Del Mar Bluffs Geotechnical Study

Part 1: Geotechnical Evaluation
Volume I

prepared for

NCTD
North County Transit District

prepared by

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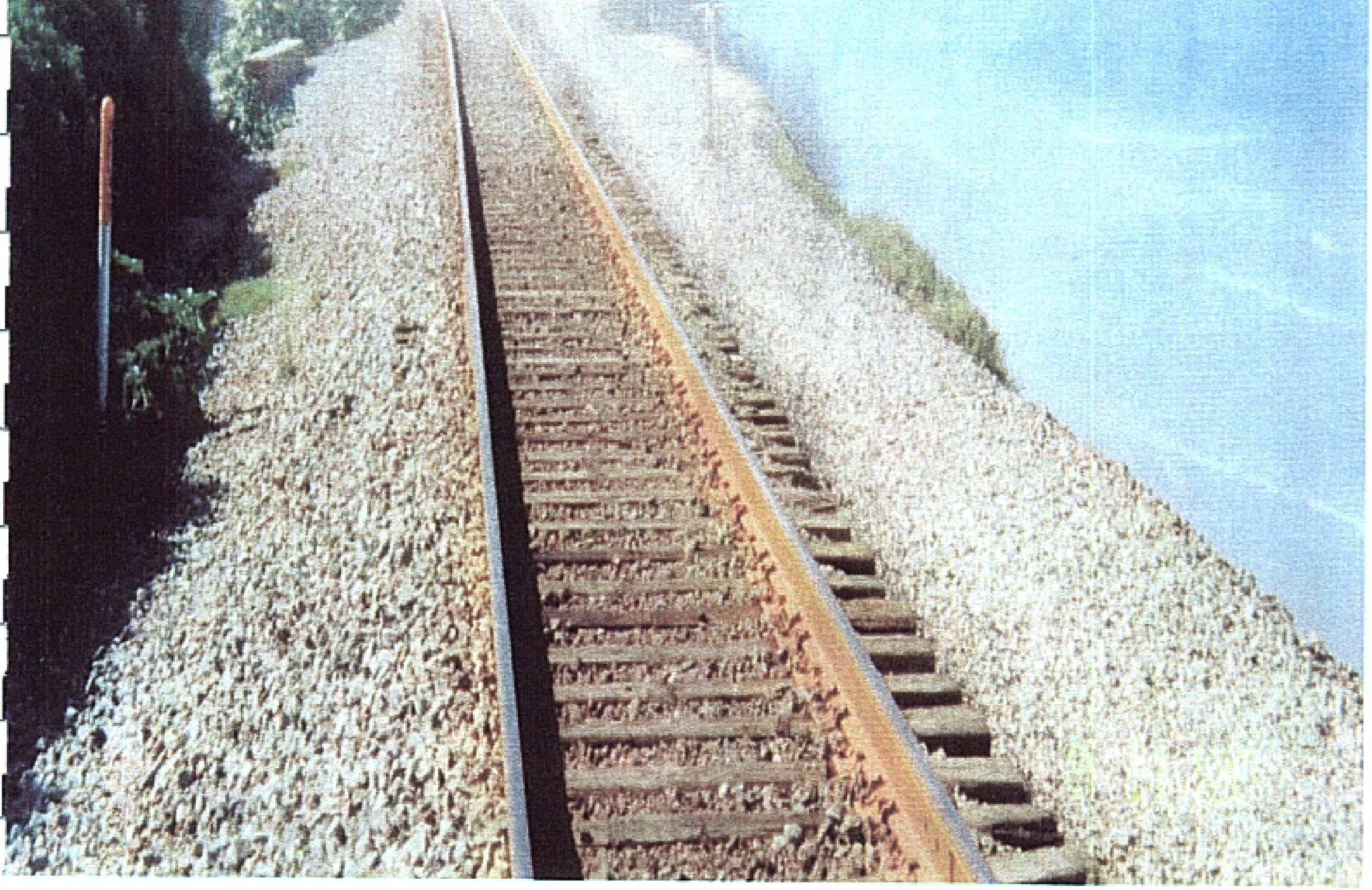
January 2001



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Leighton and Associates

GEOTECHNICAL CONSULTANTS

DEL MAR BLUFFS GEOTECHNICAL STUDY,
DEL MAR, CALIFORNIA,
PART I -VOLUME NO. 1
GEOTECHNICAL EVALUATION

January 11, 2001

(Revised January 31, 2001)

Project No. 040151-001

Prepared For:

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Celebrating 40 Years
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January 11, 2001
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Project No. 040151-001

To: North County Transit District
810 Mission Avenue
Oceanside, California 92054

Attention: Ms. Leslie Blanda, Manager of Capital Development

Subject: Del Mar Bluffs Geotechnical Study, Del Mar, California,
Part I - Volume No. 1 – Geotechnical Evaluation

In accordance with your request and in conjunction with HDR Engineering Incorporated, this report presents the results of our geotechnical evaluation of the Del Mar Bluffs between Milepost 244.1 and Milepost 245.7 in the City of Del Mar, California. The purpose of our study was to perform a general geotechnical evaluation of the coastal bluffs that provide support for the North County Transit District (NCTD) Rail Alignment. The following report provides a summary of the geotechnical conditions observed along the alignment, an evaluation of the bluff retreat processes affecting the alignment, and provides our conclusions with regard to preserving the bluffs so that with consideration of the geotechnical issues, the railroad right-of-way can be maintained for at least 20 years.

If you have any questions regarding this report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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PART 1 – VOLUME II

1.0 EXECUTIVE SUMMARY

In accordance with the direction of HDR Engineering and NCTD, this report presents a summary of our geotechnical evaluation of the NCTD rail alignment located atop the Del Mar Bluffs between Milepost (MP) 244.1 and MP 245.7 in the City of Del Mar, California. The study was performed so that we could prioritize bluff preservation initiatives to maintain the railroad right-of-way on the Del Mar Bluff for at least another 20 years. This was done by consideration of geotechnical and coastal processes anticipated to affect the erosion and bluff stability both now and over the next 20-year period.

As part of our evaluation, we reviewed previous documents and reports, geologically mapped the site, conducted a subsurface investigation of the site geologic conditions, mapped the site geology, performed slope stability analysis, and prepared this summary report.

In order to prioritize preservation initiatives, a matrix has been prepared to assess the risk to the rail right-of-way from geotechnical and coastal phenomena. The major factors influencing the bluff stability and risk include the presence of existing failures, coastal erosion, bluff retreat, ground water seepage, drainage issues, and the close proximity of the tracks to the bluff top. This matrix was prepared to reflect current conditions and then redone with some assumptions for anticipated risks in the year 2020.

In general, the on-going degradation of the bluff that supports the NCTD alignment is due to a combination of factors. In certain areas, individual factors carry more importance. For example, ground water seepage and poor surface drainage likely accelerate deterioration of the bluff in several areas, whereas, in other areas the lack of lateral support of the tracks is a more significant concern. Also, wooden seawalls along portions of the bluff are currently protecting portions of the base of the bluff against erosion due to typical wave impact. However, these walls are of insufficient height to block heavy storm surf and appear to be poorly maintained. As a result, although they are not actively eroding, these areas are also at a high risk.

Our analysis of stability identifies that increased factor of safety is obtained under completely drained conditions. Measures to limit the potential for ground water accumulation are necessary throughout the bluff. In addition, our analysis indicates that portions of the bluff may be subject to failure during a major seismic event on one of the active regional faults. This is particularly true for areas of the bluff that are near saturation from either prolonged periods of heavy rainfall or in areas of heavy ground water accumulation.

Even under drained conditions, segments along the bluff are judged to possess factors of safety less than 1.5 to resist local and global instability under static conditions. Efforts to provide additional lateral support and protect the existing support are necessary.

The following table provides a summary of the most critical issues identified during our study along with the priority level assigned for the implementation of preservation alternatives.

1. In terms of Drainage Improvements

Priority	Location
High	8th Street and 11th Street to Coast Boulevard
Medium	10th to 11th, 5th to 7th Streets
Low	Southern Section

2. In terms of Bluff Toe Protection

Priority	Location
High	7th to 8th Street and 11th to MP 244.2
Medium	MP 245.16 to MP 245.37
Low	Center Section, High Cliffs

3. In terms of Bluff Stability Improvement (Static Conditions)

Priority	Location
High	7th to 8th Street and 11th to MP 244.2
Medium	5th to 6th Street
Low	High Cliffs and South Section

4. In terms of Bluff Stability Improvement (Hydrostatic and Seismic Conditions)

Priority	Location
High	All Areas

The risk assessment area points out on-going maintenance or repair is critical where the bluffs are locally unstable in their present condition or where existing improvements or protective structures are already present. In addition, improvements to mitigate effects of heavy storm surf and reduce the saturation of the bluffs should be implemented. Conceptual preservation alternatives are presented for consideration in Part 2 of the Del Mar Bluff Study.

In finalizing this report review comments by NCTD, HDR Engineering, Caltrans, SEQAD Consulting Engineers (Dr. Scott A. Ashford and Dr. Frieder Seible), and Coastal Environments (Dr. Hany Elwany and Dr. Jeffery Johnson), have been considered and incorporated in portions of the report.

2.0 PURPOSE AND SCOPE

This report has been prepared in accordance with the direction of NCTD and HDR Engineering, Inc. and presents the results of our geotechnical evaluation of the Del Mar Bluffs. The study area is located along the San Diego Northern Railroad (SDNR), which is owned and operated by NCTD. More specifically, our study area is between MP 244.1 and MP 245.7 along the western boundary of the City of Del Mar, California. The site can also be referred to as being located along the bluff south of Coast Boulevard and north of Torrey Pines State Beach (Figure 1). Our primary task was to review a report previously prepared by Medall, Aragon, and Higley (MAH, 1998), perform additional investigation, and prepare the geotechnical evaluation report. The purpose of the geotechnical evaluation was to evaluate the pertinent geotechnical conditions in order to prioritize bluff preservation initiatives that may be required to maintain the railroad right-of-way on the Del Mar Bluffs for at least another 20 years. This was done by consideration of geotechnical and coastal processes effecting the erosion and the stability of the bluffs now and as anticipated during the next 20 years.

Our scope of services included:

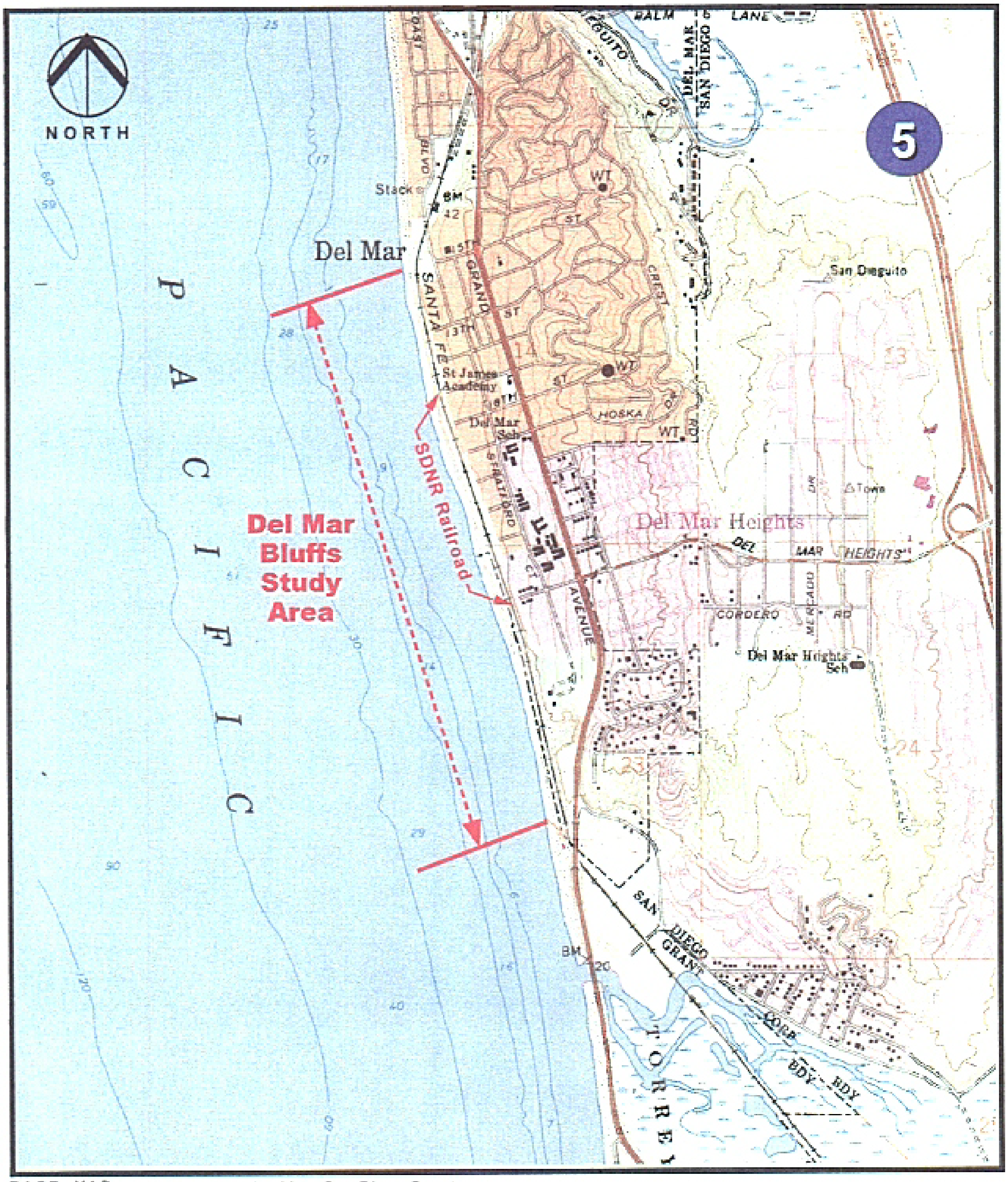
- Review of pertinent available geotechnical literature, previous geotechnical reports, geologic maps, aerial photographs (Appendix A), and historical information provided by NCTD (Appendix H).
- Meetings with NCTD and City of Del Mar representatives.
- Meetings with Project Reviewers.
- Reconnaissance and geologic mapping of the site and photo documentation of the existing site conditions. Photos of selected site locations are included as Appendix B.
- Review of a series of historical aerial photographs. Strip photos documenting the condition of the bluff in 1999 are also included as Photo Nos. 33 to 43 in Appendix B.
- A subsurface exploration program consisting of the excavation, sampling and logging of six large-diameter exploratory borings. The large-diameter borings were excavated to evaluate the characteristics of the bedrock materials on site, possible areas of landsliding, and site ground water levels. Logs of the borings are presented in Appendix D. We have also utilized and included the logs of borings previously excavated by both Leighton and Associates and by others for preparation of this report (Appendix D).
- Laboratory testing of representative samples obtained during our subsurface exploration (Appendix F). Previous laboratory testing by Leighton and by others was also considered in our evaluation and is incorporated as part of Appendix F.
- Preparation of geologic maps and seventeen geologic cross-sections in representative areas throughout the study area. Cross-Sections A-A' through V-V' are presented as Figures 8 through 20 at the rear of the text. The approximate locations of the geologic cross-sections are shown on the Site Plan and Geologic Maps, Plates 1 through 5.

- Geotechnical analysis of the data accumulated during our preliminary investigation (including slope stability analyses) and evaluation of the data collected during the previous site investigations (Appendix G).

- Evaluation of coastal bluff retreat processes and rates. This evaluation was prepared utilizing the geologic mapping of the bluffs, reviews of historical aerial photographs, interviews with local residents, and consultations with representatives of the coastal engineering firm Noble Consultants. Oceanographic data is presented as Appendix E.

- Preparation of a matrix (Table 10, Appendix C) to evaluate the risks to the railroad based on current conditions. This matrix was then utilized to identify the area of highest risk in order to prioritize preservation initiatives. In addition, Table 11, Appendix C presents a risk matrix for bluff conditions in the year 2020 assuming on-going erosion continues at or near present annualized rates.

- Preparation of this report presenting our preliminary findings and conclusions regarding the stability of the bluffs that support the rail alignment and a general summary of the factors that contribute to instability of that support. The approximate limits of the geologic units encountered during surface mapping and the boring and trench locations of our investigation and the previous site investigation are presented on the Site Plan and Geologic Maps (Plates 1 through 5).



BASE MAP: TOPO! Interactive Map, San Diego County

1"=2,000'

0 1000 2000 4000

Scale in Feet

Del Mar Bluffs Del Mar, California	SITE LOCATION MAP	Project No. 040151-001 Date 1-7-01 Figure No. 1

2.1 Site Description and History

The study areas consist of approximately 1.6 miles of coastal bluff along the western side of the City of Del Mar in the southern portion of the city. The study area extends from near Coast Boulevard at MP 244.1 south to MP 245.7. The railroad is located atop a coastal bluff with elevations in the study areas that increase from 41 feet at Coast Boulevard to 66.9 feet at MP 245 then descends to elevation 37 feet as the track goes under the Highway 101 bridge in the south. In 1880, construction began on the California Southern Railroad from Oceanside to San Diego. Construction of the railway track through Del Mar commenced in January 1881 and the tracks were originally aligned along Stratford Court. Operation of the railroad through Del Mar began in 1883 (see Figure No. 4). The tracks were relocated onto the Bluffs in 1910 to accommodate the development of the City of Del Mar and to reduce the track grade. In order to construct this new alignment, the area proposed for the train was leveled by infilling major canyon areas and excavated to the design grades by use of dynamite and a large steam shovel (see Figure No. 5). The present alignment can be seen in the attached strip photos included in Appendix B as Photo Nos. 33 to 43.

Since 1910, the bluffs have been naturally eroding and subjected to increased surface water runoff and increased ground water seepage. The urban residential development of Del Mar increases the surface water runoff and concentrates it into storm water systems that traverse the rail right-of-way. In 1938, Del Mar had 430 residents, today there are approximately 5,000 residents. As population increases, storm runoff water and irrigation water increase and, as a result of infiltration, ground water increases. In addition to surficial failures present on the bluff face, landslides that threatened rail activity occurred in 1941 (when a train was derailed), 1958, 1961, 1962, and 1978. The approximate limits of these past failures are indicated on the geotechnical maps (Plates 1 through 5). It should be noted that all of these past failures occurred in mid-winter typically after periods of increased rainfall.

Since construction of the railroad, dams have been built across all the major river systems in San Diego County that provide sediment to the beaches. With the construction of Lake Hodges in 1919, the effective sediment producing area of the San Diequito River watershed was reduced from 346 square miles to 43 square miles. This has resulted in the loss of major sediment supply for the Del Mar Beach and Bluffs area. We can conclude that the beach widths generally been reduced since 1910 when the railroad was placed on the bluffs. The reduced natural protective sand barrier will increase the risk of future storm damage to the bluffs. Mitigative measures implemented to date have included; several wooden and concrete sea walls, earthern buttresses and slope fills, improvements of the site drainage by installing a storm drain system, subdrains, and both lined and earthen drainage swales. Localized slope failures have also necessitated the construction of additional sea walls, installation of soldier pile retention systems and the most recent repair which was a sand-cement buttress constructed in 1996.

Topographically, the rail alignment is constructed across the now relatively flat mesa top. For a majority of the alignment, the track is at or slightly above the adjacent seaward bluff top elevation. However, in the central portion of the site the tracks are located in an excavated slot approximately 20 feet lower than the adjacent bluff top. The tracks themselves are situated from ±80 feet to within 10 feet of the top of the bluff. The relation of the track to the bluff can be seen on the Plates 1 through 5, Site Plan and Geologic Maps, where the "top and toe of bluff" has been plotted. A deeply incised drainage (Anderson Canyon) is also present in the southern portion of the site. Anderson Canyon has been infilled with roughly 50 feet of fill across the railroad alignment.

Site drainage is presently accomplished through a network of storm drains, the majority of which now extend down the bluff to the beach, although there are some locations where storm drains discharge near the bluff top. Previously, several major storm drains discharged near the bluff top and significant erosion had occurred. A concrete drainage ditch has been constructed in the southern portion of the alignment to intercept drainage from the east and direct this runoff to the beach. Surface flow west of the tracks is still generally controlled by surface flow that drains over the top of the bluff or ponds alongside the tracks to dissipate by evaporation and infiltration . Localized areas of concentrated runoff have also created several erosional gullies that extend down the bluff (see Photo Nos. 13 and 14, Appendix B). In several localized areas along the bluff top, a series of subsurface drains have been installed most of which are thought to remain functioning. Where known to be present, these drains are shown on the geotechnical maps. Several of the outlets can also be seen discharging on the bluff face. Areas of discharge can be seen on Photo No. 34, adjacent to the south side of the concrete frame (MP 244.4) and also on the bluff in Photo No. 25 (MP 244.97).

Pedestrian traffic is heavy on the bluff top and several trails are present extending down the bluff face. In general, foot traffic is confined to these localized areas and is not widespread across the bluff face. Evidence of rodent activity on the buff face was also observed but does not appear to be a widespread problem.

2.2 Subsurface Investigation and Laboratory Testing

Our subsurface investigation consisted of the excavation, logging and sampling of six large-diameter borings to depths between 60 and 70 feet utilizing a bucket-auger drill rig. The large-diameter borings were entered by our geologist and downhole logged. All borings were reviewed by a State of California Certified Engineering Geologist. Subsequent to the subsurface investigation, the borings were backfilled, tamped and capped with a concrete slurry. The results of previous geotechnical studies have also been incorporated into this report where appropriate. Logs of all the borings both current and from previous studies are included in Appendix C. The approximate locations of the borings (current and previous) are presented on the Site Plans and Geologic Maps (Plate 1 through 5). During the drilling operations, bulk and relatively undisturbed samples were obtained from the borings for laboratory testing and evaluation. Drive samples were utilized to collect samples during drilling operations. The drill rig Kelly bar was utilized as the driving weight for sampling. The Kelly bar weighed approximately 4,991 pounds for samples taken between the ground surface and a depth of 30 feet. For deeper samples, the weight of the Kelly bar was reduced depending on the depth due to use of telescoping methods. Actual drive weights are shown on the boring logs. The relatively undisturbed in-place samples were obtained utilizing a modified California drive sampler driven by dropping the Kelly bar 12 inches. The modified California sampler has an outside diameter of approximately 3 inches and inside diameter of 2.4 inches. Brass rings, 1-inch in height lined the sampler for "undisturbed" sample collection. The blow counts required to drive the sampler each successive 6 inches was recorded and the final 12 inches of measured blow counts is presented on the boring logs.

Laboratory testing was performed on representative soil samples obtained during our subsurface investigation. The laboratory tests included moisture/density determinations and direct shear. A discussion of the tests performed and a summary of the results are presented in Appendix F. The moisture/density determinations of the "undisturbed" ring samples obtained from the borings are shown on the boring logs (Appendix D). Also included in Appendix F are test previously performed by Leighton and by others. The previous laboratory testing has included moisture/density, direct shear, sieve and hydrometer analysis, compressibility, laboratory compaction, and Atterberg limits.

3.0 GEOTECHNICAL CONDITIONS

3.1 Regional Geology

The site is situated in the coastal section of the Peninsular Range province, a California Geomorphic province with a long and active geologic history throughout Southern California. Throughout the last 54 million years, the area known as the "San Diego Embayment" has undergone several episodes of marine inundation and subsequent, marine regression. This has resulted in a thick sequence of marine and nonmarine sedimentary deposits on rocks of the Southern California batholith with relatively minor tectonic uplift of the area.

The sedimentary rocks in the area include the Quaternary-aged Bay Point Formation and the Eocene-aged Delmar Formation and Torrey Sandstone. Subsequent to the deposition of these units, erosion and regional tectonic uplift created the valleys and ridges of the area. Human influences, recent weathering and erosional processes have produced the recent surficial units including fill soils, landslide deposits, and beach deposits which mantle the site.

3.2 Site-Specific Geology

Specifically, the site is underlain by (from youngest to oldest) undocumented fill soils, Quaternary landslide deposits, Quaternary beach deposits, and formational materials of the Quaternary-aged Bay Point Formation and the Eocene-aged Delmar Formation. The approximate areal limits of each of the geologic units are indicated on the Site Plan and Geologic Map (Plates 1 through 5). Our interpretation of the subsurface geologic conditions is also indicated on Geologic Cross Sections A-A' through V-V' (Figures 8 through 20). The geologic units present on the site are described below (youngest to oldest). Although not present within the alignment, the Eocene-aged Torrey Sandstone can be observed just east of the tracks in the southern portion of the site and within Anderson Canyon.

3.2.1 Artificial Fill Soils (Map Symbol – Af)

Artificial fill soils were observed in a number of places on the site. As observed, the fill soils were generally associated with the grading for the railroad alignment and include canyon fills (Anderson Canyon), subdrain and storm drain backfill, and minor fills placed to reach appropriate grades for the existing railway. Fills associated with previous slope repairs are also present on the site. In localized areas, minor amounts of loose fill are present on the site. These loose fills generally appear to be surficial in nature and not of significant extent. In the northern section of the bluff MP 244.3 to MP 244.45 it appears that soil has been "air dumped" from the train tracks at several different time periods (1940 and ±1990, see Figure No. 7), a majority of this loose soil has been eroded off the bluff face and only minor amounts remain. In addition, a sand-cement buttress fill covered by fill materials is present in the proximity of MP 244.45. The significant areas of fill are shown on the Site Plan and Geologic Map (Plates 1 through 5). Based on the site topography, these fill soils are estimated to range from 1 to 45+ feet in depth.

Fill materials are believed to be generally comprised of materials derived from the Bay Point and Delmar Formations. Gravel can be found within subdrains, bedding zones, and as ballast adjacent to track areas.

3.2.2 Landslide Deposits (Map Symbol-Qls)

Several areas of landslide deposits have been identified on the face of the bluff. Landslides documented by the railroad occurred in 1941, 1958, 1961, 1962, 1966, 1973, 1975, 1976 and 1978. The approximate limits of these landslide deposits are shown on the Site Plan and Geologic Maps (Plates 1 through 5) and many of the landslides are also depicted on the Geologic Cross-Sections A-A' through V-V' (Figures 8 through 20).

The landslide deposits generally appear to be shallow surficial failures and are composed of disturbed and relatively undisturbed blocks of formational material and weathered formational material surrounded by a loose matrix consisting of soils characteristic of the Bay Point and Delmar Formations. The majority of these failures occurred during winter months when soil moisture contents are somewhat higher due to rainfall. The material is generally moderately fractured, jointed, and highly weathered near the surface and at the toe of the failure areas. Many of the observed slope failures appear to be the result of failures on the upper portions of the bluff and are now deposited as debris along the base of the slope (see Photo Nos. 23 and 29, Appendix B). In addition, many of the failures appear to be directly related to areas of ground water seepage. The majority of the failures observed appear to be occurring within a weathering profile within the face of the bluff. As the bluffs retreat weathering occurs, joints and fractures develop near and parallel to the bluff face. As these features become more well developed, the surficial failures occur. The frequency of well-developed joint sets diminishes with distance away from the bluff face.

3.2.3 Quaternary Beach Deposits (Map Symbol – Qb)

Quaternary Beach Deposits consisting of varying thicknesses of beach sand were encountered at the base of the bluff, on the west side of the site. These soils generally consisted of light gray, damp, loose, fine to medium sand. Sand levels vary seasonally with generally lower sand levels in winter and higher sand levels in summer. During the time over which our study was performed, sand levels fluctuated by as much as 1 to 2 feet.

3.2.4 Bay Point Formation (Map Symbol-Qbp)

The Quaternary-aged Bay Point Formation underlies the SDNR alignment throughout a majority of the study area. These terrace materials were deposited on a now elevated, relatively level wave cut platform and overlie the Delmar Formation. As encountered during our investigation this material generally consisted of red-brown, moist, loose (near the ground surface) to dense, fine to medium silty to clayey sands and sandstone with lesser amounts of clayey sandstone. This material is generally very permeable and typically, zones of perched water have developed along the contact of these materials with the underlying Delmar Formation (see Photo No. 23). Where exposed at the surface this material is highly erodible. A majority of the terrace deposits were removed in both the northern and southern sections of the alignment during railroad construction creating the flat mesa top observed today. Although of a reddish color, this material is generally considered suitable for use as part of a beach replenishment program because of its sandy nature.

3.2.5 Delmar Formation (Map Symbol-Td)

The Tertiary-aged Delmar Formation was found at depth across the site and underlies the Bay Point Formation generally below an approximate elevation of 45 to 55 feet mean sea level (msl). Surface exposure of this unit can be observed at lower site elevations on the west face of the bluff. As observed in our borings and exposures, this unit consists of dark gray to olive-green, stiff, silty claystone, silty fine sandstone, to clayey siltstone with occasional interbeds of light yellow-brown sand. The contact between the Bay Point Formation and underlying Delmar Formation occurs at approximate elevations between 45 to 55 feet msl. This generally horizontal contact is depicted on both the Geologic Cross-Sections A-A', through V-V' and the Geological Maps. Within the Delmar Formation, we have observed several localized resistant layers. These occur both at mid-bluff heights and most importantly at the base of the bluff. This resistant layer at the base of the bluff creates a barrier to erosion and creates a shallow beach platform within the beach and surf zone areas which works to dissipate wave energy (see Photo Nos. 8 through 12, and 30, Appendix B).

3.3 Geologic Structure

Review of the geologic literature applicable to the site (Appendix A), our professional experience on-sites with similar soils, and geologic mapping of the site, indicate the on-site geologic units are generally flat-lying and massively bedded with occasional cross bedding and randomly oriented jointing. Based on the subsurface data, bedding within the formational materials generally exhibits variable bedding with dips typically horizontal to less than 5 degrees to the west. Localized areas of weak and randomly fractured claystone beds, typically about 1 foot in thickness, were encountered in several of the borings. The randomly fractured claystone beds appear to be the result of consolidation and or tectonic movement following the deposition process and are not considered to be evidence of previous deep-seated slope instability. They do however provide weak layers within the bluff that are more easily weathered and also provide possible failure surfaces for some of the failures that have occurred. In addition, in one localized area at the base of the bluff some minor "notching" was observed. This notching has occurred where a fractured claystone bed overlies a resistant sandstone unit. Jointing on-site is very variable, but predominantly trends sub-parallel to the existing slopes.

Jointing dips were found to be generally moderately to steeply dipping. Jointing was mainly encountered in the upper portion of the bedrock becoming less pronounced with depth. Materials at the bluff face were significantly more weathered, fractured and jointed. Fractures and joints typically parallel the bluff face forming a weathering profile on the bluff face. Based on our site observations and experience on similar sites, this fracturing and jointing is typically concentrated near the bluff face (±outer 10 feet) with significantly less fracturing away from the bluff face.

3.4 Bluff Retreat

Bluff retreat along the Del Mar Bluffs is controlled by a combination of marine erosion and subareal erosion. Marine erosion results from the effects of the ocean and wave action along the base of the bluffs. Subareal erosion results from those erosional influences that exist above the high water line (or wave run-up line) and includes such items as erosion due to surface runoff, ground water seepage, wind, pedestrian traffic, rodent activity, and slope instability.

In general, a bluff will lay back and flatten over time until a characteristic slope is achieved. The generalized stages of bluff retreat by erosion of less resistant materials on the bluff face are illustrated in Figure 8.

For discussion purposes, we have divided the bluffs into Northern, Central, and Southern Sections each of which is discussed below:

3.4.1 Northern Bluff Section (MP 244.1 to MP 244.48)

The northern bluff section extends from the northern end of the study area to approximately the terminus of 11th Street (MP 244.1 to MP 244.48) and consists of a bluff that varies in elevation from 32 to 58 feet msl. In this section, the bluff is in general, a more mature type bluff although it should be noted that in general the rail alignment is much closer to the top of the bluff here than in other sections. In addition, two major areas of fill soils that were the result of either previous slope repairs or the original railroad construction extend to the base of the bluff. The majority of this area has a relatively resistant base of well-indurated bedrock across the base of the bluff and a slightly flatter slope angle. Near vertical sections and areas of surficial slumping are common and some recent failures were observed, the majority of which are the result of failures on the upper portions of the bluff rather that at the base. Numerous areas of perched ground water seepage were observed on the bluff face and areas of ponded water were observed east of the tracks in the northern part of this section. Approximately 625 feet (±30% of the northern bluff section) of the bluffs have some shoreline protection existing at the base of the bluff. This protection consists primarily of short concrete walls and drainage structures, wooden seawall type structures, and rip-rap along the base of the bluff. We note that between MP 244.3 and MP 244.35, the bluff has been the site of several past slope failures. Also it appears that loose fill and between MP 244.41 and MP 244.47 has been "air dumped" over the slope face several times to maintain some cover over the eroded bluff in this area. A short wooden seawall has been constructed across this area to control further erosion. This wall has generally been effective in reducing slope toe erosion; however, the wall is not of sufficient height to counteract the infrequent higher waves that could cause significant additional erosion.

The beach in this area is of moderate width with a relatively thin veneer of sand that mantles a relatively flat beach platform consisting of dense bedrock of the Delmar Formation. This beach platform is easily identified in aerial photographs and verified by wading into the shallow waters at low tide. Sand on the beach varies seasonally. Based on review of aerial photography and conversations with long-term local residents, some sand is virtually always present. Similar beach widths can be observed in historic photos from the 1880's and early 1900's (see Figures 4 and 5). Vegetation on the northern section of the bluff locally extends to within several feet of the base of the bluff in the form of very mature vegetation indicating that this portion of the bluff is not frequently subject to strong wave action (see Photo Nos. 4, 7, and 8, Appendix B).

