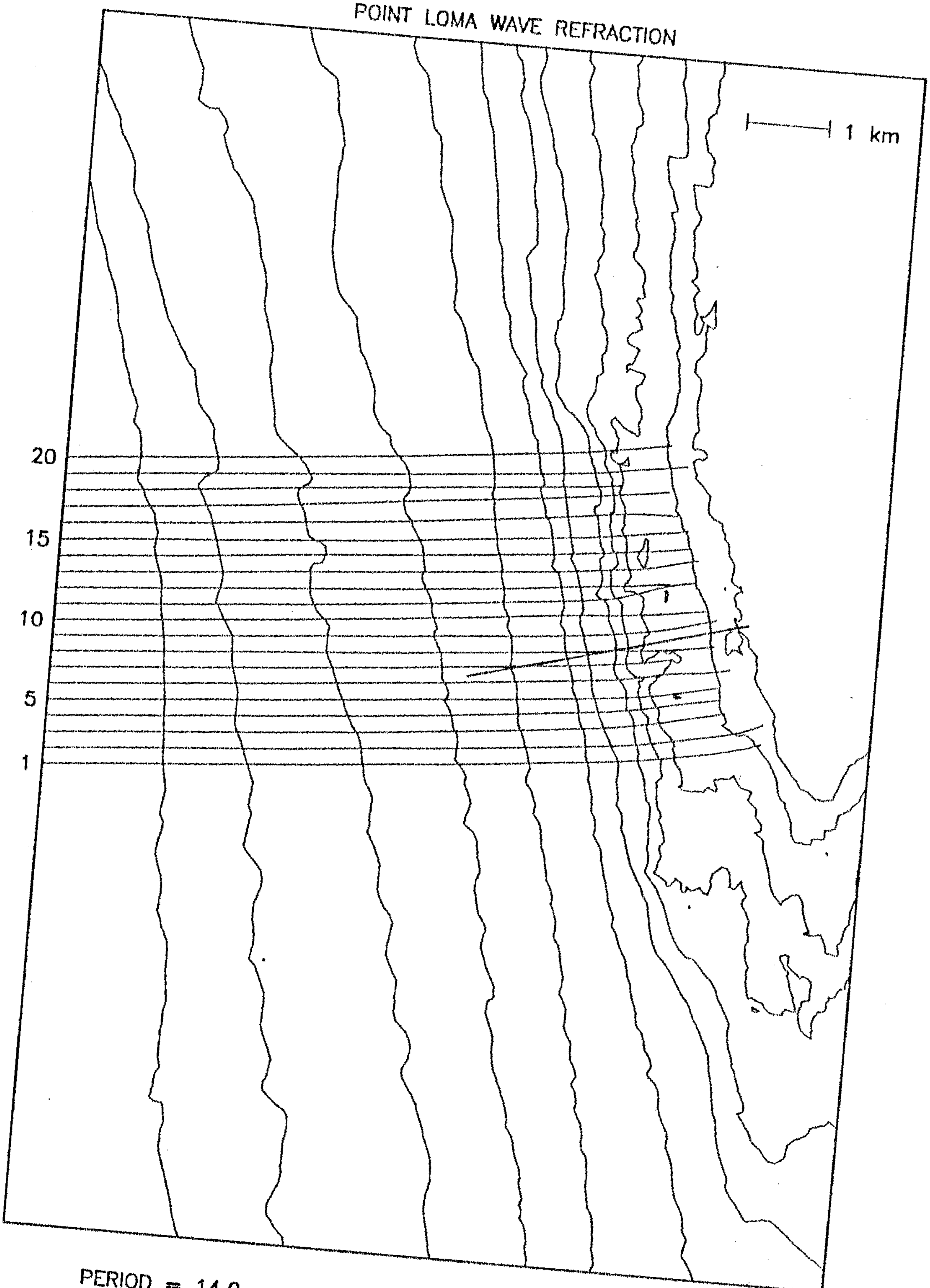
POINT LOMA WAVE REFRACTION



PERIOD = 14.0 s DIRECTION = 245.0° H_o = 5.6 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 58	117 14 50	244.7	5.0	3.9
2	32 40 3	117 15 0	240.8	8.2	6.4
3	32 40 8	117 15 6	247.6	6.8	5.5
4	32 40 12	117 15 16	247.3	9.6	7.5
5	32 40 20	117 15 9	250.5	6.4	5.0
6	32 40 22	117 15 26	242.6	12.6	9.8
7	32 40 23	117 15 25	257.0	12.1	9.4
8	32 40 38	117 15 17	249.1	5.2	4.4
9	32 40 37	117 15 32	246.5	12.5	9.8
10	32 40 47	117 15 20	248.8	7.6	5.9
11	32 40 53	117 15 22	246.7	7.8	6.1
12	32 40 50	117 15 44	252.2	9.5	7.5
13	32 41 3	117 15 22	251.7	5.5	4.3
14	32 40 59	117 15 59	250.1	12.9	10.1
15	32 41 14	117 15 29	249.8	7.0	5.4
16	32 41 18	117 15 32	249.7	8.0	6.3
17	32 41 25	117 15 34	249.5	6.8	5.4
18	32 41 27	117 15 41	258.4	10.1	7.9
19	32 41 36	117 15 38	251.9	7.5	5.7
20	32 41 38	117 15 46	256.6	11.4	8.9

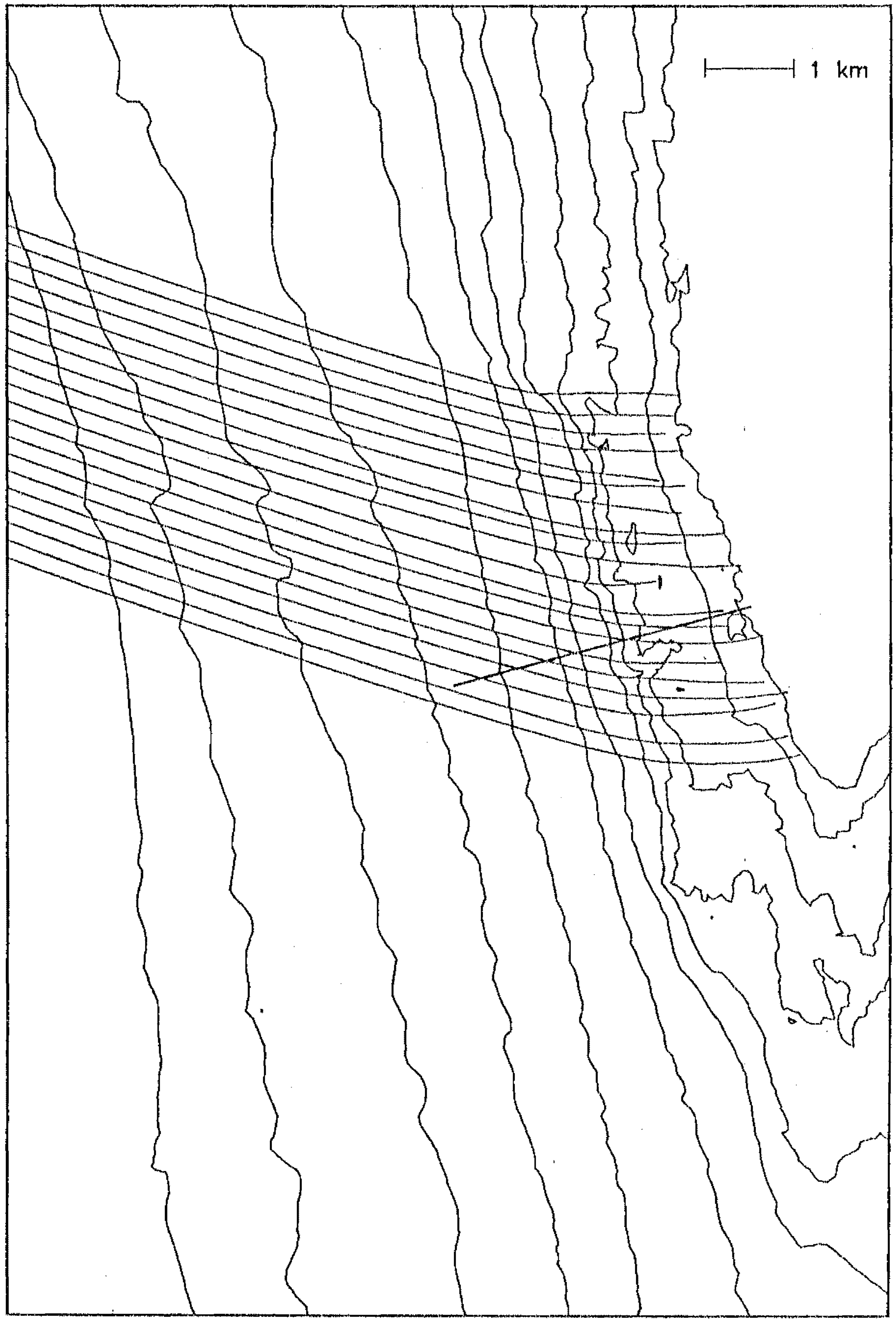


PERIOD = 14.0 s DIRECTION = 265.0° H_o = 5.6 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 59	117 14 56	249.5	8.2	6.4
2	32 40 7	117 14 55	242.9	6.7	5.2
3	32 40 10	117 15 13	256.2	8.6	6.7
4	32 40 15	117 15 16	258.2	9.1	7.1
5	32 40 20	117 15 17	257.6	9.7	7.6
6	32 40 28	117 15 11	257.2	5.2	4.1
7	32 40 30	117 15 39	258.1	14.6	11.4
8	32 40 37	117 15 20	261.3	8.1	6.4
9	32 40 47	117 15 19	253.8	7.1	5.6
10	32 40 51	117 15 24	259.7	9.8	7.7
11	32 41 2	117 15 30	256.5	9.4	7.3
12	32 41 0	117 15 42	266.3	11.2	8.7
13	32 41 10	117 15 29	257.9	7.1	5.5
14	32 41 15	117 15 33	259.2	9.2	7.2
15	32 41 22	117 15 34	258.6	8.4	6.2
16	32 41 27	117 15 38	267.0	9.0	7.0
17	32 41 36	117 15 44	260.3	10.5	8.2
18	32 41 37	117 16 7	262.8	14.9	11.6
19	32 41 47	117 15 36	261.6	5.3	4.1
20	32 41 55	117 15 44	261.0	8.0	6.2

POINT LOMA WAVE REFRACTION

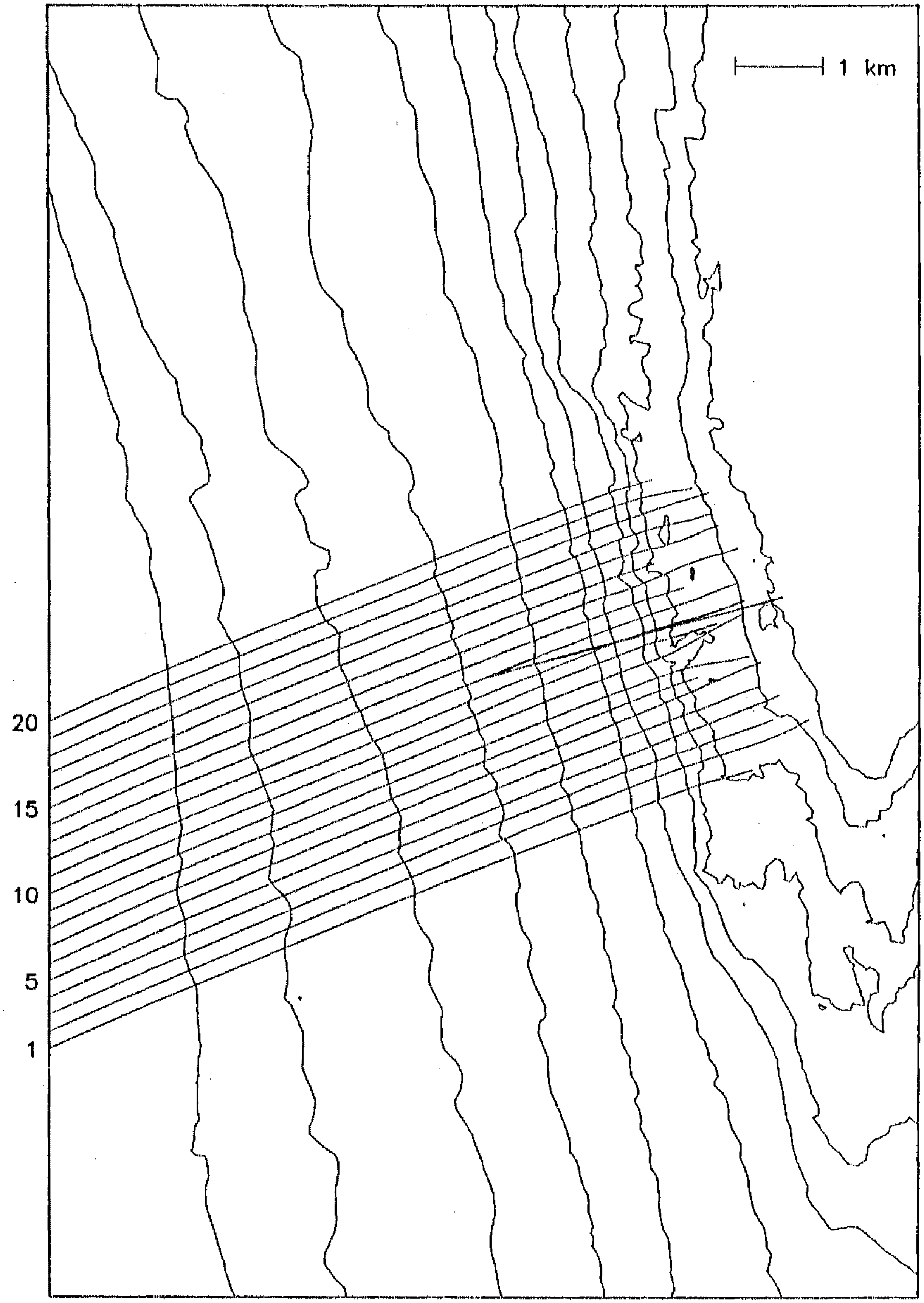


PERIOD = 14.0 s DIRECTION = 290.0° H_o = 2.9 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 51	117 14 45	253.6	5.5	4.3
2	32 39 58	117 14 50	255.2	5.0	3.9
3	32 40 10	117 14 55	252.4	6.4	4.9
4	32 40 14	117 14 49	258.4	3.4	2.6
5	32 40 18	117 14 59	266.2	5.7	4.5
6	32 40 25	117 15 18	265.6	9.2	7.2
7	32 40 35	117 14 59	253.6	3.9	3.0
8	32 40 39	117 15 15	267.6	4.0	3.1
9	32 40 45	117 15 16	264.2	5.8	4.5
10	32 40 56	117 15 45	264.7	9.6	7.6
11	32 41 2	117 15 8	265.4	2.8	2.5
12	32 41 10	117 15 33	269.4	10.0	7.8
13	32 41 14	117 15 15	262.4	3.7	2.9
14	32 41 21	117 15 32	270.6	7.2	5.6
15	32 41 31	117 15 31	266.5	4.8	3.8
16	32 41 33	117 15 42	276.5	10.1	7.9
17	32 41 44	117 15 31	267.1	3.7	2.9
18	32 41 51	117 15 38	267.0	6.1	4.7
19	32 41 57	117 15 34	274.7	4.2	3.3
20	32 42 5	117 15 32	270.9	4.5	3.8

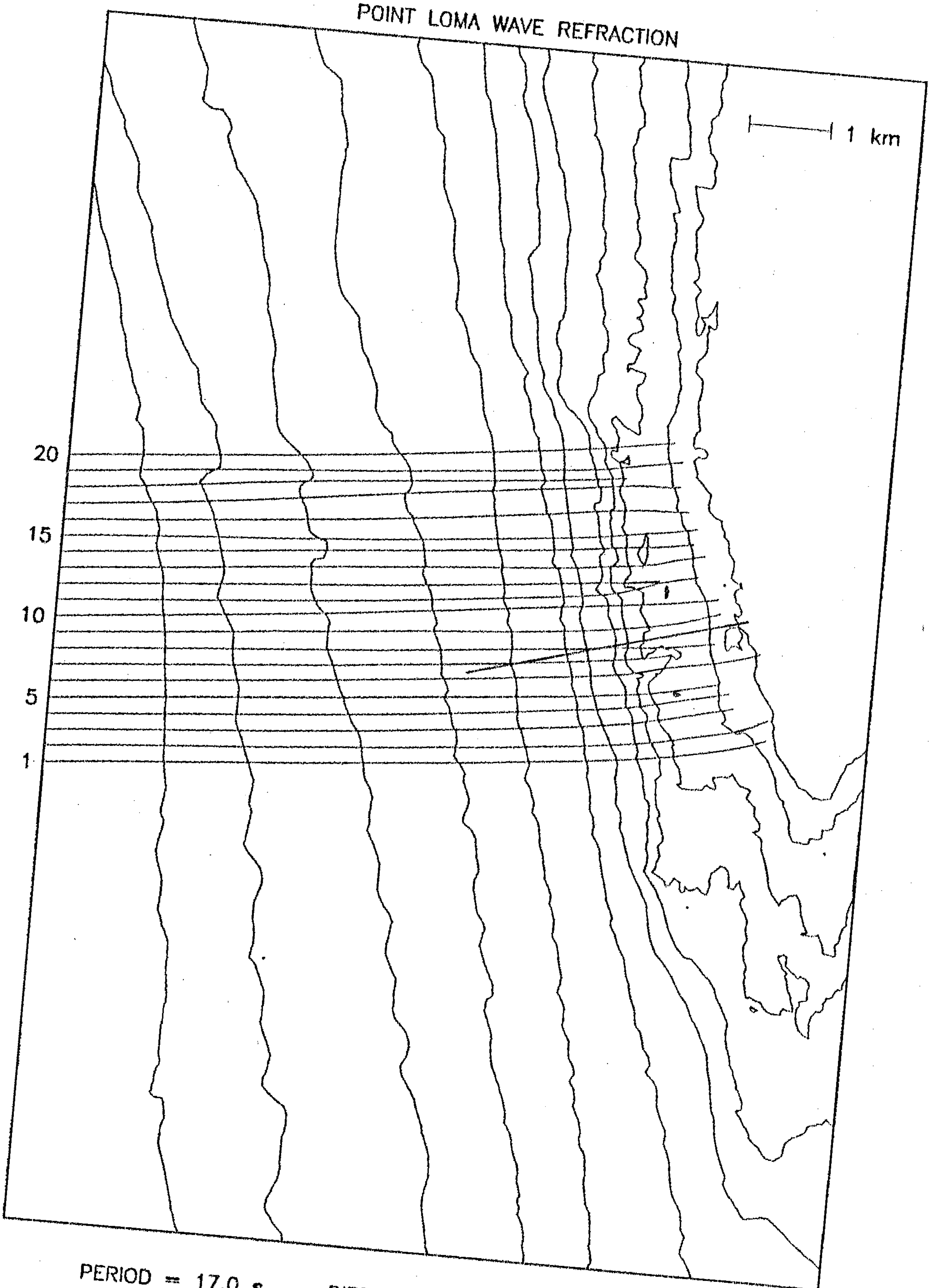
POINT LOMA WAVE REFRACTION



PERIOD = 17.0 s DIRECTION = 245.0° H_o = 5.8 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 42	117 15 20	252.0	13.7	10.7
2	32 40 1	117 14 54	241.9	7.4	5.8
3	32 40 0	117 15 10	249.7	9.8	7.7
4	32 40 9	117 15 7	247.4	6.9	5.3
5	32 40 13	117 15 15	249.2	9.1	7.1
6	32 40 22	117 15 14	248.4	8.0	6.3
7	32 40 16	117 15 42	250.0	11.0	8.6
8	32 40 24	117 15 20	256.9	10.2	8.0
9	32 40 40	117 15 22	245.0	9.5	7.4
10	32 40 36	117 15 33	249.3	12.8	10.0
11	32 40 45	117 15 21	250.3	8.6	6.7
12	32 40 52	117 15 24	247.4	9.5	7.3
13	32 40 50	117 15 47	253.6	10.9	8.3
14	32 41 5	117 15 23	252.0	5.6	4.4
15	32 40 59	117 15 60	252.8	13.2	10.3
16	32 41 14	117 15 31	249.8	8.4	6.5
17	32 41 18	117 15 35	251.6	9.8	7.5
18	32 41 26	117 15 35	250.6	7.3	5.7
19	32 41 27	117 15 42	260.8	10.3	8.1
20	32 41 31	117 15 59	253.1	10.1	7.9

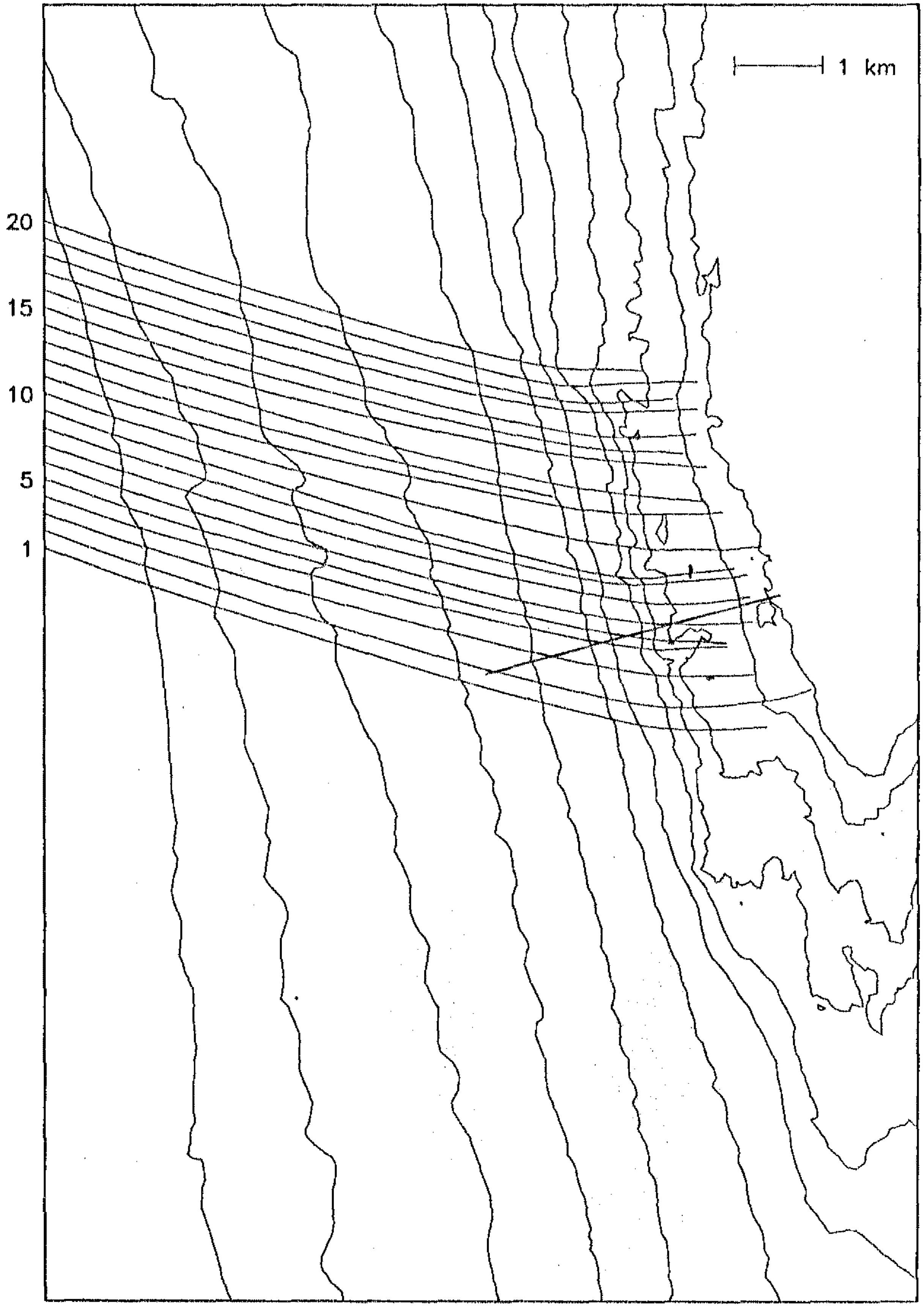


PERIOD = 17.0 s DIRECTION = 265.0° H_o = 4.2 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 40 0	117 14 53	248.6	6.8	5.3
2	32 40 8	117 14 52	243.3	5.4	4.2
3	32 40 11	117 15 9	254.9	7.2	5.7
4	32 40 17	117 15 11	257.8	7.1	5.5
5	32 40 20	117 15 16	257.3	9.0	7.0
6	32 40 34	117 14 58	250.3	4.1	3.2
7	32 40 30	117 15 39	257.2	14.5	11.4
8	32 40 36	117 15 19	260.9	7.6	5.9
9	32 40 48	117 15 17	253.2	5.6	4.4
10	32 40 54	117 15 18	256.3	5.2	4.1
11	32 41 3	117 15 28	255.6	8.6	6.7
12	32 41 0	117 15 46	261.7	11.3	9.0
13	32 41 11	117 15 26	257.7	5.7	4.4
14	32 41 15	117 15 32	259.2	8.4	6.6
15	32 41 22	117 15 30	256.2	6.3	4.9
16	32 41 28	117 15 34	264.4	6.8	5.3
17	32 41 38	117 15 39	263.3	7.7	6.1
18	32 41 39	117 16 4	261.9	12.9	10.1
19	32 41 48	117 15 39	259.1	6.2	4.8
20	32 41 55	117 15 43	261.8	7.5	5.8

POINT LOMA WAVE REFRACTION

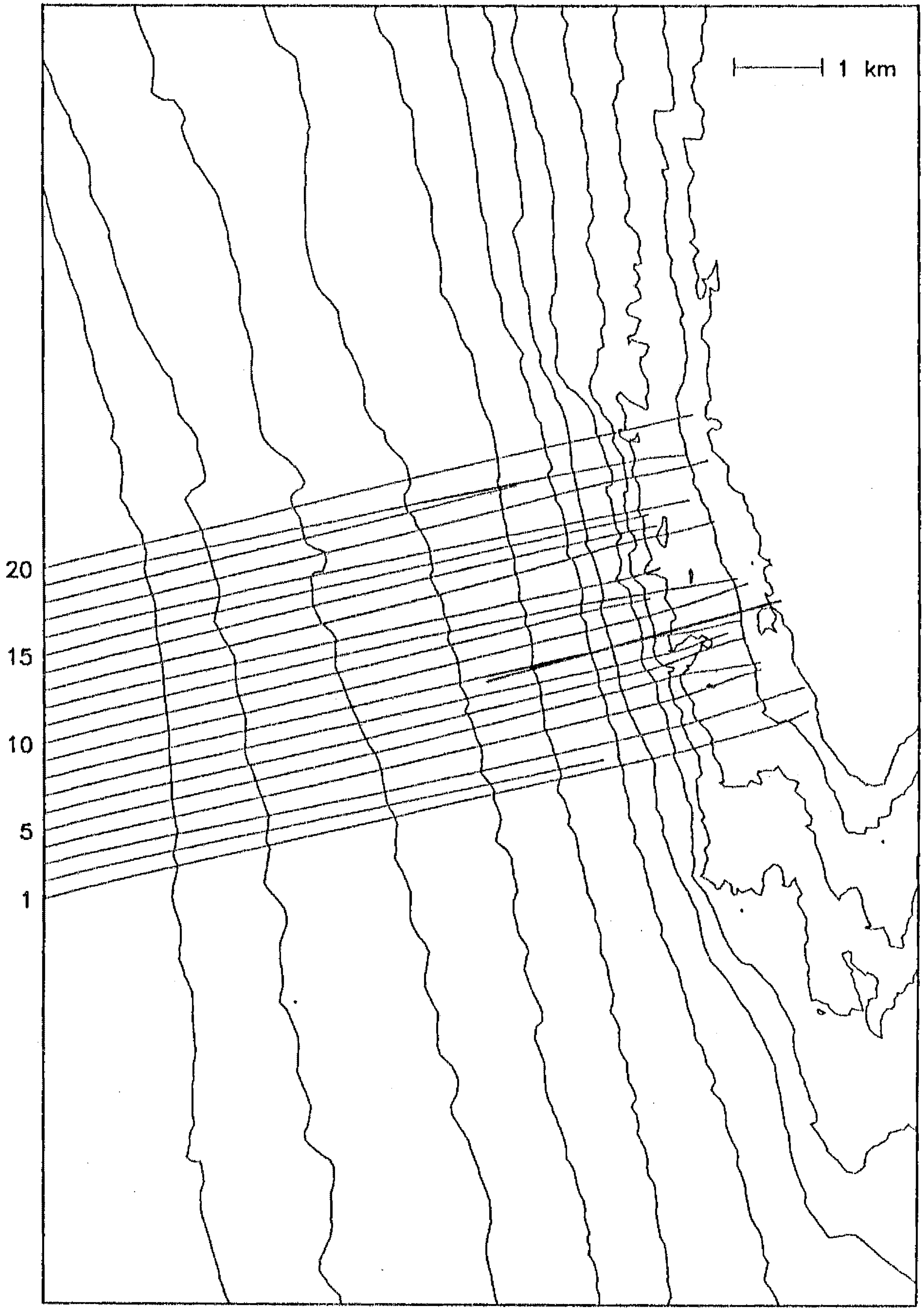


PERIOD = 17.0 s DIRECTION = 290.0° H_o = 4.0 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 57	117 15 12	265.6	11.2	8.8
2	32 40 11	117 14 52	249.5	5.2	4.0
3	32 40 16	117 15 17	266.3	9.4	7.4
4	32 40 27	117 15 28	267.8	11.6	9.1
5	32 40 28	117 15 29	271.8	11.6	9.1
6	32 40 36	117 15 17	265.2	5.7	4.6
7	32 40 46	117 15 17	261.8	6.3	4.9
8	32 40 54	117 15 18	258.8	5.2	4.1
9	32 40 57	117 15 24	259.5	7.7	5.9
10	32 41 5	117 15 11	265.7	3.7	2.9
11	32 41 18	117 15 28	262.6	6.0	4.7
12	32 41 24	117 16 42	281.1	49.8	*****
13	32 41 22	117 15 36	271.4	9.1	7.1
14	32 41 35	117 15 35	268.9	5.9	4.6
15	32 41 40	117 15 44	272.9	10.1	7.9
16	32 41 47	117 15 38	264.8	6.0	4.7
17	32 41 57	117 15 38	270.1	5.9	4.6
18	32 42 0	117 15 48	268.3	10.2	7.9
19	32 42 7	117 15 37	271.9	6.3	5.0
20	32 42 11	117 16 1	270.4	14.3	11.1

POINT LOMA WAVE REFRACTION

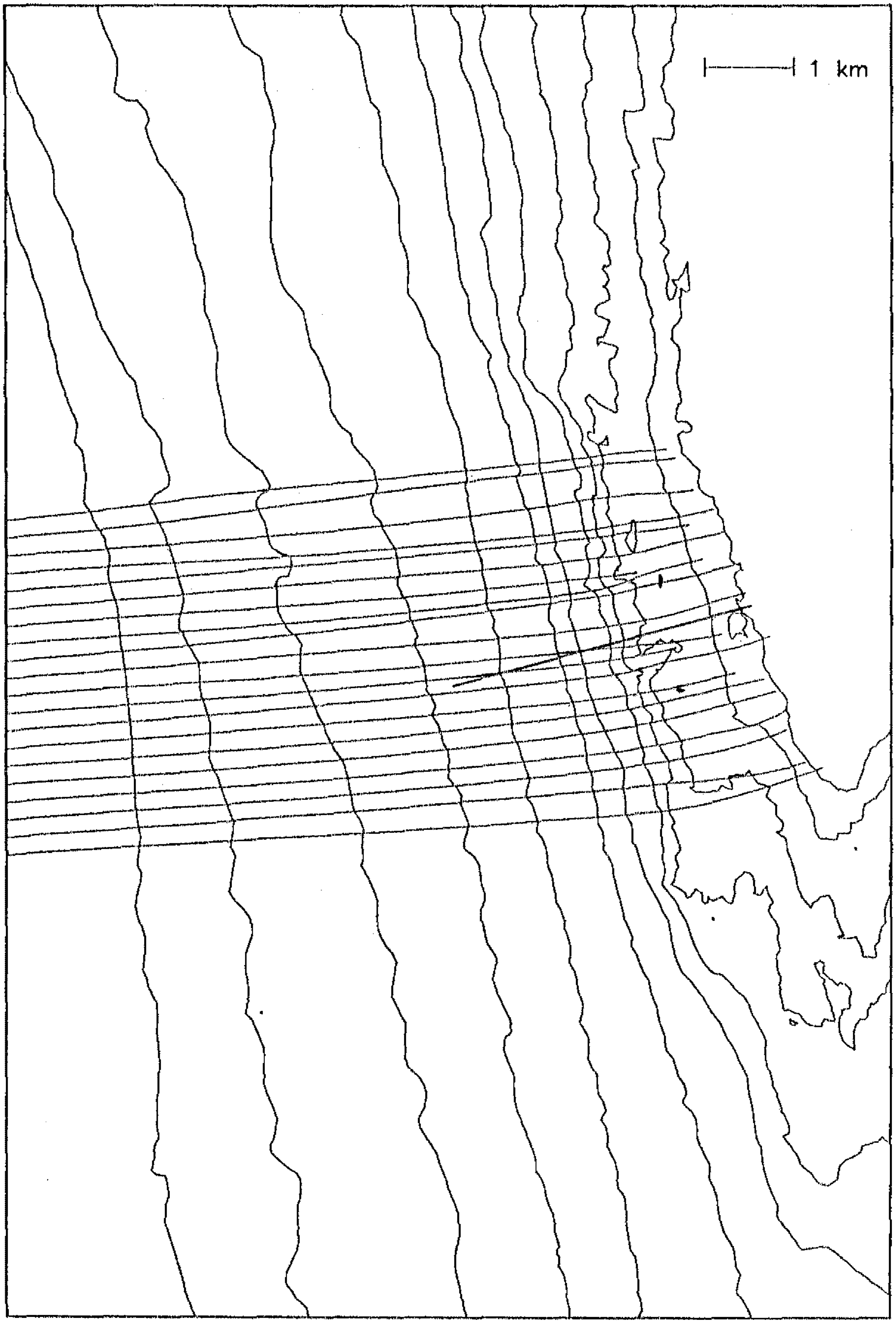


PERIOD = 20.0 s DIRECTION = 255.0° H_o = 4.5 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 40 6	117 14 54	242.0	6.2	4.8
2	32 39 48	117 16 24	257.7	51.6	40.9
3	32 40 15	117 14 55	250.3	5.5	4.3
4	32 40 21	117 15 14	252.2	7.9	6.0
5	32 40 24	117 15 14	258.4	7.6	5.9
6	32 40 33	117 15 20	253.7	9.2	7.2
7	32 40 34	117 15 28	250.7	11.1	8.6
8	32 40 39	117 15 21	259.7	8.9	6.9
9	32 40 53	117 15 18	249.3	5.5	4.3
10	32 40 47	117 16 1	258.8	16.8	13.6
11	32 40 55	117 15 23	257.5	7.3	5.7
12	32 40 58	117 15 56	250.3	13.4	10.5
13	32 41 16	117 15 32	253.3	8.4	6.6
14	32 41 14	117 15 57	260.4	9.7	7.6
15	32 41 18	117 16 1	255.4	12.5	9.8
16	32 41 23	117 15 42	259.0	11.5	9.0
17	32 41 38	117 15 34	253.7	5.2	4.1
18	32 41 29	117 16 58	256.0	56.4	*****
19	32 41 40	117 15 43	263.8	9.5	7.5
20	32 41 56	117 15 39	260.1	6.4	5.0

POINT LOMA WAVE REFRACTION

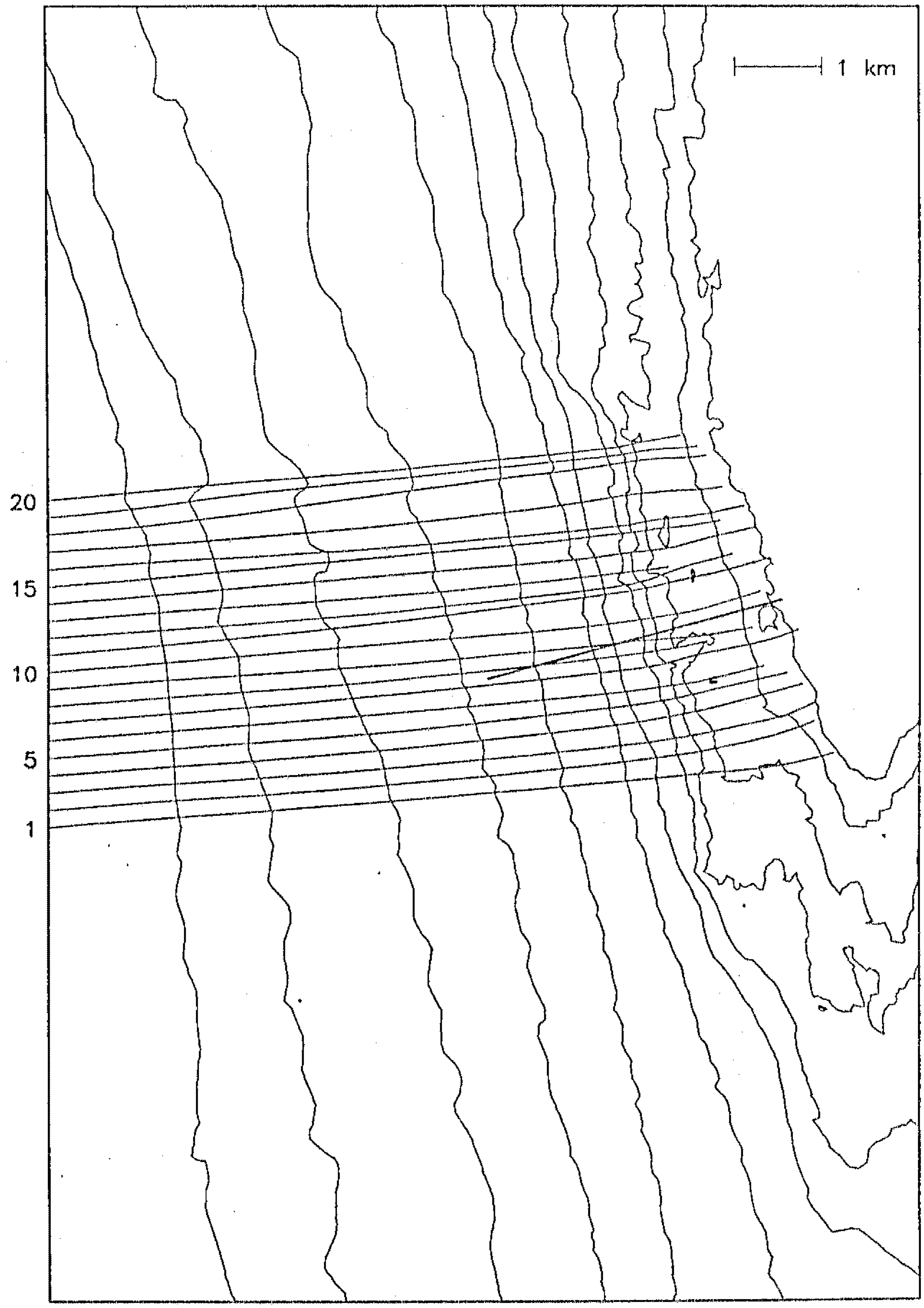


PERIOD = 20.0 s DIRECTION = 265.0° H_o = 2.7 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 47	117 14 36	245.8	2.7	2.1
2	32 39 49	117 14 43	251.6	5.2	4.1
3	32 40 0	117 14 51	248.6	5.1	4.0
4	32 40 8	117 14 50	245.2	4.4	3.4
5	32 40 15	117 14 55	252.6	5.6	4.3
6	32 40 19	117 14 60	254.3	5.3	4.1
7	32 40 21	117 15 12	256.2	6.9	5.4
8	32 40 35	117 14 57	247.4	2.9	2.3
9	32 40 31	117 15 37	257.0	14.2	11.1
10	32 40 36	117 15 17	260.1	6.7	5.2
11	32 40 51	117 15 6	254.5	4.1	3.2
12	32 41 2	117 15 8	256.0	3.7	2.9
13	32 41 4	117 15 24	254.0	6.4	5.0
14	32 40 59	117 15 53	259.7	12.9	10.2
15	32 41 13	117 15 15	254.1	3.8	3.0
16	32 41 16	117 15 30	258.5	7.0	5.5
17	32 41 21	117 15 19	258.8	4.2	3.2
18	32 41 29	117 15 28	260.3	5.0	3.9
19	32 41 40	117 15 35	260.3	4.7	3.7
20	32 41 44	117 15 38	264.2	5.8	4.5

POINT LOMA WAVE REFRACTION

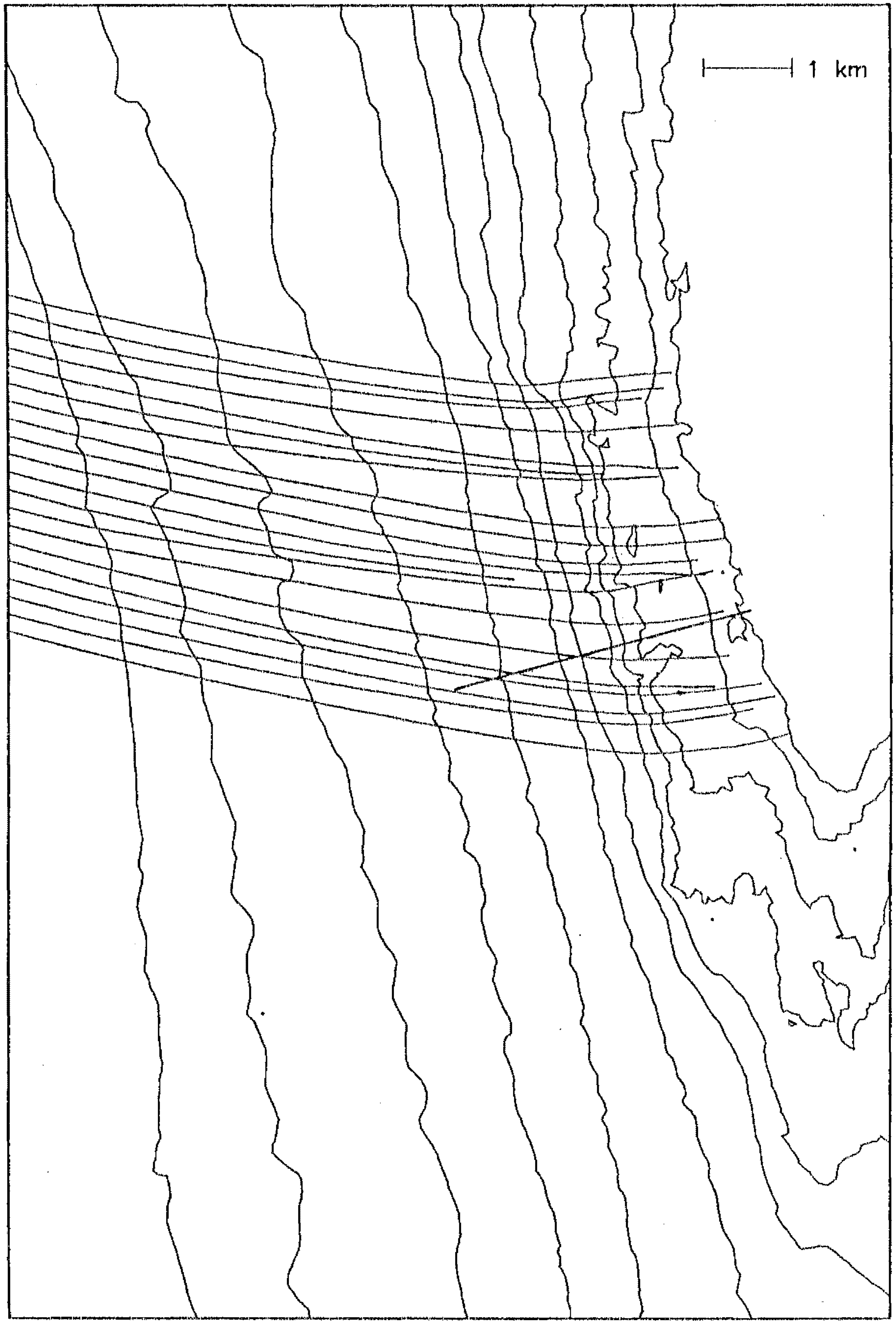


PERIOD = 20.0 s DIRECTION = 265.0° H_o = 2.9 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 39 48	117 14 43	252.2	5.6	4.4
2	32 40 0	117 14 51	248.4	5.4	4.2
3	32 40 7	117 14 51	244.8	4.7	3.6
4	32 40 14	117 14 56	253.1	5.9	4.6
5	32 40 19	117 15 3	255.7	5.6	4.4
6	32 40 21	117 15 13	256.4	7.3	5.7
7	32 40 35	117 14 57	247.3	3.1	2.4
8	32 40 31	117 15 38	257.0	14.2	11.2
9	32 40 36	117 15 18	260.2	7.0	5.5
10	32 40 50	117 15 13	253.9	4.3	3.4
11	32 41 2	117 15 8	256.1	3.9	3.1
12	32 41 3	117 15 25	254.2	6.8	5.3
13	32 40 59	117 15 54	259.9	13.0	10.2
14	32 41 13	117 15 15	254.3	3.9	3.1
15	32 41 16	117 15 30	258.7	7.4	5.7
16	32 41 21	117 15 19	258.9	4.5	3.4
17	32 41 28	117 15 29	260.7	5.3	4.1
18	32 41 40	117 15 35	260.7	4.9	3.8
19	32 41 44	117 15 39	264.3	6.2	4.8
20	32 41 48	117 15 46	258.8	9.9	7.7

POINT LOMA WAVE REFRACTION



PERIOD = 20.0 s DIRECTION = 285.0° H_o = 2.6 m

Wave Breaking Data

ray	latitude	longitude	angle	depth(m)	height(m)
1	32 40 0	117 14 49	253.1	4.5	3.5
2	32 40 9	117 15 5	255.7	6.0	4.7
3	32 40 14	117 14 55	257.3	5.7	4.3
4	32 40 19	117 15 0	259.6	5.6	4.4
5	32 40 18	117 15 21	269.0	11.3	8.8
6	32 40 29	117 15 15	263.4	6.7	5.2
7	32 40 46	117 15 16	259.0	5.7	4.5
8	32 41 1	117 15 20	258.0	5.0	3.9
9	32 40 57	117 16 47	276.0	55.4	*****
10	32 40 59	117 15 23	266.8	6.2	5.0
11	32 41 5	117 15 27	264.1	7.0	5.4
12	32 41 13	117 15 15	260.6	4.4	3.5
13	32 41 20	117 15 16	259.7	3.3	2.6
14	32 41 35	117 15 41	265.3	9.9	7.6
15	32 41 36	117 16 6	269.5	12.6	10.1
16	32 41 38	117 15 33	264.7	5.2	4.1
17	32 41 54	117 15 32	264.6	4.4	3.5
18	32 42 3	117 15 48	266.5	10.3	8.0
19	32 42 7	117 15 39	269.3	6.9	5.4
20	32 42 13	117 15 35	267.7	5.2	4.0

APPENDIX C

SHORELINE EROSION

APPENDIX C

SHORELINE EROSION

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APPENDIX C

SHORELINE EROSION

1.0 INTRODUCTION

This appendix documents the technical approach to estimation of the 50-year and 75-year bluff retreat lines drawn on Figure 1 in the main report (Site Plan and Geologic Map). Our evaluation is based on four types of analysis:

1. Estimation of the relative effectiveness of marine versus subaerial erosion;

2. Estimation of the amount of marine erosion that should be expected at the cliff-platform junction;

3. Estimation of the potential for collapse of overhangs and block fall along joints; and

4. Estimation of the amount of slope decline that may be expected for the cliff and bluff above the elevation of principal influence of marine erosion.

Some of the terminology used in this report may be unfamiliar to some readers. These terms are defined in the Glossary section of the main report.

1.1 Sea-Cliff Geomorphology

The geomorphology of a typical Point Loma sea cliff is shown in Figure C-1. The typical sea-cliff profile generally consists of a lower near-vertical cliff rising directly from the sea, an intermediate bluff or cliff at 42 to 80 degrees, and an upper bluff with moderate slopes. Little or no flat area is exposed above sea level at the base of the cliff, even at very low tides. The sea cliff is bounded at its landward edge by the coastal terrace, which may extend inland a considerable distance. Offshore from the cliff is an area of indefinite extent called the nearshore zone.

The sea bottom in the nearshore zone is a shore platform extending out to sea from the base of the cliff. Generally, such platforms vary from nearly flat to gradients of approximately 3 horizontal to 1 vertical. At the site, the gradient of the shore platform is approximately 50:1 or about 1.15 degrees.

The point at which the sea cliff and shore platform meet is called the cliff-platform junction. It is at this junction, and some distance above, where retreat of the base of the cliff occurs.

The breaking of waves defines the inshore and foreshore zones. The inshore zone starts offshore where the waves begin to break. This zone is highly variable with time because the point at which waves begin to break changes dramatically with changes in wave size and tidal level. During low tides, large waves will begin to break far out to sea. During high tide, waves may not break at all or they may break directly on the lower cliff. The foreshore represents that portion of the shore lying between the upper limit of wave wash at high tide and the ordinary low water mark. It is absent at the site.

1.2 Point Loma Sea Cliffs

The coastal bluffs at Point Loma were generally classified by Emery and Kuhn (1982) as type C(d) (Figure C-2). The letter "C" designates coastal bluffs having a resistant geologic formation at the bottom, and less resistant materials in the upper parts of the bluff. The relative effectiveness of marine erosion of the lower resistant formation compared to subaerial erosion of the upper bluff produces characteristic profiles. Rapid marine erosion compared to subaerial erosion produces a steep cliff whereas slow marine erosion produces a gently-sloping upper bluff. The letter "(d)" designates the case of comparatively slow marine erosion.

Local variations in geology at the Point Loma Treatment Plant have produced a range of profile types (Figure C-3). The geologic section includes three, instead of two, materials of varying erosion resistance. In the lower 10 to 20 feet of the sea cliff, the lower Point Loma Formation is highly resistant to erosion. The upper portion of the Point Loma Formation encountered between the elevations of about 20 and 60 feet, is intermediate in erosion resistance. The upper bluffs, which are comprised of the Bay Point Formation, typically found above elevation 56 to 60 feet, have relatively low erosion resistance. Exposure of these three materials to marine and subaerial erosion has produced different profiles for the headlands and coves.

The profile of the typical headland within the site fits classification "C(d)" for which marine erosion is generally somewhat slower than subaerial erosion. Along some of the headlands at the site, a notch has formed just above the contact of the lower, more erosion-resistant, portion of the Point Loma Formation because of a lens of locally more erodible upper Point Loma Formation. The profile of slope formers in the upper bluff indicate that marine erosion at the cliff-platform junction is slow enough to permit rather well developed slope decline to the observed gradients.

The profile of the typical cove is of type "C(a)", having steep cliffs up to about 90-feet high in all three geologic units. Undercutting by marine erosion at the base of the cliff is common. This profile indicates that the rate of marine erosion at the cliff-platform junction is much greater than the rate of subaerial erosion of the upper cliff and bluff (Figure C-2). The upper bluff tends to retreat by collapse of overhangs and block fall along steep joints in order to keep up with the marine erosion.

A variable length transition area exists between the headlands and coves reflective of both type "C(c)" and "C(b)" sea cliff profiles.

1.3 Components of Bluff-Top Retreat

Placement of facilities on the coastal terrace above the bluff must account for changes in the bluff that should be expected during the intended life of the facility. One approach has been to build as close to the bluff as desired, assuming that maintenance and repair can forestall loss of the facility. Another approach is to estimate the amount of bluff-top retreat that should be expected within the life of the facility and to build behind the influence of retreat. A combined approach may also be appropriate, wherein a facility is sited within the area expected to retreat and limited shoreline protection is provided. If the component processes of bluff-top retreat are understood, then selection and efficient design of the appropriate limited shoreline protection is possible.

In coastal engineering, the concept of intended lifetime of a facility has been replaced by required design periods set by regulatory agencies. The Corps of Engineers requires 50 years and the California Coastal Commission requires 75 years. These design periods approximate the useful life of most facilities.

Available compiled measurements of bluff-top retreat are too widely variable for use in engineering design. For cliff profiles similar to those in San Diego, the best estimates of retreat rates have been reported near Santa Barbara where cliff materials similar to those at the site experienced measured bluff recession rates of 1.87 to 12.14 inches per year (Norris, 1975). These rates were measured by comparing existing structures to the topography on plot plans filed for their building permits. For the 75-year period of interest, the indicated bluff-top retreat would be approximately 12 to 76 feet. Rates of up to 1.5 feet per year have been reported for Sunset Cliffs by the U.S. Army Corps of Engineers. At that rate, bluff-top retreat would be 112 feet in 75 years.

Large short-term variations in bluff-top retreat should be expected in steeper sea cliffs. Wherever the profile is steep enough that rock and slope stability is questionable, failures can cause an instantaneous retreat of many feet.

We have applied techniques of geomorphology to estimate rates of bluff-top retreat. This requires breaking the problem down into component processes, analyzing each component, identifying the interaction of the components, and evaluating each characteristic bluff profile for the site.

The component processes of bluff-top retreat operate on various parts of the bluff, the sea cliff, and the cliff-platform junction. The components are as follows:

1. Marine erosion at the cliff-platform junction;

2. Collapse of overhangs and block fall along joints, essentially a rock stability problem; and

3. Slope decline by subaerial erosion of the middle and upper bluffs.

The components interact in different ways on the various bluff profiles characteristic of headlands, coves and transition areas. In isolation, each component process would independently proceed to completion or to an asymptotic rate. For example, slope decline in the Bay Point Formation would eventually produce a slope somewhat flatter than the angle of repose in several million years. In reality, continued erosion at the base of the cliff keeps the cliff and bluff steep, at approximately the same profile. For this to occur, the separate components of bluff-top retreat must retain the same approximate balance over time. The process of bluff-top retreat is further complicated by the presence of existing shoreline protection which may be only partially adequate. In general, this tends to mitigate the marine erosion component.

2.0 MARINE EROSION AT THE CLIFF-PLATFORM JUNCTION

Retreat rate at the cliff-platform junction is variable. One generally uniform rate appears to affect the sea cliffs, another rate affects the cove, and a third and more variable rate affects the transitional areas. Moreover, the rate in the coves appears to be variable from cove to cove and may also vary widely over time. These differences in characteristic rate between the cliffs and coves requires separate evaluation.

The cliff-platform junction contribution to retreat of the sea cliff is from marine erosion, which includes mechanical, chemical, and biological erosion processes. Marine erosion operates horizontally on the cliff as far up as the splash zone. It is accompanied by downwearing (marine erosion measured in a vertical direction) of the shoreline platform, which operates in a vertical direction. In general, backwearing (marine erosion measured in a horizontal direction) and downwearing progress at rates that will maintain the existing slope of the shoreline platform at approximately 50:1. This suggests that the rate of downwearing is approximately 2 percent of the rate of backwearing.

2.1 Effect of Water Depth, Wave Height, and Platform Slope

The key aspects of the sea-cliff profile for the marine erosion component of bluff-top retreat are the water depth at the base of the cliff, the breaking wave height and the slope of the shore platform. At the site, water depth at the cliff-platform junction averages 4 to 6 feet relative to mean sea level. This relatively deep water at the base of the sea cliff subjects the cliff to the attack of non-breaking, breaking, and broken waves that in turn control which mechanical erosion processes are active. Forces due to non-breaking waves are primarily hydrostatic. Broken and breaking waves exert an additional force due to the dynamic effects of turbulent water and the compression of entrapped air pockets.

Breaking waves exert a considerable added erosive force called "breaking wave shock" because of trapped air cushions in a near-vertical wave front. These shock or impact pressures result in relatively high pressure fields that last a few thousandths to a few hundredths of a second. These relatively short-duration impact pressures are of questionable importance in the design of vertical seawalls, however, when acting upon jointed and fractured rock, the water-hammer effect tends to cause hydraulic fracturing which exacerbates lower sea cliff erosion. Large sections of rock can be pried off by one well-placed wave. Erosion associated with breaking waves is most active when water depths at the cliff-platform junction (ds) coincide with the respective critical incoming wave height (H) such that $ds \simeq 1.3H$.

Waves will break when their height reaches approximately 75 percent of the water depth, thus 3 to 4.5 foot wave heights, will break at the base of the sea cliff in the Plant vicinity when tides are at mean sea level. Moreover, since the waves reaching the coast are generally in the range of 2 to 5 feet, breaking waves should be expected to occur at the base of the sea cliff usually four times a day (due to semidiurnal tidal fluctuations).

The slope of the shore platform is typically 50:1 or about 1.15 degrees. Whenever wave height and water depth are sufficient to produce breakers some distance offshore from the cliff, the very gradual slope will influence the breaker to form broken waves with high turbulence. The broken waves may reform as smaller non-breaking waves. Moreover, the smaller non-breaking waves may, in turn, reform as small breakers in a repetition of the process. When waves break and reform, considerable wave energy is lost to drag on the shore platform; consequently, less erosive energy is delivered to the cliff-platform junction.

2.2 <u>Erosion Processes</u>

The types of erosion affecting the typical Point Loma profile will change with the tidal level. In addition, any local variation that changes the average water depth will significantly alter the local balance of erosive forces.

Mechanical erosion processes at the cliff-platform junction include water abrasion, rock abrasion, cavitation, water hammer, air compression in joints, breaking-wave shock, and alternation of hydrostatic pressure with the waves and tides. All of these processes are active in backwearing. Downwearing processes include all but breaking-wave shock. Backwearing and downwearing by the mechanical processes described above are both augmented by bioerosion. Bioerosion is the removal of rock by the direct action of organisms. Backwearing at the site is assisted by algae in the intertidal and splash zones and by rock-boring mollusks in the tidal range. Algae and associated small organisms bore into rock up to several millimeters. Mollusks may bore several centimeters into the rock. Both chemical and salt weathering also contribute to the erosion process.

2.3 Rate of Marine Erosion of the Sea Cliff

The general rate of marine erosion at the cliff-platform junction is the result of the combined effect of mechanical erosion and bioerosion. Reported total rates for sedimentary rock coasts vary from less than 10 mm/yr for hard-rock coasts, to 2000 mm/yr for weak sedimentary rocks such as mudstones and siltstones. The Point Loma Formation at the cliff-platform junction is in the hard-rock part of this range of rock types.

In San Diego County, rates of marine erosion have been measured for the somewhat less resistant sedimentary rocks present at the cliff-platform junction north of Point Loma. The average rate obtained was approximately 10 mm/yr during a five-year period, from 1970 to 1975 (Lee, Pinckney, and Bemis, 1976), of mild winters with few major storms and only one episode of extreme wave activity (see Appendix B). More typically, a five-year period would include three or four extreme-wave episodes suggesting a proportional increase in erosion.

The rate of bioerosion has been estimated for downwearing to be 0.6 mm/yr for sandstones in Southern California (North, 1954). No estimates are available for the horizontal component of bio-erosion.

For headlands, we have chosen to use a preliminary estimated rate of one-half inch per year (12.7 mm/yr) for marine erosion at the cliff-platform junction. This estimate is based on consideration of the worldwide data, local measurements, variations in rock type, and the long-term storm record. Beginning with the measured rate for the San Diego coast of 10 mm/yr, the one-half inch per year rate was estimated assuming an increased wave environment to be approximately balanced by more erosion-resistant rock of the lower Point Loma Formation. At this rate, approximately 3 feet of marine erosion should be expected to occur in 75 years at the base of the sea cliffs.

Shoreline erosion of the headlands was not determinable through review of historical photographs dating back to 1939. That is to say, the rate of erosion was too slow to detect any measurable rate of retreat, thereby corroborating the preliminary estimate of 1/2 inch per year.

2.4 Cove Erosion

2.4.1 Typical Profile

The typical cove profile is shown on Figure C-4. The cove is eroded back into the normal sea-cliff profile (shown in the background). The sea floor in the cove is generally incised by a 5 to 20± foot wide surge channel which extends seaward beyond the cliff front. The surge channels in the site vicinity appear to be about 2 to 5 feet deeper than the adjacent shore platform. In the cove, the bottom of the surge channel likely extends inland at a gradient similar to the shore platform at about 50:1 or 1.15 degrees.

2.4.2 Controls on Cove Location

Erosion at the cliff-platform junction of the cove is in the Point Loma Formation. The Point Loma Formation is generally resistant to erosion, except at fractures and joints. Careful geologic mapping provided close correlation of faults, shear zones, and fractures, with associated increased rates of erosion of the cliff-platform junction in these areas. Thus, coves and other discontinuities exhibiting locally higher rates of marine erosion are joint controlled along this reach of coastline.

An additional factor controlling the location of coves is a difference in erodibility between the lower section of Point Loma Formation and the overlying parts of the unit. The lower section consists of comparatively thick-bedded sediments that are more resistant to erosion. The overlying section consists of comparatively thin-bedded sediments that have many more bedding planes and fractures on which wave action can operate. Because of this difference, the upper, laminated section of Point Loma Formation will erode more rapidly than the lower, more massive section (when exposed to marine erosion).

In general, the lower, more resistant section is exposed along the sea cliffs. As indicated on Figure C-4, the Point Loma Formation dips into the cliff at an average angle of about 8 degrees. As such, continued erosion exposes progressively more of the upper more erodible portion of the Point Loma Formation to marine erosion. Wherever long-term retreat has breached the more resistant rocks, accelerated cove growth will occur. The location of many of the wider coves along the Point Loma coast is likely to be at least partly controlled by this distribution of variably resistant Point Loma Formation. The maximum extent of indentation for any cove along the Point Loma coast is 300 feet and the average for large coves is approximately 200 feet.

2.4.3 Active Erosion Processes

Erosion processes in the coves are essentially the same as those along the sea cliff. Small differences arise because, under normal day-to-day sea conditions, the wave energy is occasionally sheltered somewhat by the adjacent headland, which often leads to a comparatively quiet water environment in the cove. Extreme wave episodes often arrive directly on the coast from within 30 degrees of perpendicular to the general shoreline (see Appendix B). The direct approach of extreme waves transmits high erosive energy into the cove for short periods of time. Moreover, local offshore sea floor bathymetry tends to focus wave energy into the coves.

2.4.4 Rate of Marine Erosion

The rate of marine erosion for the coves has been estimated as a reasonable multiple of the rate for the sea cliff along the main coastline alignment and from comparison of aerial photographs taken as early as 1939, with new photographs taken for this project. A lower limit was set based on minimum rates considered necessary to maintain the near-vertical upper bluffs of the overlying Bay Point Formation. This approach is further developed in Section 4.0.

Significant differences in erosion rates are evident between the sea cliff and the coves. This is in part due to the difference in lithology and intensity of jointing, the wave-direction dependence of transmitting erosive energy into the cove, and the energy-focusing effect of surge channels.

The less-resistant upper Point Loma Formation in the coves is judged to have the approximate erosion resistance of the younger, Tertiary-aged rocks in which direct measurements were made along the coast north of Point Loma. The rate for the sea cliff at the site was, in part, based on recognition of the more resistant nature of the lower section of Point Loma Formation. Locally, jointing and faulting further reduces erosion resistance near the apex of the cove. This leads to an upward adjustment in rate in the back of the cove.

The wave direction and extreme-wave episode dependence of the cove erosion may be summarized as the proportion of extreme wave episodes that arrive from within approximately 30 degrees of perpendicular to the coast. Thirty-seven of the 60 extreme wave episodes since 1905 (Appendix B) have arrived within this range. The rate for the sea cliff at the site was based, in part, on recognition that three to four times as many extreme wave episodes of greater intensity should normally occur in the five-year period over which measurements were made by Lee and others (1976). For the coves, only two of the four episodes would be likely to cause significant erosion by arriving from the proper direction. This leads to a downward adjustment in rate.

The focusing effect of surge channels increases the erosive effectiveness of waves arriving from the same, or approximately the same, direction. This leads to an upward adjustment in the rate.

Considering the above three factors, it seems reasonable that the rate of marine erosion in the coves would be four to eight times that of the sea cliff.

Aerial photographs taken at irregular intervals from as early as 1939 to the present were compared in estimating the rate of cliff retreat. Our interpretation of these photographs indicated bluff-top retreat rates varying from approximately 2½ to 4 inches per year. Review of aerial photographs further indicated that upper bluff retreat was primarily due to sloughing, whether by undermining or due to localized slope instability. Thus, the rate of marine erosion in the backs of coves is substantially faster than subaerial erosion and the development of a more stable (flatter) upper slope cannot be initiated due to the excessive rate of undermining that occurs in the backs of the coves.

We have chosen to use a rate of 4 inches per year for the cliff-platform junction contribution to bluff-top retreat in the back of coves. This is somewhat higher than a measurement reported by Kennedy in a sea cave at Sunset Cliffs of five inches between 1965 and 1973 (Kennedy, 1973). This period included several extreme wave episodes. The Point Loma Formation exposed at Sunset Cliffs is judged to be intermediate in erosion resistance between the more erodible sediments in coves at the plant and the resistant sedimentary rocks along the sea cliff at the Plant.

Marine erosion rates were also developed for the transition zones between headlands and the back of each cove. This was somewhat more subjective and, in part, a factor of current slope geometry. We have chosen a marine erosion rate of 1 inch per year for the median point of what we considered to be representative of the transition. However, this point is typically closer to the headland than the back of the cove, resulting in a retreat rate for most of the cove and transition zone similar to that of the back of the cove.

Historical photographs dating back to the early 1900's (primarily in the vicinity of Sunset Cliffs, approximately 1½ miles to the north) were also reviewed to further evaluate erosion rates in both the Point Loma and Bay Point Formations.

3.0 BLOCK FALL

3.1 Overhangs

The maximum extent of overhangs was estimated from topographic mapping, ground measurements where access was possible, and observations, from a distance, of inaccessible caves. Existing sea caves and overhangs are shown in Figure 1 as shaded areas along the base of the sea cliff. Where overhangs and sea caves are not filled, they may collapse. Although only a certain proportion of overhangs will fall in a given year, it is difficult to predict which overhang will fall. Therefore, a rate cannot be estimated. We have assumed that overhangs and sea caves that are left unfilled will collapse, except where specific analysis and thickness of roof rock has shown that collapse should not be expected within the 75-year period of interest.

3.2 Joint-Controlled Block Fall

Block fall along joints applies to the part of the lower sea cliff where joints and other rock discontinuities intersect the cliff face. This part of the cliff, consisting of Point Loma Formation, currently stands at slope angles of about 80 degrees, parallel to the primary joint set. Prediction of the actual stability along each joint intersecting the cliff was beyond the scope of our investigation. We have assumed that parts of the cliff will retreat by block fall to a slope angle of 80 degrees.

4.0 SLOPE DECLINE

The process of slope decline is illustrated in Figure C-5. The graph (from Wallace, 1977) shows the gradual decline of slope angle as the slope ages. The curve is for weakly indurated formations. The curve has two parts. The steeper section, representing more rapid decline from about 10 to 100 years of age, shows a decline from 60 degrees to about 35 degrees. The flatter section represents slower decline after 100 years. This data has been developed from an evaluation of fault scarps where estimates of age were available.

4.1 Bay Point Formation

The Bay Point Formation is weakly to moderately indurated. It stands at slopes varying from about 30.5 degrees (1.7:1, horizontal to vertical) at headlands, to about 80 degrees in coves. Using the graph in Figure C-5, the slopes should be expected to decline from 30.5 to 29.5 degrees at the headlands, from 45 to 35 degrees at the transition median point, and from 80 to 37 degrees at the back of coves.

In reality, the base of the slope in Bay Point Formation will be eroded back along with the top of the underlying formation.

4.2 Upper Point Loma Formation

The upper Point Loma Formation is moderately to strongly indurated. It stands at slopes varying from about 1.0:1.0 at headlands, to vertical or overhanging in coves. We constructed a new curve for the Point Loma Formation based on the underlying principles of slope decline represented in Wallace's graph. These slopes should be expected to decline from 45 to 44 degrees at the headlands, from 63.5 to 54 degrees at the transition median point, and from near vertical to 57 degrees at the back of coves.

4.3 Fill Slopes

Fill slopes, where not maintained, will also retreat, but should not decline according to Figure C-5. Most fill slopes at the site are protected by paving on the flat area above the slope. This protection is likely to cause the fill slope to retreat as a series of slumps from progressive toe failures due primarily to wave runup and subsequent progressive erosion of the toe of the fill slope.

5.0 EFFECT OF EXISTING SHORELINE PROTECTION

The presence of a stone revetment at the base of the coastal bluffs mitigates direct wave impact onto the bluffs. Although direct wave impact is reduced, wave runup and the attendant splash is likely increased, which contributes to both lower bluff and upper bluff erosion. Nevertheless, bluff-top retreat will continue as the upper parts of the cliff and bluff are affected by slope decline. Since construction of the Plant in 1963, there have been several periods of riprap placement with the net effect of further reducing coastal erosion. We have reviewed available stereographic aerial photographs from the following dates:

Stereographic Aerial Photographic Coverage

Date of Photograph	Photographic Scale
1987	1:12,000 (photographically enlarged to 1"=200')
1986	1:12,000
1985	1:40,000
1982	1:24,000
1981	1:24,000 (photographically enlarged to 1"=200')
1978	1:40,000 (photographically enlarged to 1"=200')
1972	1:20,000 (photographically enlarged to 1"=200')
1964	1:24,000 (photographically enlarged to 1"=200')
1960	1:24,000 (photographically enlarged to 1"=200')
1953	1:24,000 (photographically enlarged to 1"=200')
1950	1:24,000 (photographically enlarged to 1"=200')
1949	1:20,000
1939	1:24,000 (photographically enlarged to 1"=200')

Since erosion rate was, in part, evaluated by the cliff retreat rate based on successive photo evaluation and historical storm data (Table B-2, Appendix B) the effect of existing stone revetments on bluff retreat rate was evaluated based on the annualized reduction in retreat rate after placement of rock when compared to the pre-1963 data. This information has been incorporated into the evaluation of our 50 and 75-year bluff retreat lines shown on Figure 1 in the main report.

Existing shoreline protection consists of rock revetments and walls. The revetments mitigate or nearly eliminate marine erosion at the cliff-platform junction (within the 75-year period of interest).

Sections of the upper bluff with walls create a man-made bluff-top line at the top of the wall. This line will not retreat unless the wall fails. Some walls at the site are expected to fail by undermining and/or corrosion if additional protection is not provided. This has also been incorporated into the 50 and 75-year bluff retreat lines shown on Figure 1 of the main report.

6.0 BLUFF-TOP RETREAT LINES

Estimates of the 50 and 75-year bluff-top retreat lines, shown in Figure 1 of the main report, were made by analyzing the effect of each bluff-top retreat process on each of the typical bluff profiles present at the site. An example of the transition median point is shown in Figure C-6. The retreat lines estimated from the analysis of profiles are generally not as far from the present bluff as the 50 and 75-year lines shown in Figure 1. An additional margin has been added in some areas to account for the more limited data for modeling transition areas.

Special study of the sea cave under the engineering trailer has shown the roof to consist of thick competent rock. Therefore, this cave has not been assumed to collapse as have the other caves.

Using Figure C-6, the bluff profile after 75 years of erosion was estimated by the following six-step method:

1. The overhang was assumed to collapse, forming a vertical cliff.

2. The Bay Point slope declines to 35 degrees by the process of slope decline.

3. At the same time, the upper Point Loma Formation declines to 54 degrees. In so doing, the contact between upper Point Loma Formation and Bay Point Formation retreats.

4. The new profile on the Bay Point Formation is estimated by connecting the contact point determined in Step 3 with the retreat point determined in Step 2.

5. Marine erosion at the cliff-platform junction causes retreat along the contact between the lower and upper portions of the Point Loma Formation. The amount of erosion shown is approximately 6 feet.

6. The future profile of the upper Point Loma Formation is estimated by connecting the retreat point estimated in Step 5 with the point estimated in Step 3.

The method described above was applied to the typical Point Loma profiles for headlands, coves, and transition areas. In each case, the respective marine erosion rates and slope decline rates resulted in an estimated future profile similar to the present profile. The estimated similarity suggests that the estimated rates of marine and subaerial erosion are in a balance that will maintain the current profiles.

REFERENCES

Emery, K.O., and Kuhn, G.G., 1982, "Sea cliffs: their processes, profiles, and classifications," Geological Society of America Bulletin, v.93, pp. 644-654.

Lee, Louis, Pinckney, C.J., and Bemis, C., 1976, "Sea cliff base erosion, San Diego, California: American Society of Civil Engineers, National Water Resources and Ocean Engineering Conference, April 5-8, 1976, preprint 2708, pp. 1-13.

Norris, R.M., 1968, Sea cliff retreat near Santa Barbara, California: California Division of Mines and Geology, Mineral Information Service, v. 21, No. 6, p. 87-91.

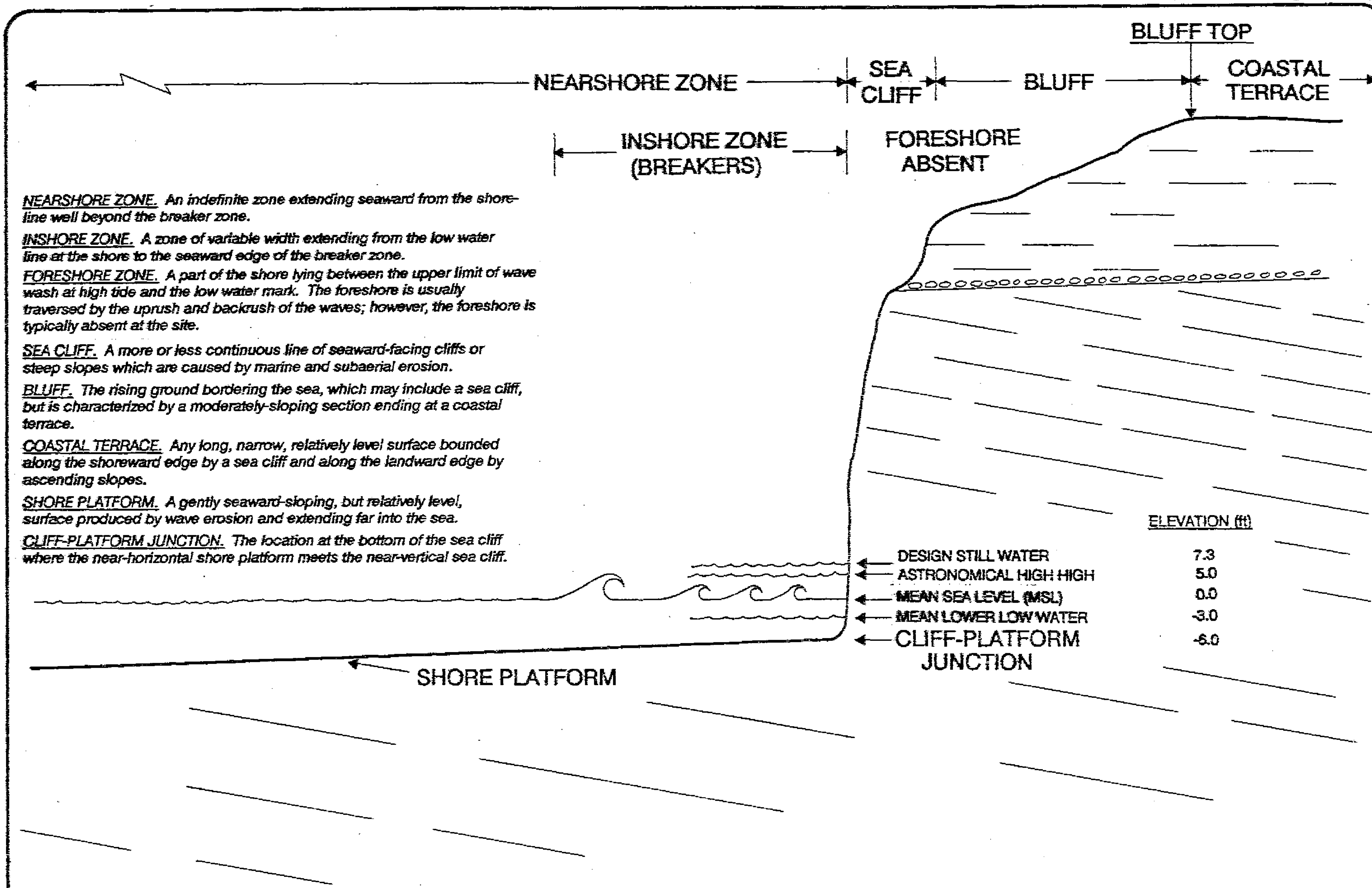
North, W.J., 1954, Size distribution, erosive activities, and gross matebolic efficiency of the marine intertidal snarls, Littorina planaxis and L. Scutulata: Biol. Bull. 106, p. 185-197.

Pease, R.C., 1979, Scarp degradation and fault history south of Carson City, Nevada: University of Nevada, Reno, Masters Thesis, 90 p.

Sanders, N.K., 1968, The development of the Tasmanian shore platforms, University of Tasmania, Ph.D. Thesis.

Trenhaile, Alan S., 1987, "The geomorphology of rock coasts," Clarendon Press, Oxford, 384 pp.

Wallace, R.E., 1977, Profiles and ages of young fault scarps, north-central Nevada: Geological Society of America Bulletin, v.88, p.1267-1281.



NEARSHORE ZONE. An indefinite zone extending seaward from the shoreline well beyond the breaker zone.

INSHORE ZONE. A zone of variable width extending from the low water line at the shore to the seaward edge of the breaker zone.

FORESHORE ZONE. A part of the shore lying between the upper limit of wave wash at high tide and the low water mark. The foreshore is usually traversed by the uprush and backrush of the waves; however, the foreshore is typically absent at the site.

SEA CLIFF. A more or less continuous line of seaward-facing cliffs or steep slopes which are caused by marine and subaerial erosion.

BLUFF. The rising ground bordering the sea, which may include a sea cliff, but is characterized by a moderately-sloping section ending at a coastal terrace.

COASTAL TERRACE. Any long, narrow, relatively level surface bounded along the shoreward edge by a sea cliff and along the landward edge by ascending slopes.

SHORE PLATFORM. A gently seaward-sloping, but relatively level, surface produced by wave erosion and extending far into the sea.

CLIFF-PLATFORM JUNCTION. The location at the bottom of the sea cliff where the near-horizontal shore platform meets the near-vertical sea cliff.

TYPICAL SEA CLIFF PROFILE (LOOKING NORTH UP THE COAST)

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure C-1

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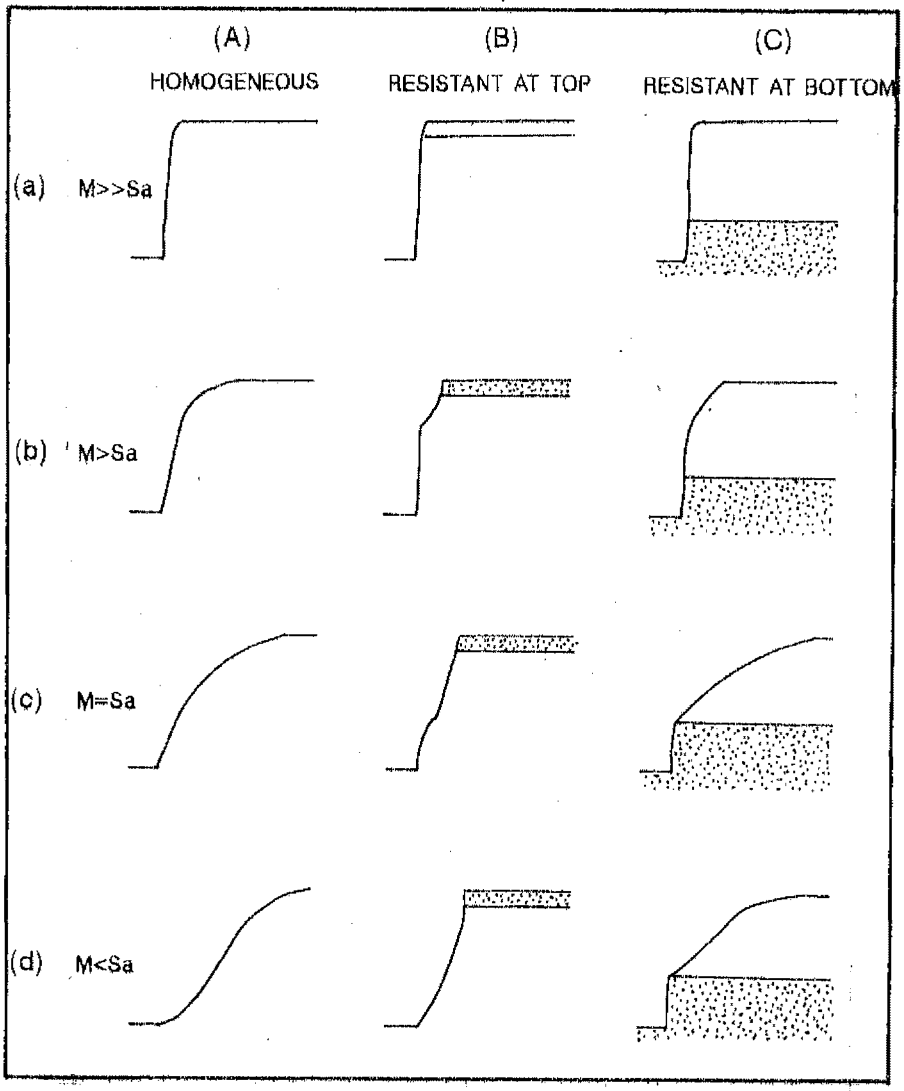

CLIFF PROFILES ACCORDING TO VARIATIONS IN ROCK RESISTANCE AND IN THE RELATIVE EFFICACY OF MARINE (M) AND SUBAERIAL (Sa) PROCESSES. THE MORE RESISTANT ROCK OUTCROPS ARE SHADED (AFTER EMERY AND KUHN 1982).

