Appendix G
Geotechnical Study

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PRELIMINARY GEOLOGICAL/GEOTECHNICAL ASSESSMENT REPORT FOR THE DANA POINT HARBOR REVITALIZATION PROJECT ENVIRONMENTAL IMPACT REPORT

Prepared For

RBF Consulting

January 15, 2004

(updated January 18, 2006)

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GeoPentech

January 15, 2004
Project No. 03006A

Mr. Eddie Torres
RBF Consulting
14725 Alton Parkway
Irvine, CA 92618-2027

SUBJECT: **PRELIMINARY GEOLOGICAL/GEOTECHNICAL ASSESSMENT REPORT FOR THE DANA POINT HARBOR REVITALIZATION PROJECT ENVIRONMENTAL IMPACT REPORT DANA POINT, CALIFORNIA**

Dear Mr. Torres:

GeoPentech, Inc. (GeoPentech) is pleased to submit this preliminary geological/geotechnical assessment for the Dana Point Harbor Revitalization Project EIR. A draft report was previously submitted on February 25, 2003. RBF comments on the draft report were provided on December 8, 2003 and were incorporated in this report. This report was prepared in accordance with our scope of work dated November 12, 2002.

The report contains the anticipated subsurface conditions and the geologic and seismic setting of the site and provides the impacts on the proposed facilities/structures that are proposed as part of the revitalization program. The report also addresses potential remediation measures for the facilities/structures.

It is noted that this preliminary report is prepared to provide input to the EIR and therefore should not be used for design or construction purposes. It is also noted that the information utilized in this report was based on readily available information provided primarily by RBF Consulting and supplemented by available in-house records and published information.

Thank you for providing GeoPentech the opportunity to participate in this project. If you have any questions or require additional information, please call. We can finalize this report once we receive your comments.

Very truly yours,

GeoPentech

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1.0 INTRODUCTION

This Preliminary Geological/Geotechnical Assessment report has been prepared to address significant geological and geotechnical impacts for incorporation into the Draft Environmental Impact Report (EIR) for the Dana Point Harbor Revitalization Project. This work was performed in accordance with our scope of work dated November 12, 2002.

Dana Point Harbor is a County owned and operated facility which was opened in 1971 from a natural cove along the Southern Orange County Shoreline, at the location shown in Figure 1, *Site Vicinity Map*. The Harbor is comprised of approximately 276.8 acres. The Harbor facilities have aged and deteriorated over time and are now due for major revitalization. The County's proposal for revitalization of the Dana Point Harbor considers both landside (Harbor) facility and seaside (Marina) improvements. The current revitalization plans are for improvements to substantial portions of the landside areas of the Harbor. Specifically, these areas are Planning Areas 1 through 5 as shown on Figure 2, *Planned Community Development Plan*. Geological and geotechnical impacts for Planning Areas 1 through 5 are addressed in this preliminary report.

Plans for the seaside (Marina) improvements, Planning Areas 9 and 10 as shown on Figure 2, *Planned Community Development Plan,* will also be considered as part of a separate planning and construction-related review and approval process by the California Coastal Commission as funding becomes available. Geological and geotechnical impacts related to elements of this work, such as the repair and/or replacement of boat slips and the seawall, are also addressed in this report.

Planning Areas 6, 7, 8, 11 and 12 (shown on Figure 2, *Planned Community Development Plan*) are not part of the current project and are not addressed in this report.

The geological/geotechnical impacts have been evaluated in accordance with the California Environmental Quality Act (CEQA) and with the County of Orange Planning and Development and Services Department instructions for completing environmental information for Public Projects.

The assessment was prepared based on the documents provided to us by RBF Consulting and available published information. No field exploration or laboratory testing was performed as part of the scope of work for this report. It is noted that this assessment addresses only the geological/geotechnical impacts associated with the proposed revitalization of Dana Point Harbor in a preliminary fashion. Other aspects, such as construction impacts from noise, dust, vibrations, and traffic are anticipated to be addressed in the Draft EIR by others and are only briefly identified in this report.