Surface runoff includes some sheet flow from the west side of the tracks which flows over the top of the bluff and into localized erosional gullies There have been attempts to control surface runoff by swales on the eastern side of the tracks with storm runoff directed to the base of the bluff by a series of pipes. In addition, a wall and subdrain system has been constructed at the top of slope between MP 244.41 and MP 244.46. Ponding was observed in areas on the east side of the tracks and some phreatotypic plants are present (see Photo Nos. 1 and 2, Appendix B). It appears that a subdrain may have been installed in this area in 1941; however, the outlet point and the current effectiveness of this drain system are unknown. Localized areas of heavy seepage can be observed on the bluff face. In general, the bluff is heavily vegetated and although foot traffic along the top of the bluff is heavy, only localized areas of foot traffic extend down the bluff face. In addition, based on our review of the construction drawings for the top of slope wall, it does not appear as though adequate lateral support for the wall foundation is present.

The rail alignment is in general in close proximity to the bluff top with the distance to the bluff edge varying from ±10 to ±80 feet (see Photo No. 5, Appendix B). We also note that review of historic topographic maps of the area (USGS, 1889) included in Appendix H, indicated a topographic depression adjacent to 12th Street. This topographic low could represent natural drainage area in the early stages of formation or as suggested by one of NCTD's reviewers, possibly an area of larger scale ancient landsliding. Geologic mapping on the bluff face indicates several relatively resistant beds extending across the area that are features not indicative of landsliding; however, the area likely consists of more highly weathered soils and areas of increased ground water flow. It appears that in this area the rail alignment may have been constructed across an area of deeply weathered and corresponding weaker soils that are more prone to erosion and bluff failure. Historically, this area has been the location of several past landslides and additional investigation is warranted.

3.4.2 Central Bluff Section (MP 244.48 to MP 244.85)

The central portion of the bluff extends from the terminus of 11th Street (MP 244.48) to roughly 4th Street (MP 244.85). This section can be further divided into a section of high cliffs (MP 244.5 to MP 244.7) and a section of lower cliffs (MP 244.7 to MP 244.85). In this section, the track elevations range from a low of 58 feet msl in the northern portion to a high of approximately 66 feet msl at the southern end of this section. However, from the north end to approximately the end of Little Orphan Alley (MP 244.75) bluff top elevations are between 70 to 78 feet msl. Adjacent to the higher section of the bluff, the rail tracks have been constructed in an excavated slot behind the bluff top. In this area the rail alignment is setback 50 to 90 feet from the bluff top. Geologic mapping of this section of the bluff indicates a much thicker sequence of terrace deposits in the upper portion of the bluff. To both the north and to the south, a thick sequence of the terrace deposits was completely removed along the bluff top as part of the original rail construction. Along the base of the bluff, a relatively resistant bed of Delmar Formation siltstone can be observed (see Photo Nos. 9 and 10, Appendix B). This resistant bed extends several feet above the base of the bluff and will severely limit the marine erosion at the base of the bluff. The base of the bluff is generally near vertical with no significant undercutting or notching noted. On all of the bluff sections observed the only notching observed was a short lateral section with notching approximately 12 to 18 inches in depth.

Surface drainage in the central section includes some minor sheet flow which is directed over the bluff top; however, a majority of the runoff is collected by earthen swales along the track and directed to a storm drain inlet or to localized erosional gullies which discharge to the beach. At approximately MP 244.7 a storm drain which collects runoff from Shipley Lane and 8th Street discharges near the tracks at the bluff top. This discharge, which appears to have a small but constant flow of water, has eroded a large gully through the terrace deposits to the underlying formational material. At this location the rail tracks are within 30 feet of a storm drain headwall that is severely distressed (Photo 31), and past erosion has required construction of a retaining wall to support the tracks. It is our understanding that this storm drain is underdesigned for peak flows (Fraser, 1993).

The shore platform adjacent to the central section of the bluff is slightly higher than adjacent areas to both the north and the south and can easily be observed at low tide and in review of aerial photographs. This beach platform is essentially flat and exhibits very little evidence of downwearing. In the central portion of the bluff, wooden pilings can be observed that are remnants of the 1885 "Natatorium" which was essentially a walled in swimming area along the beach (see Photo Nos. 11 and 12, Appendix B). Review of historical photos of this structure shows beach widths and bluff configurations similar to those that can be observed today (see Figure 5). In addition, resistant beds can be observed on the bluff at nearly identical locations to those observed today. When reviewing this photo, it is important to note that railroad construction removed a majority of the Bay Point Formation terrace deposits in the northern section. Marine erosion in this section of the bluff consists of relatively minor and infrequent wave action against the base of the bluff. Wave energy is quickly dissipated by the relatively high shore platform that forces most waves to break farther offshore. In addition, the majority of the year there is a sand berm against the resistant bluff base. This sand tends to migrate seasonally with a larger sand berm present in the summer months. While some waves do reach the base of the bluff, these waves have deposited most of their energy in the offshore zone. As such, the majority of the waves to reach the base of the bluff typically lack energy and generally only wash against the base of the bluff. Therefore, the only waves to actually reach the base of the bluff with significant energy to cause serious erosion will be large storm waves that would be coupled with a high tide during periods where low sand levels are present on the beach. This combination of events significantly reduces the frequency of wave attack at the base of the bluff. As evidenced by the mature vegetation along the base of the bluff and lack of large amounts of historic erosion, this event is historically relatively rare and the result is that significant erosion of the base of the bluff will has been infrequent.

3.4.3 Southern Bluff Section (MP 244.85 to MP 245.7)

The third section of the bluff is the southerly section. This section begins roughly at midway between the endpoints of Sherrie Lane and 6th Avenue (MP 244.85) and extends south to the end of the study area (MP 245.7). In this section, the bluff top ranges in elevation from 60 feet msl in the north to approximately 48 feet msl in the south. The setback of the rail to the bluff top varies from an average width of ±80 feet in the northerly portion of this section to an average width of ±20 feet between MP 245.2 and 245.36. Within the area south of MP 245.36 the setback is about 40 to 50 feet.

As evidenced in Photo Nos. 19 and 21, in Appendix B, this section of bluff forms somewhat of an embayment with a slightly wider beach. Most noticeable in this section is the lack of significant vegetation on the bluff face (Photo No. 24). Two factors appear to influence the lack of vegetation in this area. The first and probably the most significant is the noticeable reduction is seepage through the bluff face. The second is likely the result of a slight increase in erosion rates for this area. The shore platform that is so noticeable to the north is slightly deeper for this area. While the shore platform cannot be observed directly or from aerial photographs, it can be identified by diving in the surf zones. Based on our rough measurements, the shore platform is approximately 3 to 4 feet below the section immediately to the north (see Photo No. 20, Appendix B). The shore platform still appears to be relatively level. Sand width on this section of beach (including sand within the surf zone) is roughly 300 feet.

The bluff geology is similar to that in the northern sections with terrace deposits capping the bluff top and Delmar Formation sandstones, siltstones, and claystones below. The base of the bluff is generally covered by the debris of past bluff failures that have predominately failed as blocks from the upper portion of the bluff (see Photo Nos. 23, 29, and 31, Appendix B). The dense basal unit that can be observed along other portions of the bluff is not well exposed due to the debris that covers the base of the bluff. The dense resistant unit was however encountered in the borings drilled in this section of the bluff and can also be observed between 5th and 6th Streets (MP 244.9) where a storm drain pipe caused very significant erosion prior to its removal. At this location the storm drain discharge eroded a bowl shaped feature down into the bluff until it encountered the resistant layer at approximately 12 to 15 feet msl (see Photo No. 30 Appendix B). Also exposed on this section of the bluff is an 8-inch diameter CMP pipe that exits the bluff face near the contact between the terrace deposits and the Delmar Formation. This pipe is located opposite the terminus of 5th Street at MP 244.98 (Photo No. 25). This pipe discharges a constant flow of water and appears to be the outlet of a subdrain that was installed in 1941 (and repaired in 1978). The actual extent of this drain is unknown, as it underlies drainage improvements that were installed in 1996. In any event, it likely contributes to the reduction of seepage that can be observed in this section of bluff.

Surface drainage in the southern section is poorly defined west of the tracks and includes some areas of minor ponding following heavy rainfall and sheet flow over the bluff top. Drainage on the east side of the tracks is significantly improved by a large concrete drainage ditch that was installed in 1996 (see Plates 3, 4 and Photo No. 28 Appendix B).

3.4.4 Storm Water Runoff

Evidence of ponding and surficial erosion is evident in several areas along the SDNR alignment. In addition, surface runoff and erosion has been a long-standing problem. The 1978 AT&SF Railroad Company report states that "The steep slopes of the streets running perpendicular to the railroad right-of-way, cause high velocity runoff. In the winter storms of 1977-78 this caused heavy scour in some areas and storm water flowed across the bluff tops and down the face of the bluffs in an uncontrolled manner causing serious bluff erosion." Since 1978, there have been numerous improvements to the storm water drainage system. Today several storm drains continue to discharge at or near the bluff top and well-defined surface drainage is poor to non-existent in areas. There are obvious improvements that can be made by both the City of Del Mar and NCTD to improve the drainage conditions. For example, the appropriate party should take action to restore the drainage structure shown in Photo 27 that has caused severe erosion of the bluffs above the railroad. It is also recommended that repair of the drainage structure at the base of 8th Street (Photo 31) in the bluff preservation works be given a high priority in the bluff preservation works.

The Del Mar Drainage and Hydrology study (Fraser, 1993) addresses some of the storm drain issues and assess the capacity of the various drainage structures, several of which are reportedly under-designed for peak flows apparently due to the urbanization of the watershed area since the rail line was constructed.

3.4.5 Erosion Rates

Erosion rates along the Del Mar Bluffs are similar to those along the remainder of the San Diego County coastline. The erosion at the base of the bluffs has typically been measured in an average erosional retreat rate on the order of a few tenths of a foot per year. However, coastal bluff erosion often occurs in blocks, as the result of episodic events that erode several feet to tens of feet at one time. Review of the 1978 AT&SF Railroad Company study of the bluffs (AT&SF, 1978), indicate the average amount of bluff lost during the period between 1943 and 1978 was on the order of 5 feet. The report goes on to note that the bluff loss is inconsistent along the coastline with some areas having considerably more loss. This loss can be calculated to have an erosion rate for that 35-year period of 0.14 feet per year. Other studies along the San Diego Coastline have calculated similar erosion rates. Leighton and Associates for example calculated an annual erosional rate of 0.22 to 0.33 feet per year for some of the beaches in the San Diego County, (Leighton, 1979).

Recently, however, previously reported erosional rates along many areas of the San Diego County Coastline have been in question due to an overall loss of sand along San Diego Country beaches. The sand along the beach has historically provided a buffer to coastal erosion. It has been demonstrated on some beaches within San Diego County that due to a loss of sand on our beaches, the protective buffer is not as effective as in the past. Natural supplies of sand have severely diminished over the years due to the construction of dams along local rivers, river channelization, the construction of coastline improvements and inland development. The United States Army Corps of Engineers (USCOE) Reconnaissance Report of the Encinitas Shoreline (United States Army Corps of Engineers, 1996), which has bluffs of similar geology, calculated a natural bluff retreat rate of about 0.1 feet per year over a 6000-year period. For the Encinitas area, a historic marine erosion rate of 0.25 to 1.0 feet per year has been presented. For subareal erosion the rates presented ranged from 0.15 to 1.0 feet per year. We note that while the bluffs are of similar geology, beaches in the Encinitas area are typically narrower, steeper, and have a cobble berm along the base. In addition, these beaches that routinely receive heavy surf against the bluff are typically nearly void of sand in the winter. The cobble berm when well established provides some protection to the base of the bluff; however, the cobbles often act as a highly abrasive force in the erosion process. Where cobbles are present there typically is seen a larger amount of "notching" near the base of the bluff that acts to increase the erosion rates. Due to lack of significant cobble berms in Del Mar, typical erosion rates are somewhat less in this area.

The following table provides a summary of calculated bluff retreat rates for the Del Mar area.

Table 1

Calculated Bluff Retreat Rates

Report	Years	Bluff Retreat Rate	Location
AT&SF 1978	1943-78	0.14 ft/yr	Del Mar
L&A	1978	0.22 ft/yr	Del Mar
L&A	1979	0.33 ft/yr	Encinitas
USCOE	1996	0.1 ft/yr in absence of man's intervention	Encinitas
USCOE	1996	0.25 to 1.0 ft/yr with man's intervention	Encinitas
Benumof and Griggs	1999	0.4 to 0.6 ft/yr	Del Mar
FEMA	2000	< 1 ft/yr	Del Mar

In the Del Mar area, review of aerial photographs from historical collections of 1885, 1910, 1928, 1953, 1960, 1990, and 1999 indicate beach widths that do not vary significantly in width. We know that a seasonal migration of sand off the beaches does occur in the winter, but typically there is still a well-developed sand beach in place that only increases in size during the summer months. In addition, the abrasive cobble berms that are present on other beaches in San Diego County were not observed during our study or in the photos reviewed. Notching was only observed in very limited sections.

Previous studies by Kuhn and Osborne,1987, have shown that a high percentage of the bluff erosion and bluff retreat over the past 45 years has occurred as the result of subaerial erosion following periods of major rainfall which acts to saturate the more weathered portions of the bluffs and weaken the bluff materials to the point of failure.

Based on our review of the geologic conditions and sequences of stereoscopic aerial photographs, it does not appear in general as though erosion of the base of the Del Mar Bluffs has increased significantly in the more recent years. In fact, comparison of relatively close scale oblique aerial photographs taken in 1990 and 1999 do not indicate significant erosion. This is also confirmed when the photo sets are expanded to include the older sets of photos. Localized areas of erosion of several feet can be observed, but in general, the erosion appears to be occurring at relatively slow rates. Based on review of bluff conditions adjacent to existing structures in both the north, central, and southern sections of the bluff between 1960 and 1999 the erosion appears to be on the order of between 5 and 10 feet. This relates to an annual erosion rate of 0.12 to 0.25 feet per year, with the slightly higher rates occurring in the southern sections. This estimate appears to be confirmed by the presence of relatively mature vegetation along the base of portions of the bluff, which indicates very low erosion rates. However, a ±1940 historic photo of the bluff near MP 244.2, which is similar to the cover photo, indicates bluff retreat on the order of 5 to 10 feet in the past 60 years. A short wooden seawall has been present in this location since ±1978. Assuming all 10 feet of erosion occurred prior to 1978. This relates to a bluff retreat rate of approximately 0.26 ft/yr for this area.

In the central portion of the bluff, erosion appears to be minimized by the presence of the resistant basal layer and the relatively high flat shore platform that reduces wave energy. In the southern section, the resistant basal layer is still present and although the shore platform is slightly lower, the shoreline forms an embayment that creates a slightly wider beach and typically has more sand protecting the base of the bluff. We attempted to utilize past topographic maps (USGS, 1889) and railroad survey data to calculate actual bluff retreat but were not successful. Recent research along the San Diego County Coastline by others (Benumof and Griggs, 1999) utilized high-resolution digital photogrammetry techniques to qualify erosion along the San Diego County Coastline. For the Del Mar area, they calculated erosion rates of 0.4 to 0.6 ft/yr.

Observation of the bluff face also reveals numerous failures and signs of on-going bluff retreat. Many of these failures have occurred on those portions of the bluff not subject to appreciable marine erosion. The cap of sandy terrace deposits is often at near vertical gradients and excessive rilling can be observed in the photos and during site walks along the beach. Areas of localized seepage and poor drainage exacerbate the problem. Review of the aerial photographs indicates that in areas, the upper portion of the bluff has retreated approximately 3 to 10 feet in the last 10 years. In addition, numerous surficial failures can also be observed within the Delmar Formation, which are also generally above the area of marine erosion. These failures are much more evident in the areas of the bluff that have zones of perched seepage or where storm drains now or previously existed.

Pedestrian traffic and rodent activity although a contributing factor, does not appear to be the primary cause of accelerated erosion. Localized footpaths do exist which provide an avenue for surface runoff and erosion, but these features are relatively widely spaced within the study area.

In summary, average bluff retreat rates are calculated at 0.6 ft/yr or less for the Del Mar area. With retreat rates of this magnitude, one could anticipate that in the next 20 years up to 12 feet of bluff retreat may occur. Therefore, in several sections along the tracks, bluff retreat may impact the existing rails if mitigative measures are not implemented. For purposes of this report and for our risk assessment we have assumed bluff retreat rates of 0.5 feet per year for the next 20 years. This corresponds to a retreat of 10 feet.

3.4.6 Bluff Toe Resistance

Visual examination of the bluff toe indicates that the conditions vary from dense resistant bedrock to areas of soft and loose talus and debris. The condition of the bluff toe is somewhat variable depending on which stage of bluff retreat the individual location is experiencing in Figure No. 8. The stage of bluff retreat, the condition of the toe, and the nature of the exposed soils will vary over time. Where not protected by a wall or other protective device, the areas of soft or loose debris will rapidly erode during periods of heavy surf. The toe of the bluff has been plotted on the geotechnical maps as the interpreted base of the dense bedrock. Thus, in many locations, the actual bluff toe is buried by surficial debris and loose soil. Further investigation of toe conditions will be required if toe of slope protective devices are desired.

3.5 Coastal Processes

The Del Mar Bluffs are located in the southern portion of the Oceanside Littoral Cell, which extends from Dana Point to La Jolla (Appendix E). A littoral cell is a coastal zone that contains a complete cycle of beach sedimentation, sediment sources, sediment transport pathways and sediment sinks. The sources of sediments include discharge from rivers and streams, bluff and shoreline erosion, and the acts of man (such as beach replenishment projects). The primary source of sediment flow is toward the south where the Scripps and La Jolla Submarine Canyon act as a sink for sediments.

As a result of the damming of rivers and streams and other works of man, such as construction of sea walls and other protective devices, the amount of sediment within the littoral cell has been reduced. It is generally accepted that San Diego beaches are narrower today than they were 100 years ago and this condition will impact bluff erosion rates. The USCOE Coast of California Storm and Tidal waves study of 1983 concluded that the total cumulative deficit of sand yield from rivers to beaches in the Oceanside Littoral cell, that stretches from Dana Point to La Jolla, was estimated to be 27-million cubic yards.

Beach profiles have been measured in Del Mar by the USCOE 1934 until 1989 and by SANDAG since 1995. Copies of historical beach profiles are included as Appendix E. Based on photos taken on the beach by NCTD staff at the culvert outlets in 1998 at MP 244.3, MP 244.4, MP 245.16 and MP 245.37 and review of annual inspection reports, the following beach levels have been determined.

Table 2

Del Mar Beach Sand Survey at Culverts

		Level of sand on beach wrt datum		
Milepost	Ref. Datum and year built	1.24.94	8.1.98	9.19.00
MP 244.30	0.0" is 6" below top of side wall in 1910	-24"	-36"	+1"
MP 244.40	0.0" is 6" below top of side wall in 1910	-36"	-30"	0"
MP 245.16	0.0" is old pipe invert level in 1910	-40"	-24"	0"
MP 245.37	0.0" is old culvert invert level in 1910	0"	NR	0"

These record measurements demonstrate the movement of sand since the culverts were constructed in 1910. During the winter months 2 to 3 feet of sand is lost from the beach then after prolonged calm summer weather the sand levels return to the 1910 levels at the culvert discharge points.

This information would indicate that the beach level at the base of the bluffs is similar to the level in 1910 when the culverts were constructed. However, the beach width and protection offered by a wider beach has been reduced due to the loss of sediment in the Oceanside Littoral Cell.

3.6 Sea Level and Wave Action

It has been shown that changes in sea level greatly effect the shoreline processes. Based on studies by the United States Fish and Wildlife Service (U.S. Fish and Wildlife, 2000), it has been reported that sea level has risen approximately 0.7 feet in the last 100 years. In addition, short-term events such as high winds, low-pressure systems and storms can create a temporary raise in sea level and resulting increased potential for erosion. For example, climatic changes such as El Nino (1982-83) created a temporary raise in sea level of approximated 0.85 feet.

Waves within the study area approach from several different directions and vary in wave height and frequency. The wave exposure for the San Diego County area is illustrated in Appendix E.

Breaking waves in the study area are typically on the order of 2 to 5 feet in height with infrequent waves up to 10 feet. A shallow water wave-measuring gauge has been located off Del Mar for the last two decades. The mean characteristic wave height according to the wave gauge is 6.2 feet. Large waves can impact the study area year round and usually last about 2 to 3 days (San Dieguito Wetlands EIS/EIR, 2000).

The following table presents the significant wave height for extreme near shore waves for various return periods (recurrence intervals) at Del Mar and is based wave gauge data and hind casting conducted by Scripps Institution of Oceanography.

Table 3 Significant Wave Heights at Del Mar	
Return Period (years)	Significant Wave Height (feet)
5	13.0
10	14.5
25	16.5
50	18.0
100	19.4
Source: USCOE 1991	

Wave run-up on the bluff face will also be significant during a combination of high surf and high tide or storm surge. Previous studies in adjacent coastal areas (USCOE, 1996) have calculated wave run-up in relatively similar geologic and coastal setting of between 14 and 21 feet mean low lowest water (mllw) for events with a recurrence interval between 5 and 100 years. Based on our calculations, conversations with representatives of NCTD and Coastal Environments, and wave data presented in Table 3 and Appendix E we have calculated a wave run-up in the Del Mar area of 11 feet to 20 feet above mean sea level. Calculations were based on a still water level (SWL) of 4.26 feet, a period of 8 seconds and wave heights at the base of the bluff of 4 and 6 feet (assuming 8 to 16 foot wave breaks offshore). Additional evaluation of wave run-up should be incorporated as part of future studies and design of preservation alternatives.

3.7 Train Vibration

A train vibration study was not performed as part of this original assessment of the bluff conditions in Del Mar. We understand based on our review of the previous geotechnical report that a vibration analysis was performed by Ogden Environmental and Energy Services in the City of Encinitas in 1997. According to the previous report (MAH, 1998), "The Encinitas study focused on the potential impact of a proposed passing track. The referenced study evaluated the geologic significance of vibrations associated with rail traffic on the bluffs. The study indicated:

1. Recorded levels indicated ambient acceleration of the bluffs on the order of one-thousandth that of gravity (0.001g).

2. Bluff monitoring stations located approximately 50 feet from the tracks measured increase in vibration (ground acceleration) levels between the ambient and train passing conditions.

3. Dr. S. Dharmaragan, Professor Emeritus of Engineering Mechanics at San Diego State University was quoted as stating, "The vibration level produced by trains and automobile activity are not sufficient enough to produce effective propagation of a flaw (void) in the material (sandstone bluffs). As the data presented to me indicates, the vibration levels, and corresponding forces, produced by rail operations within the project area are far less than those forces that the bluff experiences due to the action of gravity."

The referenced report concluded that no extraordinary geological failure would occur due to the continued use of the existing rail along the SDNR alignment."

3.8 Faulting and Seismicity

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault that has had surface displacement within Holocene time (about the last 11,000 years). The State Geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years) but that has not been proven to be active or inactive. This definition is used in delineating Fault-Rupture Hazard Zones as mandated by the Alquist-Priolo Earthquake Fault Zoning Act of 1972 and as most recently revised in 1997. The intent of this act is to assure that unwise urban development does not occur across the traces of active faults. Based on our review of the Fault-Rupture Hazard Zones, the site is not located within any Fault-Rupture Hazard Zone as created by the Alquist-Priolo Act (Hart, 1997). Because of the lack of known active faults on the site, the potential for surface rupture at the site due to faulting is considered low.

San Diego, like the rest of Southern California, is seismically active as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional fault zones such as the San Andreas, San Jacinto and Elsinore Faults Zones, as well as along less active faults such as the Rose Canyon Fault Zone.

Our review of geologic literature pertaining to the site area indicates that there are no known major active faults on or in the immediate vicinity of the site (Kennedy, 1975 and Jennings, 1994). Evidence for active faulting was not encountered during our field investigation. On other portions of the San Diego Coastline, we have observed a significant number of predominately inactive faults that transect the coastal bluffs. These faults are often closely spaced and provide localized zones of weathered bedrock along which there is increased erosion. In the La Jolla area, Encinitas, and Solana Beach, these zones of preferential weathering have resulted in the formation of sea caves and increased rates of bluff retreat. In the study area, significant zones of faulting were not observed and while localized minor faults may be present, it does not seem to be a major factor that is contributing to bluff erosion/retreat.

The nearest known active fault is the Rose Canyon Fault Zone (RCFZ) located offshore approximately 1.9 miles west of the site. A regional fault map (Figure No. 3) is attached to illustrate the proximity of the site to major regional faults.

Utilizing the Caltrans Seismic Hazard Map (Mualchin, 1996), the Rose Canyon Fault Zone is considered a strike-slip type fault capable of a magnitude M7.0 maximum credible earthquake. The horizontal peak bedrock ground motion for the site according to these maps is 0.55g.

3.9 Seismic Considerations

The principal seismic considerations for most improvements in Southern California are surface rupturing of fault traces and damage caused by ground shaking or seismically induced ground settlement. The possibility of damage due to ground rupture is considered low since active faults are not known to cross the site.

The seismic hazard most likely to impact the site is ground shaking resulting from an earthquake on one of the major regional faults, causing seismically induced landslides that extend to the tracks. In fill areas such as Anderson Canyon, landsliding or large deformations may result from strong ground acceleration.

Liquefaction of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils underlain by a near-surface ground water table are most susceptible to liquefaction, while the stability of most silty-clays and clays is not adversely affected by vibratory motion. Because of the dense nature of the underlying formation and lack of a near-surface static ground water table, it is our opinion that the potential for liquefaction or seismically induced dynamic settlement of the bluff materials at the site due to the design earthquake is low. Research has shown (Youd, 1998) that Pleistocene-aged and older soils such as the on-site Delmar Formation and Bay Point Formation have a very low susceptibility to liquefaction during strong seismic shaking. With regard to areas of deep canyon fills such as at Anderson Canyon, the soils are not considered liquefiable due to the lack of a permanent high ground water table.

3.10 Ground Water

During our field investigation, ground water seepage was encountered in all of our exploratory borings. In addition, numerous seeps from perched water were observed in the exposed bluff face. Perched ground water is also likely present in some areas of the slope face that are masked by dense vegetation on loose surficial soils. Ground water seepage is primarily located at the base of the Bay Point Formation and within localized zones of sand and fractured material within the Delmar Formation. In our exploratory boring LB-2 for example, very heavy seepage was encountered within a sandy zone within the Delmar Formation to a depth of 28 feet below the existing ground surface. In this location, the sandy zone is believed to correspond to a sandy channel infill within the otherwise clayey Delmar Formation.

Previous efforts to control ground water seepage have included improvements of the storm drain system, surface drainage, and subdrains. Where recent improvements have been made in the southern section of the bluff and where a subdrain system appears to be still working, there is significantly less seepage than in the northern sections. Subdrains were also reportedly installed in the northern section of the bluff; however, we were not able to observe all subdrain outlets. Based on the amount of seepage observed in the northern areas, it is our opinion that the subdrains in this area may not be functioning as intended. Where subdrains are believed to exist (based on review of historical data), they are shown on the geotechnical maps.

The infiltration of surface water on upslope properties is believed to be the source of persistent year-round ground water within the bluffs. Water infiltrating into the soil upslope percolates through the sandy terrace deposits of the Bay Point Formation, then becomes perched on or within the clayey and sandy materials of the Delmar Formation until it reaches the bluff face. Surface drainage adjacent to the tracks is also poor in areas and is expected to contribute to ground water development during rainy weather. At MP 244.2 and between MP 244.5 to MP 244.7 water ponds after periods of heavy rainfall. In addition, in the southern section of the bluffs water seeps from the base of the bluffs above (east) of the tracks. Previous drainage improvements in the southern area have included a concrete-lined ditch with weep holes north of Anderson Canyon. This drainage structure collects a majority of the seepage north of the canyon. South of Anderson Canyon seepage from the bluff east of the tracks locally ponds near the track bed.

3.11 Slope Stability

The evaluation of the factor of safety to resist deep-seated instability was performed using the computer program GSTABL7 with STEDwin (Gregory, 1999). Profiles that were analyzed were selected from the geologic cross-sections that were developed and presented in this report. The selected profiles included the following cross-sections: D-D', F-F', H-H', J-J', K-K', L-L', M-M', N-N', Q-Q', R-R', and S-S'.

Our stability analysis was performed to evaluate the factor of safety to resist deep-seated failure that would extend to the location of the train tracks. Several scenarios were analyzed. The analyzed scenarios included the following: 1) static conditions without a ground water table present, 2) static conditions with a train surcharge, and 3) pseudo-static (seismic) conditions, and 4) static conditions with perched ground water conditions.

3.11.1 Soil Properties

Soil properties that were included in the analyses included soil unit weights and soil strength properties. Based on the tabulated values of unit weights obtained during laboratory testing by Leighton, the average moist unit weight obtained from the ring samples was approximately 129 pounds per cubic foot (pcf). According to data presented on boring logs prepared for previous investigation of the site (MAH, 1998), the average moist unit weight was 122 pcf. In addition to moisture and density determination performed on selected ring samples, moisture and density data was also recorded for samples subjected to shear strength testing by Leighton and Associates. Since direct shear testing by Leighton and Associates was performed on inundated samples, moisture and density values for field and laboratory soaked moisture conditions were obtained for these samples. Based on this testing, the average unit weight for the inundated samples was approximately 136 pcf. These average values for moist and inundated conditions were considered in assigning unit weights for the computer slope stability analysis. For moist conditions, an average unit weight of 125 pcf was utilized. For analysis of hydrostatic conditions, an average unit weight of 130 pcf was utilized.

To develop shear strength parameters considered appropriate for use in the idealized slope stability analysis, we evaluated the results obtained from testing performed during this investigation by Leighton and Associates and results from testing performed during previous investigations at the site. Composite plots of the test results for testing performed by Leighton and Associates and testing performed by others is provided as Figures G-1 through G-11. Table 4 as follows summarizes the average values obtained from these plots.

Table 4

Summary of Average Direct Shear Test Results

Source	Materials	Test Condition	Sample Preparation	Friction Angle, (deg)	Apparent Cohesion, (psf)
L&A, 2000	Bay Point	Inundated	"Undisturbed"	36	300
L&A, 2000	Delmar	Inundated	"Undisturbed"	47	650
L&A, 2000	Bay Point	Inundated	Ultimate	29	150
L&A, 2000	Delmar	Inundated	Ultimate	29	150
MAH, 1998	Bay Point	Field Moisture	"Undisturbed"	32	700
MAH, 1998	Delmar	Field Moisture	"Undisturbed"	38	900
MAH, 1998	Delmar	Field Moisture	Residual	26	800
L&A, 1978	Bay Point	Inundated	"Undisturbed"	35	150
L&A, 1978	Delmar	Inundated	"Undisturbed"	36	310
L&A, 1978	Delmar	Inundated	Residual	23	125
L&A, 1978	Fill	Inundated	Remolded	25	240

Within the analyzed cross-sections, several generalized geologic units were modeled. Utilizing the results of the laboratory testing along with our observations and experience with similar materials and our professional judgement, idealized soil strength properties were assigned to each unit. A summary of the assigned values is provided below as Table 5.

Table 5 Soil Strength Parameters for Slope Stability Modeling		
Material	Friction Angle, (degrees)	Cohesion, (psf)
Fill Soils	32	100
Bay Point Formation	36	200
Delmar Formation	36	300
Delmar Formation (within +/- 5° horizontal)	25	150
Landslide Materials	24	100
Beach Deposits	30	0

Because the Delmar Formation represents the most significant geologic unit relative to the deep-seated stability of the bluffs, further discussion regarding the selection of the idealized strength parameters for that unit is provided. Figures G-2, G-6, and G-9 depict the composite plots of "peak" direct shear test results performed by Leighton and Associates and by others on ring samples of Delmar material. Provided on each plot is a line that represents the average peak shear strength values from the test data. Also provided is a line that represents the selected friction angle and cohesion design values of 36 degrees and 300 pounds per square foot (psf). By review of Figures G-2 and G-6, it can be seen that the strength parameters (for planes not within 5 degrees of horizontal) utilized in the analysis fall along the lower bound of the plotted test results. This corresponds to a reduction of approximately 40 percent of the shear resistance at lower stresses (1,000 psf) and approximately 25 to 35 percent at higher stresses (3,000 psf). By comparison, to Figure G-9, it can be seen that the design value coincides with the average of those test results. Considering these comparisons and the composition of the bluffs, it is our opinion that the averages contained within Figures G-2 and G-6 are elevated by the presence of strong cementing within some of the samples. However, not represented within these results were the local discontinuities observed within these same materials in our borings. To account for these discontinuities and other geologic observations, a reduction in the average direct shear strength values to the selected value was made.

For failure planes within 5 degrees of the horizontal in the Delmar Formation, fine-grained beds are expected to control the failure mechanism. As some evidence of bedding plane shearing was observed within our borings, ultimate strength values rather than peak strength values were considered for selection of shear strength parameters along the horizontal planes within the Delmar Formation. Figure G-4 depicts ultimate strength values for samples from the Delmar Formation and Figure G-4a depicts the plot of the tested fine-grained materials from that same data. As can be seen on the plot, the line representing the design strength parameters was selected slightly below the line of average of the test results. Coupled with our geologic observations, the selected values are considered appropriate for planes along a failure surface that might intersect beds within 5 degrees of horizontal in the Delmar Formation.

3.11.2 Stability Analysis for Static, Drained Conditions

Table 6 as follows presents the results of our stability analysis for the two static scenarios: static and static with train surcharge. Circular and block failure surfaces were consider in our analysis. For modeling of the train surcharge, a uniform strip load of 3,000 psf was applied across a width of 5 feet. This corresponds to the static load of a Cooper E80 engine increased by approximately 40 percent to account for dynamic loading effects.

Table 6

Summary of Factors of Safety for Static Scenarios

Location	Static Scenario		Static with Train Surcharge Scenario
	Circular Failure	Block Failure	
D-D'	2.0	1.65	1.65
F-F'	1.62	1.56	1.46
H-H'	1.58	1.36	1.32
J-J'	1.53	1.37	1.33
K-K'	1.54	1.38	1.35
L-L'	1.89	1.73	1.68
M-M'	1.66	1.37	1.37
N-N'	1.51	1.56	1.50
Q-Q'	1.58	1.50	1.37
R-R'	1.35	1.33	1.19
S-S'	1.43	1.29	1.23

Based on our analysis, Cross-Sections H-H', J-J', K-K', M-M', R-R', and S-S' possess factors of safety less than 1.5 for static conditions. When the train surcharge is incorporated in the analysis, two additional Cross-Sections, F-F' and Q-Q', also fall below a factor of safety of 1.5.