MATRIX OF ACTIVE SEA CLIFF PROFILES

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure C-2

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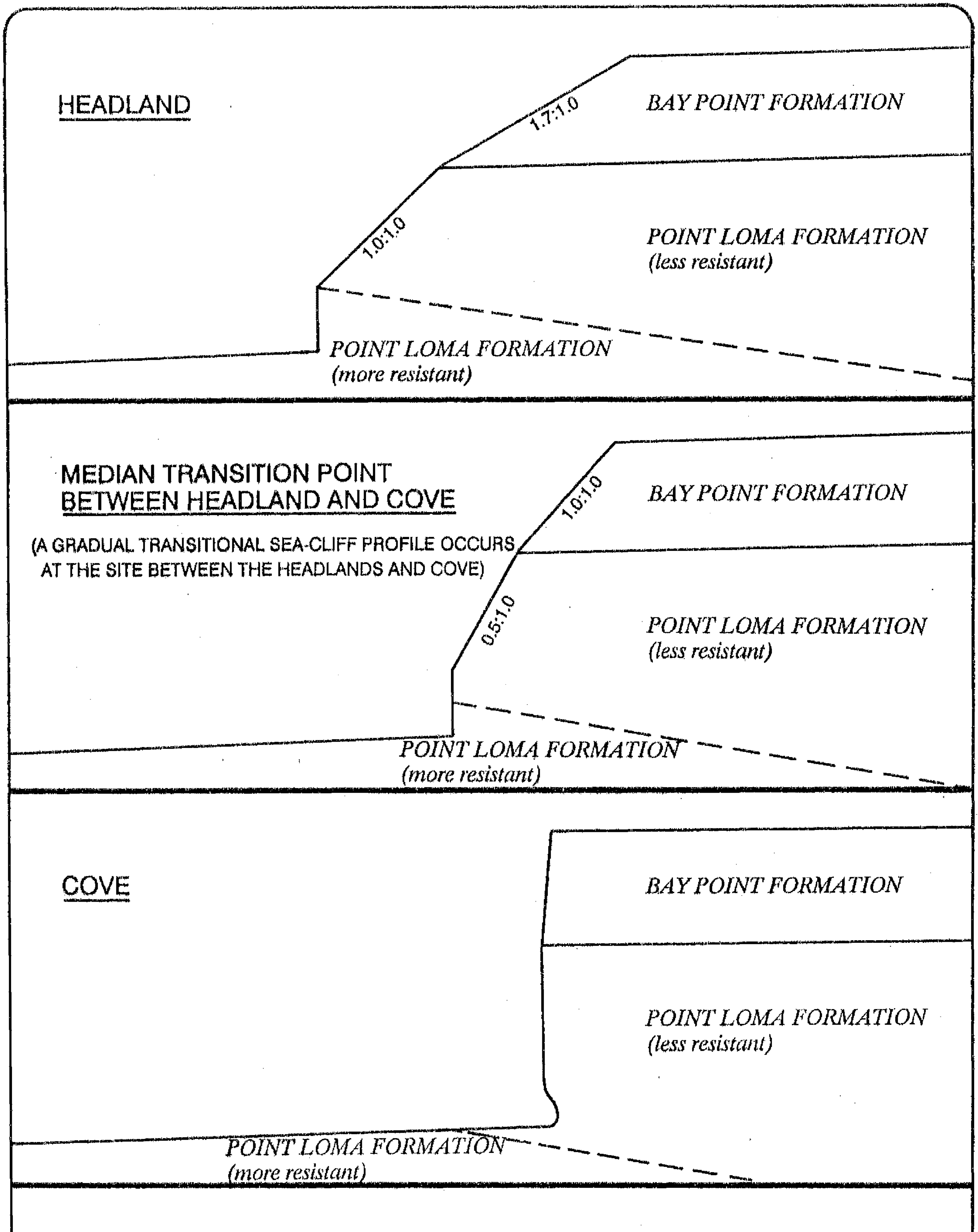

SEA CLIFF PROFILES FOR POINT LOMA

Project No.	POINT LOMA TREATMENT PLANT	Figure C-3

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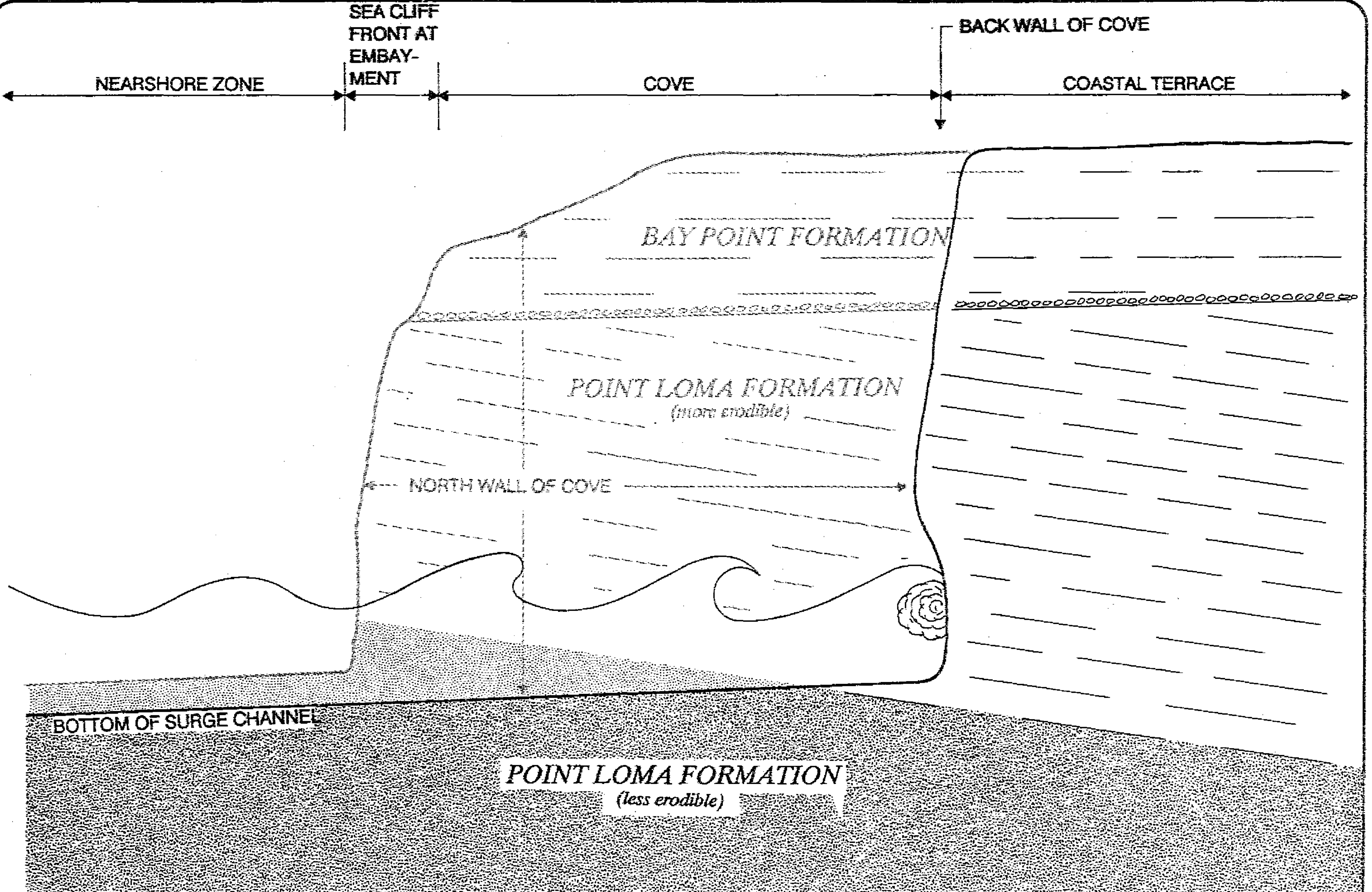

TYPICAL COVE PROFILE

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure C-4

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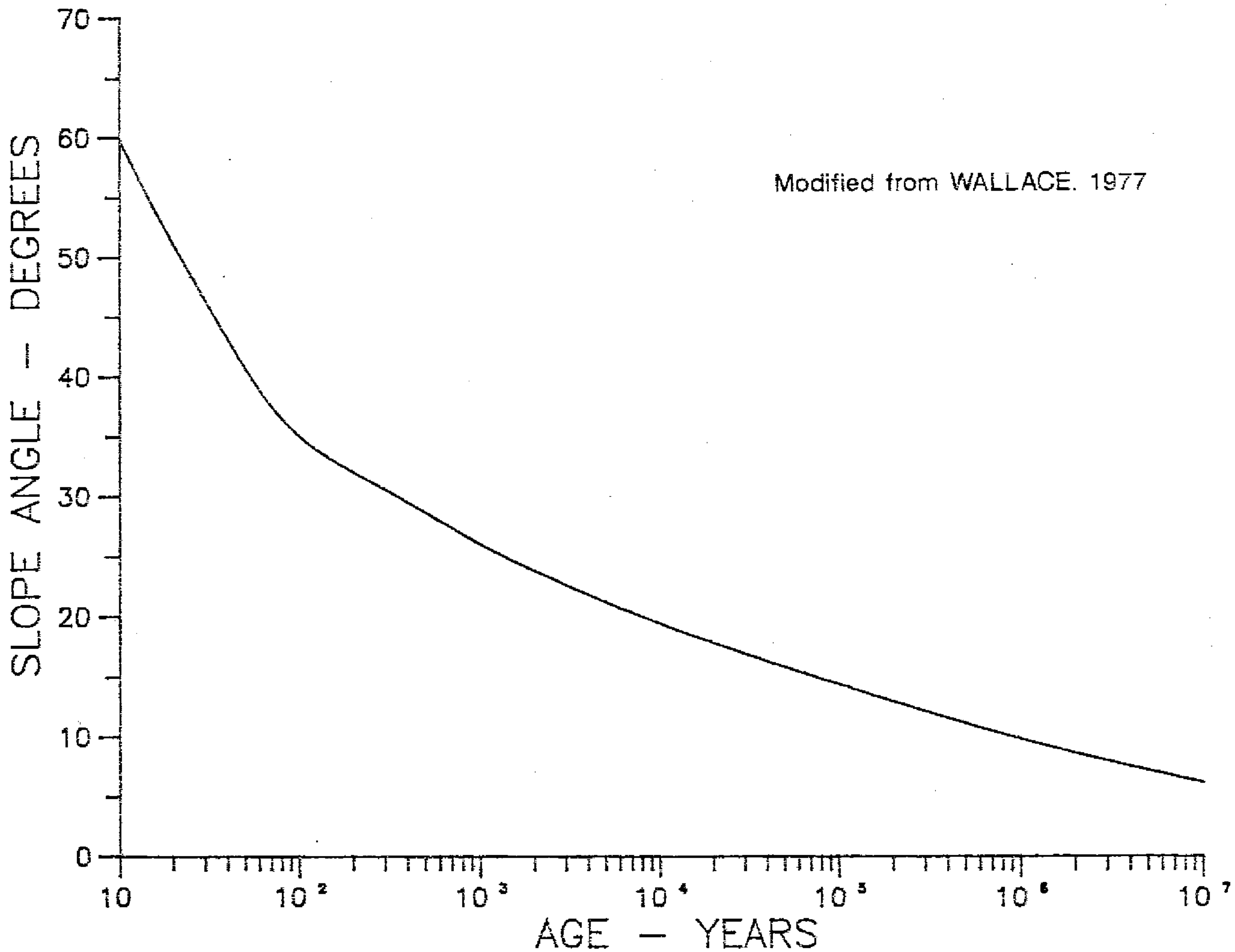

AGE-DEPENDENT SLOPE ANGLE FOR WEAKLY INDURATED FORMATIONS

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure C-5

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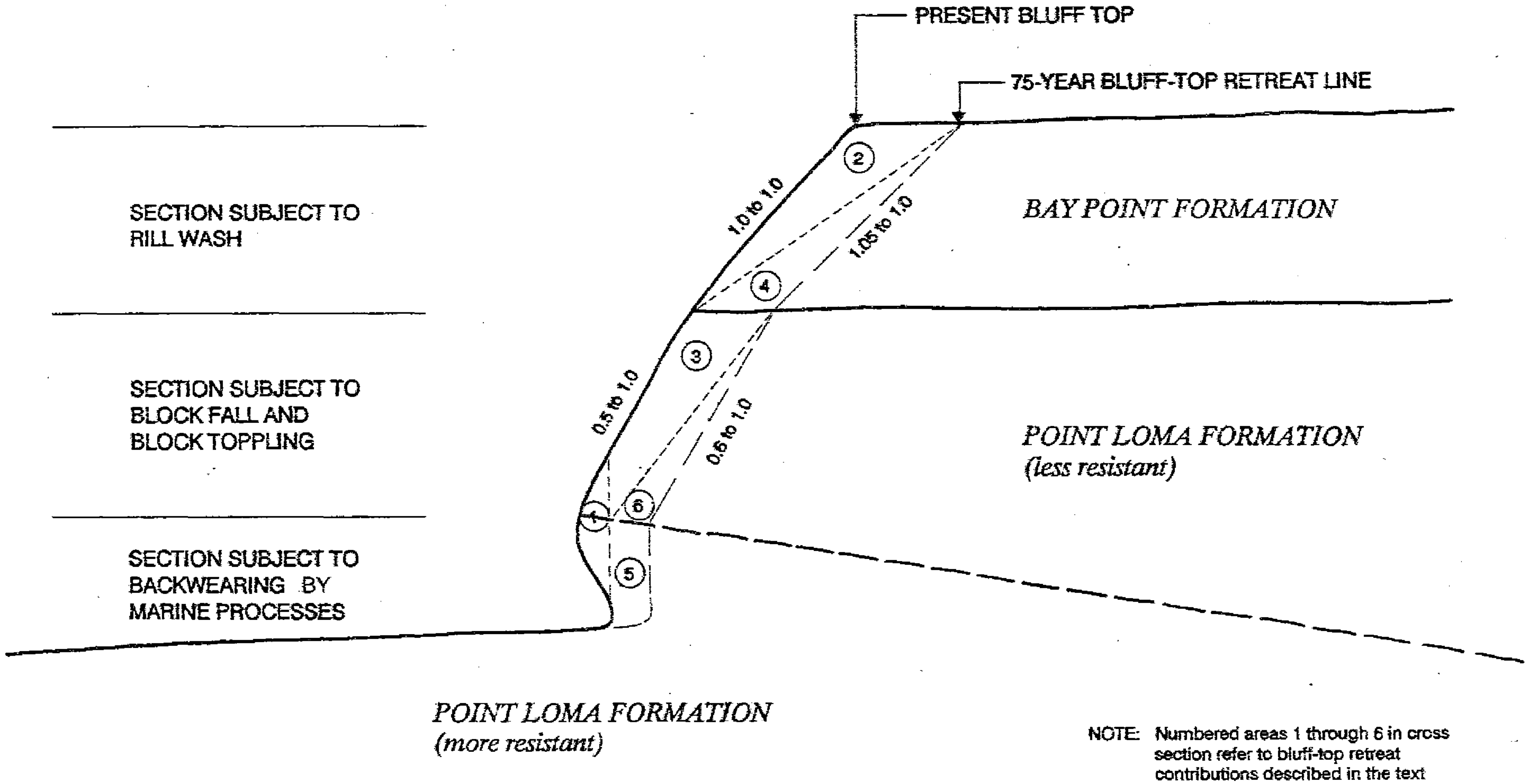


CONTRIBUTIONS TO BLUFF-TOP RETREAT

Project No. 1089-ES01	POINT LOMA TREATMENT PLANT	Figure C-6

GROUP DELTA CONSULTANTS, INC.

APPENDIX D

REINFORCED EARTH:
APPLICATION OF THEORY
AND RESEARCH TO PRACTICE

REINFORCED EARTH®: APPLICATION OF THEORY AND RESEARCH TO PRACTICE

David P. McKittrick*

This technical paper was presented as the keynote address at the Symposium on Soil Reinforcing and Stabilising Techniques sponsored by the New South Wales Institute of Technology and the University of New South Wales on October 16, 1978 in Sydney, Australia.

In the eleven years since the first commercial use of Reinforced Earth®, over 2,200 structures have been completed. These structures have included: retaining walls and bridge abutments for transportation applications; industrial structures including material processing and storage facilities; containment dikes for crude oil and liquified natural gas storage; and foundation slabs and hydraulic structures such as seawalls, flood protection structures, sedimentation basins and dams. Structures have been completed in all parts of the world in a variety of environmental settings; structures have been designed for, and been subjected to, a variety of loading conditions including static, moving and dynamic loads, thermal stresses and hydraulic and seepage forces. The performance of these structures has been closely monitored, either through gross observation or by precise instrumentation. This experience has provided the opportunity to critically examine theoretical and applied research, to compare predicted with actual performance, to refine design procedures, and to improve construction methods and technology to optimize economics. This paper reports on the present state of the art from a practicing engineer's viewpoint and proposes design procedures that are consistent with both basic soil and structural mechanics theory, as well as observed behavior of completed structures.

INTRODUCTION

In his "State-of-the-Art" address prepared for the American Society of Civil Engineers (A.S.C.E.) Symposium on Earth Reinforcement, our late colleague, Dr. Kenneth Lee (1), reviewed not only the papers that had been submitted for publication but also the status and results of research programs that he had actively directed, supported or reviewed during the several years that he and his associates had been involved in this topic. In his paper he listed several topics that he believed were in need of further study and advised caution in "drawing far reaching conclusions from limited basic research data" (this list is reproduced in Table I). A conscientious reader of his paper might, however, be somewhat puzzled by what would appear to be a contradiction contained therein; that is, his acknowledgment that "today (1978) the practice of using Reinforced Earth for appropriate geotechnical engineering projects is well established, and rational design procedures have been developed and demonstrated on many successful projects" and his admonition that " . . . the behavior of Reinforced Earth is actually very complex and . . . many more years will elapse before the basic mechanisms are clearly established to everyone's satisfaction." The same conscientious reader must then ask from what sources do practicing engineers derive the confidence and experience to design and construct civil and industrial works using this new material, knowing full well that the failure of these structures could imperil the public safety and cause significant economic disruption and monetary loss. These sources are, of course, the same theoretical and experimental studies known to and reviewed by Dr. Lee, augmented and interpreted in the light of experience with the design, construction and constant surveillance of actual Reinforced Earth structures. In this paper, I will attempt to re-examine those several topics cited by Dr. Lee in light of actual field experience in the United States and other countries, in an attempt to demonstrate that Reinforced Earth structures are designed on the basis of rational and usual engineering procedures and that, while certain behavior mechanisms may be complex, they still may be explained by basic soil mechanics theory and appropriately conservative parameters can be selected to account for these behavior mechanisms in the design of actual structures.

Proposed by Kenneth L. Lee, 1978

1. Sliding shear resistance between soil and reinforcing material
2. Fundamental behavior mechanisms and practical design parameters
3. Long term durability or corrosion of reinforcing materials
4. Backfill of cohesive soil or soil with fines

Table I: Reinforced Earth Topics for Further Study (Beginning with the most important)

Reflecting on the present state-of-the-art from a practitioner's standpoint and reviewing the amount of published data now available, I cannot disagree with the content of Dr. Lee's table. I would, however, rearrange and combine some of the topics. Table II contains a somewhat parallel listing of the factors or topics dealing with Reinforced Earth which are most important from a design and performance viewpoint. The order is derived not only from personel experience but also from the expressed concerns of the engineers with whom we deal on a daily basis.

*President, The Reinforced Earth Company, Washington, D.C.

Because they are interrelated, I have chosen to discuss items Ia and b and III in sequence. The durability question will be presented last, not because it is unimportant, but because it can be conveniently separated.

I.	Basic Mechanics of Reinforced Earth a. State of Stress in a Reinforced Earth Structure b. Frictional Relationship between Soil and Reinforcements
II.	Durability of Buried Metal Reinforcements
III.	Selection of Soil for Use in Reinforced Earth Structures

Table II: Topics of Major Importance to the Safety and Economy of Reinforced Earth Structures

In all project specific discussions of Reinforced Earth, the first question asked is "how does it work?". Having explained that the basic working mechanism depends on the efficient combination of metallic reinforcements and granular soil, the engineer's concern immediately shifts to the durability or service life question because it appears that most engineers, either through their work or educational experience, have concluded that metal buried in the earth will corrode in a time period inversely proportional to their years of experience. Engineers believe they understand the concept of friction and they have apparently decided that the complexities in data published by the researchers are the result of bad testing. They seem less interested in the selection of the backfill, believing this question to be one merely of economic concern. Before reviewing the status of research on the listed topics, it is important to understand the basic mechanics of the material under consideration.

I. Basic Mechanics of Reinforced Earth

The basic mechanics of Reinforced Earth were well understood by Vidal and were explained in detail in his early publications. A simplification of these basic mechanics can be illustrated by Figure 1. As shown in Figure 1a, an axial load on a sample of granular material will result in lateral expansion in dense materials. Because of dilation, the lateral strain is more than one-half the axial strain. However, if inextensible horizontal

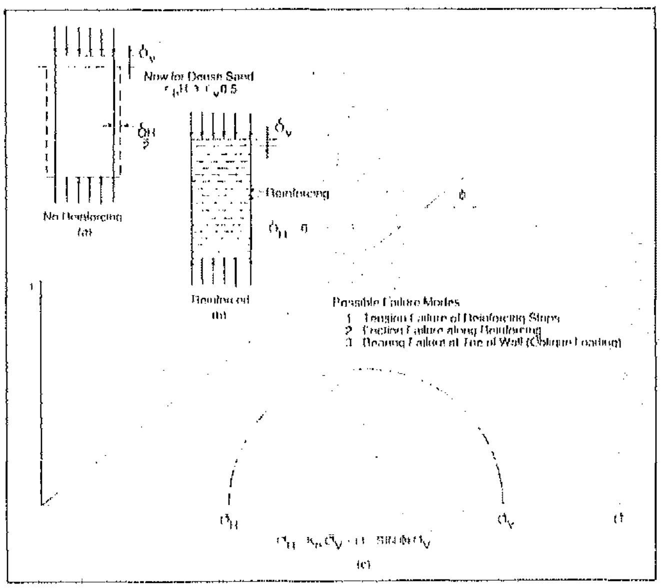


Fig 1: State of Stress in Reinforced Earth

reinforcing elements are placed within the soil mass, as shown in Figure 1b, these reinforcements will prevent lateral strain because of friction between the reinforcing elements and the soil, and the behavior will be as if a lateral restraining force or load had been imposed on the element. This equivalent lateral load on the soil element is equal to the earth pressure at rest ($K_o\sigma_v$). Each element of the soil mass is acted upon by a lateral stress equal $K_o\sigma_v$. Therefore, as the vertical stresses increase, the horizontal restraining stresses or lateral forces also increase in direct proportion. Thus, for any value of the angle of internal friction, ϕ, normally associated with granular soils, the stress circle lies well below the rupture curve at all points. Failure can occur only by loss of friction between the soil and the reinforcements, or by tensile failure of the reinforcements. This fundamental principle was examined and confirmed by Schlosser and Longe (2), Hausman (3) and others. Theoretical relationships were developed between the spacing and tensile resistance of the reinforcements and the increase in "anisotropic pseudo-cohesion" of the reinforced materials. Finding conclusions from this earlier research restrictive of wider applications of earth reinforcing, Bassett and Last (4) have further investigated this concept with analyses of a non-cohesive soil reinforced with a uni-directional reinforcement system subjected to plane strain. Using a Mohr circle of strain rate, Figure 2a, the investigators have determined the direction of the major and minor principle strains, ϵ_1 and ϵ_3, and also the direction of the zero strain planes, α and β, which define an arc segment containing the minor principle strain direction ϵ_3, within which all normal strains would be tensile and reinforcement horizontal in line with the maximum principle tensile strain. This direction is used in actual Reinforced Earth retaining walls. Figure 3b shows the effect on the same strain fields and potential failure planes when reinforcements are inserted in the soil matrix in a direction parallel to ϵ_3. Since the modulus of the reinforcing material is generally very much greater than that of the soil and as efficient frictional bonding occurs between the soil matrix and the reinforcements, the direction of the reinforcement must be aligned with one of the zero extension characteristics. Referring to Figures 2b and 3a, the β characteristics of a composite material would be rotated to become very nearly horizontal and the α characteristics are forced to follow. The potential rupture or failure mechanism would also attempt to re-align with these new characteristics. Such a re-alignment is in substantial conformity to the locus of maximum tensile strains measured in several full-scale structures, Schlosser et al (5), Figure 23. Vidal assumed that this composite material could be used to construct a coherent gravity structure, and that the properties of the structure would be similar to that of the theoretical and experimental models. Certainly, empirical adjustments would have to be made to account for horizontal and vertical discontinuity of the reinforcements. Adjustments would also be made for boundary conditions at the facing of the structure, point and magnitude of applied loading, foundation conditions, thrust of the backfill and other project-specific conditions.

In most research programs involving a study of the mechanics of materials, a fundamental behavior or failure mechanism is assumed. All studies thereafter build from that initial hypothesis. Experiments are designed to examine properties or individual components of the material, and very often many years are spent trying to rationalize and modify data to fit the initial assumption. Such has apparently been, and unfortunately continues to be, the case with Reinforced Earth. Many researchers have embarked on extensive research programs using the hypothesis that Reinforced Earth structures, in particular retaining structures, are analogous to tied-back walls. The literature on Reinforced Earth is replete with references to "tie-forces," Rankine failure planes, and other topics relevant to the analysis and design of anchor systems. These conscientious investigators have apparently neglected or misunderstood the basic mechanics of the material or the significant and substantial documentation that has existed for several years that should, when seriously considered in the light of actual structural performance, eliminate the tie-back or anchor approach as a conceivable failure mechanism. Before investigating structural behavior in more detail, I believe it is useful to compare the two hypotheses, i.e., composite material and tie-back, to determine if, in fact, they are so different.

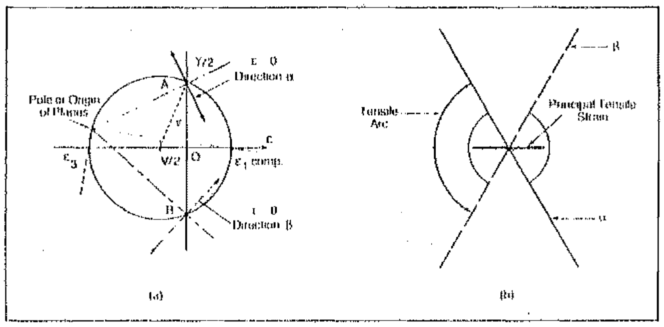


Fig 2: Mohr Circle of Strain Rate

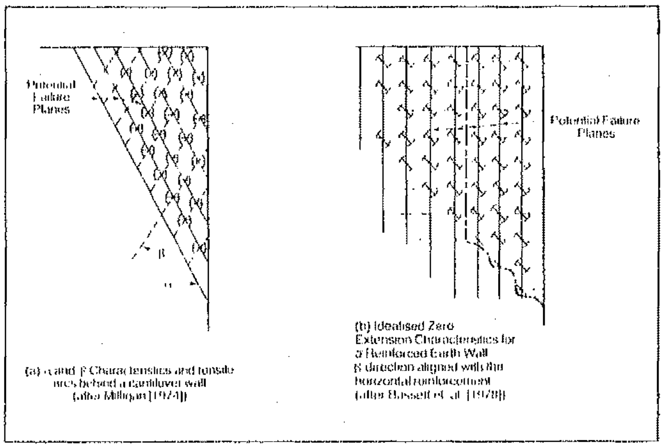
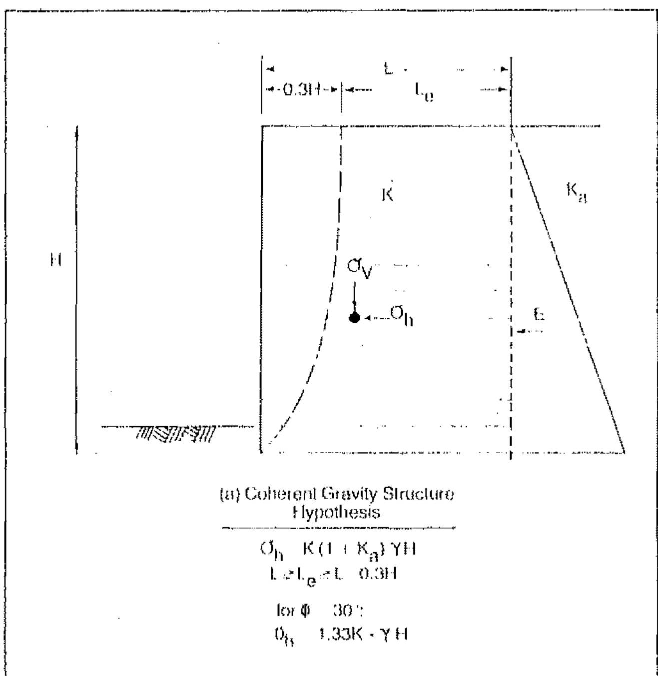


Fig 3: Strain Field Orientation-Cantilever and Reinforced Earth Retaining Walls



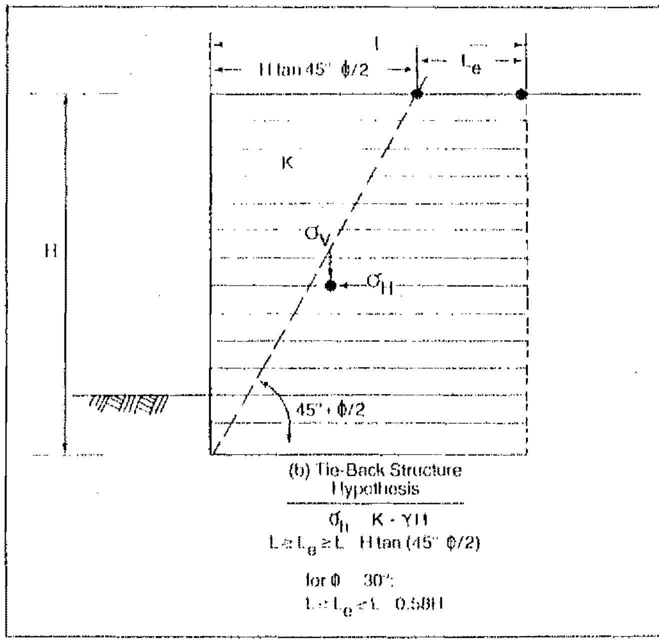


Fig 4: Design Hypotheses for Reinforced Earth Walls

Figures 4a and b demonstrate the significant differences in fundamental dimensions and stresses which can be obtained by use of the design procedures ordinarily used for the two basic mechanism theories. (The derivation of the design procedure and dimensions from the "coherent gravity structure" hypothesis will be developed later in this paper.)

For the same earth pressure coefficients, the use of Vidal's approach would require more reinforcements to resist the higher calculated stresses; but the required length of the reinforcements would be shorter than that calculated by the anchor theory. Referring briefly to a published case study reported by Al-Hussaini (6), Figure 5 demonstrated that use of the "coherent gravity structure" analysis would have predicted a conservative and safe design. It would not have been necessary to rely on empirically adjusted values.

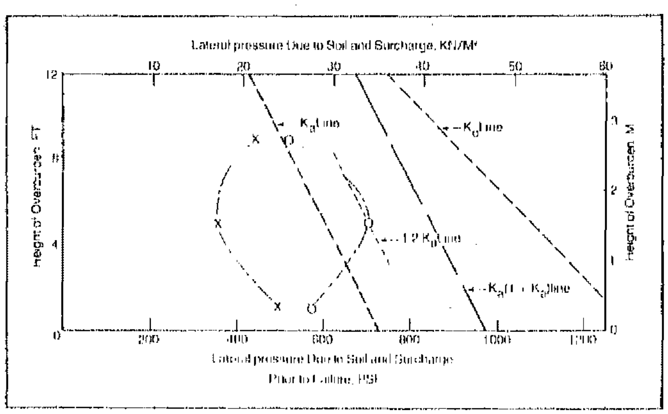


Fig 5: Measured Lateral Pressures W.E.S. Wall (after Al-Hussaini, 1976)

With the potential for calculating divergent answers to the important questions which govern safety and economics, it is important to determine what hypothesis can be supported by field experience. Unfortunately, we have convincing answers to this query. Field experience strongly supports the coherent gravity structure theory, as demonstrated by gross foundation failures under structures at Aguadilla, Puerto Rico, and Roseburg, Oregon.

At Aguadilla a nine-meter high retaining structure, presumably founded on rock, was instead constructed on a compacted structural fill placed on a clay foundation. Unknown to The Reinforced Earth Company and in violation of the specifications of the Puerto Rico Department of Public Works, the foundation had not been benched and so it sloped down and away from the rear of the structure. The backfill used to construct the structural fill and wall was a clean, uniform beach sand and the fill within the wall was compacted by ponding. As the structure neared completion, the reduced shear strength of the saturated and presumably remolded clay foundation was not sufficient to resist the mobilizing force of the mass, and the structure moved outward as a unit approximately two meters. The wall face remained essentially vertical during wall movement and no structural distress was evident in the precast concrete panels. The structure remained coherent, and it was possible to dissassemble the structure and salvage all wall panels without danger to the workers. The initial and final locations of the structure are shown in Figure 6. Photographs taken before and after sliding occurred are shown in Figures 7 and 8.

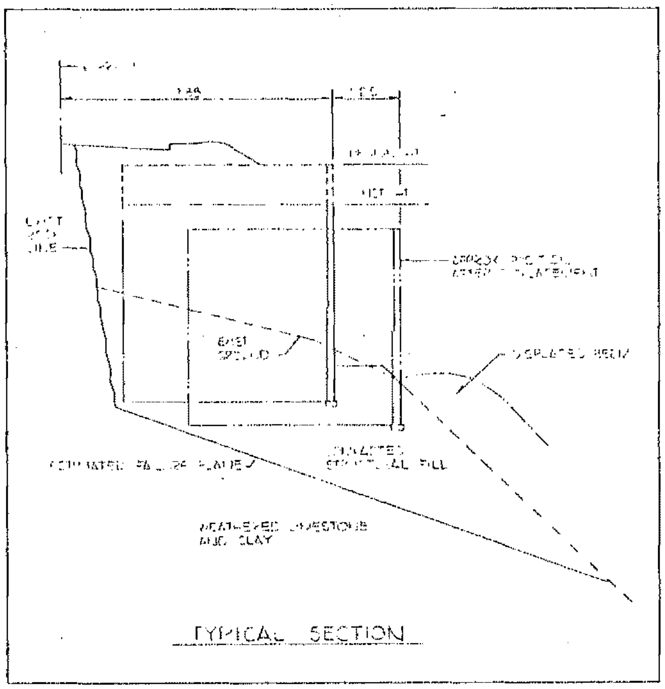


Fig 6: Movement of Structure at Aguadilla, P.R.

In Roseburg, Oregon, a 10-meter high Reinforced Earth retaining wall had been constructed to reduce the amount of fill required for a highway embankment and to prevent encroachment of the embankment on a river which paralleled the highway. A cross-section showing the wall and its relative position in the embankment is shown in Figure 9a and b. As the embankment above the wall was nearing completion, a slide occurred at the wall location. The slide failure plane, which was positively identified through continuous sampling, and by the use of inclinometers, passed behind and beneath the structure. The top of the Reinforced Earth wall was displaced seven meters horizontally and the wall dropped approximately 3.7 meters vertically. The final location of the wall and the location of the slide plane are shown in Figure 10. Remedial measures are also shown. In spite of these large movements, the structure remained intact as shown in Figures 11a, b and c. Subsequent analysis revealed that the slide was caused by overstressing the weak foundation soils located approximately six meters below the base of the wall.

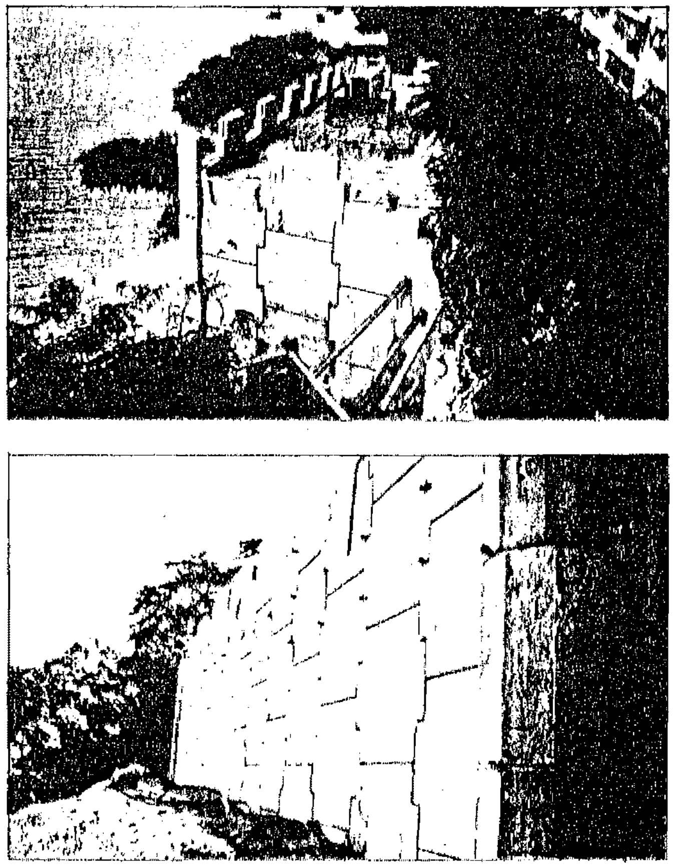


Fig 7: Aguadilla Structure Before Movement

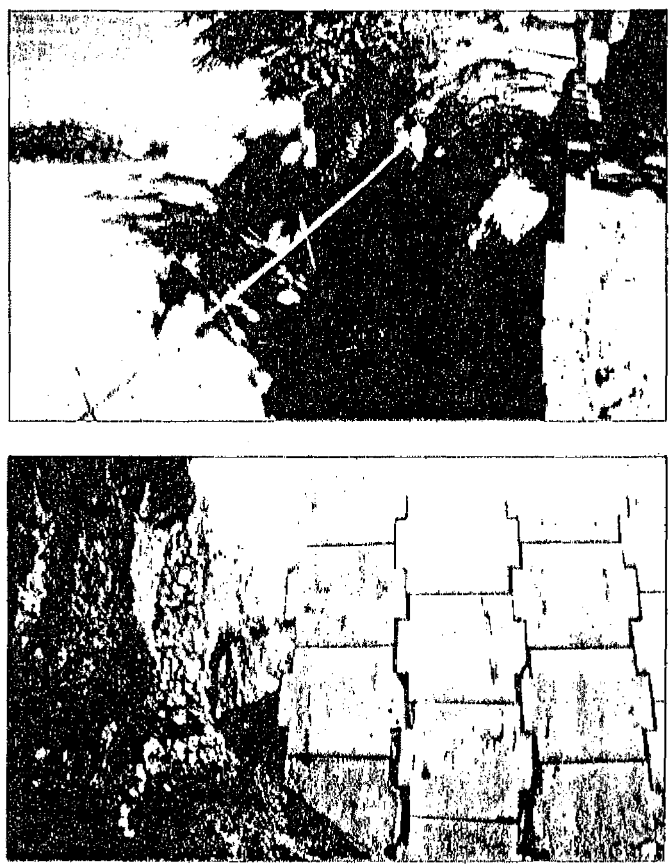


Fig 8: Aguadilla Structure After Movement

In its report on the Roseburg landslide, the Federal Highway Administration (7) concludes, "... it should be emphasized that the subject problem was a landslide problem and not a Reinforced Earth wall failure. Unfortunate as it was, the slide did provide a dramatic full-scale test of a Reinforced Earth wall (the first we know of in the world) and demonstrated (1) the internal strength of a Reinforced Earth structure and (2) that a Reinforced Earth wall does, in fact, perform as a (coherent) gravity structure."

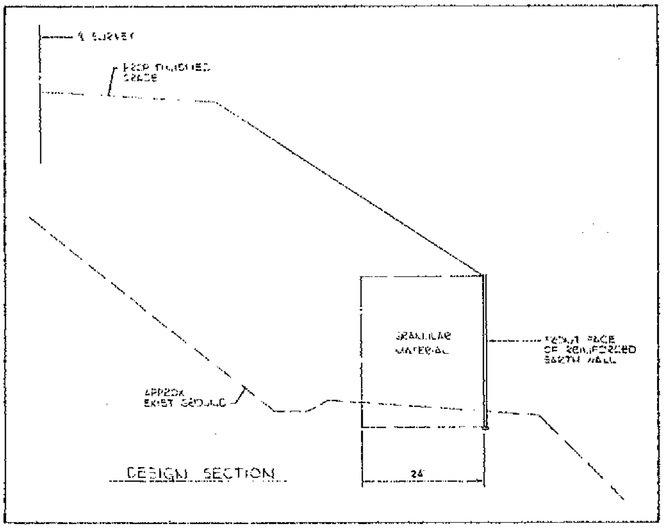


Fig 9: Position of Reinforced Earth Wall Before Slope Failure

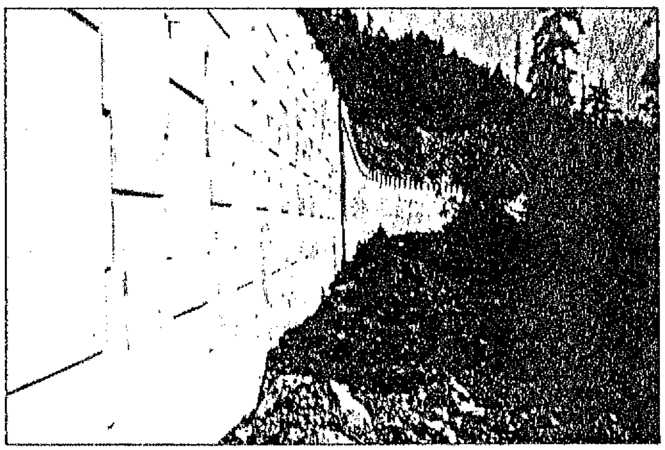


Fig 9: Roseburg Structure before Movement

In other cases, when Reinforced Earth structures have experienced important and expected settlements, careful measurements have confirmed that the reinforced volumes reacted as coherent masses.

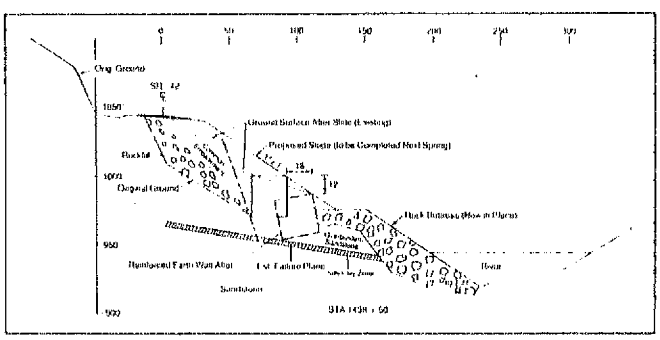


Fig 10: Roseburg-Coos Bay Highway Oregon Route 42

In light of substantial theoretical analyses supported by actual field performance, it seems reasonable to adopt the coherent gravity structure theses as a basis of design and to re-examine experimental data derived or interpreted on the basis of a "tie-back" or anchor wall failure mechanism.

(a) State of Stress in a Reinforced Earth Structure

The essential calculation in designing Reinforced Earth structures is the calculation determining the lateral or tensile stresses which must be resisted by the reinforcements. Overstress could promote tensile failure of the reinforcement which in turn would produce a catastrophic structural collapse. The calculation regarding the sliding shear resistance between the soil and reinforcements is less critical since slippage will cause only re-distribution of stress and a slow deformation of the mass.

Instrumentation of Reinforced Earth structures has shown that the state of stress within these structures varies and cannot be consistently predicted using, for example, a single earth pressure assumption adjusted as required for the effect of the thrust of the backfill. Schlosser (8) has previously reported in this conference a summary of earth pressures calculated from strain gauge measurements made in actual structures. This data, which is repeated in Figure 12 and is consistent with Vidal's early qualitative observations, as shown in Figure 13, can be explained by the relationship between the critical void ratio and applied stress. Studies such as those by Castro (9) have shown that the critical void ratio decreases with increasing stress. Accordingly, relative extension of the soil compared with the reinforcing strips becomes less for higher walls with their corresponding higher stresses. Thus, for higher structures the effective lateral stress is reduced and approaches an active state.

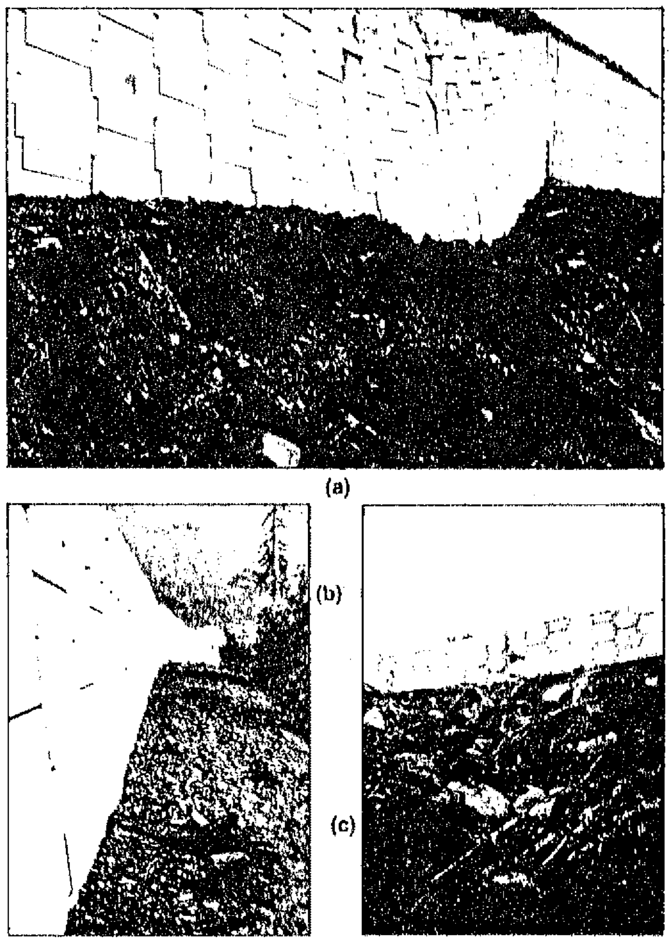


Fig 11: Roseburg Structure after Movement

The effect of the Factor of Safety in designs is to move the Mohr circle away from the failure envelope, in effect designing for a coefficient of earth pressure K, greater than the active coefficient K_a.

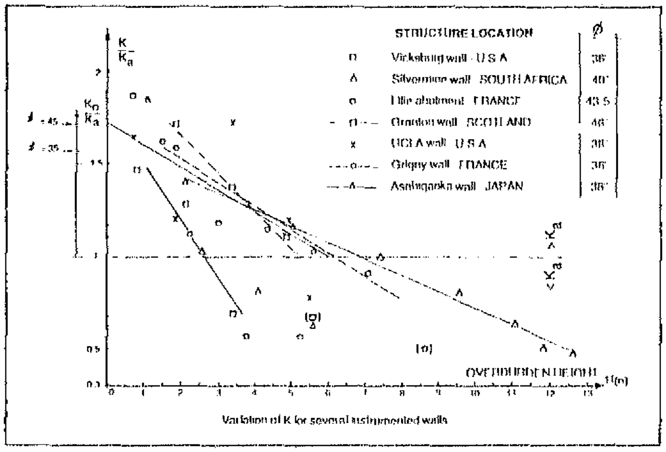


Fig 12: Earth Pressure Variation in Reinforced Earth Structures

This phenomenon was reproduced experimentally by Hausmann (10) and he reports similar observations during measurements of full-scale structures at Dunkirk, Thionville, and Granton.

The empirical distribution shown in Figure 14 has been developed to conform to observed stress distributions in Reinforced Earth structures and is consistent with results of theoretical analyses.

(b) Frictional Relationship Between Soil and Reinforcement

Having determined appropriate conservative values for the horizontal stresses in a Reinforced Earth mass and proportioned the cross-section and horizontal and vertical spacing of the reinforcements therein, the designer must satisfy himself that the horizontal stress can be effectively and efficiently transferred to these reinforcements. The designer must also be able to predict, within certain limits, the margin of safety available in the completed structure.

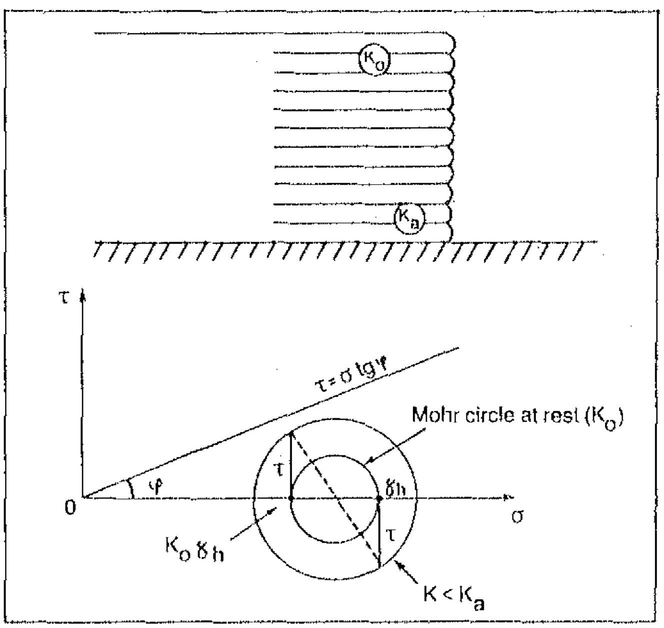


Fig 13: State of Stress in a Reinforced Earth Wall (after Vidal, 1969)

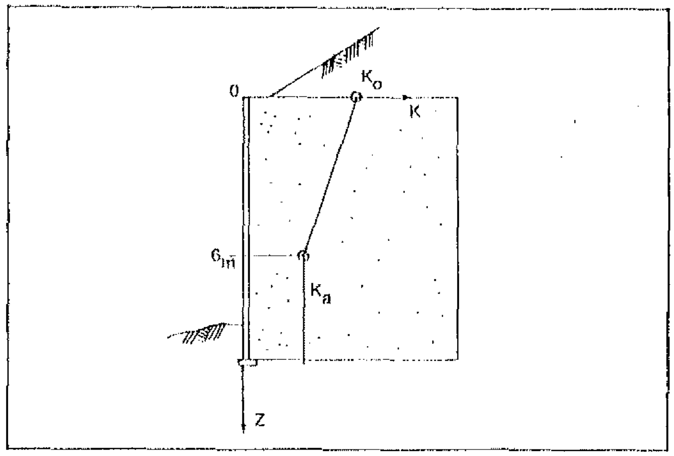


Fig 14: Earth Pressure Distribution in a Reinforced Earth Structure

Equations governing the frictional relationship are presented in Figure 15.

(1) $$FS_{Bond} = \frac{2bf^* L_e \sigma_v}{(K \cdot \sigma_v)\, \overline{S_x S_y}}$$

Which can be rewritten as follows:

(2) $$FS_{Bond} = \frac{2bf^* L_e}{K \cdot \lambda}$$

Where:

b = width of reinforcement

f^* = apparent coefficient of friction

L_e = length of reinforcement effective in stress transfer

K = earth pressure coefficient

$S_x \cdot S_y$ = influence area of reinforcement

Fig 15

The solution of equation (2) in Figure 15 requires not only knowledge of the geometry of the structure, but also the selection of appropriate values for the apparent friction coefficient, f^*, the effective length of the reinforcement, L_e, and the earth pressure coefficient, K. Let us examine first the apparent friction coefficient.

(b-1) Apparent Friction Coefficient, f^*.

The topic of sliding shear resistance between the soil and reinforcements has been the subject of numerous research studies in several countries. These studies have produced abundant data that on first examination are difficult to explain but will, after more detailed scrutiny, generally yield to the usual concepts of the shear strength properties of granular materials and sliding friction between materials. Several types of tests have been used to measure the value of f^*. These include:

(1) Direct shear (sliding shear) tests between soil and reinforcing material-model and prototype scale.

(2) Reinforcing strip pullout from a Reinforced Earth wall-model, prototype and full scale.

(3) Reinforcing strip pullout tests from embankments.

(4) Reinforcing strip pullout tests from a rigid moving wall-model scale.

(5) Reinforcing strip pullout tests during vibration-model scale.

Of all the testing procedures used, the direct or sliding shear test is the one most available to practicing engineers for the evaluation of design parameters. Other testing procedures require more specialized equipment, and generally involve higher cost which may not be justified by either the size of the project or the economic gain that may result from more refined and extensive data. From a designer's standpoint, therefore, it is important to know if direct shear test results can be used with reliability. Let us first generally examine the results of the other testing programs. In the following discussions, the terms "apparent friction coefficient," "pullout resistance," and "shearing resistance" will be used interchangeably to describe the frictional bond between soil and reinforcement.

Reinforcing strip pullout tests in the laboratory and in full-size structures have shown the peak and residual shearing resistances to be dependent on the density of the soil, the effective overburden pressure and the geometry and surface roughness of the reinforcements. Some typical results from field pullout tests are shown in Figures 16 through 19a, b and c.

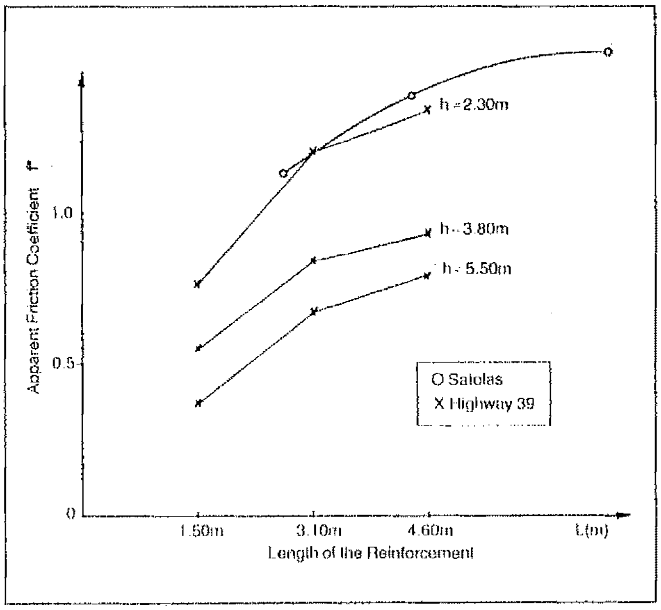


Fig 16: Pull-out Tests: Apparent Friction Coefficient (Influence of the Length of the Reinforcement)

Data from field pullout tests have shown the shearing resistance developed by reinforcements to be directly proportional to length. Results from both the Satolas (11) and Highway 39 (12) tests show that the apparent coefficient of friction reaches a maximum value at a strip length of about eight meters. For longer lengths, strips experience ductile flow and, therefore, the testing procedure is no longer relevant to the determination of the sliding shear resistance.

The surface roughness has an obvious and long-understood effect on the sliding shear resistance. Schlosser and Vidal (13) reported results of direct shear tests performed on samples of leucate and calcareous sand sheared along, and in contact with, smooth and roughened reinforcements. The results of these tests are shown in Figure 20. Examination of the reinforcements after shearing revealed striations on the smooth strip oriented in the direction of the displacement. This is evidence that sliding of the soil particles along the strip had occurred. Examination of the roughened reinforcement did not reveal such striations, evidence that shearing had taken place along a soil-soil interface. Examination of the high adherence reinforcements now in use tested in prototype direct shear and full-scale field pullout tests reveals similar evidence. Thus, it is reasonable to assume that the use of reinforcements with appropriately designed surface roughness can result in an apparent friction coefficient approximately equal to the shear strength of the soil as determined by direct shear or sliding shear tests.

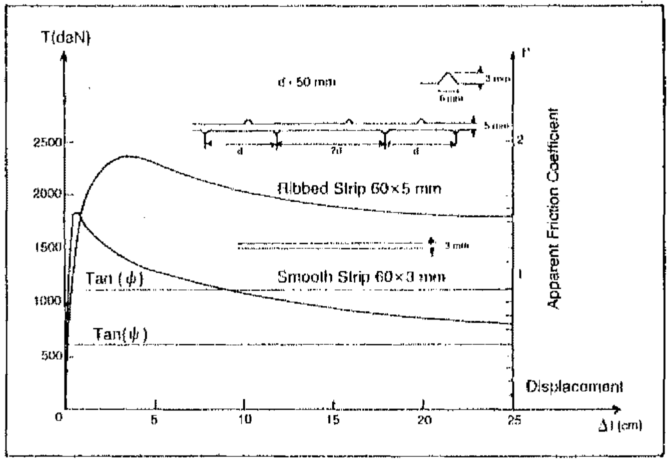


Fig 17: Pull-out Test in Reinforced Earth Walls (Influence of the Nature of the Strip Surface)

The relationships shown in Figure 19a, b and c are calculated using the expression $T = \sigma_v \tan \Psi \cdot 2bL$ to determine the apparent friction coefficient f^* (i.e., $f^* = \tan \Psi$). As shown in these Figures, frictional values exceed those which could be calculated using a value of Ψ or ϕ determined by direct shear testing procedures. Since these results could strongly influence overall safety and economy, it is important to determine if they are a function of the testing procedure or in fact represent a phenomenon which can be expected in the performance of actual structures. To more easily understand the influence of density and overburden pressure on a Reinforced Earth mass, let us first examine the influence of these parameters on the shear strength of a granular material. Figure 21 shows the effect of density on the stress-strain-volumetric relationships in granular soils tested under drained shear conditons. Dense sands exhibit a high peak deviator stress at low strains, and a residual deviator stress, less than the peak stress, at high strain. The dense material expands, or dilates, during shear. The loose sand exhibits a much lower deviator stress, no peak value, and a volumetric contraction or compression during shear. A further insight is gained by examining the same stress-strain-relationships during undrained shear where no volume changes are allowed to occur. This restriction of volumetric expansion is a condition like that which exists in an actual structure. The undrained relationships are shown in Figure 22. The negative pore pressures which are induced in a saturated sample during shear may be used qualitatively to estimate the apparent increase in overburden stress when volumetric expansion in unsaturated samples (or structures) is not allowed to occur (a localized

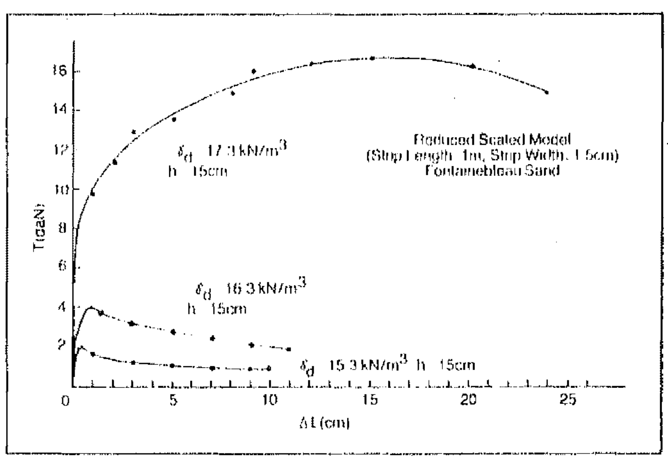


Fig 18: Influence of the Density in Pull-out Test

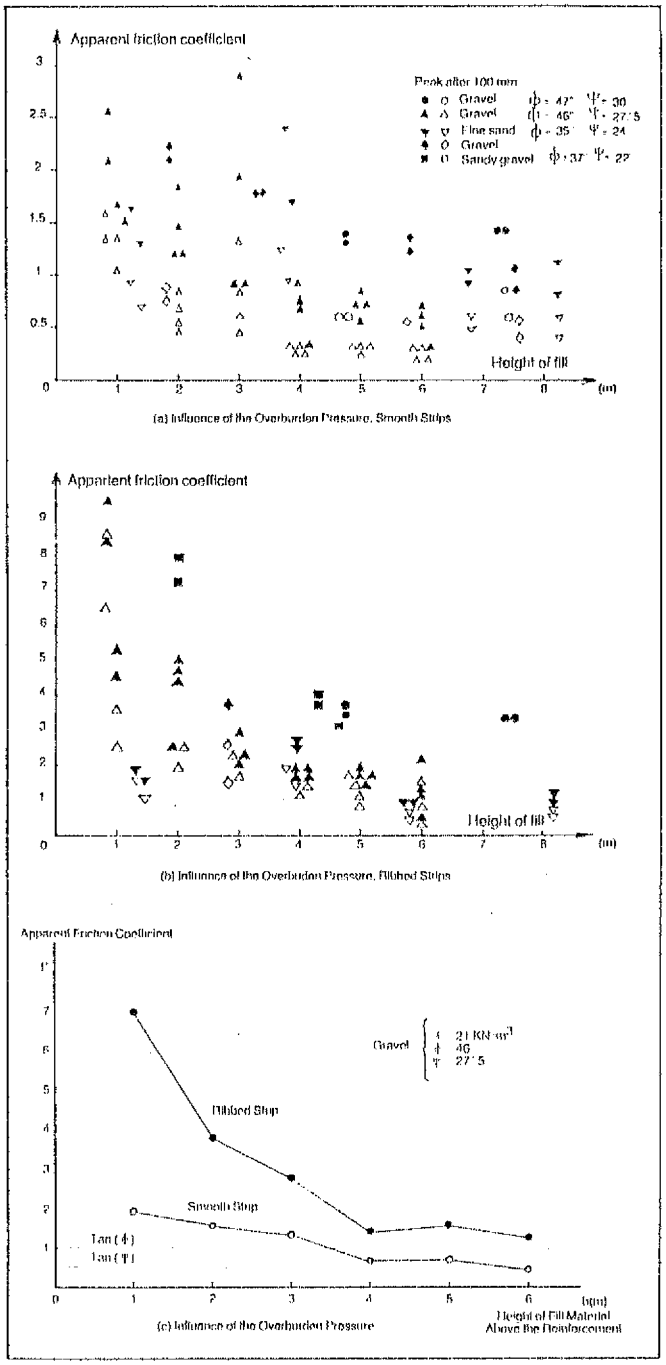


Fig 19: Pull-out Tests: Apparent Friction Coefficient

condition). Higher densities can, therefore, increase the normal stresses acting on the strip and the apparent coefficient of friction, at least in those ranges of overburden pressures where the soil will be dilatant (i.e., void ratios less than the critical void ratio). Test results which indicate that at high density the values of the apparent coefficient of friction are much greater than the value of tan Ψ, as determined in direct shear results on soil-strip interface, can therefore be rationalized by taking into effect a factor to allow for the dilatancy of the soil. (In recent tests where volumetric expansion was not allowed to occur during direct shear tests, calculated values of Ψ were 10-15 degrees higher than tests where free expansion could occur.)

An interrelated phenomenon is the effect of the overburden stress on the apparent friction coefficient. To gain insight into this, let us investigate the effect of normal or overburden stresses on the shearing resistance of granular soils.

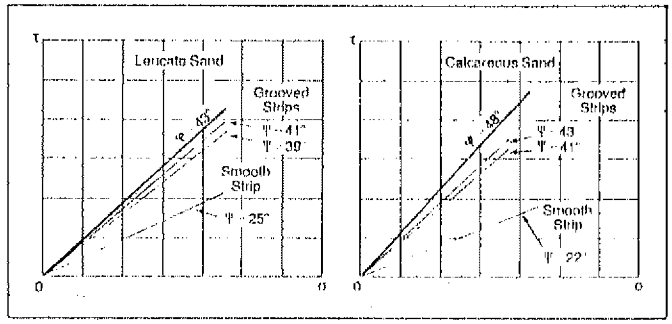


Fig 20: Coefficient of Earth-Strip Friction (f*=tg Ψ).

Figure 23 contains a display of peak stress ratios determined from 60 mm square direct shear box tests and 80 mm square plane strain compression samples compacted to a relative density of 0.70 (17.0kN/M²) (14). At small normal stresses (~20kN/M2) the peak angle of shearing resistance is approximately 50°, but this drops to 42° as the normal stress rises. Peak results from direct shear test and plane compression test results are significantly greater than 33°, the ultimate strength determined after the peak in direct shear tests and from loose slope angles (14). These results are consistent with the results of

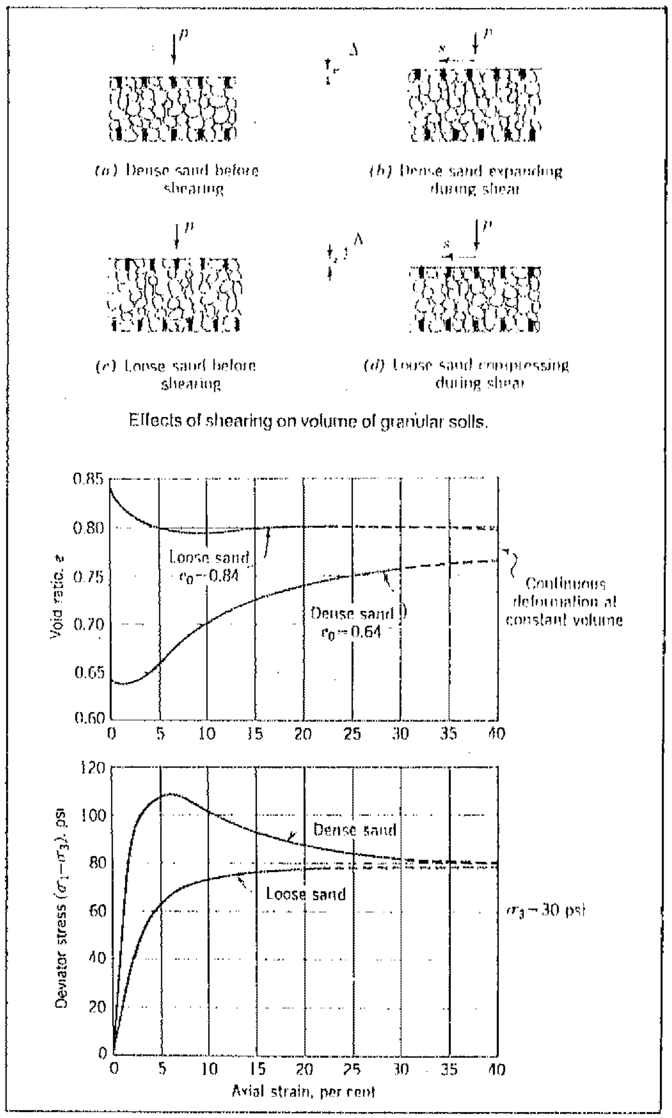


Fig 21: Effect of Density on Stress-Strain-Volume Relationships in Granular Soils. Drained Shear Tests. (after Leonards, 1962)

Ponce and Bell (15), who report an angle of shearing resistance increasing with the reduction of normal stress, and with Cornforth (16) who established that peak (strength) angles will exceed ultimate angles by up to 17° in a typical dense sand in plane strain.

As soils used in Reinforced Earth structures are (1) granular, (2) subjected to low strain levels, (3) normally stressed in plane strain (except under point loads) and (4) compacted to relatively high densities, it is reasonable to evaluate their static performance in a reinforced state using peak values determined in plane strain compression tests adjusted as required for the appropriate overburden conditions. Using such assumptions, it is possible to rationalize the values of f*, shown in Figure 19, with soil mechanics theory.

For example, Schlosser (8) reports the average of residual and peak values for f* equal to approximately 1.35 for pullout tests of smooth reinforcements in fine sand with an ultimate strength $\phi = 35°$. If we assume a 30 percent increase in normal stress due to the effect of restrained volumetric expansion, a value of 53° is calculated, a peak strength result consistent with Cornforth's observations.

The factors affecting sliding shear resistance are thus seen to be at least qualitatively consistent with basic soil mechanics theory. A knowledge of the density, shear strength, void ratio and strain state is, therefore, important in selecting the value of f* to use in design of actual structures. It is also important to consider the nature of the reinforcement surface.

For smooth reinforcements, the value of f* obtained in direct shear tests, selected at strain conditions consistent with anticipated structural performance, should be used. In most cases this value will be equal to the residual sliding shear value (tan ω).

For reinforcements with deformations or transverse ribs, such as those shown in Figure 24, values of f* consistent with soil parameters adjusted for the effects of plane compression, dilatancy, and overburden pressure can be used with confidence. In the case of smooth strips, the soil-strip friction characteristics will control behavior; in the case of ribbed or roughened strips, the soil-soil characteristics will most often control.

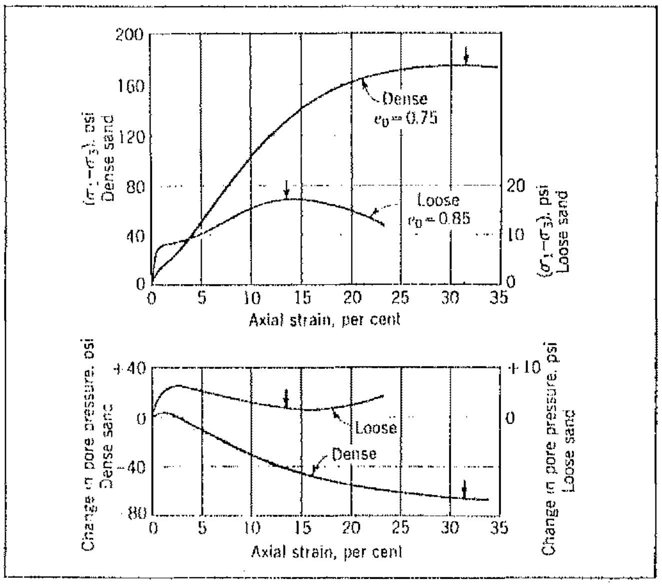


Fig 22: Undrained Shear Tests on Very Loose and on Moderately Dense Saturated Sand. (After Bishop and Henkel, 1957)

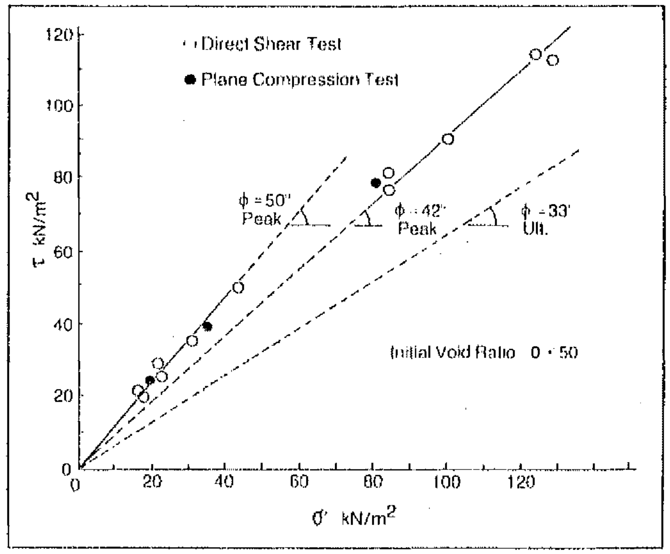


Fig 23: τ / σ on Plane of [τ / σ] Peak (Bolton et. al., 1978)

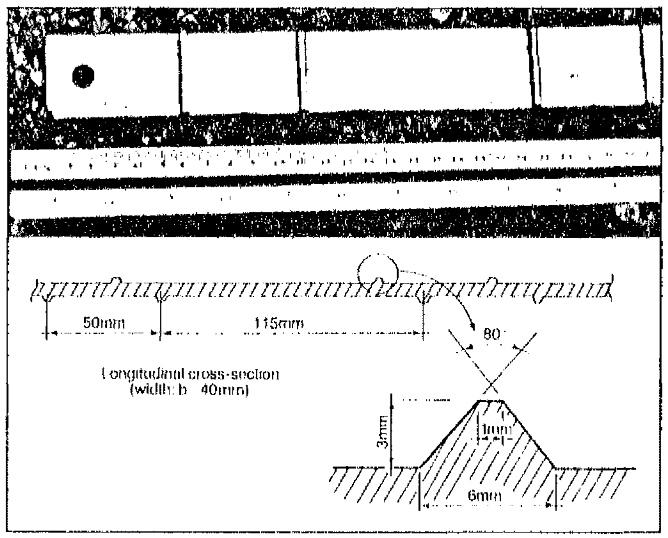


Fig 24: Ribbed Strips

(b-2) Effective Reinforcement Length.

Analysis of Reinforced Earth structures has shown that the tensile stress in the reinforcements of a structure is not maximum at the facing, but at a distance behind the facing. As shown in Figure 25, the locus of the points of maximum tension define two zones within the structure: an active zone in which the shearing stresses exerted by the earth on the reinforcement are directed outward, towards the facing; and a resistant zone in which the shearing stresses are directed towards the free end of the reinforcement. The boundary of this "active zone" varies with the type of structure, the foundation soil, and the location and magnitude of applied external loading. The boundary of the active zone, as determined by instrumentation of full-scale structures designed in accordance with working stress principles, as well as the boundary determined by theoretical procedures as previously reported by Schlosser (8) in this conference, can be enveloped by the straight lines shown in Figure 25.

This boundary is qualitatively consistent with hypothetical ideal distributions of shear stress along reinforcing strips presented by Hausmann (10). This graphical illustration is reproduced in an extended form in Figure 26. The ideal reinforcing case can be symbolically represented by Vidal's paper-reinforced rock pile, Figure 27. Here, all particles are in contact with the reinforcements and shear is a maximum at the center. In an actual Reinforced Earth wall, the discontinuity of the reinforcements, the soil loading imposed on the facing by the construction procedures, the stiffness of the facing, and other factors cause the redistribution of stress shown.

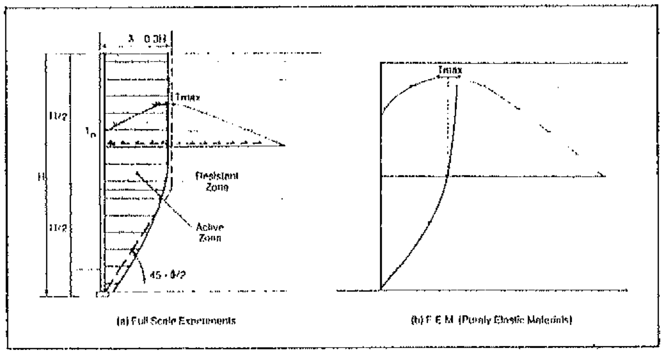


Fig 25: Tensile Forces Distribution Along the Reinforcements

(b-3) Design Procedures.

Based on these foregoing discussions, it is possible to formulate design procedures to adequately (and safely) proportion the reinforcement surface area. This procedure is shown schematically in Figure 28a and b. For granular backfills (compacted to at least 90 percent standard Proctor density) and ribbed reinforcements, the use of values for f* indicated in the figure can be supported by empirical data. However, one may also select a value of f* S= tan ϕ' (peak stress determined from plane compression tests) approximately adjusted for the effects of dilatancy. For nondilatant soils, a value for f* = tan Ψ (direct shear test value) should be used for both ribbed and smooth reinforcements. Appropriate safety factors should be used.

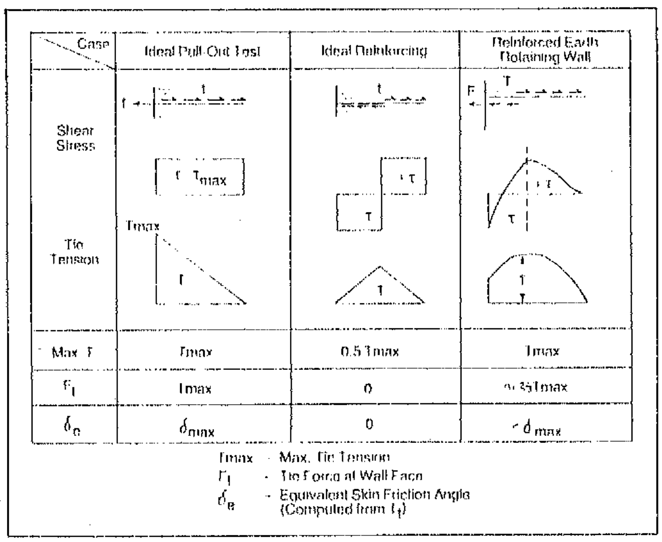


Fig 26: Hypothetical Ideal Distributions of Shear Stress Along Reinforcing Strip & Tie-Back (after Hausman, 1978)

II. Selection of Soils For Use in Reinforced Earth Construction.

Three principal considerations which influence the selection of soils for use in Reinforced Earth structures are:

(1) Long-term stability of the completed structure

(2) Short-term (or construction phase) stability

(3) Physiochemical properties of the materials.

It is evident from the previous discussions that granular soils compacted to densities that result in volumetric expansion during shear are ideally suited for use in Reinforced Earth structures. Where these soils are well-drained, effective normal stress transfer between the strips and soil backfill will be immediate as each lift of backfill is placed, and shear strength increase will not lag behind vertical loading. In the range of loading normally associated with Reinforced Earth structures, granular soils behave as elastic materials. Therefore, for structures designed at working stress levels, no post-construction movements associated with internal yielding or readjustments should be anticipated.

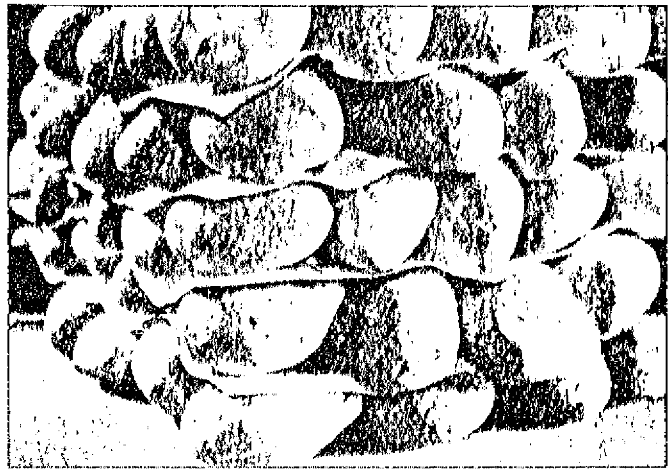


Fig 27: Idealized Reinforced Mass

On the other hand, fine-grained materials are not especialy suitable for Reinforced Earth structures. They are normally poorly drained, and effective stress transfer will not be immediate, thus requiring a greatly slowed construction schedule or an unacceptably low factor of safety in the construction phase. Fine-grained materials often exhibit elasto-plastic or plastic behavior, thereby increasing the possibility of post-contruction movements. In addition, if a significant portion of the strength of the fine-grained material is derived from its clay content, the rational design procedures used heretofore to evaluate the safety of the structure will not be applicable. With such sharply contrasting performance, it is necessary then to define clearly the boundary between granular and fine-grained soils, as this boundary or limit applies to Reinforced Earth construction.

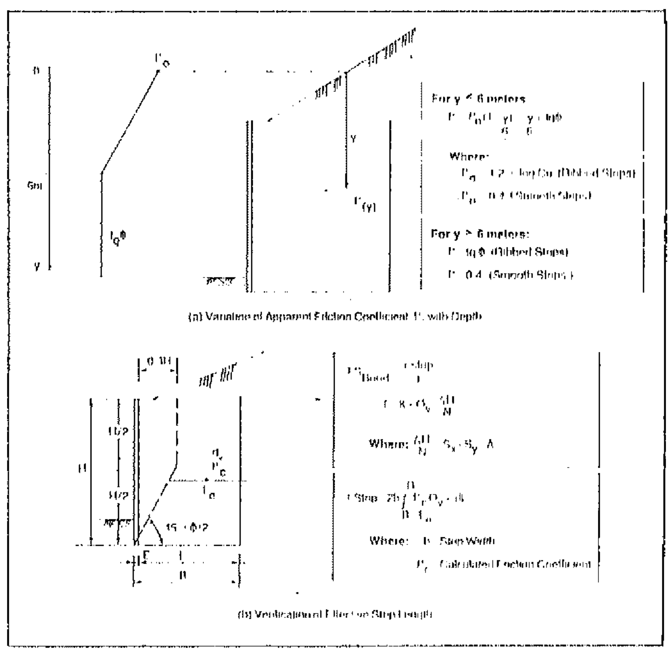


Fig 28: Design Methods for Evaluating Safety Against Failure by Lack of Adherence

Writers of the first specifications for Reinforced Earth projects (including, of course, Vidal) clearly understood the shear strength, density and dilatancy relationships and specified clean granular materials for use in all structures. Reflecting on this, it is interesting to note that by specifying such a material, they virtually eliminated from concern such problems as drainage, corrosion and post-construction movements. The first specification published by the U.S. Federal Highway Administration in 1974 was derived from the early French specification and allowed the following limiting gradation:

Sieve Opening or Screen Size	Percent Passing (By Weight)
10 inches (254mm)	100
4 inches (101.6mm)	100-75
No. 200 (75 μm)	0-15

Table III: Specification for Select Granular Backfill Material from F.H.W.A. Specification FP-74(17)

In addition, the specification further required that all backfill material exhibit an angle of internal friction of 25 degrees as determined by standard triaxial or direct shear testing methods. This caveat was added in recognition of the fact that many gravels in the western part of the United States are highly degradable and normal design assumptions would not be applicable to them. The specifications were appropriately conservative given the state-of-the-art at the time they were published. However, as more experience with actual structures was gained and as the results of theoretical and applied research were analyzed, it became evident that a significant relaxation and broadening of these specifications could be done safely, thus extending the spectrum of usable materials and further improving the potential for economy to users of the system.

From 1970 to 1974 an extensive research program was carried out by Schlosser and Long (18) to study the relationship between the fine-grained portion of a soil and the development of the angle of internal friction. In this study two types of soil test were conducted: (1) an artificial soil made with a mixture of glass balls and powdered clay and (2) mixtures of natural soils. Saturated soil samples of both types with varying amounts of fines were tested in a direct shear box. Results were conclusive in demonstrating that the parameter controlling shear strength is the relative volume of the fine-grained portion to the granular portion. Some typical results are shown in Figure 29. Additional tests have shown that the grain size which separates the fine grained portion from the granular portion is 15μ.

While this theoretical research was carried on, several projects were constructed using materials which differed from the original specification in the amount of fines passing the 75μ sieve. These materials were typically nonplastic residual soils such as those derived from decomposed granites and metasedimentary formations high in quartz and mica content (schistose and schistose gneisses of the Piedmont Plateau). Fine content (percent greater than 75μ) varied from the allowable 15 to as much as 40 percent. This field experience was generally favorable but did require the designers to focus more closely on questions of short-term stability, and to develop construction procedures necessary to effectively incorporate soils with higher fine contents into Reinforced Earth structures. Two examples illustrate this point.

At Cove Point, Maryland over 800 linear meters of single and double-faced Reinforced Earth containment dikes 3 to 5 meters high were constructed using sandy silts, with as much as 40 percent passing a 75μ screen. Extensive laboratory tests were conducted to evaluate the apparent coefficient of friction between the aluminum reinforcements and the sandy silts. These test results were used to determine safety factors for the completed structures, which were designed with the usual working stress factor of safety allowed by U.S. practice. Wall backfill had previously been excavated and stockpiled at future tank locations. During construction of the walls, work was stopped for one week due to heavy rains (which eventually totalled 25cm for the week) after which construction was immediately re-started. Backfill was brought to the wall location with 25cu meter self-loading scrapers. Two days after construction had resumed, outward deflections were noticed in several areas along the dikes. Construction was stopped so that the situation could be assessed.

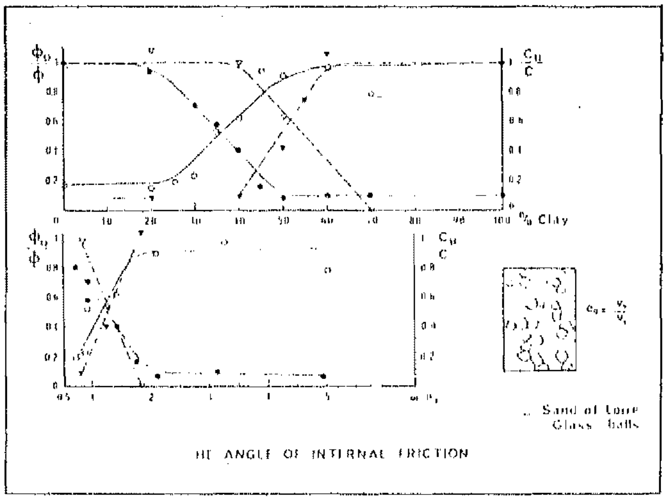


Fig 29: Evolution of the Angle of Internal Friction and Cohesion (after Schlosser and Long, 1974)

At another project along Interstate Route 70 near Vail, Colorado, a similar but slightly more dramatic episode occurred. The construction of the highway through the highly scenic Vail Pass in the Colorado Rockies required the construction of approximately 27,000 square meters of retaining wall to control embankment encroachment on streams and wilderness areas. Of the total, approximately 75 percent or 20,000 square meters were Reinforced Earth structures built using conventional, as well as curved panels. Wall heights varied from 3 to 26 meters, and walls were built vertically in a single step or in tiers. Typical structures are shown in Figures 30 and 31. Due to the short construction season, contractors often worked 20-24 hours per day on a six-day work schedule. Wall backfill was a decomposed granite with up to 25 percent passing a 75μ screen, the normal requirement for structure backfill in the State of Colorado. Early in November of 1975 as the contractor was nearing completion of a Reinforced Earth structure, a section of wall 300 feet long tilted outward during placement of the backfill. Some panels were cracked and broken, and the reinforcing strips had obviously been drawn out of the fill. As in the case at Cove Point, construction was halted and an investigation undertaken.

The two cases are, quite obviously, related to the water content and loading conditions of the soil which was placed within the Reinforced Earth structures. At Cove Point, the loading from the scrapers was far greater than the loading considered in static wall designs. This increase in overall loading, combined with a temporary decrease in shear strength caused by the higher water content and poor drainage characteristics of the soil, combined to create a marginally stable situation. Thus, outward movement of the panels occurred. Review of the calculations showed no problem with long-term stability, and construction was allowed to continue after the soil had dried out and lighter equipment was brought in. Continuing observation showed no further movement of the walls.