Data Source: Parcels provided Orange County data on March 2003
Aerial Photography Provided by Eagle Aerial Imaging

N

0 300 600 Feet

4/15/03 10-102529

Harbor Boundary
Commercial Core Boundary

Source: Dana Point Harbor Revitalization Project Notice of Preparation (October 2003)

Site Vicinity Map

Project No. 03006A	Date: JAN. 2004	Project: Dana Point Harbor Revitalization Project	Figure 1

T:\03006A\DanaPoint\Figures\Figure1.grf

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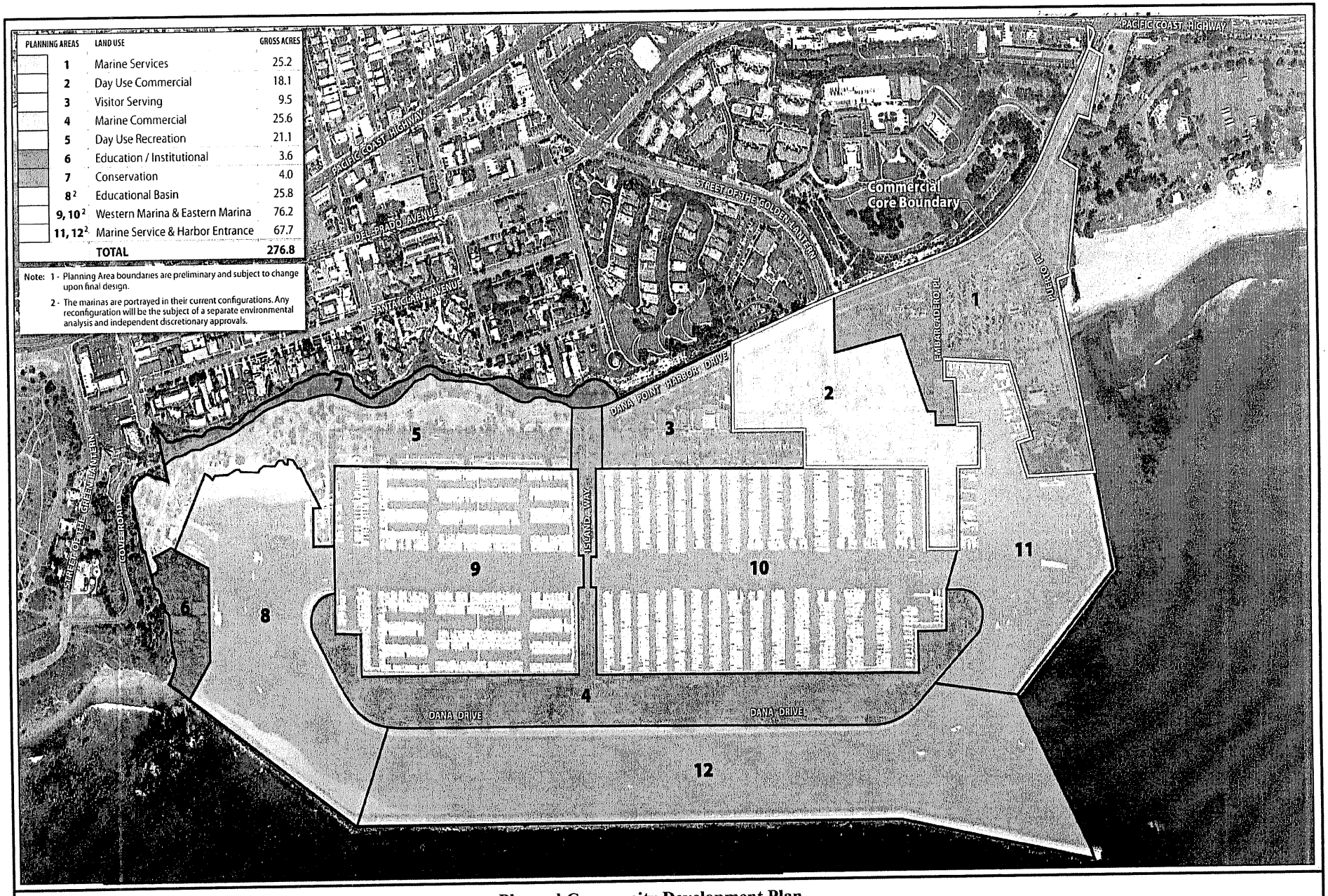