3.11.3 Stability Analysis for Seismic Conditions

For pseudo-static analysis, the cross-sections were analyzed for two values of the horizontal pseudo-static coefficient. The coefficient values considered were 0.15 and 0.28. The value of 0.15 was selected based on the range presented by Seed as reproduced in California Division of Mines and Geology Special Publication 117 – Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 1997). According to Seed, a coefficient range of 0.10 to 0.15 corresponds to maximum earthquake magnitudes of M6.5 to M8.25. Although the maximum moment magnitude of the Rose Canyon Fault

Zone is considered to be M7.0 by the California Department of Transportation (Caltrans), the upper bound of the range, k_H = 0.15 was elected for analysis. The higher pseudo-static coefficient of k_H=0.28 was determined utilizing Caltans methodology (Caltrans, 1999), where the coefficient is set equal to one-half the deterministic peak horizontal ground motion. As discussed in Section 3.8, the peak horizontal ground motion assigned to the site using Caltrans maps is 0.55g. Accordingly, this coefficient was set to be k_H = 0.28. Table 7 as follows presents the results of our pseudo-static analysis.

Table 7

Summary of Factors of Safety for Pseudo-Static Scenarios

Location	Horizontal Pseudo-Static Coefficient	
	k_H = 0.15	k_H = 0.28
D-D’	1.21	0.94
F-F’	1.14	0.90
H-H’	1.02	0.81
J-J’	1.04	0.84
K-K’	1.05	0.84
L-L’	1.22	0.95
M-M’	1.01	0.80
N-N’	1.11	0.89
Q-Q’	1.07	0.86
R-R’	1.01	0.82
S-S’	0.98	0.78

For the horizontal coefficient of k_H = 0.15, all of the analyzed cross-sections, excepting D-D’ and L-L’, fall below the pseudo-static factor of safety of 1.15 recommended by Seed (CDMG, 1997). For the horizontal coefficient of k_H = 0.28, all of the analyzed cross-sections, fall below the pseudo-static factor of safety of 1.0 required by Caltrans (Caltrans, 1999).

3.11.4 Stability Analysis for Perched Ground Water Conditions

To consider the affects of ground water on deep-seated stability, two scenarios were analyzed. One scenario consisted of models that coarsely simulate ground water build-up in 10-foot thick perched zones within the entire depth of the Delmar Formation materials. Both scenarios include a zone of perched water within the terrace deposits of at least 2 feet in thickness. The second scenario consisted of models that coarsely simulated ground water build-up in 25-foot thick perched zones within the entire depth of the Delmar Formation materials. The two scenarios are modeled by assigning uniform pore pressure values of 312 psf and 780 psf within the Delmar Formation portions of the models. These uniform pressures are approximately equal to the average horizontal pore pressure that would result from the perched ground water condition being modeled. It is noted that the vertical uplift along the toe of the failure is also equal to the average value.

Table 8

Summary of Factors of Safety for Hydrostatic Scenario

Location	Sequence of 10-ft Thick Perched Zones	Sequence of 25-ft Thick Perched Zones
D-D'	1.48	1.23
F-F'	1.35	1.08
H-H'	1.18	0.93
J-J'	1.19	0.82
K-K'	1.17	0.85
L-L'	1.58	1.38
M-M'	1.22	1.01
N-N'	1.07	0.76
Q-Q'	1.24	0.93
R-R'	1.14	0.87
S-S'	1.16	1.01

The hydrostatic models illustrate the significant impacts that ground water accumulation posses to slope stability. In all but one case, Cross-Section L-L', the factor of safety is reduced below 1.5 where the sequence of 10-foot perched zones is considered. With the greater perched depth of 25-feet, all analyzed cross-sections fall below the minimum values of 1.5. This modeling illustrates the importance of drainage, both surface and subsurface, with regard to the stability of the bluffs.

3.11.5 Surficial Stability

While not a focus of our study, we are of the opinion that the zone of materials west of the tracks, and in particular within the weathered profile along the bluff face, will possess factors of safety closer to unity (1.0) to resist deeper-seated failure under static, drained conditions. This low factor of safety closer to the bluff face is manifested as numerous surficial failures present on the bluff face.

3.12 Track Bed Support

In order to determine areas where the existing lateral support for the railroad may be lacking, we have utilized the State of California Department of Transportation, Trenching and Shoring Manual (Caltrans, 1996). Based on our review it is our understanding that all major railroads have agreed to utilize the supplemental specifications to Part 20 of Chapter 8 of the AREMA Manual for Railway Engineering regarding shoring. In order to simplify this we have prepared Figure No. 9, "Track Bed Support Line," to illustrate when there is a need for additional shoring or lateral support of the rail.

The figure shows that the minimum track bed support is defined with a line that extends 11 feet from the centerline of the rails and then outward and downward at a gradient of 1.5 to 1 (horizontal to vertical). When existing grades or zones of highly weathered or disturbed material encroach inside of this line, additional lateral support or shoring is needed. Based on our analysis, stability at the top of the bluffs is generally improved where a 10-foot weathering setback is allowed for beyond the projected track bed support line. This top of bluff weathering setback has been added to Figure 9 to illustrate this allowance.

We have found that in general this line also depicts the approximate point where slope stability analyses indicate an adequate factor of safety (without unusual geotechnical conditions). The track-bed support line at the toe of slope has been plotted on the geotechnical maps (Plate through 5). Areas where track bed support at the top of slope is now or anticipated by the year 2020 to be inside the track-bed support line are indicated on Tables 10 and 11, Appendix C.

The following Table 9 illustrates areas both now and projected to the year 2020 areas where the track-bed support line is located beyond (protrudes) the bluff toe and where additional lateral support is desirable.

Table 9

Protruding Track-Bed Support Line by Milepost

Milepost in Year 2000	Distance Protruding Support Line in 2000	Milepost in Year 2020	Distance Protruding Support Line in 2020
244.19 to 244.50	1,040 feet	244.18 to 244.55	1,955 feet
244.71 to 244.79	20 feet	244.56 to 244.62	315 feet
244.80 to 244.81	55 feet	244.69 to 244.81	635 feet
245.20 to 245.27	370 feet	245.04 to 245.05	55 feet
245.29 to 245.38	475 feet	245.13 to 245.14	55 feet
		245.15 to 245.39	1,270 feet
Total	2,960	Total	4,285

With regard to inadequate support at the top of bluff, it is our opinion that in areas when the top of bluff is within 10 feet of the reference line, or where it is anticipated to occur within the next 20 years, stabilization measures should be implemented.

4.0 RISK ASSESSMENT

In order to provide an overall evaluation of the impacts of the site geotechnical conditions to the existing rail alignment, the methodology we chose was to prepare a matrix of the various geotechnical conditions which affect the alignment. We have then ranked each item based on risk and then compiled a ranking for each specific site. This compilation was then the basis for developing a risk ranking (low to high) for the different segments of the SDND alignment. The geotechnical factors considered in our ranking include the bluff height and gradient, the location of the existing tracks in relation to the bluff top and toe, coastal erosion and bluff retreat, and geologic factors such as the various soil types, areas of ground water and existing improvements.

Our risk assessment matrix is presented as Table 10 (Appendix C) for the year 2000, and as Table 11 (Appendix C) for the year 2020. In order to prepare the matrix, we selected 28 locations along the alignment and ranked each site on the factors described above. In addition, we reviewed the areas where slope stability analyses were performed and compared the calculated factor of safety to the assigned risk. Factor of safety was not a criteria utilized in our risk matrix because slope stability analyses were performed on only selected cross sections which may not be representative of an entire section of bluff. In our opinion, the slope height, setback from rail, and the projection from the track to toe (track-bed support line), incorporate the slope stability concerns into the matrix.

The 28 "risk matrix sites" we utilized are indicated on Plates 1 through 5, Site Pan and Geologic Maps. For each identified condition, we have ranked the sites on a 1 to 5 basis, with 5 having the highest risk. We have then totaled the scores for all of the sites with the highest scores indicating the areas of highest risk. Finally, based on our knowledge of the site conditions, we have defined the limits of the areas that are representative of each risk and included those limits on the geotechnical maps. The areas with the highest risk are therefore the sites most likely to be impacted within the 20-year period we were provided.

A summary of the entire alignment is presented as follows with the lateral limits shown on the site plans and Geotechnical Maps. For convenience, a reduced copy of the maps is included as Appendix C and illustrated in Appendix F.

4.1 Mile Post Area-by-Area Review

Area from MP 244.10 to 244.19

Risk Priority	Low
Risk Matrix Sites Included:	1
Cross-Sections Included:	None
Slope Stability Analyses Included:	None

This area is generally low risk priority due to the wide distance from the track to the bluff top. In addition, the area provides adequate track bed support. Seepage is minor and foot traffic on the bluff face does not occur. Localized areas of ponding and poor surface drainage are present. A storm drain outlet at the north end is causing some accelerated erosion.

Area from MP 244.19 to 244.43

Risk Priority	High
Risk Matrix Sites Included:	2, 3, 4, 5, 6 and 7
Cross-Sections Included:	J-J', K-K', I-I', Q-Q', T-T', R-R' and S-S'
Slope Stability Analyses Included:	J-J', K-K', Q-Q', R-R' and S-S'

This area is high risk due to the proximity of the bluff top to the existing track, as little as 10 feet. Over-steepening occurs in several areas. The track-bed support line extends beyond the toe of the bluff. Seepage is generally moderate and slope failures have previously occurred in the area. A major top of slope wall and subdrain system are present in the southern portion. There does not appear to be adequate lateral support for the top of slope wall foundation.

Area from MP 244.43 to 244.46

Risk Priority	Medium
Risk Matrix Sites Included:	8
Cross-Sections Included:	P-P' and V-V'
Slope Stability Analyses Included:	None

This area is medium risk as there is only 10 feet between the tracks and the bluff top. The geologic unit is artificial fill (sand-cement), which increases the surficial stability, and appears to have been affective in reducing erosion on this steep slope. In addition, no seepage, foot traffic or existing bluff failures occur on the bluff face. The site of past failures, this section has been downgraded to medium risk because of the construction of a sand-cement buttress that has performed well to date. The track-bed support line extends beyond the toe of slope. The toe of slope protection was not upgraded with the recent buttress construction.

Area from MP 244.46 to 244.51

Risk Priority	High
Risk Matrix Sites Included:	9
Cross-Sections Included:	H-H'
Slope Stability Analyses Included:	H-H'

This area is high risk due to the presence of existing slope failures and over-steepened areas. Seepage and foot traffic are heavy in this area. This is also a major pedestrian access area. The track-bed support line extends beyond the toe. MP 244.47 has the highest total risk of all the sites due to the aforementioned conditions.

Area from MP 244.51 to 244.69 (High Bluff Area)

Risk Priority	Medium
Risk Matrix Sites Included:	10, 11 and 12
Cross-Sections Included:	L-L', G-G''
Slope Stability Analyses Included:	L-L'

This area is overall medium risk due to adequate setback from the top of bluff to the tracks. Over-steepening and moderate seepage occur on the bluff face. Potentially a high-risk area, the general 70-foot track to bluff top increases stability of track bed. The track-bed support line is generally behind the toe of slope. The high bluff has over-steepened areas and seepage zones. Block falls are anticipated but, not expected to impact rail operations due to setbacks. Water ponds along the tracks from runoff and storm drain discharge.

Area from MP 244.69 to 244.8

Risk Priority	High
Risk Matrix Sites Included:	13, 14, and 15
Cross-Sections Included:	U-U', F-F, and M-M'
Slope Stability Analyses Included:	F-F and M-M'

This area is high-risk due to heavy seepage on the bluff face and storm drain discharge on the top of bluff at the northerly end. Moving southerly to MP 244.76, seepage ends, but the track-bed support line protrudes beyond the toe, maintaining high risk. Major erosion occurs at the undersized storm drain outlet (MP 244.7). Previous failures in this area have been repaired. A soldier pile wall is present in southern section from the 1978 repair and some toe protection exists on the beach.

Area from MP 244.8 to 244.89

Risk Priority	Medium
Risk Matrix Sites Included:	16 and 17
Cross-Sections Included:	E-E'
Slope Stability Analyses Included:	None

This area is medium risk. The track-bed support line extends behind the toe. The area is not over-steepened, seepage does not occur and foot traffic is moderate. This section has adequate track to top distance and adequate track bed support.

Area from MP 244.89 to 244.92

Risk Priority	Medium
Risk Matrix Sites Included:	18
Cross-Sections Included:	None
Slope Stability Analyses Included:	None

This area is medium risk due to the presence of a large erosional gully resulting from removal of a storm drain. The area near the tracks has been backfilled. Over-steepening is moderate and seepage and foot traffic are minor. The track-bed support line is behind toe of bluff; however, a large eroded area reduces lateral support and is of higher priority.

Area from MP 244.92 to 245.21

Risk Priority	Medium
Risk Matrix Sites Included:	19, 20, 21, 22 and 23
Cross-Sections Included:	D-D' and C-C'
Slope Stability Analyses Included:	D-D'

This area is medium risk due to moderate over-steepening of the bluff face. The track to bluff top distance ranges from 150 feet to 105 feet. The track-bed support line is well behind toe of slope. Seepage is minor (possibly due to past drainage improvements) and foot traffic is absent. It should be noted that in the northerly and southerly ends of the area the slope to the east of the tracks presents some risk, even though it is excluded from this study. A ±10 foot wall at storm drain outlet MP 256.16 protects the fill area from erosion. The wall has performed well-to-date but, on-going maintenance is critical

Area from MP 245.21 to 245.37

Risk Priority	High
Risk Matrix Sites Included:	24 and 25
Cross-Sections Included:	B-B', N-N' and A-A'
Slope Stability Analyses Included:	N-N'

This area is high risk due to moderate over-steepening of the bluff face and the close proximity of the track to the top of the bluff. The projection of the track-bed support line extends beyond the toe of the bluff. In addition, the track to top of bluff distance decreases to 15 feet in the southerly end of this area. Seepage is minor and foot traffic and bluff face over-steepening are generally moderate. Again, a major wall protects the fill area at Anderson Canyon from erosion. The wall has performed well but its maintenance is critical.

Area from MP 245.37 to 245.70

Risk Priority	Low
Risk Matrix Sites Included:	26, 27 and 28
Cross-Sections Included:	O-O'
Slope Stability Analyses Included:	None

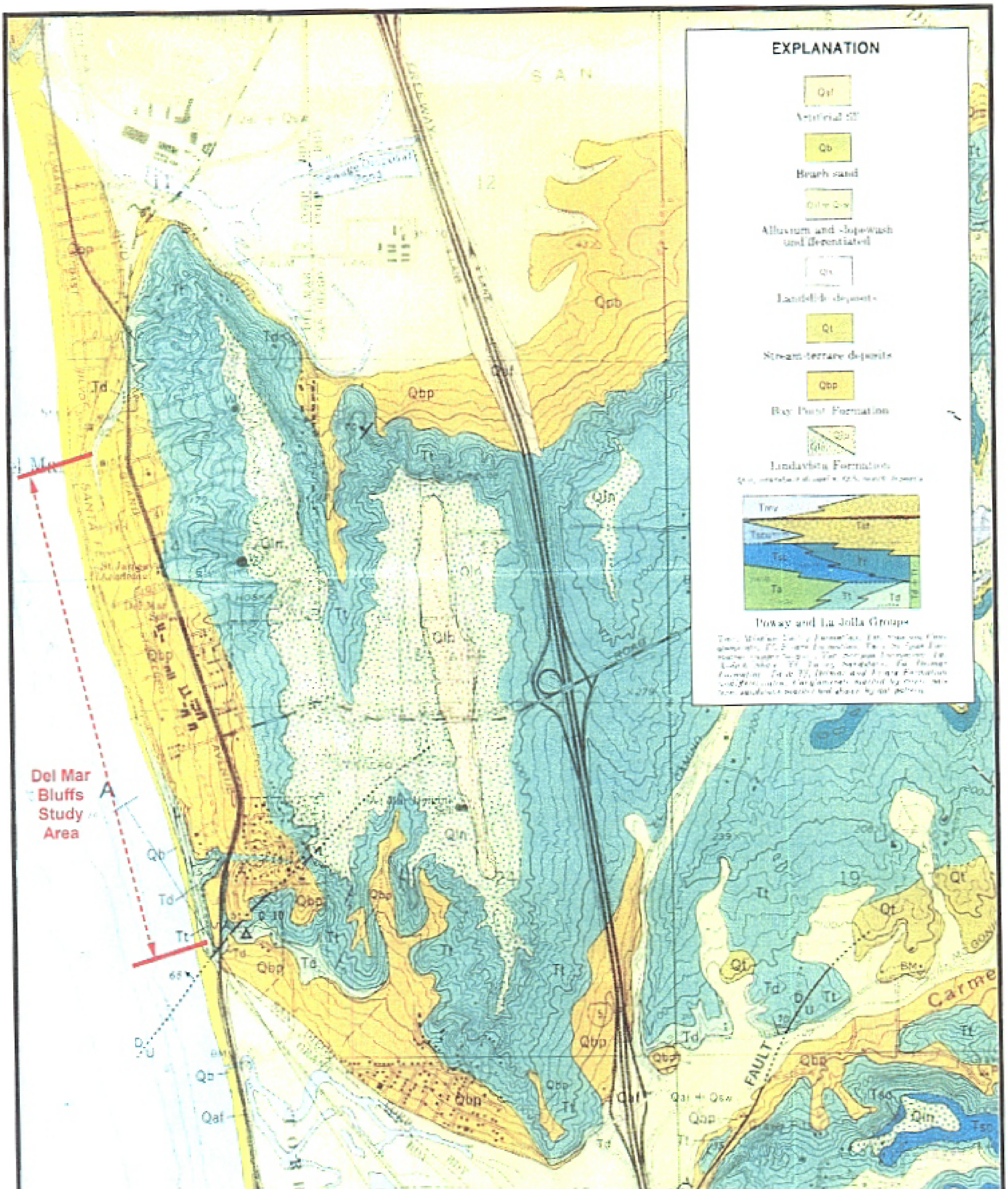
This area is low risk due to several factors including: slightly lower bluff heights and neither significant seepage nor foot traffic are present below the tracks. Seepage from the bluffs above the tracks requires mitigation to keep the track bed from being saturated. Minor over-steepening of the bluff face is present and the area has an average track to top of bluff distance of 45 feet. The track-bed support line is well behind toe of slope.

4.2 Summary of Risk Assessment

It should be understood that the ranking of the individual site factors is in a large part based on our engineering and geologic judgements of the conditions observed and studied at the site.

As previously noted, coastal erosion is quite often controlled by episodic events and the actual extent of bluff failures and coastal erosion may vary somewhat from our model risk assessment and analyses. The bluff retreat rates presented in this report are average rates; as a result, there can be essentially no erosion for a number of years and then several feet of retreat may occur in one event. In addition, changes to the surface conditions on the bluff top and to the east, whether by nature or by the acts of man, more accelerate or decrease the degradation of the bluff and impact the rail alignment.

This document has been prepared as a planning tool to help organize and facilitate preservation of the rail alignment for the next 20 years. Conceptual repair alternatives are presented under separate cover as Part 2 of the Del Mar Bluffs Geotechnical Study. It should be understood that additional investigation and design work is intended to be included as part of the selected preservation alternatives.



Source: Geology of the Del Mar Quadrangle, San Diego County, California, Scale 1"=24,000', 1975

REGIONAL GEOLOGIC MAP

Project No.
040151-001

Date
1-7-01

Leighton and Associates, Inc.

Figure No. 2

EXPLANATION

Fault traces are indicated by solid lines where well located, by dashed lines where approximate or inferred, by dotted lines where concealed and queried where uncertain. Coloring and highlighting indicate the age or regency of displacement:

PINK	Faults that show displacement during historic time (i.e. last 200 years)
ORANGE	Faults that show displacement during Holocene (i.e. last 10,000 years)
GREEN	Faults that show displacement during late Quaternary (i.e. last 700,000 years)
PURPLE	Faults that show displacement during Quaternary (i.e. last 1.6 million years)
BLACK	Fault without recognized Quaternary displacement (considered inactive faults)

Adapted from Jennings, 1994, Fault Activity Map of California and Adjacent Areas: CDMG, California Geologic Data Map Series, Map No. 6

REGIONAL FAULT LOCATION MAP
DEL MAR BLUFFS
DEL MAR, CALIFORNIA

Project No.	040151-001
Scale	1:750,000
Engr./Geol.	SAC/MRS
Drafted By	KRD
Date	1-7-01

1045 889

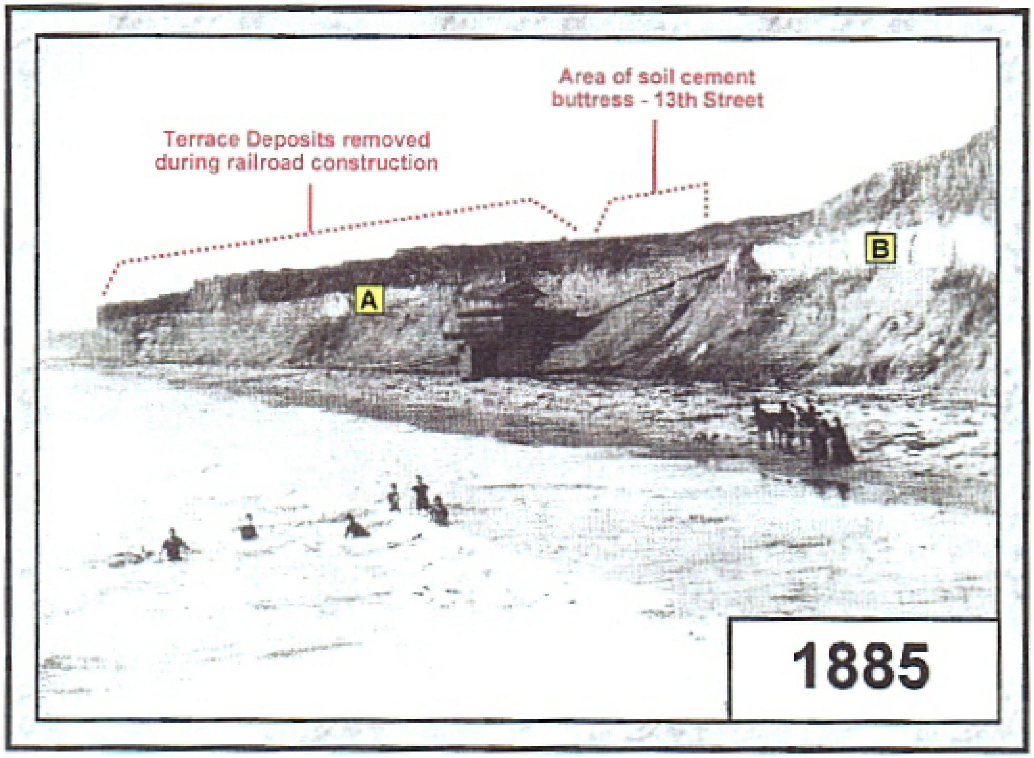
Figure No. 3


CITY OF DEL MAR
ORIGINAL MAIN TRACK CENTERLINE
PRESENT MAIN TRACK CENTERLINE
P A C I F I C
O C E A N
NOB AVE
GRAND (STATE HIGHWAY) AVE.
RAILROAD AVE.
OCEAN AVE
COAST BLVD.
℄ Original Main Track
℄ Present Main Track A.T.&S.F.Ry.Co.
Scale: 1"=400'
HDR
ORIGINAL AND EXISTING RAILROAD ALIGNMENT
(Atchison, Topeka and Santa Fe Railway Right of Way Records)
DELMAR BLUFFS GEOTECHNICAL INVESTIGATION
SCALE 1"=400'
Figure No. 4

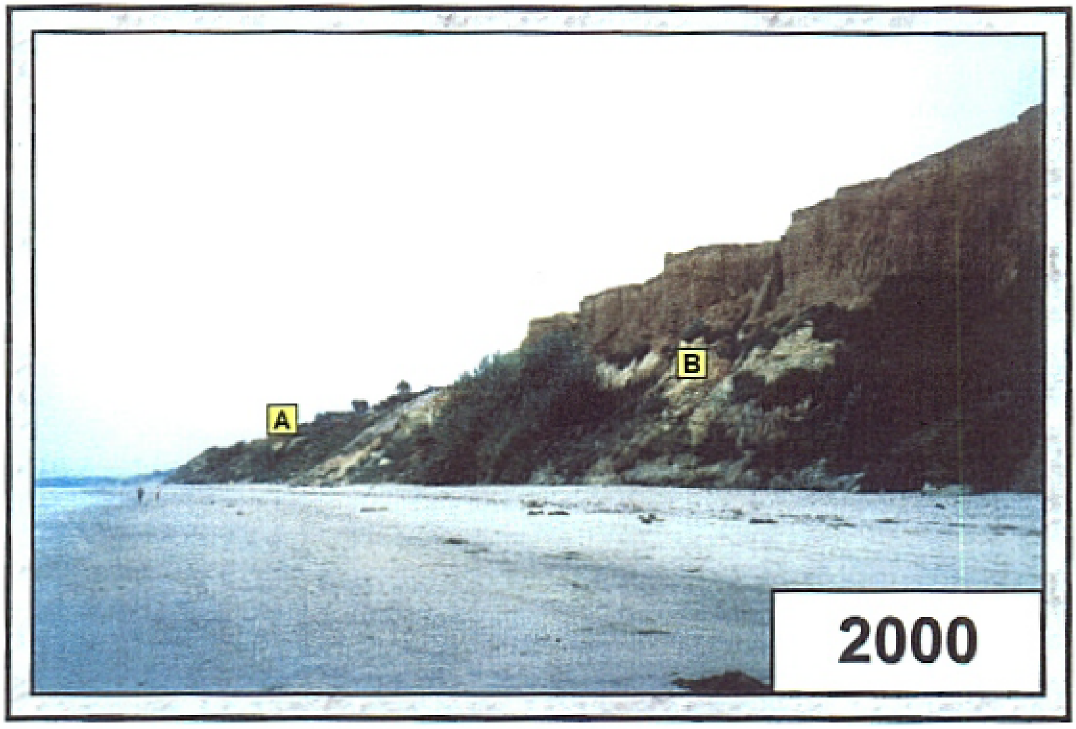
1910 RAILROAD CONSTRUCTION

Del Mar Bluffs
Del Mar, California

Figure No. 5



A
B
= Identical points



View north from approximately same location at
"Natatorium" in both 1885 and 2000.

Figure No. 6

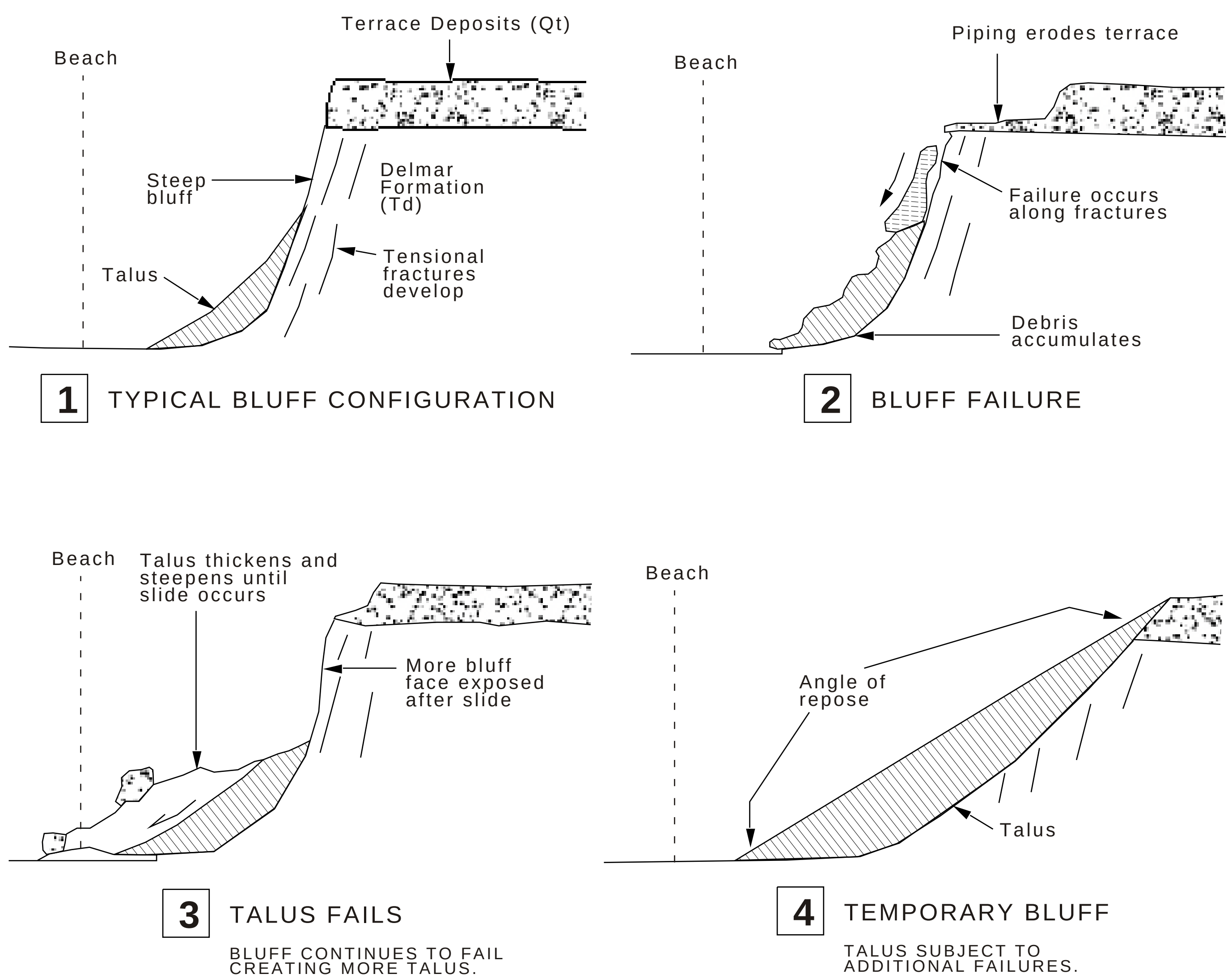
PHOTO: CIRCA 1940
M.P. 244.2
Del Mar Bluffs
Del Mar, California

Note berm of air dumped material along top of bluff.

Photo by William D. Middleton from *Santa Fe… the Railroad Gateway to the American West* by Donald Duke

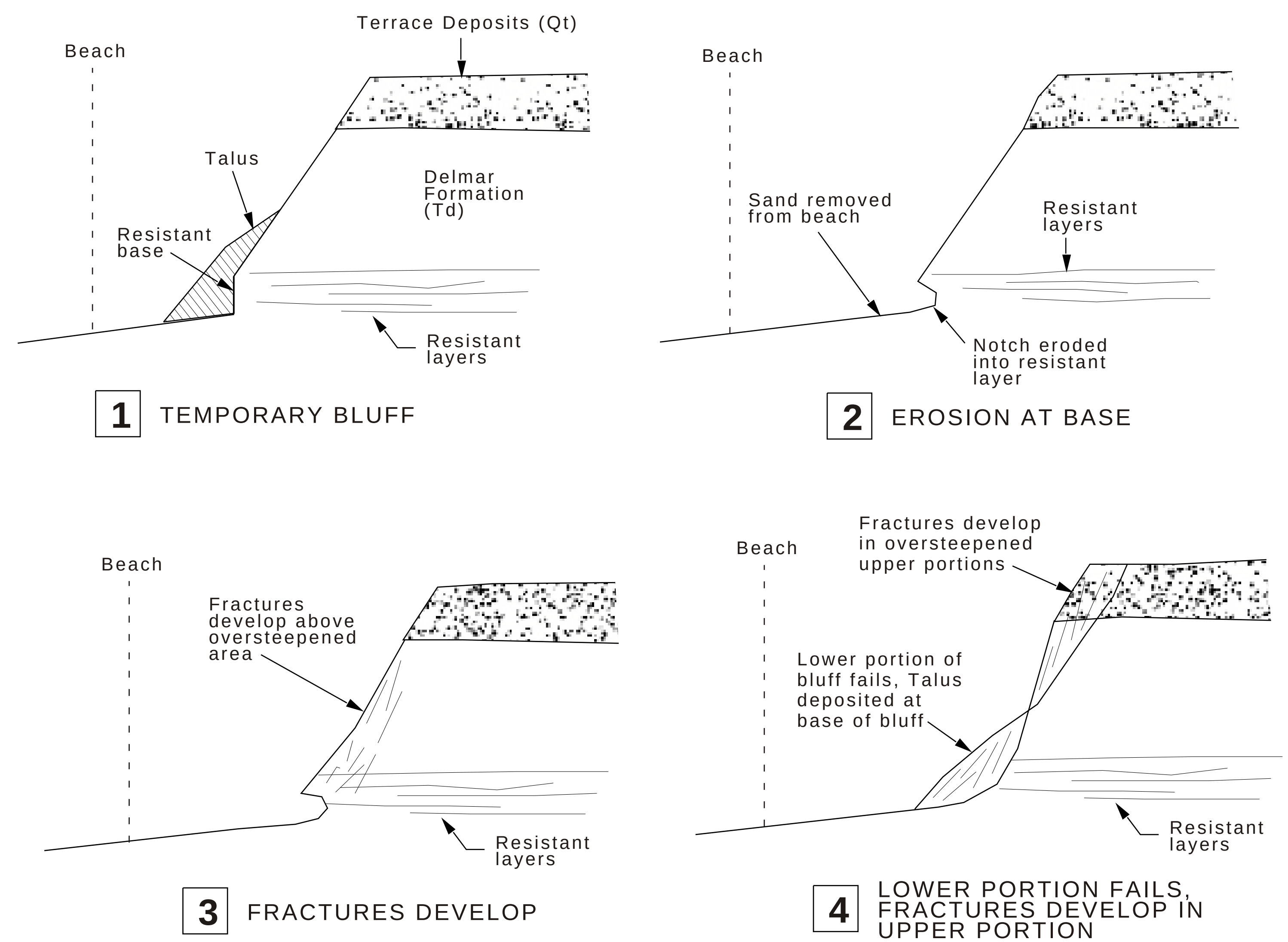
Figure No. 7

SEQUENCE IN BLUFF RETREAT WITHOUT TOE EROSION



SEQUENCE IN BLUFF RETREAT WITH TOE EROSION

NOTE: SIMILIAR SEQUENCE OCCURS AT MIDSLOPE ELEVATIONS WHERE GROUNDWATER IS PERCHED.