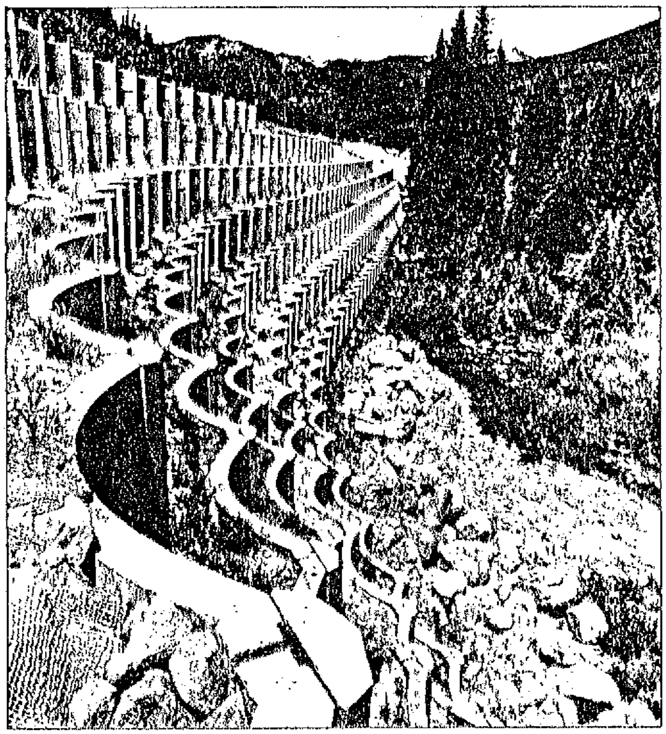


Fig 30: Vail Pass-Tiered Wall

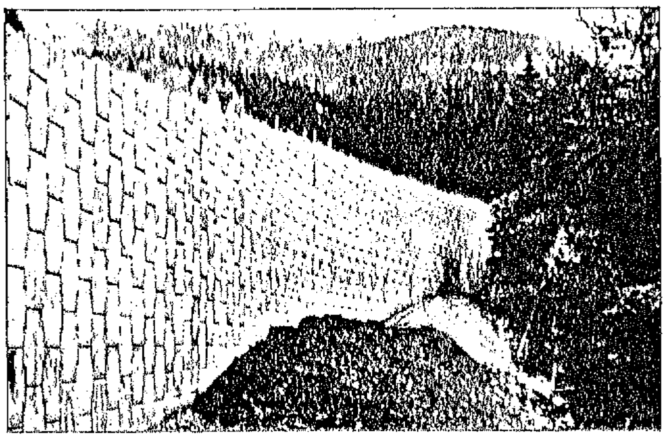


Fig 31: Vail Pass-Standard Panel Wall

At Vail Pass, investigations revealed that the filling operation had been intermittently shut down for several days prior to the wall deflections due to snow and freezing temperatures. It was not possible to determine if the contractor had cleaned, scarified and recompacted the fill surface after filling operations resumed. However, samples taken in several locations in the embankment and wall after the failure, showed water contents of 6 to 8 percent over optimum. It was, therefore, reasonable to deduce that high pore pressures were created in the wall backfill under the influence of the heavy haul equipment. Shear strength, as well as sliding resistance, were drastically reduced and the panels moved outward virtually without restraint. At Vail, the affected portion of the wall was removed, and construction resumed with proper attention to compaction water content. To completely eliminate such occurrences in the future will probably be impossible. However, if specifications are correctly written, many similar problems can be avoided.

Drawing on these and other experiences, as well as an understanding of the basic mechanics of Reinforced Earth, it has been possible to define a wider spectrum of materials suitable for use in Reinforced Earth structures. These broader limits are shown in the new F.H.W.A. specification which will be issued at the end of 1978.

Sieve Size	Percent Passing
6"	100
3"	75-100
No. 200	0-25

and P.I. < 6

OR if percent passing No. 200 is greater than 25 percent, and percent finer than 15 μ is less than 15 percent, material is acceptable if

$\phi \geq 30^\circ$ as determined by AASHTO T-236
P.I. < 6

Table IV: Minimum Specification for Select Backfill (Adopted by F.H.W.A., 1978)

When the percent finer than 75μ is greater than 15 percent, special attention to moisture-density relationships is required. The compaction specifications should include a specified lift thickness and allowable range of moisture content above and below optimum. Special attention must also be focused on design details such as internal and external drainage.

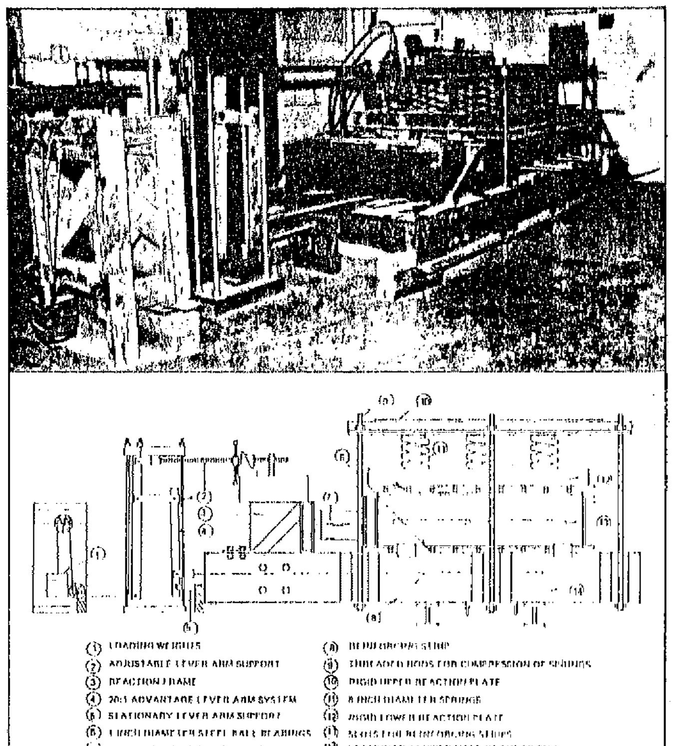


Fig 32: Large Direct Shear Box with Creep Test Setup

The broader specifications still restrict the spectrum of suitable materials to those which are non-plastic and whose deformation properties will be essentially elastic in the normal load range of Reinforced Earth structures. Even assuming that a rational design procedure could be developed for cohesive soils, we believe that the potential for large post-construction movements due to creep would be significant. A laboratory testing program is now in progress to evaluate this phenomenon. Testing procedures are shown in Figure 32. Preliminary results of a reinforcing strip pullout test in residual silts with between 70 and 90 percent passing a 75μ sieve are shown in Figure 33. The tests show significantly lower values of apparent friction are obtained with these materials and, more importantly, the values are significantly less than the shear resistance of the soil, even with deformed or ribbed strips. Figure 34 shows the result of two creep tests conducted at stress levels which varied from 34 to 49 percent of measured peak load. High deflections are seen to continue after 50 hours loading. Other testing programs now in progress will evaluate the possibility of using admixtures such as flyash, lime or cement to reduce the plasticity of cohesive soils to eliminate or minimize creep. We hope to be able to report on the results of these tests in one or two years. At the present, however, it is not possible to use such soils reliably in permanent structures.

In addition to the mechanical complexities, as soils become more fine-grained, their resistivity generally decreases. Soil resistivity is an important factor controlling the rate of galvanic corrosion, and low resistivity is often associated with aggressive soils. This topic will be discussed in greater detail in the following section.

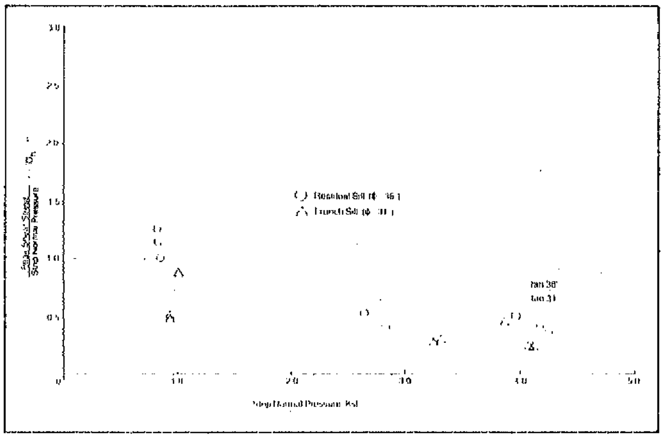


Fig 33: Strip Pullout Tests-Fine Grained Soils

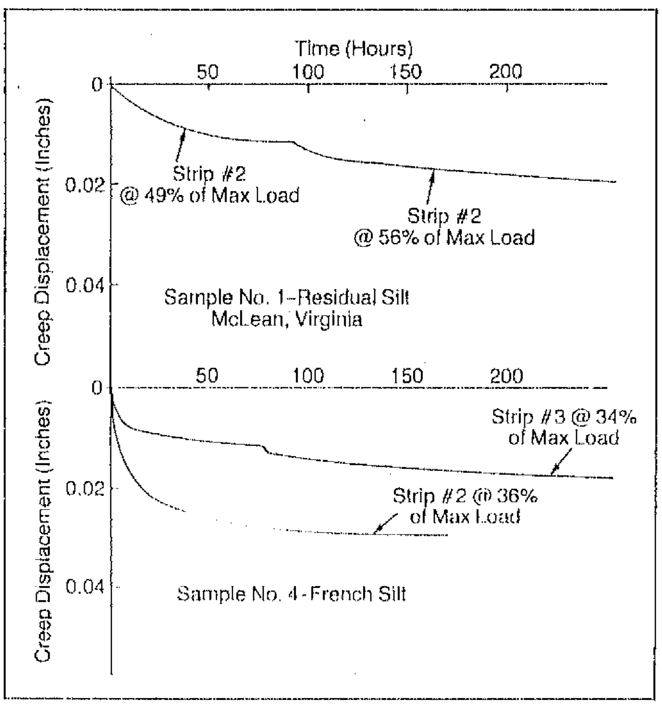


Fig 34: Creep Tests-Ribbed Strips

III. Durability of Buried Metal Reinforcements.

In the many discussions we hold with potential users of Reinforced Earth, the most frequently asked question is "How long will it last?" Everyone knows that ferrous and other metals corrode and that metallurgists might spend whole careers creating a single exotic alloy to resist the aggressive attack of a predictable environmental setting (such as aluminum boats in sea water). Reinforced Earth structures are normally designed for a service life of 75 to 100 years. Reinforcements are typically thin metal strips varying in thickness from 3 to 9 mm thick depending on the physical forces to be resisted and environmental setting in which the structure will be erected. What special information, therefore, is required to safely proportion the structural components to resist a physical phenomenon that is as undeniable as it is seemingly unpredictable? What margin will exist and what will be the consequences if the predictions are incorrect?

To attempt to answer these questions we must again reflect on the mechanism of corrosion, the results of theoretical studies and whatever actual performance data exists. In interpreting these studies and data, we must be careful to interpret them in the light of our own concerns. For example, a buried metal conduit might be considered to have failed if a pitting type corrosion completely penetrates the conduit wall and fluid or pressure is lost. In contrast, pit type penetrations of a sheet or strip may do little to reduce the effective cross-section resisting stress. Therefore, for the same corrosive effect, the strip is serviceable and the conduit is not. With this in mind, let us examine the phenomenon of corrosion as it applies to the serviceability of Reinforced Earth structures.

The corrosion process is essentially an electrochemical process. For corrosion to occur there must be a potential difference between two points that are electrically connected in the presence of an electrolyte. A typical galvanic cell is shown in Figure 35. Current flows from the anodic area through the electrolyte to the cathodic area and back through the metal to complete the circuit. In the case of buried metals, the electrolyte consists of water rich in oxygen and dissolved salts wetting the soil particles in contact with the metal. Among the factors that govern corrosivity of a given soil are (1) porosity (aeration) (2) electrical conductivity (3) dissolved salts, including depolarizers or inhibitors (4) moisture and (5) acidity or alkalinity (pH). Let us look at the influence of each of these parameters.

A porous soil may retain moisture over a longer time or may allow maximum aeration, both factors which tend to increase the initial corrosion rate. The situation is complex, however, because corrosion products formed in an aerated soil may be more protective of the base metal than those formed in an unaerated soil. In addition, it is probable that aeration of soils may affect corrosion not only by direct action of oxygen in forming protective films, but also indirectly through the influence of oxygen reacting with an decreasing concentration of the organic complexing agents or depolarizers naturally present in some soils which greatly stimulate localization cells. Another factor to be considered is that in poorly aerated soils containing sulfates, sulfate-reducing bacteria may be found. These organisms often produce the highest corrosion rates normally experienced in any soil. However, the beneficial effect of aeration extends to soils that harbor sulfate-reducing bacteria because these bacteria become dormant in the presence of dissolved oxygen.

The electrical continuity allows current to flow between anodic and cathodic zones on the metal surface. The loss of metal from the anode is proportional to the intensity of the current which in turn is directly proportional to the conductivity of the electrolyte between the two poles of the electrochemical cell. Normally, the method used to measure this important soil parameter is the resistivity, the direct inverse of the conductivity. Resistivity is dependent on the soil's content of soluble salts and varies greatly with degree of saturation.

For purposes of determining service life, the resistivity of a soil at 100 percent saturation, the worst case, is always used. Generally a high resistivity is associated with a slightly aggressive soil. Table V shows some typical resistivities of soils.

Moisture, even in small quantities, is a necessary agent in corrosion. Usually the speed of corrosion increases with increasing water content of the soil. The conductivity of water increases with increasing concentration of dissolved salts, again increasing the potential for increased corrosion.

The acidity or alkalinity (pH) of a soil also controls the rate of corrosion. Certain protective oxides that form on the surface of a metal are insoluble within certain pH ranges. For example, experience has shown that the by-products of the corrosion of zinc are insoluble within a 5 to 12 pH range.

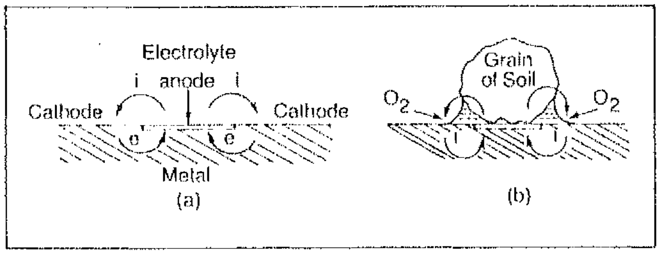


Fig 35: Idealized Galvanic Cell in Buried Metals

DESCRIPTION	pH	RESISTIVITY AT SATURATION (Ω cm)
Sands and Gravelly Sands	5.6	11,700
Silty Clay	4.3	3540
Silty or Clayey Fine Sand	5.8	2300
Fill (Derived From Natural Soils)	7.1	1000
Organic Clay & Silt (Alluvium)	3.8	1000

Table V: Some Physiochemical Properties of Soils in the Washington, D.C. Area

The potential difference between the poles of a galvanic cell is dependent upon the nature of ion concentrations on the surface. Certain ions, such as chlorides and sulfates, are aggressive; others such as magnesium and calcium are inhibitors of corrosion.

Thus, the high number of interrelated factors which influence the initial and long-term corrosion rate makes the study of corrosion and service life an inexact science, especially when one considers that many of the parameters will most certainly change with the passage of time. As in other sciences where exact solutions are not possible, and I suggest that soil mechanics is certainly one, it is necessary to determine the possible upper and lower limits to the effects or results under study and then, using prudent engineering judgment, to provide for a reasonable margin of safety. This approach is applicable to the study of corrosion of buried metals.

The most extensive series of field tests on various metals and coatings in all types of soils was begun in 1910 by the U.S. National Bureau of Standards. (NBS). These tests continued until 1955 and now constitute the most important sources of comprehensive data available in the field of underground corrosion. This information, therefore, constitutes the data base of the entire sub-science, and it is against this data that all new experience and subsequently derived empirical relationships must be compared and contrasted. It is useful, therefore, to briefly review the results of this study.

In the NBS study, samples of ferrous and nonferrous metals were buried at 128 sites. Plain and galvanized steel specimens were buried at 47 sites where the soil-water environments were different, but representative of soil conditions in the United States. The resistivity and pH were measured at each site in an attempt to determine a quantitative correlation between these measurable (but somewhat time- and environment-dependent) parameters and metal loss. Romanoff (19) the author of the NBS study, demonstrated that the rate of corrosion is greatest in the first few years after burial and decreases to a much lower constant rate thereafter. He indicated that this damping of corrosion was a more significant parameter than the initial rate. He proposed quantitative empirical relationships to calculate average loss of thickness of plain steel as a function of time.

Darbin (20), in his comprehensive review of the NBS data, has selected burial site data more or less consistent with the normal range of environments for buried reinforcing strips and extended this data in accordance with Romanoff's proposals. He compared the results of this extended or extrapolated data with other pertinent studies such as the performance of sheet piles and culverts. This comparison for galvanized steel samples and metal culverts is shown in Figure 36. This data demonstrates that even in an aggressive environment (ρ = 13,000 Ω – cm, pH = 4.7), the galvanized steel reinforcing strips currently in use would have a service life of 120 years.

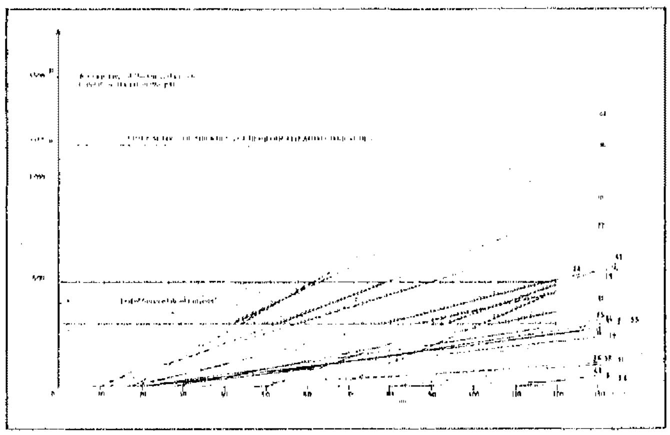


Fig 36: Synthesis of Extrapolated NBS and Metal Culvert Data (after Darbin et. al., 1978)

The extrapolation of the Romanoff data requires the solution of the exponential equation:

$$X = kT^n$$

where: X = average loss of thickness with time
k = a site characteristic
T = time in years
n = site dependent and is always less than 1.0

Since selection of k and n requires some subjective interpretation, it is useful to see if some more general quantitative conclusions can be drawn from the Romanoff data. In an attempt to obtain this, the NBS data from the 47 steel burial sites has been re-plotted. Figure 37 is an attempt to show a relationship between metal loss and resistivity. The figure shows that a well-defined relationship does not exist, but clearly demonstrates a trend of smaller metal losses with increased resitivity for sites whose pH is greater than 5. Figure 38 is an attempt to show a relationship between metal loss and pH. The figure again demonstrates that a well-defined relationship does not exist. However, it does show greater metal losses at sites with pH values less than 5. Using pH as the only guide, it is difficult to draw a conclusion. However, if only well-drained sites are plotted as in Figure 39, it can be concluded that metal losses at such sites will not exceed 0.15 oz./sq.ft./yr.

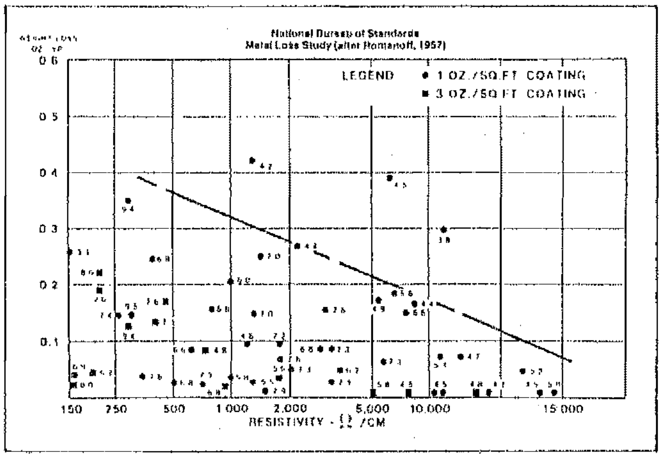


Fig 37: Weight Loss versus Resistivity

The abundance and reliability of this data and the ability to extrapolate it to the time period normally associated with engineering works has led to the selection of galvanized steel as the material most commonly used for Reinforced Earth structures. The zinc coating on galvanized steel forms a sacrificial anode which corrodes while protecting the base metal. In addition, zinc promotes a more uniform corrosion by preventing the formation of pits during the highly aggressive initial stages of burial.

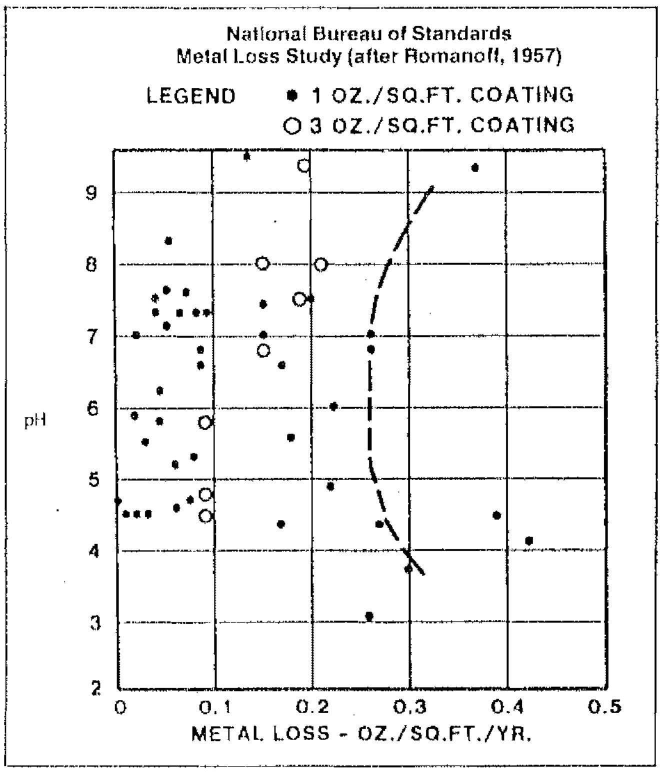


Fig 38: Metal Loss versus pH

Other materials currently used for reinforcements include plain carbon steel for temporary structures and some marine structures, and a fusion-bonded epoxy-coated steel reinforcement for highly acidic or warm marine environments.

Other materials have been used as reinforcements with mixed results. Stainless steel was used in ten structures in France. Nine of these structures showed no evidence of corrosion when examined several years after completion. In one structure, there was some evidence of surface corrosion and pitting. Aluminum magnesium reinforcements were used on several structures in France and the United States. When placed in clean, well-aerated backfills, this material demonstrated excellent performance. These passive metals such as stainless steel and aluminum, are highly reactive in the presence of oxygen, and under favorable conditions are rapidly coated with a protective film of oxides that prevents corrosion of the base metal. However, when this protective layer is destroyed, either by physical or chemical processes, rapid corrosion can occur.

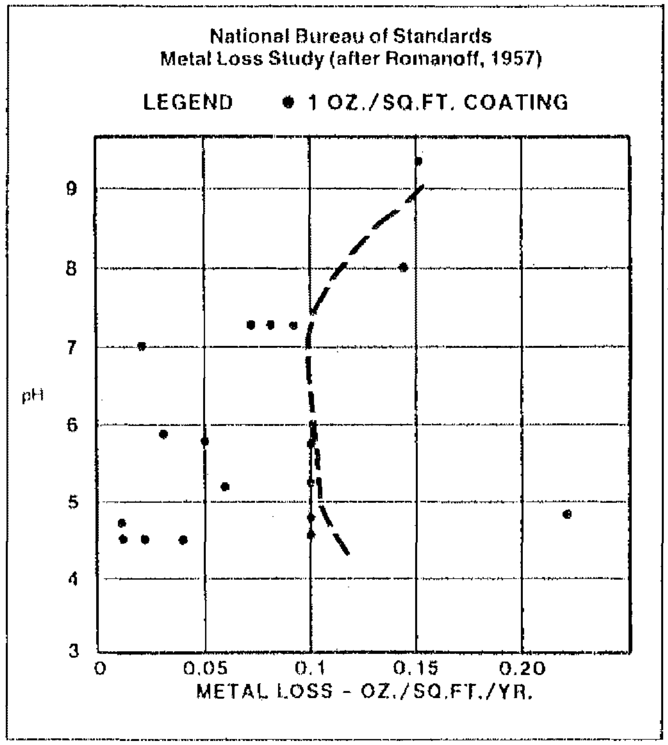


Fig 39: Metal Loss versus pH-Well Drained Sites Only

Plastics and other synthetics have also been used as reinforcements, but their performance has been disappointing. These materials are too brittle or too flexible to withstand and sustain the construction loads, and their corrosion performance is unpredictable (but in our experience uniformly poor).

In summary, we can state that there is sufficient data available to permit the selection of the cross-section and coating weight of galvanized steel reinforcements to insure a minimum service life. Design procedures for this important determination include the following:

(1) the calculation of anticipated weight loss, based on laboratory or field measured values of resistivity and pH at saturated conditions.

(2) the selection of suitable site-dependent characteristics for precise calculation according to Romanoff's formula.

(3) comparing answers found in (2) with upper limits inferred by a broad interpretation of the Romanoff data.

(4) proportioning the strip dimensions such that the stresses in the equivalent cross-section at the end of the anticipated service life will be less than or equal to the yield stress.

(5) applying whatever factor of safety to calculation (4) is required by the site and project characteristics.

CONCLUSIONS

The excellent structural performance of the more than 2,000 Reinforced Earth structures completed during the past 11 years demonstrates more than any other fact that these structures have been safely designed. Measurements and observations of movements and stresses confirm that the working stress design procedures derived on the basis of a coherent gravity structure analysis, accurately predict subsequent performance. As improvements are made in the technology, such as the recent introduction of high adherence reinforcements, basic soil mechanics theory, supported by laboratory and field testing, can be used to modify design procedures to anticipate the effects of these improvements.

This experience has demonstrated that from all considerations of performance, stress, structural deformation and corrosion, the use of wall backfill that is a well-drained granular material, compacted to a field density that results in dilation during shear, will result in an extremely safe structure with a long and highly predictable service life. As backfill materials become more fine-grained, caution must be exercised in selecting design parameters and factors of safety to allow for the more complex shear strength and corrosion characteristics of these finer grained materials. However, even with these finer grained elastic materials, adequate designs can be developed.

At present, at least, plastic materials should not be used for Reinforced Earth structures. No rational design procedure exists and their anticipated long-term performance, even assuming an adequate structural design, cannot be assured.

REFERENCES

(1) Lee, K.L., "Mechanisms, Analysis and Design of Reinforced Earth, State of the Art Report," A.S.C.E. Symposium on Earth Reinforcement, Pittsburgh, 1978.

(2) Schlosser, F., and Long, N.T., "Comportement de la Terre Armee a l'Appareil Triaxial," *Activity Report*, Laboratoire Central des Ponts et Chaussees, 1969.

(3) Hausmann, M., and Lee, I.K., "Strength Characteristics of a Reinforced Soil," *Proceedings, International Symposium on New Horizons in Construction Materials,* Lehigh, 1976.

(4) Bassett, R.H., and Last, N.C., "Reinforcing Earth Below Footings and Embankments," A.S.C.E. Symposium on Earth Reinforcement, Pittsburgh, 1978.

(5) Schlosser, F., and Juran, I., "Theoretical Analysis of Failure in Reinforced Earth Structures," A.S.C.E. Symposium on Earth Reinforcement, Pittsburgh, 1978.

(6) Al-Hussaini, M., and Perry, E.B., "Field Experiment of Reinforced Earth Wall," A.S.C.E. Symposium on Earth Reinforcement, Pittsburgh, 1978.

(7) Federal Highway Administration, "Report on the Landslide Vicinity Reinforced Earth Wall, Roseburg-Coos Bay Highway, Oregon SR42," 1977 Unpublished.

(8) Schlosser, F., "La Terre Armee: Historique, Development Actuel et Futur," *Proceedings, Symposium on Soil Reinforcing and Stabilising Techniques in Engineering Practice*, Sydney, 1978.

(9) Castro, G., "Liquefaction of Sands," *Harvard Soil Mechanics Series No. 81*, January 1969, Pierce Hall, Harvard University, Cambridge, Massachusetts.

(10) Hausmann, M.R., and Lee, K.L., "Rigid Model Wall with Soil Reinforcement," A.S.C.E. Symposium on Earth Reinforcement, Pittsburgh, 1978.

(11) Almi, I., Bacot, J., Lareal, P., Long, N.T., Schlosser, F., "Etude d'adherence Sol-Armatures," *Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering*, Moscow, 1973.

(12) Chang, J.C., and Forsyth, R.A., "Design and Field Behaviour of Reinforced Earth Wall, "*Journal of the Geotechnical Engineering Division*, A.S.C.E., July 1977, pp. 677-692.

(13) Schlosser, F., and Vidal, H., "Reinforced Earth," *Bulletin de Liaison des Laboratoires Routiers-Ponts et Chaussees Number 41*, November 1969.

(14) Bolton, M.D., Choudhurry, S.P., and Pang, P.L.R., "Reinforced Earth Walls: a Centrifugal Model Study," A.S.C.E. Symposium on Earth Reinforcement, Pittsburgh, 1978.

(15) Ponce, V.M., and Bell, J.M., "Shear Strength of Sand at Extremely Low Pressures," *Journal of the Soil Mechanics and Foundation Division*, A.S.C.E., April 1971, p. 625-638.

(16) Cornforth, D.H., "Prediction of Drained Strengths of Sands from Relative Density Measurement," *A.S.T.M. STP 523*, 1973.

(17) Federal Highway Administration, "Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects," FP 74, 1974.

(18) Schlosser, F., and Long, N.T., "Recent Results in French Research on Reinforced Earth," *Journal of the Construction Division, A.S.C.E.*, September 1974, p. 223-237.

(19) Romanoff, M., "Underground Corrosion," *National Bureau of Standards Circular 579*, 1957.

(20) Darbin, M., Jailloux, J.M., and Montuelle, J., "Performance and Research on the Durability of Reinforced Earth Reinforcing Strips".

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GEOTECHNICAL EVALUATION AND BLUFF RETREAT STUDY POINT LOMA TREATMENT PLANT ACCESS ROAD SAN DIEGO, CALIFORNIA

Prepared for
BSI CONSULTANTS, INC.

Project No. 1106-SI01
October 26, 1988

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October 26, 1988

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Attention: Mr. Joe Aroyo

GEOTECHNICAL EVALUATION AND
BLUFF RETREAT STUDY
POINT LOMA TREATMENT PLANT
ACCESS ROAD
SAN DIEGO, CALIFORNIA

Gentlemen:

In accordance with our Agreement for Consulting Services, dated November 15, 1987, we have completed our geotechnical evaluation for the proposed realignment of a portion of the access road for the Point Loma Treatment Plant, located approximately 1,000 feet south of the southerly Plant property boundary near the southerly tip of the Point Loma Peninsula, in San Diego, California.

Phase 1 of this project, the Bluff Retreat Study, was performed in conjunction with the bluff retreat study for the adjacent Point Loma Treatment Plant site, on a shared-cost basis in accordance with our Consultant Agreement with the City of San Diego, Document No. R-269683. Phase 2, the Geotechnical Evaluation for the proposed road realignment, however, differed from our proposal in that, due to environmental constraints, we were requested by the National Park Service to avoid performing excavations within the Park property. Accordingly, we agreed with BSI Consultants, Inc. that no field or laboratory tests would be performed. Our geotechnical evaluation for the proposed road realignment, therefore, is based on our detailed field reconnaissance, our investigation of the subject site for the bluff study, and on our geotechnical test borings along the roadway, within the nearby Plant site.

If you have any questions or require additional information, please give us a call.

Very truly yours,

GROUP DELTA CONSULTANTS, INC.

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

Braven R. Smillie
C.E.G. 207

WFC/BRS/jg
Attachments

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GEOTECHNICAL EVALUATION AND BLUFF RETREAT STUDY POINT LOMA TREATMENT PLANT ACCESS ROAD SAN DIEGO, CALIFORNIA

1.0 INTRODUCTION AND PROJECT DESCRIPTION

The Point Loma Treatment Plant is located approximately 3/4 miles northerly of the Point Loma Lighthouse. Coastal bluffs in this area rise to over 90 feet above sea level, and many of the improvements associated with the treatment plant, including the access road, extend relatively close to the bluffs.

The Cabrillo/Gatchell Road provides access to the lower westerly slopes of Point Loma, extending over approximately 1.3 miles from Cabrillo Memorial Drive at approximate elevation 350 feet (mean sea level datum), to the entrance to the City Sewage Treatment Plant, at approximate elevation 90 feet.

As we understand, the area of primary geotechnical concern for both the city of San Diego and BSI Consultants, Inc. is approximately 600 feet south of the entrance to the Sewage Treatment Plant site, where bluff erosion, aggravated by a collapsed sea cave, is encroaching upon the existing road alignment (see frontispiece). We have, therefore, confined our investigation to that part of the road alignment (within approximately 1,000 feet of the City property boundary) that the City is considering realigning.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of our investigation is to provide geological and geotechnical information to assist in project design. In particular, our investigation is designed to provide both 50- and 75-year lines of estimated bluff-top retreat. Additionally, our investigation is designed to address:

° The geologic setting of the site;
° Surface conditions in the site area;
° General lithologic and soil conditions;
° Evaluation and recommendations regarding the stability of temporary construction slopes, and for cut and fill slopes;
° Retaining wall, Reinforced Earth wall, or Cribwall design recommendations, if required; and
° Pavement design recommendations.

During our study, we have discussed the project with personnel from BSI Consultants, Inc. We have also been provided a 2-sheet "Preliminary Alignment and Profile, Point Loma Treatment Plant Access Road, 50-Year Alignment," (Sheet 1 of 2),; and "Preliminary Alignment and Profile, Point Loma Treatment Plant Access Road, 75-Year Alignment," (Sheet 2 of 2). These drawings, scaled 1 inch equals 40 feet, include a profile of the realigned road for each of the two alternatives. The 50- and 75-year alignment alternatives are based on the estimated 50- and 75-year bluff-top retreat lines that we have previously provided to BSI Consultants, Inc., and are also presented on the aforementioned same drawings.

Supplementary data for our study were obtained primarily from available published geologic literature and maps. These references are cited at the end of this report. Aerial photographs of the alignment area were also examined for significant geologic features.

Our field investigations for both phases of this project were conducted between November 1987 and October 1988.

This report, in part, provides the technical basis for selection of the 50- and 75-year bluff-top retreat lines previously provided to BSI Consultants, Inc., and so indicated on their above-referenced 2-sheet alignment and profile plans.

3.0 SITE CONDITIONS AND GEOLOGY

3.1 Geologic Setting

Point Loma is a long promontory, extending approximately 6 miles southward from the low land adjacent the San Diego River. Parts of its western shoreline are bordered by a narrow wave cut Quaternary-age terrace with a top elevation ranging from 25 to 95 feet above sea level. The shoreline of Point Loma is irregular, due to differences in geologic structure and in rock hardness. Wave erosion has etched out less resistant rock masses, resulting in shallow pocket coves between rocky headlands. Small pocket beaches have sporadically formed in areas where sufficient sand is available.

Offshore, the sea floor is comprised of the sedimentary rocks of the Point Loma Formation. Isolated, erosion-resistant stacks exist seaward of the intertidal zone, resulting in isolated topographic highs that cross a ledgy shelf surface. Seaward, the ledges become progressively deeper, interspersed with surge channels typically approaching the shoreline along trends of the major geologic joint sets which control the erosion resistance of this formational unit.

3.2 Site Conditions

The existing access road within the study area has been graded into a narrow Quaternary-age terrace, described above, at an average elevation of 100 feet (mean sea level datum). The frontispiece piece photograph shows the subject section of the alignment, the encroaching sea coves and caves, and the hillside, into which the road is proposed to be realigned.

3.3 Subsurface Conditions

Three geologic formations are present in the site area. These are the Point Loma and Cabrillo Formations of Cretaceous age, and the Bay Point Formation of Quaternary age. Overburden soils in the site area include alluvium, slopewash, and the topsoil which has formed on them. The following paragraphs describe these units in order, from oldest to youngest.

Point Loma Formation: The Point Loma Formation is an approximately 900-foot-thick (Kennedy, 1975) sedimentary layer that discontinuously crops out in coastal areas of northern Baja California and as far north as Carlsbad. At the site, it forms the lower, more resistant parts of the sea cliff up to elevations of 54 to 60 feet, and it dips into the sea cliff at about 8 to 12 degrees. The Point Loma Formation extends seaward, comprising the sea floor adjacent the cliff.

The Point Loma Formation consists of well-indurated marine sediments deposited by an offshore and deep-water submarine fan. Offshore deposits are represented by the thin-bedded siltstone and fine sandstone exposed in the upper part of the sea cliff. Deep-water deposits are represented by the erosion-resistant thick-bedded mudstone and sandstone exposed at the base of the cliffs. The Point Loma Formation ranges in age from approximately 70 to 80 million years within Rosario Group rocks of the Upper Cretaceous Period.

Cabrillo Formation: The Cabrillo Formation is a 560± foot thick sedimentary deposit that discontinuously crops out in coastal San Diego County from the southern tip of Point Loma to Carlsbad. At the site, it forms the slopes east of the coastal terrace on which the Plant is situated. The formation consists of moderately-indurated, massive marine sandstone and conglomerate deposited in the nearshore area of a submarine fan. The Cabrillo Formation conformably overlies the Point Loma Formation. The age of the Cabrillo Formation ranges from approximately 66 to 70 million years within the Rosario Group rocks of the Upper Cretaceous Period.

Bay Point Formation: The Bay Point Formation, deposited on the coastal terrace on which the subject section of the access road is built, ranges up to approximately 35 feet in thickness and forms the upper part of the sea cliff above elevations of 54 to 60 feet. The cliff-forming section of Bay Point Formation is approximately

4.0 SHORELINE EROSION

4.1 Lower Bluff Erosion

The Point Loma Formation is exposed along the entire base of the sea cliffs in the study area; it is vertical to near-vertical in most areas and is on the order of 50 to 60 feet high in the project area. Erosion at the base of the cliff, up to approximately elevation +10 feet (MSL), is due predominantly to direct wave impact acting upon small joints and fissures in the lower, more massive rock unit, and by water-hammer effects between parallel joints. Much of the Point Loma Formation exposed in the shoreline bluffs is quite intact, and appears to have experienced little erosion in the last 50 years. In other areas, where fractures and joints in the rock are more prevalent, erosion is occurring more rapidly. Where shear zones are present, surge channels and caves have developed.

4.2 Upper Bluff Erosion

The upper bluffs are comprised of the Point Loma Formation above approximate elevation +10 to +20 feet (MSL) and the overlying Bay Point Formation beginning at elevations ranging from approximately 50 to 60 feet. The Bay Point sands form the uppermost portion of the bluffs and are approximately 35 feet in thickness. These sands are subject to several different forms of erosion as a result of the following actions:

- Wave spray and wave splash;
- Undermining of the basement rock and caving of the resulting oversteepened slopes; and
- Wind, rain, irrigation, and uncontrolled surface runoff.

The upper bluffs, which support little or no vegetation, are exposed to the elements throughout most of the site. Wave spray and splash often reach these unprotected sands, causing saturation of the outer layer and subsequent sloughing of oversteepened slopes.

In areas where the Point Loma Formation is experiencing erosion at the base of the sea cliff, the overlying upper bluffs become undermined and subsequently fail through loss of vertical support. This results in oversteepened slopes that stand nearly vertically. The Point Loma Formation and the lower cliff-forming section of marine Bay Point Formation form the existing sea cliff. The upper slope-forming section of Bay Point Formation stands vertically, until the pore-water tension within the soil has had a chance to dissipate; then sloughing occurs. The slopes are relatively stable when they attain inclinations of about 1 to 1.

Wind, rain, irrigation, and uncontrolled surface runoff contribute to minor erosion of the upper cliff face, especially on the more exposed, oversteepened portions of the friable sands. A considerable amount of rilling has occurred along portions of the upper cliffs as a result of these actions.

4.3 Rates of Bluff Retreat

When studying and reporting sea cliff erosion and retreat, care must be taken to distinguish between cliff retreat rates based on: (a) bluff or cliff top retreat, (b) shoreline or cliff base retreat, and (c) averages between the top and bottom at various locations along the cliff. The degree of erosion can vary significantly from spot to spot on a sea cliff, and is influenced by many independent and dependent variables (that is, lithology, joints or fracture characteristics, beach configuration, offshore bottom conditions, climate, impacting wave configuration and energy, and human effects). Because erosion does not necessarily act uniformly over a sea cliff, nor necessarily at a uniform rate, the lack of clarification of the basis for the qualitative erosion rate values can lead to confusing and misleading results.

Kennedy (1973) provides a good general discussion of the erosion processes and forces acting on the Point Loma peninsula. Based on a comparison of old and new photographs, Kennedy reported that 75 percent of the sea cliff area has undergone no appreciable erosion during the last 75 years; only 20 percent of the cliff has undergone very rapid retreat of 10 feet in the last 75 years (0.13 feet per year), with nearly 5 feet of retreat occurring in the late

1940's. Kennedy's reported average erosion rate was 3 feet in 75 years (0.04 feet per year).

4.3.1 Site-Specific Bluff Retreat Rate

In order to evaluate the rate of bluff retreat in the vicinity of the proposed Cabrillo Road realignment, a review was made of the following data:

- Stereographic aerial photographs from 1939 to the present;
- Pertinent Historical Society photographs and supporting data (some of which were taken as early as the 1800s);
- Topographic maps and supporting field notes dating back to 1859;
- Applicable geologic and geotechnical literature;
- Historical storm data; and
- Wave climate.

A detailed geologic site reconnaissance was then performed to map sediments exposed in the bluffs in order to develop an understanding of the soil characteristics and strength of individual stratigraphic units, the bedding attitudes, faulting, joint and fracture patterns, and to look for evidence of perched groundwater seepages. An inventory was also made of adjacent and nearby bluffs in order that they could be compared to the site-specific stratigraphy, structure, slope geometry and stage of development.

After evaluating the data collected, geologists and oceanographers most conversant with the Point Loma shoreline processes were contacted at the following agencies:

- The Army Corps of Engineers;
- The United States Geological Survey;
- Scripps Institution of Oceanography;
- San Diego County; and
- The State of California.

Based on a review of the available data, our geologic inventory of the site vicinity, and discussions with other experts, we have developed a design rate of bluff retreat and a 50 and 75-year bluff retreat line. A detailed description of the methods used for determining the bluff retreat rate is included in Appendix A.

The design bluff retreat rate is, in part, based on our combined evaluation of historical storm data (Table 1) and stereographic aerial photographs from the following dates:

Stereographic Aerial Photographic Coverage

Date of Photograph	Photographic Scale
1987	1:12,000 (photographically enlarged to 1"=200')
1986	1:12,000
1985	1:40,000
1982	1:24,000
1981	1:24,000 (photographically enlarged to 1"=200')
1978	1:40,000 (photographically enlarged to 1"=200')
1972	1:20,000 (photographically enlarged to 1"=200')
1964	1:24,000 (photographically enlarged to 1"=200')
1960	1:24,000 (photographically enlarged to 1"=200')
1953	1:24,000 (photographically enlarged to 1"=200')
1950	1:24,000 (photographically enlarged to 1"=200')
1949	1:20,000
1939	1:24,000 (photographically enlarged to 1"=200')

The 50- and 75-year bluff retreat lines generated from our design rate of bluff retreat are shown on a 2-sheet set of preliminary drawings titled "Preliminary Alignment and Profile, Point Loma Treatment Plant Access Road, 50-Year

Alignment," (Sheet 1 of 2),; and "Preliminary Alignment and Profile, Point Loma Treatment Plant Access Road, 75-Year Alignment," (Sheet 2 of 2). These drawings, prepared by BSI Consultants, Inc., show a preliminary date of May 9, 1988, and were prepared under the supervision of Mr. Joe Aroyo, R.C.E. No. 36086, Engineer in Responsible Charge.

5.0 SLOPE DESIGN CONSIDERATIONS

In order to evaluate the general stability of proposed construction cuts and finished slopes along the alignment, nearby exposures and cut slopes in the site vicinity were examined, and information determined regarding their height, inclination and any geologic structure. With this information, soil strength parameters were back-calculated assuming a static factor of safety of 1.2 (equivalent to a seismic factor of safety of 1.0). This enables a general comparison of the lower limit of soil strengths necessary for stability of both temporary construction cuts and permanent slopes. Overall, slope stability was also evaluated in determining design lateral earth pressures for retaining walls.

In our analyses, we have assumed an internal angle of friction of 33 degrees and a soil unit weight of 115 pcf for the formational soils. The Janbu method of analysis was used to back-calculate the required cohesion necessary for a static factor of safety of 1.2.

6.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Our investigation did not reveal any major adverse geologic conditions on the site, such as faults or landslides, which would preclude the proposed road realignment and any associated retaining walls, if they should be required. Additionally, competent

materials exist, which should provide adequate foundation support for these walls.

6.2 Grading

It is our opinion that the formational soils on site can be ripped, with heavy duty equipment, to design grades throughout the entire alignment.

In our opinion, all materials generated on site during grading operations are suitable for use as structural fill soils. The majority of the on-site fill soils are considered nonexpansive in nature and would be suitable for use at finish grade.

A limited amount of porous alluvial and slopewash materials (generally less than 2 feet) presently exist on the slopes within the realignment area. In our opinion, excavation for the realignment will remove most or all of the porous overburden soils within the realignment area.

We expect that both the 50- and 75-year proposed alternates for realignment will remove most or all of existing variable thickness, porous alluvial soils at the site. These soils are considered unsuitable in their present condition for the direct support of any settlement-sensitive structural elements. Asphaltic concrete pavement is, however, a semi-flexible structure and can accommodate slight differential movements without distress to the pavement section. In this regard, we recommend that, as a minimum, road bed excavations be examined and, where porous alluvial soils are exposed at pavement subgrade elevations, these materials should be scarified in place to a depth of 12 inches, moisturized as necessary, and compacted to a minimum relative compaction of 95 percent of the laboratory standard, as determined from ASTM Test Designation D 1557-78.

6.3 Slopes

It is our opinion that proposed cut slopes having maximum slope inclinations of 2.0:1.0 (horizontal to vertical) and maximum unsupported slope heights of up to 20 feet will have adequate factors of safety against both deep-seated and surficial slope failure.

It is our opinion that properly engineered fill slopes having maximum slope inclinations of 2.0:1.0 and maximum slope heights of up to 10 feet will have adequate factors of safety against both deep-seated and surficial slope failure.

6.4 Retaining Wall Design Criteria

From a geotechnical standpoint, the Pleistocene-age terrace deposits, which will provide support for any proposed retaining walls, are relatively competent in nature and suitable for relatively heavily loaded foundation elements. We recommend that an allowable bearing pressure of 5,000 psf be used for design of footings, bearing on undisturbed Bay Point formational sands. This value may be increased by 1/3 for transient loads, such as seismic and wind loads.

We recommend that all backfill materials behind any proposed retaining walls consist of properly compacted, nonexpansive granular soils. The backfill should be placed and compacted in accordance with the current Standard Specifications for Public Works Construction. We recommend that all backfill be compacted to a minimum relative compaction of 90 percent of the laboratory standard, as determined from ASTM Test Designation D-1557-78.

We recommend that retaining walls be designed to resist a lateral earth pressure equal to an equivalent fluid pressure weighing 40 pounds per cubic foot. This assumes that soil behind the retaining wall will consist of compacted granular soils, and that a sloping surcharge condition exists at an inclination of 6 units horizontal to 1 unit vertical. Recommended earth pressures do not include hydrostatic loading. If drainage is not provided or maintained immediately behind the wall, hydrostatic forces need to be included in the design.

To provide resistance for design lateral loads, we recommend that passive pressure be assumed equivalent to a fluid pressure of 350 pcf for footings and shear keys poured neat against sides of excavations. This value assumes a horizontal surface for the soil mass extending at least 10 feet from the base of the wall or 3 times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of materials in areas susceptible to erosion should not be included in design for passive resistance to lateral loads. If friction is to be used to resist lateral loads, we recommend a coefficient of friction of 0.35 between the soil and the base of the footing. If it is desired to combine friction and passive resistance in design, we recommend using a reduced friction coefficient of 0.25.

6.5 Pavement Design

The appropriate pavement section depends primarily on the shear strength of the subgrade soils, traffic load, and planned life. We have not been allowed the opportunity to expose the actual pavement section or subgrade soils, and have performed no laboratory tests to evaluate the soils' support characteristics for pavement subgrade. As we have previously discussed, since the existing pavement section appears to be in excellent condition, we would suggest that the existing as-built pavement section be determined and utilized as the basis of design for any new pavement sections.

We recommend that surface drainage be provided to prevent ponding of water and to reduce infiltration of water into the subgrade materials. We suggest that paved areas have a minimum gradient of 1 percent.

7.0 LIMITATIONS

Coastal engineering and the earth sciences are characterized by uncertainty. Professional judgements represented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgements rendered meet the current professional standards; we do not guarantee the performance of the project in any respect. This warranty is in lieu of all other warranties, express or implied.

REFERENCES

California Division of Mines and Geology, 1963, County Report 3.

Gastil and Higley, 1977, Guide to San Diego area stratigraphy: Department of Geological Sciences, San Diego State University.

Kennedy, M.P., and others, 1975, Character and recency of faulting, metropolitan San Diego area: California Division of Mines and Geology Special Publication 123.

Leighton and Associates, 1984, Seismic safety study for the City of San Diego.

Vertical stereographic aerial photographs flown in 1953 (scale 1:24,000).

MAPS AND CHARTS

1967, (photo revised 1975), "Point Loma, California," USGS Topographic Quadrangle, scale 1:24,000.

November 1, 1960 (plans approved), November 3, 1961 (as-built), Plans for the Construction of Cabrillo Road and Site Grading for Sewage Treatment Plant, 17 sheets, City of San Diego Drawing Nos. 9074-1D to 9074-17D.

February 10, 1960, Untitled topographic map, No. 186-1689 (18-76), City Engineers Office, City of San Diego, scale 1":200'.

1953, "Point Loma, California," USGS Topographic Quadrangle, scale 1:24,000.

1942 ed., (surveyed 1938 - 1939), "Point Loma, California," USGS Topographic Quadrangle, scale 1:31,680.

June 12, 1941, "Desired improvements, San Diego Harbor, California," scale 1:15,000.

1916 and 1917, "California, San Diego Bay, Point Loma to San Diego," U.S. Coast and Geologic Survey, scale 1:10,000.

July 1, 1911, "Entrance to San Diego Harbor, California," U.S. Army Corps of Engineers, scale 1:1000±.

REFERENCES (CONTINUED)

MAPS AND CHARTS (continued)

04-01-1910, "Submarine cable route, Fort Rosecrans, San Diego Harbor, California," U.S. Army Signal Office, scale 1:10,000.

1904, "San Diego Quadrangle," U.S.G.S. Topographic Quadrangle, scale 1:62,500.

July 1902, "San Diego Bay, California," U.S. Coast and Geodetic Survey, scale 1:40,000.

May 9, 1902, "Entrance to San Diego Harbor, California," U.S. Army Corps of Engineers, scale 1:10,000.

March 13, 1901, "Entrance to San Diego Harbor, California," U.S. Army Corps of Engineers, scale 1:10,000.

1859, "San Diego Bay, California," U.S. Coast Survey, scale 1:40,000.

July 1858, "Plat of the Pueblo Lands of San Diego," U.S. Surveyor General, scale 1":80 chains.

1856, "Hydrography of San Diego Bay and vicinity," U.S. Coast Survey, scale 1:10,000.

1853, "San Diego entrance and approaches," U.S. Coast Survey, scale 1:20,000.

1852, "Map of False Bay near San Diego, California," U.S. Coast Survey, scale 1:10,000.

1851, "Map of San Diego Bay, California, from Point Loma to New Town," U.S. Coast Survey, scale 1:10,000.

1851, "San Diego Harbor, California," U.S. Coast Survey, scale 1:10,000.

Undated (prior to 1848), untitled map of San Diego area - Spanish, scale in varas.

APPENDIX A

SHORELINE EROSION

APPENDIX A

SHORELINE EROSION

The coastal bluffs at Point Loma consist of a resistant geologic unit at the bottom, and less resistant materials in the upper parts of the bluff. The relative effectiveness of marine erosion of the more resistant lower bluff, compared to subaerial erosion of the less resistant upper bluff, produces characteristic sea cliff profiles. Rapid marine erosion compared to subaerial erosion produces a steep cliff whereas slow marine erosion produces a gently-sloping upper bluff.

Local variations in geology along Point Loma have produced a range of profile types. In the lower 10 to 20 feet of the sea cliff, the Point Loma Formation is typically highly resistant to erosion. The upper portion of the Point Loma Formation, encountered between the elevations of about 20 and 60 feet, is intermediate in erosion resistance. The upper bluffs, which are comprised of the Bay Point Formation, typically found above elevation 56 to 60 feet, have relatively low erosion resistance. Exposure of these three materials to marine and subaerial erosion has produced different profiles for headlands and coves.

Weathering patterns in the Point Loma Formation are variable, due to the localized presence of joint and fracture zones. Unfractured portions of the Point Loma Formation are relatively resistant to weathering, and have experienced little erosion over the last 50 years. Where fractures and joints are more prevalent, erosion rates are correspondingly greater. A number of shear zones are also present in the lower bluffs which exhibit intense fracturing or brecciation. These areas are subject to high levels of erosion and often form the loci for surge channels and sea caves.

Marine erosion is generally somewhat slower than subaerial erosion for the typical headland profile adjacent the site. The profile of slope formers in the upper bluff indicate that marine erosion is slow enough to permit a rather gentle slope to develop on the upper Bay Point Formation.

The profile of the cove exhibits relatively steep cliffs up to about 90-feet high in both geologic units. Undercutting by marine erosion at the base of the cliff is common. This profile indicates that the rate of marine erosion at the base of the cliff is much greater than the rate of subaerial erosion of the upper cliff. The upper portion of the cliff tends to retreat by collapse of overhangs and block fall along steep joints in order to keep up with the marine erosion.

DESIGN CONSIDERATIONS OF BLUFF-TOP RETREAT

Placement of improvements on the coastal terrace above the bluff must account for changes in the bluff that should be expected during the intended life of the improvement. One approach has been to build as close to the bluff as desired, assuming that maintenance and repair can forestall loss of the improvement. Another approach is to estimate the amount of bluff-top retreat that should be expected within the life of the improvement and to build behind the influence of retreat.

In coastal engineering, the concept of intended lifetime of an improvement has been replaced by required design periods set by regulatory agencies. The Corps of Engineers requires 50 years and the California Coastal Commission requires 75 years. These design periods approximate the useful life of most improvements.

Available compiled measurements of bluff-top retreat are too widely variable for use in engineering design. For cliff profiles similar to those in San Diego, the best estimates of retreat rates have been reported near Santa Barbara where cliff materials similar to those at the site experienced measured bluff recession rates of 1.87 to 12.14 inches per year (Norris, 1975). These rates were measured by comparing existing structures to the topography on plot plans filed for their building permits. For the 75-year period of interest, the indicated bluff-top retreat would be approximately 12 to 76 feet. Rates of up to 1.5 feet per year have been reported for Sunset Cliffs by the U.S. Army Corps of Engineers. At that rate, bluff-top retreat would be 112 feet in 75 years.

Large short-term variations in bluff-top retreat should be expected in steeper sea cliffs. Wherever the profile is steep enough that rock and slope stability is questionable, failures can cause an instantaneous retreat of many feet.

We have applied certain geomorphic techniques to estimate rates of bluff-top retreat. This requires breaking the problem down into component processes, analyzing each component, identifying the interaction of the components, and evaluating each characteristic bluff profile for the site.

The component processes of bluff-top retreat operate on various parts of the bluff and the sea cliff. The components are as follows:

1. Marine erosion at the base of the sea cliff;

2. Collapse of overhangs and block fall along joints, essentially a rock stability problem; and

3. Slope decline by subaerial erosion of the middle and upper bluffs.

The components interact in different ways on the various bluff profiles characteristic of headlands and coves. In isolation, each component process would independently proceed to completion

or to an asymptotic rate. For example, slope decline in the Bay Point Formation would eventually produce a slope somewhat flatter than the angle of repose in several million years. In reality, continued erosion at the base of the cliff keeps the cliff and bluff steep, at approximately the same profile. For this to occur, the separate components of bluff-top retreat must retain the same approximate balance over time. The process of bluff-top retreat is further complicated by the presence of existing shoreline protection which may be only partially adequate. In general, this tends to mitigate the marine erosion component.

MARINE EROSION AT THE BASE OF THE SEA CLIFF

Retreat rate at the base of the sea cliffs is variable. One generally uniform rate appears to affect the headlands, and another rate affects the coves. Moreover, the rate in the coves appears to be variable from cove to cove and may also vary widely over time. These differences in characteristic rate between the cliffs and coves requires separate evaluation.

Marine erosion which affects the base of the sea cliff includes mechanical, chemical, and biological erosion processes. Marine erosion operates horizontally on the cliff as far up as the splash zone. It is accompanied by downwearing (marine erosion measured in a vertical direction) of the seafloor adjacent the cliff, which operates in a vertical direction. In general, backwearing (marine erosion measured in a horizontal direction) and downwearing progress at rates that will maintain the existing slope of the seafloor at approximately 50:1. This suggests that the rate of downwearing is approximately 2 percent of the rate of backwearing.

<u>Effect of Water Depth, Wave Height, and Seafloor Slope</u>

The key aspects of the sea-cliff profile for the marine erosion component of bluff-top retreat are the water depth at the base of the cliff, the breaking wave height and the slope of the seafloor near the base of the cliff. Tidal and seasonal variations subject the cliff to the attack of non-breaking, breaking, and broken waves that in turn control which mechanical erosion processes are active. Forces due to non-breaking waves are primarily hydrostatic. Broken and breaking waves exert an additional force due to the dynamic effects of turbulent water and the compression of entrapped air pockets.

Breaking waves exert a considerable added erosive force called "breaking wave shock" because of trapped air cushions in a near-vertical wave front. These shock or impact pressures result in relatively high pressure fields that last a few thousandths to a few hundredths of a second. These relatively short-duration impact pressures are of questionable importance in the design of vertical seawalls, however, when acting upon jointed and fractured rock, the water-hammer effect tends to cause hydraulic fracturing which exacerbates lower sea cliff erosion. Large sections of rock can be pried off by one well-placed wave. Erosion associated with breaking waves is most active when water depths at the base of the sea cliff (ds) coincide with the respective critical incoming wave height (H) such that ds $\simeq$ 1.3H.

Waves will break when their height reaches approximately 75 percent of the water depth, thus 3 to 4.5 foot wave heights, will break at the base of the sea cliff when tides are at mean sea level. Moreover, since the waves reaching the coast are generally in the range of 2 to 5 feet, breaking waves should be expected to occur at the base of the sea cliff usually four times a day (due to semidiurnal tidal fluctuations).

The slope of the seafloor adjacent the bluff is typically 50:1 or about 1.15 degrees. Whenever wave height and water depth are sufficient to produce breakers some distance offshore from the cliff, the very gradual slope will influence the breaker to form broken waves with high turbulence. The broken waves may reform as smaller non-breaking waves. Moreover, the smaller non-breaking

waves may, in turn, reform as small breakers in a repetition of the process. When waves break and reform, considerable wave energy is lost to drag on the seafloor; consequently, less erosive energy is delivered to the base of the sea cliff.

Erosion Processes

The types of erosion affecting the typical Point Loma profile will change with the tidal level. In addition, any local variation that changes the average water depth will significantly alter the local balance of erosive forces.

Mechanical erosion processes at the base of the sea cliff include water abrasion, rock abrasion, cavitation, water hammer, air compression in joints, breaking-wave shock, and alternation of hydrostatic pressure with the waves and tides. All of these processes are active in backwearing. Downwearing processes include all but breaking-wave shock. Backwearing and downwearing by the mechanical processes described above are both augmented by bioerosion. Bioerosion is the removal of rock by the direct action of organisms. Backwearing at the site is assisted by algae in the intertidal and splash zones and by rock-boring mollusks in the tidal range. Algae and associated small organisms bore into rock up to several millimeters. Mollusks may bore several centimeters into the rock. Both chemical and salt weathering also contribute to the erosion process.

The general rate of marine erosion at the base of the sea cliff is the result of the combined effect of mechanical erosion and bioerosion. Reported total rates for sedimentary rock coasts vary from less than 10 mm/yr for hard-rock coasts, to 2000 mm/yr for weak sedimentary rocks such as mudstones and siltstones. The Point Loma Formation at the base of the sea cliff is in the hard-rock part of this range of rock types.

Measured Rates of Marine Erosion in the Region

In San Diego County, rates of marine erosion have been measured for the somewhat less resistant sedimentary rocks present at the base of the sea cliff north of Point Loma. The average rate obtained was approximately 10 mm/yr during a five-year period, from 1970 to 1975 (Lee, Pinckney, and Bemis, 1976), of mild winters with few major storms and only one episode of extreme wave activity. More typically, a five-year period would include three or four extreme-wave episodes suggesting a proportional increase in erosion.

The rate of bioerosion has been estimated for downwearing to be 0.6 mm/yr for sandstones in Southern California (North, 1954). No estimates are available for the horizontal component of bio-erosion.

Headland Erosion

For headlands, we have chosen to use a preliminary estimated rate of one-half inch per year (12.7 mm/yr) for marine erosion at the base of the sea cliff. This estimate is based on review of available stereographic aerial photographs, consideration of the world-wide data, local measurements, variations in rock type, and the long-term storm record. Beginning with the measured rate for the San Diego coast of 10 mm/yr, the one-half inch per year rate was estimated assuming an increased wave environment to be approximately balanced by more erosion-resistant rock of the lower Point Loma Formation. At this rate, approximately 3 feet of marine erosion should be expected to occur in 75 years at the base of the sea cliffs.

Shoreline erosion of the headlands was not determinable through review of historical photographs dating back to 1939. That is to say, the rate of erosion was too slow to detect any measurable rate of retreat, thereby corroborating the preliminary estimate of 1/2 inch per year.

Cove Erosion

The Point Loma Formation is generally resistant to erosion, except at fractures and joints. Careful geologic mapping provided close correlation of faults, shear zones, and fractures, with associated increased rates of erosion of the base of the sea cliff in these areas. Thus, coves and other discontinuities exhibiting locally higher rates of marine erosion are joint controlled along this reach of coastline.

Active Cove Erosion Processes

Erosion processes in the coves are essentially the same as those along the sea cliff. Small differences arise because, under normal day-to-day sea conditions, the wave energy is occasionally sheltered somewhat by the adjacent headland, which often leads to a comparatively quiet water environment in the cove. Extreme wave episodes often arrive directly on the coast from within 30 degrees of perpendicular to the general shoreline. The direct approach of extreme waves transmits high erosive energy into the cove for short periods of time. Moreover, local offshore sea floor bathymetry tends to focus wave energy into the coves.

Rate of Marine Erosion for Coves

Significant differences in erosion rates are evident between the sea cliff and the coves. This is in part due to the difference in lithology and intensity of jointing, the wave-direction dependence of transmitting erosive energy into the cove, and the energy-focusing effect of surge channels.

The less-resistant upper Point Loma Formation in the coves is judged to have the approximate erosion resistance of the younger, Tertiary-aged rocks in which direct measurements were made along the coast north of Point Loma. The rate for the sea cliff at the site was, in part, based on recognition of the more resistant nature of the lower section of Point Loma Formation. Locally, jointing and faulting further reduces erosion resistance near the apex of the cove. This leads to an upward adjustment in rate in the back of the cove.

Aerial photographs taken at irregular intervals from as early as 1939 to the present were compared in estimating the rate of cliff retreat. Our interpretation of these photographs indicated bluff-top retreat rates varying from approximately 2½ to 4 inches per year. Review of aerial photographs further indicated that upper bluff retreat was primarily due to sloughing, whether by undermining or due to localized slope instability. Thus, the rate of marine erosion in the backs of coves is substantially faster than subaerial erosion and the development of a more stable (flatter) upper slope cannot be initiated due to the excessive rate of undermining that occurs in the backs of the coves.

We have chosen to use a rate of 4 inches per year for the base of the sea cliff contribution to bluff-top retreat in the back of coves. This is somewhat higher than a measurement reported by Kennedy in a sea cave at Sunset Cliffs of five inches between 1965 and 1973 (Kennedy, 1973). This period included several extreme wave episodes. The Point Loma Formation exposed at Sunset Cliffs is judged to be intermediate in erosion resistance between the more erodible sediments in coves in the vicinity of the site and the resistant sedimentary rocks along the sea cliff in this area.

REFERENCES

Lee, Louis, Pinckney, C.J., and Bemis, C., 1976, "Sea cliff base erosion, San Diego, California: American Society of Civil Engineers, National Water Resources and Ocean Engineering Conference, April 5-8, 1976, preprint 2708, pp. 1-13.

Norris, R.M., 1968, Sea cliff retreat near Santa Barbara, California: California Division of Mines and Geology, Mineral Information Service, v. 21, No. 6, p. 87-91.

North, W.J., 1954, Size distribution, erosive activities, and gross metabolic efficiency of the marine intertidal snarls, Littorina planaxis and L. Scutulata: Biol. Bull. 106, p. 185-197.

Pease, R.C., 1979, Scarp degradation and fault history south of Carson City, Nevada: University of Nevada, Reno, Masters Thesis, 90 p.

Sanders, N.K., 1968, The development of the Tasmanian shore platforms, University of Tasmania, Ph.D. Thesis.

Trenhaile, Alan S., 1987, "The geomorphology of rock coasts," Clarendon Press, Oxford, 384 pp.

Wallace, R.E., 1977, Profiles and ages of young fault scarps, north-central Nevada: Geological Society of America Bulletin, v.88, p.1267-1281.

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FINAL PRELIMINARY REPORT

NORTH SHORELINE
PROTECTION IMPROVEMENTS
POINT LOMA WASTEWATER
TREATMENT PLANT
SAN DIEGO, CALIFORNIA

CIP NO. 46-175.0

Prepared for
BLACK & VEATCH
San Diego, California

Project No. 1601
July 3, 1995

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FINAL PRELIMINARY REPORT
NORTH SHORELINE PROTECTION IMPROVEMENTS
POINT LOMA WASTEWATER TREATMENT PLANT
SAN DIEGO, CALIFORNIA

CIP NO. 46-175.0

Gentlemen:

In accordance with our consultant agreement, we have completed the final preliminary report, including site information and constraints, and preliminary studies for certain shoreline and upper-bluff stabilization measures considered necessary to protect the northwest corner of the Point Loma Wastewater Treatment Plant in San Diego, California.

The accompanying report presents the results of the various engineering support studies, site information and constraints, and alternate concept designs for shoreline and upper-bluff stabilization in this area.

If you have any questions or require additional information, please give us a call.

for BAJADA/GDC ASSOCIATES

Very truly yours,

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R.C.E. 23792, R.G.E. 245

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WFC/BRS/jc

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EXECUTIVE SUMMARY

Coastal erosion and bluff retreat have characterized coastal geomorphic processes in the San Diego area for an estimated 18,000 years and, consequently, the construction and maintenance of shoreline protection have been necessary since the opening of the Point Loma Metropolitan Treatment Plant in 1963. The most recent shoreline and upper-bluff stabilization project, completed in 1992, required extensive construction at five separate sites within the plant boundaries, and included the construction of stone revetments, tied-back walls, gravity walls, and the resurfacing and structural stabilization of a deteriorated steel binwall. The City currently intends to construct the North Shoreline Protection Improvements (NSPI) at the landward end of an approximately 100-foot-wide, 200+-foot-long sea cove (collapsed sea cave) at the northern boundary of the plant site, an area of accelerated erosion not addressed or mitigated by the 1992 construction. The NSPI project area was formed along an existing fault lineament, which has weakened the cliff-forming bedrock unit in this area, originally allowing the formation of a large sea cave, the roof of which eventually collapsed, forming the cove that exists today. This fault-controlled cove area continues to exhibit significantly higher erosion rates, necessitating the currently-proposed improvements.

The proposed improvements include a rock revetment at the base of the coastal bluff to essentially arrest ongoing marine erosion, and the construction of a 30±-foot-high reinforced shotcrete structural tied-back wall at the top of the bluff to increase the stability of the currently oversteepened and marginally stable upper portion of the bluff. Following removal of the existing talus/debris pile, the rock revetment would be built upwards from the shore platform to elevation 25 feet (MSL Datum) using approximately 3,000 cubic yards of imported 8-ton riprap. The structure would have a crown width of approximately 11 feet and an inclination of 1.7:1.

Upper-bluff stabilization would commence above the Point Loma contact near elevation 57 feet starting behind a 12-foot setback, essentially forming a 12-foot-wide sacrificial bench. This concept is similar to the existing shoreline protection southwesterly of the Administration Building at the entrance to the Plant, and minimizes the visual impact and landform alteration. The free-form structural tied-back shotcrete surface would be carved and otherwise shaped to conform to the natural geologic structure, and will extend from the Point Loma contact up to the top of the bluff, protecting both the terrace deposits and the existing overlying fill soils.

Following complete removal of the existing unsightly debris pile, a smaller-scale and less obtrusive rock revetment would be placed at the base of the bluffs. This singular improvement would result in re-exposing a 35-foot-tall vertical cliffed portion of the bluff, which is currently covered with debris. A chemical stain would then be used, which reacts with the alkalinity in the concrete to provide a mottled natural appearance similar in color to that of the adjacent rock for the upper 30±-foot-high shotcrete wall. These improvements would be essentially identical to the southerly cove adjacent to the Administration Building at the entrance to the Plant, which has been judged by the California Coastal Commission as a premier example of acceptable coastal protection.

FINAL PRELIMINARY REPORT
NORTH SHORELINE PROTECTION IMPROVEMENTS
POINT LOMA WASTEWATER TREATMENT PLANT
SAN DIEGO, CALIFORNIA

CIP NO. 46-175.0

1 INTRODUCTION

The City of San Diego owns and operates the metropolitan sewerage system which currently provides service to a population of approximately 1.5 million customers in San Diego and 16 surrounding municipalities and sewerage districts. The Point Loma Wastewater Treatment Plant (PLWTP) began operations in 1963, and the City now processes approximately 200 million gallons of sewage on a daily basis, and discharges the treated effluent into the ocean through a 9-foot-diameter pipe extending approximately 5 miles offshore to a water depth of 320 feet.

The PLWTP is situated on approximately 37 acres of land, with approximately 2,150 lineal feet of ocean frontage and is located approximately 3/4 miles northerly of the Point Loma Lighthouse. Coastal bluffs in this area rise to over 90 feet above sea level, and many Plant improvements are sited relatively close to the bluffs.

Erosion and cliff retreat are ongoing processes along the San Diego coastline. At the PLWTP, limited amounts of rock slope protection have been placed during, and several times since, construction to help control erosion. By 1984, stone revetments had been placed at the base of approximately 50 percent of the bluffs supporting the PLWTP. By 1987, shoreline erosion had advanced to the point where Gatchell Road was becoming undermined near the entrance to the Plant, and the existing Armco Binwall immediately to the north was similarly being undermined. Shoreline erosion had also encroached upon the lower hydro access road, limiting access for certain maintenance equipment necessary for servicing the lower pump house and outlet structure.

In December 1987, the City of San Diego contracted with Group Delta Consultants, Inc. (GDC) to provide construction documents, environmental documents, and permit acquisition for a variety of shoreline stabilization measures, extending from the southerly entrance to the PLWTP to the northerly end of the lower hydro access road just beyond the lower pump house. That shoreline stabilization work, completed in 1992 (Resolution No. R-269683), stabilized the southerly 1,450± feet of shoreline fronting the Plant.

As we understand, the City now desires to stabilize a limited area of ongoing coastal erosion along the northern boundary of the Plant limits, northerly of the shoreline stabilization work completed in 1992. The site currently houses the sheet metal maintenance building and paint warehouses, and is also used as a staging and work area for miscellaneous Plant activities. A 100±-foot-wide sea cove fronts this area and in previous meetings with MWWD Staff, we have characterized this northernmost cove area as being similar to the southerly cove adjacent the PLWTP Administration Building, referred to as Site 5 in the Shoreline Stabilization Project completed in 1992. Both of these coves originated as sea caves, which formed along existing fault and joint lineaments in the lower cliff-forming bedrock unit. Continuing differential erosion enlarged the sea caves to the point where roof-rock instability resulted in collapse, forming the coves that exist today. These fault/joint-controlled cove areas continue to exhibit significantly higher erosion rates than the remaining unfaulted areas of the bluff within the Plant limits. Previous attempts to retard erosion in these two cove areas included the placement of considerable riprap at the base of the southerly cove in the early 1970s, and miscellaneous construction debris in the northerly cove (presumably placed by the Navy) at some time in the past. Ongoing erosion in the southerly cove area, which threatened Gatchell Road, necessitated the shoreline improvements recently implemented at Site 5. Similar conditions exist in the vicinity of the northerly cove and, as we understand, the City now desires to stabilize this northerly-most portion of the PLWTP.

Bajada/GDC Associates was retained to provide construction documents and the supporting basis of design for the currently-proposed North Shoreline Protection Improvements (NSPI) at the Treatment Plant. This report presents the results of the various engineering support studies, including a geotechnical investigation and coastal bluff retreat study, along with concept designs for the proposed stabilization of the coastal bluffs in this area.

2 SCOPE OF WORK

The scope of work for this project has been performed in general accordance with our Consultant Agreement with Black & Veatch dated December 28, 1994.

Specifically, the scope of work includes that effort necessary for providing construction documents and the supporting technical basis of design for shoreline and upper-bluff stabilization measures considered necessary to stabilize the seaward edge of the coastal bluff in the vicinity of the northwest corner of the plant site. This document describes the results of the various engineering support studies, site information and constraints, and alternative concept designs for shoreline and upper-bluff stabilization in this area. The specific tasks to be accomplished during this phase of work include the following:

- <u>Develop Site Information and Constraints</u>
  - Data Collection from Various Agencies
  - Bathymetry
  - Geotechnical Investigation
  - Estimate of Bluff Retreat
  - Develop Design Waves

- <u>Preliminary Studies</u>
  - Identification and Evaluation of Alternate Design Concepts
  - Concept Designs and Cost Estimates

This report includes the technical background for alternative design concepts considered feasible, and includes preliminary cost estimates associated with each alternative. This report further addresses the effectiveness of proposed coastal protection works, and the necessity for structures within the northerly cove area; probable post-construction maintenance requirements; degree of physical impacts, beneficial or adverse, on abutting property; and impact on existing infrastructure in compliance with City of San Diego standards and Army Corps of Engineers standards.

3 FIELD STUDIES

Field studies, conducted during the period between January and March 1995, included a detailed geologic mapping of the site, the field work associated with the geotechnical investigation, and a detailed assessment of potential site constraints that could impact proposed improvements. Survey work was also conducted to field-edit the topographic base map provided by the City of San Diego, to survey-in geologic contacts, and to develop offshore bathymetry. A more detailed description of the geotechnical and coastal studies is included in Appendix A.

Nearshore sea-floor bathymetry was surveyed, extending 300± yards offshore from the sea cove. We have digitized our recent bathymetric survey, along with the original offshore bathymetric surveys conducted as part of the 1988 GDC studies, onto the City's topographic base, and have provided the offshore bathymetry in digital format in a separate package.

4 GENERAL SITE CONDITIONS

4.1 Existing Improvements

As can be seen on the frontispiece, the northerly limits of the Plant boundary essentially bisect the subject cove with the Navy property, which contains the northerly parking lot and construction staging areas for several ongoing PLWTP improvement projects. This photograph, taken in late 1994, shows the considerable debris at the base of the slope and active erosion undermining the existing improvements in this area. Virtually the entire site, including much of the adjacent Navy property, has been graded since construction of the facility in 1963, resulting in essentially no natural open space areas on the site.

4.2 Geologic Environment

The geologic strata exposed in the face of the coastal bluffs represent two principal geologic units. The lower cliff-forming unit, which extends up to an elevation of approximately 57 feet, is the Cretaceous-age (70 to 90 million years old) Point Loma Formation. Where not affected by fractures and jointing in the rock, this cliff-forming unit is relatively resistant to erosion. The various headlands represent relatively unfractured rock, and the cove areas represent zones of more intense fracturing, which are thus more susceptible to erosion. This cove originated as a sea cave, which formed along existing fault and joint lineaments in the lower cliff-forming bedrock unit. Continuing differential erosion enlarged the sea cave to the point where roof rock instability resulted in collapse, forming the cove that exists today. This fault/joint-controlled cove continues to exhibit significantly higher erosion rates than the remaining unfaulted areas along this portion of the Point Loma Peninsula.

The lower cliff-forming geologic unit has experienced upwards of 45 feet of tectonic uplift in the last 125,000 years, raising the relic abrasion platform to approximate elevation 57 feet. Subsequent deposition of the Bay Point formational soils on top of this wave-abraded surface (approximately 120,000 years ago) mantled the older Point Loma Formation to form a cap of more-erodible marine terrace deposits. Prior to grading for the development of the Plant in the early 1960s, this area sloped toward the bluff-top at a relatively gentle gradient of 12 percent. Subsequently, upwards of 20 feet of fill was placed in the area to create the relatively level present-day topography. A more detailed description of the geologic environment is contained in the Geotechnical and Coastal Studies report, Appendix A.

4.3 Coastal Environment

In evaluating the wave climate that controls coastal erosion, considerable hindcast data are available, which indicate likely future trends. Accordingly, it is feasible to establish geotechnical design criteria for coastal structures. Waves along the San Diego County shoreline generally range in height from 2 to 5 feet; however, large waves ranging from 6 to 10 feet in height are not uncommon. These large waves can arrive at almost any time during the year and may continue for 3 to 4 days. These high-wave episodes are frequently unaccompanied by strong winds. Breakers with estimated heights of 15 to 20 feet have been observed off the coastline within the study area (USCOE, 1960; National Marine Consultants, Inc., 1960).

Seymour, et. al. (1984) have produced storm wave hindcast estimates for the period 1900 - 1984. This resulted in a list of 59 storms in which the resulting offshore significant-wave-height exceeded 3 m (10 feet), all having periods equal to or exceeding 12 seconds. The tropical cyclone of September 1939, a major wave event in southern California, was added for a total of 60 storms.

It is of interest to note that extreme deep-water wave episodes exceeding 6 meters were only reported on eight occasions during the period 1900 - 1979, while the period from February 1980 through February 1984 experienced a total of ten storm events with deep-water waves exceeding 6 meters. Further, the storm of January 17-18, 1988, produced the highest measured deep-water waves approaching the southern California coast. The significant wave height was 10.0 meters (Seymour, 1989), higher than any reported in the 1900 - 1984 database. This storm was likely on the order of a 200-year storm, and was reported by Seymour to be ". . . remarkably similar to Richard Henry Dana's observations in Two Years Before the Mast of the dangerous Southeasters [significant storm arriving from the south] off this same coast during the 1830's."

Continued coastal erosion, in part accelerated by more energetic wave activity during the last 10 years, has subjected the Southern California coastline to a progressively more severe wave energy environment than that experienced during the preceding 40 to 50 years. This historical database is used by most consultants to estimate shoreline erosion rates, which are typically then used to forecast erosion during the useful life of a proposed structure. A more detailed description of the coastal environment is contained in the Geotechnical and Coastal Studies report, Appendix A.

4.4 Shoreline Erosion

The rate of marine erosion for the coves has been estimated as a reasonable multiple of the rate for the sea cliff along the main coastline alignment and from comparison of aerial photographs taken as early as 1939, with recent photographs and mapping for this project.

Significant differences in erosion rates are evident between the relatively linear coastal headland, and the sea coves. This is in part due to the difference in lithology and intensity of jointing, to the wave-direction dependence of transmitting erosive energy into the cove, and to the energy-focusing effect of surge channels.

Aerial photographs taken at irregular intervals from as early as 1939 to the present were compared in estimating the rate of cliff retreat. Our interpretation of these photographs indicates bluff-top retreat rates varying from approximately 0.21 to 0.33 feet per year. Review of aerial photographs further indicates that upper-bluff retreat was primarily due to sloughing, whether by undermining or due to localized slope instability. Thus, the rate of marine erosion in the backs of coves is substantially higher than subaerial erosion.

Preliminary estimates of ongoing marine erosion indicate a long-term annualized rate of bluff-top retreat on the order of 0.2 to 0.3 feet per year at the head of the cove in the project area. Subaerial erosion, primarily from aging drainage facilities that are themselves being undermined, is also contributing to retreat of the bluff-top.

5 DESIGN CONSIDERATIONS

5.1 General Considerations

In order to determine an appropriate shoreline cliff stabilization program, especially in an area of extreme environmental sensitivity, it is important to address: the problems of visual aesthetics; current uses of the area; the present hazards associated with already unstable oversteepened slopes; the potential for future erosion; and the impact of minimal or no stabilization in certain areas. Relevant sections of the California Coastal Act require that coastal protection be limited to only those areas where continued erosion will impact existing improvements. To satisfy this concern, proposed shoreline and upper-bluff stabilization measures have been limited to only the head of the sea cove, where a relatively high erosion rate exists.

5.2 Coastal Erosion

The main erosion problems at the base of the sea cliff in this area are associated with the direct impact of waves and/or wave runup in areas where joints and/or fractures are present in the Point Loma Formation. The sea cove itself has been formed by a northeasterly-trending fault zone and an associated northwesterly-trending joint system, resulting in a relatively wide zone of fractured rock at the head of the existing cove, which is susceptible to ongoing marine erosion. The presence of a large debris pile at the head of the cove, along with the shingle beach, itself partially derived from the debris, has provided temporary protection from direct wave impact and a corresponding temporary reduction in the rate of marine erosion. This debris pile is comprised of two components: 1) concrete and associated rubble derived from demolition of structures (estimated 30 to 50 percent of debris volume); and 2) natural soil slump debris, caused by periodic failure (collapse) of the oversteepened upper-bluff soils (estimated 50 to 70 percent of debris volume).

Placement of the demolition debris, based on photographic reconnaissance, likely occurred, periodically, during the middle to late 1960s. The debris pile has been temporarily effective in mitigating marine erosion; however, marine erosion appears to be rapidly reducing its effectiveness, thus allowing an increase in the rate of erosion in the Point Loma Formation, undermining the overlying Bay Point Formation, and reinitiating progressive collapse of the upper part of the coastal bluff.

Unless some form of lower shoreline stabilization is provided to replace the rapidly eroding debris pile, the rate of bluff-top retreat will again accelerate, resulting in a continual loss of existing bluff-top improvements. Of more concern, however, is the mechanism of upper-bluff retreat, which poses a life-safety issue due to the substantial amount of old fill soils that mantle the upper Bay Point formational soils. When destabilized, these old fill soils tend to collapse in a rapid progression, possibly taking upwards of 10 to 15 feet of bluff-top improvements with the failure. More troublesome is the rapidity with which the failure would occur, which could pose a life-safety issue, were anyone to be in the vicinity of the failure.

Although various structural measures were considered to reduce the rate of marine erosion, rock revetments are considered to be the most appropriate solution, as they currently stabilize over 50 percent of the coastline fronting the Plant boundaries. This structural solution would be most appropriate to maintain visual consistency throughout the Treatment Plant.

5.3 Aesthetics

The existing visual character in this area has been degraded somewhat by the existing debris that currently exists within this cove. The placement of a rock revetment would require that all of the debris be removed and replaced with a more natural-appearing rock structure of considerably smaller scale at the base of the bluffs. This singular improvement would result in re-exposing a 35±-foot-tall vertical cliffed portion of the bluff, which is currently covered with debris.