PLANNING AREAS	LAND USE	GROSS ACRES
1	Marine Services	25.2
2	Day Use Commercial	18.1
3	Visitor Serving	9.5
4	Marine Commercial	25.6
5	Day Use Recreation	21.1
6	Education / Institutional	3.6
7	Conservation	4.0
8 [2]	Educational Basin	25.8
9, 10 [2]	Western Marina & Eastern Marina	76.2
11, 12 [2]	Marine Service & Harbor Entrance	67.7
	TOTAL	276.8

Note: 1 - Planning Area boundaries are preliminary and subject to change upon final design.

2 - The marinas are portrayed in their current configurations. Any reconfiguration will be the subject of a separate environmental analysis and independent discretionary approvals.

Planned Community Development Plan

Project No. 03006A	Date: JAN 2004	Project: Dana Point Harbor Revitalization Project	Figure 2

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2.0 REVITALIZATION PROJECT PROGRAM DESCRIPTION

Dana Point Harbor (Harbor) is approximately 276.8 acres, owned and operated by the County of Orange (County). The Harbor is bordered by the Pacific Ocean to the south, Dana Headlands and the Old Cove Marine Life Preserve to the west, Doheny State Beach to the east and a variety of commercial, hotel, residential, and public park uses to the north. The Harbor is a man-made regional recreational facility built in a cove formed by the headlands of Dana Point to the north, in Capistrano Bay. The Harbor construction was completed in the early 1970s. The County has not remodeled, refurbished any of the Harbor facilities since that time. Planning for the Revitalization project began in the late 1990s.

As shown on Figure 3, *Harbor Revitalization Plan*, Planning Areas 1 and 2 consists of the replacement and/or remodeling of all existing retail and restaurant buildings, a new 610-space parking deck, new boater loading and drop-off areas, a new dry-stack boat storage building and improvements to boater service and public restroom buildings. On a programmatic level, the Revitalization Plan provides for a number of future improvements in Planning Areas 1, 3 through 7 (landside) and 8 through 12 (waterside). Plans for Planning Area 1 include a second dry stack boat storage building and the construction of a new lighthouse facility by the Dana Point Lighthouse Society at the terminus of Puerto Place.

3.0 EXISTING GEOLOGIC CONDITIONS

3.1 Topography/Landform

Dana Point Harbor is located within the protected sheltered cove known as Dana Cove. This cove is bordered on the north by steep, highly eroded sandy cliffs or bluffs that are about 100 to 200 feet in elevation. To the west, the Dana Point headland extends outward from the main line of the coast. Prior to construction of the Harbor, the base of the cliffs was similar to the headland, and consisted of rocky shoreline in which a sandy shore became more prominent toward San Juan Creek (Environmental Sciences, 1973). The cliff areas descend in height from the headland to San Juan Creek.

The existing Harbor topography gently slopes from the northwest to the southeast, with a grade change of approximately 20 feet from Dana Point Harbor Drive to the top of the bulkhead. A distinct grade change of 5 to 10 feet occurs at the northerly edge of the surface parking lots, where an existing 2:1 fill slope exists adjacent to Dana Point Harbor Drive.