TYPICAL STAGES OF BLUFF RETREAT

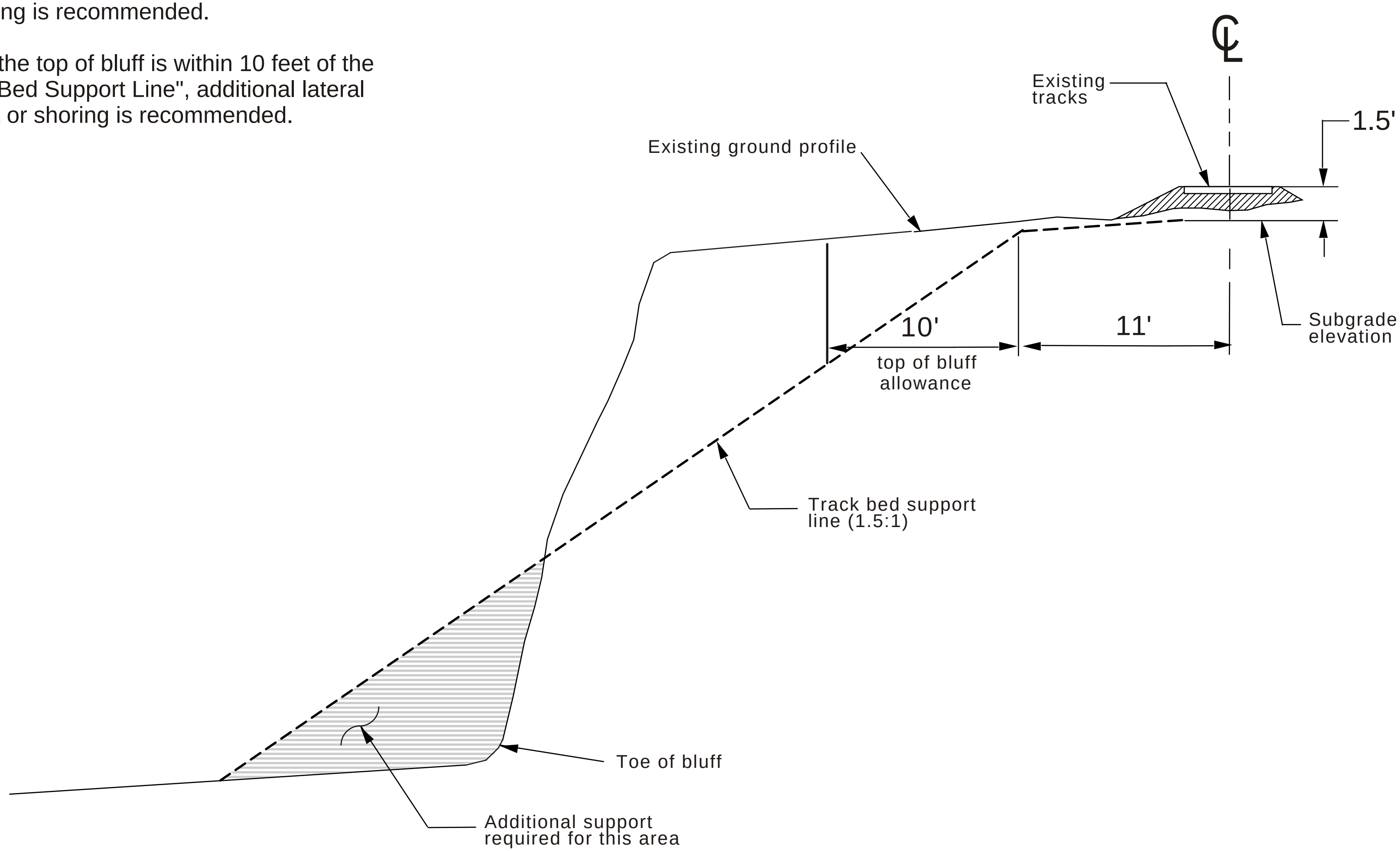
Project No.	040151-001
Scale	Not to scale
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 8

Where the "Track-Bed Support Line" is above toe of bluff, additional lateral support or shoring is recommended.

Where the top of bluff is within 10 feet of the "Track-Bed Support Line", additional lateral support or shoring is recommended.



Modified from: State of California, Department of Transportation Trenching and Shoring Manual, January 1990, Revised December 1996.

TRACK-BED SUPPORT LINE

Project No. **040151-001**

Date **January 2001**

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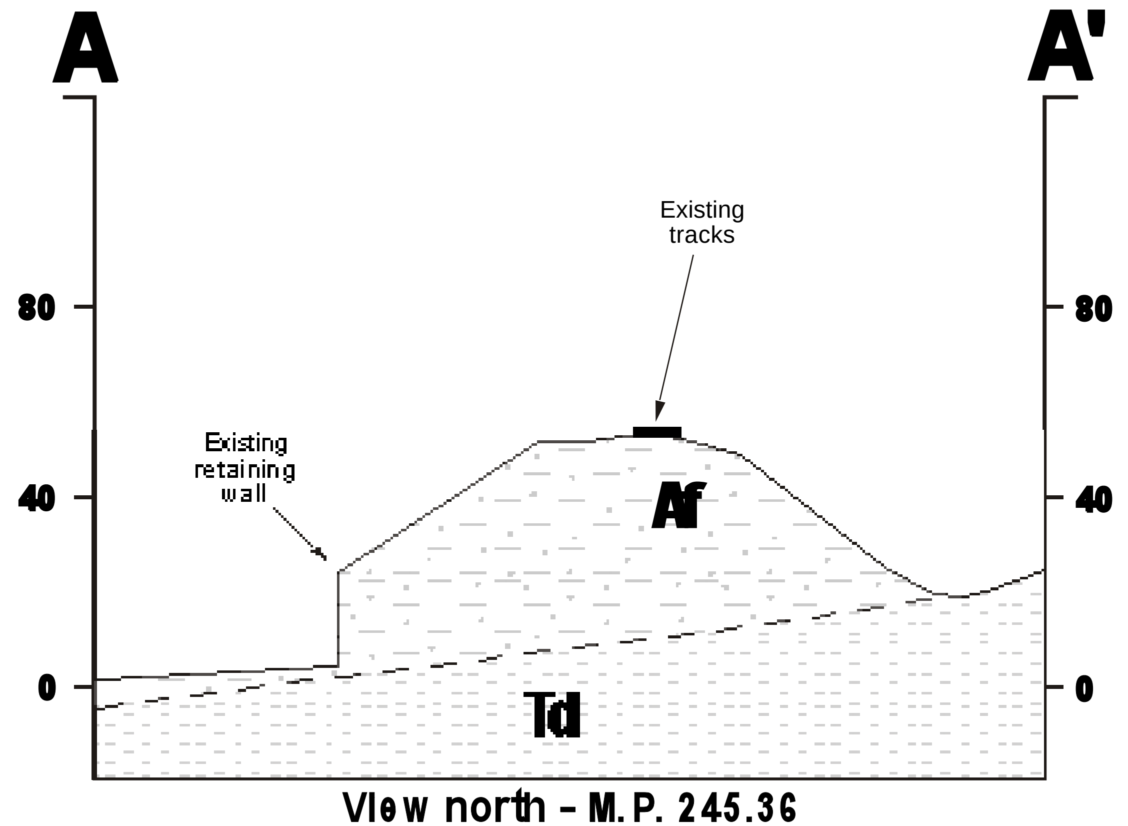
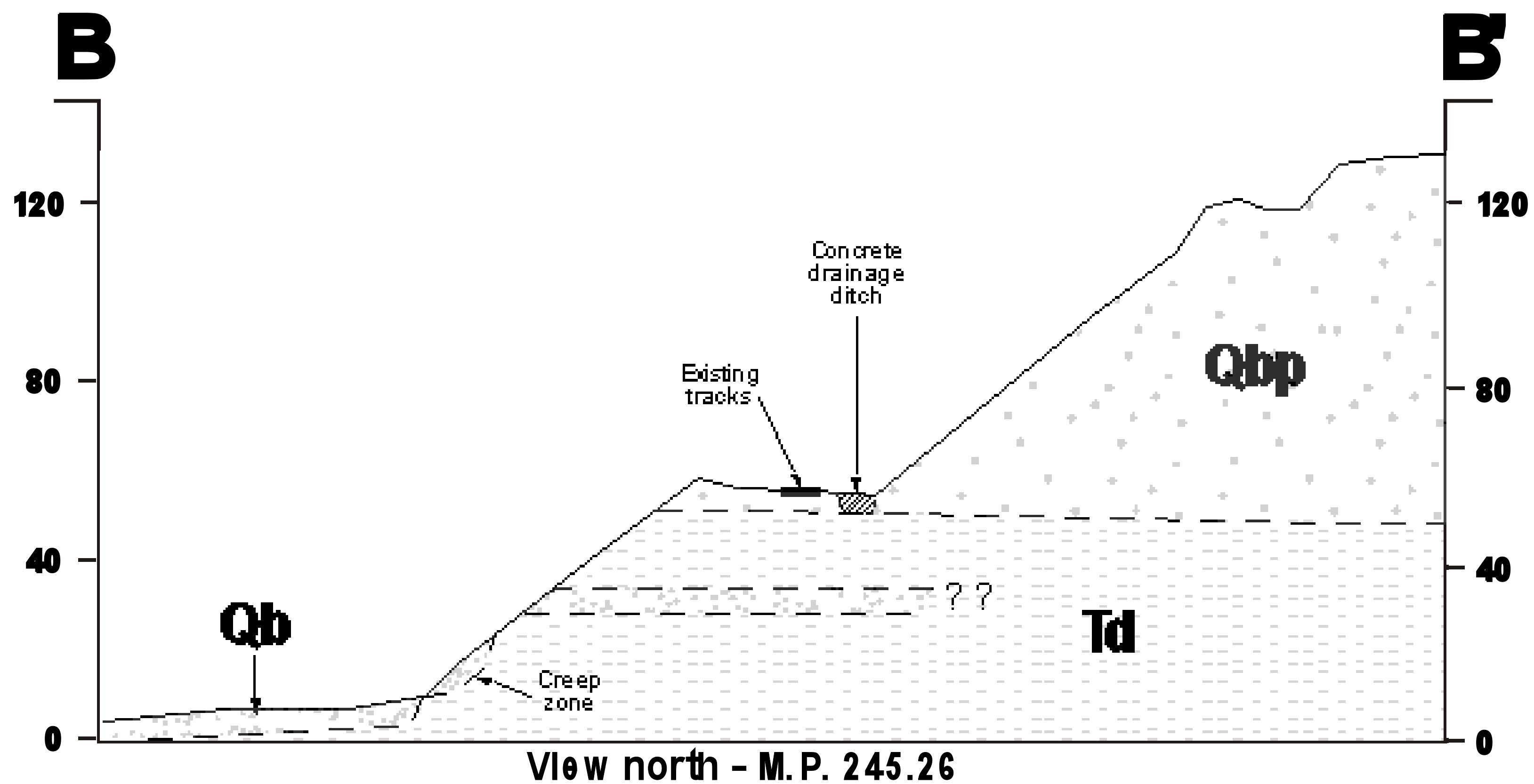
Figure No. 9





LEGEND

Af	Artificial fill soils
Qls	Landslide deposits
Qb	Beach deposits
Qbp	Bay Point Formation
Td	Delmar Formation

CROSS-SECTIONS A-A' AND B-B'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

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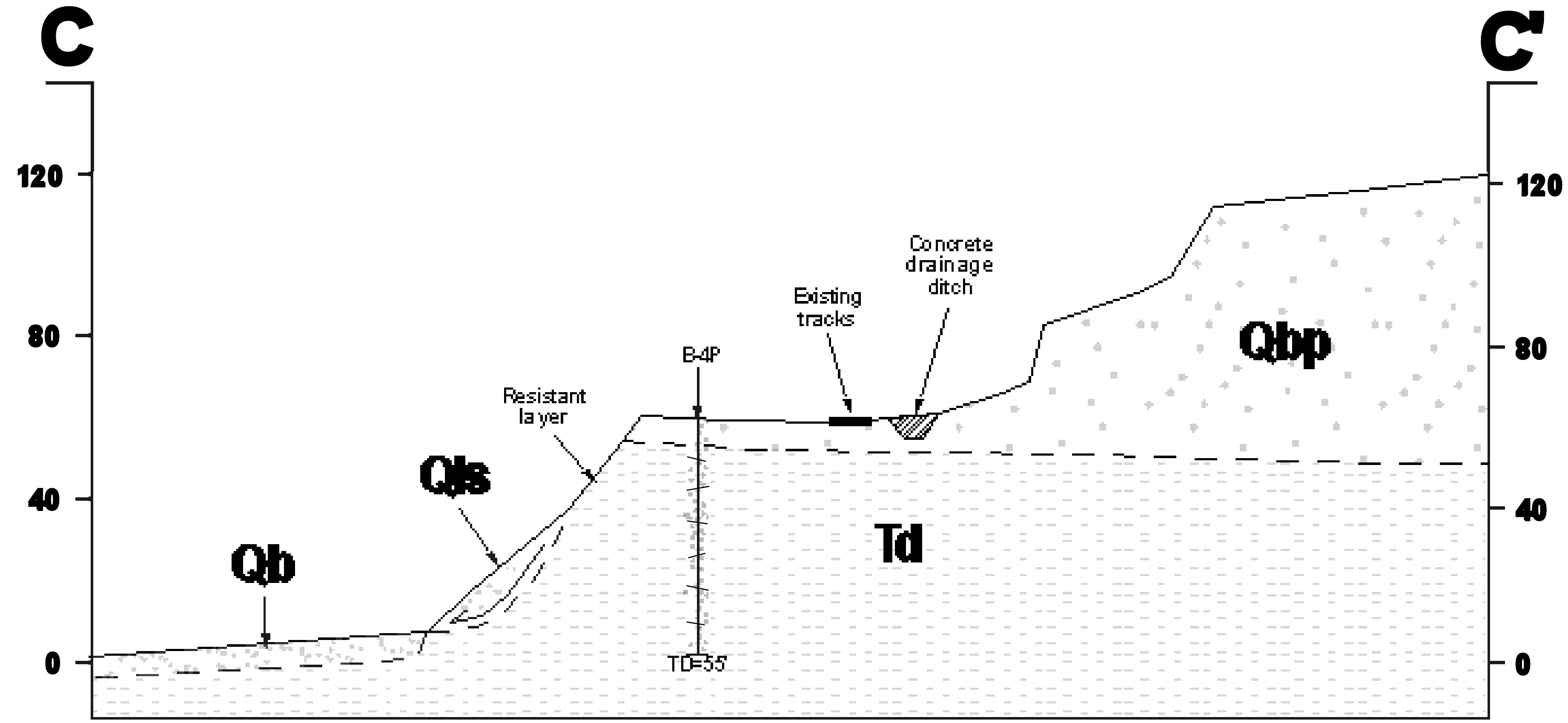
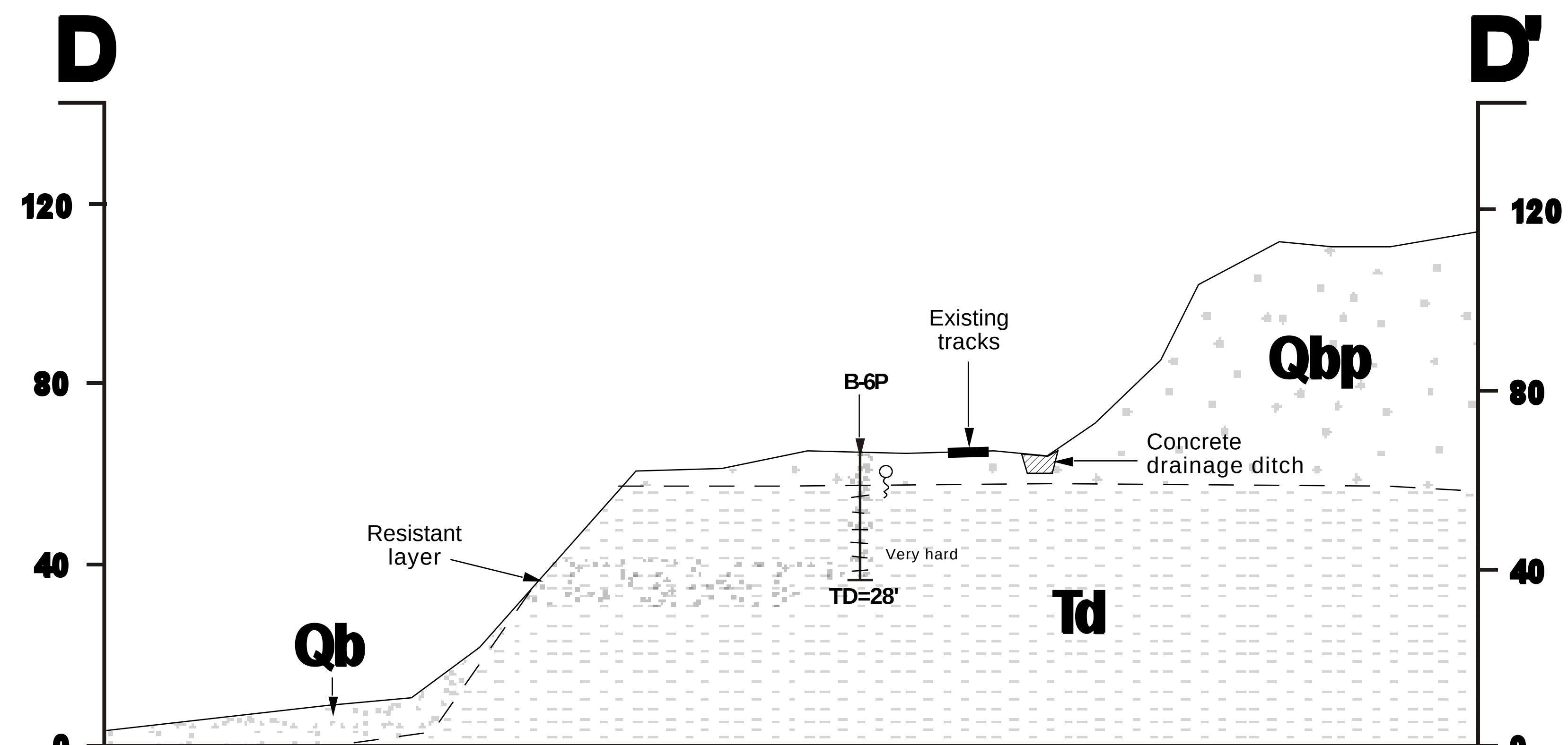
Figure No. 10



View north - M.P. 245.19



View north - M.P. 245.04

See Figure No. 10 for Legend.

CROSS-SECTIONS C-C' AND D-D'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

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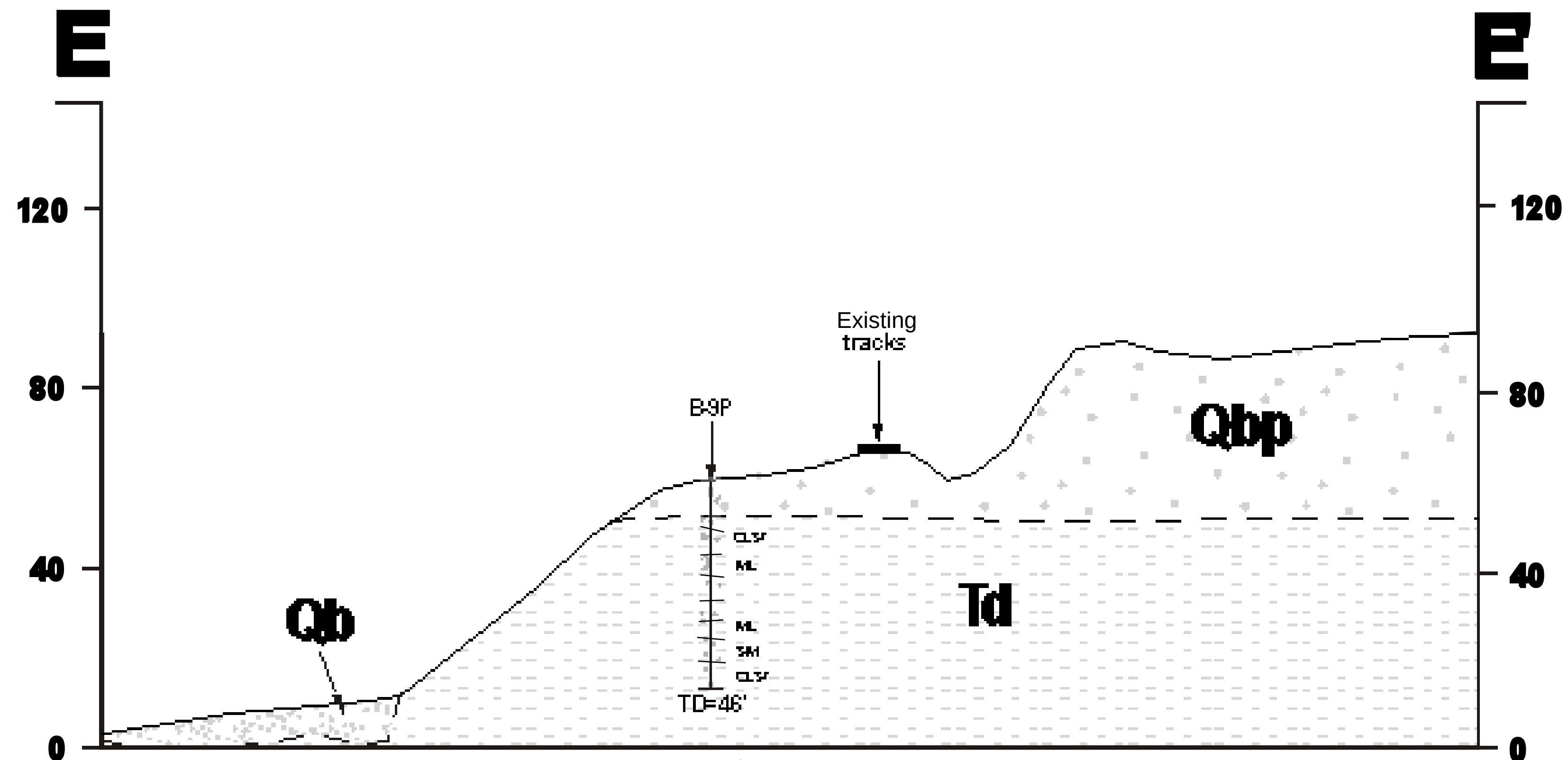
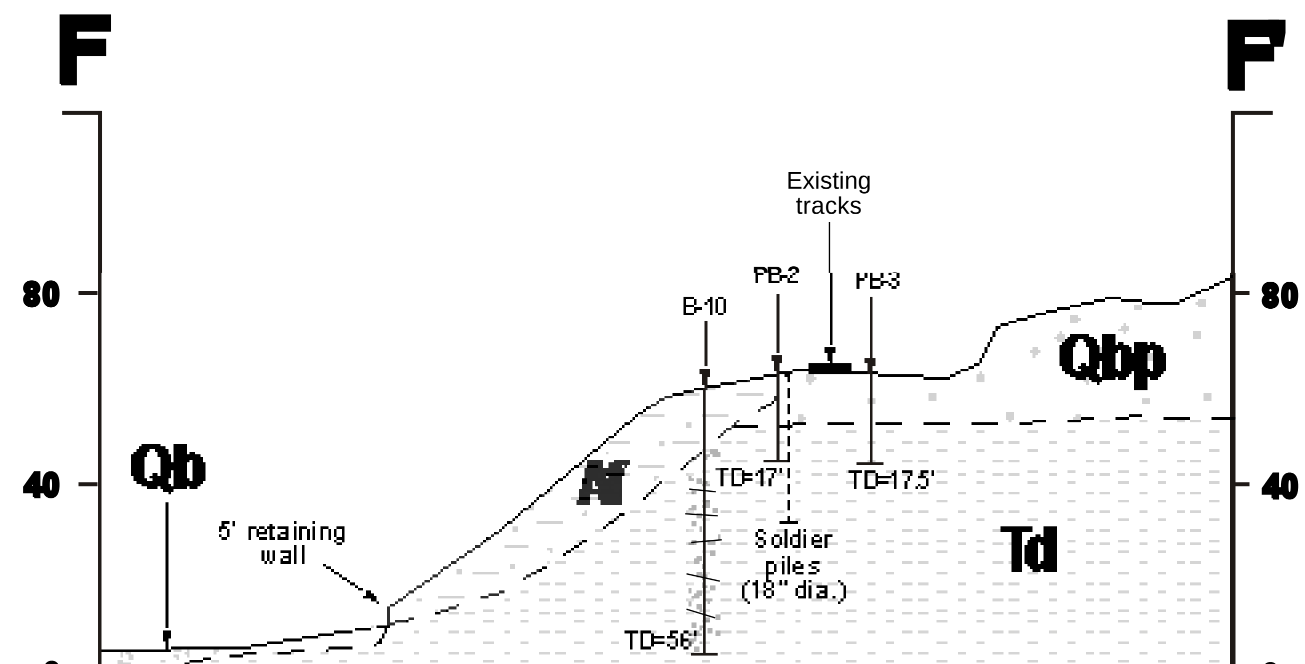
Figure No. 11



View north - M.P. 244.81



View north - M.P. 244.78

See Figure No. 10 for Legend.

CROSS-SECTIONS E-E' AND F-F'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

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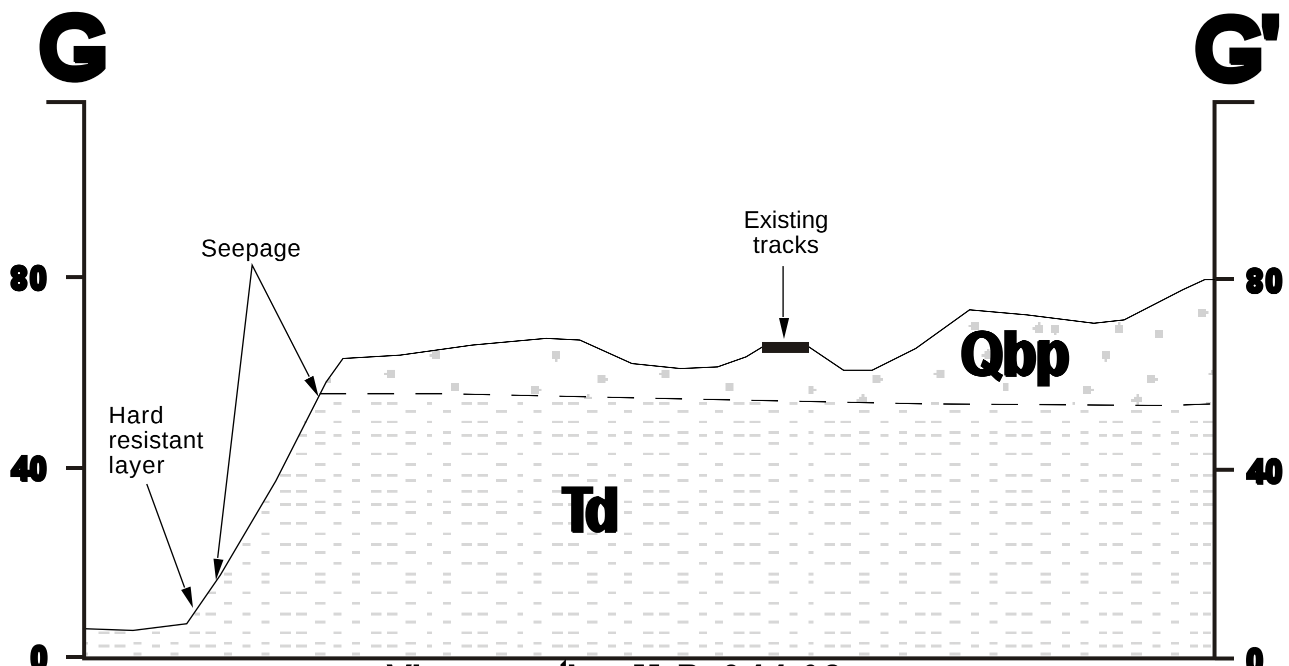
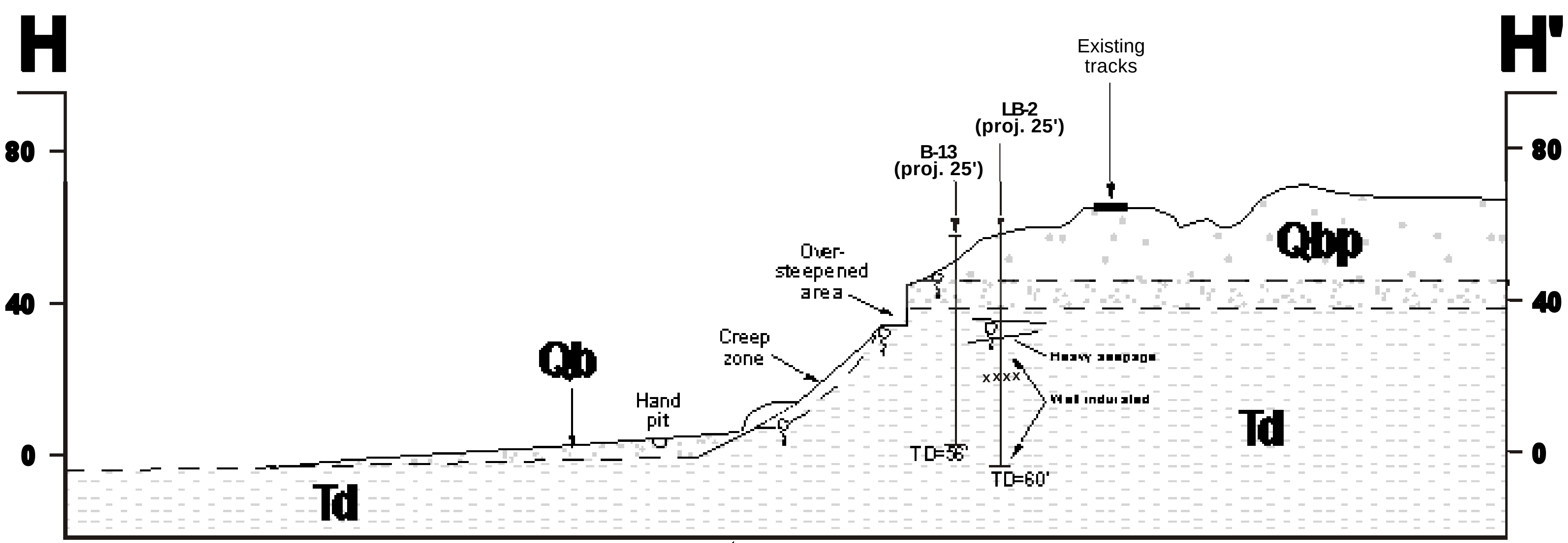
Figure No. 12



View north - M.P. 244.68



View north - M.P. 244.47

See Figure No. 10 for Legend.