5.4 Upper-Bluff Stabilization

Once the lower cliffed portion of the coastal bluff has been stabilized, some remedial work is still necessary to improve the stability of the upper portion of the bluff, addressing both the Bay Point Formation and the overlying marginally stable fill soils. A variety of upper-bluff stabilization measures are appropriate for improving the stability and safety of the upper portion of the bluff headward of the sea cove. We have included both structural and nonstructural alternatives and a general description of some of the merits and constraints of each option. The no-project option was not considered as we believe the existing upper portion of the bluff is in a marginally-stable and potentially hazardous condition that should be mitigated.

All of the upper-bluff stabilization measures considered herein address only stabilization of the upper terrace deposits and old fills. Although other options exist, in order to minimize both the scale and cost of this shoreline stabilization project, we have assumed that a rock revetment would be constructed at the base of the coastal bluff, leaving a 35± foot natural vertical-cliffed section of bluff extending up to the Point Loma contact near elevation 57 feet. In order to accommodate the anticipated [albeit reduced] future marine erosion affecting the lower-cliffed section of the bluff, we are proposing to initiate any upper-bluff stabilization measures starting behind a 12-foot setback, measured from the seaward face of the Point Loma contact, essentially forming a 12-foot-wide sacrificial bench. This concept is similar to the existing shoreline protection southwesterly of the administration building at the entrance to the Plant. This 12-foot setback line is shown on the Site Plan, Figure 1.

5.5 Revetment Design

A stable riprap design section requires consideration of such factors as the maximum anticipated deep-water design wave height and wave period that could be expected to occur over the life of the structure. Upon reaching the coastline, the design wave reaches a depth of water so shallow that the waves collapse or break. This depth is equal to about 1.3 times the wave height. During periods of extreme high tide, small swells of approximately 2 to 4 feet in height may actually maintain most of their wave energy and break directly on the structure. During periods of heavy storms, where deep-water wave heights are tens of feet high, these waves break quite a distance offshore, reform as smaller waves, and eventually impart a portion of the original wave energy onto the shore protection structure.

Deep water significant wave height and significant wave period may be determined if wind speed, wind direction and fetch length are known. This information, with water level data, is used with refraction analyses to determine wave conditions at the site. Wave conditions at a site depend critically on the water level and the corresponding sea-floor elevation at the base of the structure. Consequently, knowledge of sea-floor bathymetry and the design still-water level (SWL) must be established to evaluate the wave forces on a coastal structure.

The foreshore slope also affects the height of a particular design wave approaching the coastline. For a given beach elevation at the base of the coastal bluff, a steeper foreshore slope allows a larger wave to break upon the coastal bluff. Local subsurface anomalies in offshore bathymetry may also result in wave focusing or otherwise increased wave forces that may impact upon the bluff.

Our evaluation of the maximum design wave height is based on criteria set forth in the U.S. Army Corps of Engineers Shore Protection Manual (1984 Edition), and the 1980 NOAA published data for combined maximum astronomical tide plus storm surge for a 100-year return period. A maximum still-water level of 6.3 feet (MSL Datum) was selected for design, which includes both the highest high yearly tide, combined with a statistical 100-year storm surge, 1 foot of wave setup, and 1/2 foot of additional height to account for long-term rise in sea level.

6 UPPER-BLUFF STABILIZATION ALTERNATIVES

We have considered a variety of upper-bluff stabilization alternatives, ranging from simply regrading the upper slope to a more stable inclination, to various types of gravity structures, including mechanically-stabilized embankments, to stabilizing the upper bluff by a free-form structural tied-back shotcrete surface similar to the four recently-completed shotcrete walls as described in the 1988 GDC study. As indicated in Section 5.2, Coastal Erosion, all of the upper-bluff stabilization alternatives considered start at the top of the Point Loma contact, near elevation 57 feet, behind a 12-foot-wide setback to maintain economical, yet long-term, stability for the upper-bluff stabilization. Of the 40 feet upper bluff material considered for stabilization, the lower 20± feet, comprised of the Bay Point Formation, is more stable than the overlying fill soils; however, is still susceptible to subaerial erosion. As indicated in Figures 2 through 5, there are a variety of suitable upper-bluff stabilization alternatives, and the alternatives themselves can be combined in many ways. For example, the lower Bay Point formational soils may be cut back to a stable inclination and landscaped, and a structure built on top of the Bay Point Formation, to create the finished seaward edge at the top of the coastal bluff.

The most expensive and durable option would be the free-form structural tied-back shotcrete wall, which would be carved and otherwise shaped to conform to the natural geologic structure. A chemical stain would then be used, which reacts with the alkalinity in the concrete to provide a mottled natural appearance similar in color to that of the adjacent rock. These improvements would be essentially identical to the four walls completed to the south in 1992, which were judged by the California Coastal Commission as a premier example of acceptable coastal protection. Although most expensive, this alternative provides the most long-term stability to the upper portion of the bluff, as well as that of the lower cliffed portion of the Point Loma Formation, due to its lateral restraint originating 100± feet back into the interior bedrock mass of the Point Loma Peninsula. In addition to the ability to reclaim a reasonable amount of additional useable bluff-top square footage, this alternate is likely the easiest to process through the various regulatory agencies in view of the Coastal Commission's strong support for the recent shoreline stabilization efforts conducted at the Plant. It may be of interest to note that, in the November 1993 Coastal Commission hearings in San Diego, the Director of the California Coastal Commission, while referencing photographs of the PLWTP's recently completed Shoreline Stabilization Project, stated in a general message to the audience that this is the type of visual appearance that future shoreline stabilization projects should attempt to achieve.

The various gravity walls, which would include patented products such as Stresswall® and Reinforced Earth®, impose specific foundation loading conditions that must be addressed as part of the overall bluff stability considerations. The 12-foot setback significantly improves foundation performance, especially in view of the existing 2- to 4-foot-thick weathered veneer that comprises the outer face of the vertical cliffed section of the Point Loma Formation. Seismic loading conditions must also be considered, as they affect both the stability of the upper-bluff stabilization alternate and the entire coastal bluff.

At this concept design level, we have tentatively sized all structures and developed preliminary costs assuming a 0.25g earthquake-induced site acceleration, with a structure design factor of safety of 1.2.

The relatively steeply-inclined, mechanically-stabilized embankment (MSE) slope alternative shown on Figure 4 is the least-durable alternate, in part due to its 1:1 slope inclination. The durability of this option increases with a flatter slope inclination. However, there is an attendant loss of useable bluff-top space in the process.

As indicated on the drawings, several options require, or can be designed to accommodate, landscaping, and this also has an attendant O&M cost that may be objectionable.

Product literature has been included in Appendix B for Reinforced Earth® walls, Stresswalls®, and Tensar® (MSE) slope reinforcement. It should also be noted that many of the patented products, especially when 40±-feet high, would be difficult to visually blend into the existing coastal bluff and will likely meet with some resistance during the regulatory process.

The geotechnical factors influencing the design of these various upper-bluff stabilization alternatives include global stability of the slope, bearing capacity, earth pressures, and, for the tied-back wall system, the anchor design and the forces acting on the tied-back wall system. A detailed description of the geotechnical design aspects affecting the various structures is presented in Appendix A.

6.1 Post-Construction Maintenance Requirements

Post-construction maintenance requirements for the various alternatives are presented in the following paragraphs. In all instances, we have assumed that a rock revetment similar to that shown in Figure 2 will be constructed to eliminate the hydrodynamic forces from breaking waves impacting upon the coastal bluff. Although wave runup is occasionally expected to overtop the structure, contributing to limited additional marine erosion, we have not proposed any additional stabilization measures for the remainder of the vertical cliffed section of the Point Loma Formation, which extends up to elevation 57 feet. A 12-foot-wide sacrificial bench has been proposed at the top of the Point Loma contact to allow some additional erosion of the lower cliffed section of the bluff. This sacrificial bench should preclude the possibility of any undermining impacting any of the upper-bluff alternates well beyond the 75-year design life suggested by the Coastal Commission.

No additional post-construction maintenance requirements are anticipated for the rock revetment during its design life. As a flexible rubble mound structure, some consolidation is expected, and some stones may become dislodged during major storm events; however, the bulk of the section will remain confined within the cove area, and its permeable nature will continue to provide suitable dissipation of wave energy well beyond its 75-year design life. It should also be noted that the 1.7:1 inclination recommended for the revetment has been found to be considerably more stable than the original industry-recommended structure inclination of 1.5:1, and the coastal community now routinely recommends this slightly flatter inclination for improved section performance when the expanded footprint requirement does not pose significant environmental impacts, thereby negating its benefit.

6.1.1 *Stresswall Alternate*

The Stresswall, like many of the other patented pre-fabricated component walls, should provide reasonable performance over the design life of the structure. It is slightly more expensive than any of the other proposed patented alternates; however, the pre-cast elements themselves are quite robust and more durable than any of the other MSE structures. The relatively large pre-cast elements minimize the potential for future corrosion-induced degradation to the concrete. The stepped alternate depicted on Figure 2 has been successfully used at various locations in San Diego County, and provides an attractive alternate to a vertical wall, especially when heights on the order of 40 feet are contemplated. As implied in the sketch, the Stresswall counterforts (excluding the bottom counterfort) would all be founded in properly compacted engineered fill soils, and the proposed terrace pad between successive walls could be landscaped, thereby masking the height of the structure. As with any landscaping, there are attendant O&M costs for both sustaining the planting itself, along with the soil exposed on the terraced surfaces, as any erosion of the exposed soil will tend to undermine the upslope wall. Any uncontrolled water that may scour away engineered fills must be precluded and, as with any "plantable" type walls, this potential must be guarded against.

As indicated in Figure 1, all of the proposed upper-bluff stabilization measures extend both southerly and northerly of the existing cove limits, stepping up the sloping coastal terrace surface, which represent the flanks of the sea cove. The Stresswall alternate requires 8-foot vertical steps in both of these return walls necessitating the construction of relatively extensive keyways stepping up along each return. Any uncontrolled drainage that may extend down the face of any of these keyways could easily undermine the edge of any Stresswall termination and judicious control of surface drainage in these areas is necessary to preclude localized erosion within these keyways.

6.1.2 *Tied-Back Wall*

The tied-back wall, and the combination tied-back wall and gravity counterfort wall, are clearly the most expensive; however, they are also the most durable. A relatively thick, high-strength structural concrete section is rather impermeable, especially when using some of the special additives, and the likelihood of reinforcing steel corrosion within the design life of the structure is quite low. The corrosion protection systems currently available for tiebacks are both excellent and dependable, and can essentially provide a guaranteed service life in excess of 100 years. The counterfort design for either the tied-back wall or the counterfort gravity wall also eliminates any potential problems associated with the potential for low density backfill soils, as the wall essentially develops its lateral restraint from a combination of the tied-back anchors and the counterfort foundations resting on the top of the Bay Point formational contact. The tied-back wall alternate, as proposed, does not have any exposed soil to accommodate landscaping; however, the previous shoreline stabilization project to the south has demonstrated that a natural appearance can be maintained, essentially masking the appearance of the structure. As with all of the proposed structural alternates, the two return walls become gravity structures stepping up the sloping coastal terrace surface, requiring at least some attention to uncontrolled surface runoff along the face of the two return walls.

6.1.3 *Regraded Upper Slope*

As indicated on Figure 4, this is clearly the most economical approach to stabilizing the upper bluff; however, this approach places the graded edge of the upper bluff approximately 80 feet easterly of the seaward edge of the Point Loma contact. Although most economical, this alternate also requires that the finished slopes be landscaped, and 40-foot-high landscaped slopes do have certain O&M costs and an ongoing commitment to maintain and revegetate, as necessary, the sloping surface. This option is likely least desirable in view of the limited space available within the Plant boundaries, since considerable usable bluff-top surface is lost, and there is still a significant O&M cost to maintain the integrity and stability of the slope.

6.1.4 *Geogrid-Reinforced Earth Slope*

The geogrid-reinforced earth slope alternate shown on Figure 4, although less expensive than all alternates except the regraded slope described in the previous section, also has relatively high O&M costs. In general, O&M costs increase as the inclination of the geogrid-reinforced slope face becomes steeper. However, the attractiveness of this alternate is to enable the construction of slopes on the order of 1:1 or steeper. As with the previous alternate, the slope face must be landscaped, and the landscaping does require some ongoing effort to maintain its health. One attractive feature of the geogrid reinforcement is that it does provide tensile reinforcement of the MSE volume, making the slope quite stable against deep-seated instability. Maintenance requirements are, in part, a function of the desired level of visual performance, and are greatly reduced if surface waters are precluded from discharging over the slope face. When compared to the regraded upper slope alternate, depending upon the inclination, the presence of the primary and intermediate geogrid synthetic fabric reinforcement helps to stabilize the front face of the slope, significantly reducing the potential for surficial instability.

6.1.5 *Reinforced Earth Retaining Wall*

The Reinforced Earth (RE) alternate undoubtedly provides the most usable bluff-top space for dollars spent; however, it is relatively difficult to mask its massive linear appearance. Several of these walls have been used as coastal protection structures along the San Diego County coastline, and one such structure was recently constructed approximately 400 feet to the south (partway down the lower hydro access road) as part of the 1992 Shoreline Stabilization Project. One of the true values of this type of construction is its flexibility and, in fact, it was chosen for this purpose in the previous shoreline protection project as it spanned the front face of a since-buried sea cove and its flexible structure was a requisite at that location. If a vertical structure were contemplated similar to that shown on Figure 5, it should provide reasonable performance with the same cautionary notes regarding its stair-stepped returns up the sloping coastal terrace surface. Typical RE facing elements are 5-foot by 5-foot in dimension, have cruciform shapes, and ship lapped joints. Similar to the Stresswall, as the RE panels step up the slope, they require stepped keyways to continue the erection of the RE wall. As with all of the other return walls, care must be given to precluding erosive waters flowing down the front face of the RE panels, which, if unchecked, could undermine the return wall.

6.1.6 *Composite Systems*

As indicated in the text, and illustrated on Figures 2 through 5, a variety of upper-bluff stabilization alternatives exist, both in terms of material components and finished slope geometry and alignment. Slope alternates that incorporate landscaping have additional O&M costs attendant with the upkeep of the landscaping, and any of the MSE-type structures must have their MSE volume remain intact to preserve their long-term performance.

6.2 Existing Fill Soils

The entire site area extending easterly to First Street is underlain by a variable thickness fill pad placed on the gently, westerly-sloping coastal terrace surface during the original site grading in 1963, which has resulted in upwards of 20 feet of fill near the westerly edge of the site. This fill is likely unsuitable in its present condition for support of any future improvements that may be contemplated as part of the Plant's master planning considerations. More importantly, the existing fill soil is unsuitable as a reaction for a post-tensioned tied-back structural wall alternate due to its anticipated excessive deformation from post-tensioning anchors. With the exception of the regrading alternate, all of these slope stabilization measures will require removal and recompaction of the outer approximately 20 to 80 feet of the existing fill, including the construction backcut (see Figures 2 through 5). Any subsequent site remediation work that may be contemplated as part of any future site improvements may, if in close proximity to the proposed shoreline stabilization improvements, compromise these upper-bluff stabilization measures.

It is critically important that any future contemplated site improvements be conducted with due consideration given to any upper-bluff stabilization measures implemented as part of the currently proposed NSPI project. In discussions with personnel from both Black & Veatch and the City of San Diego, it was agreed that the removal and proper recompaction of this entire 1960's vintage fill be included as part of the NSPI project in order to eliminate any future site remediation work that may be considered necessary. We suggested this additional earthwork in consideration of the current master plan for the plant, which proposes a new North Operations Building be located in this area. The fill would be replaced to elevation 87 feet adjacent to the wall, reducing the depth of fill to 10± feet at this location. This will also reduce the cost of any upper bluff stabilization measures. The net volume of fill to be moved is estimated at 5200 cubic yards.

We have offered the previous discussion in view of our own experience at the Plant with ongoing improvements conflicting with previously constructed Plant projects. We would like to illustrate this with several examples in hopes that any contemplated future site work in this area does not in any way adversely affect the NSPI project.

- The Sludge Pumping Station currently under design by Metcalf and Eddy, to be located at the "Y" intersection of Gatchell Road, First Street, and Second Street contemplates a 30-foot excavation to accommodate the new pump house. This excavation conflicts with four existing post-tensioned tieback anchors, which provide support for the adjacent 35-foot-high structural shotcrete tied-back wall fronting the coastal bluff. Fortunately, these conflicts were recognized early in the design process, and the original design team members of the tied-back wall were retained to assist in the redesign of these anchors to accommodate the currently proposed sludge pumping station.

- The Digestor Upgrades and Expansion project included a pump house to be located on the lower hydro access road near the southerly edge of the tied-back shotcrete seawall constructed as part of the Site 2 improvements for the recently constructed Shoreline Stabilization Project. The geotechnical consultant for that project was contemplating a drilled pier foundation for the pump house, with the possibility existing for potential conflicts with the southerly most set of tiebacks for the adjacent slope stabilization project. As original designer of the tied-back wall, we were happy to assist in the resolution of this matter.

As indicated in the two previous examples, conflicts can and do occur, and the available space within the Plant boundaries necessitates maximum utilization of all of the available land, which is bound to result in conflicts with existing improvements on site. Tied-back structures, although highly efficient, do result in underground embedded anchors that may exceed 100 feet in length. If utilized, it is important that these anchors be considered in any future site improvements contemplated within the influence of the anchors. Similarly, the other upper-slope stabilization alternates, particularly the mechanically-stabilized embankment (MSE) type products depicted on Figures 4 and 5, utilize either geosynthetic or steel reinforcing strips embedded throughout the MSE volume, which, if compromised, reduces stability of the MSE structure.

A corollary concern is best illustrated with the geogrid-reinforced earth slope alternate shown on Figure 4. The MSE volume necessary to support a 1:1 inclination, with appropriate surcharge loading conditions, would be as shown on Figure 4. The base of the MSE volume would extend approximately 62 feet in from the seaward edge of the Point Loma contact, and the top of the MSE volume would extend approximately 88 feet in from the face of the Point Loma contact. As previously indicated, the existing 1960's vintage fill in this area may be deemed as unsuitable for future improvements, or the improvements themselves may encroach upon the MSE volume. If the fill were to be removed and recompacted, or the MSE volume itself compromised to accommodate some future improvement, the geogrid-reinforced earth slope would likely require redesign and possibly reconstruction to accommodate some future need.

The tied-back wall alternate shown on Figure 3 may be the least sensitive structural alternative, since the wall itself would not be compromised with any subsequent removal of existing fill soils. This alternate, however, does require that any future site utilization recognize the presence of existing embedded anchors that may extend 100 feet into the pad, albeit well below the top of the Bay Point formational contact. Any deep excavations to accommodate some subterranean structure similar to the sludge pump station, or any drilled pier foundations to accommodate some future utilization of the site, should include a careful evaluation of the existing subsurface components constructed as part of the NSPI project.

At this point, it should be noted that the estimated total project cost listed in Section 8 assumes all upper-bluff stabilization alternates to be 40 feet in height, essentially extending from elevation 57 to 97 feet. We have also analyzed a 30-foot-high tied-back wall alternate, which may be an attractive option in view of the current master plan considerations in this area. Total project cost for shoreline stabilization in this area will obviously be a function of the ultimate plan bluff-top elevation, and these costs will be refined as part of the 60 percent submittal, pending final City recommendations.

6.3 Site Drainage

Approximately 1.1± acres of upland watershed currently drains to this portion of the coastline, with all surface water discharging over the coastal bluff near the northwest corner of the site, which, over the last 30 years, has been the principal contributor to the significant incised gully in this area. Additionally, an existing 24-inch RCP drain line exits the face of the coastal bluff near the southerly margin of the sea cove, with the pipe invert elevation near 81 feet. Although surface drainage has extensively incised the original engineered fill placed in this area, the existing 24-inch drain line has also caused severe erosion of the upper bluff along the southerly margin of the sea cove. Site drainage in this area must be improved as part of the NSPI project in order to avoid damage to existing improvements from inadequately controlled surface runoff. We would suggest that all site drainage be directed to a suitable drainage structure 30± feet landward of the seaward edge of the selected bluff-top remedial measures. We anticipate that the structure would consist of a vault with an angle-bored drain line exiting the lower cliffed section of the coastal bluff near the top of the proposed revetment at about elevation 25 feet. This approach to accommodating site drainage is essentially the same as that installed at Site 5 of the 1992 Shoreline Stabilization Project. We would further suggest that the existing 24-inch drain line also be rerouted to drain into the proposed drainage vault.

7 CONSTRUCTION CONSTRAINTS

The PLWTP is a continuously-operating facility having need for continual Plant access. It will be necessary for staging areas to be made available to the contractor adjacent to the face of the bluff. The contractor will be required to cooperate with Plant personnel and maintain access along First Street and, presumably, if staging off the Navy property, will have additional requirements imposed by the Navy.

8 RECOMMENDED SHORELINE STABILIZATION PROJECT

The preferred shoreline and upper-bluff stabilization approach within the PLWTP boundaries limits proposed improvements to only those areas where continued erosion within the next 75 years will likely impact existing improvements. Specific wall types and dimensions described herein have been selected based on estimates of future erosion rates, the cost and effectiveness of proposed coastal works, and the long-range needs of the Plant. Erosion rates have been evaluated from a geologic inventory of erosion processes along the bluffs, a review of available historical aerial photographs further corroborating erosion processes, and discussions with Plant personnel eliciting their past experience with erosion processes in the Plant area.

Based on discussions with Plant personnel, we have developed the following proposed shoreline and upper-bluff stabilization measures, in part due to the durability, long-term performance, and minimum maintenance requirements, all of which are considered to be important issues when considering the long-term stability of the coastal bluff.

At present, we are recommending that the finish grade adjacent the top of the coastal bluff be lowered to elevation 87± feet, in order to reduce the scale of the proposed stabilization measures and to accommodate the long-term master plan objectives for this area. The proposed improvements include a rock revetment at the base of the coastal bluff (Figure 2) to essentially arrest ongoing marine erosion, and the construction of a 30±-foot-high reinforced shotcrete structural tied-back wall at the top of the bluff (Figure 3) to increase the stability of the currently oversteepened and marginally stable upper portion of the bluff. As indicated on Figure 1, the base of the wall would essentially follow the 12-foot setback; however, both ends of the shotcrete wall would extend beyond the width of the sea cove, stepping up and keying into the more competent intact Bay Point formational soils extending beyond either edge of the sea cove alignment. The total length of the proposed tied-back wall would be approximately 130 feet, and would result in a new useable bluff-top configuration, essentially as shown on Figures 1. We are also proposing drainage improvements, essentially as described in Section 6.3, incorporating a single inlet structure and diagonally-bored gravity drain line exiting at the top of the rock revetment, essentially similar to that which currently exists immediately southerly of the PLWTP Administration Building.

The lower rock revetment is proposed to have a crown elevation of 25 feet (MSL datum), be approximately 100 feet in length (the width of the sea cove), and consist of 8-ton armor stone constructed at an inclination of 1.7:1. The approximate geometry of this revetment is illustrated on Figure 2. All of the existing debris would be removed from the head of the cove, and the existing shingle beach would be removed within the footprint of the proposed revetment and the revetment itself keyed a minimum of 2 feet into the Point Loma shelf rock comprising the base of the sea cove.

The structural shotcrete wall will incorporate a 3½-foot-high concrete parapet designed to visually blend into the existing coastal bluffs, and designed to accommodate impact loading from HS-20 vehicular traffic impacting the wall at a speed of 10 miles per hour.

9 DESIGN CRITERIA

9.1 Stone Revetments

Sea-floor bathymetry in front of proposed structure is on the order of elevation of +1 foot (MSL datum). The average nearshore slope extending out a distance of 600± feet is on the order of 40:1. The sea-floor materials are comprised of the sedimentary rocks of the Point Loma Formation, and are judged to be relatively erosion resistant. For design purposes, we have assumed 1 foot of additional erosion from gravel abrasion at the base of structure.

A maximum design still-water level of 6.3 feet (MSL Datum) was selected for design which includes both the highest high yearly tide, combined with a statistical 100-year storm surge, 1 foot of wave setup, and 1/2 foot of additional height to account for long-term sea level rise. This results in a maximum design breaker height of approaching 8 feet, and a required minimum stone size of 6 tons. We would suggest specifying 8-ton stone, since it is more readily available (this is a Caltrans and County of San Diego specification rock size). Runup calculations indicate that design storm waves with periods on the order of 6 seconds may result in upwards of 25 feet of wave runup above the design still-water level reaching elevations of over 30 feet.

9.2 Tied-Back Walls

Surface preparation of the bluff would consist of the removal of small protrusions which would be difficult to install reinforcement around. Slots would be cut in the bluff surface at the vertical edges of the concrete in order to key the concrete into the bluff. Pockets of rock would be cut from the bluff face at the anchor locations so as to provide additional concrete thickness where tieback anchor stresses applied to the shotcrete skin are the greatest.

A matrix of holes on approximately a 12± foot square grid would be drilled to a depth of about 100± feet in the bluff face. High strength, multi-strand, prestressing tendons utilizing a double corrosion protection system would be installed in these holes and grouted.

Epoxy-coated reinforcing steel would be attached to the bluff surface. Two layers of steel reinforcing would be used at anchor locations. Colored shotcrete, a form of pneumatically installed concrete, would be blown on the bluff surface to encase the reinforcing steel in a minimum 12-inch-thick shotcrete wall. After the shotcrete has cured, the tiebacks will be tensioned and locked-off at their design loads to prevent development of active pressures in any fractured rock behind the wall. The anchorage pockets would then be filled to match the surrounding shotcrete to complete the wall.

10 ESTIMATED TOTAL PROJECT COST

Estimated costs for construction of proposed improvements described herein were developed in dollars per facial square foot for total in-place cost for all wall elements; in dollars per cubic yard for in-place cost of all reinforced or graded slopes and riprap. The cost estimates include installation of all elements of a proposed concept, plus any required grading and/or landscaping. (Grading would include the construction backcuts necessary to accommodate the alternates shown in Figures 2 through 5, respectively). It should be noted that these costs are based on manufacturer's suggested prices and present contractor's average installation cost. These costs could vary somewhat depending on availability, suppliers, and bidding costs. A breakdown of total estimated project costs is provided in Table 1. Unit costs used in our estimate of construction costs are listed below:

Estimated Unit Costs

ITEM	UNIT COST
Rock Revetment Lower Shoreline Protection	\$73/ton (\$110/cu.yd.)
Stresswall®	\$44/sq.ft.
Tied-Back Wall	\$124/sq.ft.
Tied-Back Wall and Counterfort Gravity Wall Combination	\$117/sq.ft.
Regrade Upper Slope	\$10/cu.yd.
Geogrid-Reinforced Earth Slope	\$31/cu.yd.
Reinforced Earth Retaining Wall	\$40/sq.ft.

TABLE 1
ESTIMATED PROJECT COSTS

ROCK REVETMENT LOWER SHORELINE PROTECTION		
Earthwork (removal of talus and debris):		
3000 cu.yd. @ $20/cu.yd.	$	60,000
Riprap:		
4500 tons @ $60/ton		270,000
Total Rock Revetment ...	$	330,000
STRESSWALL®		
Structure Excavation and Backcut:		
4075 cu.yd. @ $10/cu.yd.	$	40,750
4400 sq.ft. Stresswall @ $35/sq.ft.		154,000
Total Stresswall	$	194,750
TIED-BACK WALL		
Earthwork (excavation & recompact fill within 15 feet of wall):		
920 cu.yd. @ $20/cu.yd.	$	18,400
Tied-Back Wall:		
4400 sq.ft. @ $120/sq.ft.		528,000
Total Tied-Back Wall	$	546,400
30-FOOT-HIGH TIED-BACK WALL		
430 cu.yd. @ $20/cu.yd.	$	8,600
3300 sq.ft. @ $120/sq.ft.		396,000
Total Tied-Back Wall	$	404,600
TIED-BACK WALL AND COUNTERFORT GRAVITY WALL		
Earthwork (excavate and recompact fill within 15 feet of wall):		
920 cu.yd. @ $20/cu.yd.	$	18,400
Tied-Back Wall:		
2750 sq.ft. @ $120/sq.ft.		330,000
Counterfort Wall:		
1650 sq.ft. @ $100/sq.ft.		165,000
Total Tied-Back Wall and Counterfort..	$	513,400
REGRADE UPPER SLOPE		
Excavation:		
6000 cu.yd. @ $10/cu.yd.	$	60,000
Total Regrade Upper Slope	$	60,000
GEOGRID-REINFORCED EARTH SLOPE		
Structure Excavation:		
10,312 cu.yd. @ $8/cu.yd.	$	82,496
Reinforced Earth:		
7645 cu.yd. @ $20/cu.yd.		152,900
Total Geogrid-Reinforced Earth Slope	$	235,396

TABLE 1
ESTIMATED PROJECT COSTS

(continued)

REINFORCED EARTH RETAINING WALL		
Structure Excavation:		
6519 cu.yd. @ $10/cu.yd.	$	65,190
Reinforced Earth Wall:		
4400 sq.ft. @ $25/sq.ft.		110,000
Total Reinforced Earth Retaining Wall	$	175,190
REINFORCED EARTH WALL WITH GEOGRID-REINFORCED EARTH SLOPE		
Structure Excavation:		
6519 cu.yd. @ $10/cu.yd.	$	65,190
Reinforced Earth Slope:		
1527 cu.yd. @ $20/cu.yd.		30,555
Reinforced Earth Wall:		
2750 sq.ft. @ $25/sq.ft.		68,750
Total RE Wall w/Geogrid-Reinforced ...	$	164,495

REFERENCES

BOOKS, ARTICLES, AND REPORTS

1. Abbott, P.L., Ed., 1982, "Geologic studies in San Diego," prepared for San Diego Association of Geologists Field Trips, April 1982, 160 pp.

2. Anderson, J.T., Rockwell, T.K., and Agnew, D.C., 1989, "Past and possible future earthquakes of significance in the San Diego region," Earthquake Spectra, v. 5, no. 2, p. 299-333.

3. Apex Geotechnology/Group Delta Consultants, January 1994, Geotechnical Design Criteria for Scum Improvement Project, Point Loma Metropolitan Wastewater Treatment Plant, San Diego, California

4. Benton Engineering, Inc., 1960, "Soil and foundation investigation, sewer treatment plant site, City of San Diego, California."

5. Bird, E.C.F., 1985, "Coastline changes - a global review," John Wiley and Sons, 219 pp.

6. Bowden, K.F., 1983, "Physical oceanography of coastal waters," Halsted Press, 302 pp.

7. Campbell, K.W., 1987, Predicting strong ground motion in Utah, in Gori, P.L., and Hays, W.W., eds., Assessment of regional earthquake hazards and risk along the Wasatch Front, Utah, U.S. Geological Survey Open-File Report 87-585, v. II, Sect. 1, p. 1-90.

8. Curray, J.R., 1965, Late Quaternary history; continental shelves of the United States, p. 723-735 in H.E. Wright, Jr. and D.G. Frey (eds), *The Quaternary of the United States*, Princeton Univ. Press, 922 p.

9. Curray, J.R., 1961, Late Quaternary sea level: a discussion. Geological Society of America Bulletin 72, p. 1707-12.

10. Curray, J.R., 1960, Sediments and history of Holocene transgression, continental shelf, northwest Gulf of Mexico, p. 221-266, in F.P. Shepard, F.B. Phlefer, and Tj.H. van Andel (eds), *Recent Sediments, Northwest Gulf of Mexico*, 1951-1958, Amer. Assoc. Petroleum Geologists, Tulsa, Oklahoma, 394 p.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

11. Ellis, M.Y., Ed., 1978, "Coastal mapping handbook," U.S. Government Printing Office, Washington, 113 pp., Appendices.

12. Emery, K.O., and Aubrey, D.G., 1991, Sea levels, land levels, and tide gauges: Springer-Verlag Publishers, New York, NY, 237 p., 113 figures.

13. Emery, K.O., and Kuhn, G.G., 1980, "Erosion of rock shores at La Jolla, California," Marine Geology, 37, pp. 197-208.

14. Emery, K.O., and Kuhn, G.G., 1982, "Sea cliffs: their processes, profiles, and classifications," Geological Society of America Bulletin, v.93, pp. 644-654.

15. Fang, Hsai-Yang (ed.), 1991, Foundation Engineering Handbook, 2nd Edition, Van Nostrand Reinhold.

16. Fulton, K., 1981, "A manual for researching historical coastal erosion," Report No. T-CSGCP-003, University of California, Santa Cruz, Writing Program, published by California Sea Grant College Program, 56 pp.

17. Fulton-Bennet, K., and Griggs, G.B., undated, "Coastal protection structures and their effectiveness," jointly published by California Department of Boating and Waterways, and the Marine Sciences Institute, 48 pp.

18. Gastil, G., and Higley, R., 1977, "Guide to San Diego area stratigraphy," prepared for AAPG-SEG Petroleum Exploration School Field Trip, December 10, 1977, Published by The Edwin C. Allison Center, 62 pp.

19. Geotechnical Consultants Inc., May 1988, Geotechnical Investigation, Sludge Pumping Building at Point Loma Wastewater Treatment Plant, San Diego, California.

20. Geotechnical Consultants, Inc., September 1988, Geotechnical Investigation, Sludge Dewatering Pad, Point Loma Wastewater Treatment Plant.

21. Geotechnical Consultants, Inc., February 1980, Geotechnical Reconnaissance, proposed Point Loma Wastewater Treatment Plant excavation, accelerated projects, City of San Diego, California.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

22. Group Delta Consultants Inc., November 1992, Vortex Structure Tie Back Design Criteria, Point Loma Wastewater Treatment Plant, San Diego, California.

23. Group Delta Consultants Inc., June 1991, Construction-term Stability and Long-term Lateral Loading, Throttling Valve Structure, Point Loma Wastewater Treatment Plant, San Diego, California.

24. Group Delta Consultants, Inc., May 1988, Shoreline Protection Project, Point Loma Treatment Plant, San Diego, California.

25. Hileman, J.A., 1979, Seismicity of the San Diego region in Abbott, P.L., and Elliott, W.J. (eds), Earthquakes and other perils, San Diego region, p. 11-20.

26. Horrer, P., 1960, Southern hemisphere swell and waves from a tropical storm at Long Beach, California: Beach Erosion Board Bulletin, v. 4, no. 1, 18 p.

27. Inman, D.L., R.T. Guza, D.W. Skelly, and T.E. White, 1986, "Southern California coastal processes data summary," Ref. No. CCSTWS 86-1, Feb 1986, U.S. Army Corps of Engineers, 572 pp.

28. Inman, D.L., and Veeh, H.H., 1966, Dating the 10-fathom terrace off Hawaii. American Geophysical Union, Trans. 47, 125.

29. Jennings, C.W., 1975, Fault map of California: California Division of Mines and Geology Geologic Data Map No. 1.

30. Kennedy, M.P., 1973, Seacliff erosion at Sunset Cliffs, San Diego: Geology, v. 26, no. 2.

31. Kennedy, M.P., 1975, "Geology of the San Diego metropolitan area, California," California Division of Mines and Geology, Bulletin 200, 56 p., map scale 1"=2000'.

32. Kennedy, M.P., Clark, S.H., Greene, H.G., and Legg, M.R., 1980, Recency and character of faulting offshore metropolitan San Diego, California: California Division of Mines and Geology Map Sheet 42.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

33. Kuhn, G.G., and Shepard, F.P., 1979, "Accelerated beach-cliff erosion related to unusual storms in Southern California," California Geology, March 1979, pp. 58-59.

34. Kuhn, G.G., 1980, "Greatly accelerated man-induced coastal erosion and and new sources of beach sand, San Onofre State Park and Camp Pendleton, northern San Diego County, California," Shore and Beach, October 1980, pp. 9-13.

35. Kuhn, G.G., and Shepard, F.P., 1984, "Sea cliffs, beaches, and coastal valleys of San Diego County, California," University of California Press, 193 pp.

36. Kuhn, G.G., and Shepard, F.P., 1983, "Beach processes and sea cliff erosion in San Diego County, California," in: CRC Handbook of Coastal Processes and Erosion (Chapter 13), Paul D. Komar, ed., CRC Press, Inc., Boca Raton, FL, pp. 267-284.

37. Kuhn, G.G., and Shepard, F.P., 1980, "Coastal erosion in San Diego County, California," Proceedings of the Conference: "Coastal Zone '80," American Society of Civil Engineers, pp. 1899-1918.

38. Le Mehaute, B., and J.D. Wang, 1982, "Wave spectrum changes on a sloped beach," Jour. Water, Port, Coastal and Ocean Div., Amer. Soc. Civil Eng., v. 108, p. 33-47.

39. Lee, Louis, Pinckney, C.J., and Bemis, C., 1976, "Sea cliff base erosion," American Society of Civil Engineers, National Water Resources and Ocean Engineering Convention, Preprint 2708, 14 pp.

40. Legg, Mark, and Agnew, D.L., 1979, The 1862 earthquake in San Diego in Abbott, P.L., and Elliott, W.J. (eds), Earthquakes and other perils, San Diego region, p. 139-142.

41. Marine Advisers, 1961, "A statistical survey of ocean wave characteristics in Southern California waters," prepared for the U.S. Army Corps of Engineers, Los Angeles District, Marine Advisers, Inc., La Jolla, CA, 30 pp.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

42. Masters, P.M., and N.C. Fleming, 1983, Quaternary coastlines and marine archaeology: towards the prehistory of land bridges and continental shelves: Academic Press, New York, 641 p.

43. May, K.D., 1987, "Historical chronology of extreme storm and wave events in San Diego County," prepared for Moffatt and Nichol, Engineers, 37 pp.

44. McGrath, J., Ed., 1985, "California's battered coast - proceedings from a conference on coastal erosion," San Diego, February 6-8, 1985, Sponsored by California Coastal Commission, 403 pp.

45. Munk, W.H., and M.A. Traylor, 1947, "Refraction of ocean waves, a process linking underwater topography to beach erosion," Jour. Geol., v. 55, n. 1, p. 1-26.

46. Munk, W.H., G.H. Miller, F.E. Snodgrass, and N.F. Barber, 1963, "Directional recording of swell from distant storms," Phil. Trans. Roy Soc. London, A, v. 55, n. 1062, p. 505-584.

47. National Marine Consultants, Inc., 1960, Wave statistics for seven deep water stations along the California coast. Prepared for the U.S. Army Corps of Engineers, Los Angeles District.

48. Naval Facilities Engineering Command, 1982, "Coastal protection, design manual 26.2," 314 pp., references, glossary, symbols.

49. National Oceanic and Atmospheric Administration, 1980, A climatology and oceanographic analysis of the California Pacific outer continental shelf region.

50. Ninyo and Moore Inc., February 1994, Report of Geotechnical Evaluation, Point Loma Wastewater Treatment Plant Trailer Complex, San Diego, California.

51. Norris, R.M., 1968, Sea cliff retreat near Santa Barbara, California: California Division of Mines and Geology, Mineral Information Service, v. 21, No. 6, p. 87-91.

52. Pawka, S.S., and R.T. Guza, 1983, "Coast of California waves study - site selection," SIO Ref. 83-12, 51 pp., Scripps Inst. of Oceanogr., La Jolla, Calif.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

53. Pease, R.C., 1979, Scarp degradation and fault history south of Carson City, Nevada: University of Nevada, Reno, Masters Thesis, 90 p.

54. Richter, C.F., 1958, Elementary seismology: W.H. Freeman, San Francisco, 768 p.

55. Ross, A., and Dowlen, R.J., Eds., 1973, "Studies on the geology and geologic hazards of the greater San Diego area, California," San Diego Association of Geologists, 152 pp.

56. Schnabel, P.B., and Seed, H.B., 1973, Accelerations in rock for earthquakes in the western United States: Bulletin of the Seismological Society of America, v. 63, no. 2, p. 501-516.

57. Seed, Bolton, and Whitman, Robert V., 1970, "Design for earth retaining structures for dynamic loads," in Lateral stresses in the ground and design of earth-retaining structures, American Society of Civil Engineers, New York, p. 103-147.

58. Seymour, R.J., and M.H. Sessions, 1976, "A regional network for coastal engineering data," Proc. 15th Coastal Eng. Conf., Amer. Soc. Civil Eng., p. 60-71.

59. Seymour, R.J., R.R. Strange, III, D.R. Cayan, and R.A. Nathan, 1984, "Influence of El Ninos on California's wave climate," Proc. 19th Coastal Eng. Conf., Amer. Soc. Civil Eng., p. 577- 592.

60. Seymour, R.J., 1989, Wave observations in the storm of 17-18 January 1988. Jour. of Amer. Shore and Beach Preservation Assn., v. 57, n. 4, p. 10-14.

61. Shepard, F.P., and Grant, IV, U.S., 1947, "Wave erosion along the Southern California coast," Geological Society of America Bulletin, v.58, pp. 919-926.

62. Slemmons, David B., 1977, State-of-the-art for assessing earthquake hazards in the United States, Report 6, Faults and earthquake magnitudes, miscellaneous paper S-73-1, U.S. Army Engineer Waterways Experiment Station 95 p.

63. Sunamura, T., 1977, A relationship between wave-induced cliff erosion and erosive forces of wave. J. Geol. 85, p. 613-18.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

64. Trenhaile, Alan S., 1987, "The geomorphology of rock coasts," Clarendon Press, Oxford, 384 pp.

65. U.S. Corps of Engineers, Los Angeles District, March 1, 1960, "Beach erosion control report, A cooperative study of San Diego County, California," Appendix D, Plate 8.

66. U.S. Army Corps of Engineers, 1987, "Coast of California, storm and tidal waves study - coastal cliff sediments, San Diego region," 21 sectins, plates.

67. U.S. Army Corps of Engineers, 1986, "Coast of California, storm and tidal waves study - Southern California coastal photography and beach profile index," 66 pp., plates.

68. U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - geotechnical data inventory, Southern California," 3 sections, references, plates.

69. U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - hydraulic data inventory," 35 pp.

70. U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - hydrologic data inventory, Southern California coastal zone," 102 pp., appendices.

71. U.S. Army Corps of Engineers, 1985, "Coast of California, storm and tidal waves study - shoreline movement data report, Portugese Point to Mexican border (1852-1982)," 48 pp., appendix.

72. U.S. Army Corps of Engineers, 1984, "Coast of California, storm and tidal waves study - geomorphology framework report, Dana Point to the Mexican border," 9 sections, references, appendices.

73. U.S. Army Corps of Engineers, 1977, Geologic investigation of Segment A at Sunset Cliffs, San Diego, California.

74. U.S. Army Corps of Engineers, 1984, Shore Protection Manual, U.S. Army Coastal Engineering Research Center, Fort Belvoir, Virginia, v. I and II.

REFERENCES

BOOKS, ARTICLES, AND REPORTS (continued)

75. U.S. Army Corps of Engineers, 1970, Specifications for Shore Protection for Ocean Beach Area, Sunset Cliffs Segment B, San Diego, California, U.S. Army Engineer District, Los Angeles, California.

76. U.S. Army Corps of Engineers, 1965, Special study of City of San Diego (Sunset Cliffs): Beach Erosion Control Report on Coast of Southern California, Contract No. W04193ENG5196, U.S. Army Engineer District, Los Angeles, California.

77. Wallace, R.E., 1977, Profiles and ages of young fault scarps, north-central Nevada: Geological Society of America Bulletin, v.88, p.1267-1281.

78. Woodward-Clyde Consultants, February 1987, Pavement Design of Crane Areas, Point Loma Wastewater Treatment Plant, San Diego, California.

79. Woodward-Clyde Consultants, August 1981, Design memorandum, geotechnical and civil engineering design services for Sunset Cliffs shoreline and upper cliff stabilitation project, San Diego, California.

VERTICAL AERIAL PHOTOGRAPHS

Nov/Dec, 1987	-	Pountney & Associates/B&W
February, 1985	-	USGS-EROS/Color
March, 1982	-	NOAA/Color
November, 1981	-	USGS-EROS/B&W
April, 1978	-	California Department of Water Resources/B&W
November, 1976	-	California Department of Water Resources/B&W
September, 1972	-	USGS-EROS/B&W
October, 1966	-	USGS-EROS/B&W
August, 1966	-	NOAA/B&W
September, 1964	-	U.S. Department of Agriculture/B&W
September, 1960	-	NOAA/B&W
May, 1953	-	U.S. Department of Agriculture/B&W
November, 1950	-	NOAA/B&W
1949	-	U.S. Department of Agriculture/B&W
1939	-	USGS - NCIC

REFERENCES

MAPS AND CHARTS (continued)

July 1858, "Plat of the Pueblo Lands of San Diego," U.S. Surveyor General, scale 1":80 chains.

1856, "Hydrography of San Diego Bay and vicinity," U.S. Coast Survey, scale 1:10,000.

1853, "San Diego entrance and approaches," U.S. Coast Survey, scale 1:20,000.

1852, "Map of False Bay near San Diego, California," U.S. Coast Survey, scale 1:10,000.

1851, "Map of San Diego Bay, California, from Point Loma to New Town," U.S. Coast Survey, scale 1:10,000.

1851, "San Diego Harbor, California," U.S. Coast Survey, scale 1:10,000.

Undated (prior to 1848), untitled map of San Diego area - Spanish, scale in varas.

GLOSSARY OF TERMS

BLOCKFALL: Rapid decent of a large angular rock fragment derived from breaking of the parent rock mass, usually along joints.

BLUFF (COASTAL BLUFF): The rising ground bordering the sea, which may include a sea cliff, but is characterized by an upper, moderately-sloping, section ending at a coastal terrace.

BLUFF-TOP: The boundary between the bluff and the coastal terrace.

BLUFF-TOP RETREAT: Landward migration over time of the bluff-top caused by marine erosion on the sea cliff and subaerial erosion of the bluff.

CAUSTIC: In refraction of waves, the name given to the curve to which adjacent wave rays refracted by a bottom whose contour lines are curved, are tangents. The occurrence of a caustic always marks a region of crossed wave rays and high wave convergence.

CLAPOTIS: Nonbreaking waves.

CLIFF-PLATFORM JUNCTION: The location at the base of the sea cliff where the near-horizontal shore platform meets the near-vertical sea cliff.

COASTAL TERRACE: Any long, narrow, relatively level surface bounded along the shoreward edge by a sea cliff and along the landward edge by ascending slopes.

DIURNAL: Having a period or cycle of approximately 1 tidal day.

EROSION: The mechanical destruction of the land or sea floor, and the removal of rock and soil by running water, waves and currents, moving ice, wind, and gravity. It includes the processes of weathering, solution, corrosion, and transportation.

FETCH: The horizontal distance (in the direction of the wind) over which a wind generates seas.

FORESHORE ZONE: A part of the shore lying between the upper limit of wave wash at high tide and the low water mark. The foreshore is usually traversed by the uprush and backrush of waves; however, the foreshore is typically absent at the site.

GEOMORPHOLOGY: That branch of both physiography and geology which deals with the form of the earth, the general configuration of its surface, and the changes that take place in the evolution of landform.

GLOSSARY OF TERMS

GRABEN: An elongate, relatively downthrown, crustal unit or block that is bounded by faults on its long sides.

HEADLAND (HEAD): A high steep-faced promontory extending into the sea.

HORST: An elongate, relatively uplifted crustal unit or block that is bounded by faults on its long sides.

INSHORE ZONE: A zone of variable width extending from the low water line at the shore to the seaward edge of the breaker zone.

NEAP TIDE: A tide occurring near the time of quadrature of the moon with the sun. The neap tidal range is usually 10 to 30 percent less than the mean tidal range.

NEARSHORE ZONE: An indefinite zone extending seaward from the shoreline well beyond the breaker zone.

REFRACTION: The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than the part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours.

SEA CLIFF: A more or less continuous line of seaward-facing, high, steep rock faces or precipices that are caused by marine and subaerial erosion.

SEMIDIURNAL TIDE: A tide with two high and low waters in a tidal day.

SHORE: The narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a beach.

SHORE PLATFORM: A gently seaward-sloping, but relatively level, surface produced by wave erosion and extending far into the sea.

STANDING WAVES: Nonbreaking waves.

STILL-WATER LEVEL: The elevation that the surface of the water would assume if all wave action were absent.

GLOSSARY OF TERMS

SUBAERIAL EROSION: Erosion that occurs on the land surface due to removal of surface material by wind, water, and gravity in its broadest sense. This also includes the weathering process which produces more erodible material. Contrasted with marine erosion.

WASTING: The gradual destruction or wearing away of a landform surface by wind, gravity, and rill wash, but excluding subaqueous erosion.

WAVE HEIGHT: The vertical distance between a crest and the preceding trough.

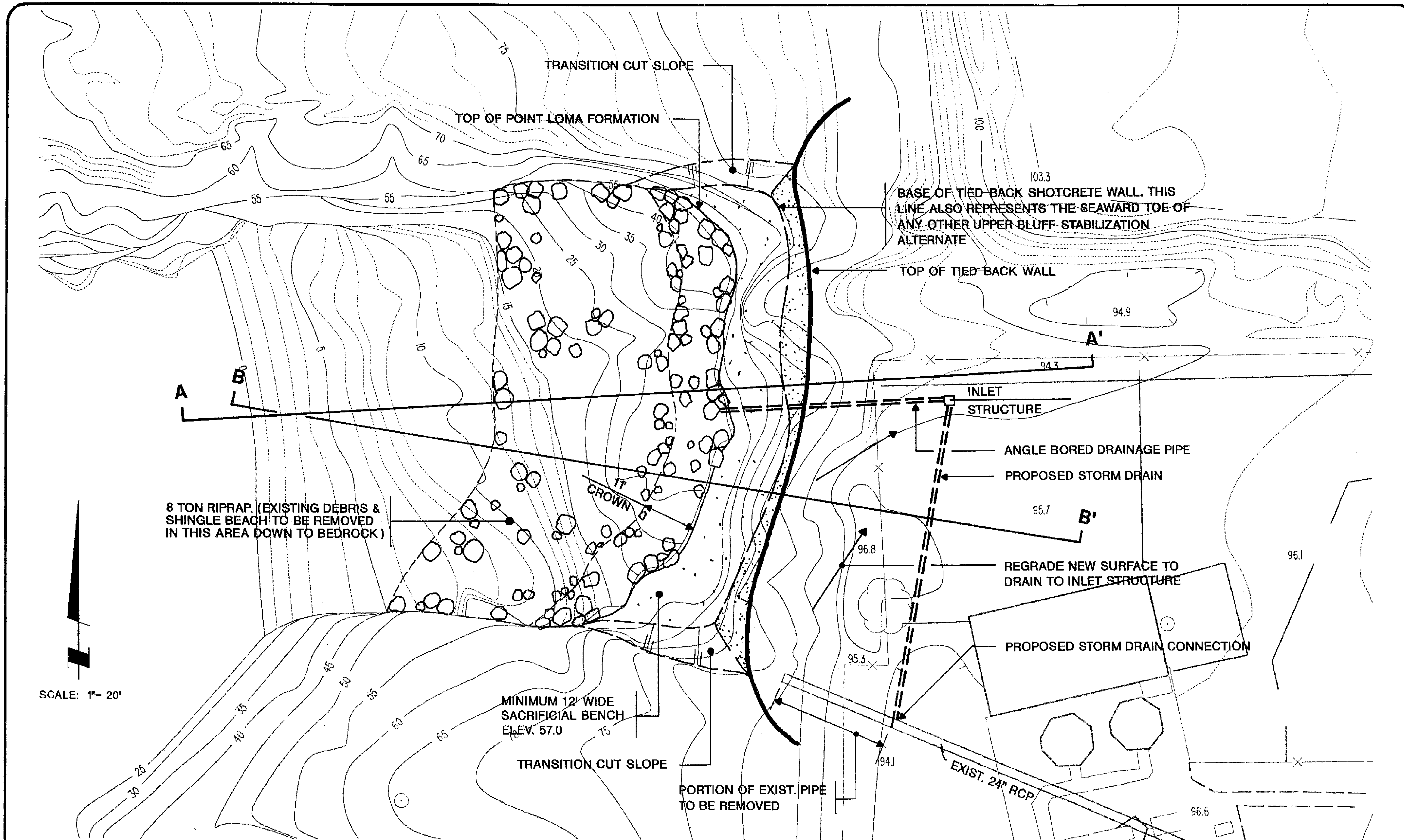
WAVE LENGTH: The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

WAVE RAY (ORTHOGONAL): On a wave-refraction diagram, a line drawn perpendicularly to the wave crests.

WAVE SETUP: Superelevation of the still water surface over normal surge elevation due to onshore mass transport of water by wave action alone.

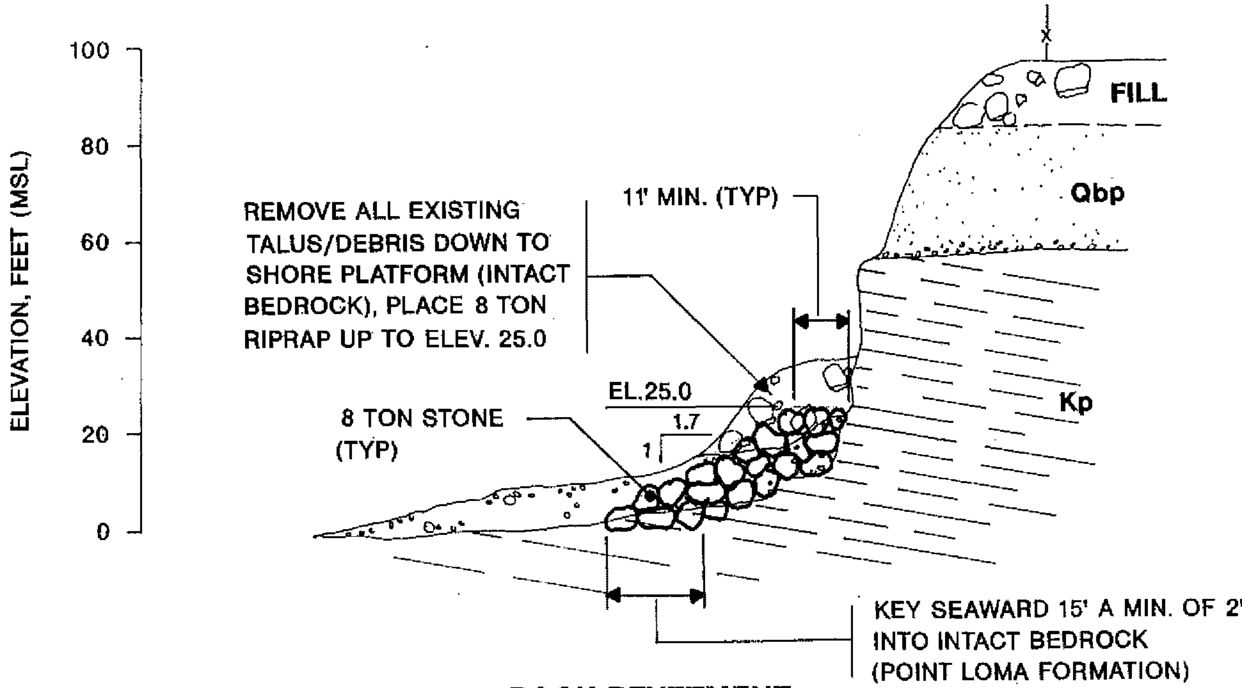
WEARING: The gradual destruction of a landform surface by movement of loose rock fragments or particles driven by wind, waves, running water or ice that causes rubbing, grinding, knocking, scraping, and bumping against the landform surface.

WEATHERING: The physical disintegration and chemical decomposition of rock that produces an in-situ mantle of softer material that is more easily eroded.

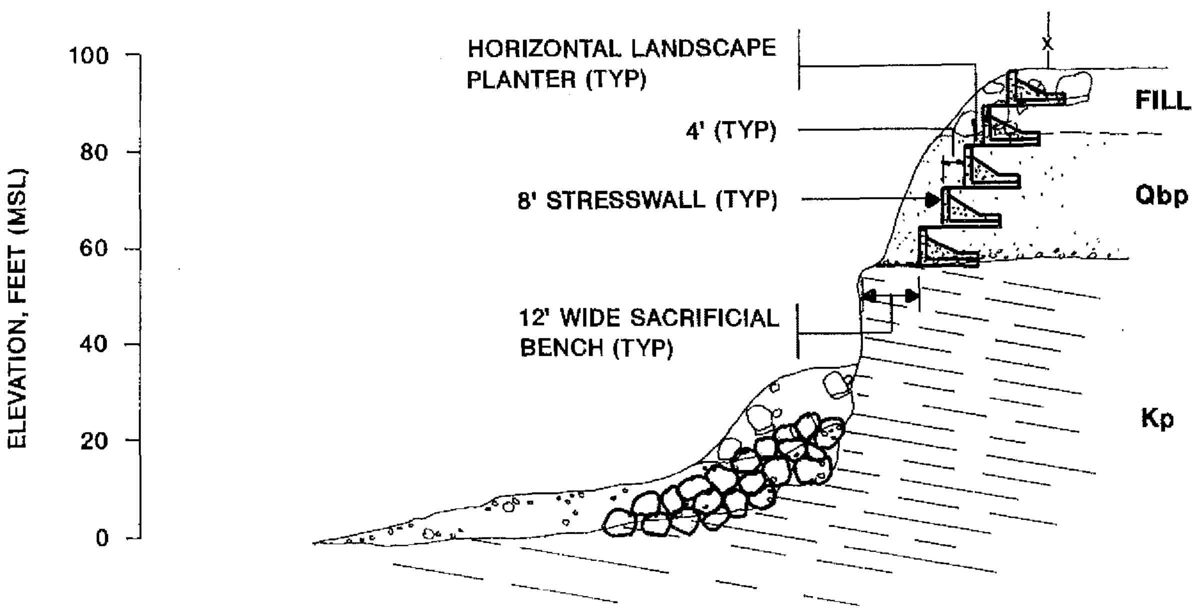


Project		Project No.	Figure
NORTH SHORELINE PROTECTION IMPROVEMENTS	SITE PLAN	1601BG	1

BAJADA/GDC ASSOCIATES



ROCK REVETMENT
LOWER SHORELINE PROTECTION



STRESSWALL

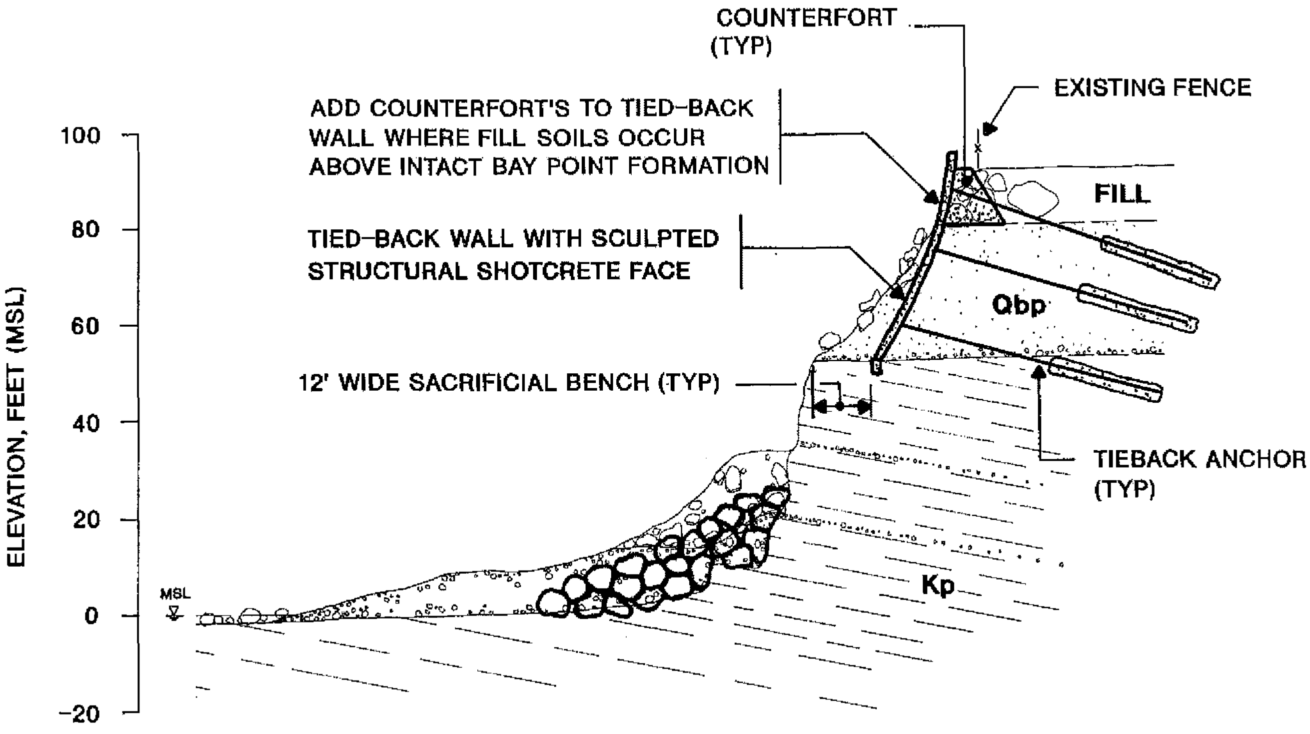
SECTION B–B'

SCALE: 1"= 40'

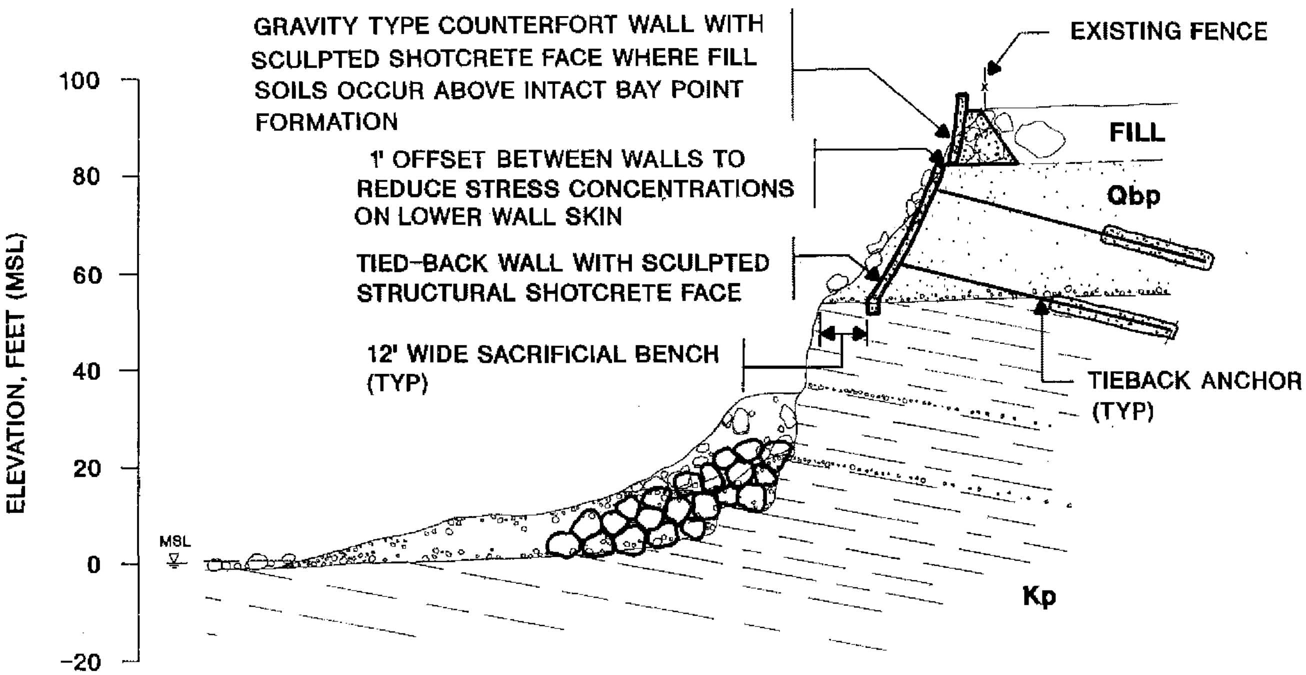
ROCK REVETMENT AND STRESSWALL ALTERNATES

Project No. 1601BG	NORTH SHORELINE PROTECTION IMPROVEMENTS	Figure 2

BAJADA/GDC ASSOCIATES



TIED-BACK WALL



TIED-BACK WALL AND COUNTERFORT GRAVITY WALL

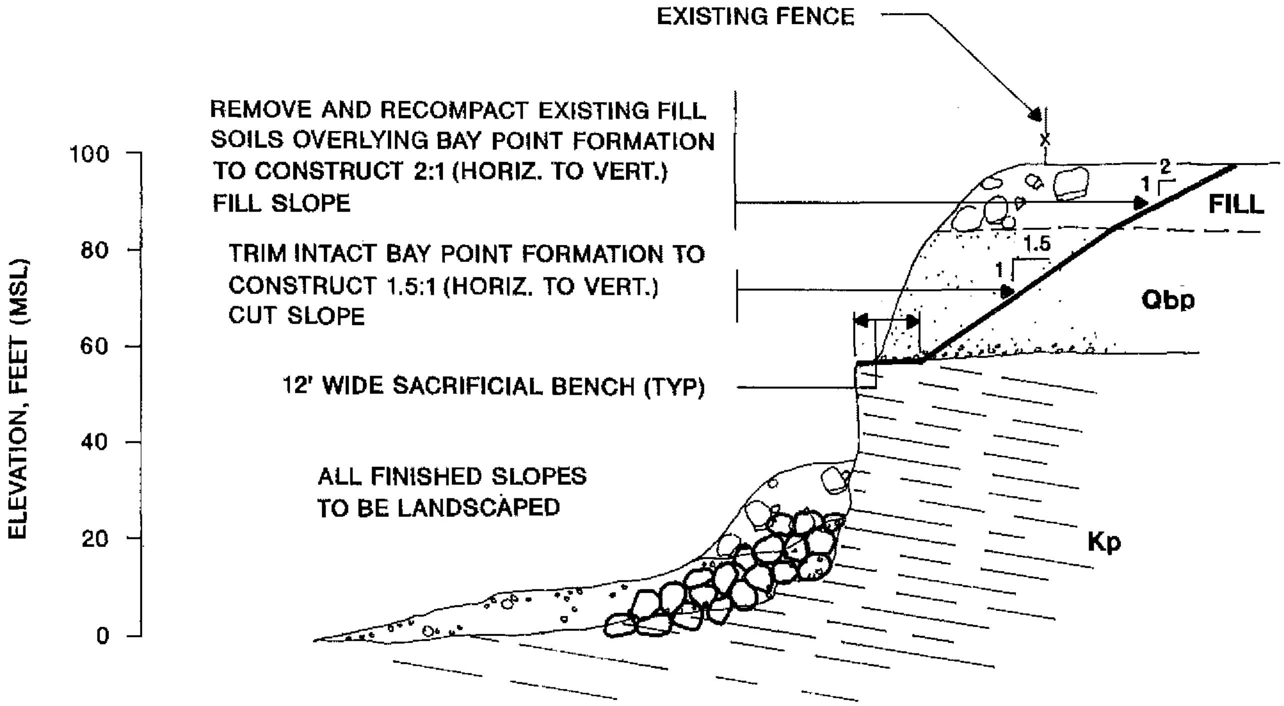
SECTION A-A'

SCALE: 1"= 40'

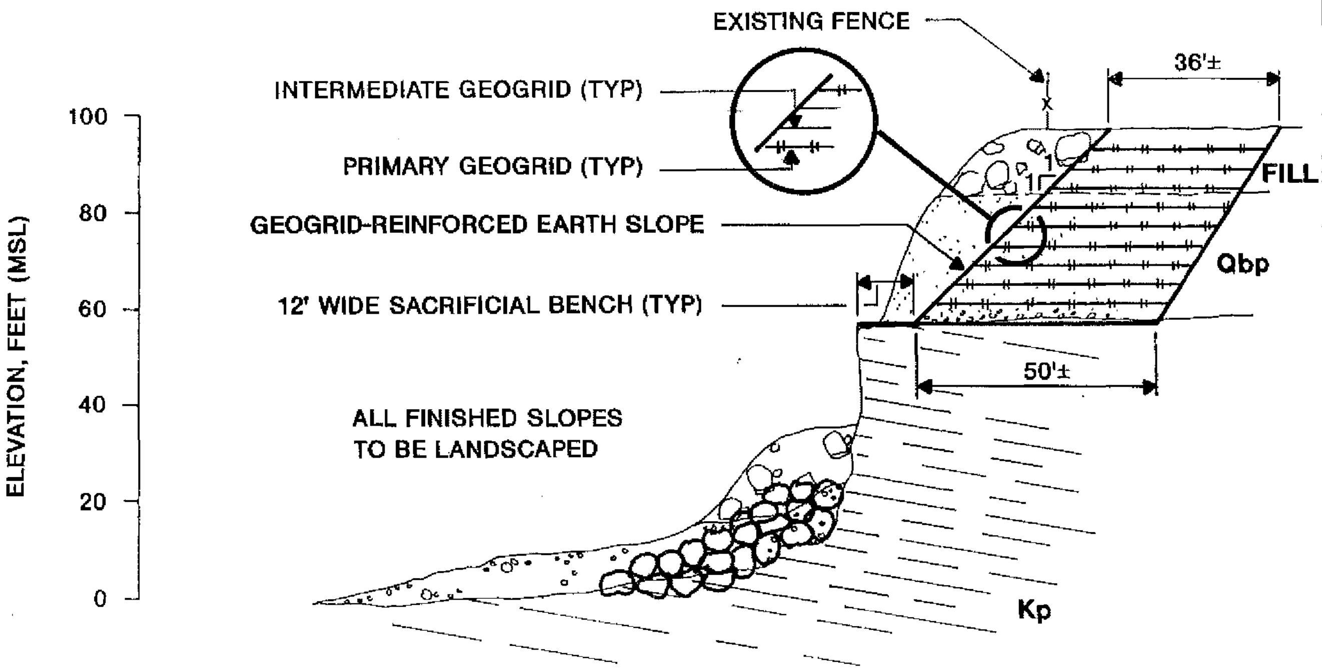
TIED-BACK WALL AND COUNTERFORT GRAVITY WALL ALTERNATES

Project No. 1601BG	NORTH SHORELINE PROTECTION IMPROVEMENTS	Figure 3

BAJADA/GDC ASSOCIATES



REGRADED UPPER SLOPE



GEOGRID-REINFORCED EARTH SLOPE

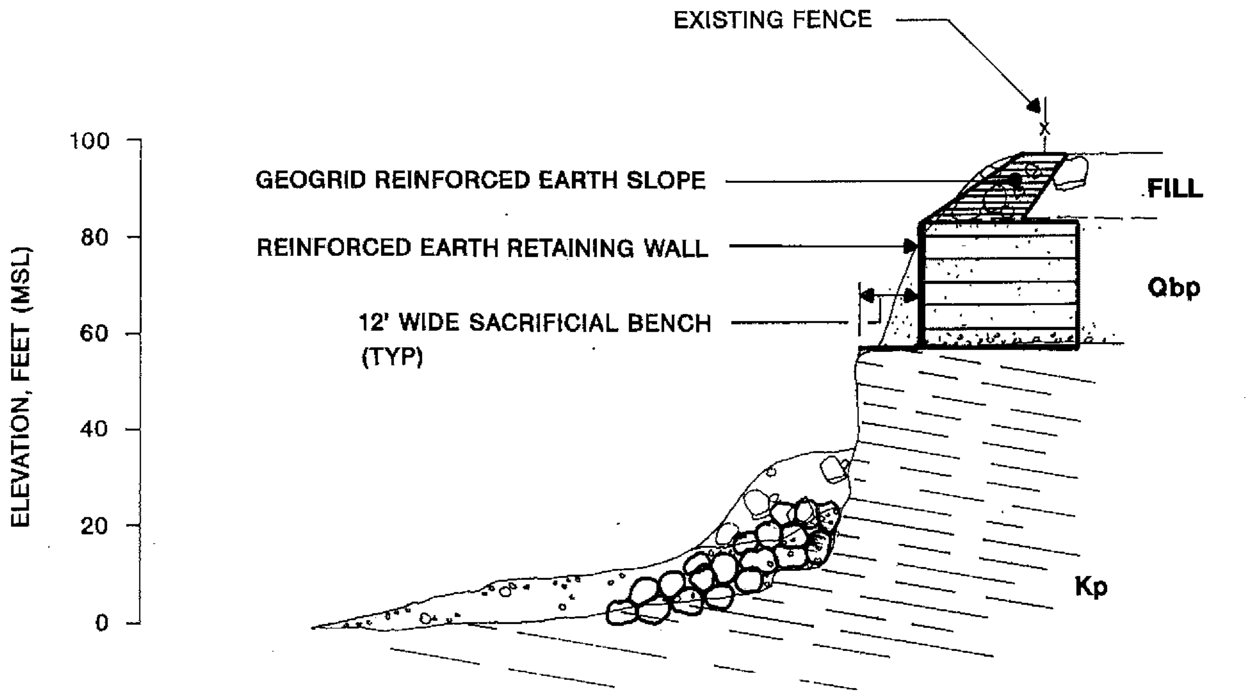
SECTION B–B'

SCALE: 1"= 40'

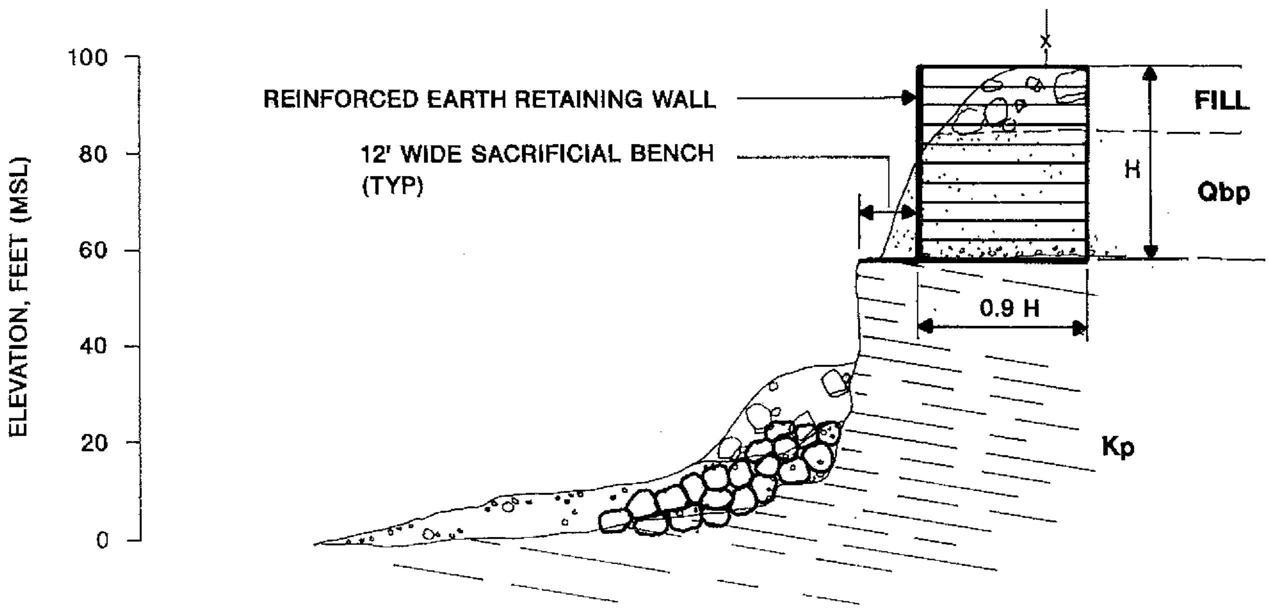
REGRADED UPPER SLOPE AND GEOGRID–REINFORCED EARTH SLOPE

Project No. 1601BG	NORTH SHORELINE PROTECTION IMPROVEMENTS	Figure 4

BAJADA/GDC ASSOCIATES



REINFORCED EARTH RETAINING WALL WITH GEOGRID-REINFORCED EARTH SLOPE



REINFORCED EARTH RETAINING WALL

SECTION B–B'

SCALE: 1"= 40'

REINFORCED EARTH AND GEOGRID–REINFORCED EARTH SLOPE ALTENATES

Project No. 1601BG	NORTH SHORELINE PROTECTION IMPROVEMENTS	Figure 5

BAJADA/GDC ASSOCIATES

APPENDIX A

GEOTECHNICAL AND COASTAL STUDIES
NORTH SHORELINE PROTECTION IMPROVEMENTS
POINT LOMA METROPOLITAN WASTEWATER TREATMENT PLANT
SAN DIEGO, CALIFORNIA

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APPENDIX A

GEOTECHNICAL AND COASTAL STUDIES
NORTH SHORELINE PROTECTION IMPROVEMENTS
POINT LOMA METROPOLITAN WASTEWATER TREATMENT PLANT
SAN DIEGO, CALIFORNIA

1 PROJECT DESCRIPTION

Coastal erosion and bluff retreat have characterized coastal geomorphic processes in the San Diego area for an estimated 18,000 years and, consequently, the construction and maintenance of shoreline protection has been necessary since the opening of the Treatment Plant in 1963. The most recent shoreline and upper-bluff stabilization project, completed in 1992, required extensive construction at five separate sites within the plant boundaries. The design of that project by Group Delta Consultants, Inc. (GDC) included the construction of stone revetments, tied-back walls, gravity walls, and the resurfacing and structural stabilization of a deteriorated steel binwall. We understand that the City currently intends to construct the North Shoreline Protection Improvements (NSPI) at the landward end of an approximately 100-foot-wide, 200+-foot-long sea cove (collapsed sea cave) at the northern boundary of the plant site, an area of accelerated erosion not addressed or mitigated by the 1992 construction. Figure A-1, the Site Plan and Geologic Map, shows the current topography, geology, and existing improvements in the general site area.

2 SCOPE OF WORK

Our geotechnical and coastal studies were performed to evaluate subsurface soil and geologic conditions, and to provide geotechnical and coastal engineering input to the design of shoreline and upper-bluff stabilization structures.

The field studies, performed during the period between January and March, 1995, were conducted to provide both geotechnical and coastal engineering data for design, and for assessment of potential site constraints that could impact the proposed improvements.

Survey work was also conducted to field-edit the topographic base map provided by the City of San Diego, to survey-in geologic contacts, and to develop offshore bathymetry.

Nearshore sea-floor bathymetry was surveyed, extending 300± yards offshore from the sea cove. We have digitized our recent bathymetric survey, along with the original offshore bathymetric surveys conducted as part of the 1988 GDC studies, onto the City's topographic base, and have provided the offshore bathymetry in digital format.

2.1 Geotechnical Investigation

Field studies for this project, as agreed with the client, were designed to supplement and complement the geotechnical studies for the HOG project, and the very detailed field and laboratory investigations performed over a three-year period during both the design and construction phases of the 1989-92 shore and upper-bluff stabilization project at the Plant site.

Geologic data points were surveyed in with the use of total station instrument. A Gradall Model G-880 excavator with a 30-foot boom was employed to expose geologic contacts on the steep slope at the head of the cove, and to assist the field geologist in reaching difficult access survey points. Additionally, the Gradall was used to excavate two test trenches through the mantle of natural overburden soils and fill at the top of the bluff. Figure A-1, the Site Plan and Geologic Map, presents the alignment of mapped geologic contacts, bedding attitude measurement points, and the locations of Test Trenches T-1 and T-2.

2.2 Coastal Studies

Our coastal engineering work on the project was performed to address three interdependent areas of investigation:

1) Bathymetry of the shore platform;
2) Selection of design waves; and
3) Development of a design rate of bluff retreat.

2.2.1 *Bathymetry*

Sea-floor bathymetry was determined immediately offshore of the study area, since, to a certain extent, sea-floor bathymetry controls the magnitude of design waves approaching and impacting upon the proposed coastal structures. We have made use of the offshore bathymetric surveys, conducted by GDC in 1988, just southerly of the study area as part of the recent shoreline stabilization work, and have augmented this baseline information with survey points extending several hundred feet offshore, and with a sufficient number of point elevations at the base of the bluffs to define local anomalies that may cause wave focusing. We have digitized our recent bathymetric survey, along with the original offshore bathymetric surveys conducted as part of the 1988 GDC studies, onto the City's topographic base, and have provided the offshore bathymetry in digital format under separate cover.

2.2.2 *Design Waves*

We have developed project design waves based on criteria set forth in the U.S. Army Corps of Engineers Shore Protection Manual (1984 edition). Foreshore slopes were determined from the results of the bathymetric surveys described above. Design still-water levels were obtained from NOAA reports with storm surge, wave setup, and sea level rise included.

2.2.3 *Rate of Bluff Retreat*

A geologic site reconnaissance was made to map the sediments exposed in the bluffs. The project site bluff geometry, stratigraphy, structure, and stage of development were compared with those of adjacent and nearby bluffs. Present day drainage patterns on the relatively gentle slopes above the bluffs were compared to those in historical photos. Using available data, soil strengths were assigned to the mapped soil units, and erosion susceptibility analyzed. Finally, utilizing all of the significant data obtained, a design rate of bluff retreat was developed.

3 SITE, SOIL, AND GEOLOGIC CONDITIONS

3.1 Site History and Surface Conditions

Virtually the entire area easterly of the project site has been graded since construction of the plant facility in 1963, resulting in essentially no natural open space areas. The 17-sheet set of plans for the construction of Cabrillo Road and Site Grading for Sewage Treatment Plant - City of San Diego Drawing Nos. 9074-1D to 9074-17D (plans approved November 1, 1960, as-built plans dated November 3, 1963), indicates that the site vicinity originally consisted of a gently westerly-sloping coastal terrace surface ranging in elevation from a low of 80 feet near the edge of the coastal bluff, to a high of approximately 106 feet in the vicinity of First Street. Original site grading completed in 1962 developed a variable-thickness fill pad in this area, with a finished surface along its seaward margin around elevation 95 feet, where it remains today.

As indicated in the original construction drawings, up to approximately 20 feet of fill soils were placed in this area in 1962. Ongoing erosion has removed portions of this material, undermining adjacent bluff-top improvements. In addition to the engineered fill shown on the above-referenced grading plans, relatively extensive debris fills also exist to the north, including a considerable volume of this material dumped into the head of the cove area, presumably to retard past coastal erosion encroaching upon bluff-top improvements.

The area easterly of the project site currently houses several existing structures, and is also used as a staging and work area for miscellaneous plant activities. The entire area is currently covered with asphalt concrete paving. Chain-link fencing and "K"-rail barriers extend along the northerly and westerly limits of the site.

3.2 Setting

Point Loma is a 6-mile-long promontory, extending southward from the lowland adjacent to the mouth of the San Diego River. The Point Loma coastal bluffs are bordered by a narrow wave-cut Quaternary-age terrace or bench, with elevations ranging up to approximately 95 feet above sea level. Wave-impact erosion has etched out less resistant faulted and jointed rock in the lower, cliffed portion of the bluffs, resulting in numerous sea caves and coves.

3.3 Soil and Geologic Units

The areal distribution and stratigraphic sequence of soil and geologic units are presented on Figure A-1, Site Plan and Geologic Map. Figure A-2, Generalized Geologic Cross Section A-A', indicates the general structural and stratigraphic relationships between the various soil and geologic units at the site. Figure A-3, Sea Cave Features and Structural Discontinuities, presents an aerial view of the cause-and-effect relationship between fault and joint trends, and the accelerated marine erosion (exemplified by sea caves and overhang features), that result from these structural discontinuities. This figure also shows one very large sea cave (approximately 160-feet long, 15- to 20-feet wide, and 35±-feet maximum height). Figure A-4, Generalized Geologic Cross Section B-B' is drawn through this very large cave, filled in 1963, as described in notes on the cross section.

Two geologic formations, two natural surficial soil deposits, and two types of man-placed (and dumped) earth materials are present in the general site area. The following paragraphs describe these units, in order from oldest to youngest.

Point Loma Formation (Kp): The Point Loma Formation is an approximately 900-foot-thick (Kennedy, 1975) sedimentary layer that discontinuously crops out in coastal areas of northern Baja California and as far north as Carlsbad. At the site, the Point Loma Formation forms the lower, more resistant, cliffed portion of the coastal bluff, up to approximately 50 to 60 feet in elevation, and dips landward (into the bluff) at about 10 to 15 degrees. This 70- to 90-million-year old formational unit extends seaward, comprising the shore platform, and extends landward beneath the coastal terrace and the entire Point Loma peninsula.

The Point Loma Formation consists of well-indurated marine sediments deposited by an offshore and deep-water submarine fan. Offshore deposits are represented by the thin-bedded siltstones and fine sandstones exposed in the upper part of the bluffs. Deep-water deposits are represented by the erosion-resistant thick-bedded mudstone and sandstone exposed in the lower-cliffed part of the bluff.

Bay Point Formation (Qbp): The Bay Point Formation, deposited on an elevated abrasion terrace, ranges up to approximately 35 feet in thickness at the site, and forms the upper part of the coastal bluff above approximate elevation 57 feet. The lower, relatively steeply-inclined 20±-foot section of this unit is comprised predominantly of locally-derived, relatively clean marine sands. The upper 15±-foot section is comprised predominantly of dense, non-marine, clayey to silty, medium to fine sands with sparse gravels, covered by 2 to 3 feet of silty sand topsoil and slopewash. The log of Test Trench T-1, presented on Figure A-5, shows the Bay Point Formation soils encountered near the surface of the coastal terrace.

The Bay Point Formation, deposited approximately 120,000 years ago on an approximately 125,000-year-old wave-cut platform, abraded during the last interglacial period when worldwide sea level was approximately 20 feet higher than today. Geologic evidence indicates that, since deposition of the Bay Point Formation, Point Loma has been uplifted by tectonic forces, approximately 40 feet at a rate of about 0.4-inches per 100 years.

Shingle Beach Deposits (Qsb): The cove contains a pocket beach consisting of gravel and cobbles. This coarse-grained "shingle" beach rests on the shore platform and is estimated to be up to approximately 10 feet in thickness.

Slump Deposits (Qsd): Slump-fall materials, which exist at the head (easterly end) of the cove are partly the result of oversteepening of Bay Point formational soils in the upper bluff, and partly of loading of the upper bluff soils with dumped fill and construction debris. In general, these loose deposits consist of light brown to tan, silty to clayey sands, and are mixed with construction debris (concrete, steel, etc.).

Artificial Fill (Qaf): Artificial fill, placed by man (as opposed to filling by sedimentary deposition), is exposed continuously along the top of the coastal bluff. This fill, placed to expand the useable flat area of the coastal terrace, can be expected to range in thickness up to approximately 20 feet in filled natural erosion gullies incised into the Bay Point Formation along the top of the bluff. Test Trench T-2, presented on Figure A-6 encountered an abundance of construction debris (large pieces of concrete and reinforcing steel) encountered in one of these filled drainages.

Rock Revetments: Figure A-1, the Site Plan and Geologic Map, shows the northerly edge of the existing rock revetment, previously placed to protect the lower hydro access road and outfall structure. These materials consist predominantly of angular, metamorphic quarry rock, which measures up to approximately 5½ feet in maximum dimension.

3.4 Groundwater

No significant free groundwater seepages were observed during our recent site reconnaissance, logging of test pits, and mapping of the coastal bluff slopes; however, we did observe several areas of locally damp to moist soil, indicating minor, evaporating perched seepages within the Bay Point Formation in the upper bluff, and a few such locally damp areas on the exposed Point Loma Formation strata in the lower part of the bluff. A broken 24-inch RCP storm drain, located in the southeasterly corner of the sea cove (see Figure A-1) discharges storm flow onto the coastal sediments below approximate elevation 81 feet. In addition to the damaging accelerated subaerial erosion caused by surface flow from this discharge, it is apparent that stress-relief jointing in the outer 2 to 4 feet of the bluff face is constantly saturated in the general area of the storm drain.

With the exception of the above-described effects from the broken storm drain pipe, only a few locally damp soil areas were observed, representing minor evaporating seepages in the coastal bluff soils. This condition is consistent with our review of the logs of geotechnical test borings drilled into the coastal terrace, easterly of the project site area. It should be expected that an extensive, regional groundwater table exists at, and slightly above, sea level.

3.5 Geologic Hazards

3.5.1 *Sea Caves*

The Point Loma Formation, which forms the lower cliffed portion of the bluff, has generally receded at a very uniform rate, forming long stretches of straight headlands, except where structural discontinuities (faults and related rock joints) have provided conduits for accelerated marine erosion. The subject sea cove has advanced landward some 200 feet, at the cliff/platform junction, in advance of the overall bluff headland alignment. Figure No. A-3, Sea Cave Features and Structural Discontinuities, and Figure No. A-4, Generalized Geologic Cross Section B-B', show the general cause-and-effect relationship between structural discontinuities and accelerated marine erosion.

In general, a sea cave begins to advance as wave energy propagates along weakened rock joints or faults. The cave begins to widen by water erosion, sand abrasion, and by slight differences in hydraulic pressure between neighboring joint planes. Eventually, sand particles suspended in larger and larger volumes of backwash from the cave abrade a surge channel seaward, which, in turn, creates a path for increased volumes of wave attack. The maximum measured extent of indentation for any cove along the Point Loma coast is approximately 300 feet, and the average for large coves is approximately 200 feet. Ultimately, sea caves widen to the point where the cave roof is unable to support its own weight, and a collapse occurs. Figure A-9 shows sea cave development resulting from wave energy focused along the alignment of a joint trend (N52°W). Figure Nos. A-3 and A-4, respectively, show areal and cross-sectional views of a large cave (approximately 160-feet long, 15- to 20-feet wide, and 35±-feet maximum height) that was infilled as part of the original Plant construction in 1963.

Of importance in the growth of sea caves is an understanding of the mechanics of its growth and its stability after cessation of erosive energy. Sea caves form along structural weaknesses in the geologic formation, with the base [floor] of the sea cave developing along the shore platform surface. Although locally deeper surge channels may exist along a fault lineament, the base of the sea cave will not extend below the offshore projection of the shore platform. Localized surge channels within the floor of the sea cave (on the fault lineament) seldom exceed 2 to 4 feet in width, with maximum depths limited to 1 to 3 feet. Subsidiary sea caves only form along subsidiary joint sets and, depending upon lithology and availability of erosive wave forces, can also grow to be quite large in dimension.

When the erosive wave energy is eliminated, further growth of the cave ceases, unless the cave geometry was only marginally stable at cessation of wave energy. Infilling sea caves is highly effective in mitigating their presence, as was done for the sea cave depicted on Figure A-4. Most importantly, based on our understanding of the mechanics of sea cave development, it is our opinion that the existing sea caves in the site vicinity pose no direct, or indirect, impact to the long-term (75 years±) stability or structural integrity of the proposed North Shoreline Protection Improvements.

3.5.2 *Landsliding and Soil Slumping*

Our field investigation did not reveal the presence of any existing ancient natural landslides in the project site area. No landslides have been mapped as being present on, or immediately adjacent to, the subject north boundary sea cove. A relatively large soil slump, apparently the result of oversteepened upper bluff geometry, is shown on Figures A-1 and A-2. This soil mass, along with construction debris carried down with it, and apparently dumped over the top of the bluff, is currently serving to protect the cove from erosion, and should continue to do so until it is removed by the next major storm event, or as part of the construction activities associated with the proposed improvements. The potential for landsliding in the coastal bluffs at the project site is discussed in greater detail in this report under Section 6.3, Bluff-Top Retreat.

3.5.3 *Regional Faulting and Seismicity*

The site is located in a moderately-active seismic region of Southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking from ten major active and potentially-active fault zones could affect the site in the event of an earthquake. Those within 60 miles of the site are the Rose Canyon, Coronado Banks, La Nacion, San Diego Trough, San Clemente, and Elsinore fault zones. The location of the site and the proximity to these faults is illustrated on Figure A-7 , the Regional Fault Map.

To assess the level of seismic exposure at the site, we estimated the maximum credible and maximum probable earthquake magnitudes for these fault zones. The maximum credible earthquake is the maximum earthquake for a given fault, based upon the geologic characteristics and constraints of the fault. The maximum probable earthquake defined by the CWP Guidelines is the earthquake that is likely to occur for any given 100-year period with a 10 percent probability of being exceeded. The resulting maximum probable earthquakes in Table A-1 are slightly more conservative than used for hospital design, which allows a 20 percent probability of being exceeded.

Based on our analysis the mean site acceleration for the maximum probable earthquake is approximately 0.25g. The Rose Canyon fault zone and the Coronado Banks fault zone produce a similar level of seismic exposure. Therefore, we recommend a design site acceleration of 0.25g. This acceleration is based on attenuation relationships by Campbell 1987, which emphasize near-field strong ground motion in the distance range of active faults near the site.

Acceleration of 0.25g is the mean acceleration. The mean-plus-one standard deviation acceleration, having a 16 percent probability of being exceeded during the maximum probable earthquake, is also shown for comparison but typically would not be used for design of a sewage facility. The additional conservatism resulting from use of the mean-plus-one standard deviation acceleration is usually applied only to nuclear power plants and similar potentially hazardous facilities.

3.5.4 *Local Faulting*

The Point Loma Peninsula is a tectonically upthrown-block or "horst" on the westerly side of the downthrown, San Diego Embayment Graben, a major regional north-south trending structural feature. The Point Loma Fault, which generally trends north-south along the easterly shoreline of Point Loma, separates the horst and graben fault blocks. The project site is situated in an area of the peninsula traversed by numerous faults antithetic to the regional north-south trending structure. All, or nearly all, of the sea caves and coves in the plant site area appear to have been formed by accelerated erosion along these secondary fault zones and/or the abundant jointing associated with the faulting. Figure A-3, Sea Cave Features and Structural Discontinuities, indicates the approximate alignment of a northeasterly-trending fault zone and a northwesterly-trending joint system, two structural features that have played a large role in the erosion of the north boundary sea cove. Like all of the structural discontinuities mapped in the Point Loma Formation at the plant site, these two features do not offset (nor are they reflected in) the overlying Quaternary-age soils, and thus can be classified as "not active." Section 6 of this report contains additional discussion on the relationship between structural discontinuities and the propagation of sea caves and coves.

4 DESIGN CONSIDERATIONS

4.1 Design Parameters

A critical factor in any geotechnical evaluation is the selection of design parameters. Our selection of design parameters was based upon our site reconnaissance, field investigation, correlations in published reports and literature, and parametric evaluations of existing conditions. Design parameters were selected for the unclassified fills and the formational materials. Parameters utilized in our evaluation are summarized in Table A-2. Information pertaining to the existing fills was unavailable at the time of our investigation. As such, parameters for the fill soils were based on typical conservative values for compacted soils obtained from the literature. The parameters chosen for the formational materials represent a lower estimate of anticipated strengths.

4.2 Tied-Back Walls

The geotechnical factors influencing the design of a tied-back wall system includes the stability of the slope, the strength of the anchor zone, the type of anchor system, and the forces acting on the tied-back wall system. The results of our analyses indicate that the sea cliffs are generally stable with respect to deep-seated, rotational-type failures. However, the bluffs are susceptible to local instabilities associated with undermining and erosion due to wave action. The undermining appears to produce a series of block-type failures (blockfall) resulting in bluff retreat. Results of our investigation also indicate that the formational materials are highly jointed in localized areas. The joint pattern is both perpendicular and parallel to the cliff face. Formational materials encountered in our investigative work, in general, are suitable for supporting the proposed anchor systems.

4.3 Gravity Structures

The gravity structures under consideration include conventional reinforced concrete walls, reinforced concrete counterfort walls, Stresswalls, and mechanically-stabilized walls. Geotechnical considerations associated with the design of gravity structures include bearing capacity, settlement, lateral earth pressures acting on the gravity structure, and other external forces such as those due to waves. Gravity-type structures may be founded on either the Bay Point Formation or the underlying Point Loma Formation. Recommendations for gravity walls, excluding mechanically-stabilized earth walls, are provided in Section 5.3. Recommendations for mechanically-stabilized earth walls are presented in Section 5.4.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on geotechnical considerations, the structures currently proposed for shoreline protection at the PLWTP may be constructed utilizing the materials encountered at the site, provided the recommendations in this report are incorporated into the design of the structures.

5.2 Tied-Back Walls

5.2.1 *General Anchor Design*

We anticipate that anchor design will consist of multi-strand prestressing tendons grouted into an inclined anchor hole and post-tensioned against the face of the wall. Either friction anchors or belled anchors could be used for the tiebacks. However, for the conditions at this site, it has been our experience that friction anchors involve fewer installation problems and provide more uniform support than belled anchors. Therefore, we recommend that anchors utilize friction from straight shaft cylindrical-bored holes, with the design capacity based upon the surface area of the bonded zone. Allowable design anchor loads can be calculated from the following equation:

$$T_{all} = 1.65\ L\ \left(\frac{d}{12}\right)\ (0.032z + 0.7)$$

Where: L = effective length (bonded length) of anchor measured in feet.
d = anchor diameter measured in inches.
z = depth of anchor (below the ground surface), measured in feet.
T_{all} = allowable anchor capacity in kips.

For example, the computed allowable anchor capacity for an anchor with 50 feet of bonded length, a diameter of 12 inches, and approximately 10 feet below the ground surface is 84.15 kips. This equation utilizes a factor of safety of 2.0 against ultimate pullout resistance.

All tiebacks should be proof tested in the field in general accordance with the City of Los Angeles Code Requirements for Anchor Testing. The City of San Diego Building Inspection Department is currently in the process of updating their requirements for anchor testing, and these requirements, although not currently in effect, are also considered acceptable for anchor testing.

5.2.2 *Tied-Back Walls Supporting Upper Fill Materials*

Tied-back anchors used to restrain a concrete structure supporting the upper fill soils should be designed so that the bonded zone for the anchors is calculated from only that portion of the anchor embedded into either Bay Point or Point Loma formational materials. In all likelihood, the tied-back anchors will be incorporated within a reinforced concrete counterfort wall, which bears on the Bay Point Formation. Tied-back anchors should be inclined on a minimum 9½ degree downward slope measured from the horizontal. Larger inclinations approaching 45 degrees are acceptable. We recommend that a minimum unbonded anchor length of 15 feet be used for all tiebacks.

The tied-back and counterfort walls should be designed to accommodate earth pressures and vehicle loads. Recommended design earth pressures are presented in Table A-3. Due to the limited amount of anticipated wall movements, the design earth pressures assume "at-rest" conditions and include an increase to account for compaction-induced pressures (Fang, 1991). In addition, the soil behind the wall is assumed to consist of granular soils. Recommended earth pressures do not include hydrostatic loading. If drainage is not provided or maintained immediately behind the wall, hydrostatic forces need to be included in the design.

To simulate the loading due to the periodic movement of heavy truck traffic, an additional uniform pressure of 60 psf should be assumed to act on the entire wall. This value assumes that the vehicle will be a minimum of 10 feet from the tied-back wall and will be limited to a tandem axle truck with 18,000 pound maximum axle load weight.

Walls subjected to other surcharge loads, such as fills or footings, applied within a distance behind the wall equal to or less than the wall height, should be designed to accommodate the surcharge load. For uniform pressure surcharge loads, the walls should be designed for an additional uniform pressure equal to 0.5 times the uniform surcharge pressure.

5.2.3 *Tiebacks for Bay Point Formation Stabilization*

Tiebacks used to restrain walls fronting the Bay Point formational soils (the lower 20± feet of the tied-back wall) should be designed to resist an equivalent fluid pressure of 60 pcf with surcharge loading due to the upper fill and gravity structure footing modeled as a uniform pressure equal to the maximum vertical pressure times a lateral earth pressure coefficient of 0.5. Tiebacks should be constructed at a 10± degree downward angle measured from horizontal at the face of the bluff.

5.3 Gravity Retaining Walls

Allowable Bearing Pressures - All gravity retaining walls should be founded in intact formational Bay Point or Point Loma soils. We recommend that gravity-retaining wall structures be designed for an allowable bearing pressure of 5,000 pounds per square foot. We estimate that total settlements for the gravity structures, including mechanically-stabilized walls, will be less than 1 inch.

Active Earth Pressures - We recommend that these structures be designed to resist the load imposed by an active earth pressure equivalent to a fluid weighing 35 pounds per cubic foot. This assumes that soil behind the retaining wall will consist of compacted granular soils. It also assumes level backfill conditions and that no surcharge loads exist. For walls where backfill slopes are anticipated to be constructed at inclinations as steep as 2 to 1, the walls should be designed for an active equivalent fluid pressure of 40 pounds per cubic foot. Recommended earth pressures do not include hydrostatic loading. If drainage is not provided or maintained immediately behind the wall, hydrostatic forces need to be included in the design. Walls subject to surcharge loads, applied at a distance behind the wall equal to, or less than, the wall height, should be designed for an additional uniform pressure equal to 0.3 times the surcharge load. Surcharge loading from vehicular traffic, as described in Section 5.2.2, should be modeled as an additional uniform load of 60 psf acting along the entire wall.

Lateral Resistance - To provide resistance for design lateral loads, we recommend that passive pressure be assumed equivalent to a fluid pressure of 350 pcf for footings and shear keys poured neat against sides of excavations. This value assumes a horizontal surface for the soil mass extending at least 10 feet from the base of the wall or 3 times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of materials in areas susceptible to erosion should not be included in design for passive resistance to lateral loads. If friction is to be used to resist lateral loads, we recommend a coefficient of friction of 0.35 between the soil and the base of the footing. If it is desired to combine friction and passive resistance in design, we recommend using a reduced friction coefficient of 0.25.

5.4 Mechanically-Stabilized Embankments (MSE)

Overview

A mechanically-reinforced fill is an embankment in which planar reinforcing elements are incorporated horizontally into the fill volume during construction. Through the mobilization of tensile stresses in these reinforcing elements, an embankment can be constructed at a steeper inclination than if no horizontal reinforcement was present. Typical horizontal reinforcement can include geogrids, metal strips, or wire mesh.

For mechanically-reinforced fills constructed at inclinations of less than approximately 1:1, no facing element is generally required where the mechanical reinforcement terminates at the slope face. At fill slope inclinations greater than approximately 1:1, a facing element is typically incorporated where the mechanical reinforcement terminates at the slope face as the embankment is constructed. These facing elements mitigate surficial loss of soil at the slope face and provide a more aesthetically and psychologically acceptable appearance. Facing elements can typically be composed of concrete panels, welded-wire panels, geotextile wraps, or landscape blocks. In applications where usable space at the top of the fill is at a premium, mechanically reinforced embankments are often constructed at vertical inclinations with concrete panels as facing elements, giving the appearance of a conventional gravity retaining structure.

Several companies currently manufacture a variety of patented applications that are essentially mechanically-reinforced fills that utilize a specific type of reinforcing element and facing element. These companies include:

- Tensar Corporation (geogrid reinforcement and geotextile facing elements);
- Hilfiker (welded-wire reinforcement and welded-wire facing elements);
- Reinforced Earth (metal strip reinforcement and concrete panel facing elements); and
- Mirafi (geogrid reinforcement and landscape block facing elements).

Design Considerations

The geometry of a mechanically-reinforced fill is largely governed by space requirements at the top of the proposed fill. Steeper fill inclinations typically cost more to construct but provide more space at the top of the fill. For a selected inclination and height, a mechanically-reinforced fill can be designed after appropriate strength values of the proposed backfill material are determined. We recommend that the strength parameters for fill material as listed in Table A-2 be considered for design.

It is recommended that computer stability analyses be conducted on proposed mechanically reinforced fills so that a variety of postulated failure planes can be considered. In these analyses, it is important that both the internal stability of the mechanically-reinforced fill and the global stability of the proposed design be considered. A variety of computer programs such as SLOPE/W and STABGM are available which can analyze the stability of slopes that have been designed with horizontal reinforcement. Additionally, some manufacturers of patented mechanically-reinforced fill systems have design software that is product specific that can be utilized provided that the designer is comfortable with the program design methodology.

For slope stability analyses, we recommend that the proposed improvements be designed to have a static factor of safety of at least 1.5. We also recommend that the proposed improvements incorporate a minimum pseudo-static factor of safety of 1.2 when a horizontal site acceleration of .25g is considered.

5.5 Seismic Design Considerations

Dynamic lateral forces are imposed upon retaining structures during seismic shaking. These forces could result in net deformation in a retaining structure, development of residual forces in anchors, and increased load requirements of the various structural elements. Although it may not be mandatory to include seismic loading in the sizing of structures and/or anchor assemblies, consideration should be given to mitigating a potential failure from overstressing foundation components during a design earthquake such as the maximum probable earthquake, as well as assessing magnitudes of net displacement of retaining structures and induced residual forces.

Seismic loading on any earth retaining structures was based on a Mononobe-Okabe analysis (Seed and Whitman, 1978) method. For a design acceleration of 0.25g, and a total unit weight of 130 pcf, we recommend an equivalent seismic-induced earth pressure component of 26 pcf. This equivalent seismic-induced earth pressure should be applied as an additional inverted triangular loading acting at 2/3 of the wall height.

6 MARINE AND SUBAERIAL EROSION

6.1 Wave Climate

In evaluating the wave climate that controls coastal erosion, considerable hindcast data are available, which indicate likely future trends. Accordingly, it is feasible to establish geotechnical design criteria for coastal structures. Waves along the San Diego County shoreline generally range in height from 2 to 5 feet; however, large waves ranging from 6 to 10 feet in height are not uncommon. These large waves can arrive at almost any time during the year and may continue for 3 to 4 days. These high-wave episodes are frequently unaccompanied by strong winds. Breakers with estimated heights of 15 to 20 feet have been observed off the coastline within the study area (USCOE, 1960; National Marine Consultants, Inc., 1960).

The San Diego County coastline is exposed to wave action, undiminished by island interference, through only two relatively narrow corridors of wave approach (USCOE, 1960). Waves with periods longer than 10 seconds approach the shore from the northwest between Santa Rosa Island and San Nicolas Island, and from the southwest between Cortez Bank and Los Coronados Islands. The longer-period waves approaching from other directions are obstructed by the various channel islands, Tanner Bank and Cortez Bank, and the Los Coronados Islands.

Short-period waves, with periods of 8 seconds or shorter, generated from the nearshore waters within the various channel islands and offshore banks, have a fetch of 50 to 100 nautical miles, and approach the study area from the northwest through the southwest.

Ocean waves off the coast of southern California fall into three main categories:

1. "Northern hemisphere swell" consisting of waves generated in the North Pacific and Gulf of Alaska;
2. "Southern hemisphere swell" consisting of similar waves generated south of the equator; and

3. "Sea" consisting of waves generated within the local area (Munk and Traylor, 1947).

Northern Hemisphere Swell: Winds that produce northern hemisphere swell are usually associated with one of the following meteorological situations (Marine Advisors, 1961):

1. Japanese-Aleutian storms, which move from west to east in relatively high latitudes, often stagnating in the Gulf of Alaska. Occasionally, especially during winter and spring, this storm track shifts southward and the maximum wave heights occur at central or southern California latitudes. These extra-tropical cyclones are the most important source of severe waves reaching the California coast.

2. Hawaiian storms, which move from west to east in mid-latitudes.

3. Tropical hurricanes, which commonly develop off the west coast of Mexico. The resulting swell rarely exceeds 2 m (6.5 feet), but a strong tropical storm will occasionally move far enough north to cause destructive high waves. The storm of September 1939, which passed directly over southern California causing very high waves locally, is an example (Horrer, 1960).

Southern Hemisphere Swell: Munk, et al., (1963) point out three major source areas: The Ross Sea, the New Zealand-Australia-Antarctic sector, and the Indian Ocean. These southern ocean source areas are partially blocked by island chains in the South Pacific Ocean. The South Pacific is such a large area that waves from several southern storms commonly reach southern California simultaneously. Southern swell is most important during the southern winter from April through September.

Sea: Sea is the term applied to short, steep waves which are still in or near the area in which they are generated. Wind conditions which generate sea vary greatly as one moves offshore from the southern California coast, changing from relatively mild winds over the inner channels to strong, gusty winds outside the islands.

Summaries of Wave Data: Directional wave information is available from various sources. Among others, Marine Advisors (1961) has produced hindcast estimates for a station approximately 65 nautical miles southwest of San Clemente Island, thereby providing deep-water wave data unaffected by island sheltering. Their data are compactly expressed in wave roses, and reproduced herein as Figure A-8. The radiating bars represent direction classifications, and the concentric circles which intersect them form a frequency scale, expressed in percentage of the average total number of hours in a year (8766 hrs/yr). For example, the longest bar in the upper wave rose represents all northern hemisphere swell approaching from 300 to 310 degrees. The inner segment, out to the numeral "1", gives the frequency of waves from that direction in the 0.1 to 0.9 foot height group. It measures approximately 6.9 percent, which indicates that waves of this classification can be expected 0.069 x 8766 = 605 hours per year. Note that maximum south swell heights arriving from the southern hemisphere are only about 25 percent as large as north swell heights from the northern hemisphere.

Seymour, et. al. (1984) have produced storm wave hindcast estimates for the period 1900 - 1984 using a single methodology, which is spectral throughout. The hindcast location is near 35°N, north of Point Conception and the Channel Islands. Only waves with deep-water-approach directions between SW and WNW were considered because waves approaching more obliquely would be considerably diminished by refraction as they approached the shoreline. Further, the waves were ranked by their power (energy multiplied by period). This resulted in a list of 59 storms in which the resulting offshore significant-wave-height exceeded 3 m (10 feet), all having periods equal to or exceeding 12 seconds. The tropical cyclone of September 1939, a major wave event in southern California, was added for a total of 60 storms. These are listed in Table A-4.

It is of interest to note that extreme deep-water wave episodes exceeding 6 meters were only reported on eight occasions during the period 1900 -1979, while the period from February 1980 through February 1984 experienced a total of ten storm events with deep-water waves exceeding 6 meters. Further, the storm of January 17-18, 1988, produced the highest measured deep-water waves approaching the southern California coast. The significant wave height was 10.0 meters (Seymour, 1989), higher than any reported in the 1900 - 1984 database. This storm was likely on the order of a 200-year storm, and was reported by Seymour to be ". . . remarkably similar to Richard Henry Dana's observations in Two Years Before the Mast of the dangerous Southeasters [significant storm arriving from the south] off this same coast during the 1830's."

Continued coastal erosion, in part accelerated by more energetic wave activity during the last 10 years, has subjected the San Diego County coastline to a progressively more severe wave energy than that experienced during the preceding 40 to 50 years. This historical database is used by most consultants to estimate shoreline erosion rates, which are typically then used to forecast erosion during the useful life of a proposed structure.

6.2 Short-Term Sea Level Rise

The effect of waves on the coast is highly dependent on the sea level during the wave episode. Large waves at low sea level cause limited erosion since they break well offshore. When episodes of large waves combine with short-term high sea level from tides and other factors, rapid retreat may occur along vulnerable coastlines.

Tides: Tides are caused by the gravitational pull of astronomical bodies, primarily the moon, sun, and planets. Tides along the San Diego County coast have a semi-diurnal inequality. On an annual average basis, the lowest tide is about -1.7 feet (MLLW datum) and the highest tide is about 7.3 feet MLLW datum (+4.42 feet MSL datum).

Storm Surge: Storm surge results from strong storms pushing sea water against the coast. Extreme storm surges are presented as a function of return period at selected California tide stations (NOAA, 1980), with those for La Jolla shown below:

Return Period Years	Storm Surge Feet
5	2.0
10	2.1
25	2.2
50	2.3
100	2.4

When storm surge occurs at the same time as a tidal maximum, the combination results in a statistical extreme water elevation as follows (NOAA, 1980):

Return Period Years	Extreme Water Elevation Feet (MSL Datum)
5	4.4
10	4.5
25	4.6
50	4.7
100	4.8

Wave Setup: Wave setup results from superelevation of the water surface over the normal surge elevation due to onshore mass transport of the water by wave action alone. Wave setup is a function of both the still-water level, and the elevation and slope of the shore platform. For San Diego County, the typical maximum range in wave setup would likely vary from 1/2 to 1 foot, which would be added to the extreme water elevation resulting from storm surge and astronomical tide.

Design Still-Water Level: For design of coastal structures, a conservative high sea level is determined that accounts for all of the factors that may increase sea level during the design life of the structure. Tides, storm surge, and wave setup add up to a 100-year high-water elevation of 5.8 feet (MSL datum). To this, an additional 0.5 feet is added to account for long-term sea-level rise. Most designers use 0.5 foot, although best estimates of expected rise in the next 75 years are 4 inches (10 cm per 100 years). For the San Diego County coast, the design still-water level is 6.3 feet (MSL).

6.3 Bluff-Top Retreat

This section documents the technical approach used for estimation of the 75-year bluff-retreat line, as presented on Figure A-1, the Site Plan and Geologic Map. Our evaluation of bluff retreat is based on two principal factors, both to be considered in the absence of the proposed North Shoreline Protection Improvements:

1. Estimation of the amount of marine erosion expected at the cliff-platform junction and evaluation of the resulting effect upon erosion of the coastal bluff; and

2. Estimation of the amount of slope decline expected for the bluff above the elevation of principal influence of marine erosion.

6.3.1 *Bluff-Top Defined*

For the purpose of this report, and in general accordance with currently-accepted coastal agency standards, bluff top or top-of-bluff is defined as the boundary, or line of intersection, between the inclination of the upper natural bluff and the natural surface of the coastal terrace, excluding incised drainages.

6.3.2 *Bluff-Top Retreat and Engineering Design*

Placement of facilities on the coastal terrace above the bluff must account for changes in the bluff expected during the intended life of the structure. Historically, the typical approach has been to build as close to the bluff as desired, assuming that maintenance and repair would forestall loss of the structure. Another approach has been to estimate the amount of bluff-top retreat expected within the economic life of the structure, and to build behind the influence of retreat.

In coastal engineering, the concept of intended lifetime of a structure has been replaced by required design periods and bluff setbacks set by regulatory agencies. The California Coastal Commission requires a 75-year period to approximate the useful design life of most structures.

6.3.3 *Methodologies of Analyses*

The methodologies most useful in the assessment of relative rates of coastal erosion can be divided into five general categories:

- Historical analyses;
- Geomorphic analyses;
- Anthropic influences;
- Impact of long-term sea-level change; and
- Empirical and analytical techniques.

<u>Historical Analyses</u>

Historical data include maps, charts, photographs, survey notes, reports, newspaper clippings, and eye-witness accounts. The comparison of contemporary maps is subject to error, especially when the maps are produced only a few years apart. We have found that, in general, successive high-resolution photographs showing readily-identified coastal features provide the best record of progressive shoreline change. Analysis of stereographic aerial photographs dating from 1938 has proven an effective tool in evaluating bluff retreat at the north boundary sea cove.

Geomorphic Analyses

Geomorphic analyses include all of the factors that contribute to the shaping of coastal landforms. Coastal erosion and coastal-bluff retreat are caused by both marine and terrestrial processes. Surf action is usually the dominant marine agent producing both hydraulic (wave) impact and abrasion. The principal geomorphic factors that contribute to coastal erosion are climate, wave energy, lithology and the structure of coastal bluffs, groundwater, bluff geometry and the angle of upper bluff slope decline.

A basic understanding of the various geomorphic processes is requisite to the assessment of variations in shoreline erosion. Geomorphic analysis, including coastwide geologic inventory, measurements of offshore bathymetry, and research to determine historic climatic conditions permits the assessment of likely future coastal erosion.

Anthropic Influences

Human activity significantly influences shoreline changes. Grading operations at the Plant site, which began in 1963, have had a major impact upon upper-bluff erosion, focusing uncontrolled surface drainage over the bluff-top at the head of the cove.

Impact of Long-Term Sea-Level Change

An entirely independent method of assessing the rate of coastal erosion is to consider long-term (geologic) sea-level change, which is the major factor determining coastal evolution (Emery and Aubrey, 1991). Sea-level curves show a relatively rapid rise of about 1 meter per century from about 18,000 years before present to about 8,000 years ago, as indicated in Masters and Fleming (1983).

About 8,000 years ago, the rate of sea-level rise slowed, ultimately to a relatively constant rate of about 10 centimeters per century since about 6,000 years ago (Curray, 1960; 1961; 1965). Most researchers agree that, along the Southern California coastline, the sea level approximately 6,000 years ago was 12 to 16 feet below its current elevation (Curray, 1960; 1965; Inman and Veeh, 1966). More importantly, the world coastline, including that of California, has been shaped largely within this 6,000-year period with the sea at, or within 16 feet of, its present level (Bird, 1985).

In its simplest form, a sea-level model can be written as follows:

$$dx/dt = \frac{\textit{Sea Level Rise}}{\textit{Shore Platform Slope}}$$

where: dx/dt is the horizontal rate of erosion

If we assume a constant sea-level rise for the last 6,000 years of 10 cm/century and a shore platform slope of 1:40, then dx/dt equals 0.13 feet per year. Using this model, this represents the average erosion rate of the headland. This model further assumes uniform lithology and a uniform elevation of the cliff-platform junction.

Empirical and Analytical Techniques

The scientific community has been actively engaged in developing numerical models to assess rates of shoreline erosion. Numerical models attempt to address both the landward retreat of the sea cliff, and the development of the shore platform.

In its simplest expression, predictive cliff-erosion models take the following form (Sunamura, 1977):

$$dx/dt \propto \ln\left(\frac{f_w}{f_r}\right)$$

where: dx/dt is the horizontal rate of erosion

f_w is the wave force

f_r is the rock resistance, which is proportional to its unconfined compressive strength.

Depending upon the proportionality constants used, a three-fold reduction in compressive strength can result in a four- or five-fold increase in erosion rate.

Of particular interest in numerical modeling is the fact that a minimum or critical wave height capable of causing erosion exists, below which, for a given rock lithology, no erosion would occur. Additionally, the rate of erosion increases in logarithmic proportion to increase in wave force, which is substantially less than a linear increase in wave energy. Importantly, however, these numerical models describe the mechanical erosion of intact rock of assumed uniform lithology, and do not account for the accelerated erosion caused by the hydrodynamic component of wave forces that occurs in fractured rock.

6.3.4 *Field Investigative Work*

Because relatively extensive existing engineered fills mask much of the near-surface site geology, we conducted a field exploratory program utilizing a "Gradall" excavator with a 30-foot reach to expose geologic and soil units on the face of the coastal bluff where necessary. Geologic contacts and bedding attitudes were mapped by the geologist rappelling down the bluff slopes, and surveyed-in with a Total Station Instrument in order to locate contacts as precisely as possible, and to record local variations in bedding attitudes.

6.4 Findings and Conclusions

6.4.1 *Rate of Marine Erosion*

The rate of marine erosion for the coves has been estimated as a reasonable multiple of the rate for the sea cliff along the main coastline alignment and from comparison of aerial photographs taken as early as 1939 with new photographs taken for this project. A lower limit was set based on minimum rates considered necessary to maintain the near-vertical upper bluffs of the overlying Bay Point Formation.

Significant differences in erosion rates are evident between the coastal bluff headlands and the coves. This is, in part, due to the difference in lithology and intensity of jointing, the wave-direction dependence of transmitting erosive energy into the cove, and the energy-focusing effect of surge channels.

The less-resistant upper Point Loma Formation in the coves is judged to have the approximate erosion resistance of the overlying Quaternary rocks in which direct measurements were made along the coast north of Point Loma. The rate for the sea cliff at the site was, in part, based on recognition of the more resistant nature of the lower section of Point Loma Formation. Locally, jointing and faulting further reduces erosion resistance near the apex of the cove (see Figure Nos. A-3 and A-9). This leads to an upward adjustment in rate of erosion in the head of the cove.

The focusing effect of surge channels increases the erosive effectiveness of waves arriving from the same, or approximately the same, direction. This leads to an upward adjustment in the rate of erosion.

Aerial photographs taken at irregular intervals from as early as 1939 to the present were compared in estimating the rate of cliff retreat. Our interpretation of these photographs indicated bluff-top retreat rates varying from approximately 0.21 to 0.33 feet per year. Review of aerial photographs further indicates that upper-bluff retreat was primarily due to sloughing, whether by undermining or due to localized slope instability. Thus, the rate of marine erosion in the heads of coves is substantially greater than subaerial erosion, and the development of a more stable (flatter) upper slope cannot be initiated due to the excessive rate of undermining that occurs at the head of the coves. This was the case at the northerly cove, until grading operations caused surface drainage to be directed over the bluff-top at the head of the cove. We have chosen to use a rate of 0.33 feet per year for bluff-top retreat at the head of the cove.

TABLE A-1
SITE ACCELERATIONS FOR ACTIVE FAULTS AND POTENTIALLY ACTIVE FAULTS

Fault-Zone Alignment	Least Distance to Site[3] (miles)	Maximum Credible (MC) Earthquake[4] (M_w)	Peak Horizontal Acceleration for MC[5](g)		Maximum Probable (MP) Earthquake[6] (M_w)	Peak Horizontal Acceleration for MP[5](g)	
Active[1]:			Mean	M+σ		Mean	M+σ
Rose Canyon	6	7	0.33	0.45	6½	0.25	0.35
Coronado Banks	8	7¼	0.32	0.44	6½	0.22	0.30
San Diego Trough	17	7½	0.22	0.30	6½	0.12	0.15
San Clemente	38	7¾	0.12	0.15	6¾	0.07	0.09
Elsinore	47	7½	0.09	0.12	7¼	0.08	0.10
Agua Blanca	62	7½	0.06	0.09	7	0.04	0.06
San Jacinto	68	7½	0.06	0.07	7¼	0.05	0.06
San Miguel	70	7¼	0.05	0.06	7	0.04	0.05
San Andreas	96	8¼	0.06	0.08	8	0.05	0.07
Potentially Active[2]:							
La Nacion	11	6½	0.16	0.22	5½	0.09	0.12

Notes:

1) Fault zones reported to have displaced Holocene-age (11,000 years old or younger) geologic units, with geologic evidence of high slip rate, the probable sources of recorded earthquakes of M_w 5.0 or greater, or classified by professional judgement of available information.
2) Fault zones of low slip rate that displace Quaternary-age (11,000 to 1.6 million years old) geologic units.
3) Measured from Regional Fault Map reproduced in this report.
4) Estimated to be the maximum earthquake capable of occurring. Derived from the maximum rupture length using length-magnitude relationship like Slemmons (1977, Fig. 27). Relationship used is for California earthquakes greater than or equal to M_w 6.0, plus one standard deviation. Multiple segment ruptures from Anderson and others, 1989, are also considered.
5) Estimated from Campbell (1987) unconstrained mean and mean +1σ for use as an index to prioritize earthquake sources. Acceleration value is for shallow soil. Ground acceleration may be less for sites on significant thicknesses of sedimentary rock. Design accelerations usually vary.
6) Estimated to be the earthquake with a 10 percent probability of being exceeded during a 100-year interval. Based on single segment ruptures having return periods up to 1000 years (Anderson and others, 1989). Magnitude values are a judgement based on regional seismicity, earthquake activity near the fault, and geologic expression of fault displacement.

TABLE A-2

GEOTECHNICAL PARAMETERS

MATERIAL TYPE	STRENGTH CHARACTERISTICS C (psf)	STRENGTH CHARACTERISTICS ϕ (deg)	Total Unit Weight γt (pcf)	Qualitative Compressibility
Recompacted On-Site	200	31	120	Moderate
Bay Point Formation	200	34	125	Low
Point Loma Formation				
Peak	1800	36	130	Low
Residual	1100	33	130	Low

TABLE A-3
COMPACTION-INDUCED LATERAL EARTH PRESSURES
FOR NON-YIELDING WALLS

Vertical Distance from Top of Wall (feet)	Pressure (psf)
0	0
1	450
1 <Z <12.14	Ps = 425+25Z
Z ≥12.14	Ps = 60Z
Z (feet)	Z (feet)

Notes: 1) Compaction equipment assumes either vibratory plate or roller, and does not include effects due to rammer plates.

2) Example 1: For Z = 10 feet Ps = 425 + 25(10) or Ps = 675 psf

Example 2: For Z = 15 feet Ps = 60(15) or Ps = 900 psf

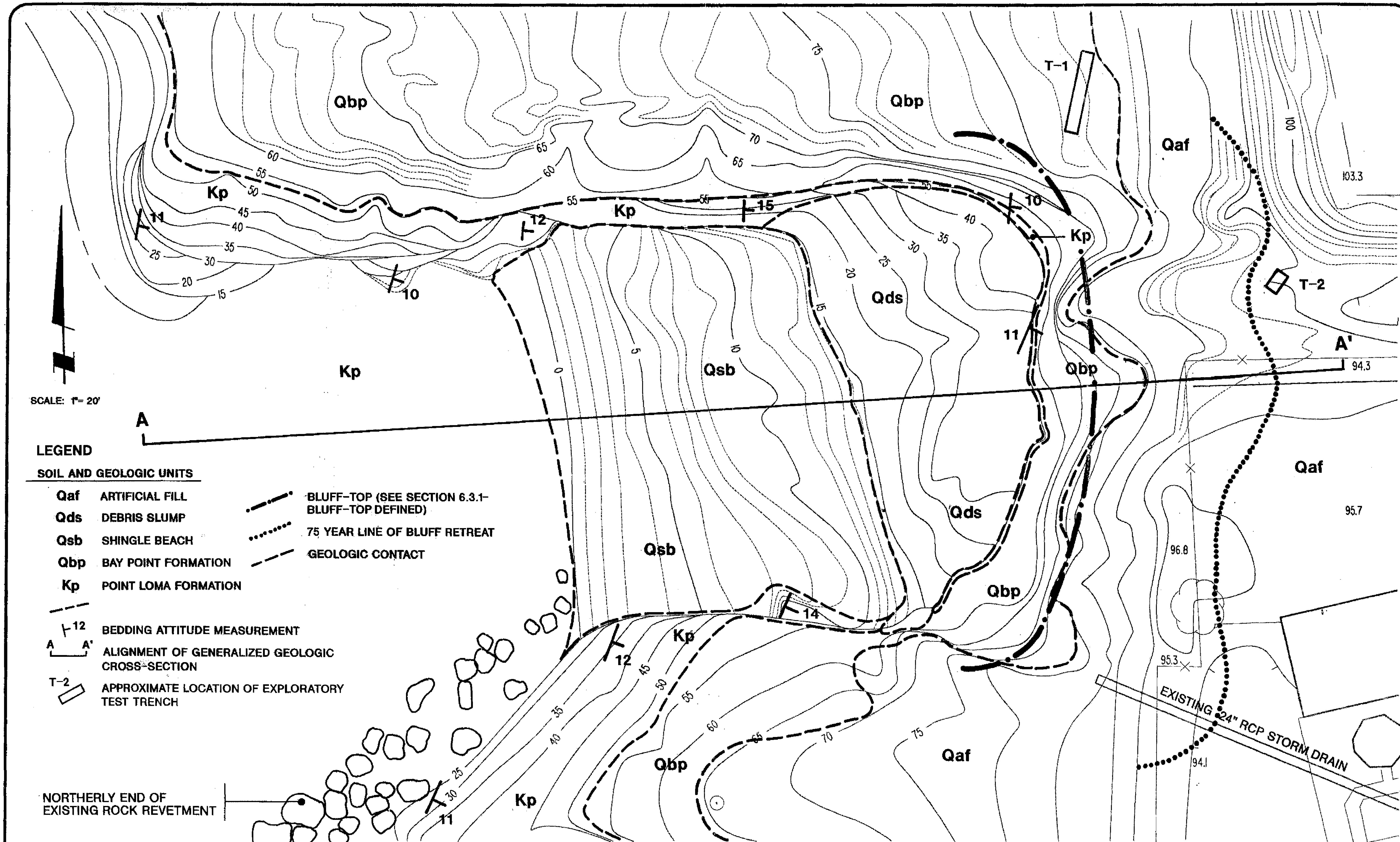
3) Hydrostatic forces are not included. Hydrostatic forces need to be included, unless the wall has been designed to preclude the buildup of hydrostatic pressures.

TABLE A-4

Hindcast (1900-84) Waves Exceeding 3 m Height Near 35° N
(Seymour et. al., 1984)

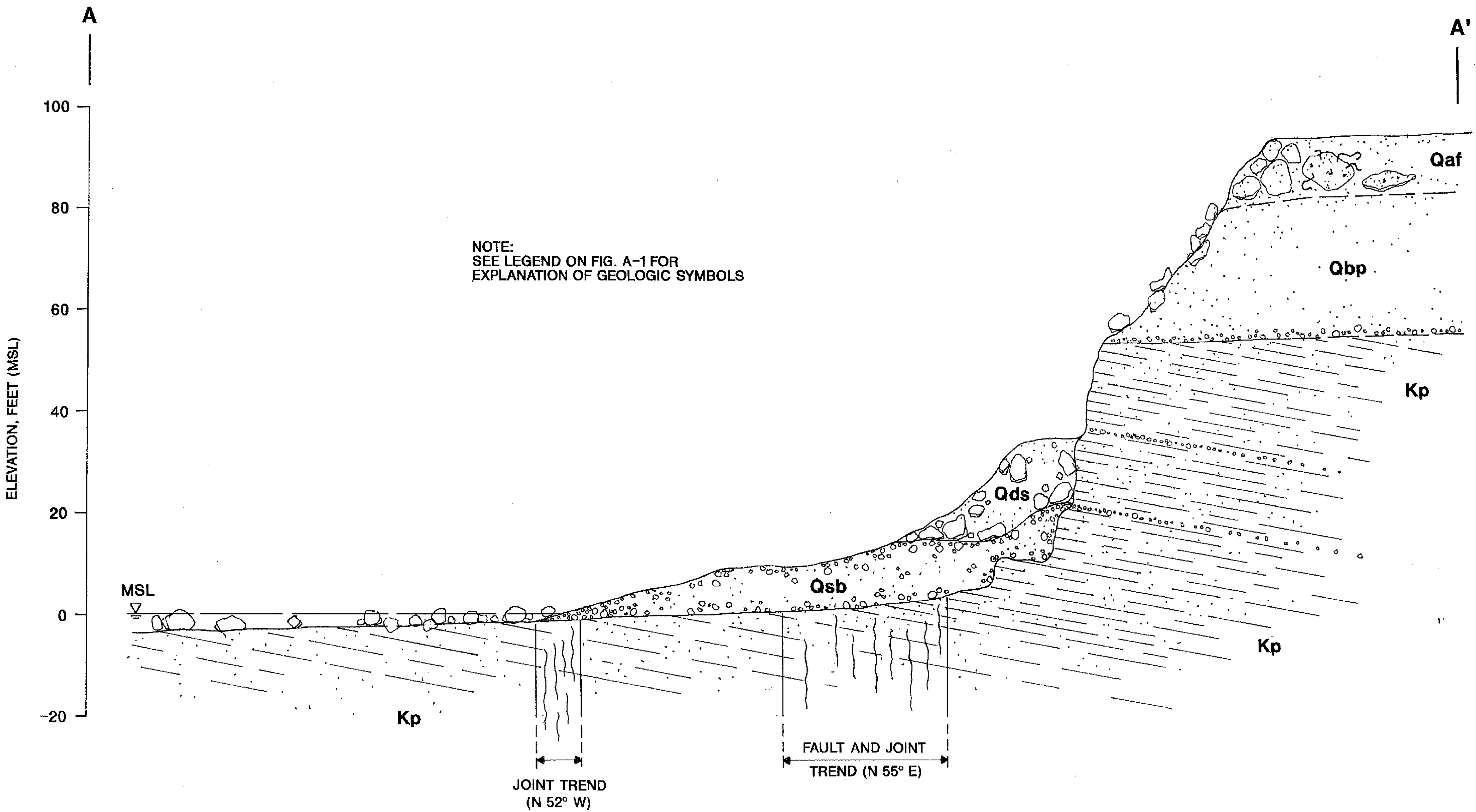
EXTREME WAVE EPISODES EXCEEDING 3 M. (BASIC SERIES)
1900 - 1984

DATE	SIG. HT. (m)	MAX. PERIOD	DIRECTION
13 MAR 05	8.8	15	247
17 NOV 05	3.3	17	286
31 DEC 07	5.3	16	282
12 MAR 12	3.2	12	220
26 JAN 14	5.8	13	223
03 FEB 15	7.5	14	235
01 JAN 18	3.7	16	280
12 FEB 19	5.3	12	299
20 DEC 20	4.7	13	301
15 OCT 23	3.7	16	296
01 FEB 26	6.9	15	257
03 JAN 27	5.8	20	287
06 NOV 28	4.0	17	294
01 JAN 31	3.9	16	276
28 DEC 31	7.4	18	288
19 DEC 35	4.7	16	267
13 DEC 37	4.5	16	272
06 JAN 39	7.9	19	285
25 SEP 39	4.5	15	205
24 JAN 40	4.3	16	267
25 DEC 40	5.7	16	270
20 OCT 41	3.3	17	294
30 DEC 45	3.9	19	285
13 FEB 47	3.9	16	265
04 NOV 48	4.7	18	300
15 NOV 53	5.7	17	269
15 JAN 58	3.1	22	280
26 JAN 58	6.8	14	259
05 APR 58	7.7	18	289
16 FEB 59	5.1	14	244
09 FEB 60	8.1	19	295
22 DEC 60	3.4	17	276
31 JAN 63	4.2	16	260
10 FEB 63	5.9	15	256
19 NOV 65	4.0	15	277
07 DEC 67	4.0	15	298
06 FEB 69	4.7	13	222
04 DEC 69	3.6	17	278
06 DEC 69	4.9	22	274
14 DEC 69	5.7	17	290
19 DEC 69	4.7	18	281
26 DEC 72	4.1	15	289
21 FEB 77	5.2	18	280
29 OCT 77	5.5	20	299
16 JAN 78	6.0	13	240
01 JAN 80	4.7	20	272
17 FEB 80	6.1	18	249
22 JAN 81	4.3	20	258
28 JAN 81	7.0	17	262
13 NOV 81	4.9	18	284
01 DEC 82	6.4	14	295
18 DEC 82	6.4	20	288
25 JAN 83	6.1	17	278
27 JAN 83	7.3	22	279
10 FEB 83	6.7	25	281
13 FEB 83	4.9	17	268
01 MAR 83	8.2	20	258
14 NOV 83	5.0	17	290
03 DEC 83	7.0	17	285
25 FEB 84	6.4	17	300



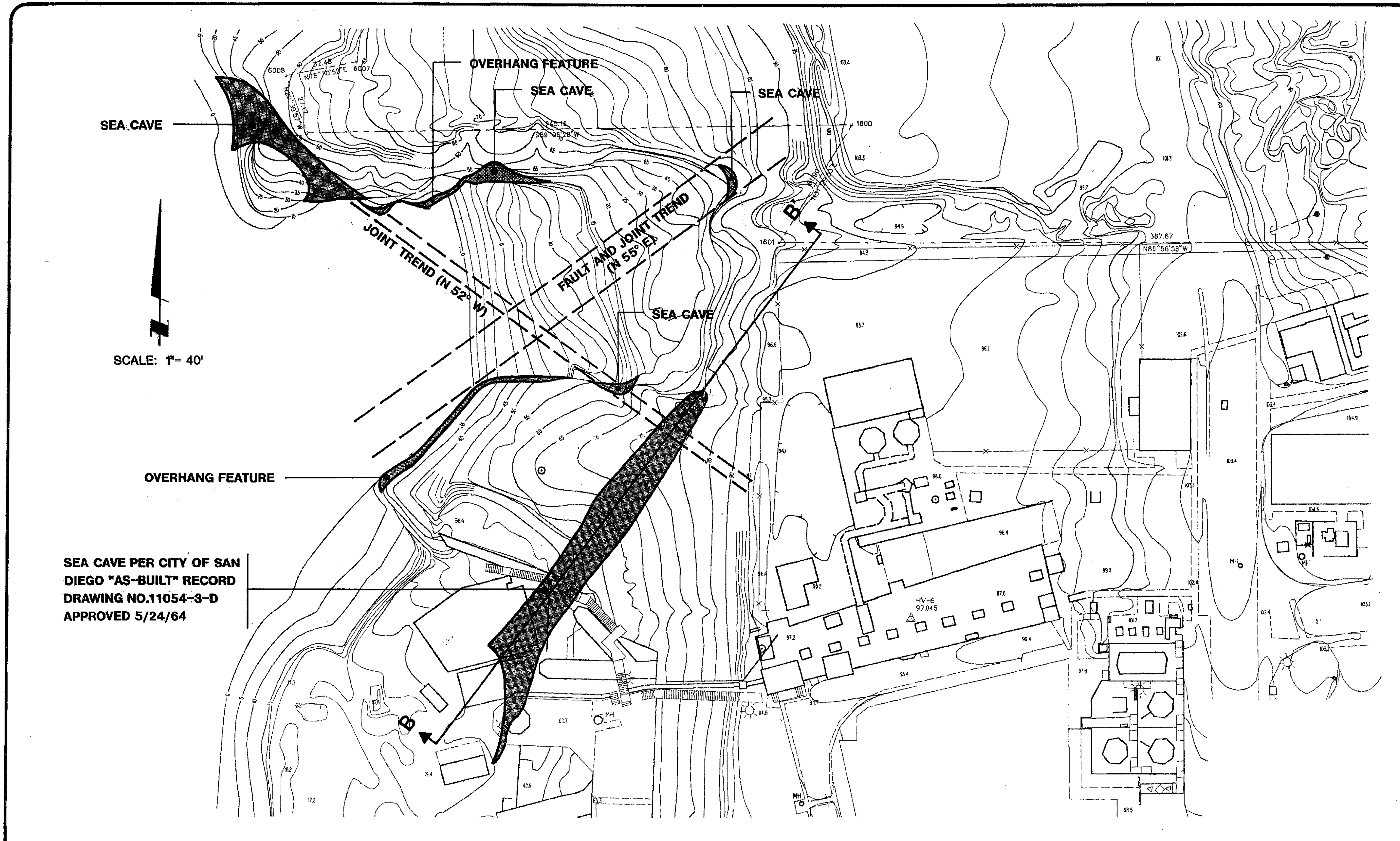
Project NORTH SHORELINE PROTECTION IMPROVEMENTS	SITE PLAN AND GEOLOGIC MAP	Project No. 1601BG	Figure A-1

BAJADA/GDC ASSOCIATES



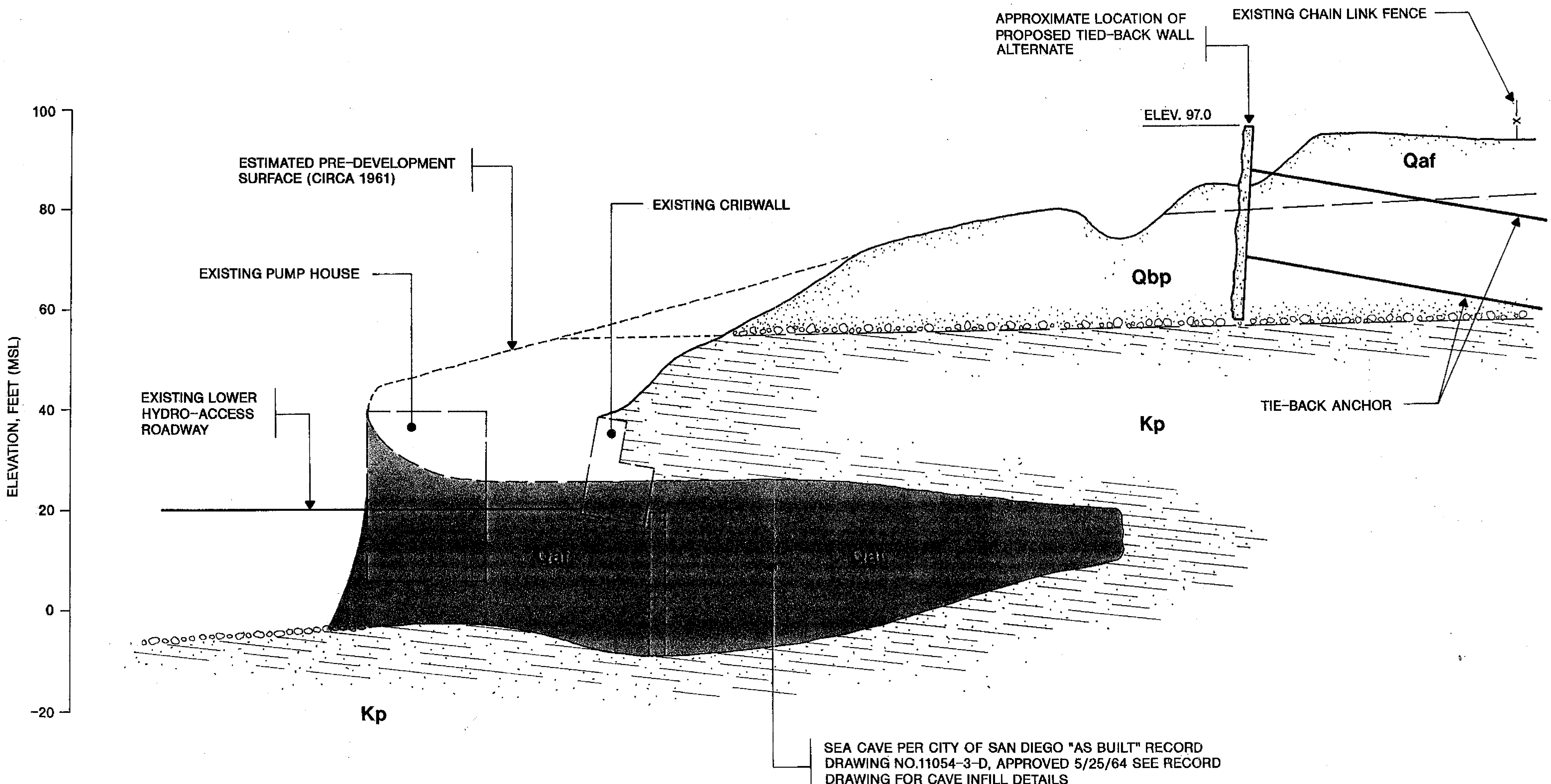
Project NORTH SHORELINE PROTECTION IMPROVEMENTS	GENERALIZED GEOLOGIC CROSS-SECTION A-A'	Project No. 1601BG	Figure A-2

BAJADA/GDC ASSOCIATES



Project NORTH SHORELINE PROTECTION IMPROVEMENTS	SEA CAVE FEATURES AND STRUCTURAL DISCONTINUITIES	Project No. 1601BG	Figure A-3

GROUP DELTA CONSULTANTS, INC.



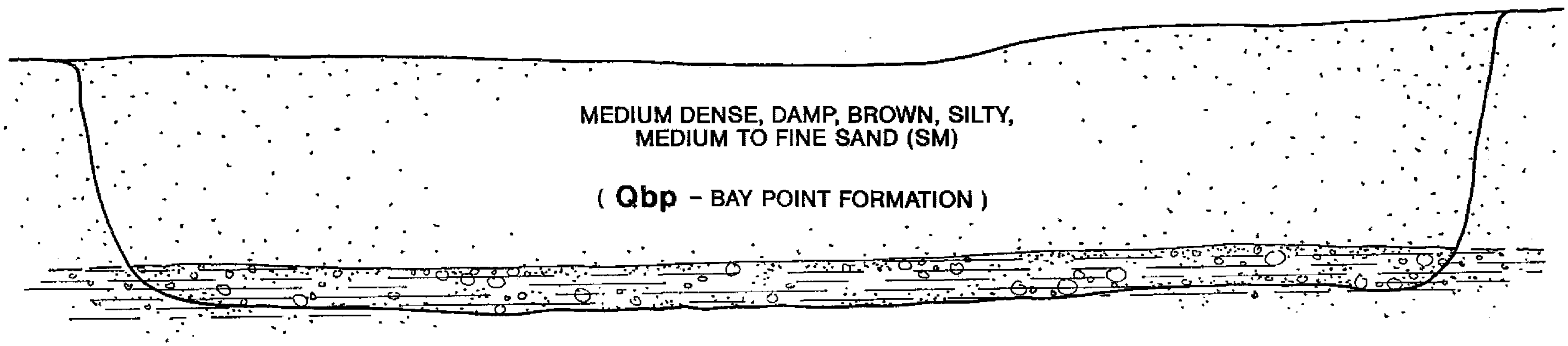
NOTE:
SEE LEGEND ON FIG. A-1 FOR EXPLANATION OF GEOLOGIC SYMBOLS

SECTION B-B'

SCALE: 1"= 20'

Project		Project No.	Figure
NORTH SHORELINE PROTECTION IMPROVEMENTS	GENERALIZED GEOLOGIC CROSS SECTION B-B'	1601BG	A-4

GROUP DELTA CONSULTANTS, INC.



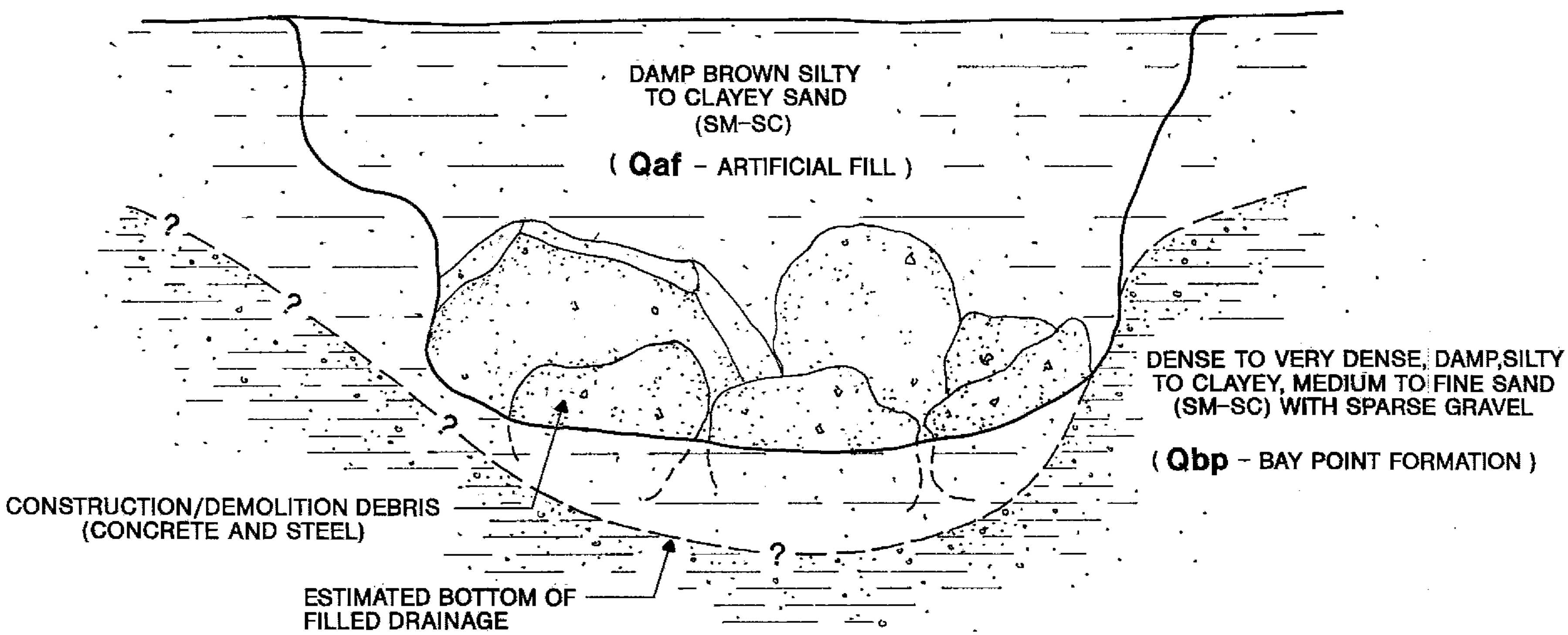
SCALE: 1"= 2'

TEST TRENCH T-1

Project No. 1601BG	NORTH SHORELINE PROTECTION IMPROVEMENTS	Figure A-5

BAJADA/GDC ASSOCIATES

NOTE
EXTENSIVE CONSTRUCTION
DEBRIS (CONCRETE AND STEEL)
ENCOUNTERED IN TEST TRENCH.

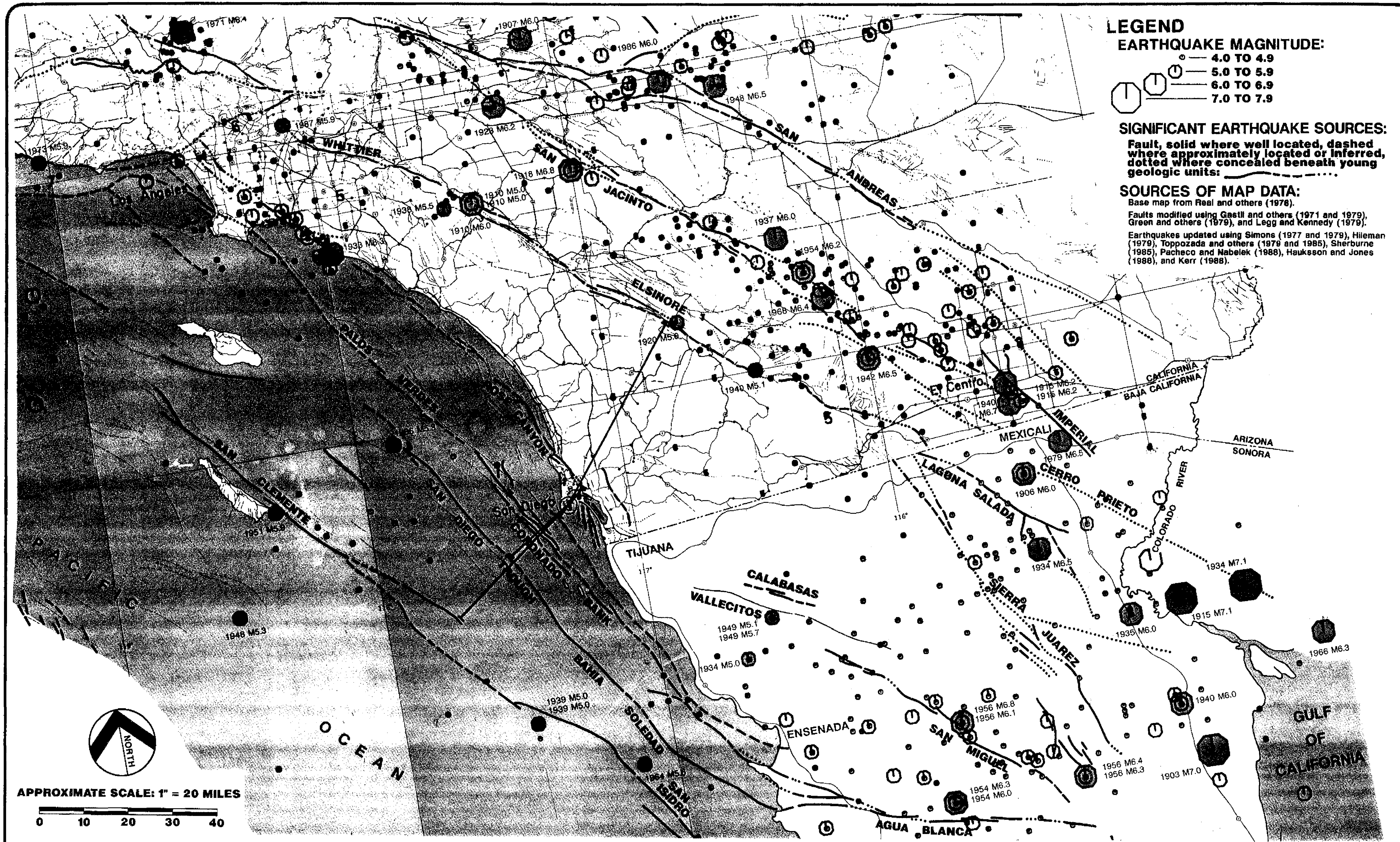


SCALE: 1"= 2'

TEST TRENCH T–2

Project No. 1601BG	NORTH SHORELINE PROTECTION IMPROVEMENTS	Figure A–6

BAJADA/GDC ASSOCIATES



Project		Project No.	Figure
NORTH SHORELINE PROTECTION IMPROVEMENTS	REGIONAL FAULT MAP	1601BG	A-7

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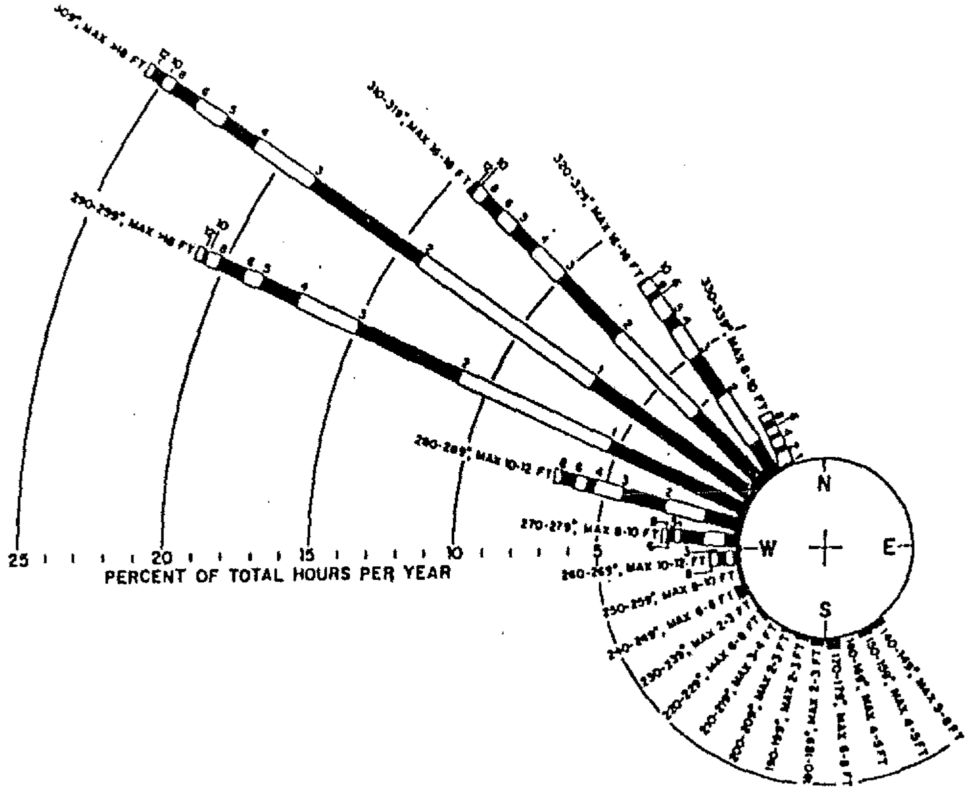
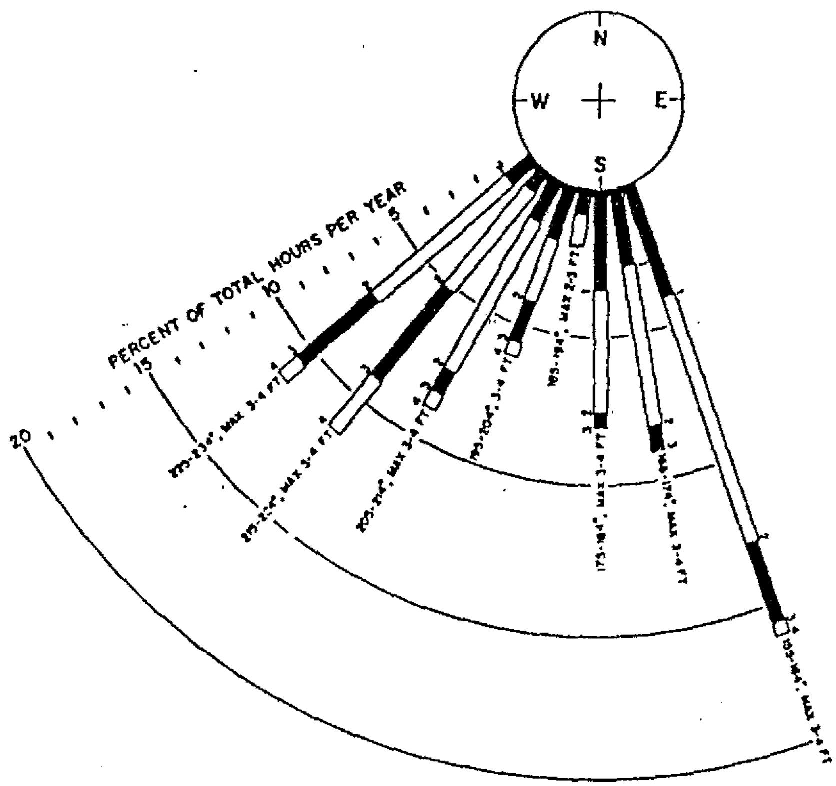


Figure 3.2.2-7 Wave rose, Station A, annual average (1956-58) northern hemisphere swell (Marine Advisers, 1961).



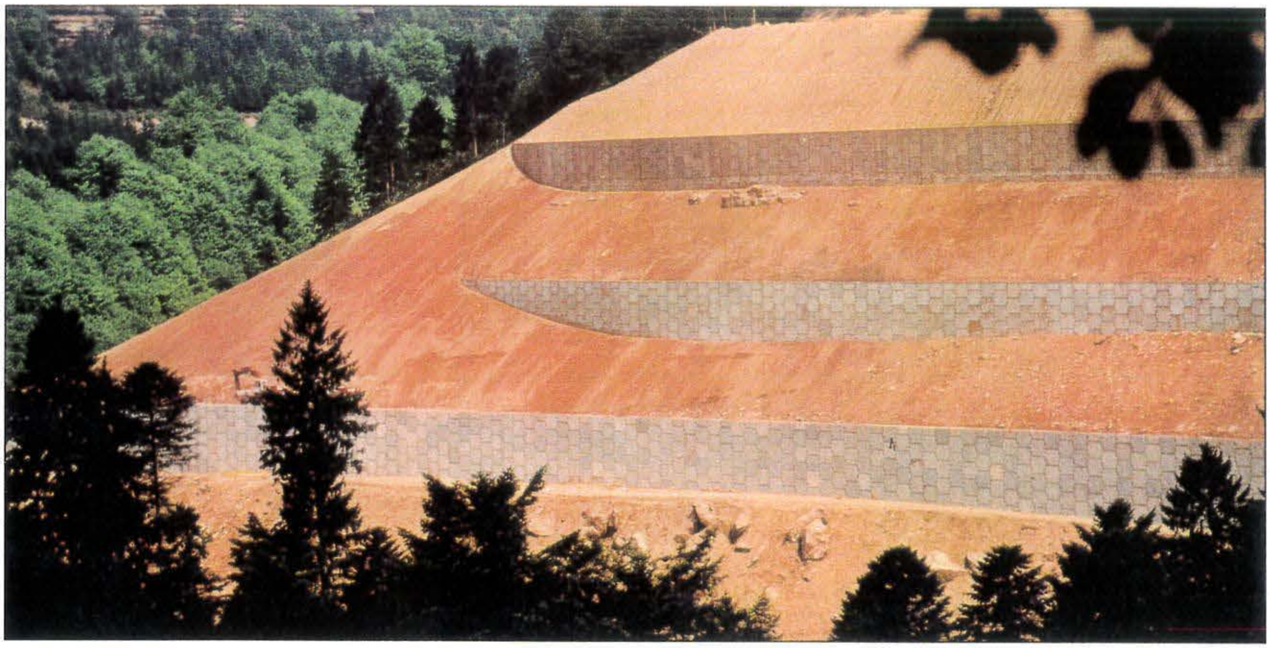
Wave rose, Station A, annual average (1948-50) southern hemisphere swell (Marine Advisers, 1961).

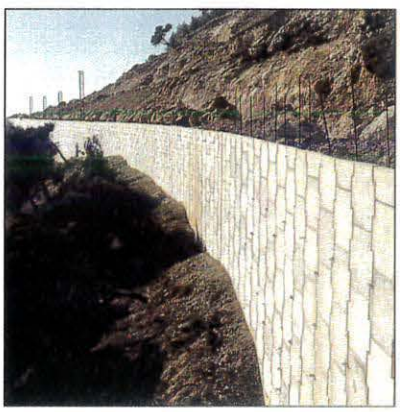
FIGURE NO. A-8

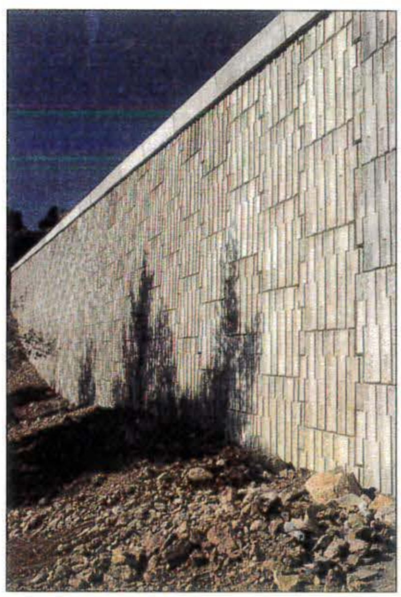
WAVE ENERGY, FOCUSED ALONG THE ALIGNMENT OF A JOINT TREND (N52°W), HAS ERODED TWO SEA CAVES, ONE AT THE NORTHWEST CORNER (ABOVE PHOTO), AND THE OTHER AT THE SOUTHEAST CORNER (LEFT PHOTO) OF THE COVE. THE COVE ITSELF REPRESENTS A REMNANT COLLAPSED SEA CAVE.

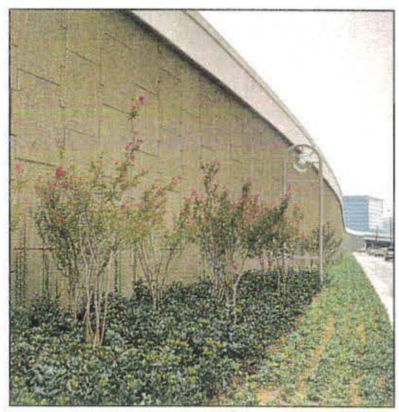
APPENDIX B
PRODUCT LITERATURE

REINFORCED EARTH® RETAINING WALLS

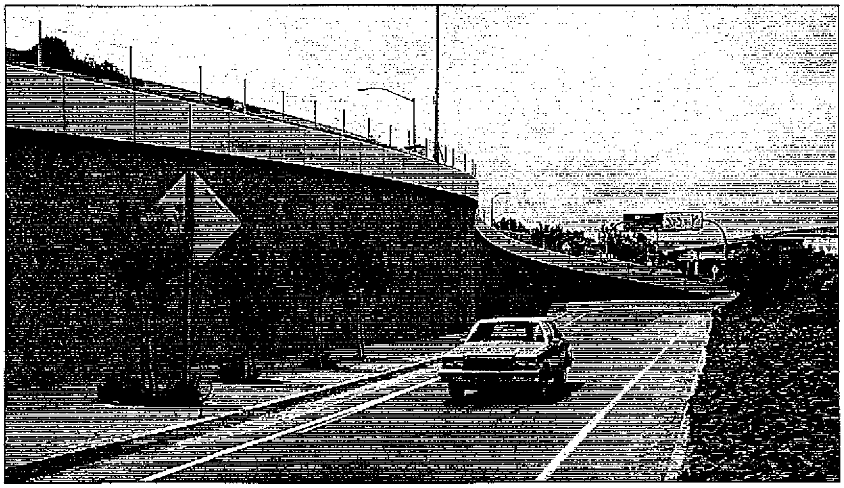








reinforced earth®



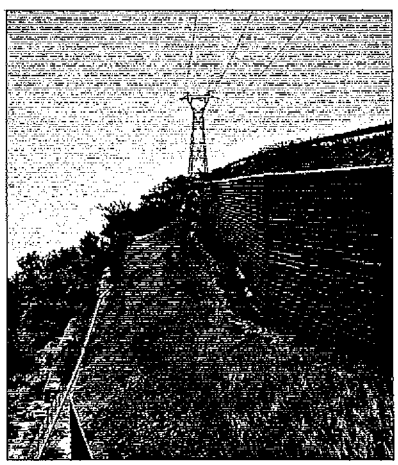
Since the invention of Reinforced Earth® a little over 20 years ago, and its subsequent development for use in transportation and civil engineering applications, more than 7,000 retaining walls have been constructed throughout the world.

On first examination, the design and performance of these structures may appear simple, as in the case of large gravity walls. However, the internal mechanism of Reinforced Earth is unique and complex. Following the conception and initial studies by Henri Vidal, the Reinforced Earth Group continued research in order to extend the knowledge of the behavior of Reinforced Earth in retaining wall applications. That research continues today. It has involved varied but complementary methods, ranging from laboratory studies on reduced-scale models and instrumentation of actual projects and full-scale experimental walls, to computerized mathematical studies including finite element method analysis.

The synthesis of this considerable mass of data has made it possible to develop practical, accurate design procedures for current projects. It has also enabled designers to optimize the geometry of Reinforced Earth structures to their intended use and environmental setting.

In addition to the experience acquired on difficult sites, research results have contributed to the careful design of retaining structures for large, sloped embankments (or terraced structures), for walls built on poor or compressible ground, and for projects built in mountainous areas, on both rocky flanks and unstable talus slopes.

The following pages chronicle some of the research conducted on the performance of Reinforced Earth retaining structures, and describe how the results of this research have been applied to the internal and geometric designs of the projects.



List of Symbols

T — tensile force in reinforcing strip

σ_v — vertical stress

σ_h — horizontal stress

K — ratio between σ_h and σ_v

K_a — coefficient of active earth pressure

K_o — coefficient of earth pressure at rest

γ — unit weight of soil

L — reinforcing strip length

H — wall height

φ — angle of internal friction

N — number of strips per unit of wall facing surface

f^* — coefficient of apparent friction

β — inclination angle of resultant earth pressure behind fill

q — surcharge load

Z — depth of fill above reinforcing strip

Gravity Structure

Massive gravity-type retaining walls were the first, and remain the most widespread application of Reinforced Earth (Fig. 1). The term "massive" clearly implies that the material, although composite and flexible, forms a continuous, homogenous block. It is this block, as heavy and stable as a large masonry wall, that retains the fill or soil. In addition to its own weight, the block transfers the effects of surcharges and earth pressures to the foundation, and distributes them evenly over the entire width of its base. Due to the flexibility of the wall, this wide foundation prevents concentrations of loads, making it possible to build a Reinforced Earth retaining wall directly on the ground, even on very poor foundation soils.

Research and Development of Overall Structure Dimensions

Therefore, designers of Reinforced Earth structures initially preferred the stable shape of a wide rectangular form, one which imposed a minimum and uniform stress on the ground (Fig. 2).

While early research was performed on structures of this type, nearly triangular wall shapes were also conceptualized for use where appropriate (Fig. 3). Whereas the shape of the structure was adapted to the profile of the ground, a somewhat intuitive effort was also made to make the design encompass all potential internal failure planes.

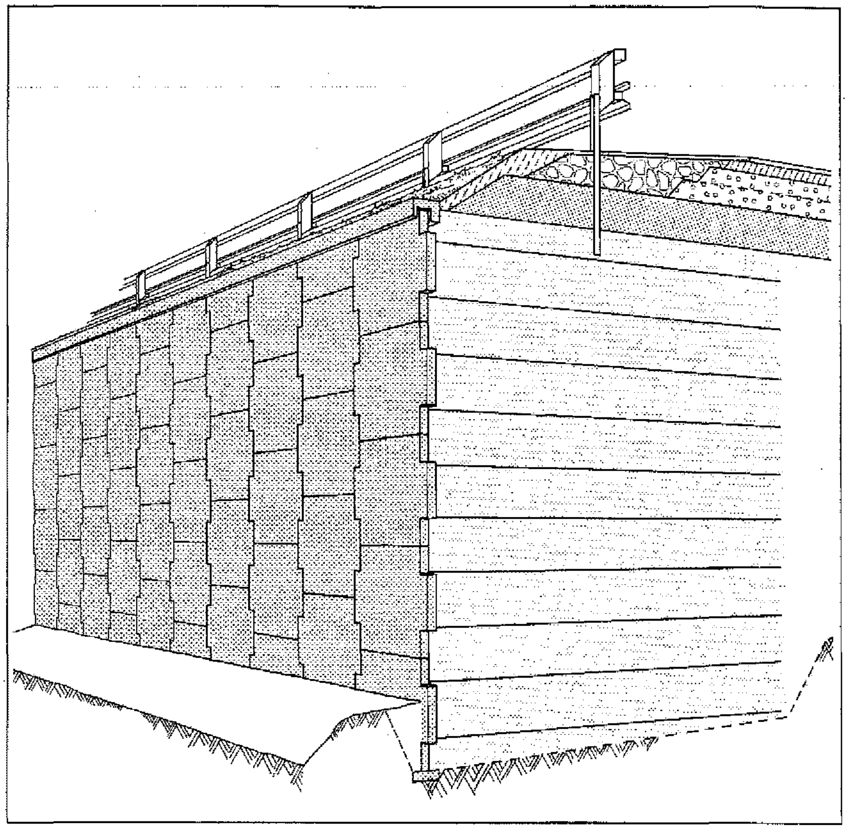


Figure 1: Cut-away view of a typical Reinforced Earth retaining wall.