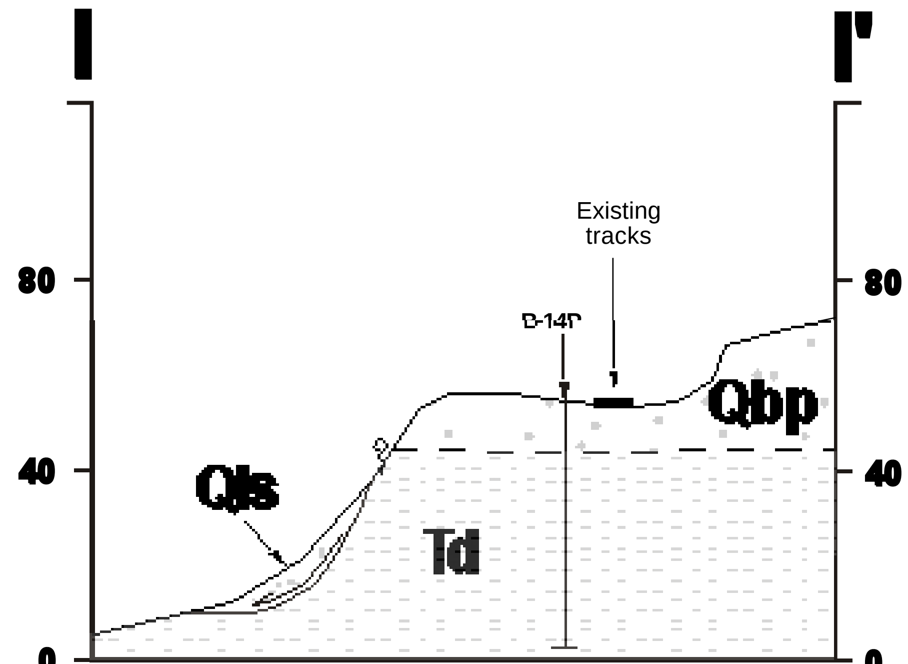
CROSS-SECTIONS G-G' AND H-H'

Del Mar Bluffs
Del Mar, California

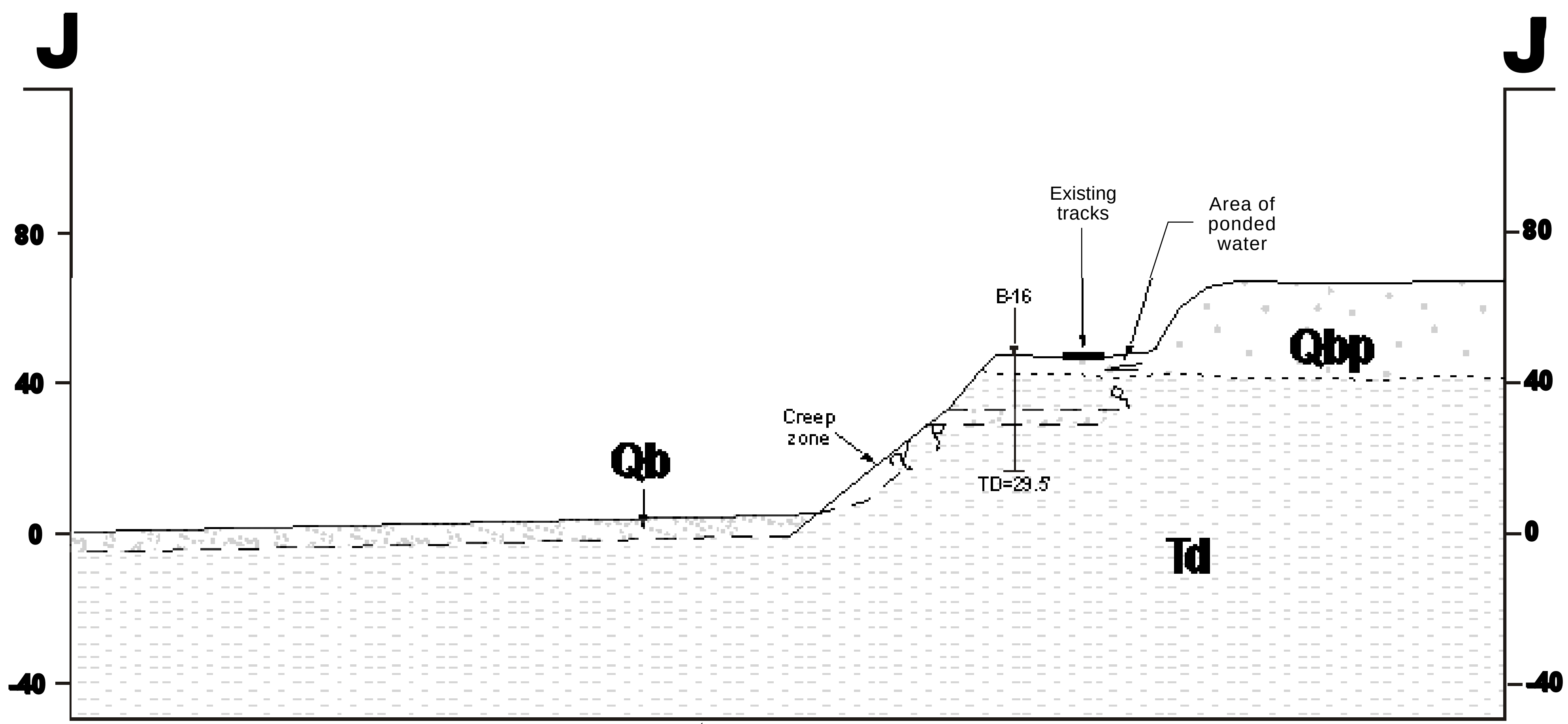
Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 13



View north - M.P. 244.37



View north - M.P. 244.2

See Figure No. 10 for Legend.

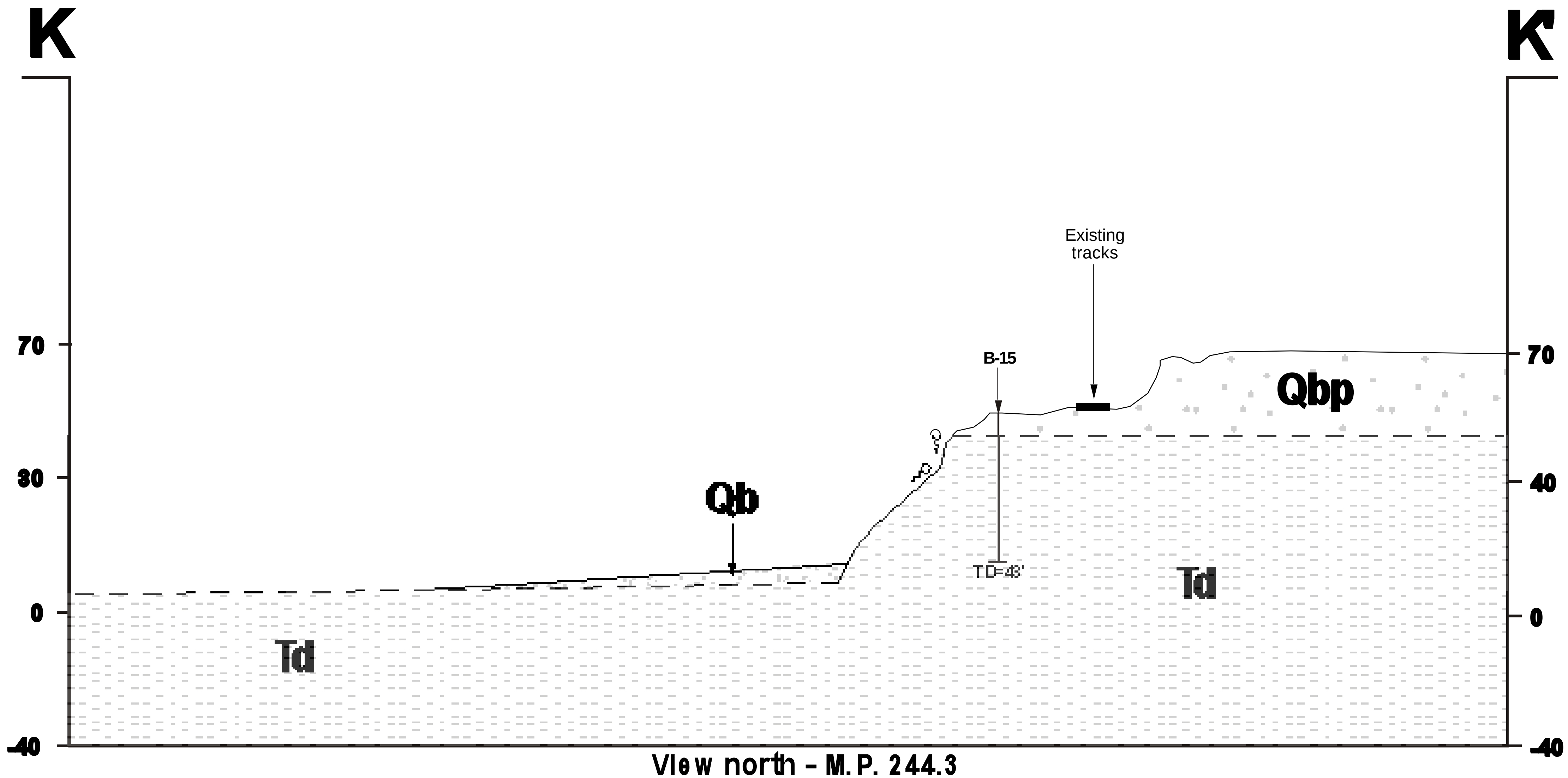
CROSS-SECTIONS I-I' AND J-J'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 14



See Figure No. 10 for Legend.

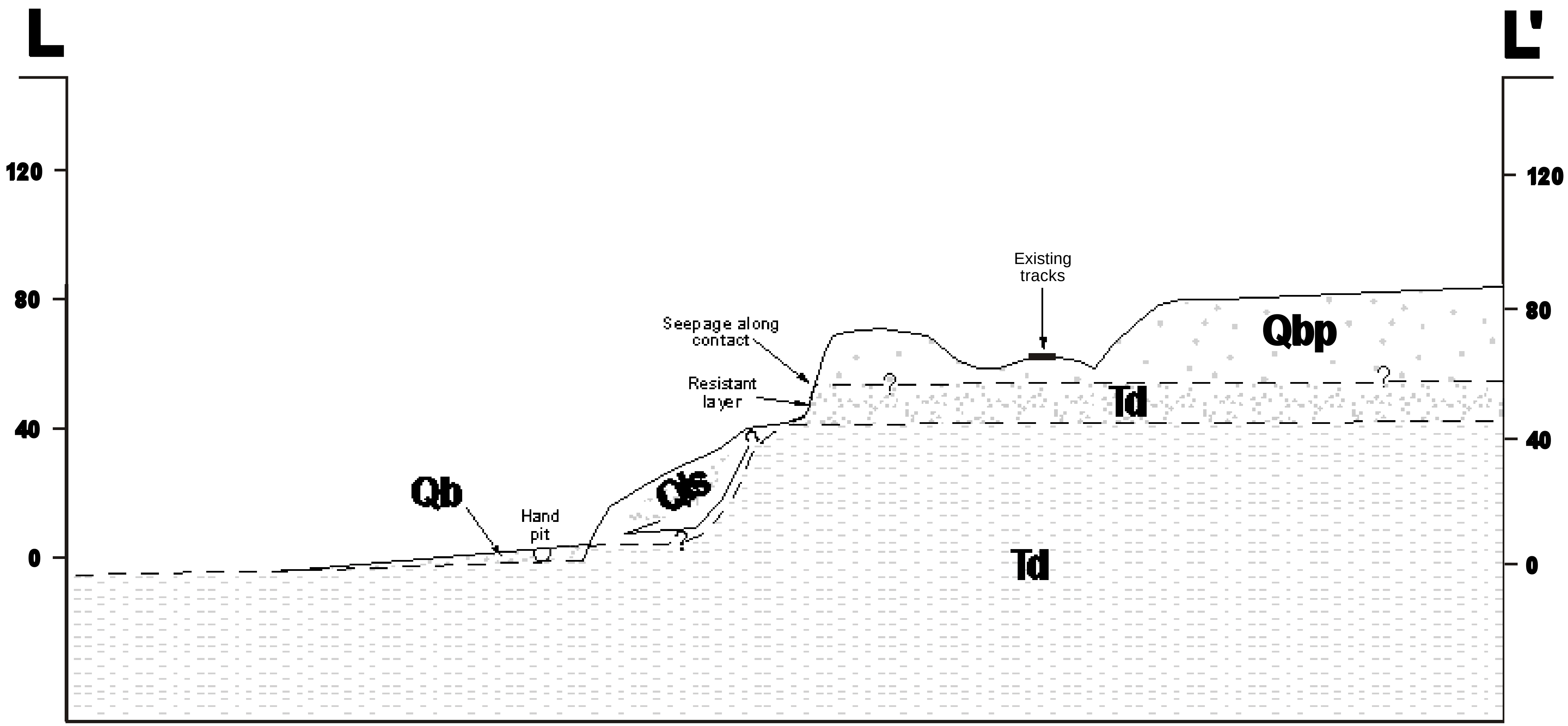
CROSS-SECTION K-K'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 15



View north - M.P. 244.64

See Figure No. 10 for Legend.

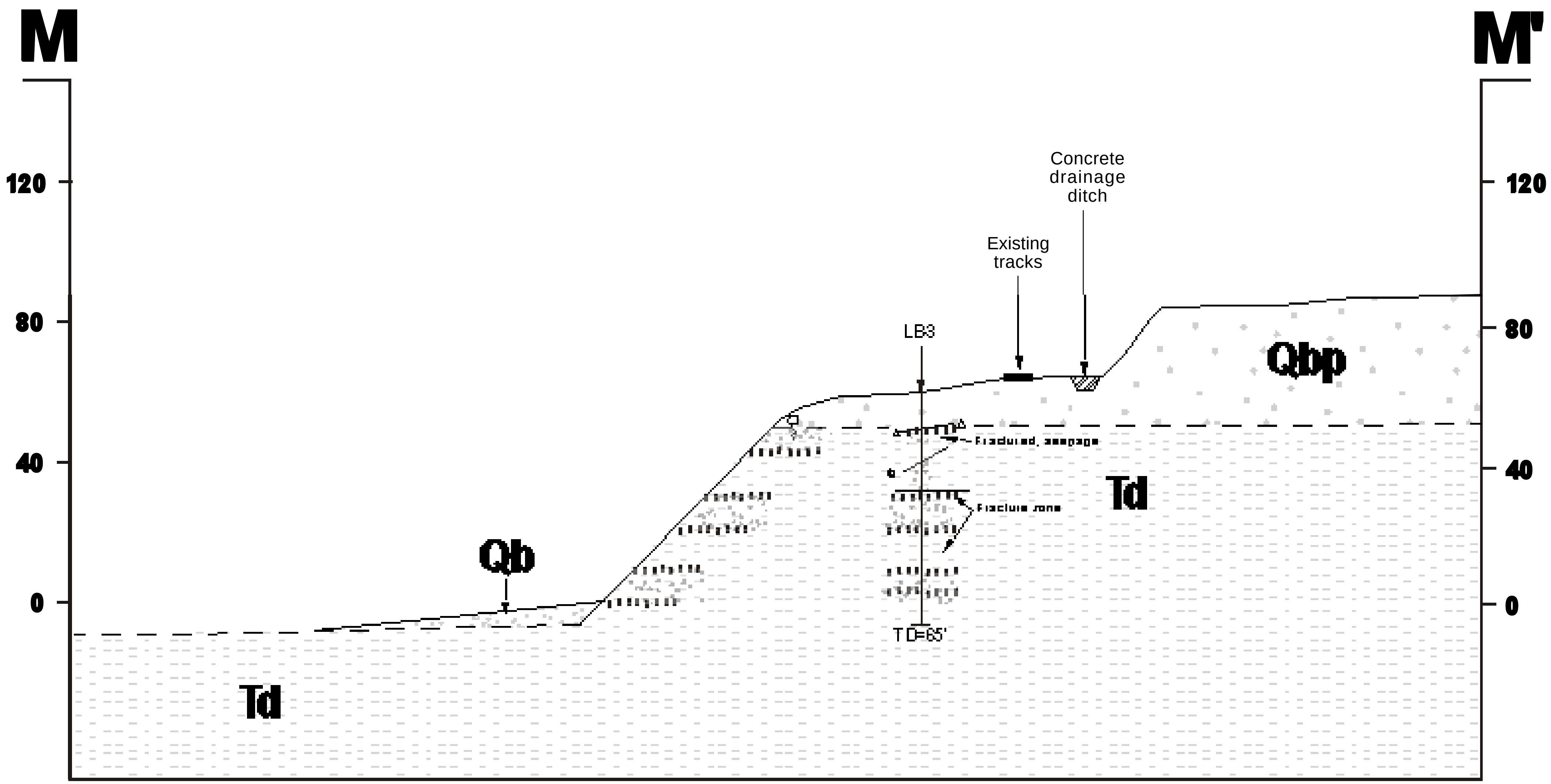
CROSS-SECTION L-L'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 16



View north - M.P. 244.79

See Figure No. 10 for Legend.

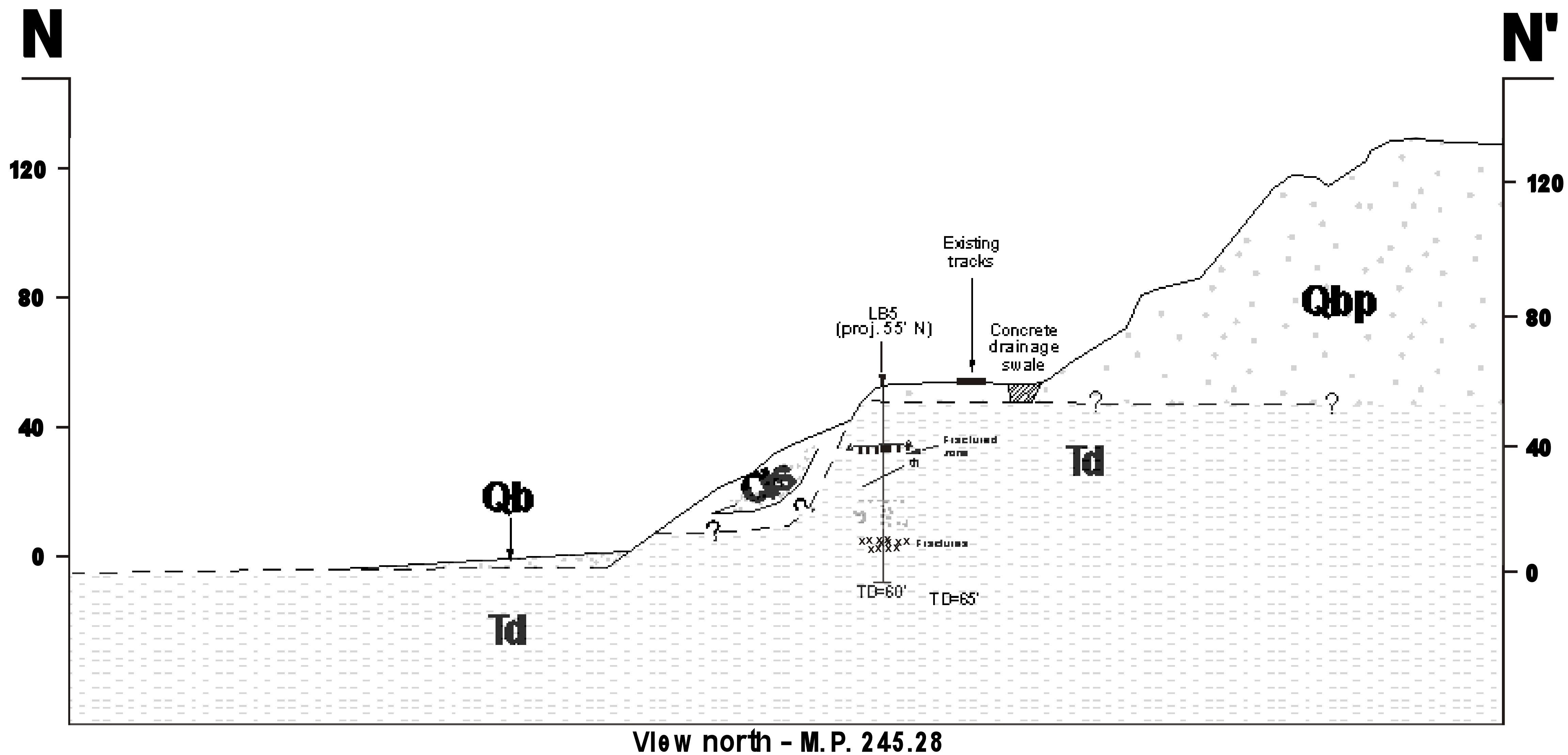
CROSS-SECTION M-M'

Del Mar Bluffs
Del Mar, California

Project No. 040151-001
Scale 1"=40'
Engr./Geol. SAC/MRS
Drafted By KAM
Date 1-7-01

Leighton and Associates, Inc.

Figure No. 17



See Figure No. 10 for Legend.

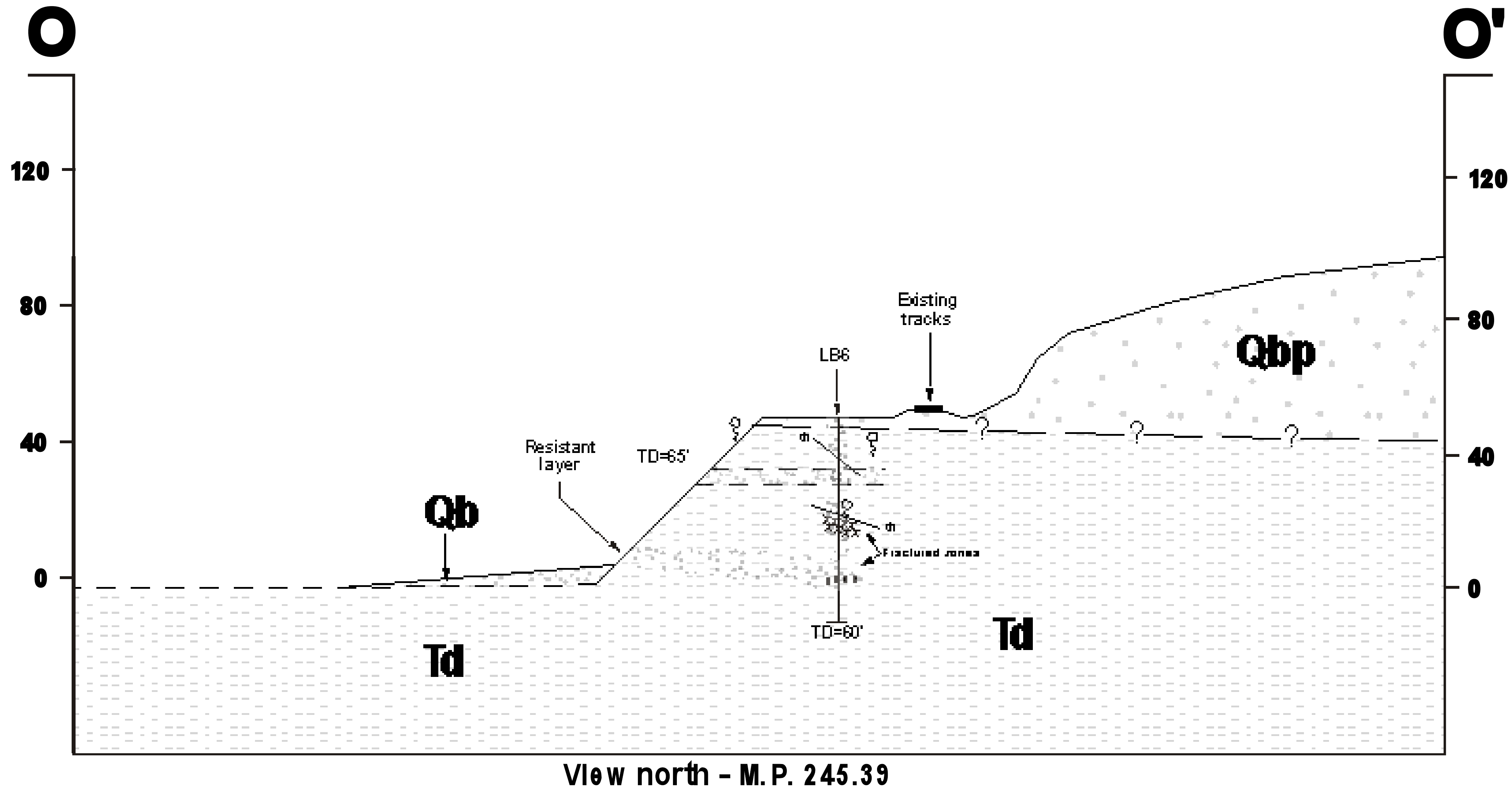
CROSS-SECTION N-N'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 18



See Figure No. 10 for Legend.

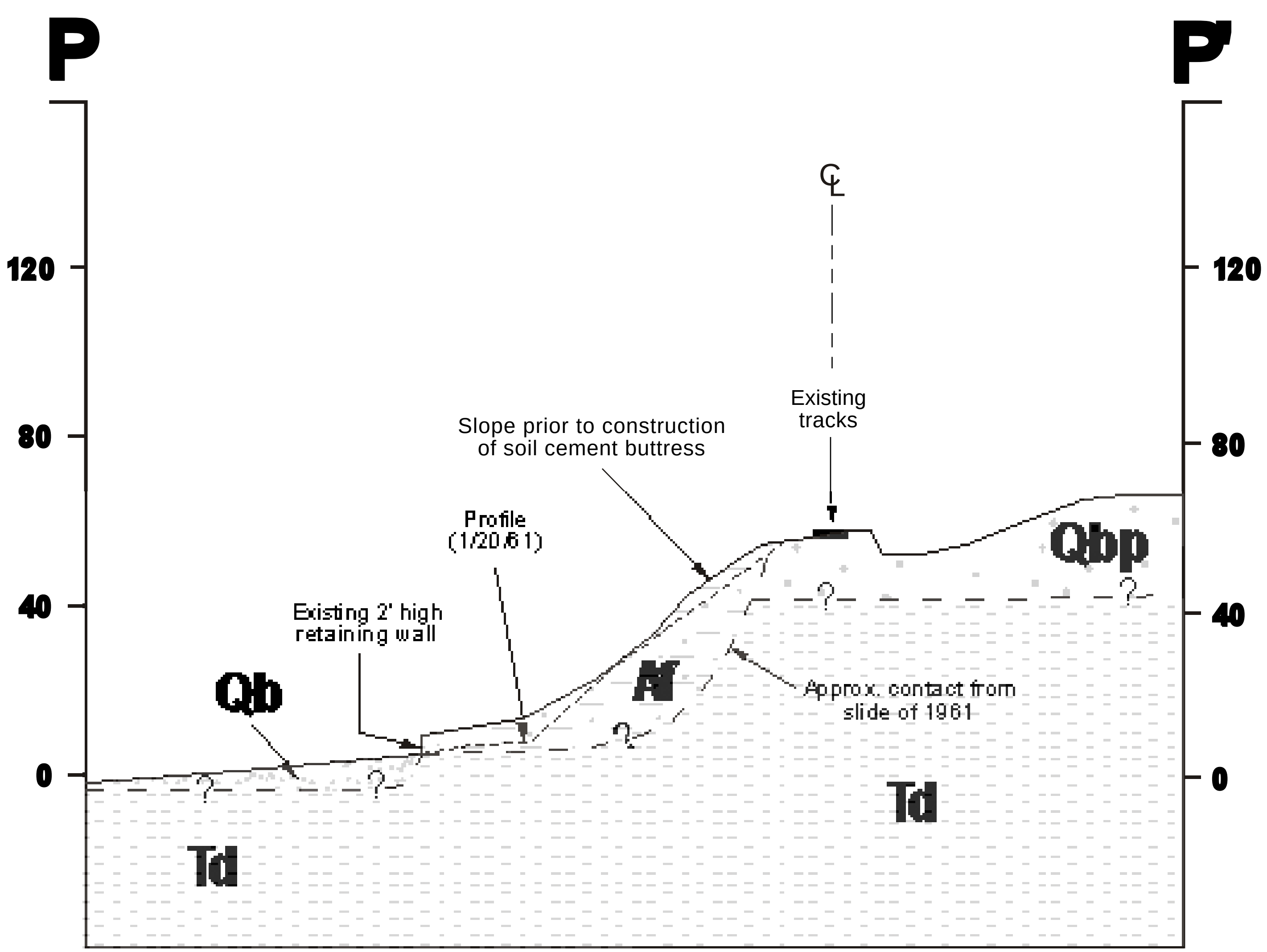
CROSS-SECTION O-O'

Del Mar Bluffs
Del Mar, California

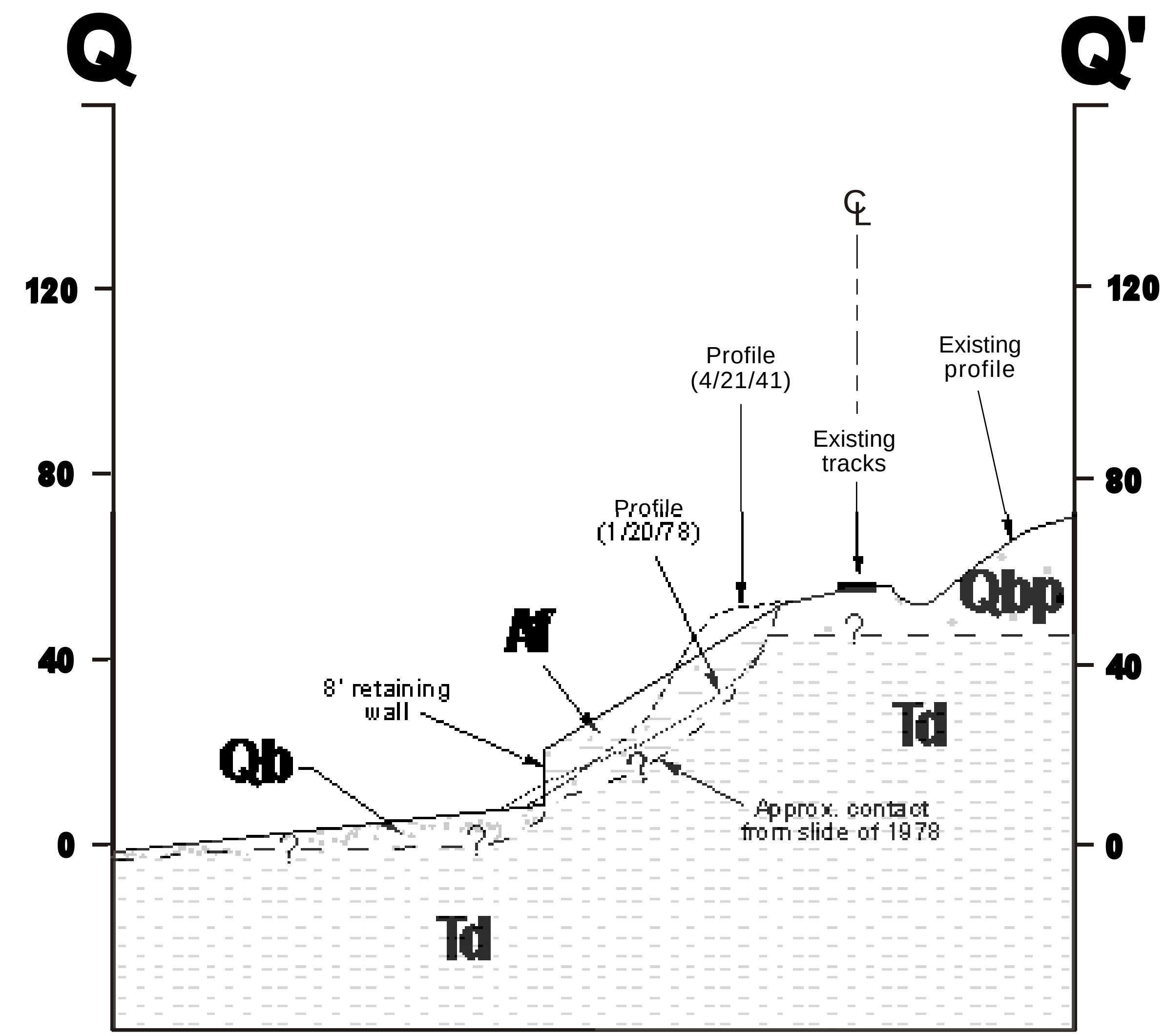
Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 19



View north - M.P. 244.44



View north - M.P. 244.33

See Figure No. 10 for Legend.