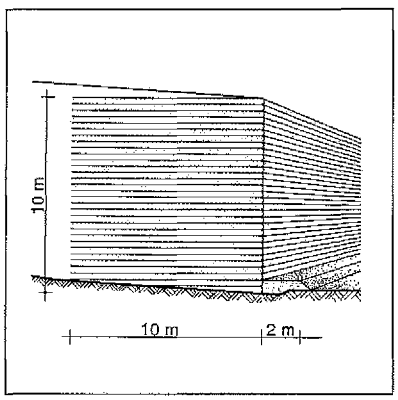


Figure 2: Normandy Highway, Incarville, France, 1968.

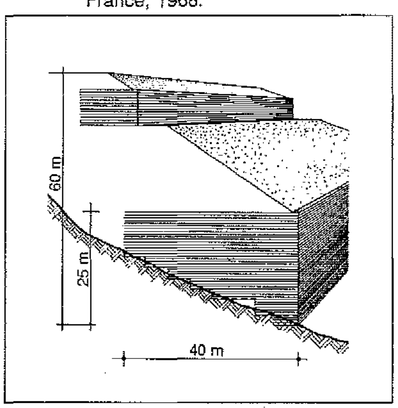


Figure 3: French-Italian Highway, Le Peyronnet, 1968.

Thus, in addition to theoretical and laboratory studies on the fundamental behavior of Reinforced Earth (including, among others, triaxial tests), a considerable amount of applied research on Reinforced Earth retaining walls was conducted. There were three primary objectives of this research:

- To study the behavior of the structure; to analyze how such external factors as the structure's own weight, surcharge loading, and earth pressures behind the Reinforced Earth mass affect the development and transmission of internal stresses within the backfill, the reinforcing strips, and the facing.
- To analyze the influence of a structure's overall geometry, shape, and width-to-height ratio on its behavior — and the resulting foundation loading; and to study the effects of variations in the modulus of elasticity of the reinforcements on overall structural performance.
- To develop new designs for Reinforced Earth retaining walls. For example, the development of a narrow profile wall for projects where it is more important to reduce the overall width of a structure built in a cut than to limit the pressures exerted by the retaining wall on the foundation (Fig. 4). This has been a topic of recent research. Results of the most important research activities are discussed in following sections.

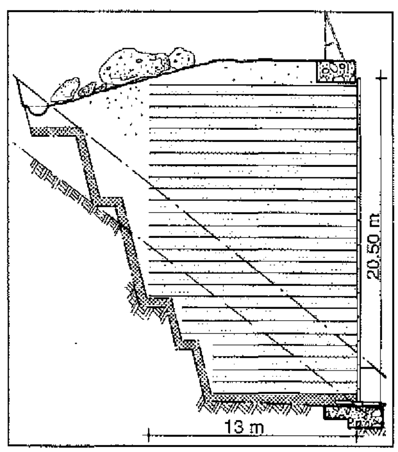


Figure 4: Austria-Italian Highway, Casello, 1985.

Since the introduction of Reinforced Earth technology, laboratory-scale models have proven useful in gaining an understanding of the behavior of full-scale structures. Henri Vidal, the inventor of Reinforced Earth, constructed and studied sand and paper models in the early 1960s. From 1969 to 1982, with the cooperation and financial support of the Reinforced Earth Group, independent laboratories around the world conducted more than a dozen scale-model research projects.

Bidimensional Models

At the Laboratoire Central des Ponts et Chaussées (LCPC) in Paris, two-dimensional models of steel rods and sheets of aluminum made it possible to distinguish between failures due to slippage or to breakage in the reinforcing strips (Fig. 5).

Tridimensional Models

Subsequently, with the use of three-dimensional sand models built by LCPC and the Institut National des Sciences Appliquées (INSA) in Lyon, it was possible to observe the shape of failure surfaces. Computational methods were deduced from these model studies.

Centrifuge Testing

Unfortunately, stresses and other forces in small-scale models are themselves small, and therefore difficult to measure directly.

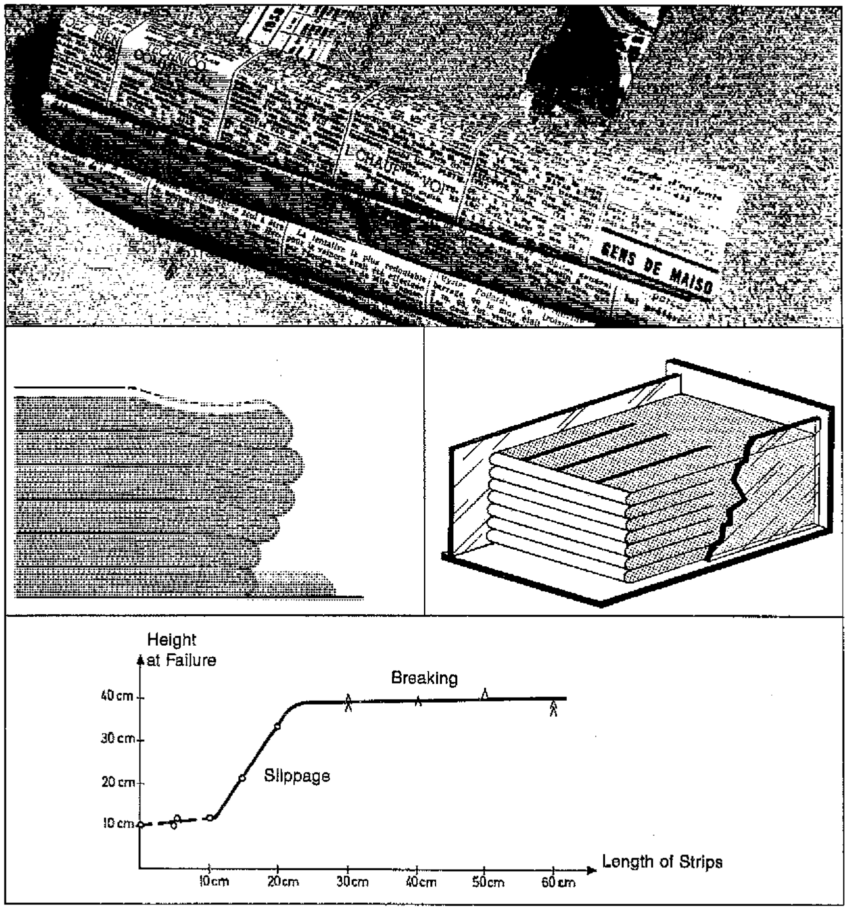


Figure 5: Scale Models. Different modes of failure.

In a centrifuge, on the other hand, an acceleration of 100g in a 20cm-high model can produce the same levels of strain that are found in a 20 meter-high wall. These loads can then be measured using sensors and strain gauges. In Manchester, England, the Transport and Road Research Laboratory (TRRL) used this method to measure tensile force levels in reinforcing strips and the pressure levels at the base of a model wall (Fig. 6).

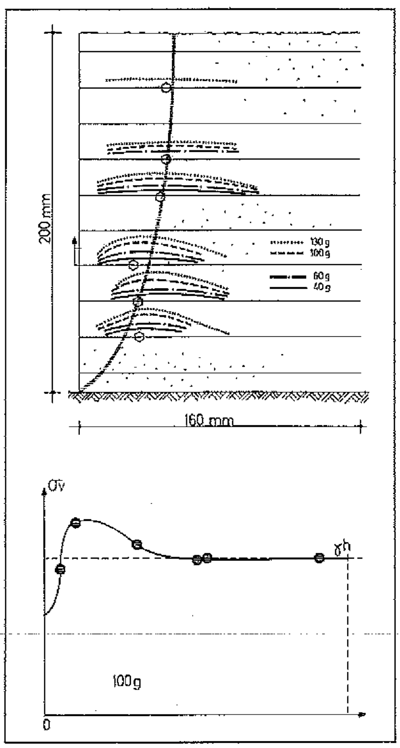


Figure 6: Centrifuge.

Large Scale Models

Subsequently, using "static" models more than one meter high (Fig. 7), LCPC obtained consistent results for the variations in tensile force levels along reinforcing strips as a function of depth, and for the foundation pressures transmitted to the ground. Analogous methods were used at the Ecole Nationale des Ponts et Chaussées (ENPC) in France to study sloped and heavily surcharged structures.

However, scale models do not faithfully reproduce the true flexibility of the wall facing, the effects of compaction or dilatancy of the fill. Tests on full-size structures were indispensible during initial development, and remain so today.

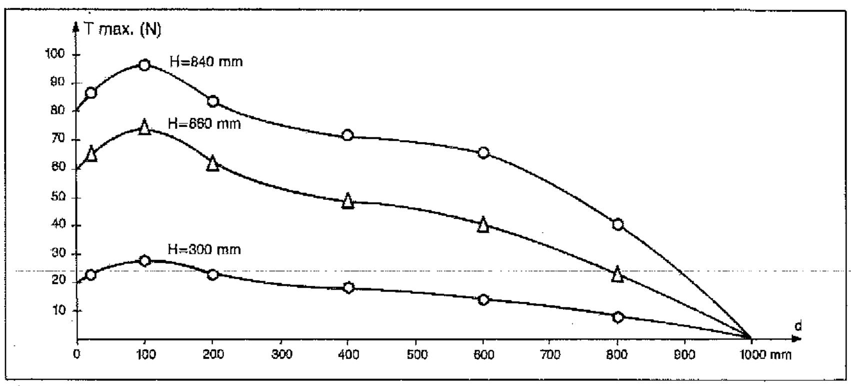
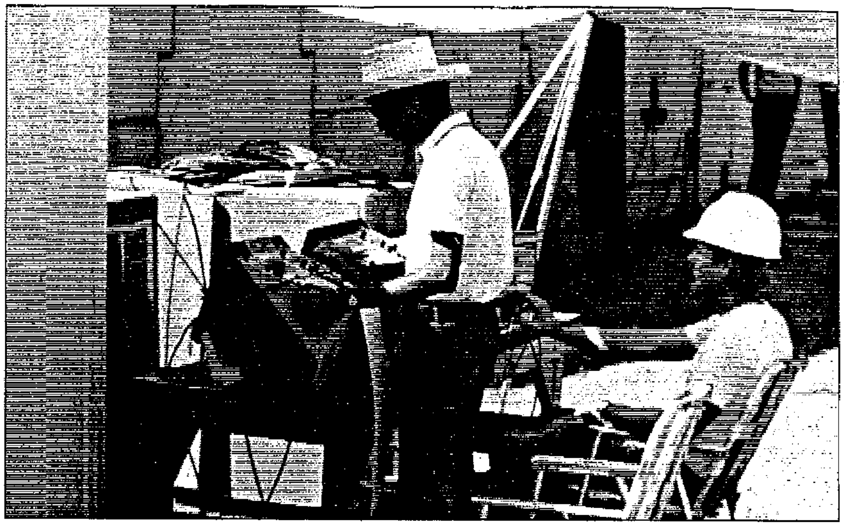
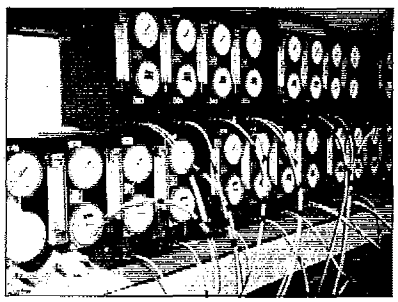


Figure 7: Large Scale Models. Variation of tension in bottom strips as height of wall increases.

The best method for obtaining reliable data on the behavior of walls or embankments is the instrumentation of structures built under normal field conditions. This has been done at a number of sites around the world, beginning with several of the early Reinforced Earth projects. Many of these experiments were financed in whole or in part by companies of the Group. Today, results from measurements taken on 20 actual structures and nine experimental structures are available for analysis. The findings of this instrumentation program are presented below.




Reinforcing strips fitted with gauges.


Readers of pressure cells.

Instrumentation

Instrumentation for studying Reinforced Earth structures typically includes:

- Strain gauges set at predetermined intervals along a majority of the reinforcing strips in a single vertical cross-section. Each point requires a gauge on both the upper and lower surfaces to eliminate the effects of local bending. To minimize localized irregularities due to the normal construction process, several reinforcing strips in the same layer are usually instrumented.
- Total pressure cells for measuring stress levels in the fill, particularly behind the wall facing and at the base of the structure. Here, too, measurements may sometimes vary due to local irregularities.
- Surveys, inclinometers, and settlement indicators to record movements of the structure.

Of course, it has not been possible to instrument all monitored Reinforced Earth walls in such a complete way. However, cross-checks using a sufficient number of experimental and instrumented structures, as shown in Figure 8, have enabled researchers to draw solid, convincing conclusions regarding the behavior and mechanisms of Reinforced Earth.

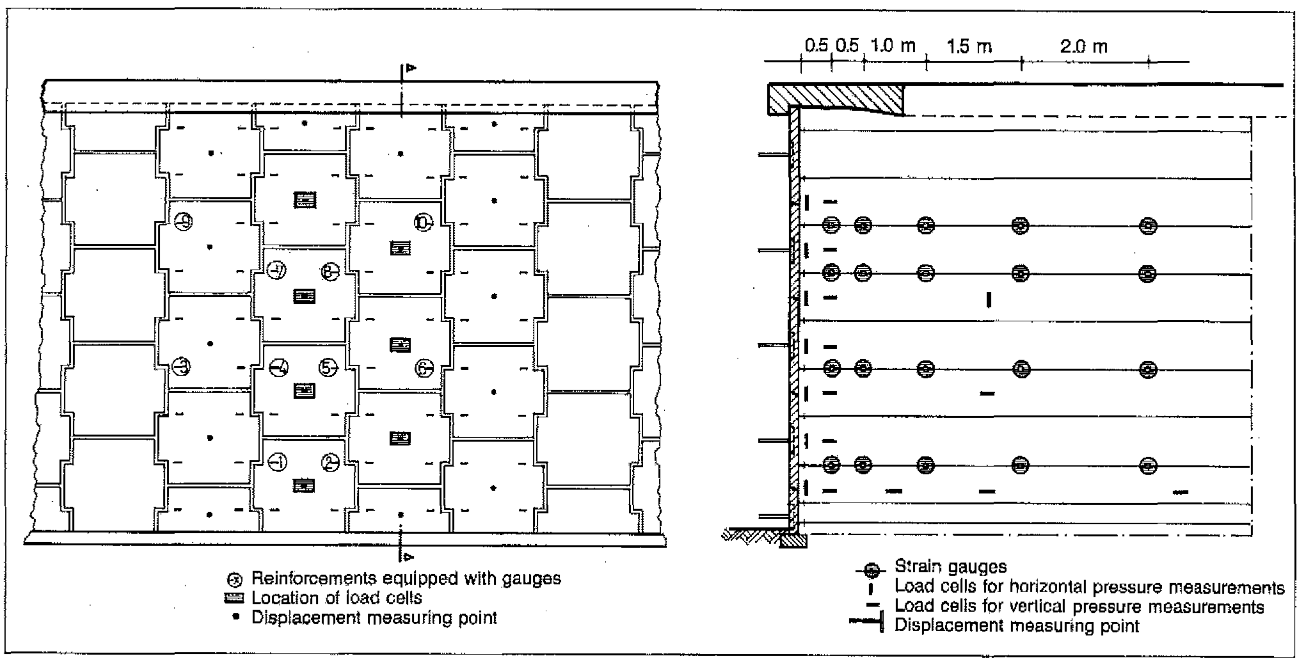


Figure 8: Instrumentation of the Fremersdorf wall, Germany.

Tensile Forces in the Reinforcing Strips

Strain gauge measurements show the variations in tensile forces along reinforcing strips, or (at a minimum) the averages of these variations (Fig. 9).

From these curves, it is possible to locate the point of maximum stress at each level of reinforcing strips. By connecting these points, one can derive the line of maximum tensile force in the structure.

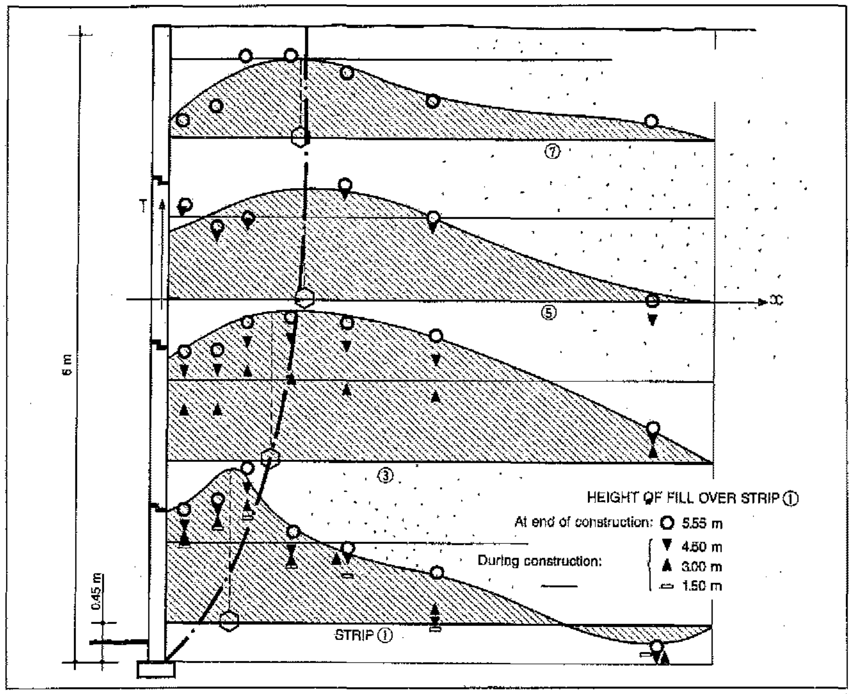


Figure 9: Silvermine wall, South Africa, 1976.

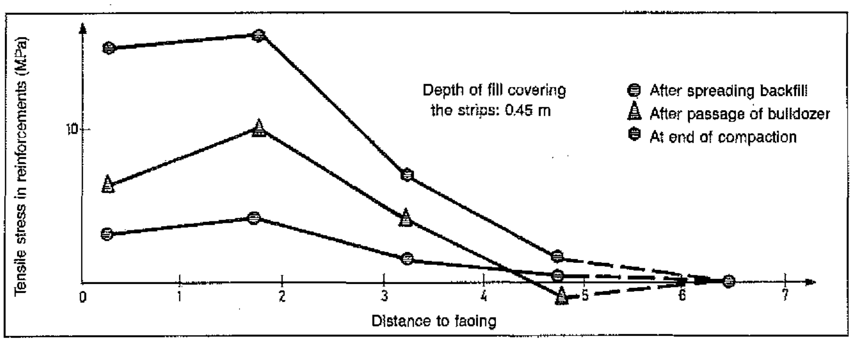


Figure 11: Granton wall, Great Britain, 1973.

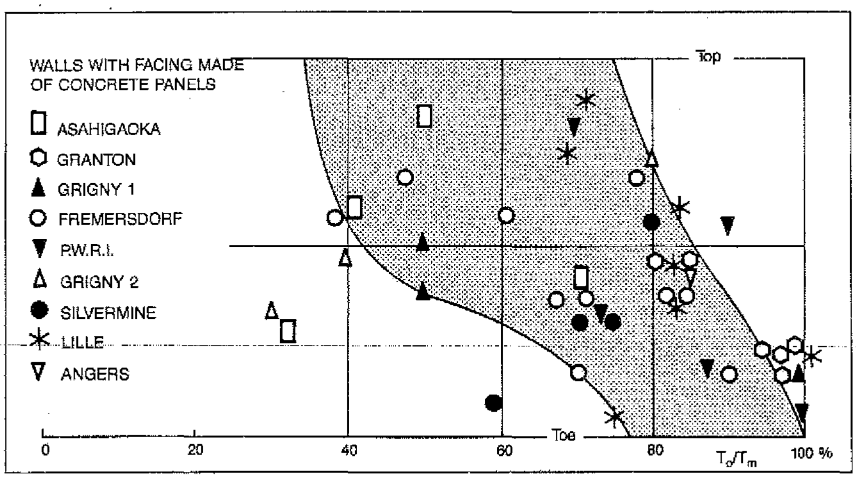


Figure 12: Measured tension of the connection compared to measured maximum tension.

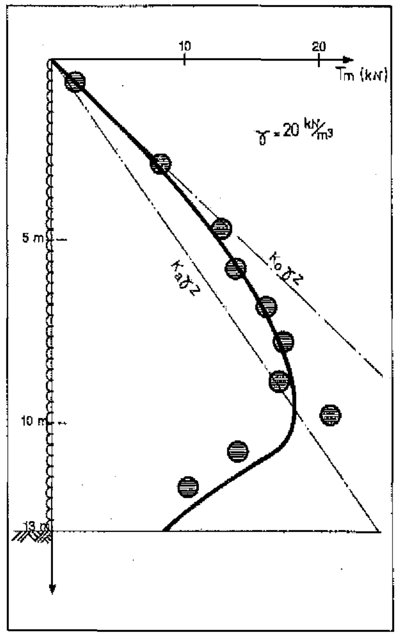


Figure 10: Thionville wall, France, 1972 (prior to construction of the bridge beam seat).

Variation of the Tension with Depth

A graph showing variations in maximum tensile force as a function of depth can also be derived from these measurements (Fig. 10). In all projects, it has been observed that stress is not entirely proportional to depth. Stresses are higher at the top of the wall and lower at the base.

This was confirmed with measurements taken on a wall built at Granton, Great Britain, where it was shown that higher stresses at the top of the structure are caused by forces developed during compaction (Fig. 11). In contrast, stresses are often reduced at the base of the wall because the foundation soil — due to its cohesion — "relieves" the lower levels of reinforcing strips.

Stresses at the Wall Facing

When gauges are placed close enough to the facing of a Reinforced Earth retaining wall, it is possible to estimate forces at the connection between facing and the reinforcing strips. Otherwise, these forces are derived from the horizontal stresses measured behind the facing where total pressure cells have been installed. These measurements have shown that stress levels at the face are lower than the maximum levels (Fig. 12). However, test results are far from uniform, thus demonstrating the necessity and value of numerous measurements.

Pressure at the base. Earth pressure due to the retained fill. When an adequate number of pressure cells are installed under the base of a structure, it is possible to determine the variation and magnitude of the foundation loading exerted by the wall on the underlying soil (Fig. 13). The Fremersdorf wall (Fig. 8) provided such an opportunity. Tests on that structure demonstrated that loading is greater toward the front of the structure due to earth pressure imposed by the retained fill behind the wall. In addition, the total load was slightly greater than the total weight of the wall, indicating that the thrust behind the structure was inclined.

The difference between total loading and weight, and the location of the resultant, makes it possible to compute the thrust angle β.

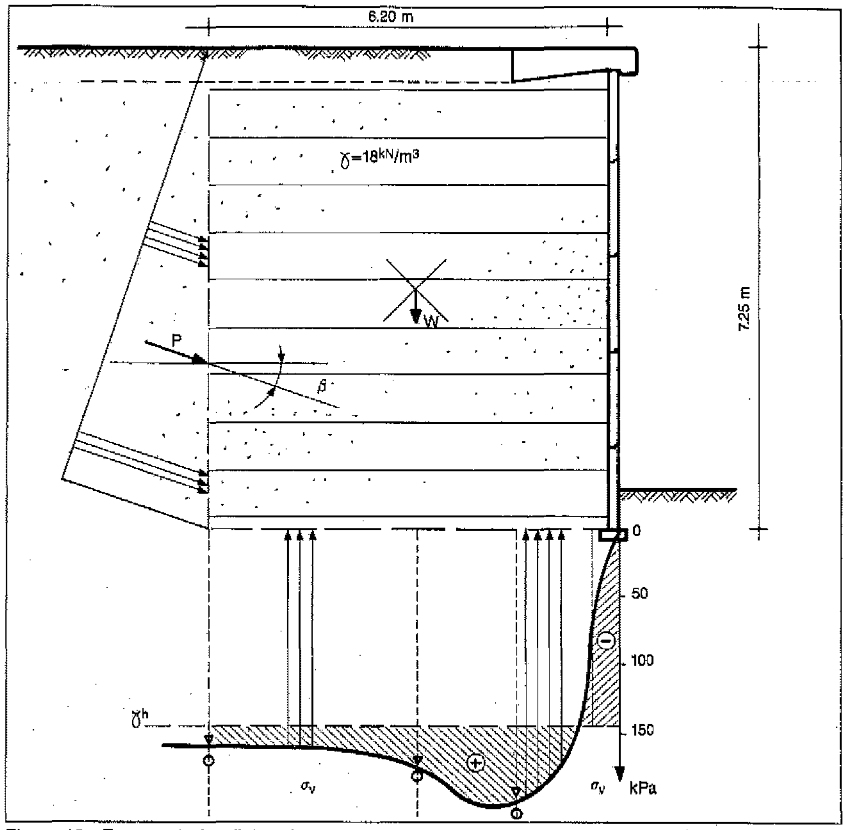


Figure 13: Fremersdorf wall; bearing pressure.

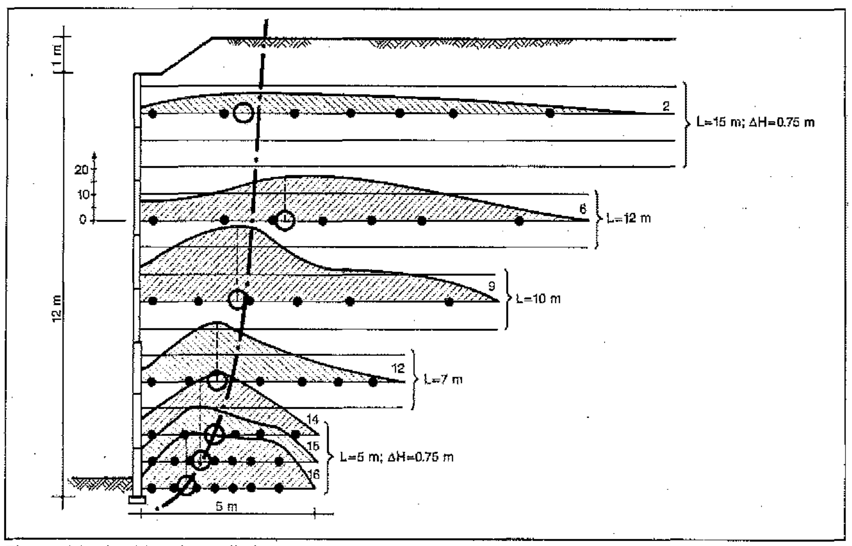


Figure 14: Asahigaoka wall, Japan.

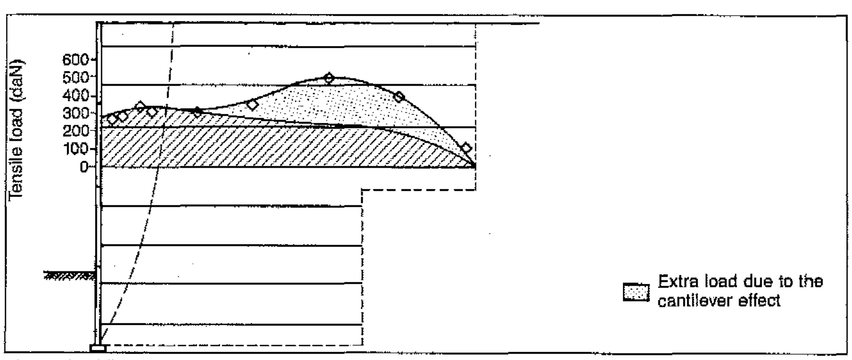


Figure 15: Lille wall, France, 1972 (before construction of abutment superstructures).

Trapezoidal Walls

Among the structures in service that have been instrumented, several were built with reinforcing strips shorter at the base than at the top. The Asahigaoka wall in Japan is a case in point. The results of field measurements on this structure are in very close agreement with those obtained on walls with a rectangular cross-section of the same general dimensions (Fig. 14).

As observed in a structure in Lille, France, excessively extended lengths of reinforcing strips used for abutment loading can result in the secondary maximum of tensile stress due to the cantilever of the extended Reinforced Earth fill over the more compressible random backfill (Fig. 15).

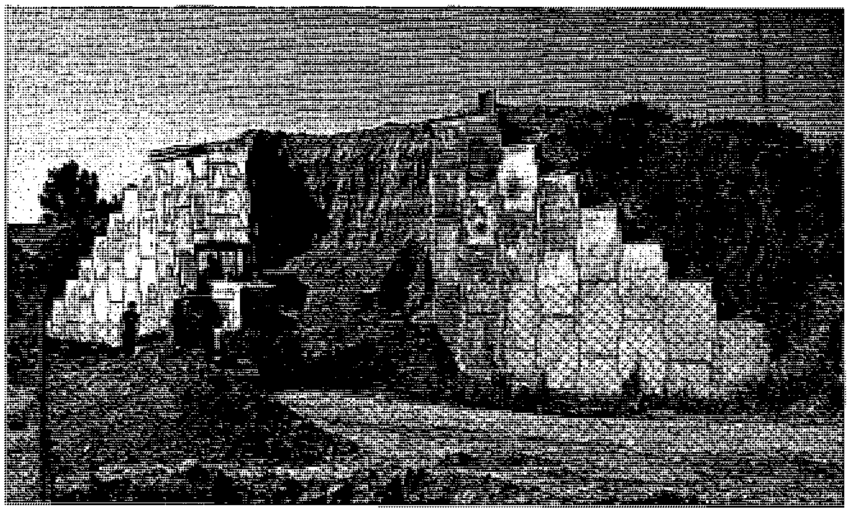
Actual projects that have been instrumented were, of course, designed according to the current state-of-the-art standards. These walls have been subjected to no more than design loadings.

Experimental walls, on the other hand, are generally intended for testing new designs or technologies. They are often loaded to failure.

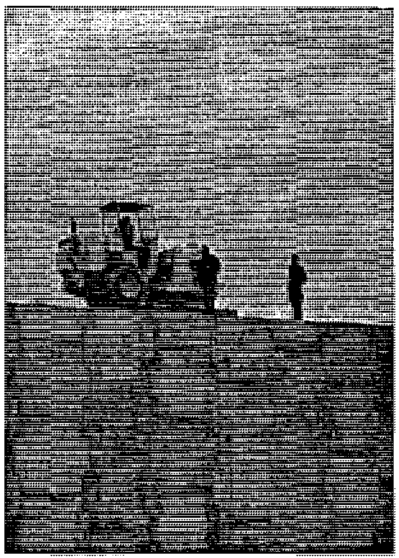
The Triel wall, for example, built in 1975 by La Terre Armée S.A., France, was constructed to verify the behavior of high-adherence reinforcing strips, and to check the effects of vibrations.

Another experimental wall, erected in 1976 by the U.S. Army Corps of Engineers in Vicksburg, Mississippi, was built using non-standard metal facing panels of the military's design. It was then loaded to failure.

As part of the research into the durability of buried steel, Tierra Armada S.A., Spain, built a six-meter-high wall in Madrid in 1977 using thin reinforcing strips (0.6 mm). The structure was then inundated with brine to accelerate corrosion. Failure developed along a surface very similar to the line of maximum tensile force established by previous research. In addition, at the moment of failure the overall residual resistance of the reinforcements was very close to the sum of the theoretically predicted maximum tensile forces.



Madrid



Triel

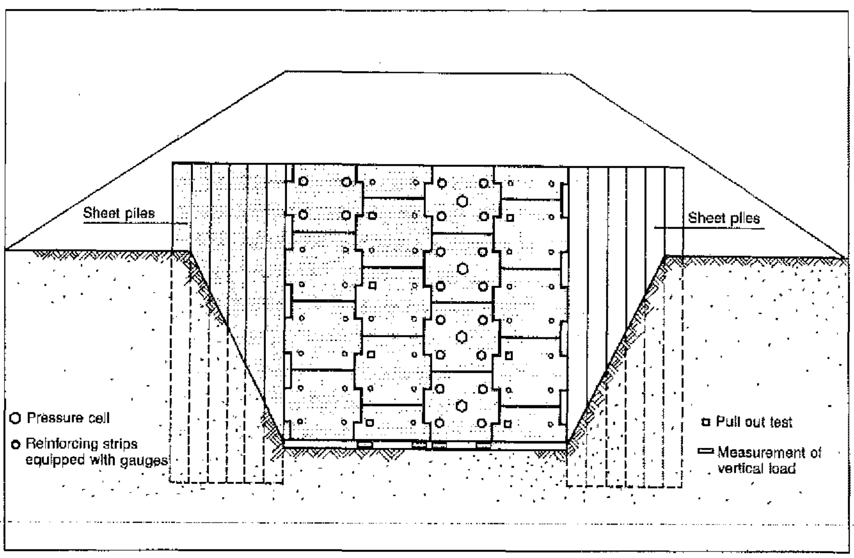


Figure 16: PWRI wall in Japan, 1978.

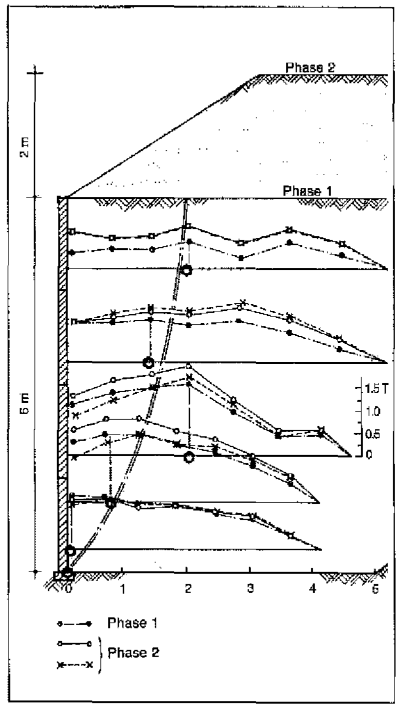


Figure 17: PWRI wall. Cross section and tensile forces.

Surcharge

In 1978, the Public Works Research Institute (PWRI) of Japan built a six-meter-high experimental wall (Fig. 16) to study the effects of a sloping surcharge and the consequent role of the wall facing. When the reinforcing strips were unbolted from the outside, the face at first remained stable, confirming the hypothesis that it plays only a minor structural role. When the structure was loaded to failure, the break in the fill closely followed the theoretical line of maximum tensile stress, as shown in Figure 17.

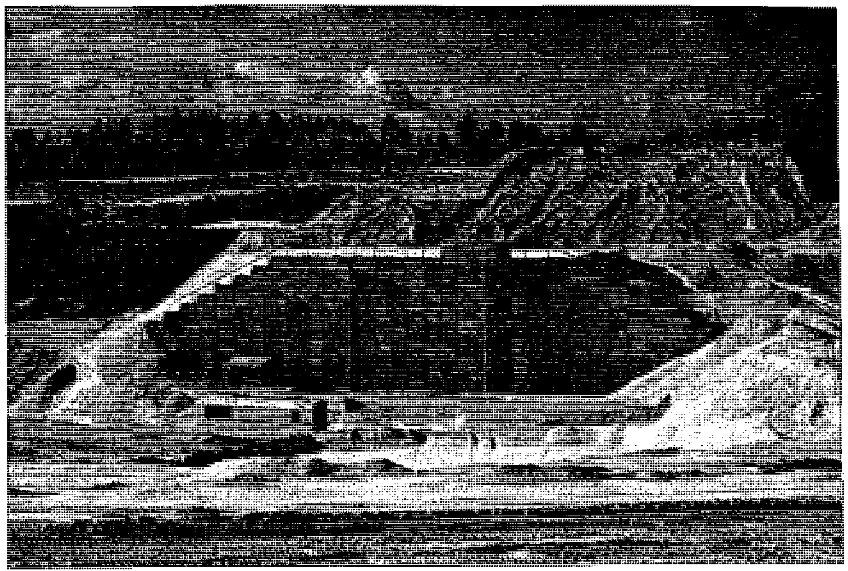
Narrow Profile Walls

In 1983, The Reinforced Earth Company, United States, built and instrumented a six-meter-high wall with short reinforcing strips at Millville, West Virginia. The structure has two cross-sections: one is rectangular, with reinforcing strips of 2.7 meters (L/H = 0.45); the other is trapezoidal, with reinforcing strips of 1.8 to 3.0 meters (L/H = 0.30 to 0.50)

An experimental wall of the same type, but with a height of 10.5 meters, was constructed in France near Fontainebleau by La Terre Armée S.A. and LCPC (1986/7). Building the wall, equipping it with complete instrumentation, and taking measurements cost about $360,000, of which 90 percent was provided by the Reinforced Earth Group.

As in Millville, the project included two separate sets of measurements. (Fig. 18). One set involves a rectangular cross-section and reinforcing strips of five meters (L/H ≅ 0.5); the cross-section of the other set is trapezoidal, with reinforcements four to six meters in length (L/H ≅ 0.4 to 0.6).

Deliberately built on relatively poor foundation soils and with fill of average quality, this experimental wall was designed to test practical construction conditions as well as the overall stability and design principles associated with a narrow profile structure.



Fontainebleau wall, France.

Instrumentation of Fontainebleau

In total, the instrumentation shown in Figure 18 included:

- 355 locations for determining the tensile stresses in reinforcing strips (two strain gauges per location).
- 52 locations for measuring the vertical stresses in the soil (Glotzl cells).
- 24 load cells, specially developed for this purpose and designed to measure the force transmitted from one panel to another within the wall face.
- 28 settlement meters, six inclinometers, plumb lines, and topographical benchmarks for assessing movements.

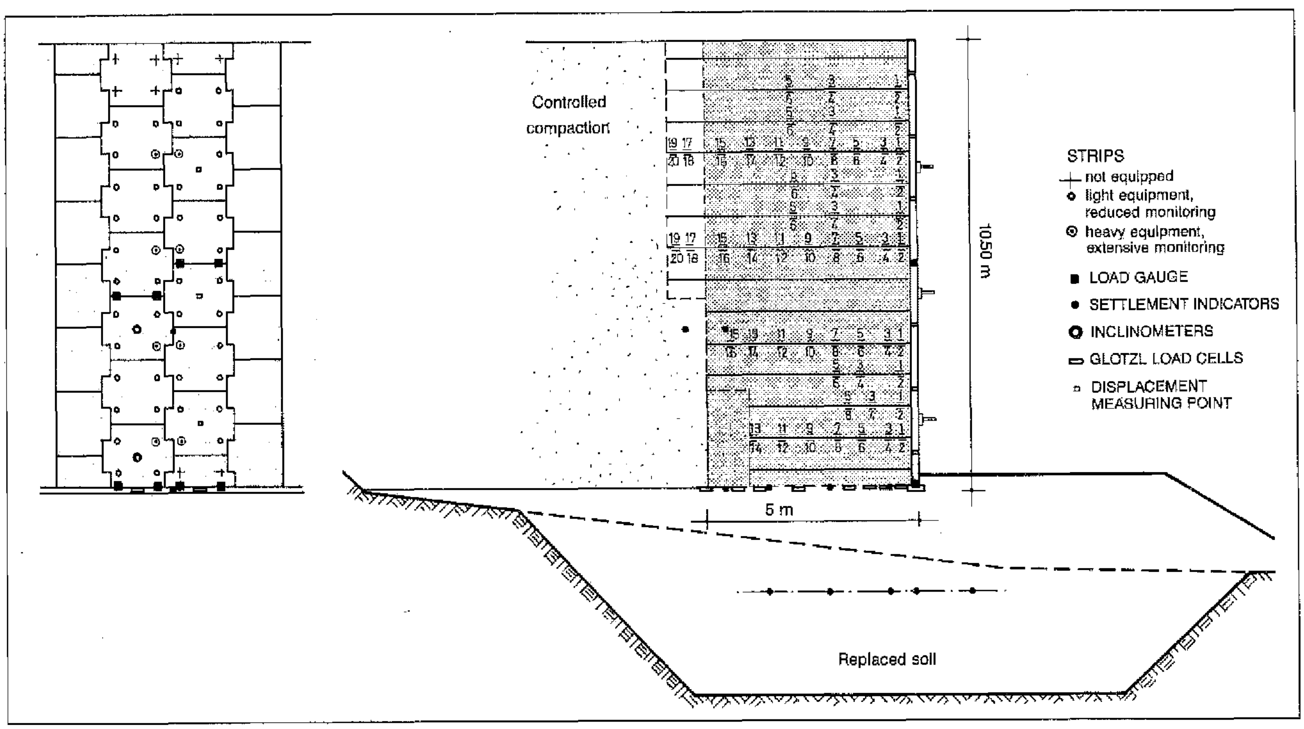


Figure 18: Instrumentation.

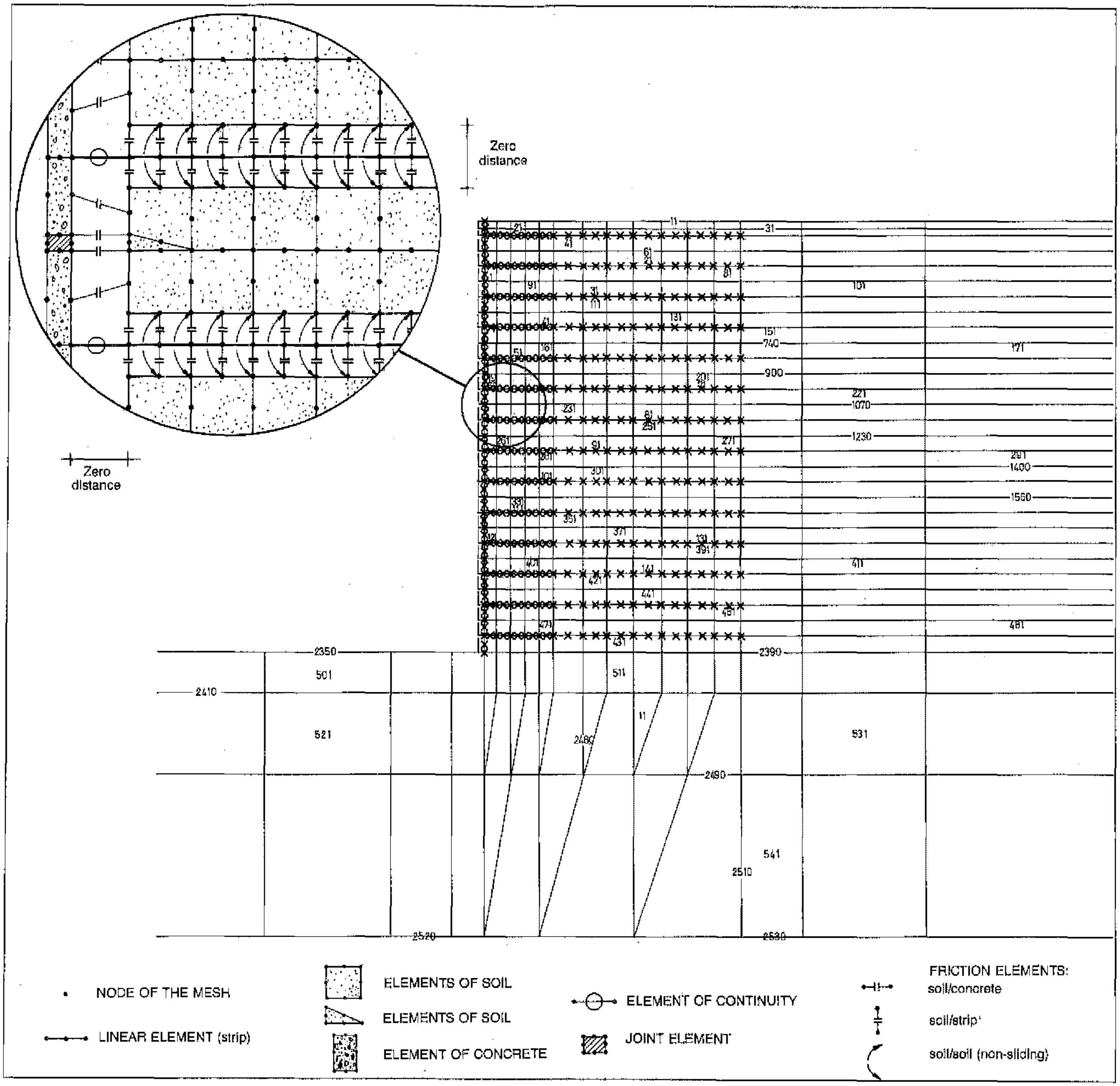


Figure 19: Detailed section of one of the finite element models.

Scale models can only provide limited results. Full-scale experiments, on the other hand, are very expensive and require a great deal of time. Thus, only a limited number of structures and loading conditions can be studied. However, computer analyses using the finite element method make it possible to vary many parameters, and to examine stress and deformation created by a wide number of loading conditions at any point in the model.

The Model

From 1982 to 1984 Terre Armée Internationale employed the Rosalie program to conduct Finite Element Method (FEM) studies on elastoplastic models that included friction-separation elements. This method allows a bar element to slide when the force it is transmitting becomes greater than the limiting friction. The system is thus computed by successive iterations until stabilization or failure. Moreover, the model in Figure 19 is practically three-dimensional. In fact, the soil is in contact with the reinforcing strips through the friction-separation elements; at the same time the soil is continuous between the reinforcements and through non-sliding soil-soil friction elements.

Variation of Design Parameters

The reliability of this model was initially tested by verifying that it produced results that were very close to those previously obtained by instrumentation of actual structures. This was followed by a study of 50 different walls, not including works subjected to concentrated loads. The following parameters were varied simultaneously:

- height H (10.5 or 6m),
- section through wall rectangular $0.7 < L/H \leq 0.4$ or trapezoidal,
- uniform surcharge,
- distribution of metallic reinforcing strips,
- type of wall facing, and
- modulus of the foundation soil.

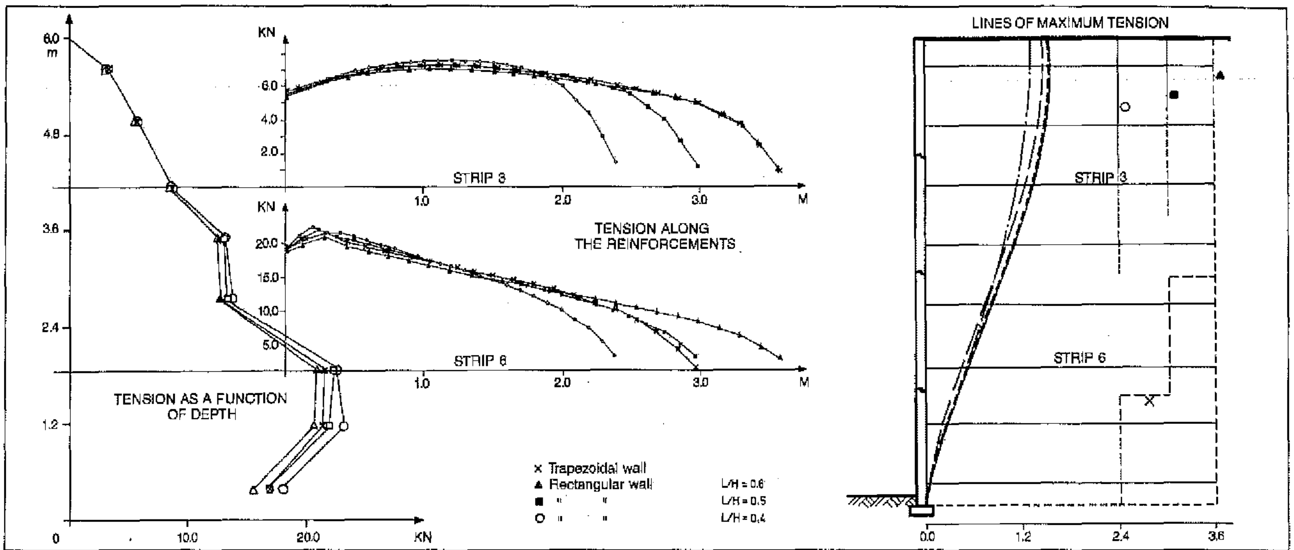


Figure 20: Variations of tensile stress forces.

Results

For each of the models considered, results of the computations were expressed graphically by computer (Fig. 20). Superimposing the curves made it possible to analyze the influence of the length of reinforcing strips on: variations in tensile stress along the strips; increases in maximum tensile stress with depth; and the shape of the active zone. These comparisons confirmed that a Reinforced Earth structure with metallic reinforcing strips as short as 0.4H always behaves in the same way, even when the structure has a trapezoidal cross-section.

These test results also made it possible to evaluate the effect of the type of facing on the tensile stresses at the connections. One can also produce an amplified representation (Fig. 21) of the natural deformations of the structure, which are always small with steel reinforcing strips.

Detailed analyses of the stresses on the boundaries of the structure also make it possible to determine variations in the earth pressure beyond that boundary, and its inclination (which is relatively sensitive to the width of the structure—see Fig. 22), as well as the pressure transmitted to the foundation (Fig. 23).

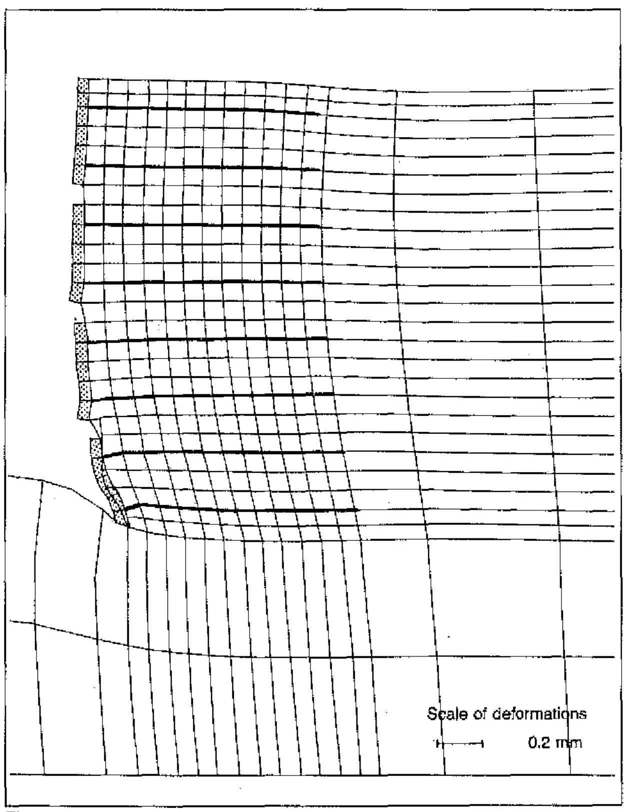


Figure 21: Amplified deformations.

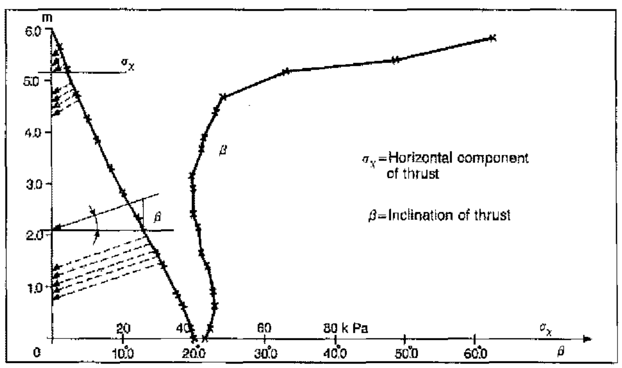


Figure 22: Variation and inclination of earth pressure behind Reinforced Earth.

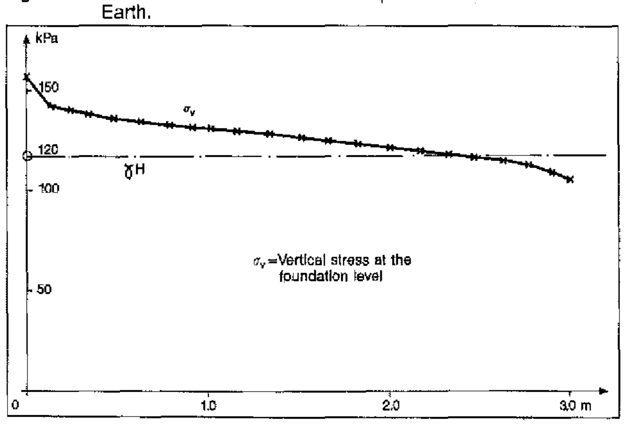


Figure 23: Bearing pressure.

Overall Behavior

The available composite data confirms that a Reinforced Earth structure behaves like a gravity wall. To the weight of the wall and the superimposed loads are added the effects of earth pressure from the retained fill (Fig. 24). Because of the wall's flexibility, this earth pressure corresponds to the active state; as a general rule, the slope of its resultant becomes steeper as the embankment becomes narrower (Fig. 25).

Instrumentation of actual structures and finite element analyses demonstrate that at the base of the structures and at intermediate levels, vertical stress in the embankment is higher closer to the facing and is on average greater than γZ. Use of the Meyerhof formula provides a good estimate of maximum stress in the structure (Fig. 26).

Internal Stress Distribution

Tensile stresses within the reinforcing strips are at a maximum at a certain distance behind the facing. The line joining the points of maximum tensile force separates the active zone, in which the reinforcing strips retain the fill, from the passive zone, in which the friction of the fill retains the reinforcing strips.

All data confirms that when metallic reinforcing strips are used, the line separating the two zones begins at the toe of the structure and follows a nearly vertical path to a point less than 0.3H from the facing at the top of the structure. This is true regardless of the structure's dimensions (up to L/H = 0.4), even for structures with a trapezoidal cross section (Fig. 27).

The shape of the active zone has also been established by kinematic analysis of the mechanism of rupture along a logarithmic spiral.

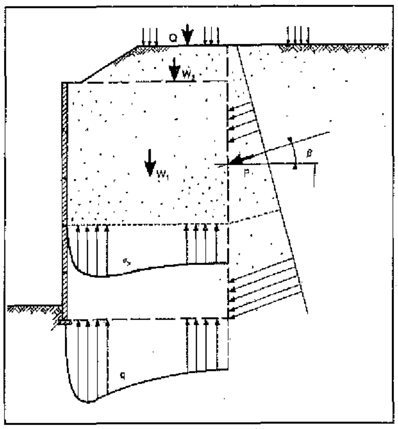


Figure 24: External loads and global equilibrium.

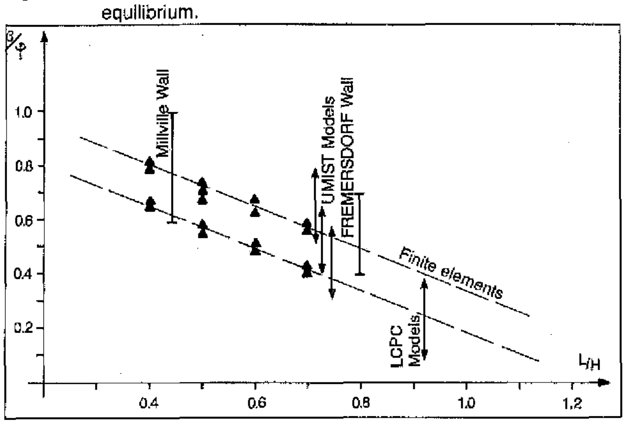


Figure 25: Thrust gradient.

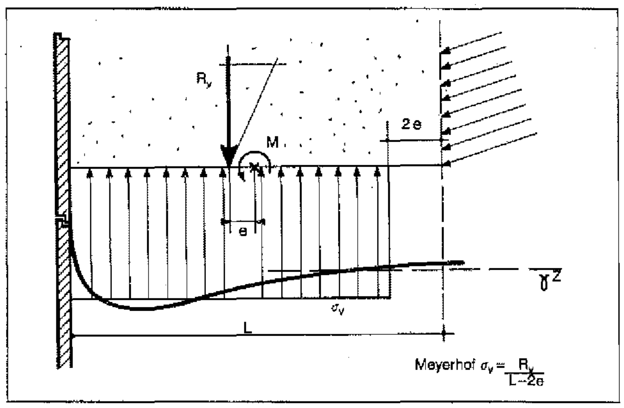


Figure 26: Meyerhof formula.

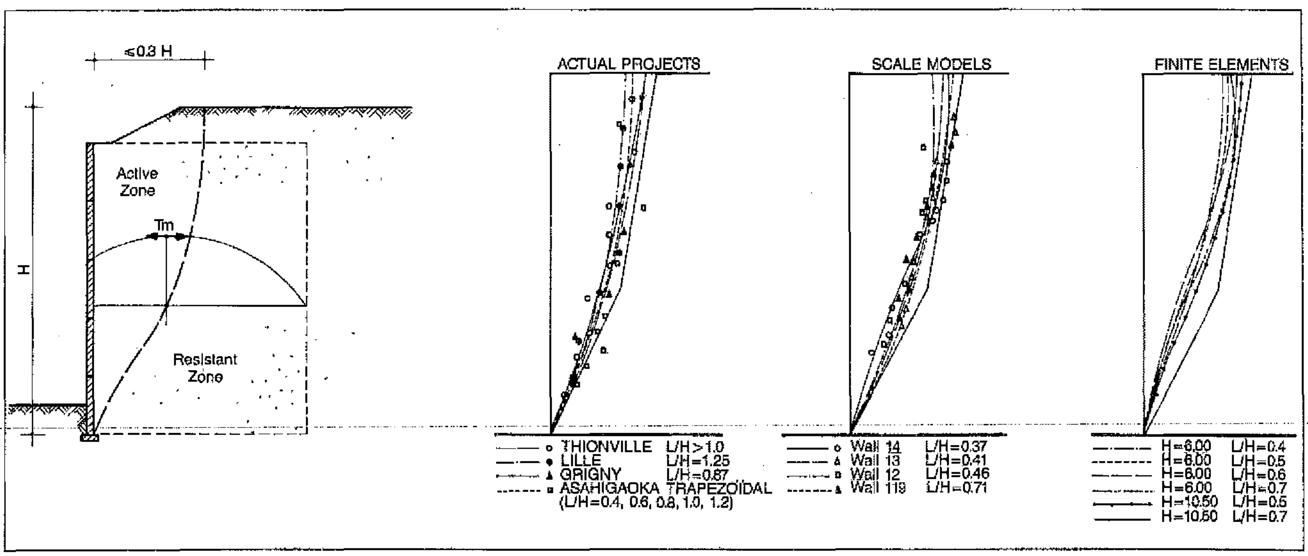


Figure 27: Active zone; measurements taken on actual projects; line obtained using scale model; results of finite element analysis.

Maximum Stress Levels in the Reinforcing Strips

Adopting the hypothesis that the shear force is zero in planes lying midway between reinforcing strips (Fig. 28), the equilibrium of the zone thus established within the active zone implies that N Tmax = σ_h (with N the number of reinforcing strips per unit of surface of the wall facing). Horizontal stress within the structure at the point of maximum tensile stress is a function of the vertical stress at this maximum point, expressed by the equation: $\sigma_h = K \sigma_v$ with σ_v the value calculated by the Meyerhof formula. The experimental results and finite elements study (Fig. 29) show that K is practically equal to the active earth pressure coefficient, K_a.

At the top of walls and to a depth of approximately six meters (Fig. 30), the values of K calculated from measurements of actual structures are significantly higher, largely due to the stresses induced by compaction. They approach a value in the region of K_o, the coefficient of earth pressure at rest.

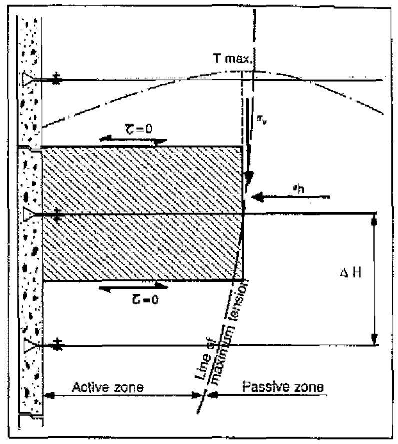


Figure 28: Local equilibrium.

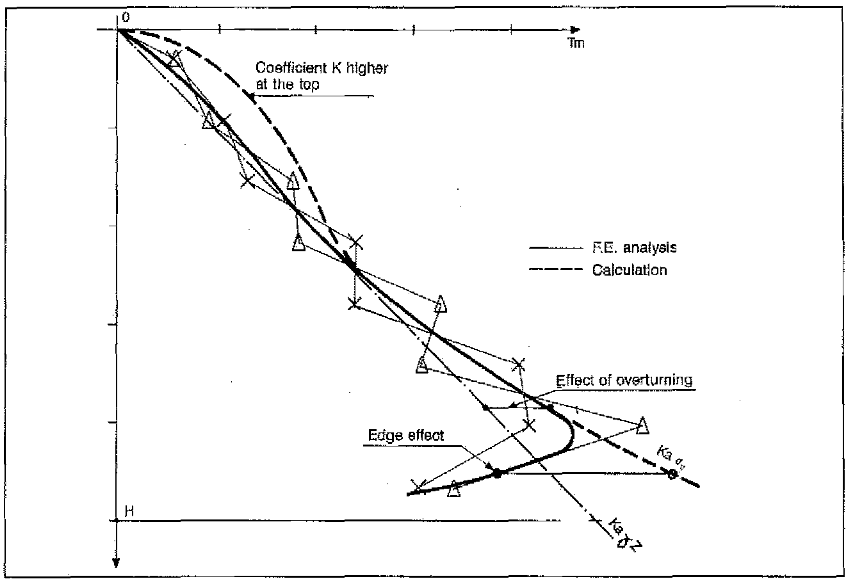


Figure 29: Analyse de $T_m = K\sigma_v / N$

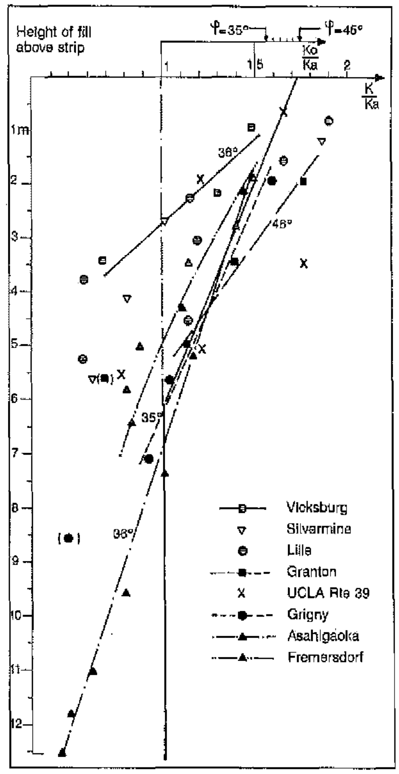


Figure 30: Measured values of coefficient K.

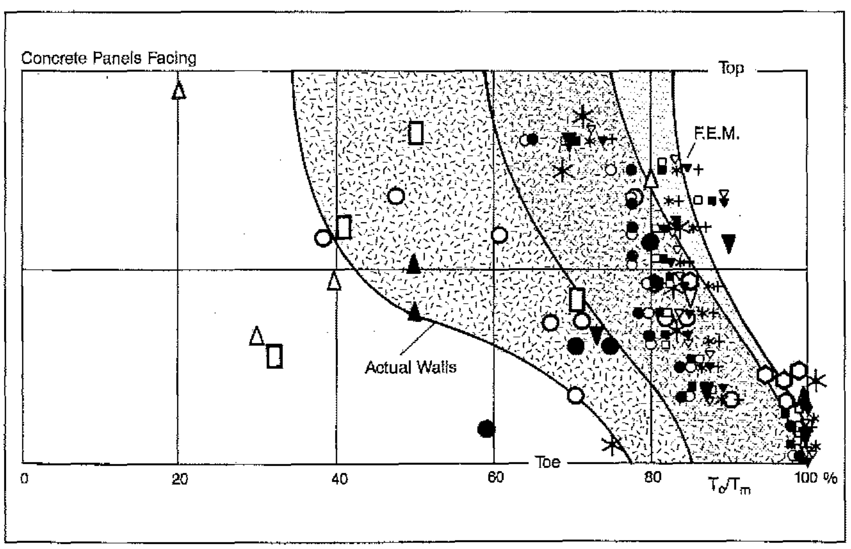


Figure 31: Ratio between tensile loads at connection and maximum tensile forces.

Tensile Stress Levels at Connection

Based on a conservative evaluation of actual measurements and finite element computations, tensile stress levels at the connections to the concrete facing panels may reach 85 percent of maximum tensile stress, and up to 100 percent at the base of the wall (Fig. 31).

With metal facing elements, this stress level never exceeds 75 percent of the maximum tensile force in the strip.

Geometry of Structures

A synthesis of the results previously summarized makes it possible to define practical design methods for Reinforced Earth retaining walls.

These methods are applicable to structures of rectangular cross-section with a maximum height-to-width ratio of 2(L/H $\geq$ 0.5). They can be adapted to "trapezoidal" structures, narrower at the base (H/3 and 2H/3 wide at the base and the top, respectively) provided the dimensions do not affect the overall stability of the structure (Fig. 32). In these formulae, height H is understood to extend to the top of low embankments supported by the structure. Reinforced Earth structures that support long slopes or heavy surcharges are subject to special design procedures.

Earth Pressure Behind the Structure

Generally, the resultant of earth pressure due to the fill behind the structure is assumed to be inclined at the angle: $\beta = (1.2 - L/H)\varphi_2$, with φ_2 the angle of internal friction of the fill. Earth pressure is computed using the Coulomb coefficient, which has as its horizontal component:

$$K_h = \frac{\cos^2 \varphi_2}{[1 + \sqrt{\sin(\varphi_2 + \beta)\sin\varphi_2/\cos\beta}\,]^2}$$

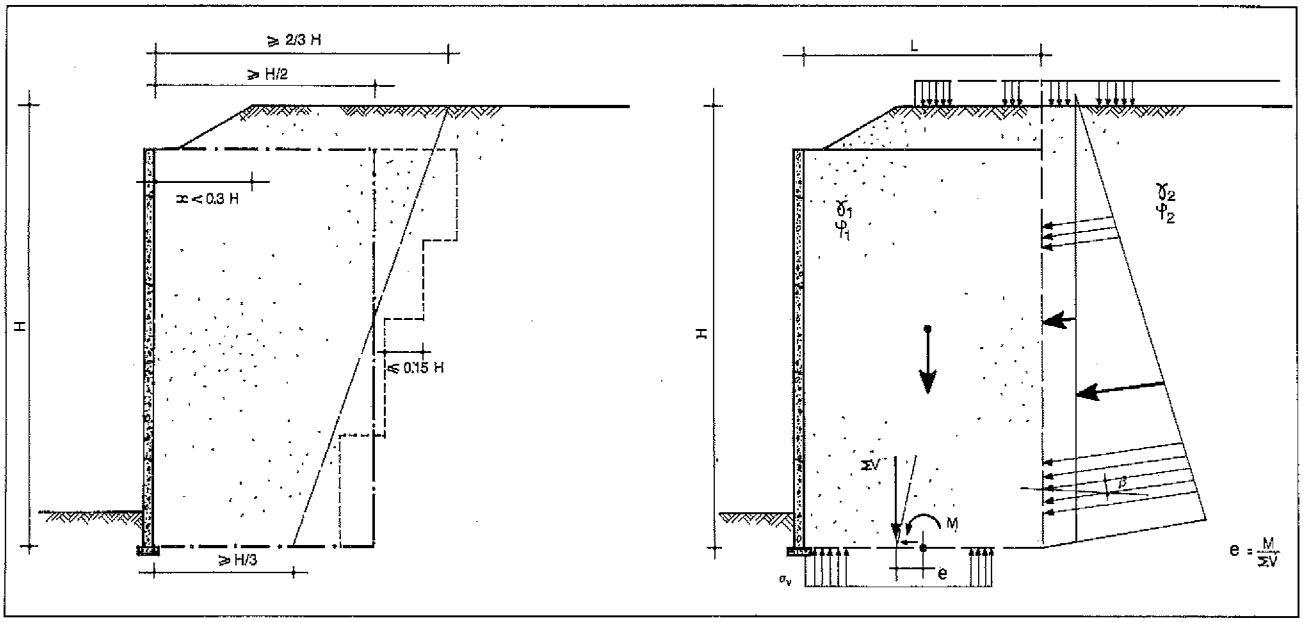


Figure 32: Geometry of typical Reinforced Earth retaining walls; overall equilibrium.

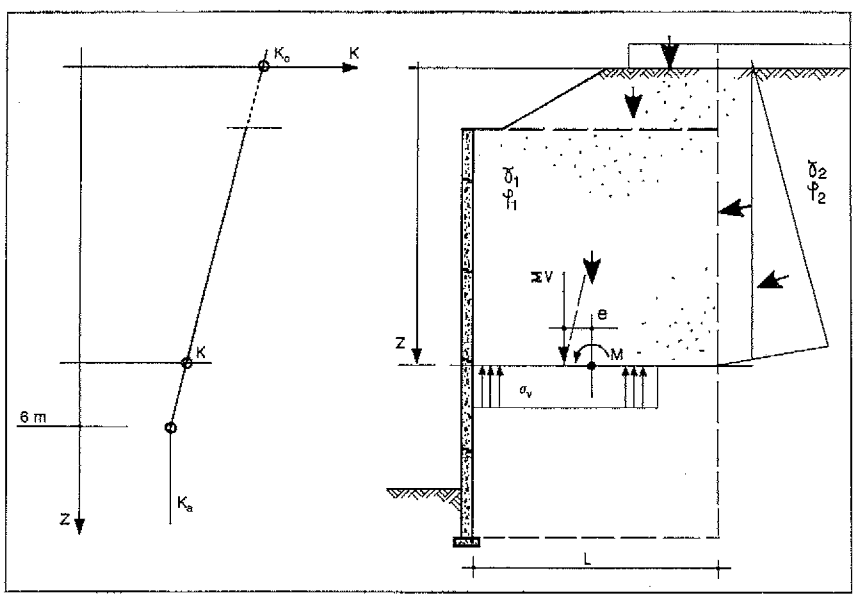


Figure 33: Calculation of σ_v and $\sigma_h = K\sigma_v$.

Stresses Within the Structure

At the foundation level and at any intermediate reinforcement level, vertical stress σ_v is computed from the vertical resultant ΣV of the loads applied to the structure using the Meyerhof formula. With M the moment of those loads with respect to the midpoint of the base,

$$\sigma_v = \frac{\Sigma V}{L - 2M/\Sigma V}$$

The maximum horizontal stress, σ_h, is computed using the relation $\sigma_h = K\sigma_v$, where K varies from $K_o = 1 - \sin\varphi_1$ at the surface to $K_a = \tan^2(\pi/4 - \varphi_1/2)$ below depths of six meters, with φ_1 the angle of internal friction of the Reinforced Earth fill (usually $\geq 36°$). See Figure 33 for resultant loads of stresses.

Tensile Stresses in the Reinforcing Strips

The maximum tensile stress in the reinforcing strips is distributed at the given reinforcement level into the number of reinforcements per unit of facing surface. This is equal to $T\ max = \sigma_h/N$. The line joining the points of maximum tensile force starts at the toe of the structure, as shown in Figure 34.

At the connection point between the reinforcing strip and the facing, tensile force, T_o, varies (depending on the level in the structure) between 85 to 100 percent of T_m with concrete facing, and equals 75 percent of T_m when metallic facing is used.

The adequacy of the cross-section of reinforcing strips shown in Figure 35 is checked at the point of maximum tensile force and at the connection (net section) as a function of the permissible stress, applicable codes, and any sacrificial thickness applied for service life design.

Adherence (Soil-Strip Interaction)

The assumed length L, the width b and the number N of reinforcing strips are then checked with respect to the minimum available frictional capacity in the passive zone. Research on friction shows (Fig. 36) that in backfills normally used in Reinforced Earth construction—correctly compacted and unsaturated—the coefficient of apparent friction (f*) between earth and high-adherence reinforcements, decreases as a result of dilatancy from a maximum value of $f_o^* = 1.5$ at the surface, to $\tan \varphi_1$ at a depth of six meters and beyond. For smooth reinforcing strips, $f^* = 0.4$ is used throughout.

The design consists of satisfying the following equation:

$$T_m \leq T_f = 2b\ f^*\ (L-D)\ \gamma_1\ Z / \gamma_f$$

where γ_f is a safety factor, L-D the length of the reinforcing strip in the stable zone, and $\gamma_1 Z$ the weight of the fill above the reinforcing strip; that is, a value slightly but conservatively smaller than the actual vertical stress, σ_v.

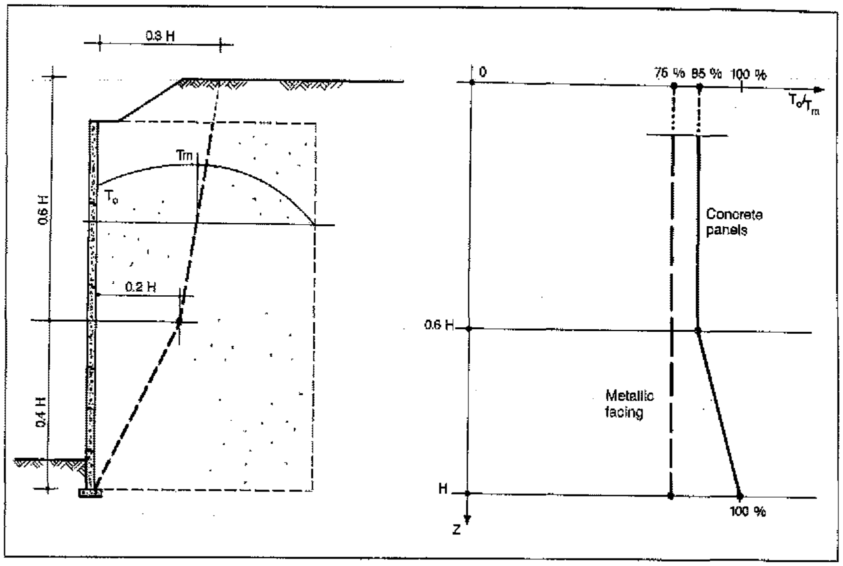


Figure 34: Line of maximum tensile forces; tensile loads at connection.

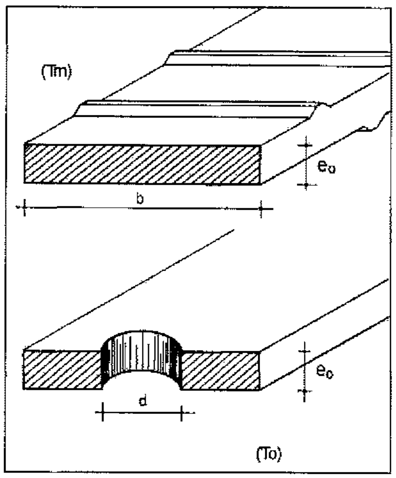


Figure 35: Strip cross-sections.

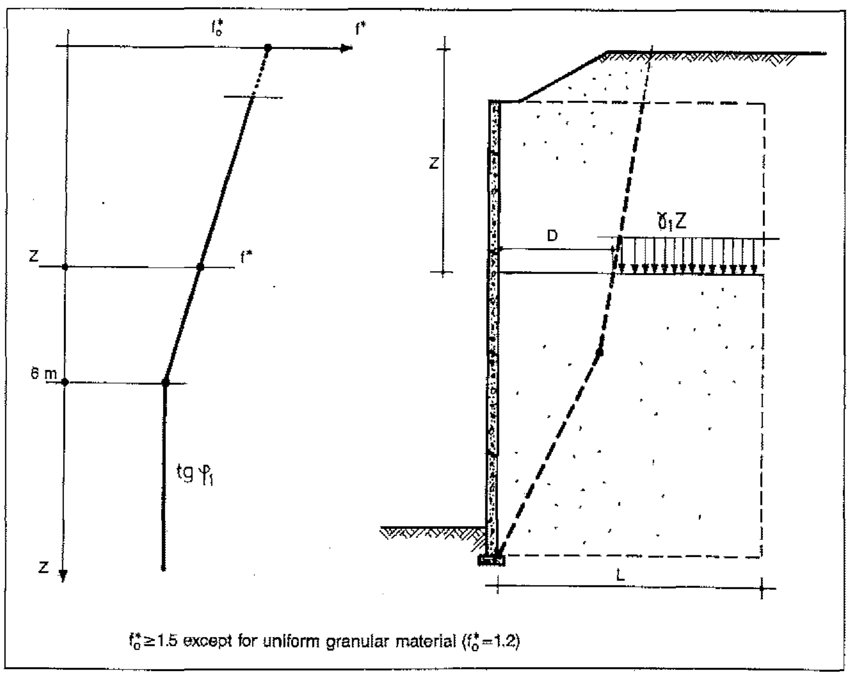


Figure 36: Adherence capacity.

Sliding Beneath The Base

The internal design of a Reinforced Earth mass does not predetermine the general stability of the structure, the soil it supports, or the soil on which it is founded.

As in the engineering of any soil-supported retaining wall, with Reinforced Earth one must first determine that there is no risk of horizontal sliding under the base of the structure (Fig. 37). In the typical case of a structure supporting a horizontal backfill, this criterion rarely governs. When it does, because of the characteristics of the foundation soil or the backfill, it may require a widening of the structure; that is, a lengthening of the reinforcing strips.

Slope Loads

The criterion for horizontal sliding is often more critical when there is a heavy surcharge behind the structure; for example, when supporting a high, sloping embankment (Fig. 38a). The minimum proportions of the structure depend primarily on the magnitude and inclination of earth pressure behind the wall.

Superimposed Structures

A special case of this problem is encountered when one Reinforced Earth structure is built on top of another Reinforced Earth-supported embankment (Fig. 38b). Such cases are not rare. They are possible because Reinforced Earth structures can be built on both embankments and soft natural ground.

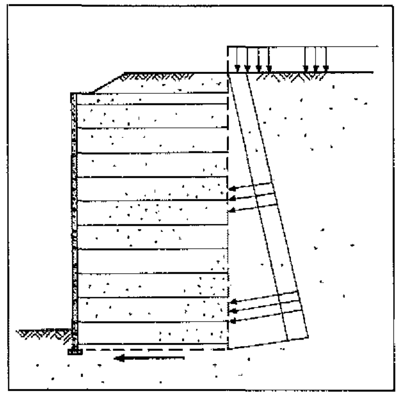


Figure 37: Resistance to horizontal sliding.

In these cases, the two Reinforced Earth embankments are two independent structures, each requiring its own internal design.

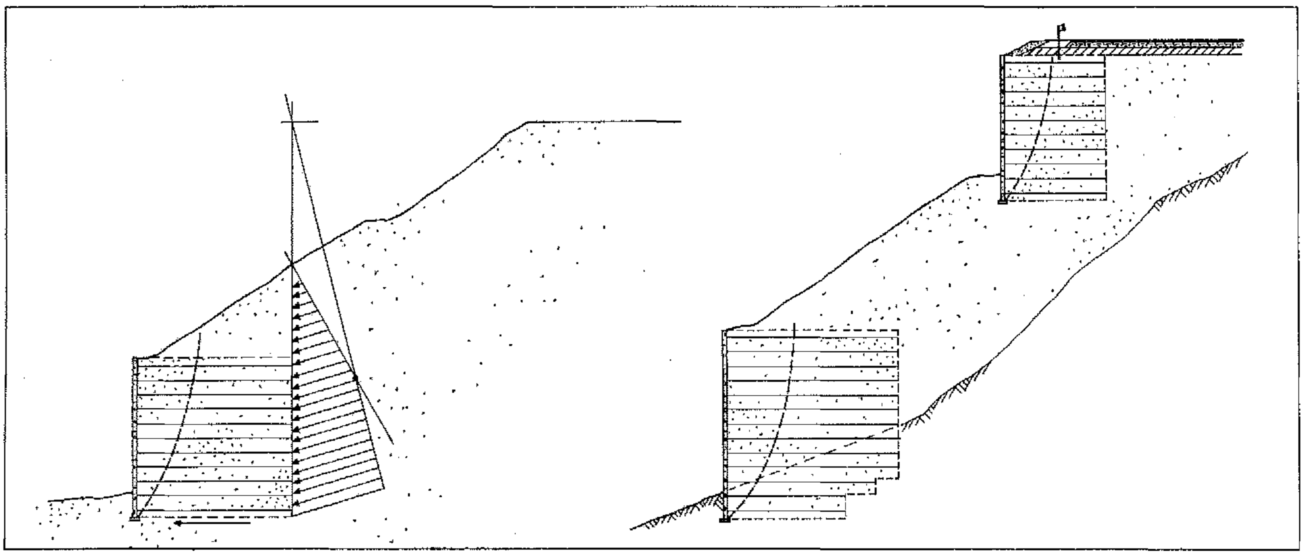


Figure 38a and b: Reinforced Earth walls supporting sloping embankments or superimposed Reinforced Earth structures.

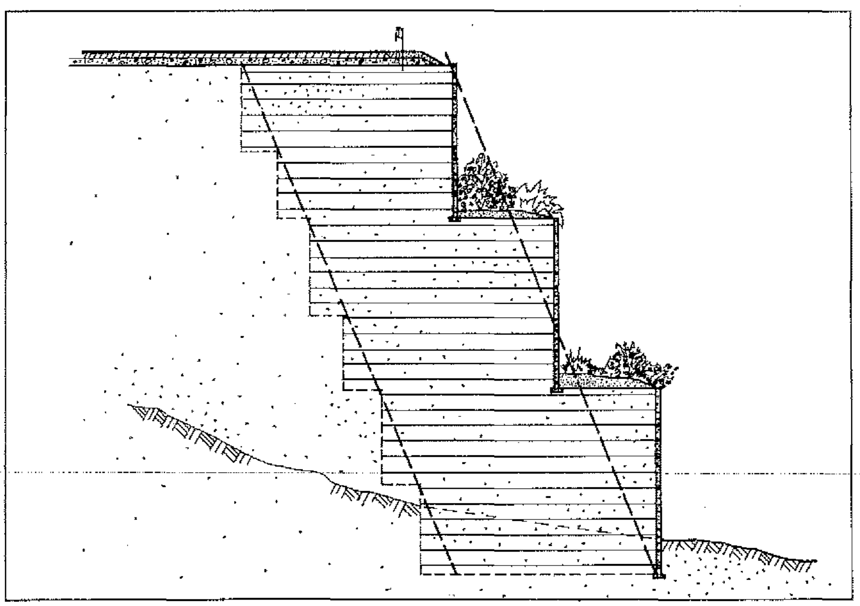


Figure 39: Terraced, tiered structures.

Terraced Structures

This is no longer true when the Reinforced Earth structures are directly superimposed, one on another, which is done when a very high wall is divided into a number of terraces (Fig. 39).

These offset structures are obviously similar to a single embankment with a sloping face. In fact, they exhibit essentially the same overall behavior, and are designed as sloping faced Reinforced Earth walls. Inclined retaining walls have been used extensively in the mining industry for construction of large ore storage structures and have been the subject of considerable, specific research.

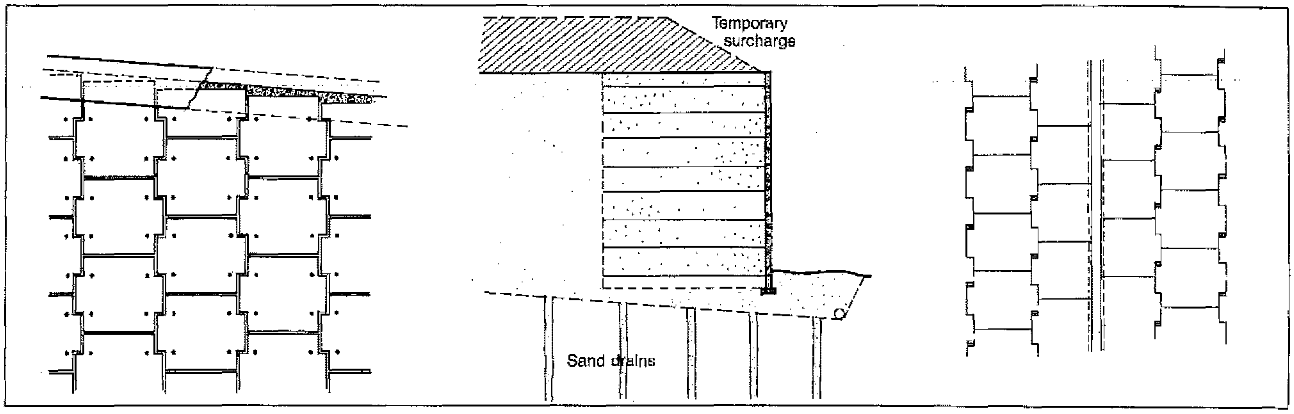


Figure 40a, b, and c: Adaptation to settlements: coping; preloading; vertical joints.

External stability design also requires a determination that the bearing capacity of the foundation is capable of supporting the load superimposed by the structure, and requires a calculation of the total anticipated differential settlement.

Settlements

Depending on the amount of settlement and the time required for it to occur, there are several methods of adapting a Reinforced Earth retaining wall to the site conditions or of accelerating the consolidation of the site, should that be required.

Rapid settlements can generally be accommodated during the construction process by final adjustments to the dimensions of the top course of facing panels, as in Figure 40a.

Slower settlements, if large in magnitude, must often be accelerated to allow for corrective measures during construction. Methods for accomplishing this include temporary surcharging of the structure foundation, installing vertical drains, and other traditional methods (Fig. 40b).

When differential settlement along the facing is expected to exceed the one-to-two percent which can be accepted by the facing system without risk of damage, the wall face can be given additional "degrees of freedom" by installing vertical slip joints (Fig. 40c).

Soil Improvement

On soils of poor quality, both settlement and bearing capacity problems may be solved by improving the foundation, by partial replacement, preloading, or even use of stone columns or a lightweight backfill (Fig. 41a and b).

Adaptation of the Reinforced Earth Structure

On a marginal foundation soil, it is possible to use the flexibility of Reinforced Earth to build retaining walls without resorting to special procedures.

This may be done:

1) By building the project at a rate which will allow the soil to consolidate, including building the project in stages if necessary;

2) or, in some cases, by lengthening the lower levels of reinforcing strips to widen the foundation of the structure, thus reducing the foundation pressure and providing assurance against a deep bearing capacity failure (Fig. 41c).

For all Reinforced Earth retaining walls built on problem soils, the best approach is often a combination of these solutions. However, by attempting first to take advantage of the ability of Reinforced Earth structures to accept deformation, it may be possible to reduce the requirements for some of the more expensive soil-improvement techniques.

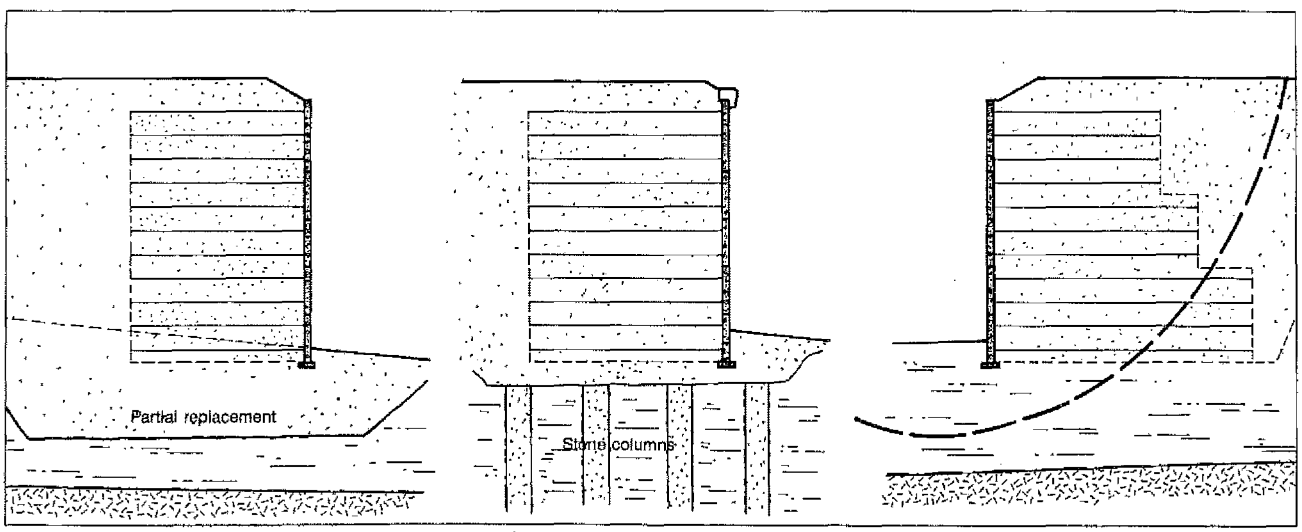


Figure 41a, b, and c: Soil improvement ; widening of foundation.

Research is being conducted on Reinforced Earth walls of narrow or trapezoidal cross-section with the aim of developing reliable design procedures for structures suitable to sites where excavation must be limited.

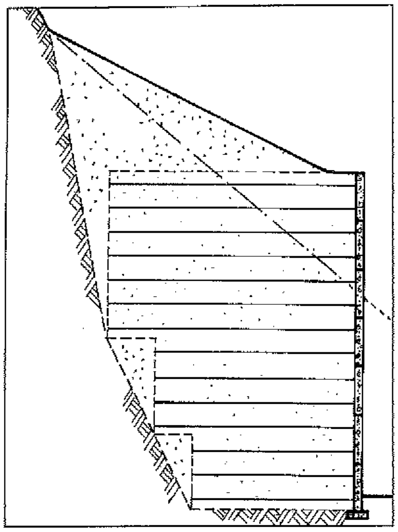


Figure 42: Reinforced Earth structure in a cut.

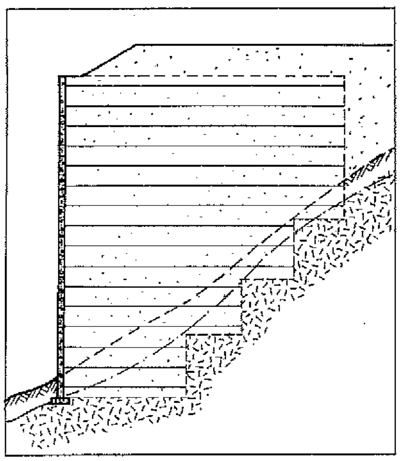


Figure 43: Reinforced Earth structure on a rock slope.

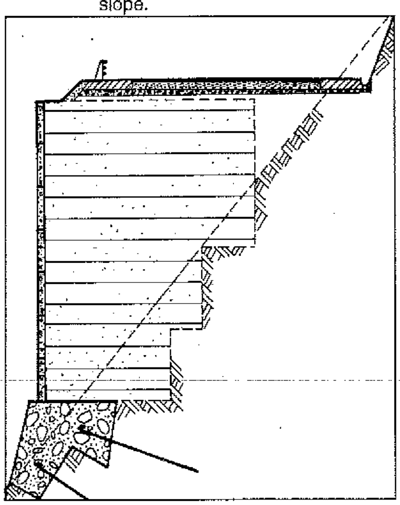


Figure 44: Cross-section showing special concrete foundation.

Excavation

One such case (Fig. 42) is a project constructed in a cut and on a very stable or rocky natural ground, where it would be paradoxical to replace too much of the existing terrain with fill, even if it was reinforced.

Walls On Rock Slopes

For embankment projects built on rocky slopes, the length of reinforcements at the base of the wall is reduced (Fig. 43).

When a slope is very steep, it may be economically and technically advantageous to support the base of the Reinforced Earth structure with a concrete substructure tied into the rock, as in Figure 44.

Unstable Slopes

The most difficult cases are those where the slope consists of relatively unstable, or marginally stable materials, such as thick debris or talus. Dimensions of the Reinforced Earth embankment must then provide a compromise between the stability of the slope above the excavation during construction and the stability of the ground below, once the latter is supporting the load of the structure (Fig. 45).

The first condition requires that the reinforcing strips be shortened, whereas the second calls for lengthening them to tie the overload to the existing ground more securely and to deepen any potential rupture lines. (The solution may also call for a temporary shoring of the excavation, and construction proceeding in small and progressive sections.) Such an optimization process requires careful geotechnical studies as well as stability calculations that take into consideration the resistance of any reinforcing strips intersected by the lines of possible major faults or slides.

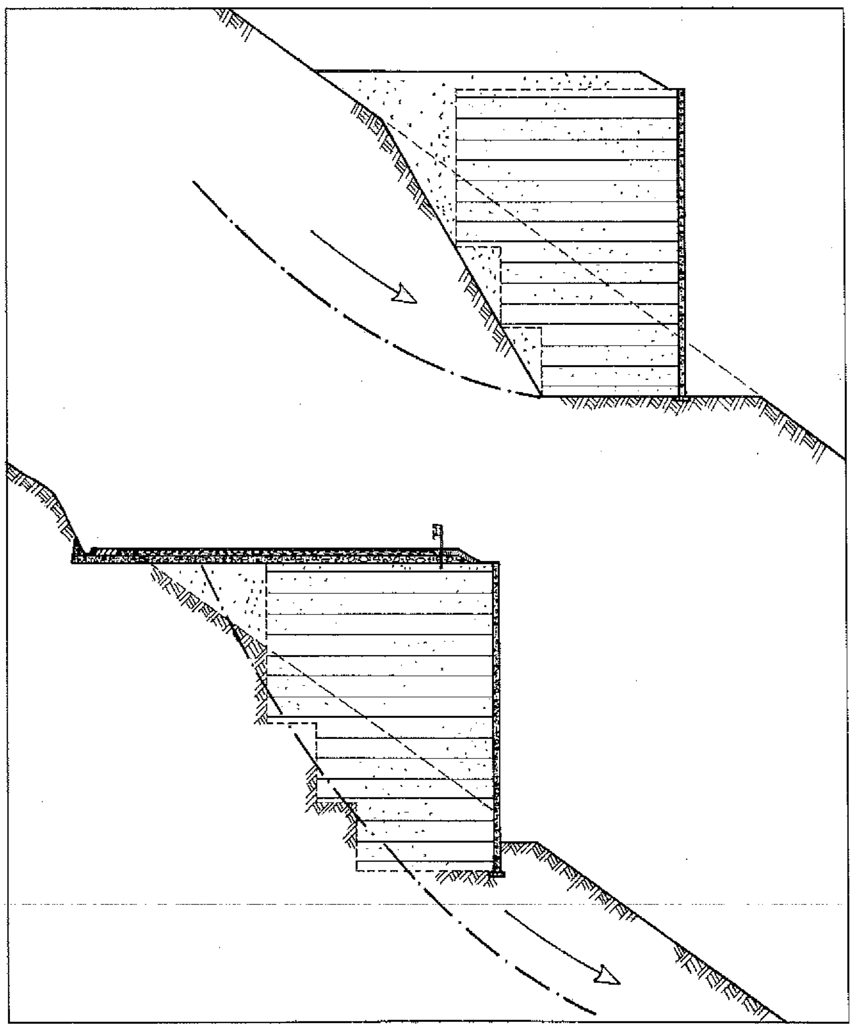


Figure 45: Unstable slope; stability of upstream excavation and of downstream slope.

Autoroute 40

A still more difficult case was presented near Nantua, France, on a section of the highway linking Lyon, France with Geneva, Switzerland. The talus slope there was in such precarious balance that it was practically impossible to excavate it, even to modest depths. For the same reason, the slope could not support heavy fill surcharges. Therefore, the highway was designed with two vertically offset roadways to follow as closely as possible the natural contours of the terrain. The wall now separating the two grades of the roadways was built approximately half cut and half fill.

The lower section is in the form of a tieback-anchored wall built downward from the surface. The upper part is a Reinforced Earth structure, constructed almost without excavating into the existing terrain. The anchors provided stability first for the excavated talus slope, and then for the slope supporting the fill surcharge and the Reinforced Earth embankment.

The downhill roadbed is itself supported by a Reinforced Earth structure that was also set close to the slope line when necessary, after the slope had been stabilized by one or two levels of tiebacks. In other sections, where stability was not as critical, the lower wall of Reinforced Earth was terraced to blend with the site.

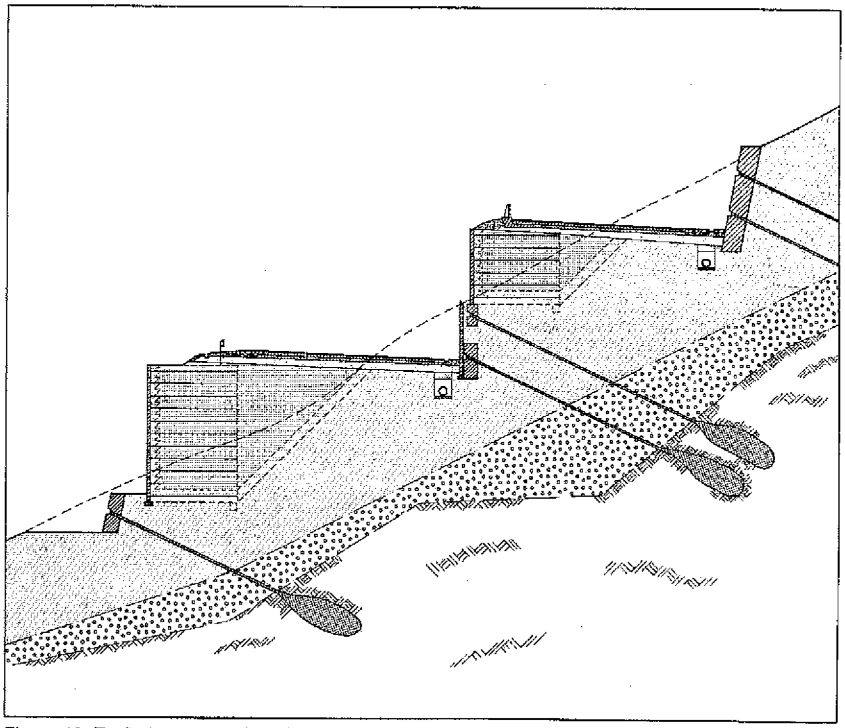
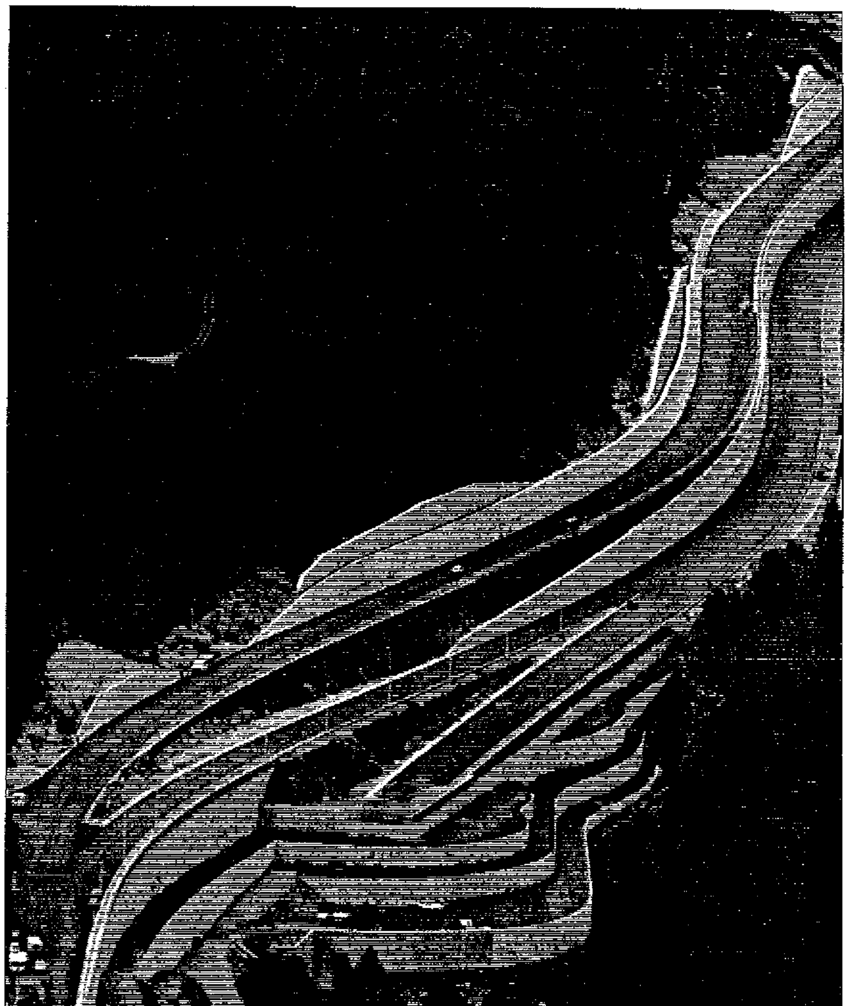


Figure 46: Typical cross-section highway A40, France.


A40 near Nantua, France.

Conclusion

Because of its technical superiority and economic advantages, Reinforced Earth has become widely used in the construction of earth-retaining structures.

Presently, more than 7,000 retaining structures are in place throughout the world. The international Reinforced Earth Group of companies continues to expand the state-of-the-art and to adapt this technology to an increasing range of problems, sites, and requirements.

Licensed under the patents issued to Henri Vidal throughout the world, the Reinforced Earth Group of companies operates in 34 countries on six continents. Although part of the same Group, each company is independently managed by nationals of that country who are professional engineers that understand local conditions, codes of practice, and construction capabilities and techniques.

All Reinforced Earth companies are fully staffed with experienced engineers and project managers and provide a complete range of services:

- Engineering, from conceptual designs to finished construction plans and shop drawings.
- Materials specification, production and delivery.
- Specification and production of customized components, such as traffic safety barriers, seawall wave deflectors, and architectural finishes.
- Detailed cost estimates.
- On-site construction assistance and advice.
- Fixed-price contracts covering both services and materials.

Research and other technological activities among the different companies are coordinated from Paris by Terre Armée Internationale. For new applications and special or unusual projects, Terre Armée Internationale can pool the resources of several companies to create optimum project designs and material specifications. It also acts as the central technical service organization, and maintains the primary information data base collected from new applications, special projects and research.

Terre Armée Internationale takes the lead in organizing and developing research projects, both under its own direction and through coordination among the other companies. Analysis and synthesis of research and technical data by Terre Armée Internationale result in technical recommendations and design improvements published in routine reports disseminated to all the Reinforced Earth companies.

The dynamics of this organization allow each Reinforced Earth company to offer government agencies, owners, consultants and contractors the understanding and flexibility of a local business, combined with the vast resources and technological advantages of a global concern.

Worldwide, the Reinforced Earth Group has a staff of some 500 professional and administrative employees. More than 10,000 Reinforced Earth structures have been completed in 56 countries.

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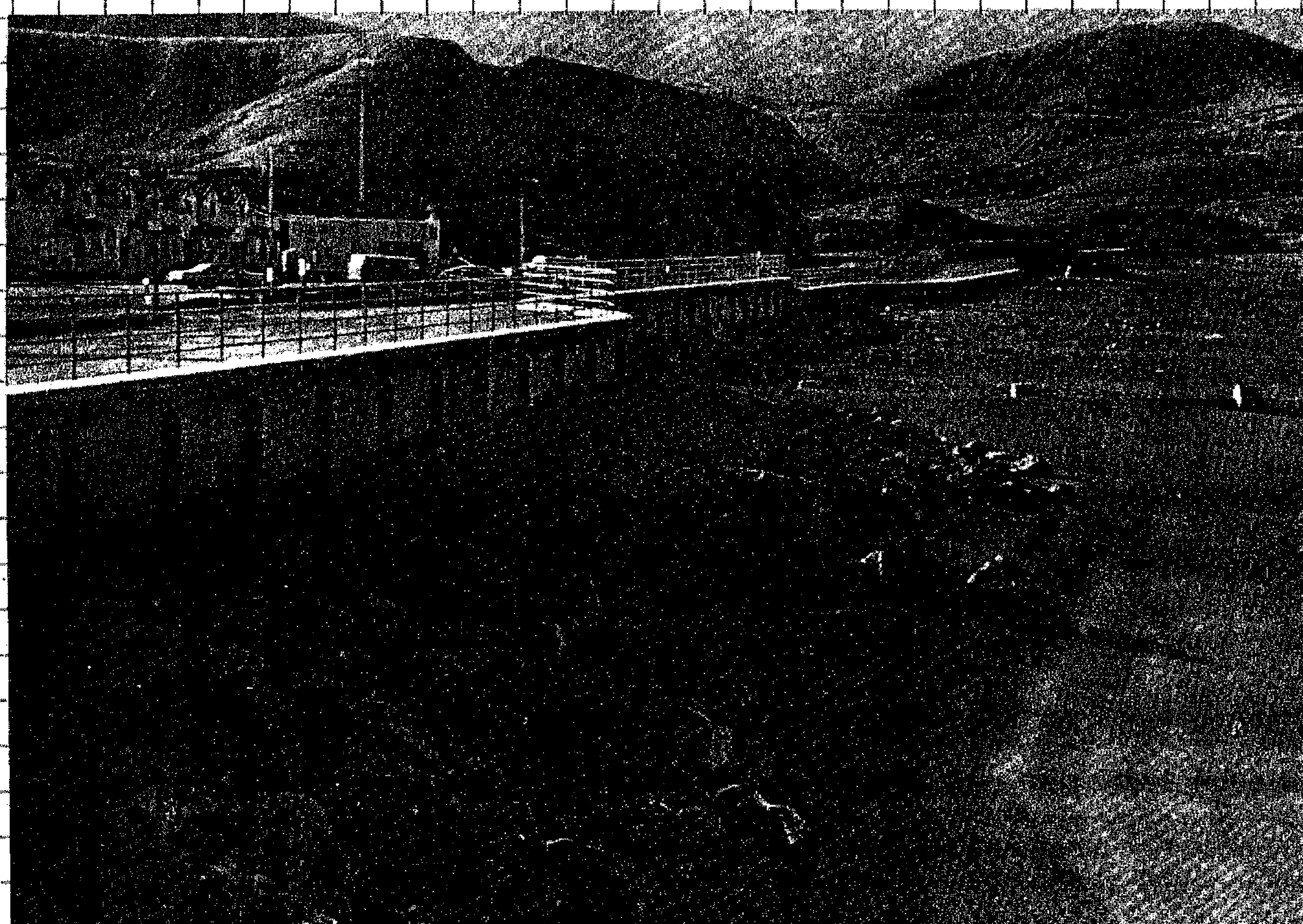
STRESSWALL

RETAINING WALL SYSTEMS



SEAWALL   CITY OF PACIFICA

Stresswall International, Inc.
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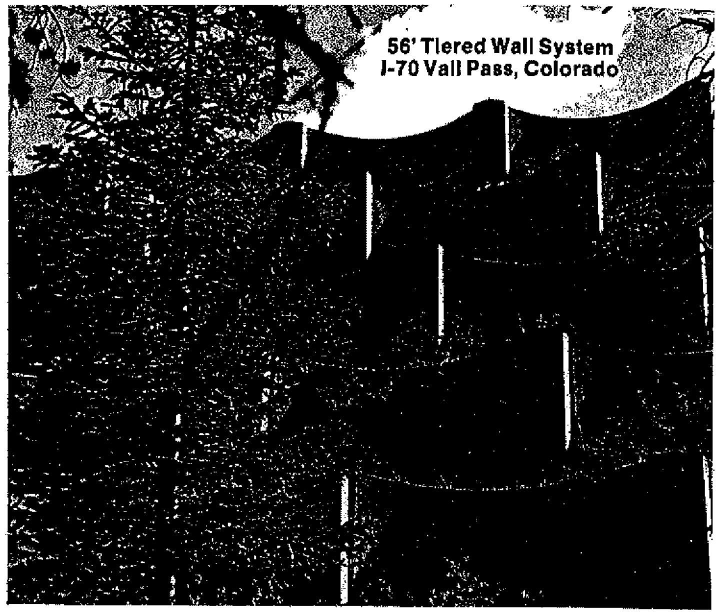
Tel: (303) 221-3894

Stresswall Canada Ltd.
3522 West 41st Avenue
Vancouver, B.C. V6N 3E6
Tel: (604) 263-2696

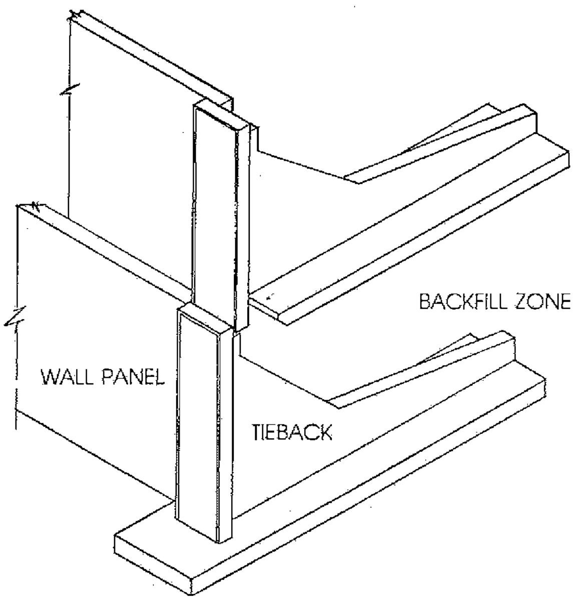
Through the application of advanced soil mechanics, Stresswall has developed an innovative design approach to earth retention structures. This design enables the Stresswall system to exhibit cost effectiveness through all stages of procurement, installation and maintenance.

This versatile product can be produced in various configurations resulting in attractive as well as practical solutions to your earth stabilization requirements.

Stresswall was initially developed by Morrison Knudsen Engineers in response to the Colorado Department of Highways needs in the mid 1970's. Through their licence agreement with Stresswall International, MKE continues to fully support the newly developed Stresswall systems with their engineering expertise.



56' Tiered Wall System
I-70 Vail Pass, Colorado

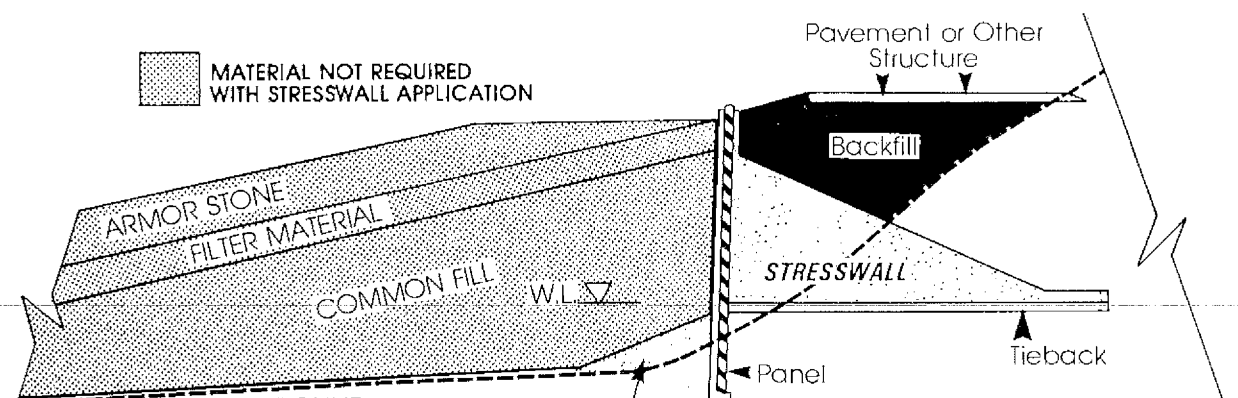


Stresswall is comprised of two precast concrete elements:

- **L-shaped tie-backs are placed perpendicular to the structure's face and extend into the backfill zone,**
- **Wall panel elements are placed between the tie-backs forming a completed wall configuration.**

Wall heights in excess of 70 feet are possible in either tiered or vertical configurations (Stresswall #1 and #2). The vertical wall system is also available in a "smooth-faced" alternative (Stresswall #3) for use adjacent to traffic lanes on roadways.

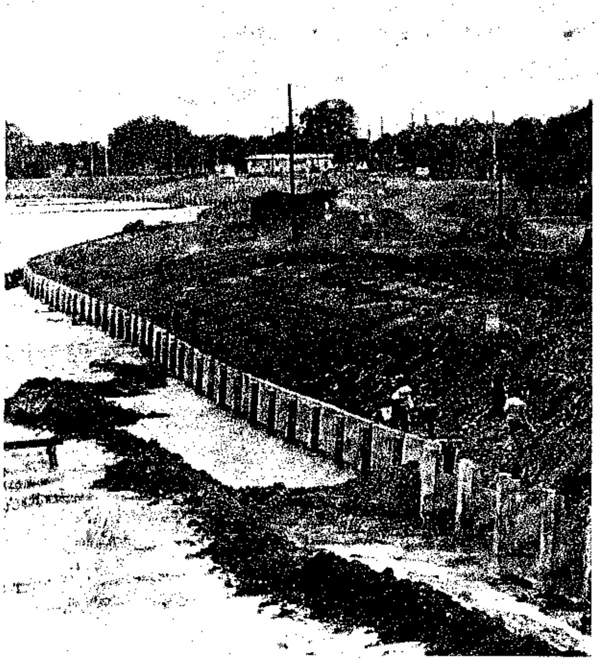
Stresswall's standard section was modified to provide an effective seawall product. The Seawall's modified section: is efficiently installed in difficult site conditions, provides scour protection, and prevents the development of overturning forces due to soil pressure. Stresswall's precast concrete elements, as well as being highly resistant to corrosion and wear, are very cost effective when compared to other shore protection alternatives.

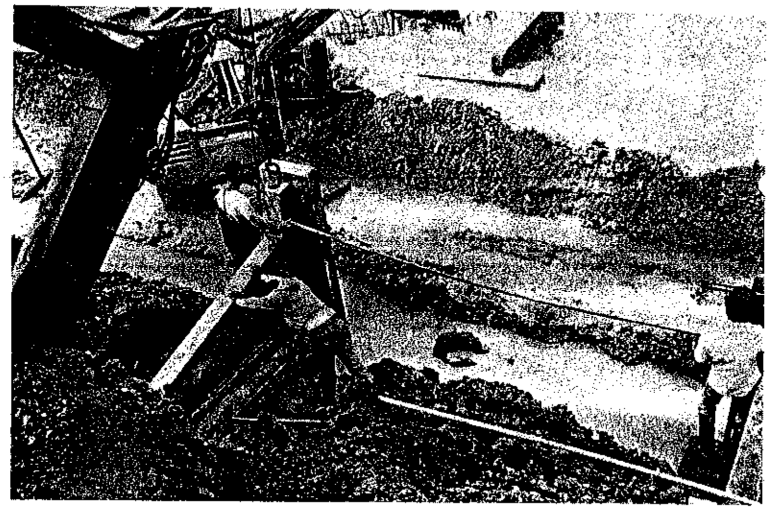


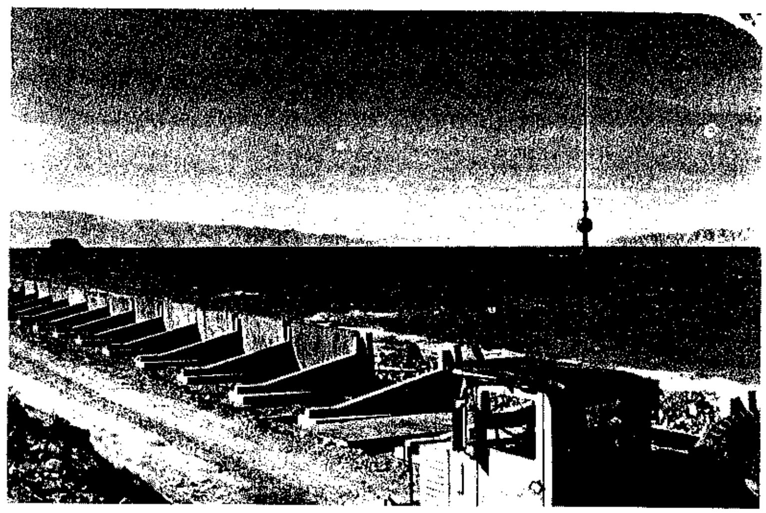
SEAWALL SECTION nts

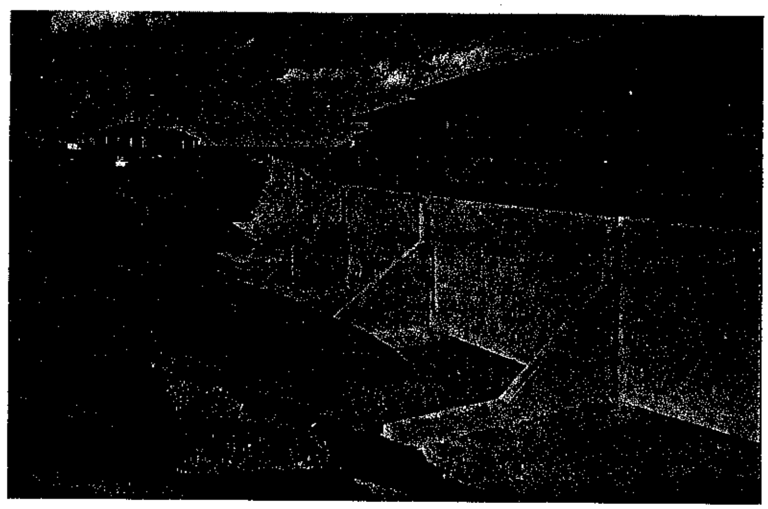
ADVANTAGES

- No mechanical fasteners are required eliminating problems with corrosion.
- Fast erection time because of the lack of fastening devices and size of components. (120 sf plus per wall panel.)
- The amount of excavation required to get the wall in place is less than with most systems.
- Backfill can be placed and compacted with normal heavy equipment. Backfill compaction requirements are significantly less than with other systems. Hand-placment and compaction are minimized or eliminated.
- Precast components can be manufactured locally.
- Can be designed to use local cut material for backfill.
- No requirement for any cast-in-place concrete.
- Walls can be designed with various architectural configurations and surface finishes.
- Large precast components enable walls to be dismantled and re-used in similar applications.
- Precast concrete elements in addition to being inherently durable, can be modified to enhance specific performance requirements.
- Individual elements allow grade and alignment variations (curves, etc.), without special consideration or cost.









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Seawall & Earth Retention Systems By **STRESSWALL**

OWNER: Palacios Seawall Commission
CONTRACTOR: Ercon Inc.
PRECASTER: Manco Inc.
ENGINEER: Jones & Neuse

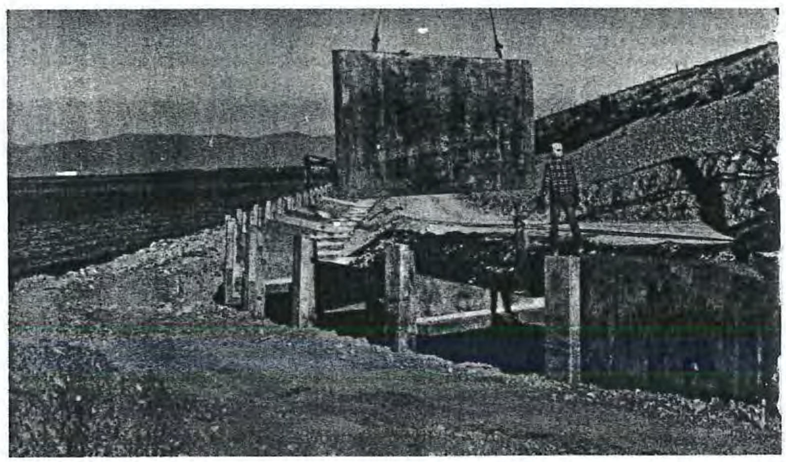
UNIQUE CONDITIONS

- Built in adverse conditions. Saturated environment due to constant wind driven high tides.
- Required the use of innovative geotextile drainage system and cast in drainage slots.
- Existing Clayey/sand embankment material used for wall backfill.

OWNER: Utah Dept. of Transportation
CONTRACTOR: L.A. Young & Sons, Inc.
PRECASTER: Lay's Rock Products

UNIQUE CONDITIONS

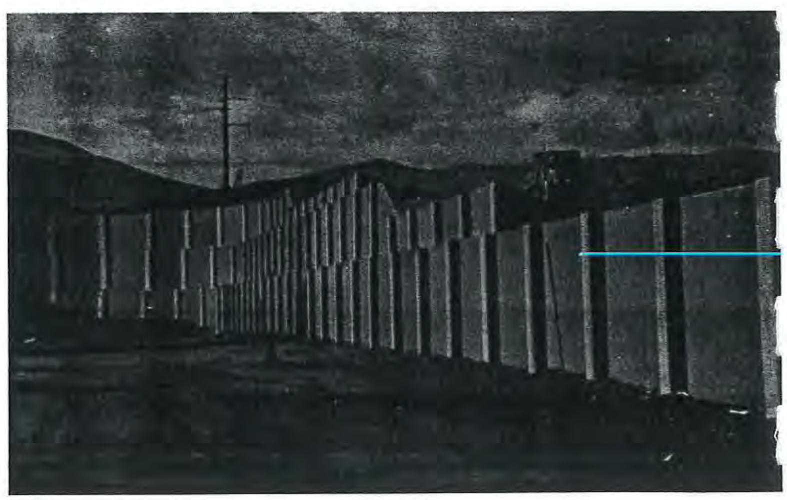
- First precast/prestressed structure used for erosion control on Great Salt Lake.
- Special design considerations required for high salt concentrations.
- Locally available "slag" from Kennecott Copper used for wall backfill.



OWNER: Colorado Division of Highways
CONTRACTOR: Ames Construction
PRECASTER: Amcor — Arvada Colorado

UNIQUE CONDITIONS

- One of first structures used on a privately funded interstate project
- Twenty-eight foot high three tier wall
- Architectural exposed aggregate surface on wall panels



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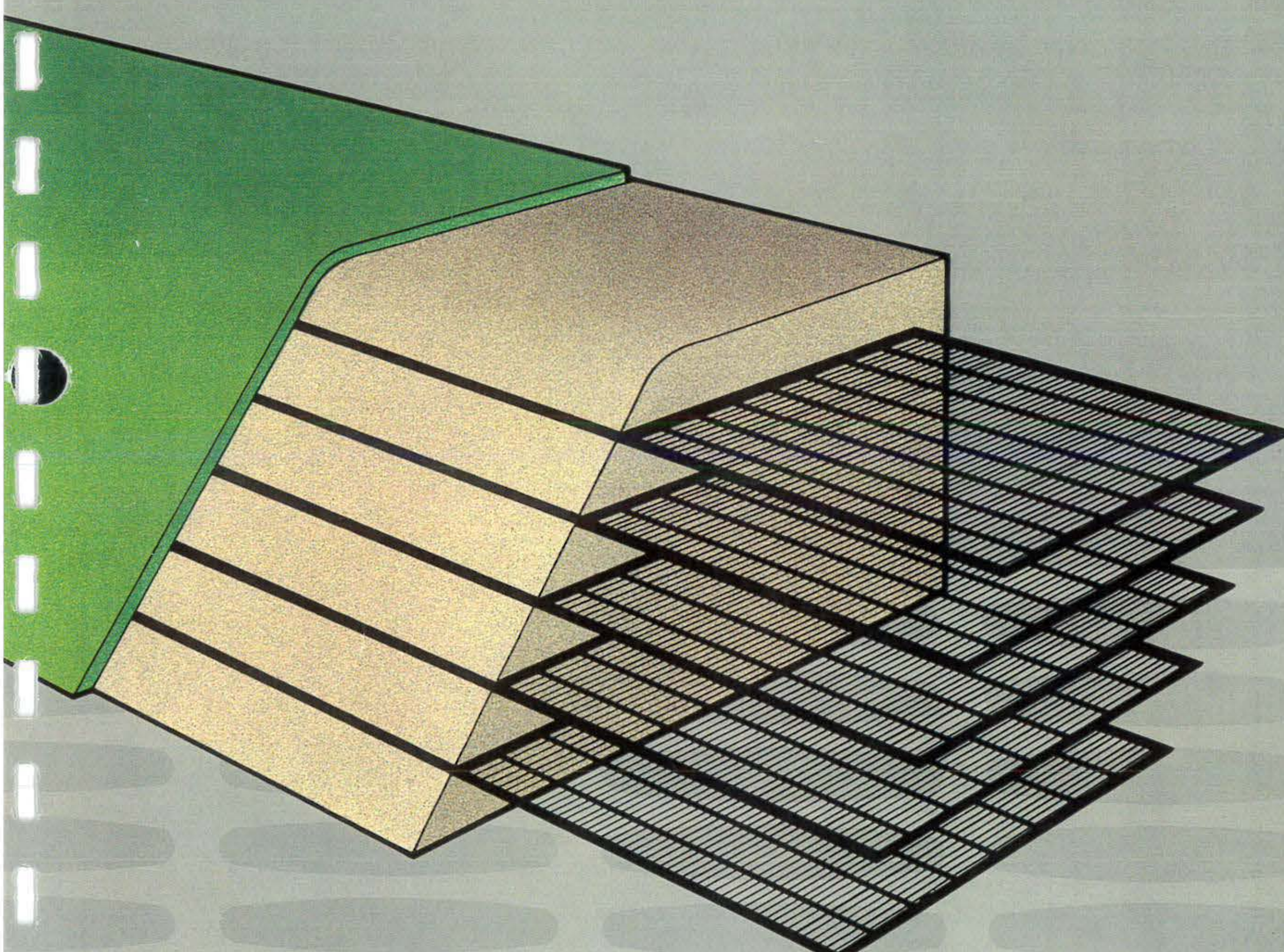
Suite 880
Fort Collins, CO 80521
Tel: (303) 221-3894

Stresswall Canada Ltd.

3522 West 41st Avenue
Vancouver, B.C. V6N 3E6
Tel: (604) 263-2696

TENSAR®

Slope Reinforcement



Stability When You Need It

When you're faced with weak construction soils, such as silts or clays, Tensar Geogrids provide reinforcement which permits use of these poor quality fill soils.

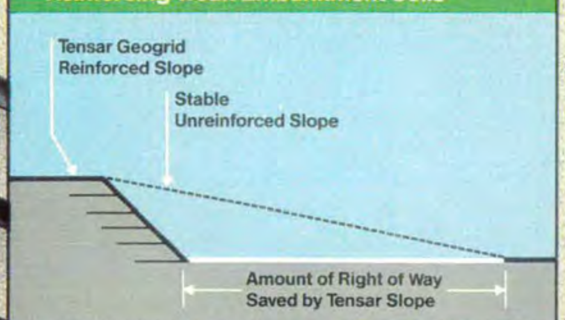
When right-of-way is limited, a reinforced soil slope can be constructed with Tensar Geogrids to fit available space at a substantially lower cost than a retaining wall.

When fill is scarce or expensive, Tensar Geogrids permit steeper slope construction so that fill and transportation expenses are minimized.

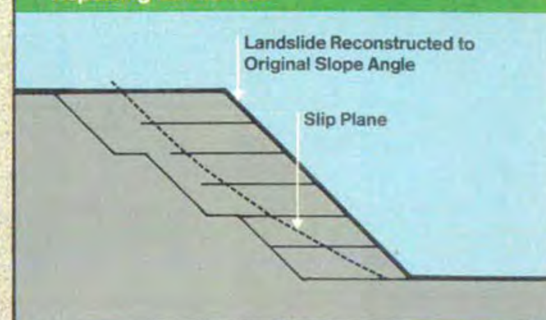
When soils are corrosive, Tensar Geogrids provide reinforcement that will not deteriorate, even in acidic, alkaline, or saltwater environments.

The examples shown here illustrate the most common situations in which reinforced soil slopes are constructed with Tensar Geogrids. Tensar Geogrids have proven their endurance, practicality, and economic advantage in these and many other applications through their use in thousands of structures in North America and around the world.

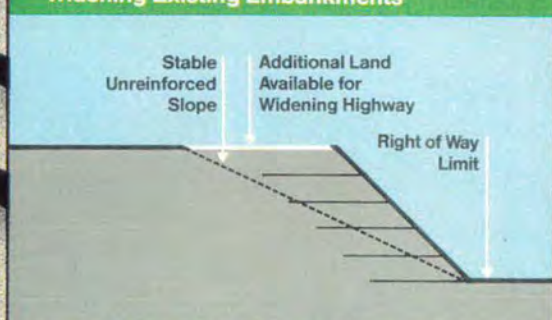
Reinforcing Weak Embankment Soils



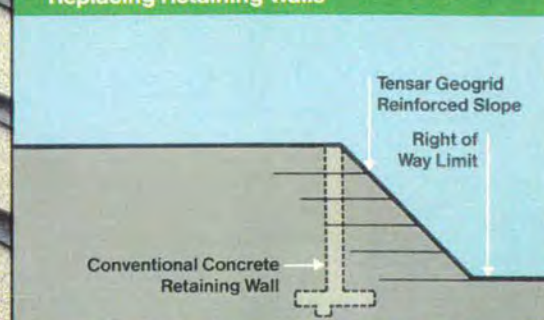
Repairing Landslides



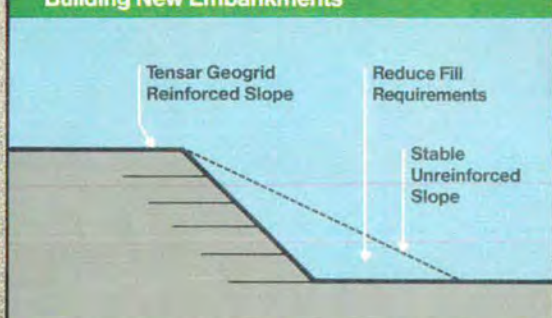
Widening Existing Embankments



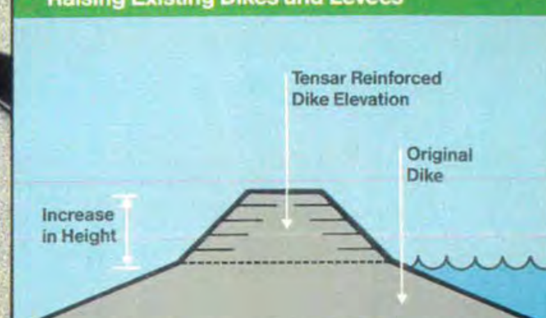
Replacing Retaining Walls

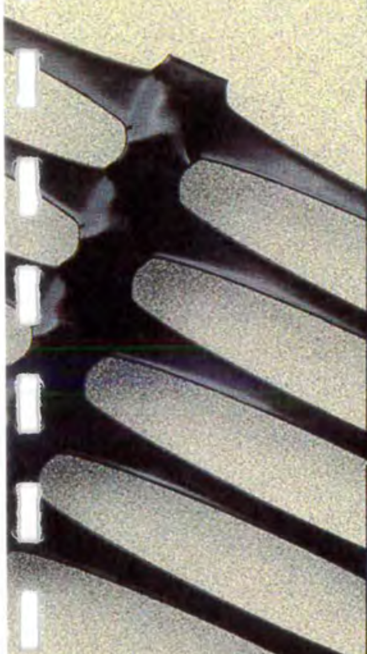


Building New Embankments



Raising Existing Dikes and Levees





Tensar Uniaxial Geogrids are used as primary reinforcement to provide overall stability in all types of reinforced soil slopes.

Stability by Design

An unreinforced slope has a natural angle of repose that is dictated by the shear strength of its soil. Typically, this may range from 20° for clay to 35° for sand and gravel. With a typical factor of safety, unreinforced soil slopes are usually constructed at angles of 18° to 26° from the horizontal.

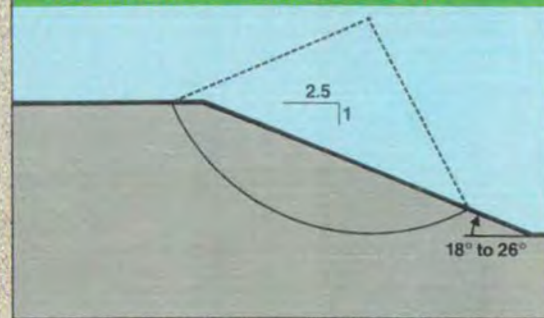
Tensar Geogrids, when embedded in soil, form a reinforced soil composite that has high internal stability. *Tensar Geogrid reinforced slopes can, therefore, be constructed far more steeply than the soil's natural angle of repose.* The result is much more efficient construction and use of land than conventional methods can safely allow.

Design of reinforced soil slopes with Tensar Geogrids involves determining:

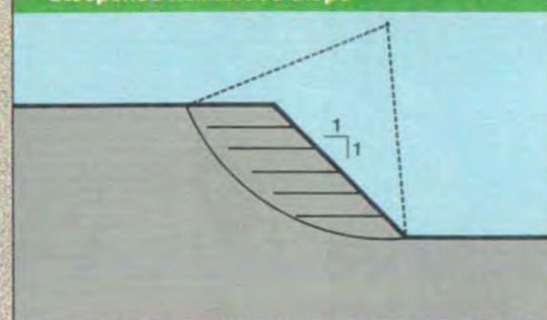
- the desired slope geometry (cross section);
- the forces acting on the soil structure which must be offset through reinforcement to ensure stability;
- the required number and type of Geogrid reinforcement layers;
- the vertical spacing of the Geogrid reinforcement layers; and
- the embedment lengths of the Geogrid reinforcement layers.

Detailed instructions for determining these parameters are included in Tensar Technical Note: *Slope Reinforcement with Tensar Geogrids, Design and Construction Guidelines.*

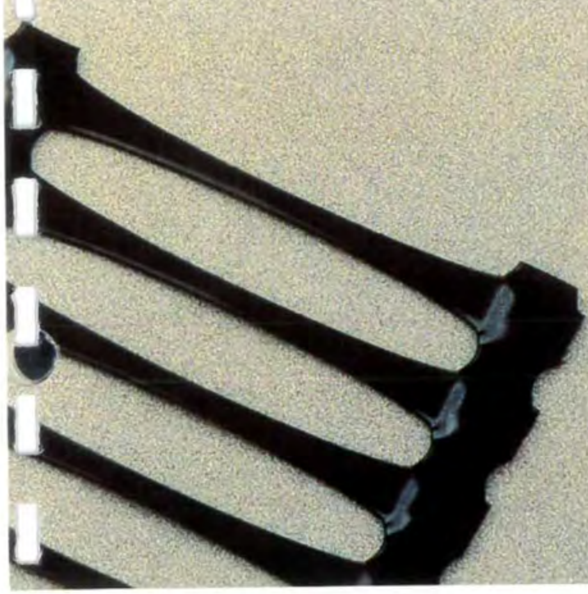
Conventional Unreinforced Soil Slope



Steepened Reinforced Slope



The Tensar Corporation also has design software and technical support services available for developing reinforced slope designs for complex stability problems.



Designing with Tensar Geogrids

Design of reinforced soil slopes with Tensar Geogrids is straight forward, as is shown in the typical example which follows. All designs should, of course, always be confirmed by adequate site investigations and finalized by a qualified professional engineer.

Example: Design a Tensar Geogrid reinforced soil slope for the conditions shown in Figure 1, where:

H = Slope height

q = Uniform surcharge loading on top of slope

γ_m = Moist unit weight of the soil fill

ϕ' = Effective soil friction angle

FS = Minimum Factor of Safety

β = Slope angle

c' = Effective soil cohesion

u = Pore pressure

1. Determine the modified slope height, H', to take surcharge loading, q, into consideration as follows:
$H' = H + q/\gamma_m$

$$\mathbf{H' = 38 + \frac{240psf}{120pcf} = 40\ ft}$$

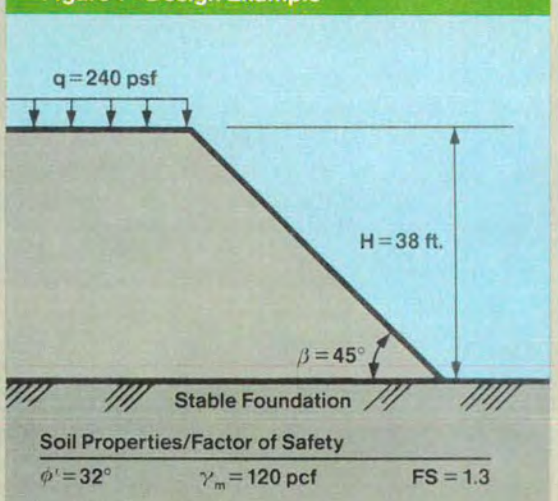
(Note: If H' exceeds 1.2H, contact The Tensar Corporation for further guidance).

2. Determine the factored soil friction angle, ϕ'_f, to incorporate the desired minimum factor of safety, FS, in the design as follows:

$$\phi'_f = \tan^{-1}\left(\frac{\tan\phi'}{FS}\right)$$

$$\phi'_f = \tan^{-1}\left(\frac{\tan 32°}{1.3}\right) = \tan^{-1}\frac{.624}{1.3} = 25.7°$$

Figure 1—Design Example



3. Determine the Tensar Geogrid force coefficient, K, as indicated in Figure 2 by the proposed slope angle, β, and the factored soil friction angle, ϕ'_f.

K = 0.14

4. Determine the total horizontal force, T, to be resisted by the Tensar Geogrid reinforcement as follows:
$T = 1/2\, K\, \gamma_m (H')^2$

$$\mathbf{T = (1/2)(0.14)(120)(40)^2 = 13{,}440\ lbs/ft}$$

5. Determine the minimum number of reinforcement layers required, N_{min}, as indicated by long-term allowable design tensile strength of the Geogrid, T_A, as follows:

$$N_{min} = \frac{T}{T_A} \qquad \mathbf{N_{min} = \frac{13{,}440}{1{,}000} \sim 14\ (for\ SR1)}$$

$$\mathbf{N_{min} = \frac{13{,}440}{2{,}000} \sim 7\ (for\ SR2)}$$

$$\mathbf{N_{min} = \frac{13{,}440}{3{,}000} \sim 5\ (for\ SR3)}$$

where: T_A = 1,000 lbs/ft for UX1100 (SR1) Geogrids
T_A = 2,000 lbs/ft for UX1200 (SR2) Geogrids
T_A = 3,000 lbs/ft for UX1300 (SR3) Geogrids

6. Determine the required geogrid embedment length, at the top (L_T) and at the bottom (L_B) of the slope as indicated in Figure 3 by the proposed slope angle β, and the factored soil friction angle, ϕ'_f. The minimum embedment lengths required for geogrid layers at intermediate levels between the top and bottom of the slope should vary linearly between L_T and L_B. In actual construction any convenient length in excess of these minimums may be used.

$L_B/H' = .73$ $\qquad$ $\mathbf{L_B = 29.2'}$

$L_T/H' = .53$ $\qquad$ $\mathbf{L_T = 21.2'}$

7. Determine the maximum vertical spacing of the Tensar Geogrid reinforcements, S_{max}, at each elevation as follows:

$$S_{max} = \frac{T_A}{K\gamma_m z}$$

where z = distance beneath the actual slope crest plus q/γ_m.

This computation will result in a theoretical maximum spacing curve. Figure 4 shows the spacing curves for the three UX type geogrids described above, based on the design problem in Figure 1. Vertical spacing increases as you approach the top of the slope.

8. Determine the actual vertical geogrid spacing as shown in Figure 4. Geogrid spacing should not exceed four feet. If calculations indicate a spacing greater than four feet, you should utilize a lower strength geogrid. For simplicity of construction, uniform geogrid spacings are normally established. Figure 4 assumes that the fill is placed in 8" lifts. A maximum spacing of 36" would, therefore, result in placement of the next lowest multiple of 8" (i.e., 32"). See TTN:SR1 for a complete discussion on geogrid spacing.

9. An example of the completed design including intermediate reinforcing layers is shown in Figure 5. Note the use of BX 1100 (SS1) geogrids as intermediate layers to provide stability at the face of the slope.

Figure 2—Force Coefficients

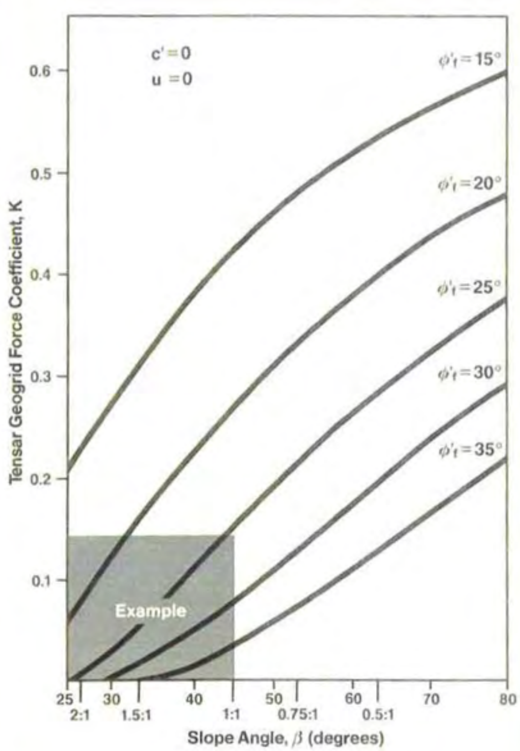
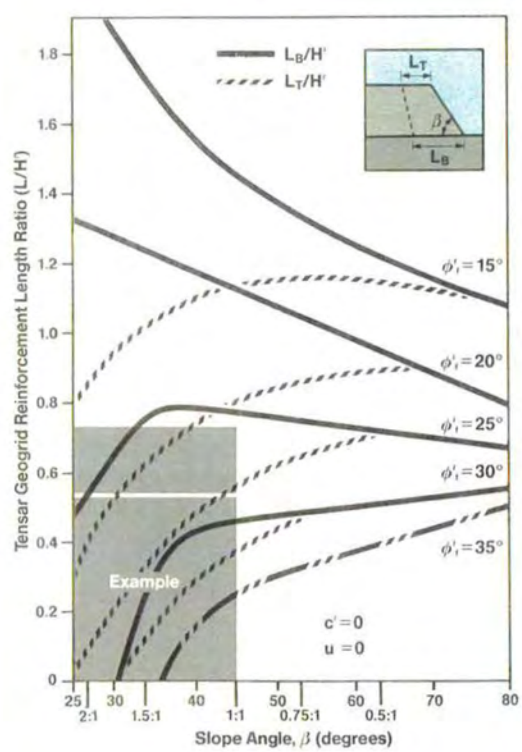


Figure 3—Embedment Lengths



Design Assumptions:
Figures 2 and 3 indicate the number and embedment length of Tensar Geogrids layers needed in the design of a reinforced soil slope. These relationships are based on hundreds of trial wedge analyses for homogeneous compacted soil slopes built over stable foundations. These analyses also assume that the slope face is planar, the top of the slope is horizontal, there are no seismic forces and that the pore water pressure in the slope is zero. Note that these relationships apply only to Tensar Geogrids.

When the slope angles exceed 50 degrees, wrap-around construction is recommended to simplify construction. When using this method the maximum vertical spacing of geogrids should not exceed two feet.

Figure 4—Reinforcement Spacing

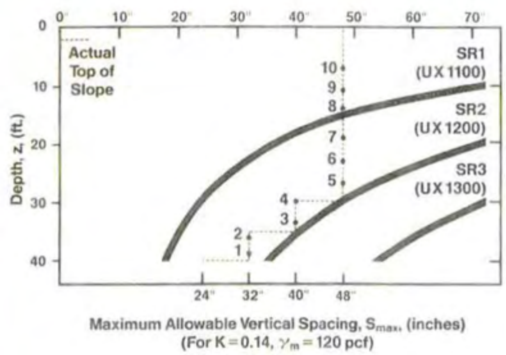
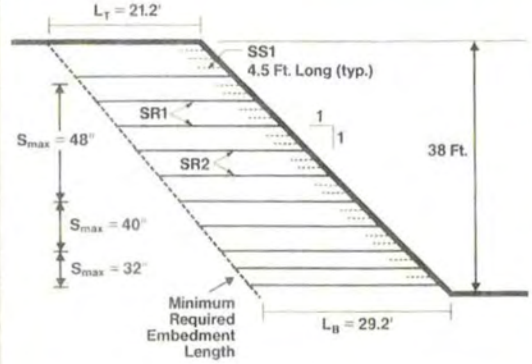


Figure 5—Illustrated Design



The Tensar Corporation

The Tensar Corporation is a manufacturer of high performance, premium quality products for use in reinforcement, support, containment, enclosure, and drainage applications. Tensar products have undergone extensive performance testing by private industry, universities, engineering consultants, and state and federal agencies. Tensar products have been used in hundreds of construction, industrial, marine, and military applications around the world. Several thousand Tensar installations are now in service.

Technical Assistance

Tensar's active research and technical assistance programs are geared to meet your needs. For a preliminary cost evaluation or design assistance for your next reinforced slope project, contact your local Tensar representative.

For additional information about Tensar products and applications, call or write:

The Tensar Corporation
1210 Citizens Parkway
Morrow, GA 30260
(404) 968-3255
1-800-845-4453

The Tensar Corporation also supplies Retaining Wall Systems for those situations in which a reinforced soil retaining wall is required. For additional information ask for our Tensar Geowall brochure.

TENSAR

the engineered advantage™

Tensar Geogrids are distributed throughout the United States by Contech Construction Products Inc. For more information on Tensar designs and applications, contact your Contech representative or the Contech Construction Products office nearest you.

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GEOTECHNICAL INVESTIGATION

POINT LOMA WASTEWATER TREATMENT PLANT SITE IMPROVEMENTS

San Diego, California

PREPARED FOR:

Gail Masutani, PhD, PE
HDR
591 Camino de la Reina, Suite 300
San Diego, CA 92108

PREPARED BY:

Atlas Engineering West Inc.
6280 Riverdale Street
San Diego, CA 92120

September 29, 2020

ATLAS

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(877) 215-4321 | oneatlas.com

September 29, 2020

Atlas No. 190329P4.3
Report No. 1

GAIL MASUTANI, PHD, PE
HDR
591 CAMINO DE LA REINA, SUITE 300
SAN DIEGO, CALIFORNIA 92108

Subject: Geotechnical Investigation
Point Loma Wastewater Treatment Plant Site Improvements
San Diego, California

Dear Ms. Masutani:

Atlas (formerly SCST) is pleased to present our report describing the geotechnical investigation performed for the subject project. We conducted the investigation in general conformance with the scope of work presented in our revised proposal dated December 18, 2019. Based on the results of our investigation, we consider the planned development feasible from a geotechnical standpoint, provided the recommendations of this report are followed. If you have any questions, please call us at (619) 280-4321.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
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NO. C87787
EXP: 09/30/21
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Chief Geologist

PFL:DM:GLC:AKN:ds

Distribution: gail.masutani@hdrinc.com

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APPENDICES

1. INTRODUCTION

This report presents the results of the geotechnical investigation Atlas (formerly SCST) performed for the subject project. We understand that the project includes the design and construction of storm water site improvements. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project. Figure 1 presents a site vicinity map.

2. SCOPE OF WORK

We conducted this investigation in general conformance with the scope of work presented in our Proposal No. 19-0830R2 dated December 18, 2019. Our scope of work consisted of the following.

2.1 Subsurface Exploration

We explored the subsurface conditions by drilling five borings (B-1 through B-4 and B-6) to depths between about 4 and 20 feet below the existing ground surface using a truck-mounted drill rig equipped with a hollow stem auger. We were not able to drill one of the planned borings (B-5) due to existing utilities. Figure 2 shows the approximate locations of the borings. Two Atlas engineers logged the borings and collected samples of the materials encountered for laboratory testing. The logs of the borings are presented in Appendix I. Soils are classified according to the Unified Soil Classification System illustrated on Figure I-1.

2.2 Laboratory Testing

Selected samples obtained from the borings were tested to evaluate pertinent soil classification and engineering properties and enable development of geotechnical conclusions and recommendations. The laboratory tests consisted of in situ moisture and density, particle-size distribution, Atterberg limits, corrosivity, expansion index and direct shear. The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix II.

2.3 Analysis and Report

The results of the field and laboratory tests were evaluated to develop conclusions and recommendations regarding:

- Subsurface conditions beneath the site
- Groundwater levels and the necessity for dewatering
- Potential geologic hazards
- Criteria for seismic design in accordance with the 2019 California Building Code (CBC)
- Site preparation and grading
- Foundation alternatives and geotechnical engineering criteria for foundation design
- Estimated foundation settlements
- Support for concrete slabs-on-grade
- Excavation characteristics of the subsurface materials encountered

- Backfill recommendations and the suitability of excavated materials for use as backfill
- Allowable temporary excavation side slope and shoring recommendations
- Lateral earth pressures and resistance to lateral loads
- Support for the pipeline
- Potential pipeline settlements
- Appropriate types of bedding and backfill materials as well as placement and compaction procedures
- Suitability of excavated materials for use as backfill
- Soil modulus E' for pipeline design
- Corrosivity

3. SITE DESCRIPTION

The planned project is located at the Point Loma Wastewater Treatment Plant, at 1902 Gatchell Road, on Point Loma San Diego, California. The site is generally bound by the Pacific Ocean to the west, a hillside to the east, military property to the north and Cabrillo National Monument to the south. Existing improvements consist of pavements, hardscape, pipelines and the facility buildings. Site elevations range from about 18 feet on the west side of the site to about 95 feet MSL on the south and east portions of the site.

4. PROPOSED DEVELOPMENT

Based on our review of the provided schematic site plans and discussions with the team, we understand the project will consist of the design and construction of storm water site improvements including pipelines, force mains, various drains, catch basins, and two pump stations.

5. GEOLOGY AND SUBSURFACE CONDITIONS

The site is located within the Peninsular Ranges Geomorphic Province of California, which stretches from the Los Angeles basin to the tip of Baja California. This province is characterized as a series of northwest trending mountain ranges separated by subparallel fault zones and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith, while the coastal plain is underlain by subsequently deposited marine and non-marine sedimentary formations. The site is located in the coastal plain and is underlain by fill, very old paralic deposits and Cabrillo Formation. Descriptions of the materials encountered are presented below. Figure 3 presents a geologic cross-section. Figure 4 presents the regional geology in the vicinity of the site.

- **Fill (Qf):** Fill was encountered beneath the existing pavement sections in borings B-1 through B-4, and at the ground surface in boring B-6. As encountered, the fill generally consisted of loose to very dense silty and clayey sand and hard sandy clay with varying amounts of gravel and cobbles. Soil cement was encountered in boring B-2 from depths between about 1 foot and 6 feet below the existing ground surface. Sand cement slurry was encountered in boring B-4 from about 1½ to 4 feet below the ground surface.

- **<u>Very Old Paralic Deposits (Qvop):</u>** Very old paralic deposits were encountered underlying the fill in boring B-2 and consisted of weakly to strongly cemented clayey and silty sandstone.

- **<u>Cabrillo Formation (Kcs):</u>** Cabrillo Formation was encountered beneath the very old paralic deposits in boring B-2, and beneath the fill in boring B-3. As encountered, the Cabrillo Formation generally consisted of strongly indurated claystone and moderately cemented silty sandstone. Interbedded sandstone and claystone layers were observed.

- **<u>Groundwater:</u>** Groundwater was not encountered in the borings. However, groundwater levels may fluctuate in the future due to tides, rainfall, irrigation, broken pipes, or changes in site drainage. Because groundwater rise or seepage is difficult to predict, such conditions are typically mitigated if and when they occur.

6. GEOLOGIC HAZARDS

The site is not located within an area previously known for significant geologic hazards. Evidence of active faulting, liquefiable soils, or collapsible soils was also not observed during our investigation. A discussion of existing and potential geologic hazards follows.

6.1 Faulting and Surface Rupture

The closest known active fault is the Newport-Inglewood-Rose Canyon fault zone, located about 2½ miles east of the site. The closest mapped fault is the potentially active Point Loma Fault, which trends through the site. The Point Loma Fault is not known to have offset Holocene sediments, indicating it is not active. The State of California does not consider this fault to be active, and as such, an Alquist-Priolo Earthquake Fault Zone has not been established. In our opinion and according to the guidelines of the State of California, the fault is not a potential source of seismic shaking or ground rupture. No active faults are known to underlie or project toward the site; therefore, the probability of fault rupture is low.

6.2 CBC Seismic Design Parameters

A geologic hazard likely to affect the project is ground shaking as a result of movement along an active fault zone in the vicinity of the subject site. Based on the subsurface conditions encountered during our investigation, the area near the proposed northern pump station (in the vicinity of boring B-4) may be classified as a Site Class C. The area near the proposed southern pump station (in the vicinity of boring B-6) may be classified as a Site Class D. The mapped site coefficients and maximum considered earthquake (MCE_R) spectral response acceleration parameters in accordance with the ASCE 7-16 (SEAOC, 2020) are presented in Tables 1 and 2.

Table 1 – ASCE 7-16 Mapped Site Coefficients, Northern Pump Station

Site Coordinates		
Latitude: 32.67854°	Longitude: -117.24723°	
Site Coefficients and Spectral Response Acceleration Parameters		**Value**
Site Class		C
Site Coefficients, F_a		1.200
Site Coefficients, F_v		1.500
Mapped Spectral Response Acceleration at Short Period, S_s		1.231g
Mapped Spectral Response Acceleration at 1-Second Period, S_1		0.424g
Mapped Design Spectral Acceleration at Short Period, S_{DS}		0.985g
Design Spectral Acceleration at 1-Second Period, S_{D1}		0.424g
Site Peak Ground Acceleration, PGA_M		0.655g

Table 2 – ASCE 7-16 Mapped Site Coefficients, Southern Pump Station

Site Coordinates		
Latitude: 32.67732°	Longitude: -117.24647°	
Site Coefficients and Spectral Response Acceleration Parameters		**Value**
Site Class		D
Site Coefficients, F_a		1.006
Site Coefficients, F_v		*See Note 1
Mapped Spectral Response Acceleration at Short Period, S_s		1.236g
Mapped Spectral Response Acceleration at 1-Second Period, S_1		0.425g
Mapped Design Spectral Acceleration at Short Period, S_{DS}		0.829g
Design Spectral Acceleration at 1-Second Period, S_{D1}		*See Note 1
Site Peak Ground Acceleration, PGA_M		0.604g

*** Note 1 – ASCE 7-16, Section 11.4.8. A site-specific ground motion analysis is required to be performed in accordance with Section 21 unless exempted in accordance with Section 20.3.1**

For a Site Class D, a site-specific ground motion analysis is required to be performed in accordance with the requirements of ASCE 7-16. However, we assume that a site-specific ground motion analysis is likely not needed for the proposed southern pump station at this time. If a site-specific analysis is required, a report addendum can be issued at a later date.

6.3 City of San Diego Seismic Safety Study

Figure 5 shows the site location on the City of San Diego Seismic Safety Study Map (City of San Diego, 2008). The project site is located in Geologic Hazard Categories 12, 44, and 53. Geologic Hazard Category 12 is a fault zone defined as potentially active, inactive, presumed inactive, or activity unknown. Geologic Hazard Category 44 is a coastal bluff defined as moderately stable, mostly stable formations, and local high erosion. Geologic Hazard Category 53 is defined as level or sloping terrain with unfavorable geologic structure and a low to moderate risk. Unfavorable geologic structure was not observed during our investigation. In our opinion, the geologic risks for the site are moderate to high.

6.4 Liquefaction and Dynamic Settlement

Liquefaction occurs when loose, saturated sands and silts are subjected to strong ground shaking. The soils lose shear strength and behave like liquid, resulting in large total and differential ground surface settlements and possible lateral spreading during an earthquake. Due to the relatively dense nature of the materials beneath the site, the potential for liquefaction and dynamic settlement to occur is considered low.

6.5 Landslides

Evidence of landslides or slope instabilities was not observed during our investigation. The potential for landslides or slope instabilities to occur at the site is considered low. The site is not mapped within a known landslide area (California Department of Conservation, 2015).

6.6 Slope Stability Analysis

The slope stability analyses were performed using SLIDE v. 6.0, a product of Rocscience, Inc. (2016). SLIDE is a two-dimensional, limit equilibrium slope stability program that evaluates the factor of safety of soil and rock slopes against both circular and non-circular failure surfaces. The Spencer's method was used. This method of analysis provides the factor of safety based on both force and moment equilibrium. Bishop's simplified method and Janbu's simplified method were also evaluated to compare and consider the results. The analyses were performed to examine both the global and local stability of the slope under static and pseudostatic conditions.

Traffic loads from existing access roads along the slope were represented as a uniform surcharge load of 250 pounds per square foot (psf) in general accordance with 2014 AASHTO LRFD. Additionally, a 100 psf distributed live load was assumed for the existing south throttling facility. The unit weight of water was also adjusted to 64 pounds per cubic foot (pcf) to account for the presence of salt water.

Three different materials were defined to represent the subsurface conditions. These materials include Cabrillo formation, very old paralic deposits and existing undocumented fill. The properties of the materials selected for the analyses are listed in Table 3.

Table 3 – Strength Parameters for Slope Stability Analyses

Material Name	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Undocumented Fill (Qf)	120	100	30
Very Old Paralic Deposits (Qvop)	127	470	37
Cabrillo Formation (Kcs)	130	670	35

The shear strength parameters were derived from laboratory test results and our experience with similar materials on previous projects in the site area.

Analyses of the existing conditions were performed on Cross-Section A-A' (Figure 3). The results are presented in Appendix III and summarized in Table 4.

Table 4 – Slope Stability Analyses Results

Cross Section	Factor of Safety	
	Static	Pseudostatic
A-A' (Global)	2.158	1.533
A-A' (Local 1)	2.166	1.650
A-A' (Local 2)	1.602	1.199

Factors of safety of 1.5 (static) and 1.15 (pseudostatic) are considered adequate in standard geotechnical practice.

6.7 Flooding, Tsunamis, and Seiches

It should be noted that the tsunami inundation line runs north-south along the west side of the site (CAL EMA, 2009). However, the site is not located within a mapped tsunami inundation area on the State of California Tsunami Inundation Maps (Cal EMA, 2009); therefore, damage due to tsunami inundation is considered low. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered low. The site is mapped within an area of minimal flood hazard (FEMA, 2019).

6.8 Subsidence

The site is not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum); therefore, the potential for subsidence due to the extraction of fluids is considered low.

6.9 Hydro-Consolidation

Hydro-consolidation can occur in recently deposited sediments (less than 10,000 years old) that were deposited in a semi-arid environment. Examples of such sediments are aeolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore spaces between the particle grains can re-adjust when inundated by groundwater causing the material to consolidate. The relatively dense materials underlying the site are not susceptible to hydro-consolidation.

7. CONCLUSIONS

Based on the results of our investigation, we consider the proposed construction feasible from a geotechnical standpoint provided the recommendations of this report are followed. In our opinion, the site conditions are suitable to construct the proposed improvements. The main geotechnical considerations affecting the proposed construction are the presence of potentially compressible soils, cut/fill transitions, and difficult excavations in soil cement, very old paralic deposits and Cabrillo Formation. We understand that project plans are not available at this time, and the locations and depths of the proposed pump stations have not been finalized. We anticipate that the proposed pump stations may be supported on shallow spread footings with bottom levels bearing either entirely on compacted fill, or entirely on formation (very old paralic deposits or Cabrillo Formation). Remedial grading is recommended to reduce the potential for distress to the proposed improvements. Remedial grading recommendations are provided herein. The recommendations presented herein may need to be updated once final plans are developed.

8. RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction as well as geotechnical recommendations for the design of the proposed structure and improvements. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations appear not to address a specific feature of the project, please contact our office for additions or revisions to the recommendations.

8.1 Earthwork

Earthwork is anticipated to include site preparation, remedial grading, excavations for foundations, temporary excavations for underground utilities, and placement and compaction of fill and backfill. Grading and earthwork should be conducted in accordance with the CBC and with the recommendations of this report. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by the geotechnical consultant during construction.

8.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, and debris. Subsurface improvements that are to be abandoned should be removed, and the resulting excavations should be backfilled and compacted in accordance with the recommendations of this report. Pipeline abandonment can consist of capping or rerouting at the project perimeter and removal within the project perimeter. If appropriate, abandoned pipelines can be filled with grout or slurry as recommended by and observed by the geotechnical consultant.

8.1.2 Remedial Grading – Pump Station Pads

Beneath the proposed pump station pads, we recommend over-excavating a minimum of 3 feet below planned subgrade elevation, or 3 feet below the proposed footing bottoms, whichever is deeper. The removal and recompaction should extend at least 5 feet outside the pump station footprint. At Atlas representative should observe the conditions exposed in the bottom of excavations to assess if additional excavation is recommended.

Additionally, the proposed pump stations should not be underlain by cut/fill transitions or transitions from shallow fill to deep fill. Where such transitions are encountered, the formational materials should be over-excavated and replaced with compacted fill to provide a relatively uniform thickness of compacted fill beneath the entire structure and reduce the potential for differential settlement. The over-excavation depth should be at least 3 feet below the planned finished pad elevation, at least 2 feet below the deepest planned footing bottom elevation, or to a depth of H/2, whichever is deeper, where H is the greatest depth of fill beneath the structure. Horizontally, the over-excavation should extend at least 5 feet outside the planned footing perimeter or up to existing improvements, whichever is less. Where practical, the bottom of excavations should be sloped toward the fill portion of the site and away from its center. Alternatively, individual pump stations can be supported on spread footings with bottoms levels bearing entirely on formational materials or on 2-sack sand/cement slurry extending down to formational materials.

8.1.3 Expansive Soil

The on-site soils tested have expansion indexes of 19 and 34, classified as very low to low expansion potential. To reduce the potential for expansive heave, the top 2 feet of material beneath footings and slabs-on-grade should have an expansion index (EI) of 50 or less determined in accordance with ASTM D4829. Horizontally, the soils having an EI of 50 or less should extend at least 2 feet outside the planned improvement or up to existing improvements, whichever is less. Based on our laboratory test results, we expect that most of the on-site silty sand, clayey sand, and clayey gravel is expected to meet the expansion index criteria. The on-site sandy clay is not expected to meet the expansion index criteria. Import material may be needed.

8.1.4 Compacted Fill

The material exposed in the bottom of excavations should be scarified to a depth of 6 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. Where gravel is provided for the stabilization of the bottom of excavation, a non-woven filter fabric should be placed between the gravel and overlying fill. Fill should be placed in horizontal lifts at a thickness appropriate for the equipment spreading, mixing, and compacting the material, but generally should not exceed 8 inches in loose thickness. Fill should be moisture conditioned to near optimum moisture content and compacted to at least 90% relative compaction. The maximum density and optimum moisture content for the evaluation of relative compaction should be evaluated in accordance with ASTM D1557. The top 12 inches of subgrade beneath vehicular pavements should be compacted to at least 95% relative compaction.

8.1.5 Imported Soil

Imported soil should consist of predominately granular soil, free of organic matter and rocks greater than 6 inches. Imported soil should be observed and, if appropriate, tested by Atlas prior to transport to the site to evaluate suitability for the intended use.

8.1.6 Excavation Characteristics

It is anticipated that excavations can be achieved with conventional earthwork equipment in good working order. Difficult excavation should be anticipated in the soil cement fill, slurry, very old paralic deposits and Cabrillo Formation. Excavations may generate oversized material that will require extra effort to crush or haul off site.

8.1.7 Oversized Material

Excavations have the potential to generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in fill, landscape material, or disposed of off site.

8.1.8 Temporary Excavations and Shoring

Temporary excavations will be required for open cut and cover trenching. Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations should be laid back no steeper than 1½:1 (horizontal:vertical). The faces of temporary slopes should be inspected daily by the contractor's Competent Person before personnel are allowed to enter the excavation. Zones of potential instability, sloughing or raveling should be brought to the attention of the Engineer and corrective action implemented before personnel begin working in the trench.

Excavated materials should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. Atlas should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Slopes steeper than those described above will require shoring. Soldier piles and lagging, sheet piles, internally braced shoring, or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring.

For design of cantilevered shoring with level backfill, the active earth pressure can be taken as equivalent to a fluid weighing 40 pcf. An additional 25 pcf should be added for shoring with 2:1 sloping ground or braced shoring. The surcharge loads on shoring from traffic and construction equipment working adjacent to the excavation can be modeled by assuming an additional 2 feet of soil behind the shoring.

8.1.9 Groundwater Seepage

Groundwater seepage may occur locally and should be anticipated in excavations.

8.1.10 Slopes

Long-term slopes should be constructed no steeper than 2:1 (horizontal:vertical). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment, or by overfilling and cutting back to design grade. Fills should be benched into sloping ground inclined steeper than 5:1 (horizontal:vertical). In our opinion, slopes constructed no steeper than 2:1 (horizontal:vertical) will possess an adequate factor of safety. An engineering geologist should observe cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. Slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

8.1.11 Surface Drainage

Final surface grades around structures should be designed to collect and direct surface water away from the structure and toward appropriate drainage facilities. The ground around the structures should be graded so that surface water flows rapidly away from the structures without ponding. In general, we recommend that the ground adjacent to the structures slope away at a gradient of at least 2%. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5% within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, saturated zones of perched groundwater can develop.

8.1.12 Grading Plan Review

Atlas should review the grading plans and earthwork specifications to ascertain whether the intent of the recommendations contained in this report have been implemented and that no revised recommendations are needed due to changes in the development scheme.

8.2 Foundations

The foundation recommendations provided herein are considered generally consistent with methods typically used in southern California. Other alternatives may be available. Our recommendations are only minimum criteria based on geotechnical factors and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or by the structural engineer. The design of the foundation system should be performed by the project structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

We understand that project plans are not available at this time, and the locations and depths of the proposed pump stations have not been finalized. However, we anticipate that the proposed pump stations may be supported on shallow spread footings with bottom levels bearing either entirely on compacted fill, or entirely on formation (very old paralic deposits or Cabrillo Formation). If the foundations are to bear entirely on formation, and isolated areas of fill exist below footings, concrete or a 2-sack sand/cement slurry can be placed between the formation and design bottom of footing elevation.

8.2.1 Spread Footings

Footings should extend at least 18 inches below lowest adjacent finished grade. A minimum width of 12 inches is recommended for continuous footings and 24 inches for isolated footings. An allowable bearing capacity of 2,500 psf can be used for spread footings supported on granular compacted fill. An allowable bearing capacity of 3,500 psf can be used for spread footings supported on formation (very old paralic deposits or Cabrillo Formation). The allowable bearing capacity can be increased by 500 psf for each foot of depth below the minimum and 250 psf for each foot of width beyond the minimum up to a maximum of 4,000 psf on compacted fill or 6,000 psf on formation. The bearing value can be increased by ⅓ when considering the total loads, including wind or seismic forces. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 7 feet exists between the lower outside footing edge and the face of the slope.

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.35 can be used. An allowable passive pressure of 350 psf per foot of depth below the ground surface can be used for level ground conditions. The allowable passive pressure should be reduced for sloping ground conditions. The passive pressure can be increased by ⅓ when considering the total loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

8.2.2 Settlement Characteristics - Compacted Fill or Old Paralic Deposits

Total foundation settlements are estimated to be less than 1 inch. Differential settlements between adjacent columns and across continuous footings are estimated to be less than ¾ inch over a distance of 40 feet. Settlements should be completed shortly after structural loads are applied.

8.2.3 Settlement Characteristics - Undocumented Fill

If the proposed pump stations are planned to be constructed in areas with relatively deep undocumented fill, Atlas should be notified so that the estimated total and differential foundation settlements may be quantified.

8.2.4 Foundation Plan Review

Atlas should review the foundation plans to ascertain that the intent of the recommendations in this report has been implemented and that revised recommendations are not necessary as a result of changes after this report was completed.

8.2.5 Foundation Excavation Observations

A representative from Atlas should observe the foundation excavations prior to forming or placing reinforcing steel.

8.3 Conventional Retaining Walls

Conventional retaining walls can be supported on spread footings. The recommendations for spread footings provided in the foundation section of this report are also applicable to conventional retaining walls.