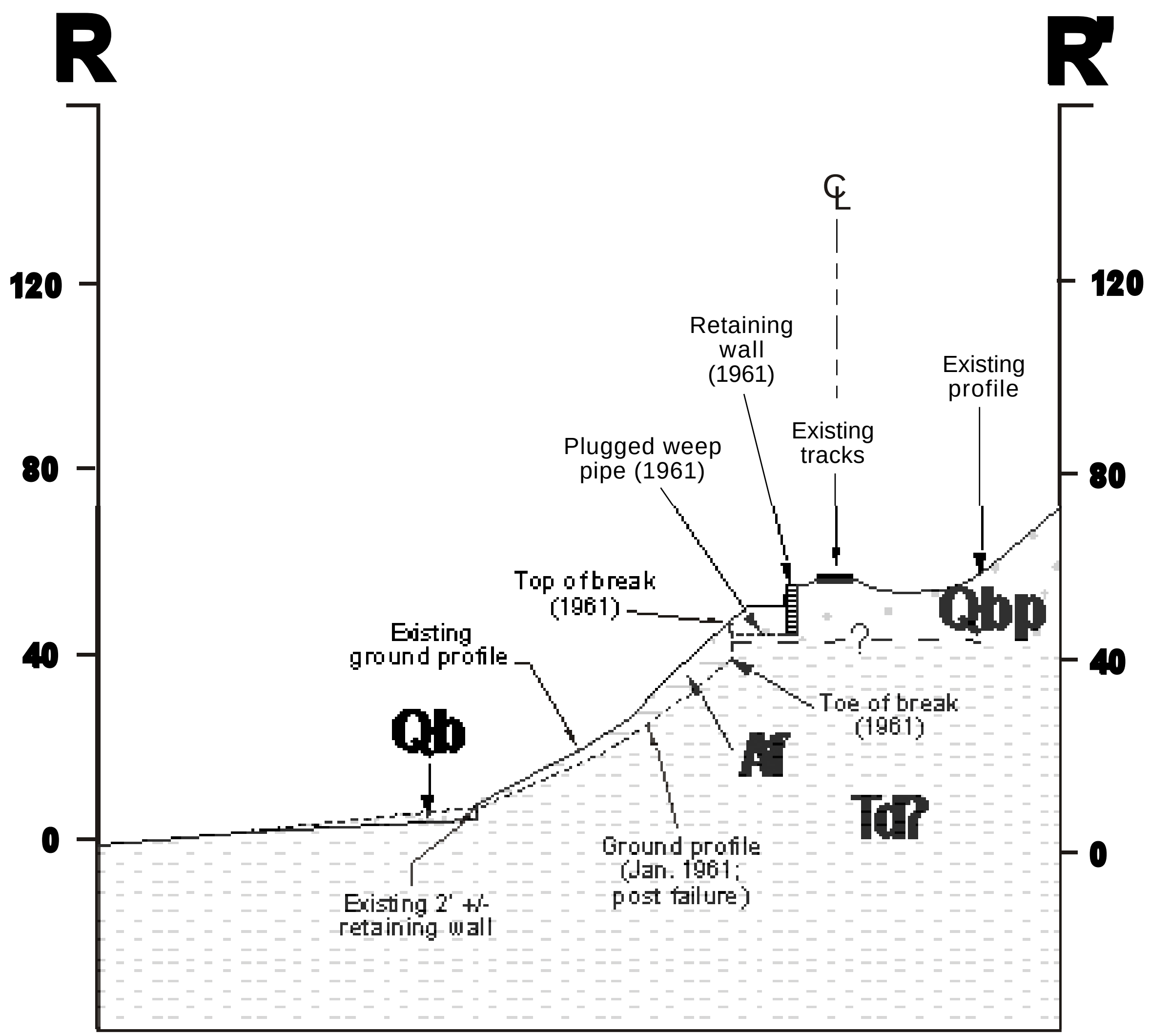
CROSS-SECTIONS P-P' AND Q-Q'

Del Mar Bluffs
Del Mar, California

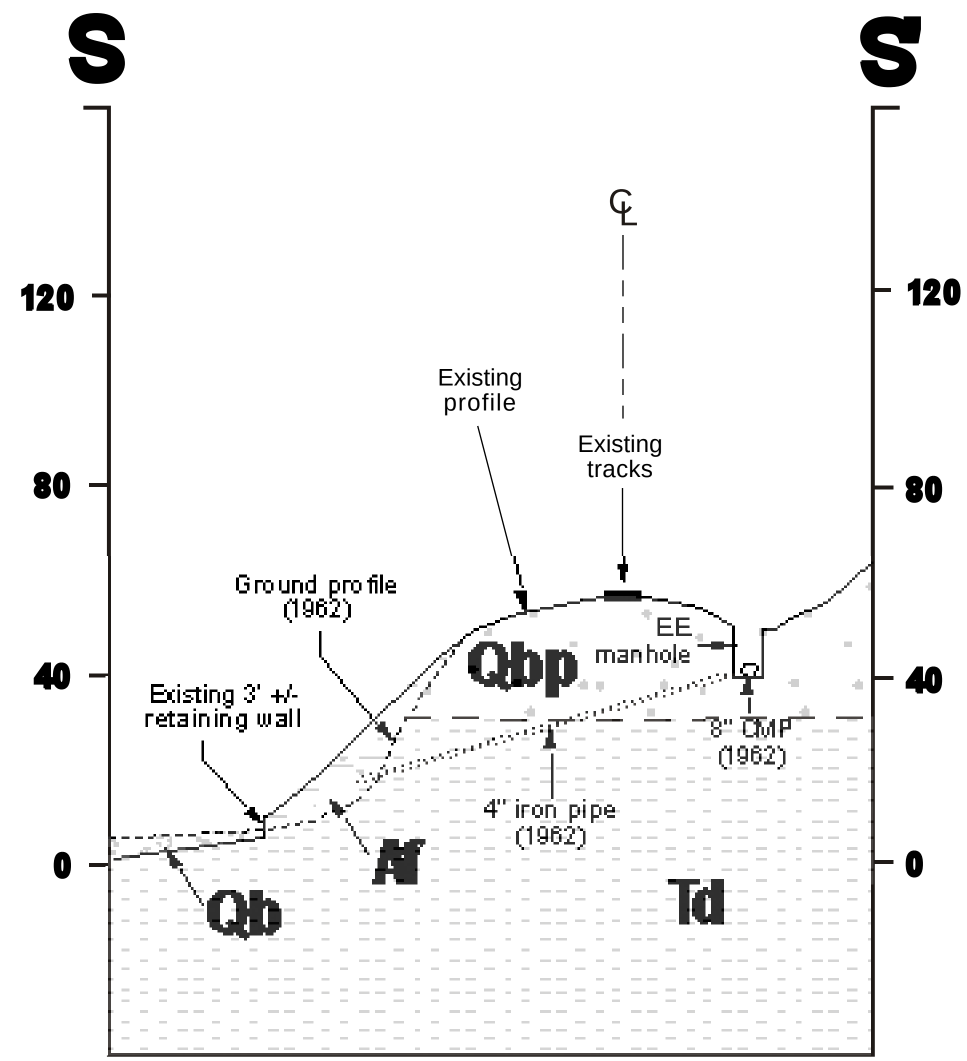
Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 20



View north - M.P. 244.42



View north - M.P. 244.40

See Figure No. 10 for Legend.

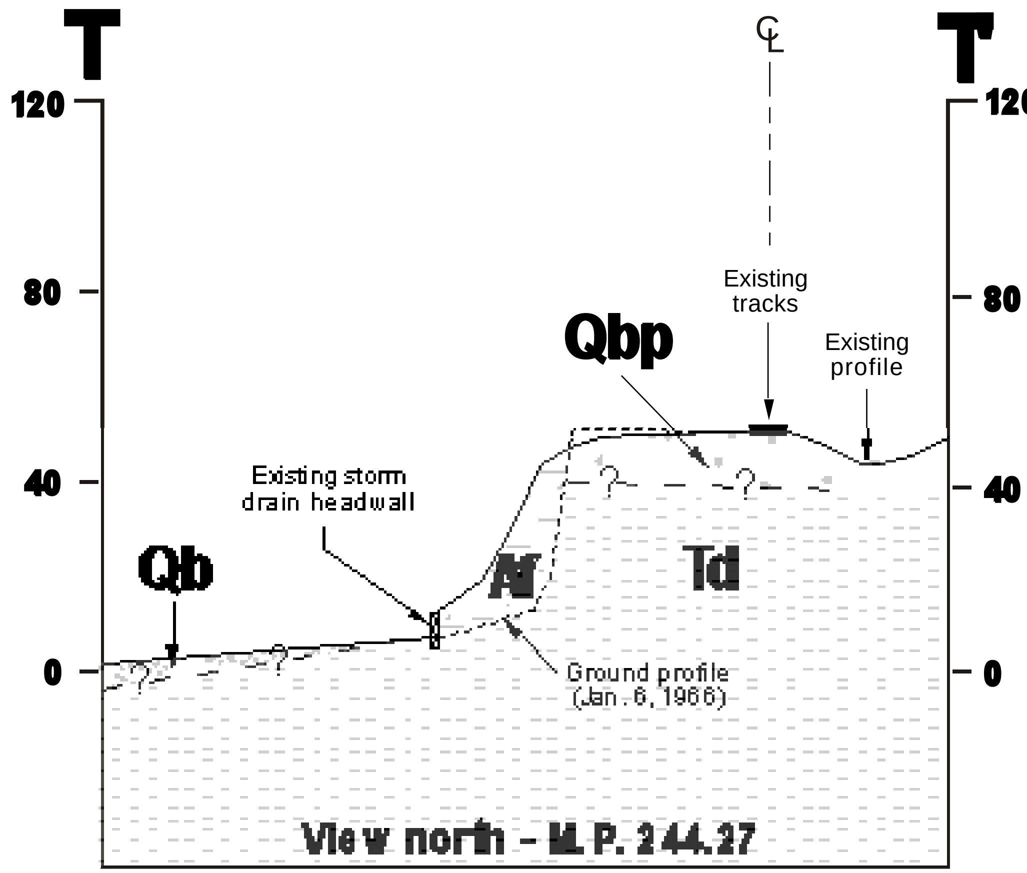
CROSS-SECTIONS R-R' AND S-S'

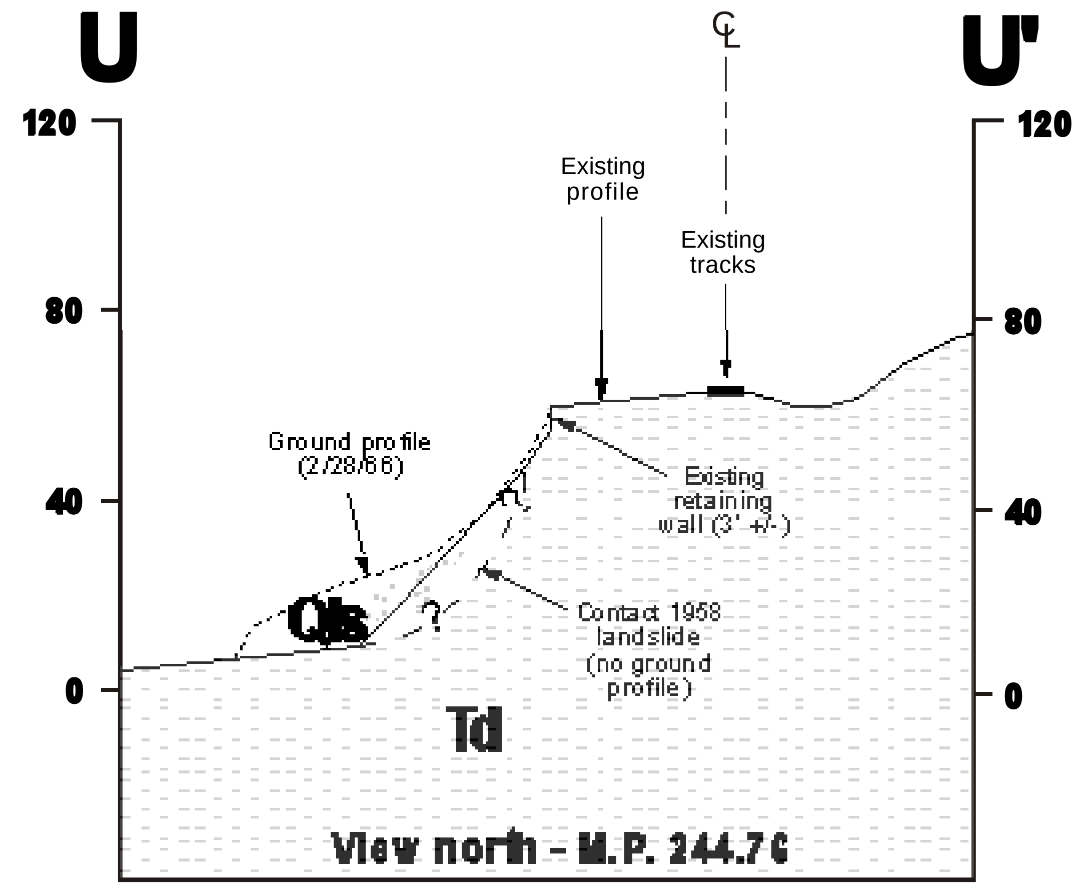
Del Mar Bluffs
Del Mar, California

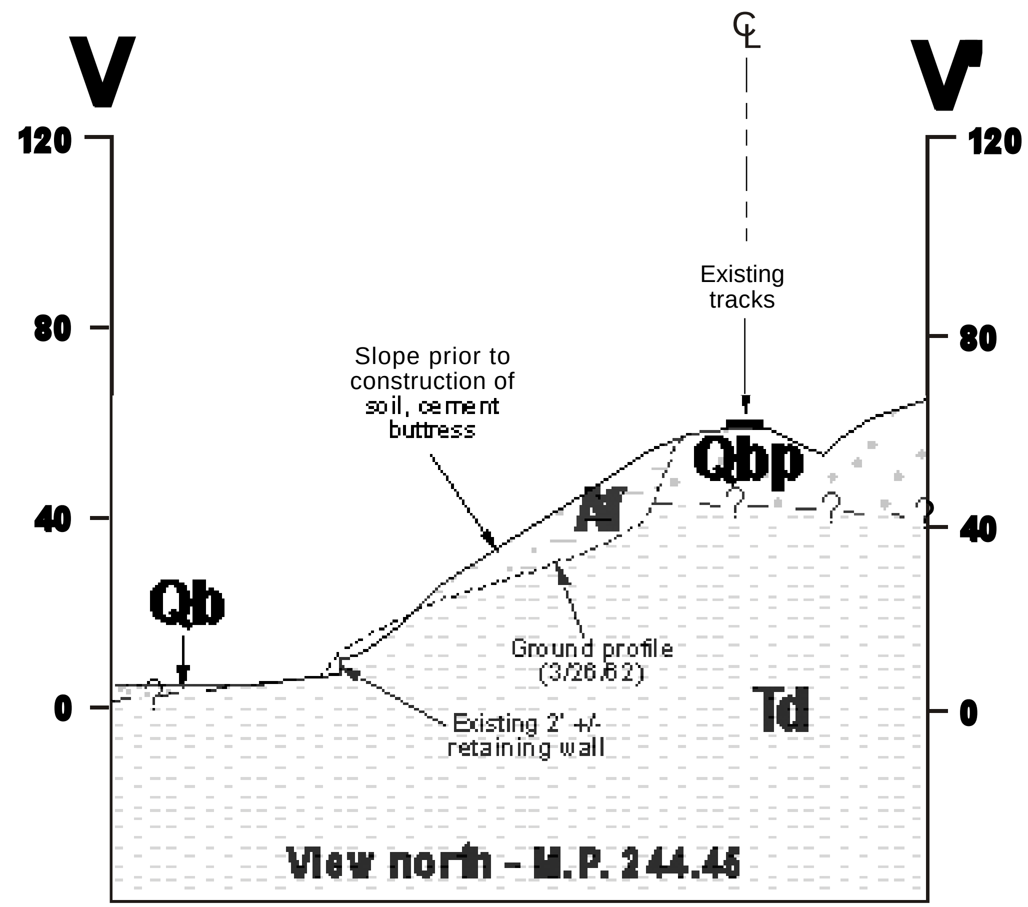
Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 21







See Figure No. 10 for Legend.

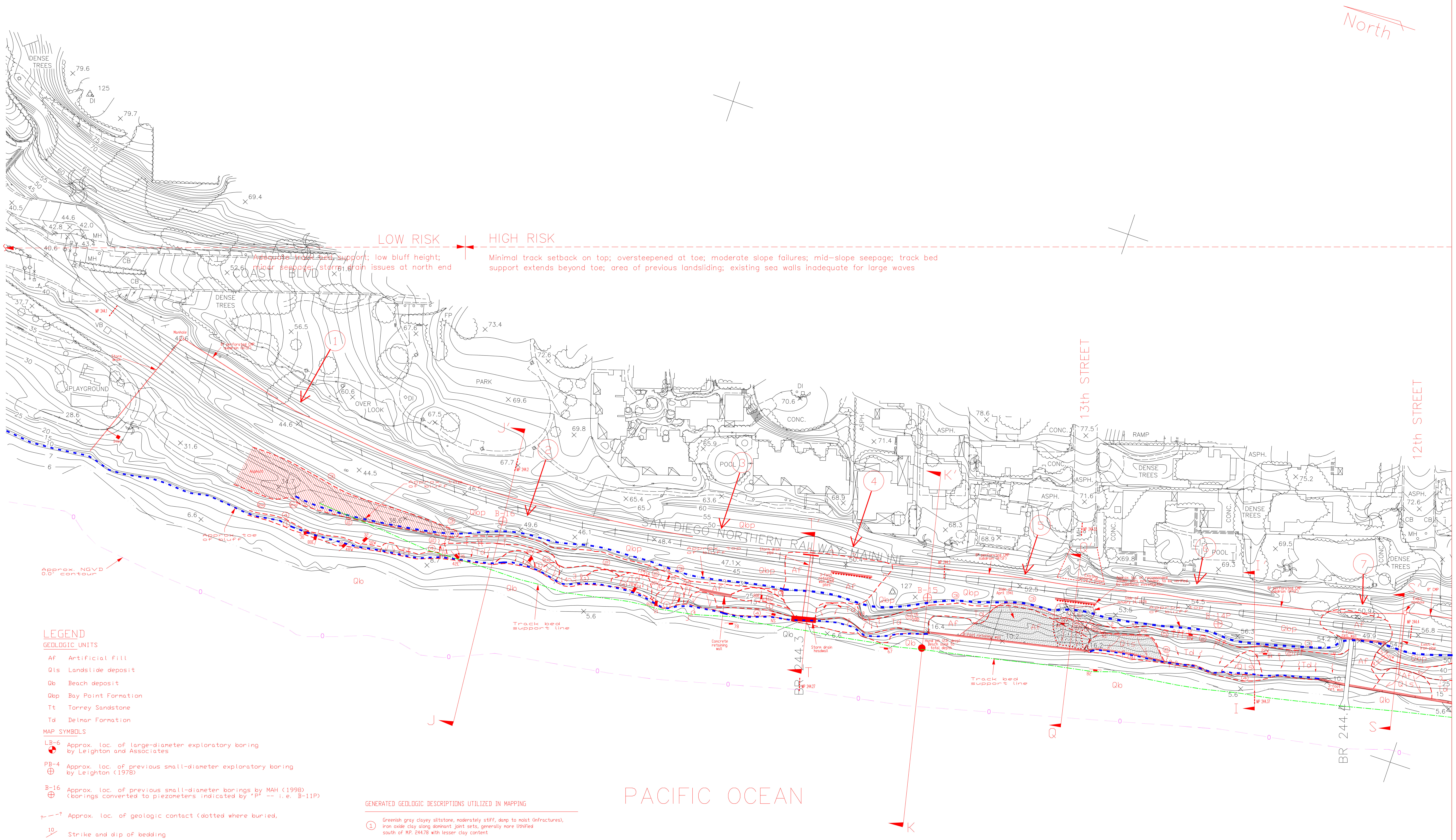
CROSS-SECTIONS T-T', U-U' AND V-V'

Del Mar Bluffs
Del Mar, California

Project No.	040151-001
Scale	1"=40'
Engr./Geol.	SAC/MRS
Drafted By	KAM
Date	1-7-01

Leighton and Associates, Inc.

Figure No. 22



LEGEND

GEOLOGIC UNITS

Af	Artificial fill
Qls	Landslide deposit
Qb	Beach deposit
Qbp	Bay Point Formation
Tt	Torrey Sandstone
Td	Delmar Formation

MAP SYMBOLS

LB-6 Approx. loc. of large-diameter exploratory boring by Leighton and Associates

PB-4 Approx. loc. of previous small-diameter exploratory boring by Leighton (1978)

B-16 Approx. loc. of previous small-diameter borings by MAH (1998) (borings converted to piezometers indicated by "P" -- i.e. B-11P)

?- - -? Approx. loc. of geologic contact (dotted where buried,

10 Strike and dip of bedding

5 Strike and dip of bedding, uncertain

2 Stike and dip of undulatory bedding

58 Strike and dip of joints

67 Fault, showing trend and plunge

Seepage

J' J Geologic cross-section

1 Risk matrix site

GENERATED GEOLOGIC DESCRIPTIONS UTILIZED IN MAPPING

1. Greenish gray clayey siltstone, moderately stiff, damp to moist (infractures), iron oxide clay along dominant joint sets, generally more lithified south of M.P. 244.78 with lesser clay content
2. Yellowish light brown fine sandy silt, stiff to very stiff, dry to Sl damp, lithifed in areas as marker bed

2b. Non-lithified version of 2

3. Reddish brown fine to medium sandstone with trace silt, very loose to very dense dry to damp
4. Gray/light brown fine to medium sandstone, dense, damp, gradual fining upwards

NOTE:

Subdrains shown on map based on review of ATSF documents (ATSF, 1978); not all subdrains observed during field mapping

040151-001

NCTD

LEIGHTON AND ASSOCIATES, INC.
3934 Murphy Canyon Road
Suite B204
San Diego, CA 92123

HDR Engineering, Inc.
One City Boulevard West
Suite 900
Orange, CA 92868

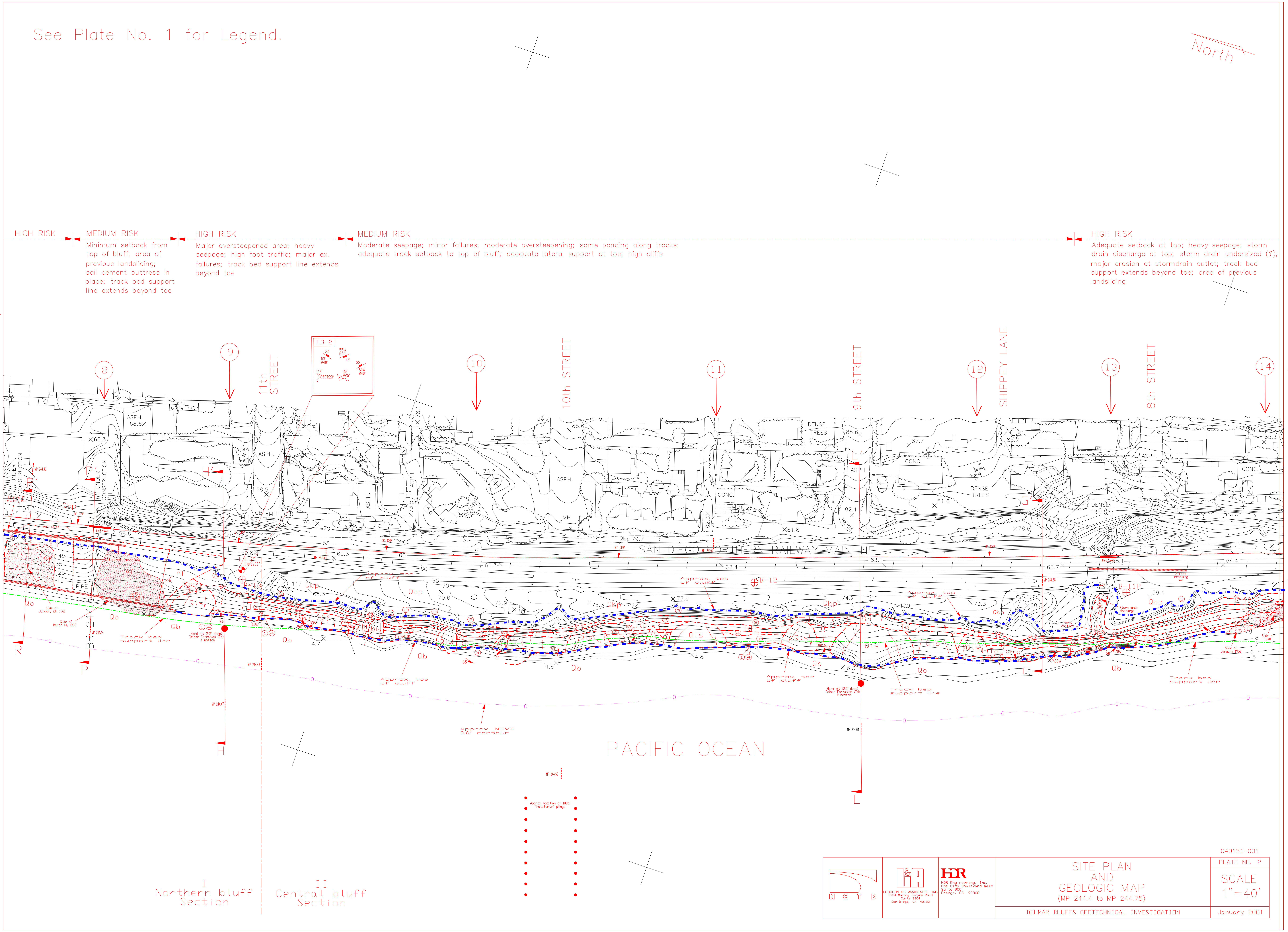
SITE PLAN AND GEOLOGIC MAP
(Coast Blvd. to MP 244.4)

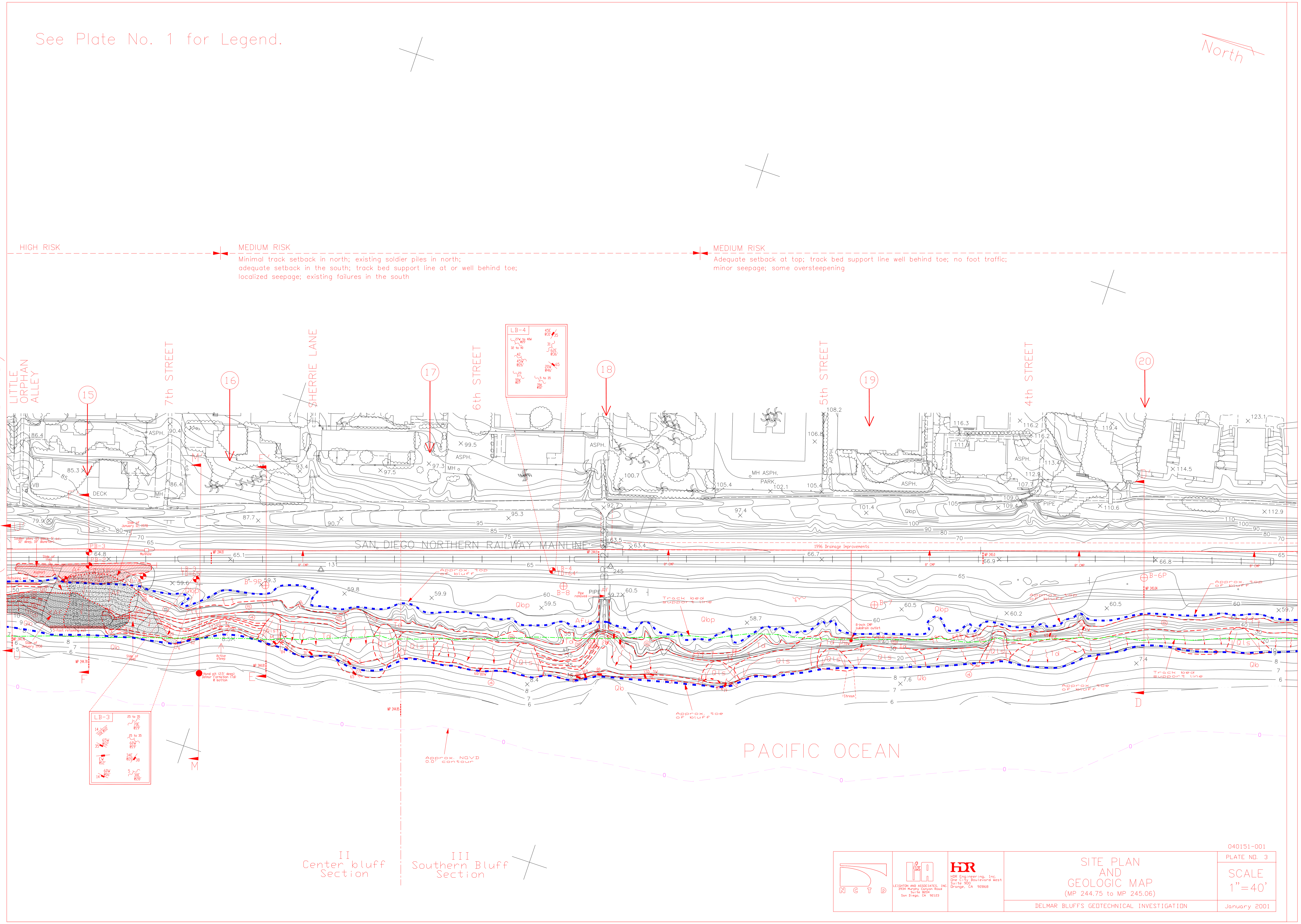
DELMAR BLUFFS GEOTECHNICAL INVESTIGATION

PLATE NO. 1

SCALE 1"=40'

January 2001


See Plate No. 1 for Legend.
North
HIGH RISK
MEDIUM RISK
Minimum setback from top of bluff; area of previous landsliding; soil cement buttress in place; track bed support line extends beyond toe
HIGH RISK
Major oversteepened area; heavy seepage; high foot traffic; major ex. failures; track bed support line extends beyond toe
MEDIUM RISK
Moderate seepage; minor failures; moderate oversteepening; some ponding along tracks; adequate track setback to top of bluff; adequate lateral support at toe; high cliffs
HIGH RISK
Adequate setback at top; heavy seepage; storm drain discharge at top; storm drain undersized (?); major erosion at stormdrain outlet; track bed support extends beyond toe; area of previous landsliding
MATCHLINE (SEE SHEET 1 OF 5)
MATCHLINE (SEE SHEET 3 OF 5)
8
9
10
11
12
13
14
11th STREET
10th STREET
9th STREET
SHIPPEY LANE
8th STREET
LB-2
UNDER CONSTRUCTION
ASPH.
CONC.
DENSE TREES
SAN DIEGO NORTHERN RAILWAY MAINLINE
Approx. top of bluff
Approx. toe of bluff
Track bed support line
Approx. NGVD 0.0' contour
Hand pit (2.5' deep); Delmar Formation (Td) @ bottom
B-12
B-11P
Storm drain discharge
Qbp
Qb
Qls
Af
PIPE
R
P
H
L
G
PACIFIC OCEAN
Approx. location of 1885 "Natatorium" pilings
I
Northern bluff Section
II
Central bluff Section
040151-001
N C T D
LEIGHTON AND ASSOCIATES, INC.
HDR
HDR Engineering, Inc. One City Boulevard West Suite 900 Orange, CA 92868
SITE PLAN AND GEOLOGIC MAP
(MP 244.4 to MP 244.75)
DELMAR BLUFFS GEOTECHNICAL INVESTIGATION
PLATE NO. 2
SCALE 1"=40'
January 2001



See Plate No. 1 for Legend.
North
HIGH RISK
MEDIUM RISK
Minimal track setback in north; existing soldier piles in north; adequate setback in the south; track bed support line at or well behind toe; localized seepage; existing failures in the south
MEDIUM RISK
Adequate setback at top; track bed support line well behind toe; no foot traffic; minor seepage; some oversteepening
MATCHLINE (SEE SHEET 2 OF 5)
MATCHLINE (SEE SHEET 4 OF 5)
LITTLE ORPHAN ALLEY
7th STREET
SHERRIE LANE
6th STREET
5th STREET
4th STREET
15
16
17
18
19
20
LB-4
LB-3
SAN DIEGO NORTHERN RAILWAY MAINLINE
1996 Drainage Improvements
Approx. top of bluff
Track bed support line
Approx. toe of bluff
Approx. NGVD 0.0' contour
PACIFIC OCEAN
II Center bluff Section
III Southern Bluff Section
SITE PLAN AND GEOLOGIC MAP
(MP 244.75 to MP 245.06)
DELMAR BLUFFS GEOTECHNICAL INVESTIGATION
040151-001
PLATE NO. 3
SCALE 1"=40'
January 2001

See Plate No. 1 for Legend.

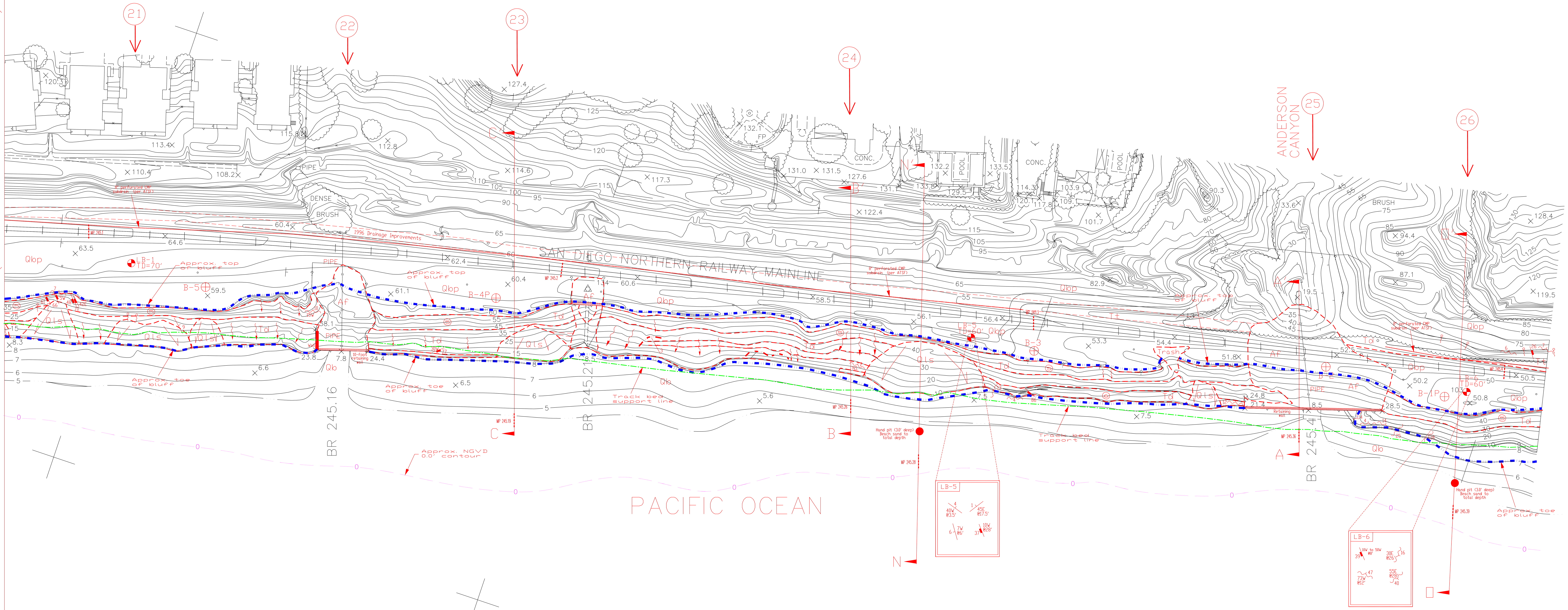
North

MATCHLINE (SEE SHEET 3 OF 5)

MEDIUM RISK
Adequate setback at top; track bed support line well behind toe; no foot traffic; minor seepage; moderate oversteepening; minor existing failures

HIGH RISK
Adequate setback at track; track bed support line extends beyond toe; maintenance of sea wall at Anderson Canyon critical; moderate oversteenening; 1.4:1 to 0.9:1; minor seepage moderate foot traffic; moderate oversteepening; track bed support line extends beyond toe

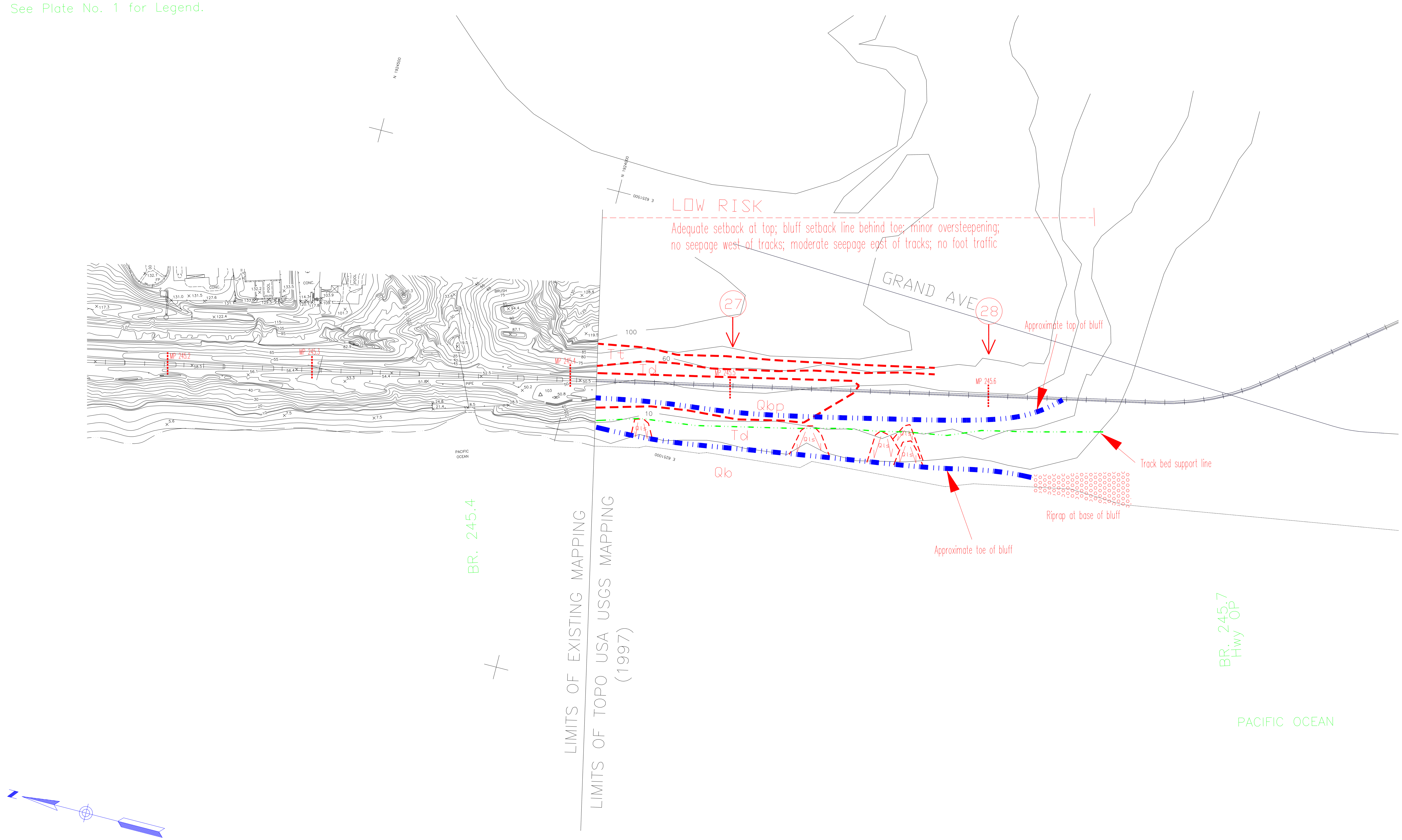
LOW RISK
Adequate setback at top; lateral support line generally behind toe; minor oversteepening; no seepage; no foot traffic



040151-001

N C T D	LEIGHTON AND ASSOCIATES, INC. 3934 Murphy Canyon Road Suite B204 San Diego, CA 92123	HDR Engineering, Inc. One City Boulevard West Suite 900 Orange, CA 92868	SITE PLAN AND GEOLOGIC MAP (MP 245.06 to MP 245.4)	PLATE NO. 4
				SCALE 1"=40'
			DELMAR BLUFFS GEOTECHNICAL INVESTIGATION	January 2001

See Plate No. 1 for Legend.



040151-001

NCTD	LEIGHTON AND ASSOCIATES, INC. 3934 Murphy Canyon Road Suite B204 San Diego, CA 92123	HDR Engineering, Inc. One City Boulevard West Suite 900 Orange, CA 92868	SITE PLAN AND GEOLOGIC MAP (MP 245.4 to Hwy. 101 Overcrossing)	PLATE NO. 5
				SCALE 1"=100'
			DELMAR BLUFFS GEOTECHNICAL INVESTIGATION	January 2001

APPENDIX A

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APPENDIX A

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Aerial Photographs

Agency	Date	Flight No.	Photo Nos.
GTI	July 29, 1990		41-44, 49-51, 54-60 (oblique photos)
GTI	January 28, 1999		86-95 (oblique photos)
GTI	November 26, 1969	15	37a BU, 37b BU
USDA	1953	AXN-8M	82 and 84
GTI	November 25, 1969	15	37a BU, 37b BU

APPENDIX B

SITE PHOTOGRAPHS

PHOTO NO. 1
Areas of water-loving plants in northern bluff section.

PHOTO NO. 2
Ponded water east of tracks, northern section.

PHOTO NO. 3
Erosional gulley near tracks, northern section.

PHOTO NO. 4
View north from the central section. Note wide beach, soil cement buttress, and areas of dense vegetation.

PHOTO NO. 5

Close proximity of tracks to bluff top, northern section.

PHOTO NO. 6

Wall supporting tracks, northern section.

PHOTO NO. 7

Resistant layer at base of bluff and mature vegetation.

PHOTO NO. 8

Wooden seawall, northern section. Note resistant layer at base of bluff.

PHOTO NO. 9
Dense vegetation on bluff. Note resistant basal layer in foreground.

PHOTO NO. 10
Resistant unit at base of bluff, dense vegetation above.

PHOTO NO. 11
View of flat shore platform in central bluff section.

PHOTO NO. 12
Horizontal shore platform in central bluff section. Note wooden pilings from 1885 "Natatorium".

PHOTO NO. 13

Erosional gulley in central section resulting from concentrated surface runoff.

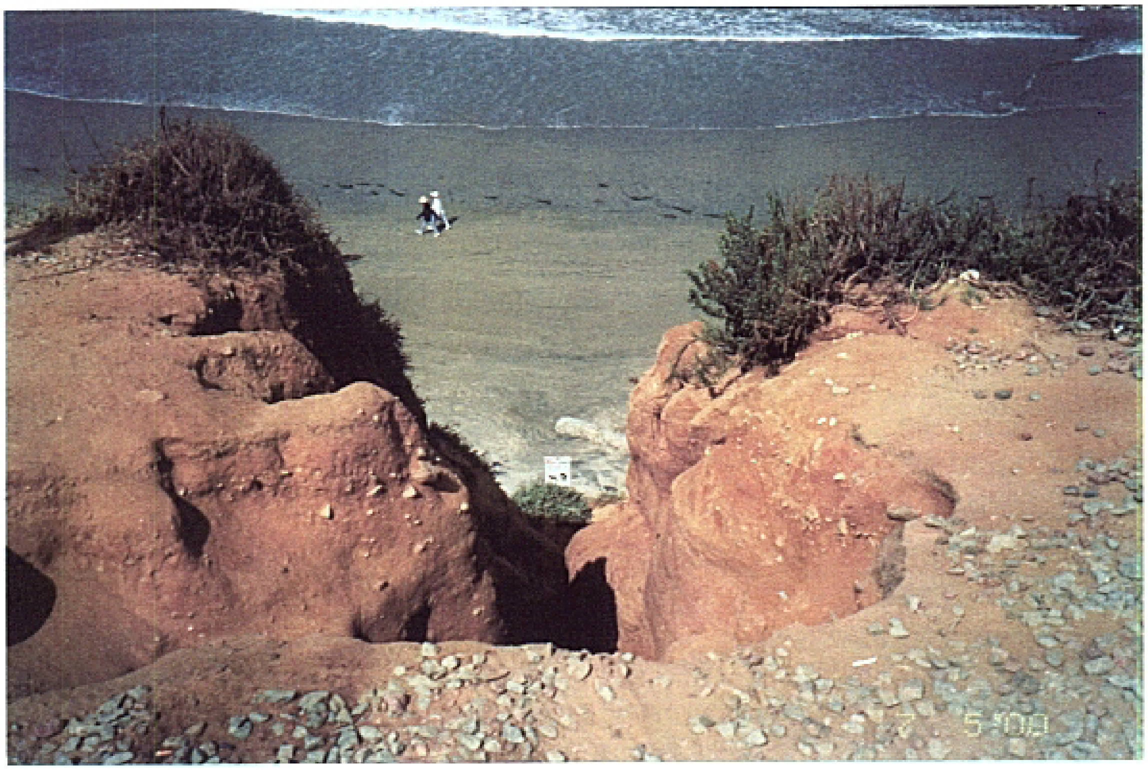


PHOTO NO. 14

Erosion along footpath to beach.

PHOTO NO. 15

Erosion caused by bluff seepage; red sand generated from erosion of terrace deposits.

PHOTO NO. 16

Evidence of ponding on east side of tracks in central bluff section.

PHOTO NO. 17

Localized area of notching in central section of bluff.

PHOTO NO. 18

Post construction erosion at base and side of structure in southern section of bluff. Post construction erosion measured at 16 to 18 inches.

PHOTO NO. 19

View of southern section. Note slight embayment.

PHOTO NO. 20

Geologist standing on level shore platform.

PHOTO NO. 21

View of southern section. Note how beach forms partial embayment.

PHOTO NO. 22

View from central section to the south.

PHOTO NO. 23
Debris resulting from block fall from upper section. Note seepage at base of terrace deposits.

PHOTO NO. 24
View of bluff at southern end of central section. Note lack of vegetation to the south.

PHOTO NO. 25

View of 8-inch CMP subdrain discharging water at bluff top.

PHOTO NO. 26

Typical weathering of exposed bedrock (Delmar Formation).

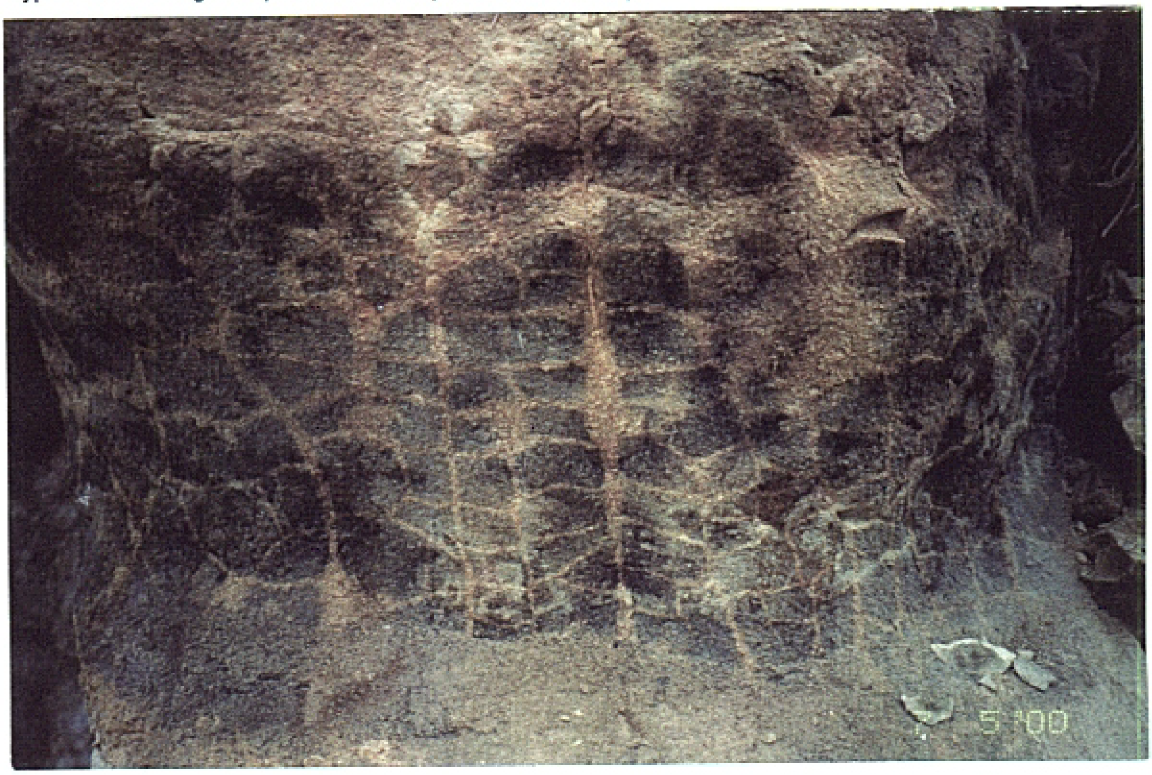


PHOTO NO. 27

Storm drain discharging to east side of tracks in southern section.

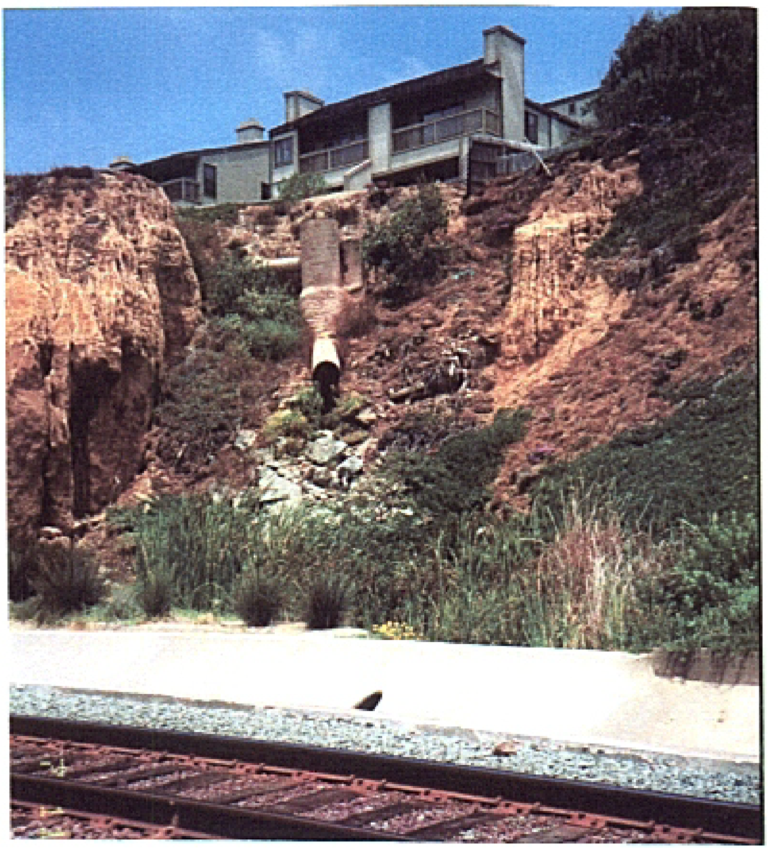


PHOTO NO. 28

Concrete drainage ditch in southern section. Note seepage at weep holes.

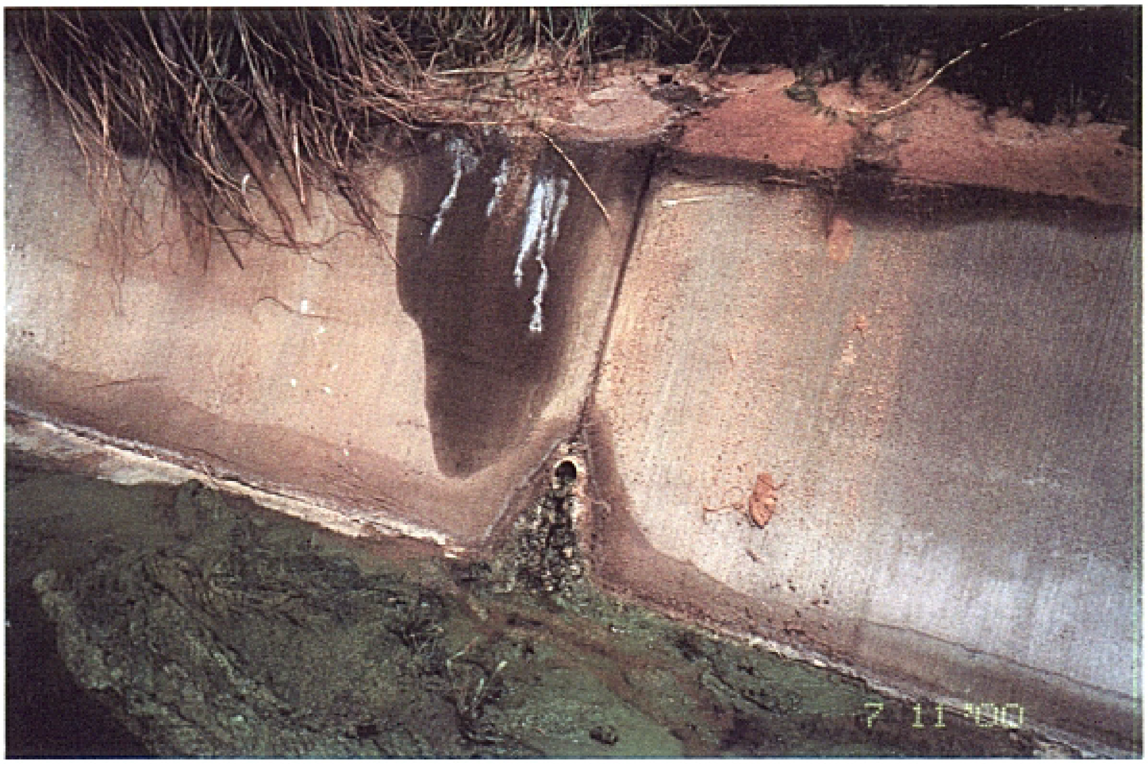


PHOTO NO. 29

Failure in upper portion of bluff.

PHOTO NO. 30

Bowl-shaped erosion in southern section, caused by now removed storm drain. Note that erosion stopped at resistant layer that extends across base of bluff.

PHOTO NO. 31

Storm drain discharging at top of bluff MP 244.7. Outlet structure requires upgrading and repair.

PHOTO NO. 32

Bluff south of Anderson Canyon in southern section. Note resistant layer at base of bluff.

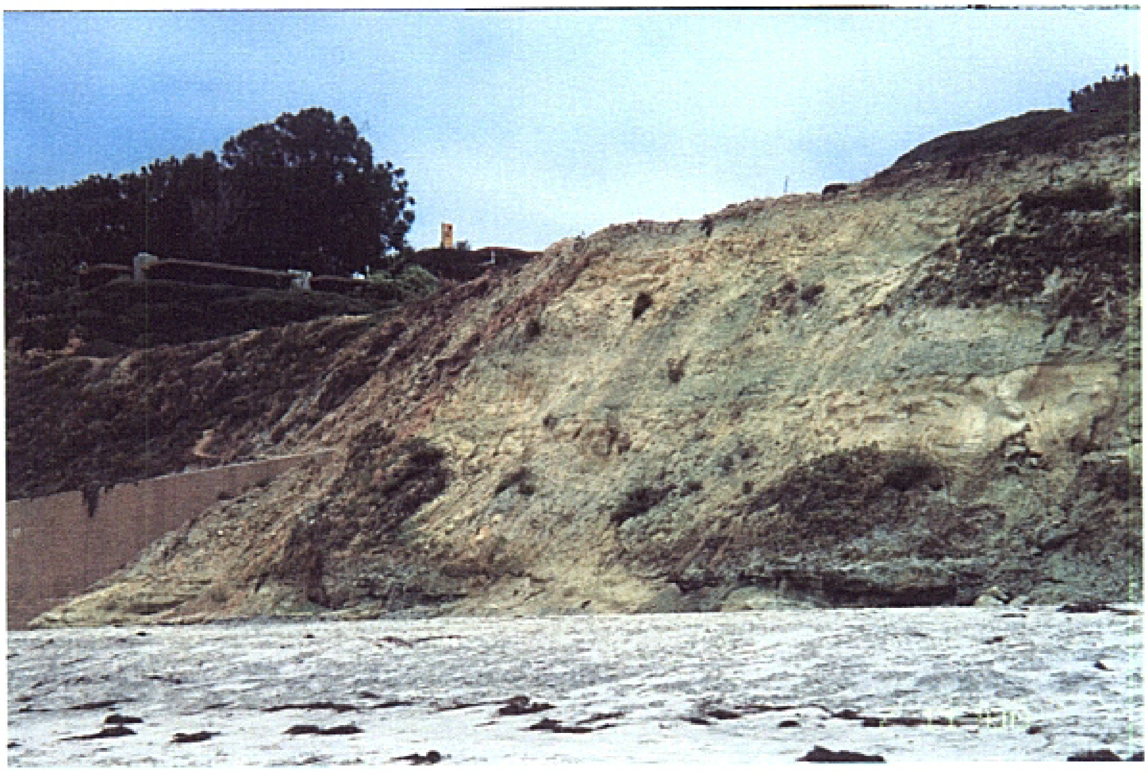


PHOTO NO. 33

PHOTO NO. 34

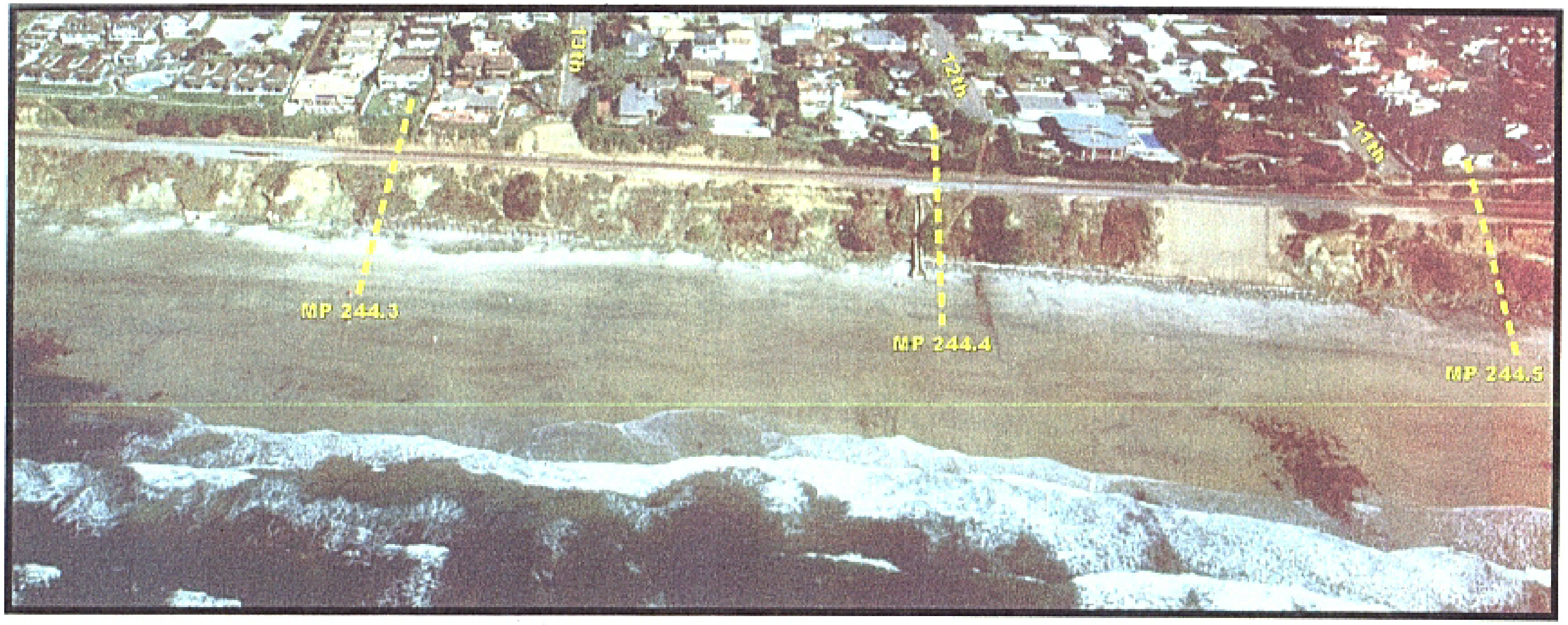
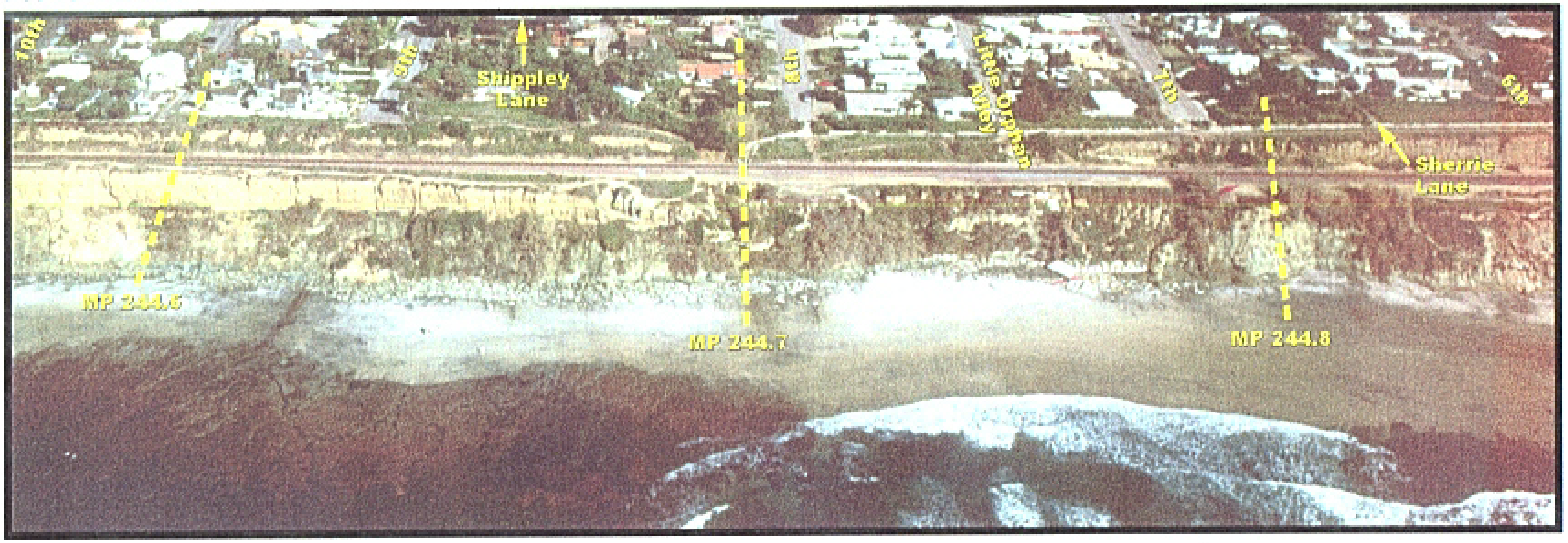


PHOTO NO. 35

PHOTO NO. 36

PHOTO NO. 37



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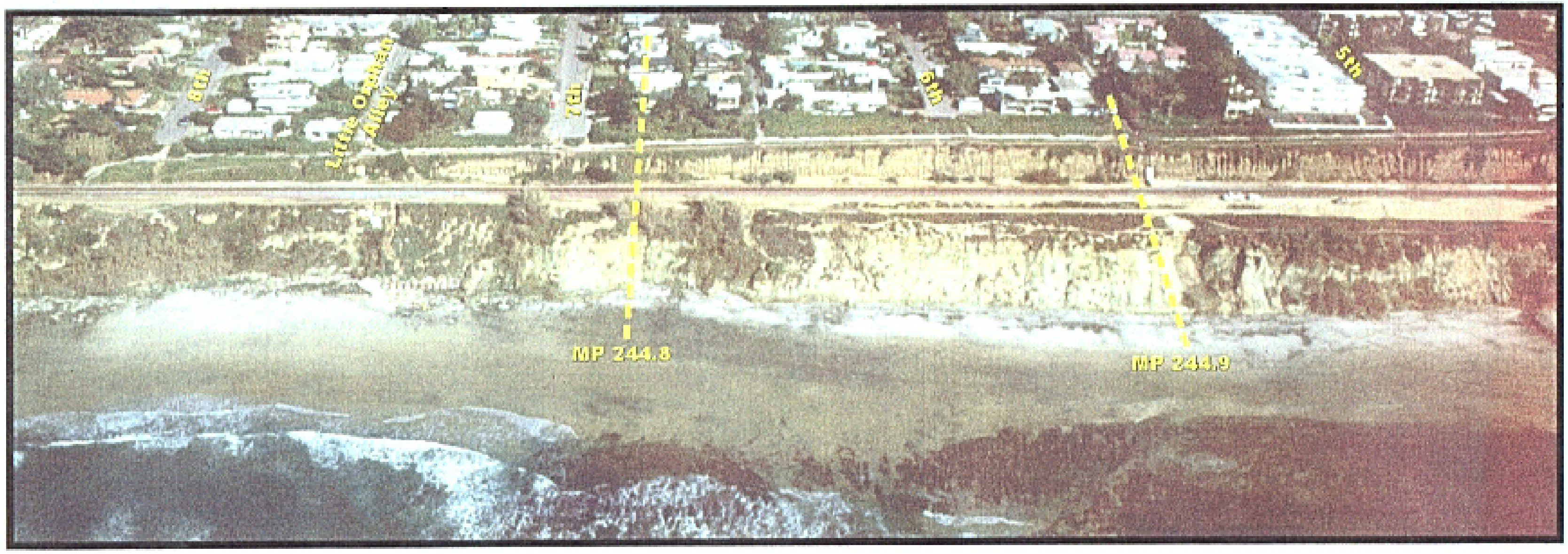


PHOTO NO. 39

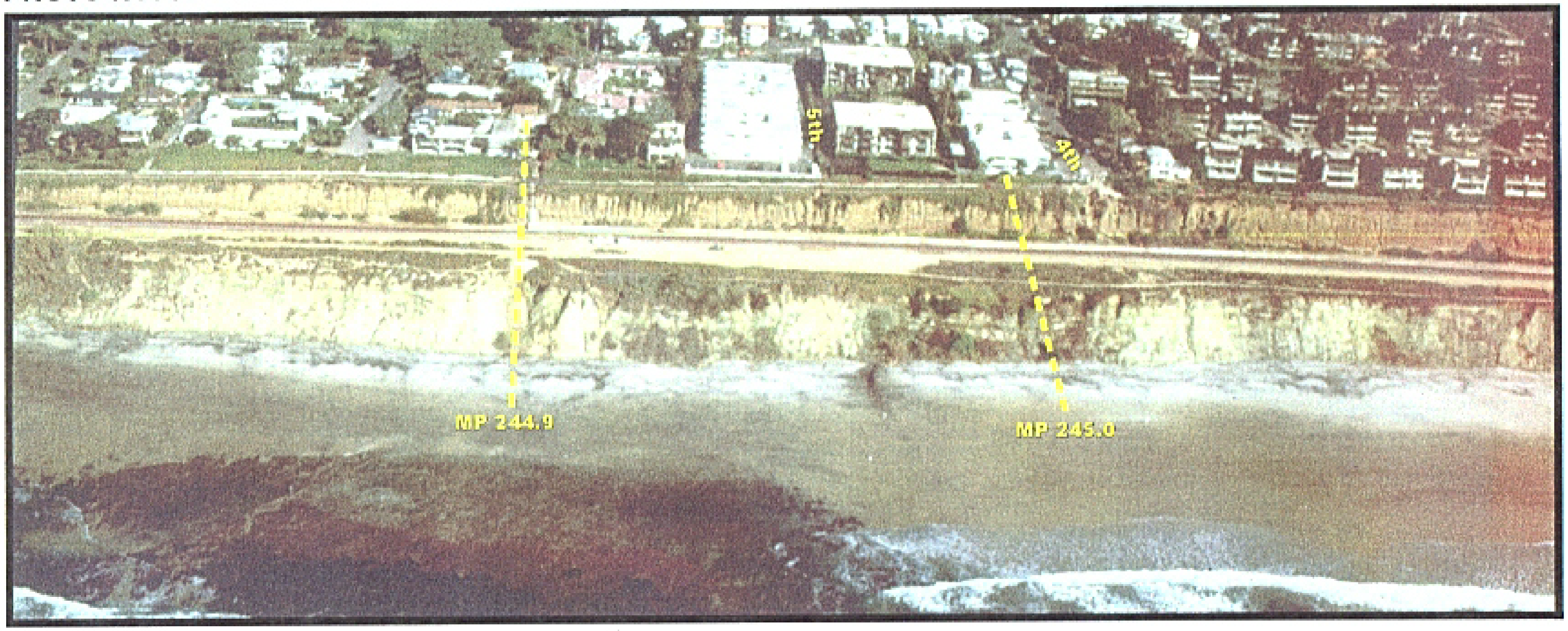


PHOTO NO. 40

PHOTO NO. 41

PHOTO NO. 42

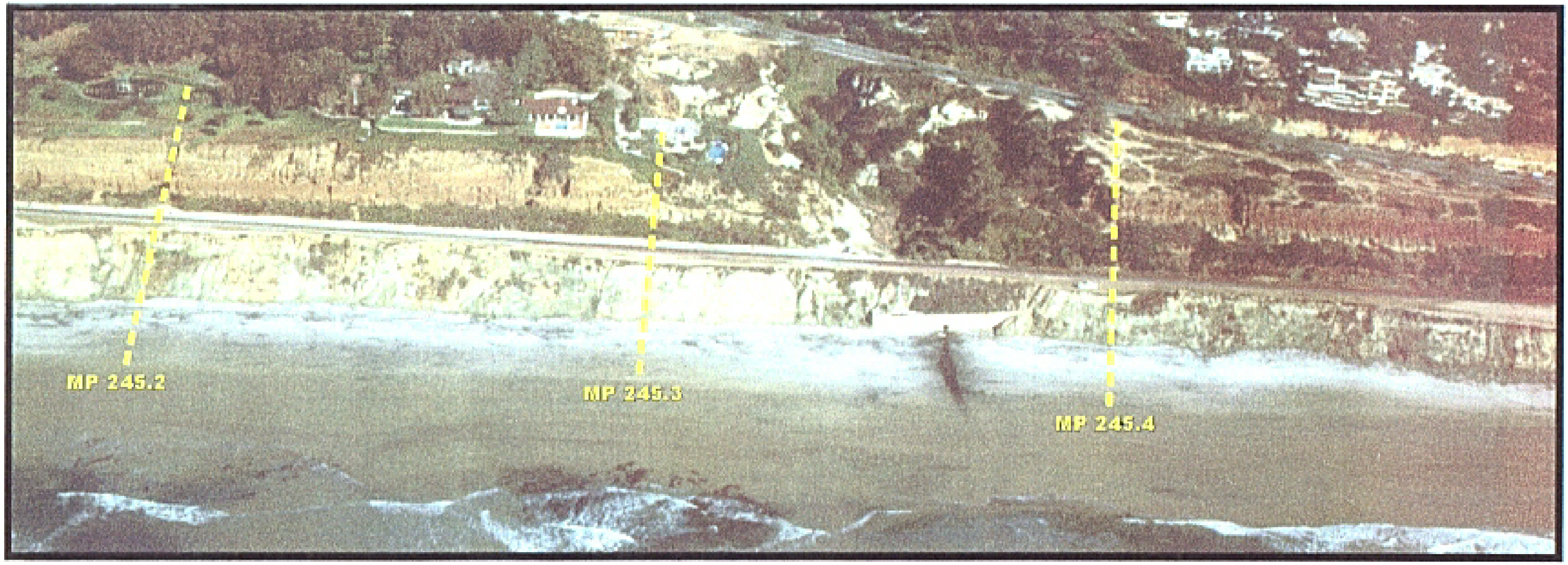
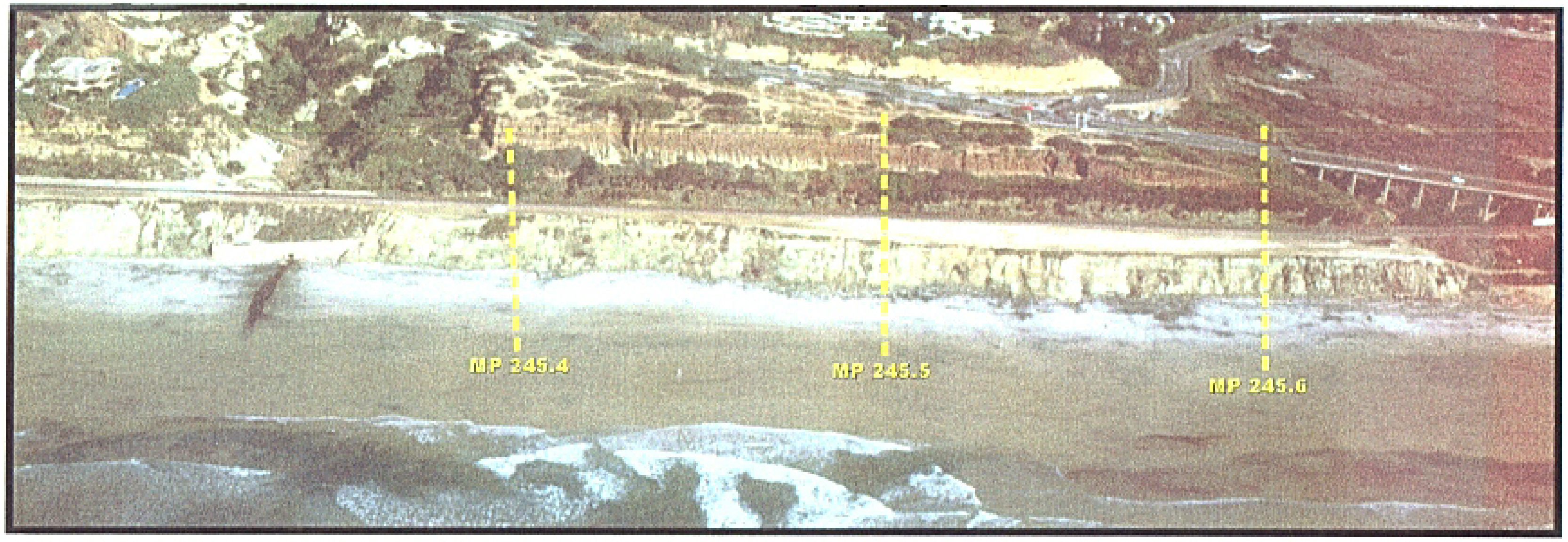