The active earth pressure for the design of unrestrained retaining walls with level backfill can be taken as equivalent to the pressure of a fluid weighing 40 pcf. The at-rest earth pressure for the design of restrained retaining wall with level backfill can be taken as equivalent to the pressure of a fluid weighing 60 pcf. These values assume a granular and drained backfill condition. Higher lateral earth pressures would apply if walls retain clay soils. An additional 25 pcf should be added to these values for walls with 2:1 (horizontal:vertical) sloping backfill. An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. The above values do not include a factor of safety. Appropriate factors of safety should be incorporated into the design. If any other surcharge loads are anticipated, Atlas should be contacted for the necessary increase in soil pressure.

Retaining walls should be designed to resist hydrostatic pressures or be provided with a backdrain to reduce the accumulation of hydrostatic pressures. Backdrains can consist of a 2-foot-wide zone of ¾-inch crushed rock. The backdrain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. Weep holes should be provided, or a perforated pipe should be installed at the base of the backdrain and sloped to discharge to a suitable storm drain facility. As an alternative, a geocomposite drainage system such as Miradrain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project architect should provide waterproofing specifications and details. Figure 6 presents typical conventional retaining wall backdrain details.

If required, the seismic earth pressure can be taken as equivalent to the pressure of a fluid weighing 28 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, static active earth pressure. The passive pressure and bearing capacity can be increased by ⅓ in determining the seismic stability of the wall.

Wall backfill should consist of granular, free-draining material having an expansion index of 20 or less. The backfill zone is defined by a 1:1 plane projected upward from the heel of the wall. Expansive or clayey soil should not be used. We anticipate that the on-site soils will not be suitable for wall backfill. Additionally, backfill within 3 feet from the back of the wall should not contain rocks greater than 3 inches in dimension. Backfill should be compacted to at least 90% relative compaction. Backfill should not be placed until walls have achieved adequate structural strength. Compaction of wall backfill will be necessary to minimize settlement of the backfill and overlying settlement sensitive improvements. However, some settlement should still be anticipated. Provisions should be made for some settlement of concrete slabs and pavements supported on backfill. Additionally, any utilities supported on backfill should be designed to tolerate differential settlement.

8.4 Pipelines

8.4.1 Pipeline Support

It is anticipated that some of the materials along the pipeline alignment will not provide adequate support for the pipe, as loose, soft, and otherwise unsuitable materials should be anticipated. Unsuitable materials encountered near trench bottom levels, as evaluated during construction by the engineer, should be excavated 1 to 3 feet as evaluated by the geotechnical consultant and replaced as compacted fill or with crushed gravel. Unsuitable materials should be removed from the full width of the trench. The bottoms of the excavations should be observed by the geotechnical consultant prior to placement of pipe bedding. The use of a stabilizing fabric such as Mirafi® HP 570 can also be used to stabilize the bottom of the excavations, if needed.

8.4.2 Modulus of Soil Reaction

A modulus of soil reaction (E') of 1,400 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

8.4.3 Thrust Blocks

For level ground conditions, a passive earth pressure of 330 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 140 psf per foot should be used below groundwater level, if encountered.

8.4.4 Pipe Bedding

Bedding material should consist of clean sand having a sand equivalent not less than 30 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The on-site materials are not expected to meet "Greenbook" bedding specifications. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

8.4.5 Trench Backfill

Excavated material free of organic debris and rocks greater than 6 inches in largest dimension are generally expected to be suitable for use as trench backfill. Imported material should not contain rocks greater than 6 inches in largest dimension or organic debris. Imported material should have an expansion index of 20 or less. Atlas should observe and, if appropriate, test proposed imported materials before they are delivered to the site. Backfill should be placed in lifts 8 inches or less in loose thickness, moisture conditioned to optimum moisture content or slightly above, and compacted to at least 90% relative compaction. All references to optimum moisture content and relative compaction in this report are based on ASTM D1557 test method. The upper 12 inches of soil beneath subgrade for pavements should be compacted to at least 95% relative compaction. We recommend that the soils in the top 24 inches below hardscape have an expansion index of 20 or less. Atlas should observe and, if appropriate, test the soils to be used within this backfill zone.

8.5 Soil Corrosivity

Representative samples of the on-site soil were tested to evaluate corrosion potential. The test results are presented in Appendix II. The project design engineer can use the sulfate results in conjunction with ACI 318 to specify the water/cement ratio, compressive strength, and cementitious material types for concrete exposed to soil. A corrosion engineer should be contacted to provide specific corrosion control recommendations.

9. GEOTECHNICAL ENGINEERING DURING CONSTRUCTION

The geotechnical engineer should review project plans and specifications prior to bidding and construction to check that the intent of the recommendations in this report has been incorporated. Observations and tests should be performed during construction. Atlas should be present during grading and construction to verify the consistency of subsurface conditions across the site with the areas explored during our subsurface evaluation. If the conditions encountered during construction differ from those anticipated based on the subsurface exploration program, the presence of Atlas during construction will enable an evaluation of the exposed conditions. Subsequently, modifications of the recommendations in this report or development of additional recommendations will be provided upon request and in a timely manner.

10. CLOSURE

Atlas should be advised of changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can, however, occur with the passage of time, whether they are due to natural processes or work on this project or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond our control. This report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations to site conditions at that time.

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

11. REFERENCES

American Concrete Institute (ACI) (2018), Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary, June.

California Emergency Management Agency, California Geological Survey, University of Southern California (Cal EMA) (2009), Tsunami Inundation Map for Emergency Planning, Point Loma Quadrangle: dated June 1.

California Department of Conservation (2015), Landslide Inventory (Beta), accessed September https://maps.conservation.ca.gov/cgs/lsi/.

City of San Diego (2008), Seismic Safety Study, Geologic Hazards and Faults, Grid Tile: 12, dated April 3.

FEMA (2019), Flood Insurance Rate Maps, National Flood Hazard Layer FIRMette, Map No. 06073C1886H: dated December 20.

International Code Council (2018), 2019 California Building Code, Based on the 2018 International Building Code, Effective Date: January 1, 2020.

Jennings, C.W. and Bryant, W.A. (2010), Fault Activity Map of California, California Geologic Survey, Geologic Data Map No. 6.

Kennedy, M.P., Tan, S.S., (2008), Geologic Map of the San Diego 30’ x 60’ Quadrangle, San Diego, California, California Geological Survey: Scale 1:100,000.

Public Works Standards, Inc. (2018), “Greenbook” Standard Specifications for Public Works Construction, 2018 Edition.

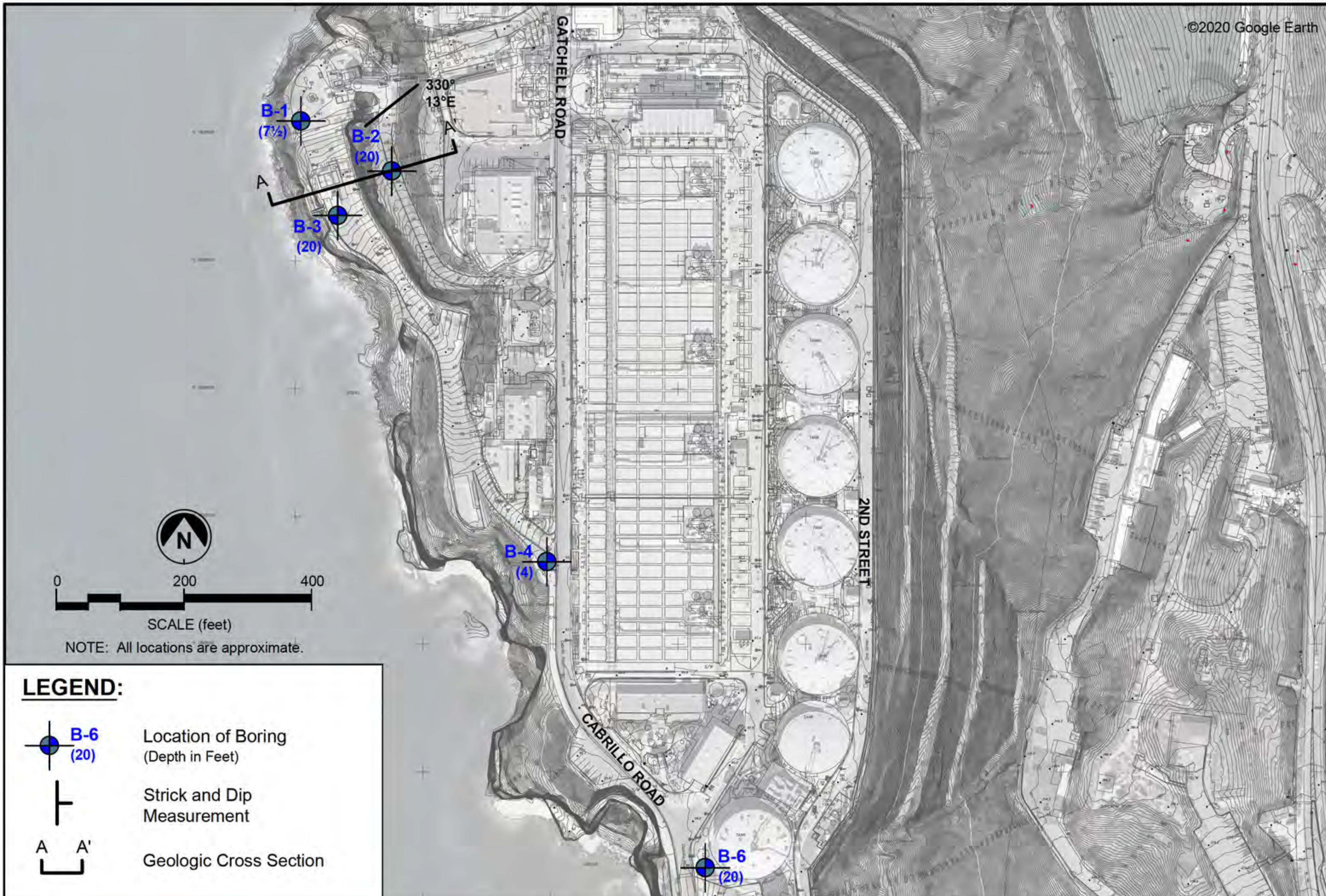
Tan, S.S., Giffen, D.G., 1995, Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, San Diego County, California, Point Loma Quadrangle, DMG Open-File Report 95-03, Map No. 33: Scale 1:24,000.

ATLAS

SITE VICINITY MAP
Point Loma Wastewater Treatment Plant
Site Improvements
San Diego, California

Date:	September, 2020
By:	ACF
Job No.:	190329P4.3-1

Figure: 1

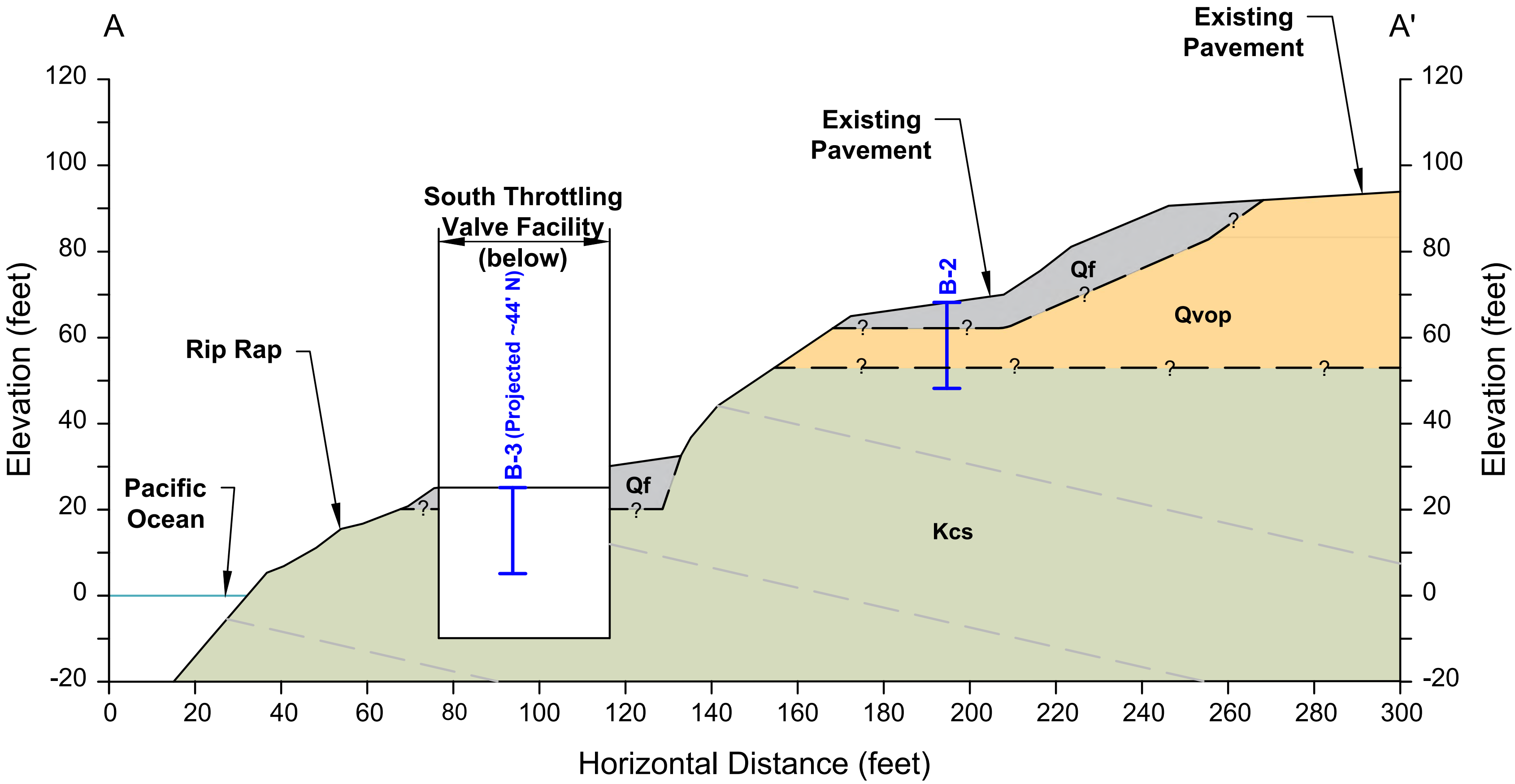


ATLAS

SUBSURFACE EXPLORATION MAP
Point Loma Wastewater Treatment Plant
Site Improvements
San Diego, California

Date:	September, 2020
By:	ACF
Job No.:	190329P4.3-1

Figure:
2



LEGEND:

B-3 Location of Boring

Attitude of measured geologic units

—?— Geologic Contact, Queried Where Uncertain

Qf	Fill
Qvop	Very old paralic deposits
Kcs	Cabrillo Formation

0 40 80

SCALE (feet)

NOTE: All locations and depths are approximate.

ATLAS

GEOLOGIC CROSS-SECTION A-A'

Point Loma Wastewater Treatment Plant

Site Improvements

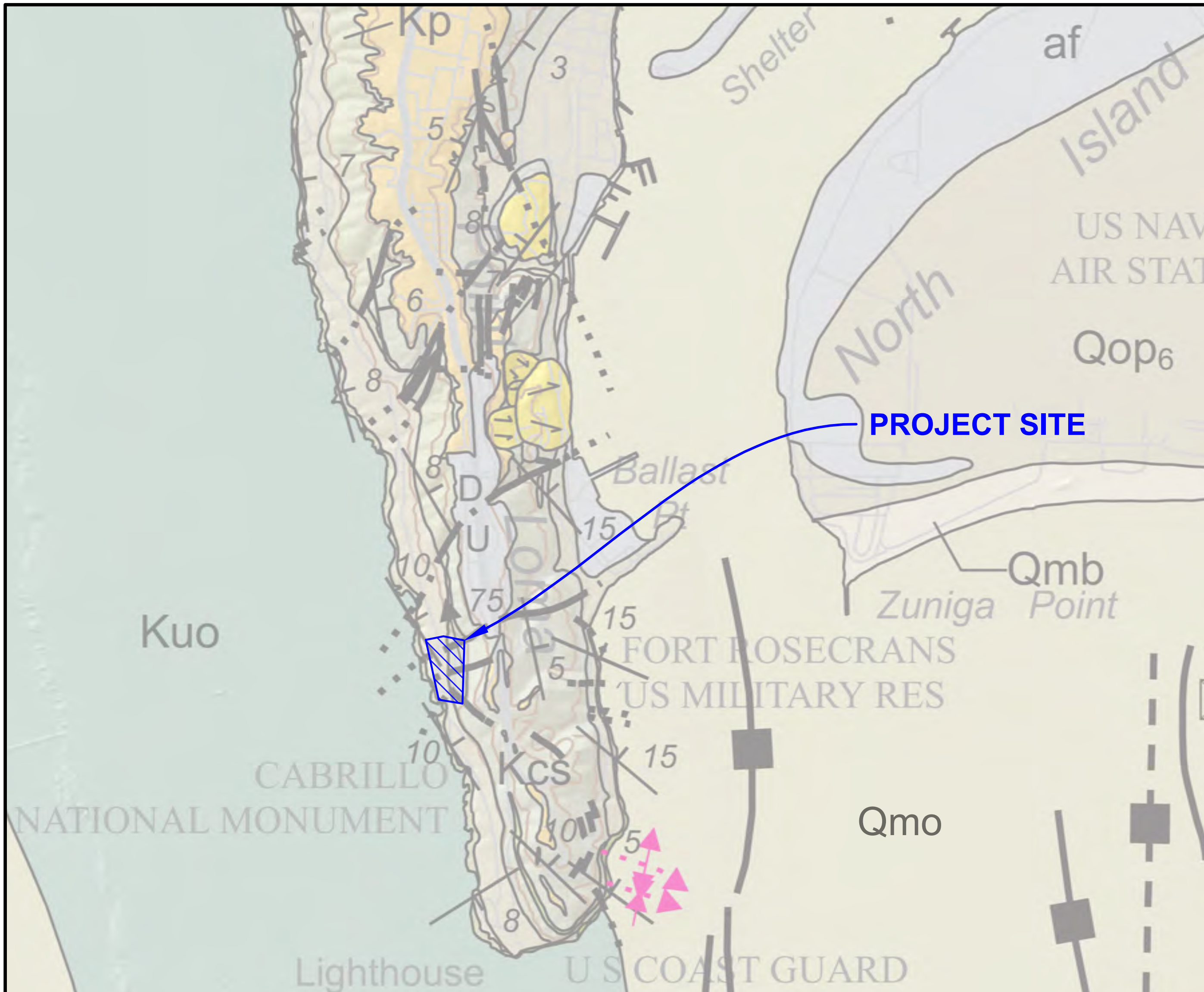
San Diego, California

Date: September, 2020

By: ACF

Job No.: 190329P4.3-1

Figure: 3



EXPLANATION:

af Artificial fill (late Holocene)

Qmb Marine beach deposits

Qmo Undivided marine deposits in offshore region

Qop_6 Old paralic deposits, undivided (late to middle Pleistocene), unit 6

Kcs Cabrillo Formation, sandstone

Kp Point Loma Formation

Kuo Undivided rocks of the Rosario Group in the offshore area

Anticline Fold - Solid where well defined; short dash where inferred

Syncline Fold - Solid where well defined; short dash where inferred

70 U D **Fault** - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.

N

0 3,000 6,000

SCALE (feet)

NOTE: All locations are approximate.

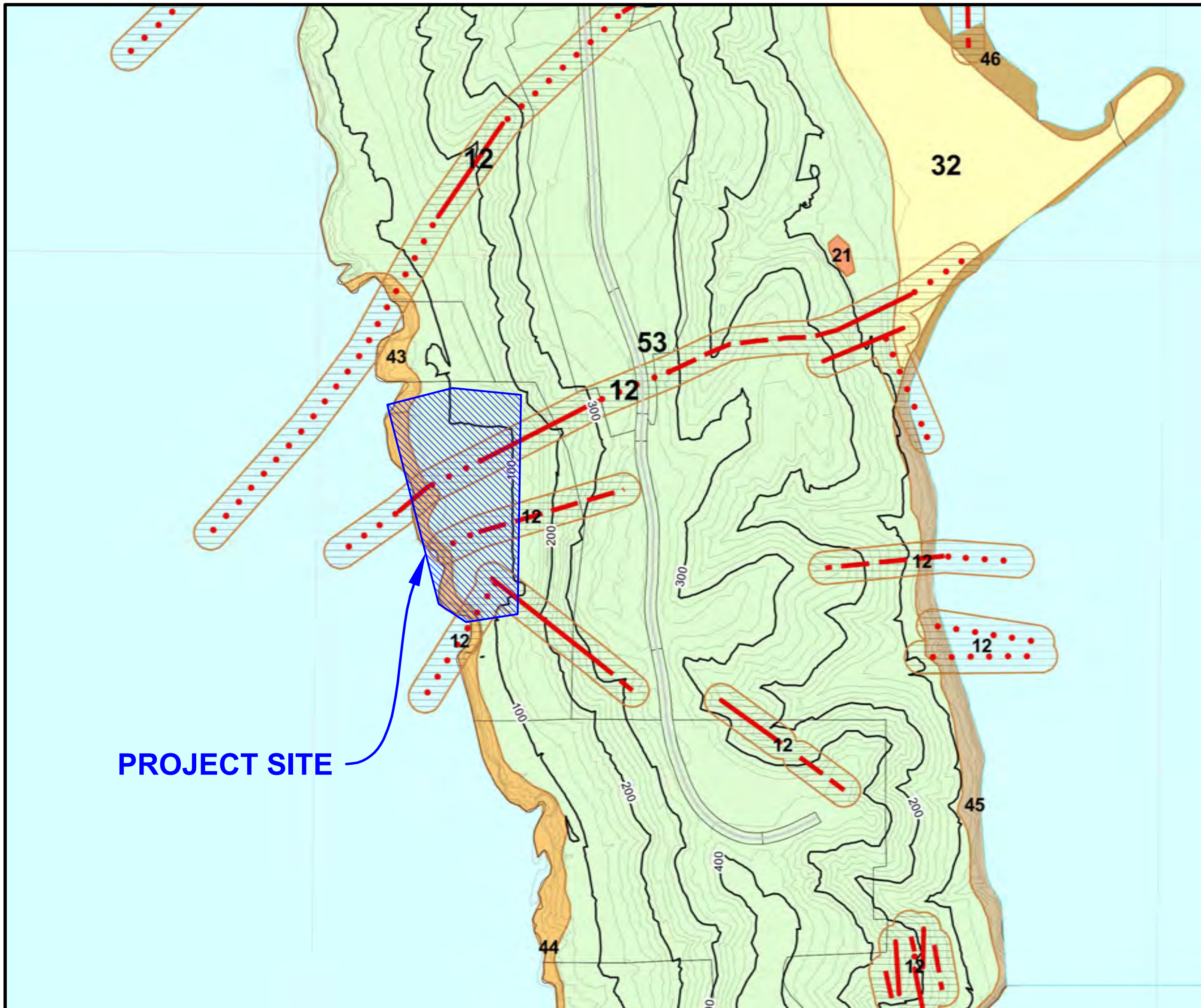
Reference:
Kennedy, M.P. and Tan, S.S. (2008), Geologic Map of the San Diego 30' x 60' Quadrangle, California, California Geological Survey, Scale 1:100,000

ATLAS

REGIONAL GEOLOGY MAP
Point Loma Wastewater Treatment Plant
Site Improvements Project
San Diego, California

Date: August, 2020
By: ACF
Job No.: 190329P4.3-1

Figure: 4



EXPLANATION:

FAULT ZONES

12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown

LANDSLIDES

21 Confirmed, known, or highly suspected

LIQUEFACTION

32 Low Potential -- fluctuating groundwater minor drainages

COASTAL BLUFFS

43 Generally unstable
Unfavorable jointing, local high erosion

44 Moderately stable
Mostly stable formations, local high erosion

45 Moderately stable
Some minor landslides, minor erosion

46 Moderately stable
Some unfavorable geologic structure, minor or no erosion

OTHER TERRAIN

53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk

Water (Bays and Lakes)

FAULTS

Fault

Inferred Fault

Concealed Fault

Shear Zone

N

0 1,000 2,000
SCALE (feet)

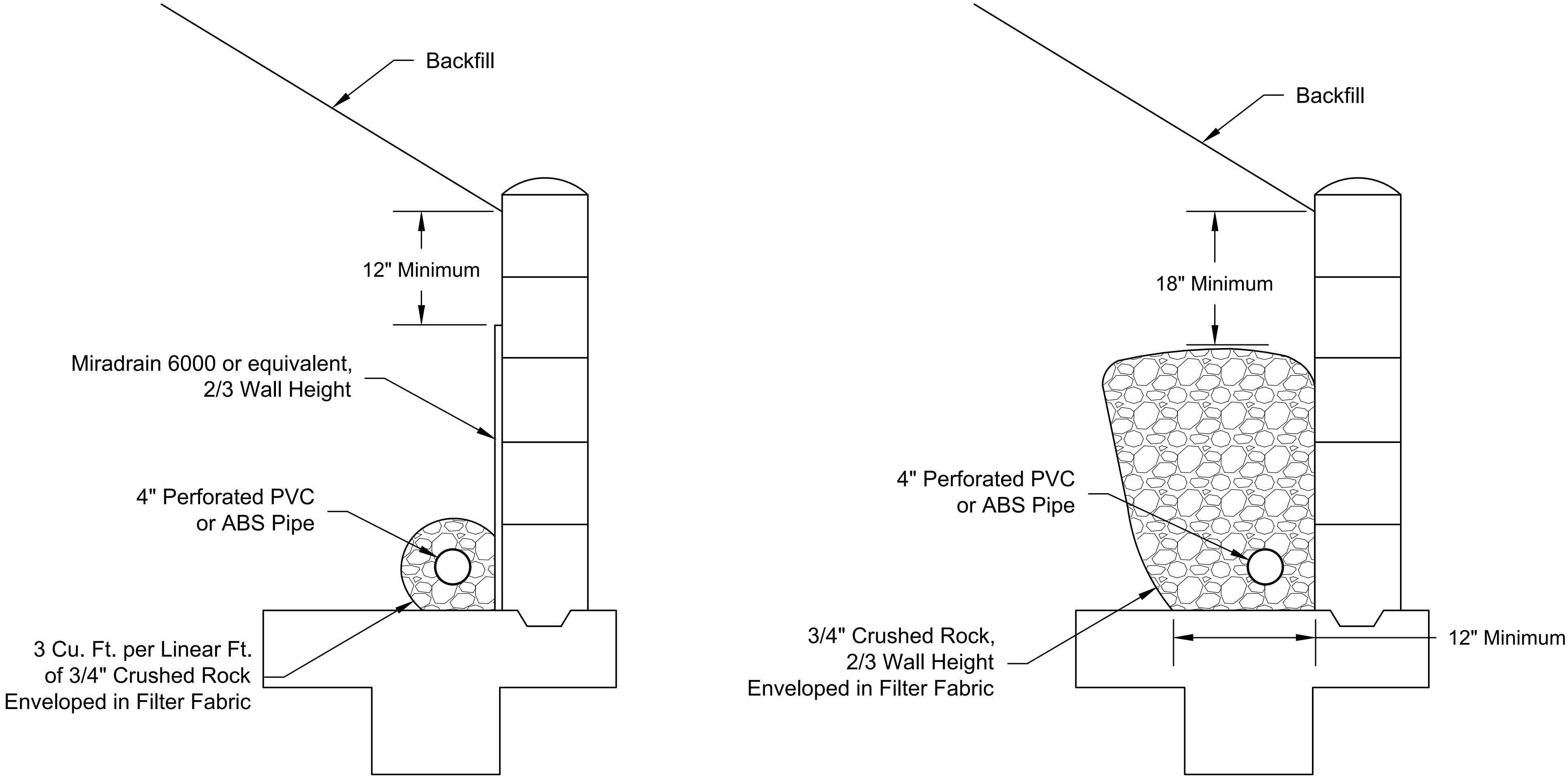
Reference:
City of San Diego (2008), Seismic Safety Study, Geologic Hazards and Faults, Grid Tile: 12 Development Services Department, April 3, Scale 1:800.

ATLAS

CITY OF SAN DIEGO SEISMIC SAFETY STUDY MAP
Point Loma Wastewater Treatment Plant
Site Improvements
San Diego, California

Date: September, 2020
By: ACF
Job No.: 190329P4.3-1

Figure: 5



NOT TO SCALE

NOTES:

1) Dampproof or waterproof back of wall following architect's specifications.
2) 4" minimum perforated pipe, SDR35 or equivalent, holes down, 1% fall to outlet. Provide solid outlet pipe at suitable locations.
3) Drain installation and outlet connection should be observed by the geotechnical consultant.

ATLAS	**TYPICAL RETAINING WALL BACKDRAIN DETAILS** Point Loma Wastewater Treatment Plant Site Improvements San Diego, California	Date: September, 2020 By: ACF Job No.: 190329P4.3-1	Figure: 6

APPENDIX I
FIELD INVESTIGATION

Our subsurface exploration consisted of drilling five borings on August 3, 2020 to depths between about 4 feet and 20 feet below the existing ground surface using a truck-mounted drill rig equipped with a hollow stem auger. Figure 2 shows the approximate locations of the borings. Our subsurface exploration was performed under the observation of two Atlas engineers who also logged the borings and obtained samples of the materials encountered.

Relatively undisturbed samples were obtained using a modified California (CAL) sampler, which is a ring-lined split tube sampler with a 3-inch outer diameter and 2½-inch inner diameter. Standard Penetration Tests (SPT) were performed using a 2-inch outer diameter and 1⅜-inch inner diameter split tube sampler. The CAL and SPT samplers were driven with a 140-pound weight dropping 30 inches. The number of blows needed to drive the samplers the final 12 inches of an 18-inch drive is noted on the boring logs as “Driving Resistance (blows/ft of drive).” SPT and CAL sampler refusal was encountered when 50 blows were applied during any one of the three 6-inch intervals, a total of 100 blows was applied, or there was no discernible sampler advancement during the application of 10 successive blows. Disturbed bulk samples were obtained from the SPT sampler and the drill cuttings.

The soils are classified in accordance with the Unified Soil Classification System as illustrated on Figure I-1. Logs of the borings are presented on Figures I-2 through I-6.

SUBSURFACE EXPLORATION LEGEND

UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION		GROUP SYMBOL	TYPICAL NAMES
I. COARSE GRAINED, more than 50% of material is larger than No. 200 sieve size.			
GRAVELS More than half of coarse fraction is larger than No. 4 sieve size but smaller than 3".	CLEAN GRAVELS	GW	Well graded gravels, gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels, gravel sand mixtures, little or no fines.
	GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
		GC	Clayey gravels, poorly graded gravel-sand, clay mixtures.
SANDS More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS	SW	Well graded sand, gravelly sands, little or no fines.
		SP	Poorly graded sands, gravelly sands, little or no fines.
		SM	Silty sands, poorly graded sand and silty mixtures.
		SC	Clayey sands, poorly graded sand and clay mixtures.
II. FINE GRAINED, more than 50% of material is smaller than No. 200 sieve size.			
	SILTS AND CLAYS (Liquid Limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, sandy silt or clayey-silt-sand mixtures with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty clays or low plasticity.
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity.
III. HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils.

SAMPLE SYMBOLS

- ☒ - Bulk Sample
- CAL - Modified California Sampler
- CK - Undisturbed Chunk sample
- MS - Maximum Size of Particle
- ST - Shelby Tube
- SPT - Standard Penetration Test sampler

GROUNDWATER SYMBOLS

- ▽ - Water level at time of excavation or as indicated
- - Water seepage at time of excavation or as indicated

LABORATORY TEST SYMBOLS

- AL - Atterberg Limits
- CON - Consolidation
- COR - Corrosivity Tests (Resistivity, pH, Chloride, Sulfate)
- CBR - California Bearing Ratio
- DS - Direct Shear
- EI - Expansion Index
- MAX - Maximum Density
- RV - R-Value
- PD - Particle-size Distribution
- SC - Soil-Cement Suitability
- SS - Soluble Sulfate

ATLAS	Point Loma Wastewater Treatment Plant Site Improvements San Diego, California		
	By: THC/PFL	Date:	September, 2020
	Job Number: 190329P4.3-1	Figure:	I-1

LOG OF BORING B-1

Date Drilled: 8/3/2020
Equipment: CME 95 with 8-Inch HSA
Elevation (ft): 18½

Logged by: THC
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVEN	SAMPLES BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		7 inches of Asphalt Concrete over 6 inches of Aggregate Base							
1	SM	**<u>FILL (Qf)</u>:** SILTY SAND with GRAVEL, loose, brown, moist, fine to coarse grained, trace cobbles.		X					
2	GC	CLAYEY GRAVEL with SAND, medium dense, light brown to brown, moist, fine to coarse grained, trace cobbles.		X					SA AL EI
3									
4									
5									
6		Very dense.	SPT		50/2"	50/2"			
7		Very difficult drilling, fragments of broken Rip-Rap encountered.	SPT		50/1"	50/1"			
8		**AUGER REFUSAL ON RIP-RAP AT 7½ FEET**							
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements
San Diego, California

By:	THC/PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	I-2

LOG OF BORING B-2

Date Drilled: 8/3/2020
Equipment: CME 95 with 8-Inch HSA
Elevation (ft): 68

Logged by: THC
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
0		**5 inches of Asphalt Concrete over 8 inches of Aggregate Base**							
1	SM	**FILL/ SOIL CEMENT (Qf):** SILTY SAND with GRAVEL, very dense, brown, moist, fine to coarse grained.							
2–4				X					
5–6			CAL		93/10"	>50	5.9	120.8	DS
6		**VERY OLD PARALIC DEPOSITS (Qvop):** CLAYEY SANDSTONE, reddish brown, moist, fine to medium grained, slightly weathered, strongly cemented, oxidized.							
10		SILTY SANDSTONE, light brown, moist, intensely weathered, weakly cemented, fine to medium grained.	SPT		40	47			
15		**CABRILLO FORMATION (Kcs):** CLAYSTONE, gray, moist, slightly weathered, strongly indurated, fine grained, interbedded sandstone layers.	CAL		50/4"	>50			
20			CAL		50/5"	>50	16.0	112.2	DS

BORING TERMINATED AT 20 FEET

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements
San Diego, California

By:	THC/PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	I-3

LOG OF BORING B-3

Date Drilled: 8/3/2020
Equipment: CME 95 with 8-Inch HSA
Elevation (ft): 33

Logged by: PFL
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVEN	SAMPLES BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		9 inches of Asphalt Concrete over 6 inches Aggregate Base							
1	SC	**FILL(Qf):** CLAYEY SAND with GRAVEL, medium dense, brown, moist, fine to coarse grained, trace cobbles.		X					
2				X					
3		Dense. Fragments of Cabrillo formation and concrete.		X					
4		**CABRILLO FORMATION (Kcs):** CLAYSTONE, gray to light brown, moist, slightly weathered, strongly indurated and cemented, fine grained, interbedded sandstone.							
5			CAL		50/1"	>50			
6			SPT		85/8"	85/8"			COR
7									
8									
9									
10			CAL		50/3"	>50			
11									
12									
13									
14									
15		SILTY SANDSTONE, yellowish brown, moist, intensely weathered, moderately cemented, fine grained.	SPT		50/6"	50/6"			
16									
17									
18									
19		Interbedded claystone layers.	SPT		75	88			
20									

BORING TERMINATED AT 20 FEET

Point Loma Wastewater Treatment Plant Site Improvements San Diego, California			
By:	THC/PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	I-4

LOG OF BORING B-4

Date Drilled: 8/3/2020
Equipment: CME 95 with 8-Inch HSA
Elevation (ft): 93

Logged by: PFL
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		6 inches of Portland Cement Concrete							
1	SM	**FILL (Qf):** SILTY SAND with GRAVEL, very dense, light brown, moist, fine to coarse grained, trace cobbles, concrete debris.							
2		SAND CEMENT SLURRY							
3									
4		**AUGER REFUSAL AT 4 FEET ON POSSIBLE UTILITY**							
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									

ATLAS	Point Loma Wastewater Treatment Plant Site Improvements San Diego, California		
By:	THC/PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	I-5

LOG OF BORING B-6

Date Drilled: 8/3/2020
Equipment: CME 95 with 8-Inch HSA
Elevation (ft): 97

Logged by: PFL
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
0–2	SM	**FILL (Qf):** SILTY SAND with GRAVEL, medium dense, brown, moist, fine to coarse grained, trace cobbles.		X					
2–5	SC	CLAYEY SAND, medium dense to dense, reddish brown, moist, fine to coarse grained, trace cobbles.		X					SA AL COR EI
5–15	CL	SANDY CLAY, hard, gray to brown, moist, fine to coarse grained, few gravel.	SPT		34	40			
10		Mangansese staining.	CAL		91	70			
15–20	SC	CLAYEY SAND, medium dense, reddish brown, moist, fine to coarse grained, trace gravel.	SPT		21	25			
19		Micaceous.	CAL		31	24			

BORING TERMINATED AT 20 FEET

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements
San Diego, California

By:	THC/PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	I-6

APPENDIX II
LABORATORY TESTING

Laboratory tests were performed to provide geotechnical parameters for engineering analyses. The following tests were performed:

- **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System.

- **IN SITU MOISTURE AND DENSITY:** The in-situ moisture content and dry unit weight were evaluated on samples collected from the borings. The test results are presented on the boring logs in Appendix I.

- **PARTICLE-SIZE DISTRIBUTION:** The particle-size distribution was evaluated on two selected soil samples in accordance with ASTM D6913. Figures II-1 and II-2 present the test results.

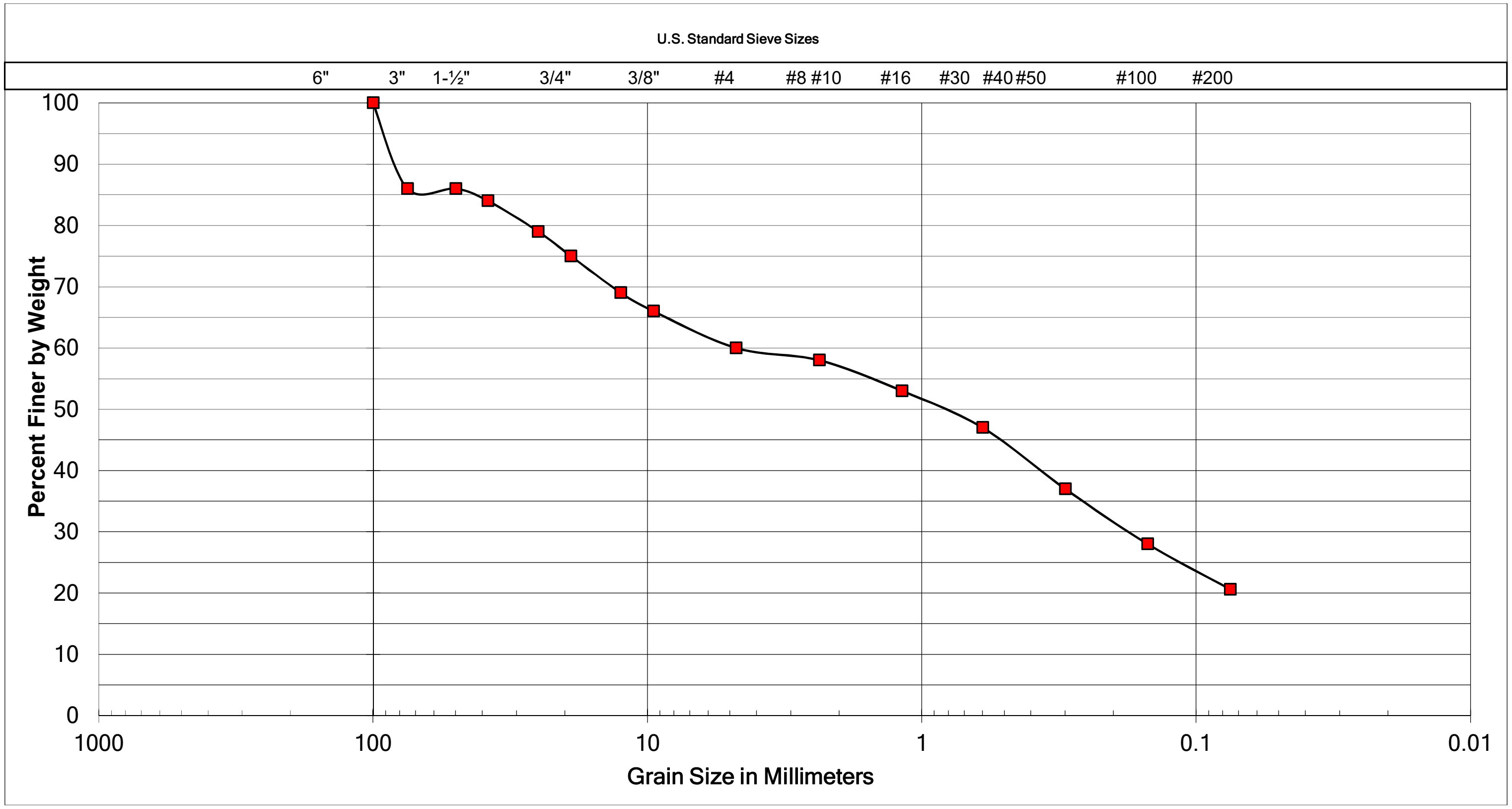
- **ATTERBERG LIMITS:** The Atterberg limits were evaluated on two selected soil samples in accordance with ASTM D4318. Figures II-1 and II-2 present the test results.

- **CORROSIVITY**: Corrosivity tests were performed on two selected soil samples. The pH and minimum resistivity were evaluated in general accordance with California Test 643. The soluble sulfate content was evaluated in accordance with California Test 417. The total chloride ion content was evaluated in accordance with California Test 422. Figure II-3 presents the test results.

- **EXPANSION INDEX:** The expansion indices were determined on two selected samples in accordance with ASTM D4829. Figure II-3 presents the test results.

- **DIRECT SHEAR:** The shear strengths were evaluated on two selected soil samples in accordance with ASTM D3080. The shear stress was applied at a constant rate of strain of 0.003 inches per minute. Figures II-4 and II-5 present the test results.

Soil samples not tested are now stored in our laboratory for future reference and analysis, if needed. Unless notified to the contrary, all samples will be disposed of 30 days from the date of this report.



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

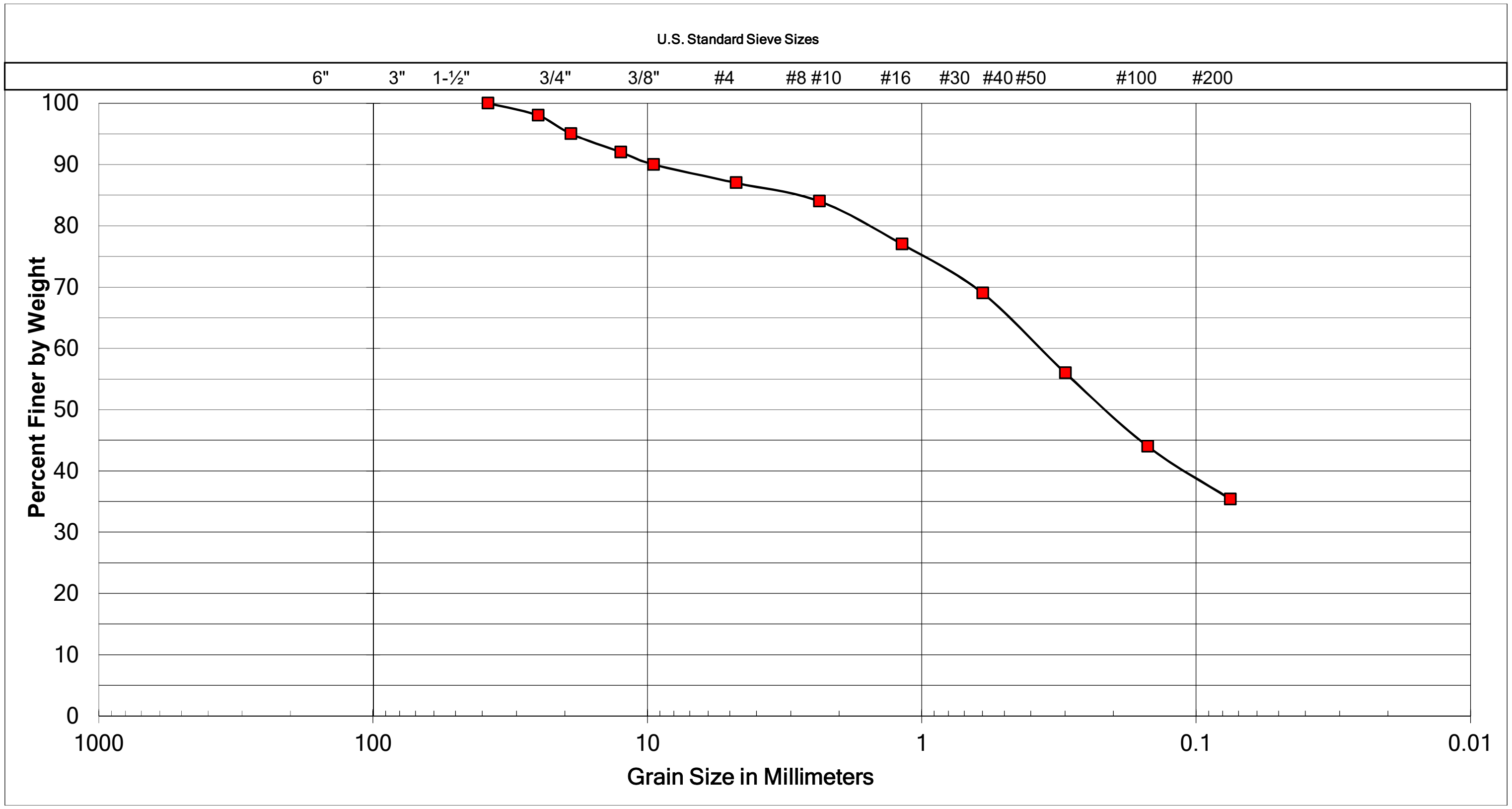
SAMPLE LOCATION
B-1 at 2 to 4 Feet
SAMPLE NUMBER
56706

UNIFIED SOIL CLASSIFICATION:	GC
DESCRIPTION	CLAYEY GRAVEL with SAND

ATTERBERG LIMITS	
LIQUID LIMIT	29
PLASTIC LIMIT	15
PLASTICITY INDEX	14

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements San Diego, California			
By:	DJM	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	II-1



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

SAMPLE LOCATION
B-6 at 2 to 4 Feet
SAMPLE NUMBER
56712

UNIFIED SOIL CLASSIFICATION:	SC
DESCRIPTION	CLAYEY SAND

ATTERBERG LIMITS	
LIQUID LIMIT	44
PLASTIC LIMIT	15
PLASTICITY INDEX	29

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements
San Diego, California

By:	DJM	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	II-2

EXPANSION INDEX

ASTM D4829

SAMPLE	DESCRIPTION	EXPANSION INDEX
B-1 at 2 to 4 Feet	CLAYEY GRAVEL with SAND	19
B-6 at 2 to 4 Feet	CLAYEY SAND	34

Classification of Expansive Soil [1]

Expansion Index	Expansion Potential
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

1. ASTM - D4829

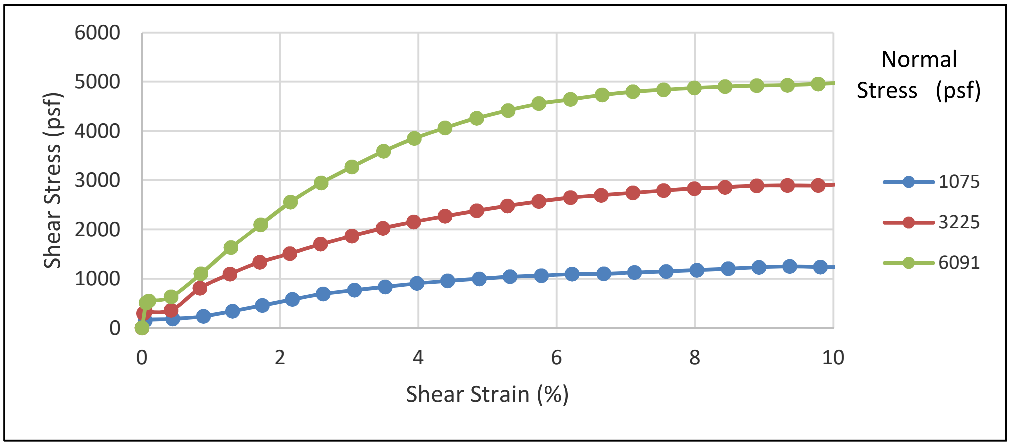
RESISTIVITY, pH, SOLUBLE CHLORIDE and SOLUBLE SULFATE

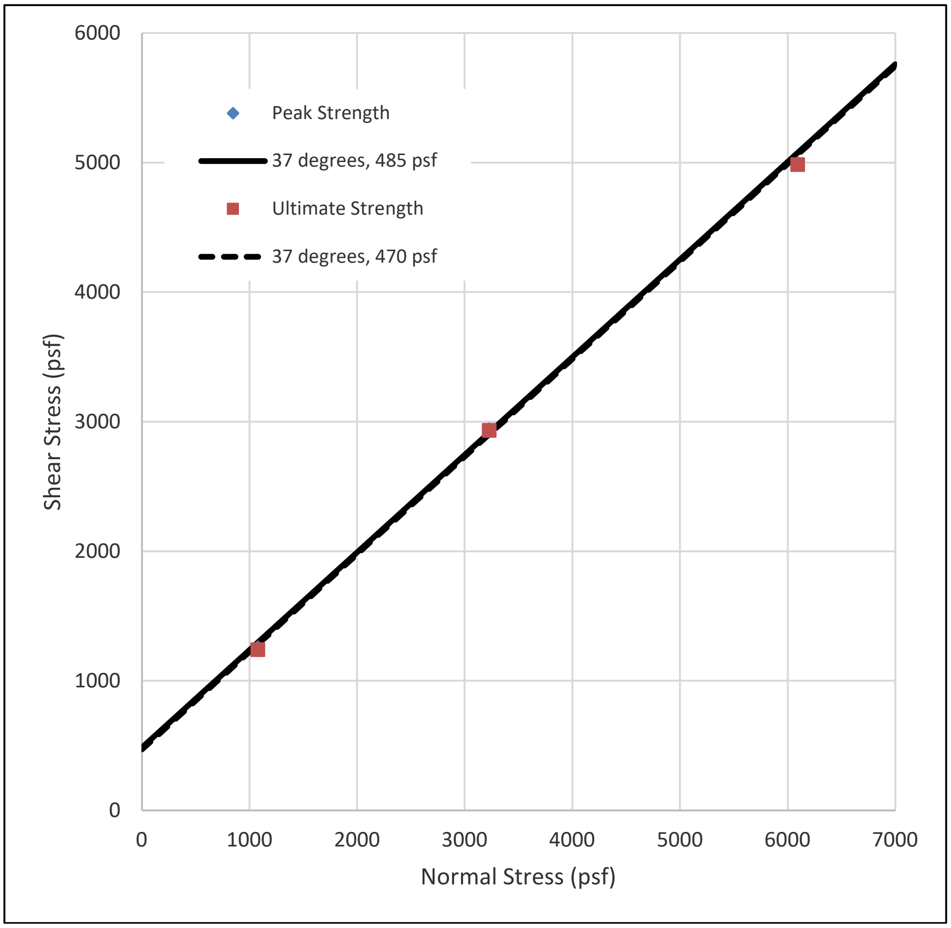
SAMPLE	RESISTIVITY (Ω-cm)	pH	CHLORIDE (%)	SULFATE (%)
B-3 at 6 to 6½ Feet	290	8.10	0.104	0.030
B-6 at 2 to 4 Feet	389	8.18	0.092	0.015

Water-Soluble Sulfate Exposure [2]

Water-Soluble Sulfate (SO_4) in soil (percent by weight)	Exposure Severity	Exposure Class	Cement Type (ASTM C150)	Max. W/C	Min. f_c' (psi)
$SO_4 < 0.10$	N/A	S0	No type restriction	N/A	2,500
$0.10 \le SO_4 < 0.20$	Moderate	S1	II	0.50	4,000
$0.20 \le SO_4 \le 2.00$	Severe	S2	V	0.45	4,500
$SO_4 > 2.00$	Very Severe	S3	V plus pozzolan or slag cement	0.45	4,500

2. Modified from ACI 318-14 Table 19.3.1.1 and Table 19.3.2.1





SAMPLE ID: B-2 at 6 to 6½ Feet

VERY OLD PARALIC DEPOSITS (Qvop): CLAYEY SANDSTONE

NOTES: Insitu

Strain Rate: 0.003 in/min

Sample was consolidated and drained

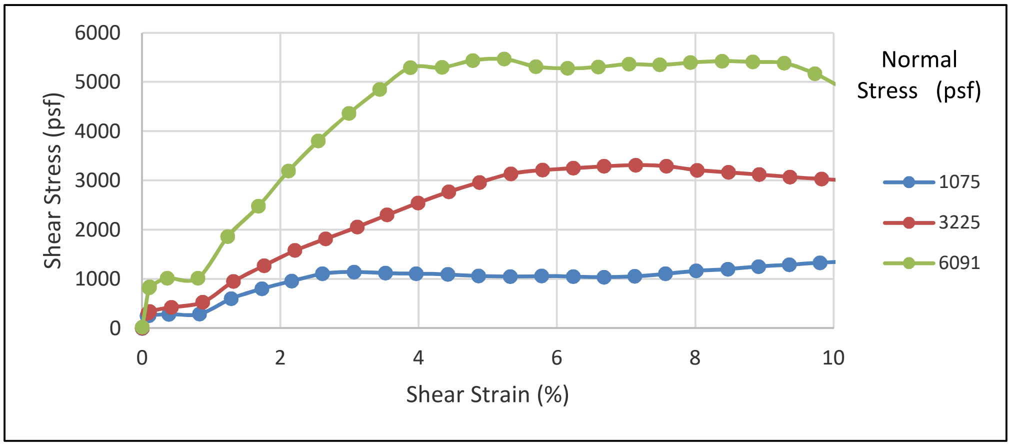
	Peak	Ultimate
Φ	37 °	37 °
c	485 psf	470 psf

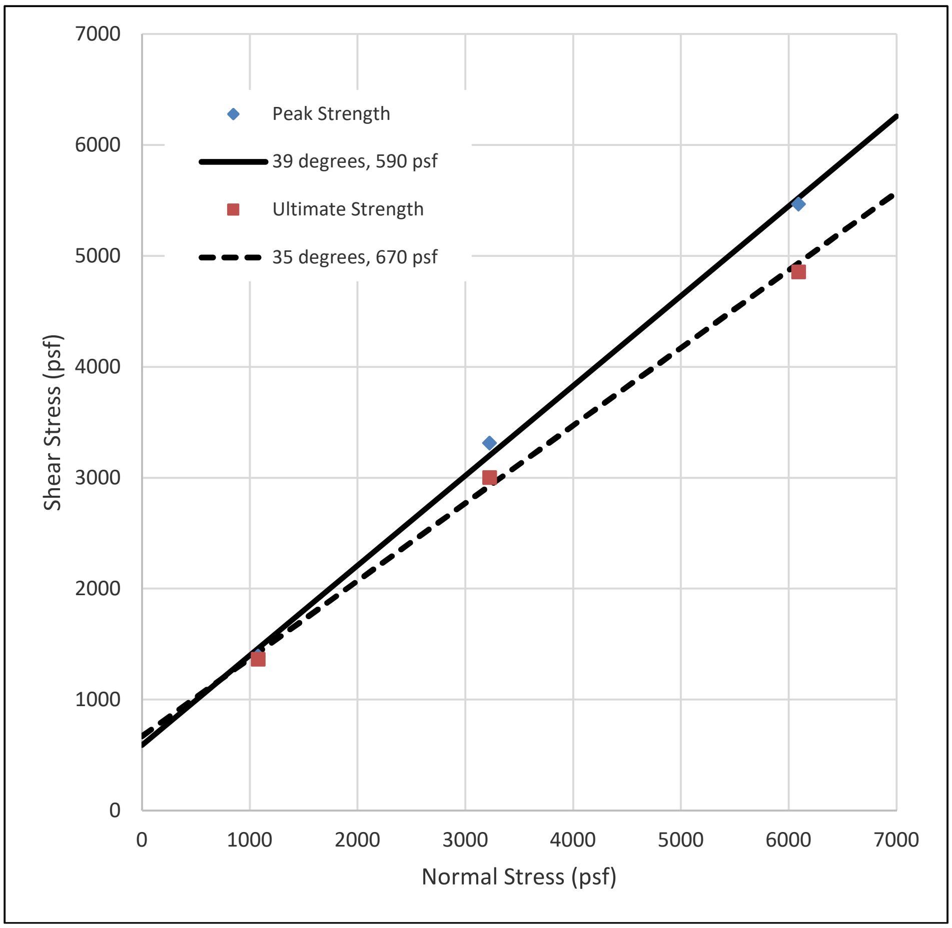
	Initial	Final
γ_d	120.8 pcf	120.8 pcf
w_c	5.9 %	13.8 %
Saturation	41 %	96 %

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements
San Diego, California

By:	PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	II-4





SAMPLE ID: B-2 at 19½ to 20 Feet

CABRILLO FORMATION (Kcs): CLAYSTONE

NOTES: Insitu
Strain Rate: 0.003 in/min
Sample was consolidated and drained

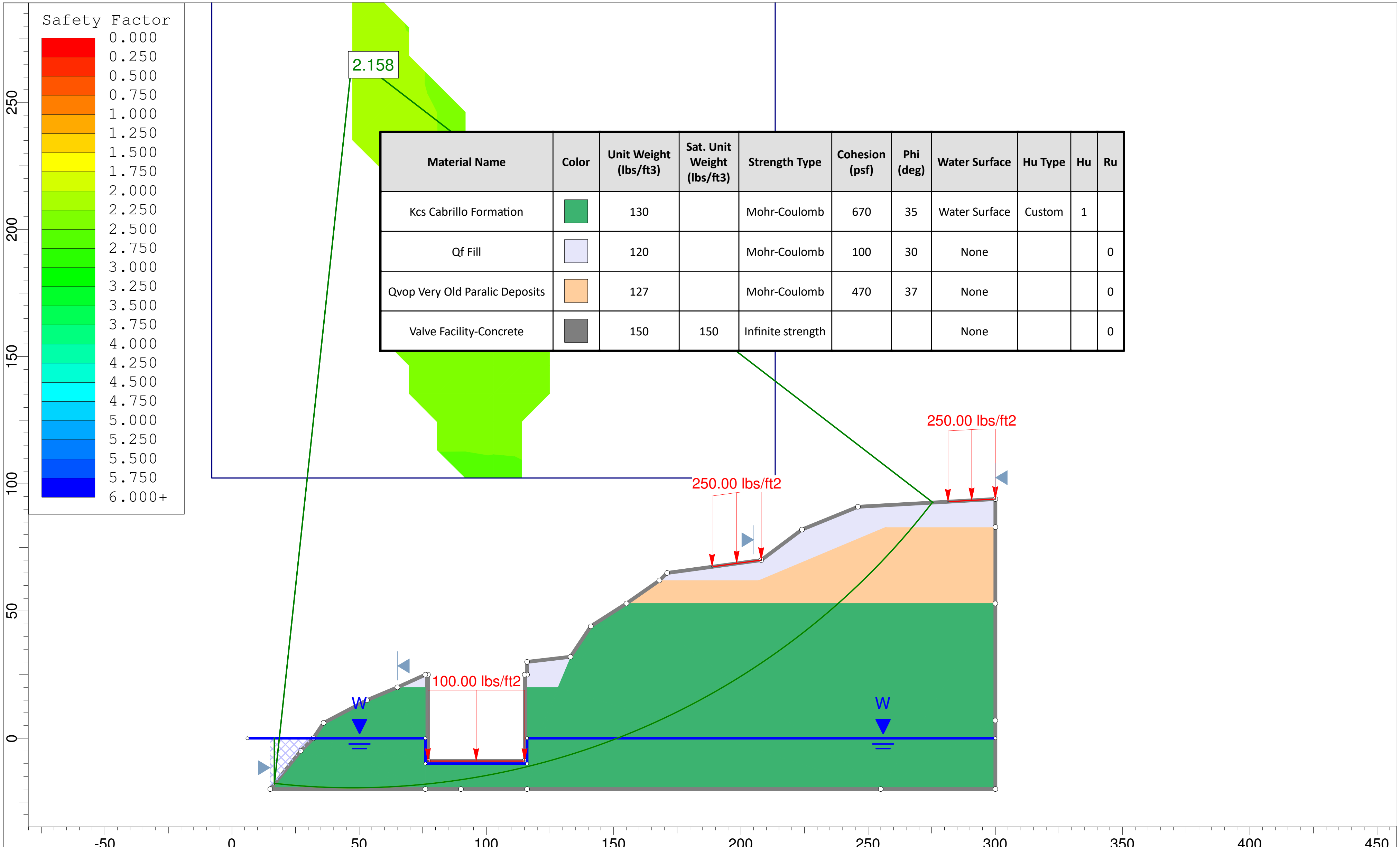
	Peak	Ultimate
Φ	39 °	35 °
c	590 psf	670 psf

	Initial	Final
γ_d	112.2 pcf	112.2 pcf
w_c	16.0 %	20.9 %
Saturation	88 %	100 %

ATLAS

Point Loma Wastewater Treatment Plant Site Improvements San Diego, California			
By:	PFL	Date:	September, 2020
Job Number:	190329P4.3-1	Figure:	II-5

APPENDIX III
SLOPE STABILITY ANALYSIS

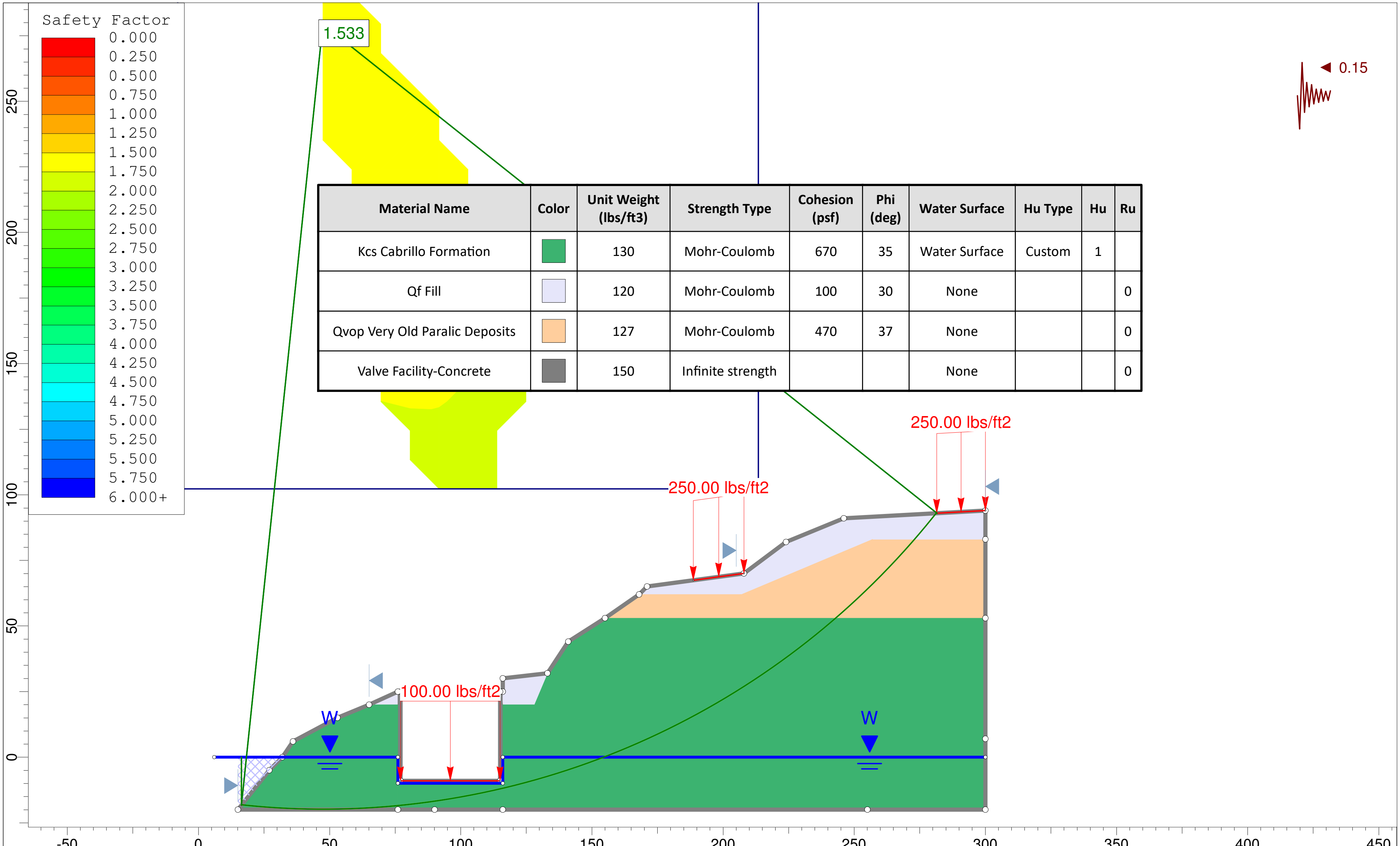


Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Kcs Cabrillo Formation		130		Mohr-Coulomb	670	35	Water Surface	Custom	1	
Qf Fill		120		Mohr-Coulomb	100	30	None			0
Qvop Very Old Paralic Deposits		127		Mohr-Coulomb	470	37	None			0
Valve Facility-Concrete		150	150	Infinite strength			None			0

ATLAS

Cross Section A-A': Global

Project: Point Loma Wastewater Treatment Plant	Location: San Diego, California
Analysis Method: Spencer Method - Static Analysis	Client: HDR
Date: September, 2020 / Analysis By: THC	Job No.: 190329P4.3 / Figure No.: III-1

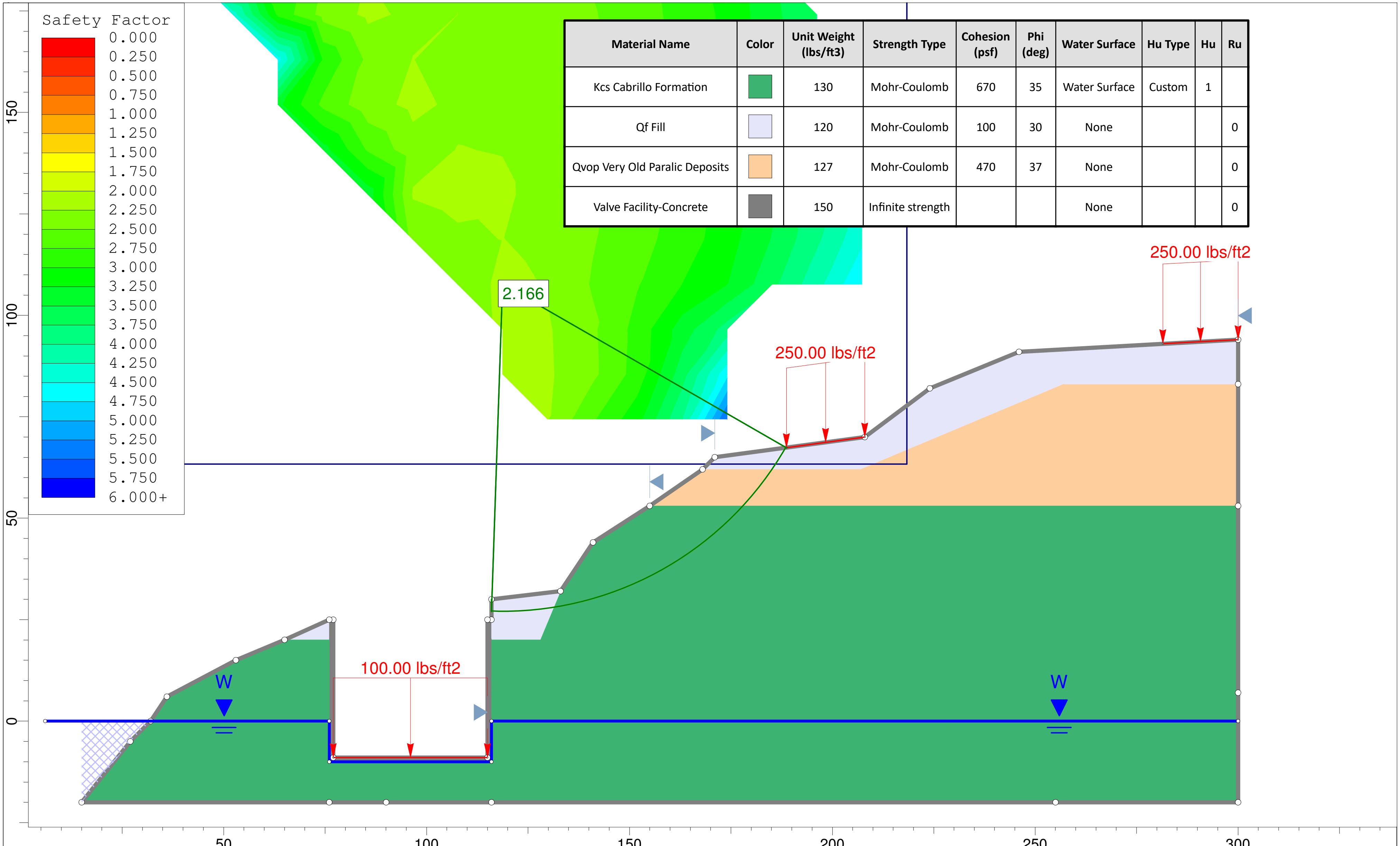


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Kcs Cabrillo Formation		130	Mohr-Coulomb	670	35	Water Surface	Custom	1	
Qf Fill		120	Mohr-Coulomb	100	30	None			0
Qvop Very Old Paralic Deposits		127	Mohr-Coulomb	470	37	None			0
Valve Facility-Concrete		150	Infinite strength			None			0

ATLAS

Cross Section A-A': Global

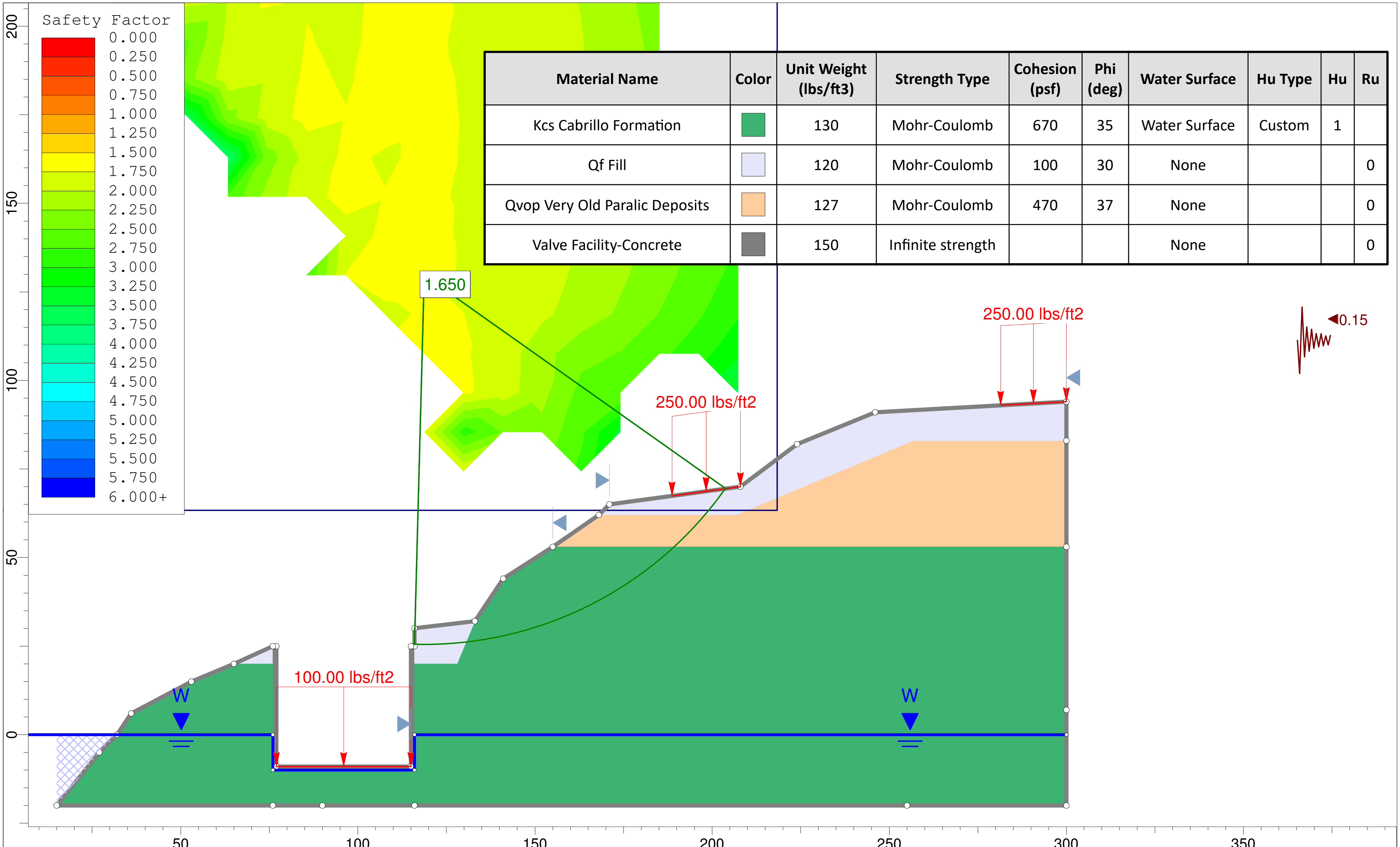
Project: Point Loma Wastewater Treatment Plant	Location: San Diego, California
Analysis Method: Spencer Method - Pseudostatic Analysis	Client: HDR
Date: September, 2020 / Analysis By: THC	Job No.: 190329P4.3 / Figure No.: III-2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Kcs Cabrillo Formation		130	Mohr-Coulomb	670	35	Water Surface	Custom	1	
Qf Fill		120	Mohr-Coulomb	100	30	None			0
Qvop Very Old Paralic Deposits		127	Mohr-Coulomb	470	37	None			0
Valve Facility-Concrete		150	Infinite strength			None			0

Cross Section A-A': Local 1

Project: Point Loma Wastewater Treatment Plant	Location: San Diego, California
Analysis Method: Spencer Method - Static Analysis	Client: HDR
Date: September, 2020 / Analysis By: THC	Job No.: 190329P4.3 / Figure No.: III-3

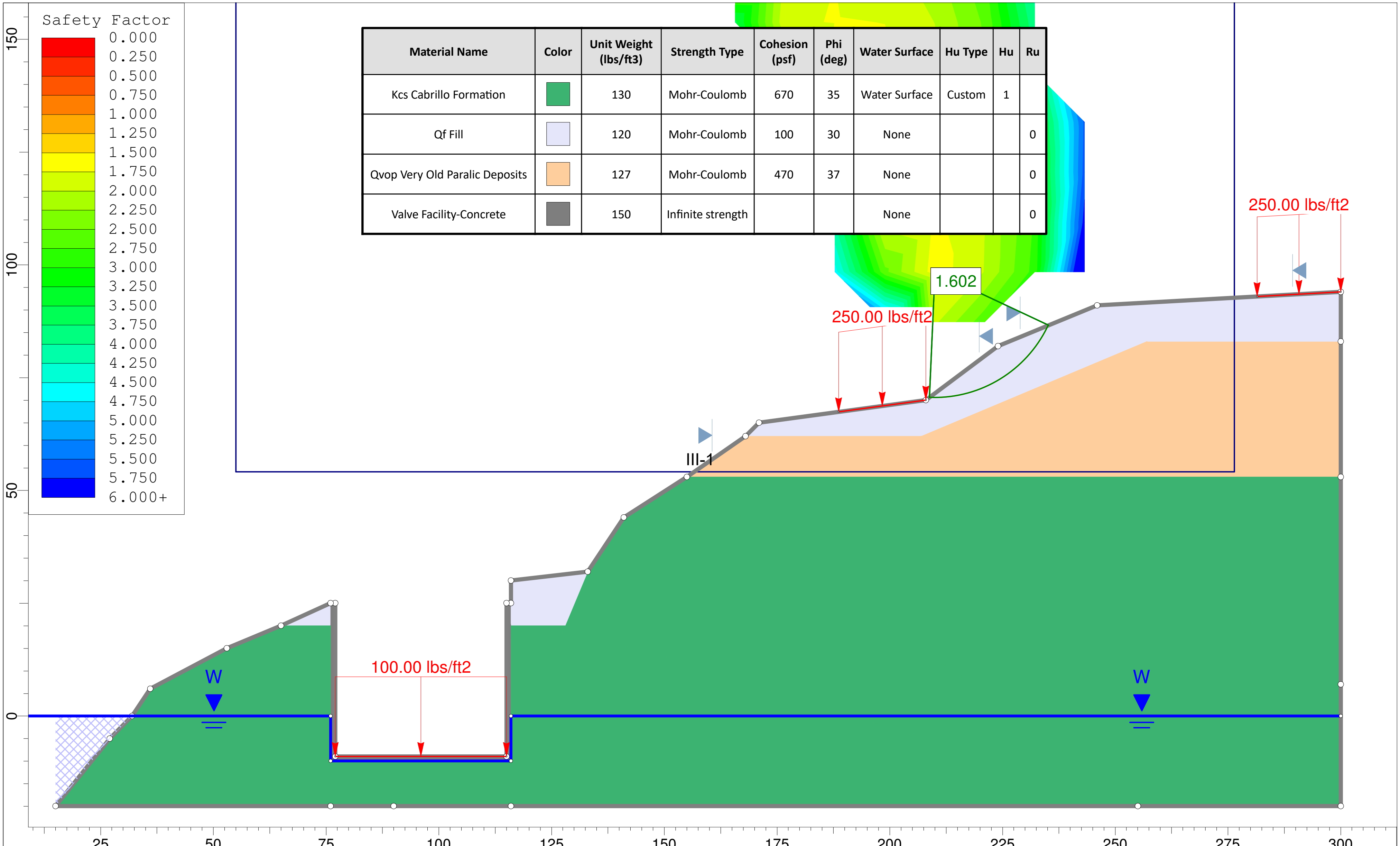


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Kcs Cabrillo Formation		130	Mohr-Coulomb	670	35	Water Surface	Custom	1	
Qf Fill		120	Mohr-Coulomb	100	30	None			0
Qvop Very Old Paralic Deposits		127	Mohr-Coulomb	470	37	None			0
Valve Facility-Concrete		150	Infinite strength			None			0

ATLAS

Cross Section A-A': Local 1

Project: Point Loma Wastewater Treatment Plant	Location: San Diego, California
Analysis Method: Spencer Method - Pseudostatic Analysis	Client: HDR
Date: September, 2020 / Analysis By: THC	Job No.: 190329P4.3 / Figure No.: III-4

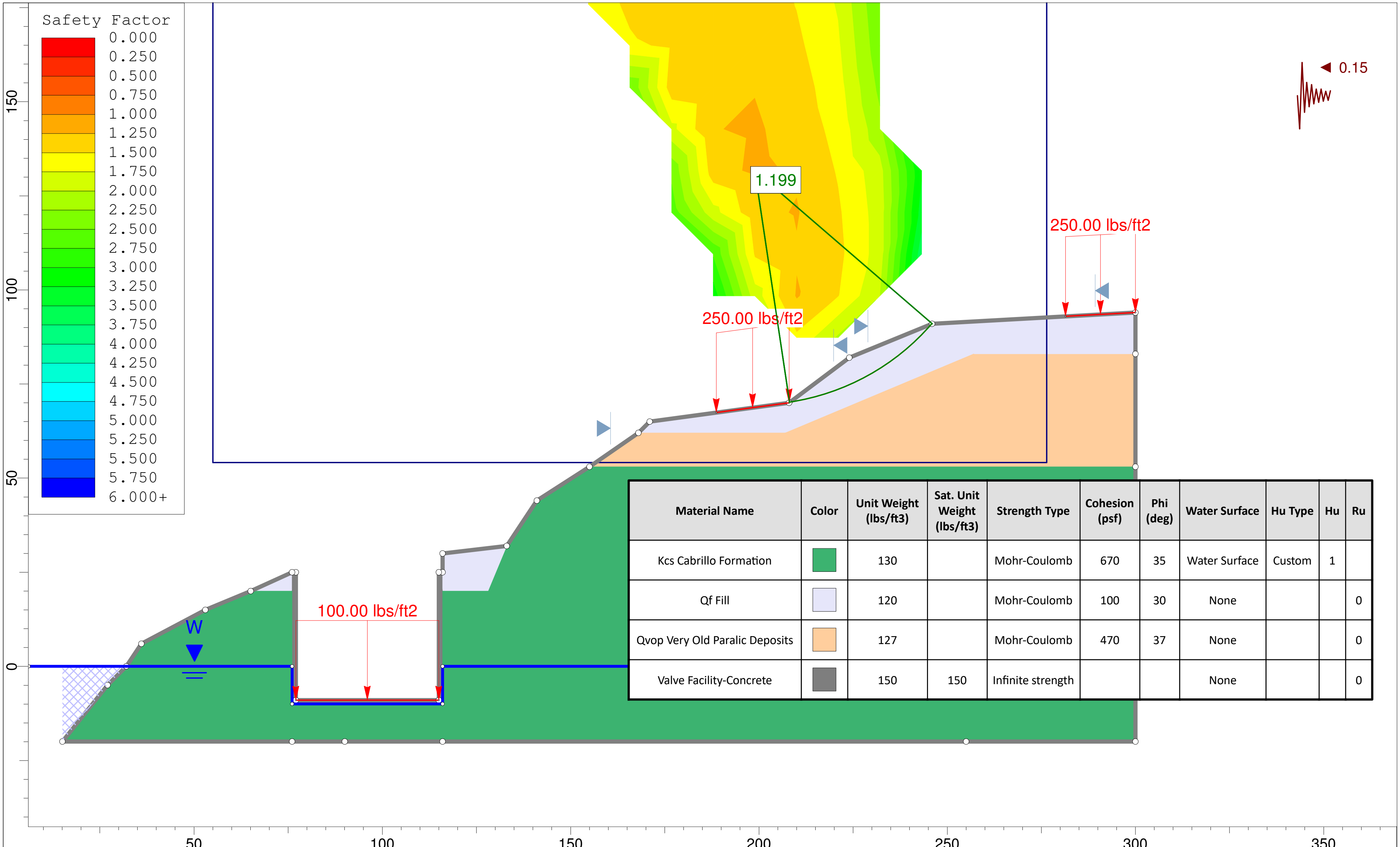


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Kcs Cabrillo Formation		130	Mohr-Coulomb	670	35	Water Surface	Custom	1	
Qf Fill		120	Mohr-Coulomb	100	30	None			0
Qvop Very Old Paralic Deposits		127	Mohr-Coulomb	470	37	None			0
Valve Facility-Concrete		150	Infinite strength			None			0

ATLAS

Cross Section A-A': Local 2

Project: Point Loma Wastewater Treatment Plant	Location: San Diego, California
Analysis Method: Spencer Method - Static Analysis	Client: HDR
Date: September, 2020 — Analysis By: THC	Job No.: 190329P4.3 — Figure No.: III-5



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Kcs Cabrillo Formation		130		Mohr-Coulomb	670	35	Water Surface	Custom	1	
Qf Fill		120		Mohr-Coulomb	100	30	None			0
Qvop Very Old Paralic Deposits		127		Mohr-Coulomb	470	37	None			0
Valve Facility-Concrete		150	150	Infinite strength			None			0

Cross Section A-A': Local 2

Project: Point Loma Wastewater Treatment Plant

Location: San Diego, California

Analysis Method: Spencer Method - Pseudostatic Analysis

Client: HDR

Date: September, 2020

Analysis By: THC

Job No.: 190329P4.3

Figure No.: III-6