PHOTO NO. 43



APPENDIX C

RISK PRIORTITY

Risk Matrix Table Year 2000

Table No. 10

#	Location (Mile Post)	Track Elev. (ft)	Bluff Top Elev. (ft)	Distance from Track to Bluff Top (ft)	R	Distance from Track to Bluff Toe (ft)	Projection 10 feet W of Track to Toe	R	Estimated Bluff Retreat Rate (ft/year)	R	Geologic Unit	R	Existing Failures	R	Foot Traffic on Bluff Face	R	Seepage	R	Over Steepened Area	R	Other	R	Existing Improvements	R	Total Risk Rank S R's	Risk Ranking
1	244.15	44.5	35	85	2	120	3.1:1	1	0.5	3	Qbp/Td	3	None	1	None	1	Minor	2	None	1			A.C. layer at top of bluff	-2	12	Low
2	244.20	47.0	49	20	4	55	1.3:1	4	0.5	3	Qbp/Td	3	Moderate	3	None	1	Moderate	3	Moderate	3					24	High
3	244.25	48.5	47	35	4	75	1.5:1	3	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	At Toe	4			Mid slope wall	0	23	High
4	244.28	50.0	48	35	4	70	1.4:1	4	0.5	3	Af/Td	3	Minor	2	None	1	Moderate	3	Minor	2					22	High
5	244.32	52.5	50	18	5	65	1.1:1	5	0.5	3	Af	4	None	1	None	1	None	1	Moderate Erosion	3			Sea wall	0	23	High
6	244.36	54.5	52	20	4	70	1.1:1	5	0.5	3	Qbp/Td	3	Major	4	None	1	Heavy	5	Moderate	3					28	High
7	244.40	56.0	55	12	5	70	1.3:1	4	0.5	3	Af	4	S.D. Erosion	3	None	1	Moderate	3	Moderate	3	Wall at track	-3	Sea wall	0	23	High
8	244.45	59.0	55	10	5	80	1.4:1	4	0.5	3	Af (soil cement)	1	None	1	None	1	None	1	None	1			Sea wall	0	17	Med
9	244.47	60.0	56	30	4	80	1.4:1	4	0.5	3	Qbp/Td	3	Major	5	Heavy	4	Heavy	5	Major	5					33	High
10	244.54	61.0	73	80	2	115	1.6:1	3	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					20	Med
11	244.60	62.0	78	60	3	115	1.6:1	3	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					21	High
12	244.67	63.0	73	80	2	130	2.2:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					18	Med
13	244.70	64.0	59	65	3	115	3.2:1	1	0.5	3	Qbp/Td	3	Minor	2	Minor	2	Heavy	5	Storm Drain	3	Storm Drain Discharge on Top of Bluff	3			25	High
14	244.75	64.5	59	30	4	80	1.3:1	4	0.5	3	Qbp/Td	3	Moderate	3	Moderate	3	Minor	2	Minor	2					24	High
15	244.78	65.0	60	30	4	90	1.7:1	3	0.5	3	Af	4	None	1	Minor	2	None	1	None	1	Soldier Piles	0	Wall at toe	0	19	Med
16	244.80	65.0	59	40	3	105	1.7:1	3	0.5	3	Qbp/Td	3	Minor	2	Minor	2	Minor	2	Minor	2					20	Med
17	244.85	66.0	58	70	3	140	2.6:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					19	Med
18	244.90	66.0	59	85	2	160	2.9:1	1	0.5	3	Qbp/Td	3	None	1	Minor	2	Minor	2	Moderate	3	Erosional Gully	2	Drain removed	0	19	Med
19	244.97	67.0	60	100	2	150	2.6:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Minor	2	Moderate	3	Sub Drain Outlet	2			19	Med
20	245.04	67.0	60	75	3	118	2.1:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Minor	2	Moderate	3					18	Med
21	245.11	65.0	60	80	2	125	2.1:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Minor	2	Minor	2					16	Med
22	245.15	64.0	61	50	2	115	2.1:1	1	0.5	3	Af	4	None	1	None	1	None	1	None	1			Sea wall	0	14	Med
23	245.19	63.0	61	50	3	105	1.8:1	2	0.5	3	Qbp/Td	3	Moderate	3	None	1	Minor	2	Moderate	3					20	Med
24	245.25	59.0	56	27	4	80	1.6:1	3	0.5	3	Qbp/Td	3	Moderate	3	None	1	Minor	2	Moderate	3					22	High
25	245.36	53.0	52	15	5	63	0.9:1	5	0.5	3	Af	4	None	1	Moderate	3	None	1	None	1			Sea wall	0	23	High
26	245.39	52.0	51	45	3	95	2.0:1	1	0.5	3	Td	2	None	1	None	1	None	1	Minor	2					14	Low
27	245.50	50.0	50	40	3	120	2.1:1	1	0.5	3	Td	2	None	1	None	1	None	1	Minor	2					14	Low
28	245.60	48.0	48	50	3	150	3.0:1	1	0.5	3	Td	2	None	1	None	1	None	1	Minor	2					14	Low

Risk Ranking

0-15	Low
16-20	Medium
>20	High

R = Risk Ranking (1-5), Higher Number Indicates Increased Risk

Lowest Possible Score = 9

Risk Matrix Table Year 2020 — Table No. 11

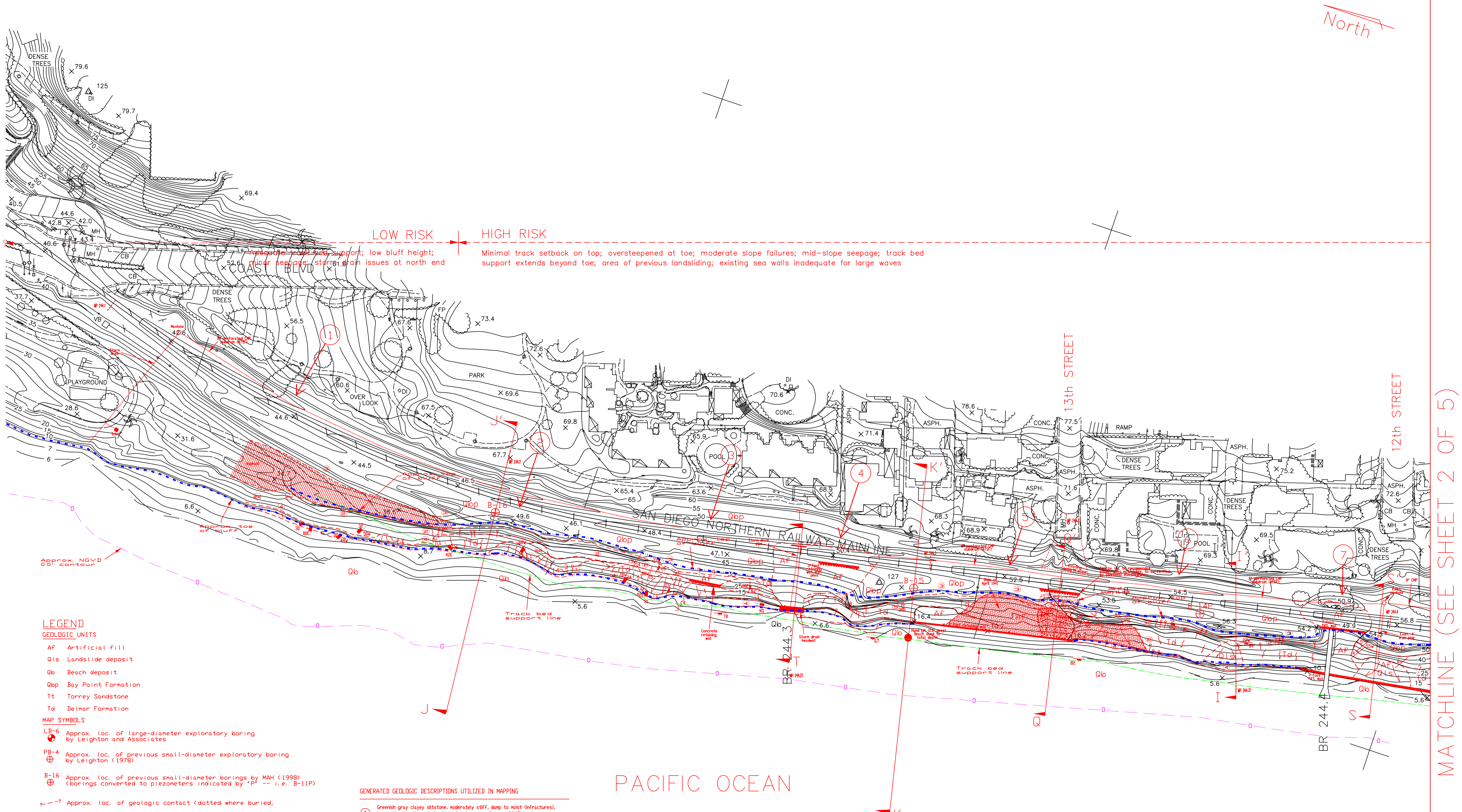
#	*Location (Mile Post)*	*Track Elev. (ft)*	*Bluff Top Elev. (ft)*	*Distance from Track to Bluff Top * (ft)*	*R*	*Distance from Track to Bluff Toe * (ft)*	*Projection 10 feet W of Track to Toe **	*R*	*Estimated Bluff Retreat Rate (ft/year)*	*R*	*Geologic Unit*	*R*	*Existing Failures*	*R*	*Foot Traffic on Bluff Face*	*R*	*Seepage*	*R*	*Over Steepened Area*	*R*	*Other*	*R*	*Existing Improvements*	*R*	*Total Risk Rank S R's*	*Risk Ranking*
1	244.15	44.5	35	75	2	110	2.9:1	1	0.5	3	Qbp/Td	3	None	1	None	1	Minor	2	None	1			A.C. layer at top of bluff	-2	12	Low
2	244.20	47.0	49	10	5	45	1.1:1	5	0.5	3	Qbp/Td	3	Moderate	3	None	1	Moderate	3	Moderate	3					26	High
3	244.25	48.5	47	25	4	65	1.3:1	4	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	At Toe	4			Mid slope wall	0	24	High
4	244.28	50.0	48	25	4	60	1.2:1	4	0.5	3	Af/Td	3	Minor	2	None	1	Moderate	3	Minor	2					22	High
5	244.32	52.5	50	8	5	55	0.9:1	5	0.5	3	Af	4	None	1	None	1	None	1	Moderate Erosion	3			Sea wall	0	23	High
6	244.36	54.5	52	10	5	60	0.9:1	5	0.5	3	Qbp/Td	3	Major	4	None	1	Heavy	5	Moderate	3					29	High
7	244.40	56.0	55	2	5	60	1.1:1	5	0.5	3	Af	4	S.D. Erosion	3	None	1	Moderate	3	Moderate	3	Wall at track	-3	Sea wall	0	24	High
8	244.45	59.0	55	0	5	70	1.2:1	4	0.5	3	Af (soil cement)	1	None	1	None	1	None	1	None	1	No lateral support @ track	5	Sea wall	0	22	High
9	244.47	60.0	56	20	5	70	1.2:1	4	0.5	3	Qbp/Td	3	Major	5	Heavy	4	Heavy	5	Major	5					34	High
10	244.54	61.0	73	70	2	105	1.4:1	4	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					21	High
11	244.60	62.0	78	50	3	105	1.4:1	4	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					22	High
12	244.67	63.0	73	70	2	120	2.0:1	2	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					19	Med
13	244.70	64.0	59	55	3	105	3.0:1	1	0.5	3	Qbp/Td	3	Minor	2	Minor	2	Heavy	5	Storm Drain	3	Storm Drain Discharge on Top of Bluff	3			25	High
14	244.75	64.5	59	20	5	70	1.1:1	5	0.5	3	Qbp/Td	3	Moderate	3	Moderate	3	Minor	2	Minor	2					26	High
15	244.78	65.0	60	20	5	80	1.5:1	3	0.5	3	Af	4	None	1	Minor	2	None	1	None	1	Soldier Piles	0	Wall at toe	0	20	Med
16	244.80	65.0	59	30	4	95	1.5:1	3	0.5	3	Qbp/Td	3	Minor	2	Minor	2	Minor	2	Minor	2					21	High
17	244.85	66.0	58	60	3	130	2.4:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Moderate	3	Moderate	3					19	Med
18	244.90	66.0	59	70	2	150	2.7:1	1	0.5	3	Qbp/Td	3	None	1	Minor	2	Minor	2	Moderate	3	Erosional Gully	2	Drain removed	0	19	Med
19	244.97	67.0	60	90	2	140	2.4:1	1	0.5	3	Qbp/Td	3	Minor	2	None	1	Minor	2	Moderate	3	Sub Drain Outlet	2			19	Med
20	245.04	67.0	60	65	3	108	1.9:1	2	0.5	3	Qbp/Td	3	Minor	2	None	1	Minor	2	Moderate	3					19	Med
21	245.11	65.0	60	70	3	115	1.9:1	2	0.5	3	Qbp/Td	3	Minor	2	None	1	Minor	2	Minor	2					18	Med
22	245.15	64.0	61	40	3	105	1.9:1	2	0.5	3	Af	4	None	1	None	1	None	1	None	1			Sea wall	0	16	Med
23	245.19	63.0	61	40	3	95	1.6:1	3	0.5	3	Qbp/Td	3	Moderate	3	None	1	Minor	2	Moderate	3					21	High
24	245.25	59.0	56	17	5	70	1.4:1	4	0.5	3	Qbp/Td	3	Moderate	3	None	1	Minor	2	Moderate	3					24	High
25	245.36	53.0	52	5	5	53	0.7:1	5	0.5	3	Af	4	None	1	Moderate	3	None	1	None	1			Sea wall	0	23	High
26	245.39	52.0	51	35	4	85	1.8:1	3	0.5	3	Td	2	None	1	None	1	None	1	Minor	2					17	Med
27	245.50	50.0	50	30	4	110	1.9:1	2	0.5	3	Td	2	None	1	None	1	None	1	Minor	2					16	Low
28	245.60	48.0	48	40	3	140	2.8:1	1	0.5	3	Td	2	None	1	None	1	None	1	Minor	2					14	Low

* Note: Matrix Assumed 10 Feet of Additional Bluff Retreat by Year 2020

Risk Ranking

0-15	Low
16-20	Medium
>20	High

R = Risk Ranking (1-5), Higher Number Indicates Increased Risk
Lowest Possible Score = 9



LEGEND

GEOLOGIC UNITS

Af	Artificial fill
Qls	Landslide deposit
Qb	Beach deposit
Qbp	Bay Point Formation
Tt	Torrey Sandstone
Td	Delmar Formation

MAP SYMBOLS

- LB-6 — Approx. loc. of large-diameter exploratory boring by Leighton and Associates
- PB-4 — Approx. loc. of previous small-diameter exploratory boring by Leighton (1978)
- B-16 — Approx. loc. of previous small-diameter borings by MAH (1998) (borings converted to piezometers indicated by "P" -- i.e. B-11P)
- Approx. loc. of geologic contact (dotted where buried,
- 10 — Strike and dip of bedding
- 5 — Strike and dip of bedding, uncertain
- 2 — Stike and dip of undulatory bedding
- 58 — Strike and dip of joints
- 67 — Fault, showing trend and plunge
- Seepage
- J' / J — Geologic cross-section
- 1 — Risk matrix site

GENERATED GEOLOGIC DESCRIPTIONS UTILIZED IN MAPPING

1. Greenish gray clayey siltstone, moderately stiff, damp to moist (infractures), iron oxide clay along dominant joint sets, generally more lithified south of M.P. 244.78 with lesser clay content
2. Yellowish light brown fine sandy silt, stiff to very stiff, dry to Sl damp, lithifed in areas as marker bed

2b. Non-lithified version of 2

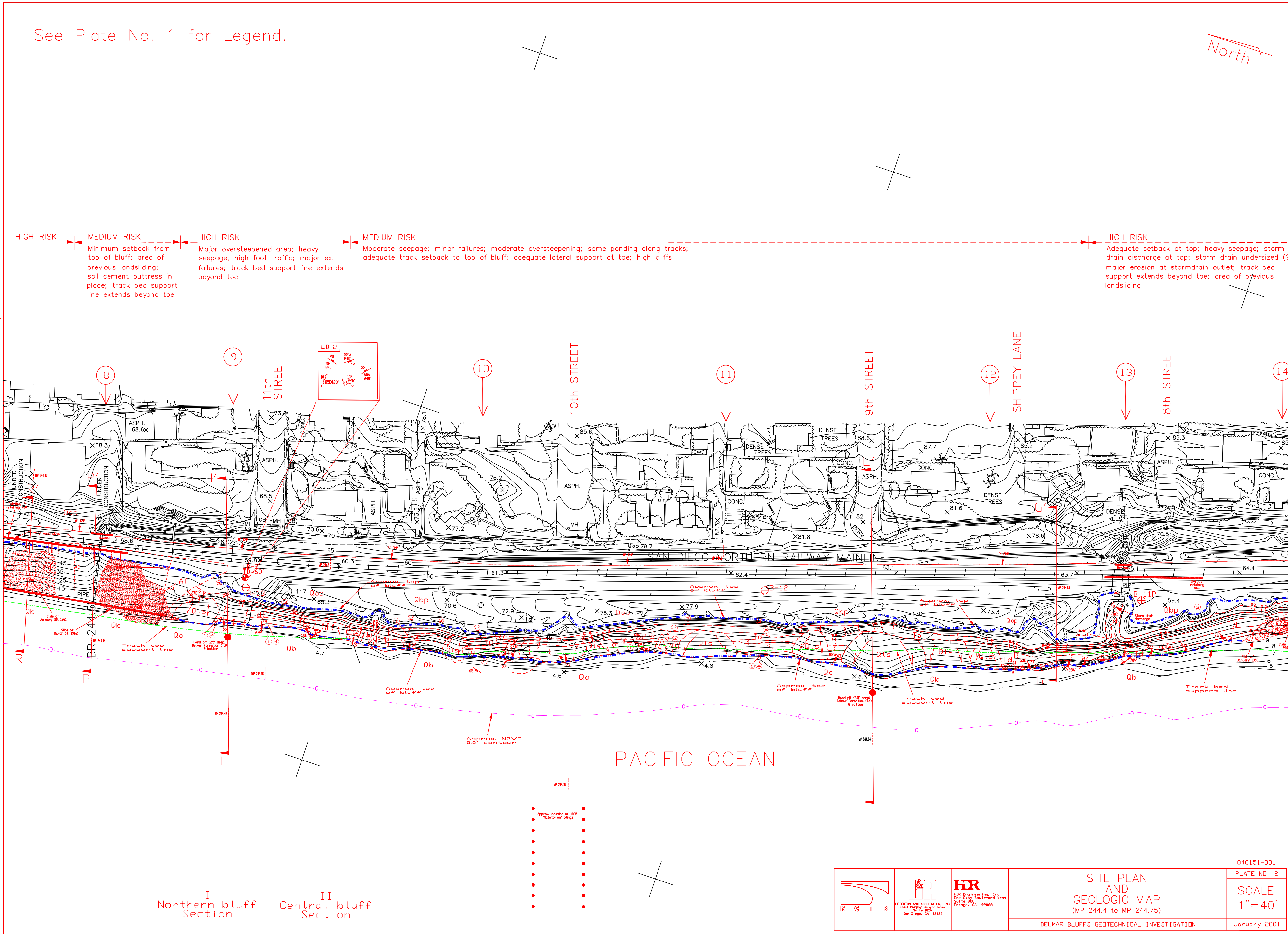
3. Reddish brown fine to medium sandstone with trace silt, very loose to very dense dry to damp
4. Gray/light brown fine to medium sandstone, dense, damp, gradual fining upwards

NOTE:

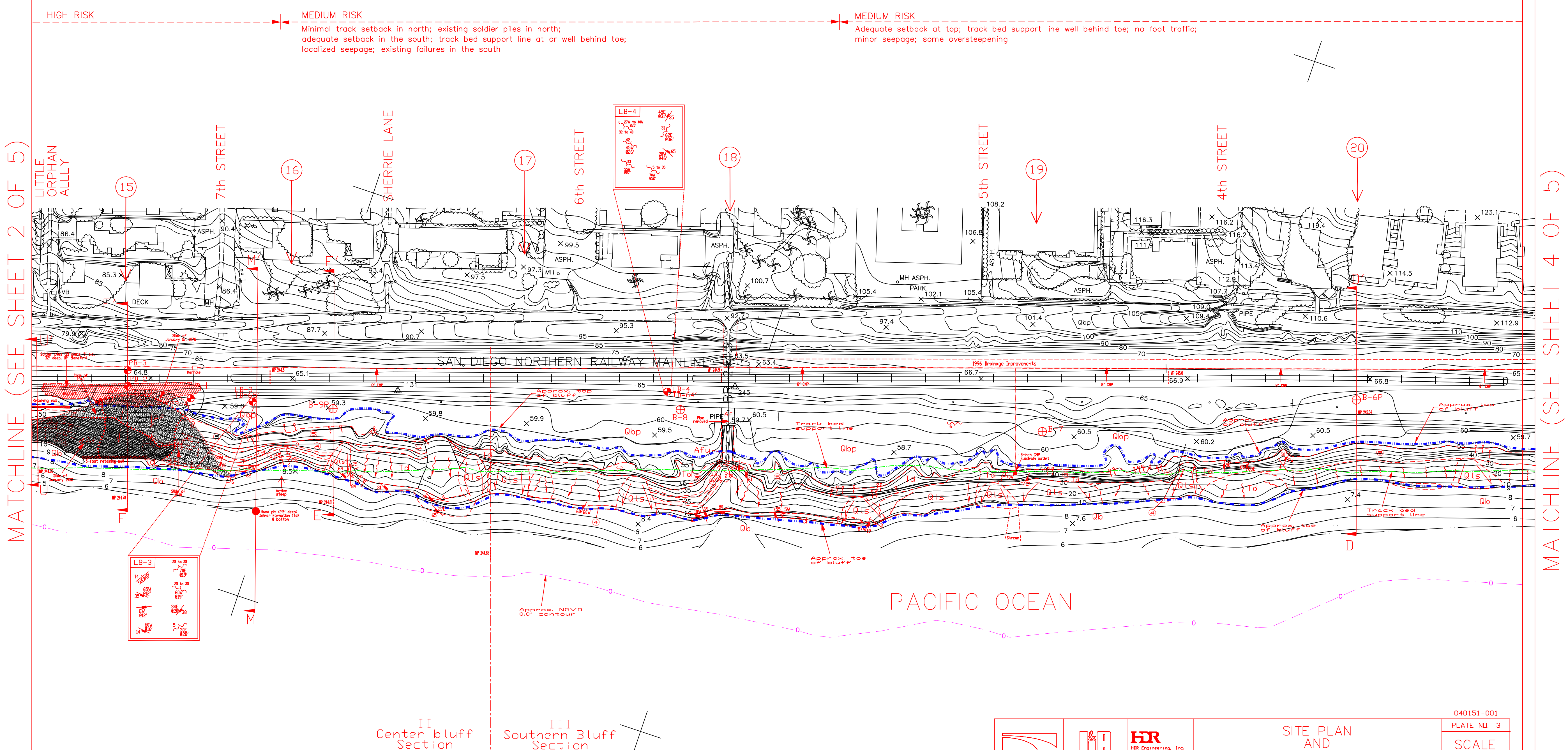
Subdrains shown on map based on review of ATSF documents (ATSF, 1978); not all subdrains observed during field mapping

040151-001

N C T D	LEIGHTON AND ASSOCIATES, INC. 3934 Murphy Canyon Road Suite B204 San Diego, CA 92123	HDR Engineering, Inc. One City Boulevard West Suite 900 Orange, CA 92868	SITE PLAN AND GEOLOGIC MAP (Coast Blvd. to MP 244.4)	PLATE NO. 1 SCALE 1"=40'
			DELMAR BLUFFS GEOTECHNICAL INVESTIGATION	January 2001


See Plate No. 1 for Legend.
North
HIGH RISK
MEDIUM RISK
Minimum setback from top of bluff; area of previous landsliding; soil cement buttress in place; track bed support line extends beyond toe
HIGH RISK
Major oversteepened area; heavy seepage; high foot traffic; major ex. failures; track bed support line extends beyond toe
MEDIUM RISK
Moderate seepage; minor failures; moderate oversteepening; some ponding along tracks; adequate track setback to top of bluff; adequate lateral support at toe; high cliffs
HIGH RISK
Adequate setback at top; heavy seepage; storm drain discharge at top; storm drain undersized (?); major erosion at stormdrain outlet; track bed support extends beyond toe; area of previous landsliding
MATCHLINE (SEE SHEET 1 OF 5)
MATCHLINE (SEE SHEET 3 OF 5)
11th STREET
10th STREET
9th STREET
SHIPPEY LANE
8th STREET
SAN DIEGO NORTHERN RAILWAY MAINLINE
LB-2
B-12
B-11P
Approx. top of bluff
Approx. toe of bluff
Track bed support line
Approx. NGVD 0.0' contour
PACIFIC OCEAN
I Northern bluff Section
II Central bluff Section
040151-001
PLATE NO. 2
SITE PLAN AND GEOLOGIC MAP (MP 244.4 to MP 244.75)
SCALE 1"=40'
DELMAR BLUFFS GEOTECHNICAL INVESTIGATION
January 2001
HDR Engineering, Inc.

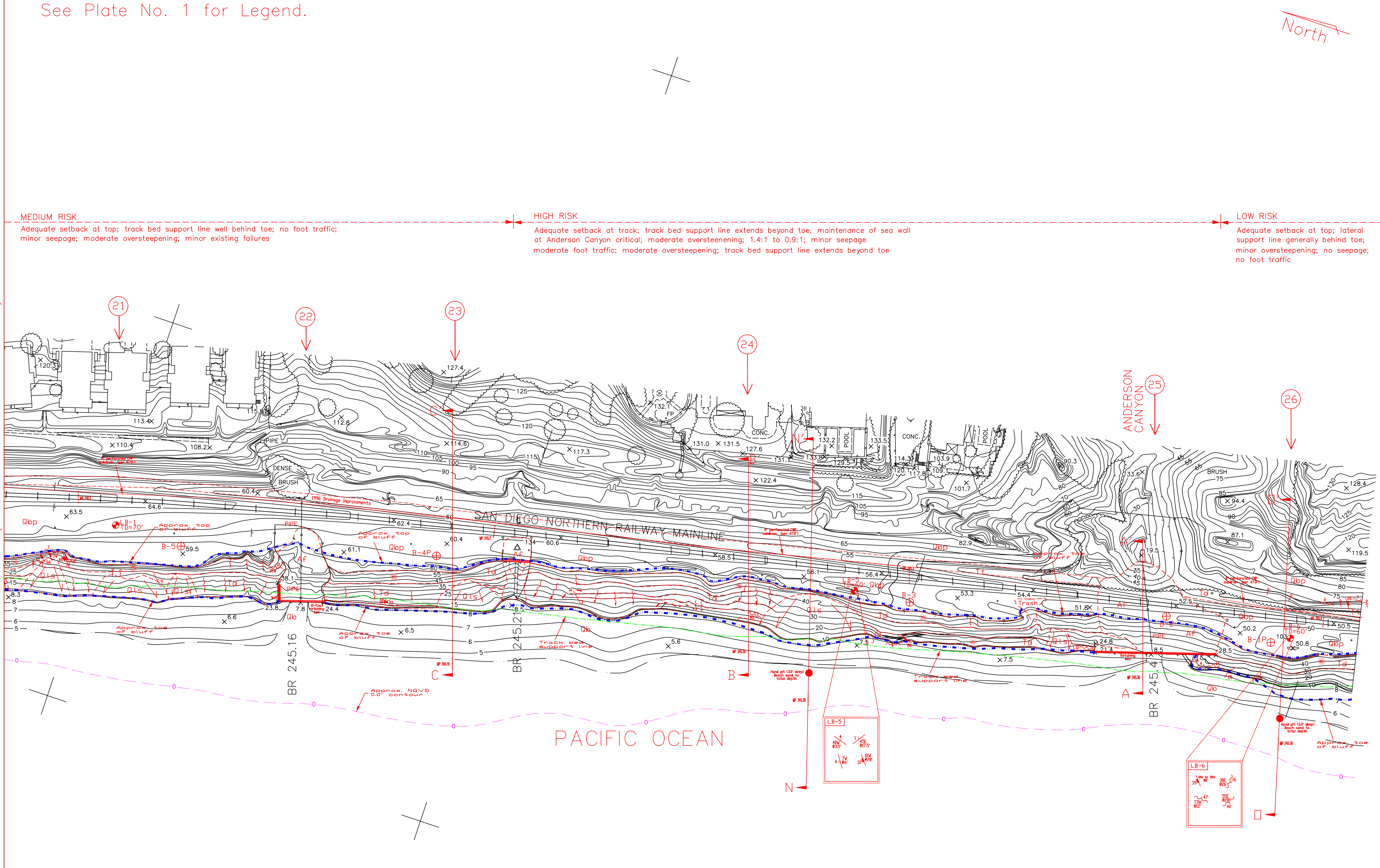
See Plate No. 1 for Legend.



N C T D	LEIGHTON AND ASSOCIATES, INC. 3934 Murphy Canyon Road Suite B204 San Diego, CA 92123	HDR Engineering, Inc. One City Boulevard West Suite 900 Orange, CA 92868	SITE PLAN AND GEOLOGIC MAP (MP 244.75 to MP 245.06)	040151-001 PLATE NO. 3 SCALE 1"=40'
			DELMAR BLUFFS GEOTECHNICAL INVESTIGATION	January 2001

See Plate No. 1 for Legend.

MATCHLINE (SEE SHEET 3 OF 5)



040151-001

N C T D

LEIGHTON AND ASSOCIATES, INC.

HDR Engineering, Inc.

SITE PLAN AND GEOLOGIC MAP

(MP 245.06 to MP 245.4)

DELMAR BLUFFS GEOTECHNICAL INVESTIGATION

PLATE NO. 4

SCALE 1"=40'

January 2001