PLANNING AREAS [1]		LAND USE	GROSS ACRES
	1	Marine Services	25.2
	2	Day Use Commercial	18.1
	3	Visitor Serving	9.5
	4	Marine Commercial	25.6
	5	Day Use Recreation	21.1
	6	Education / Institutional	3.6
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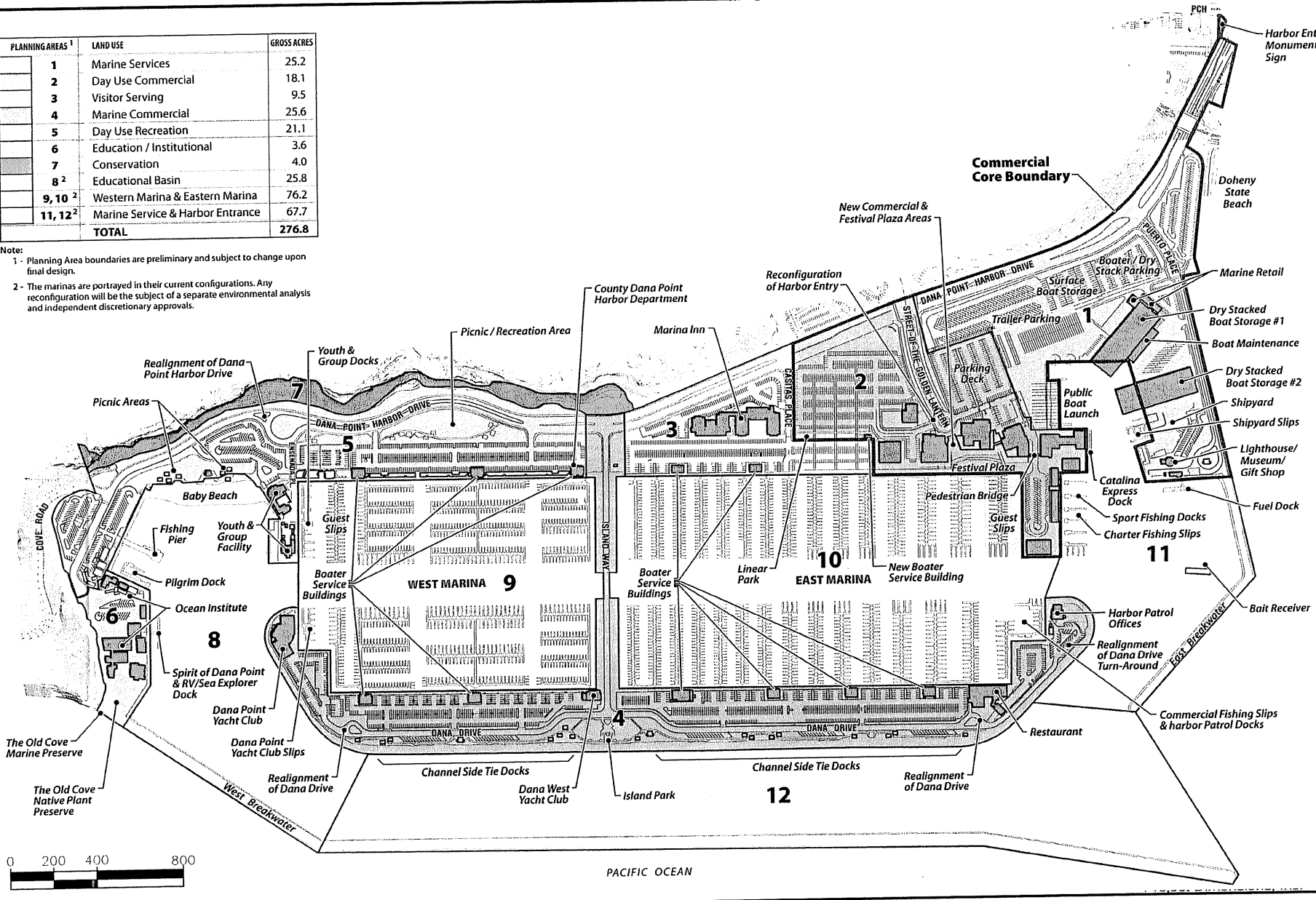

Harbor Revitalization Plan

Project No. 03006A	Date: JAN. 2004	Project: Dana Point Harbor Revitalization Project	Figure 3

T:\03006A\DanaPoint\Figures\Figure3.grf

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3.2 Geologic Setting

Dana Point Harbor is located within California's Peninsular Ranges Physiographic Province near its western margin with the offshore Continental Boarder lands. The Peninsular Ranges' predominantly northwest trending physiography is characterized by parallel northwest trending, fault-bounded mountain ranges and intervening valleys that extend southward into Baja California from the east-west Transverse Ranges Physiographic Province, located north of the Los Angeles Basin. Dana Point Harbor is situated along the shoreline below elevated marine and non-marine terraces flanking the southeastern corner of the San Joaquin Hills, one of these northwest trending ranges.

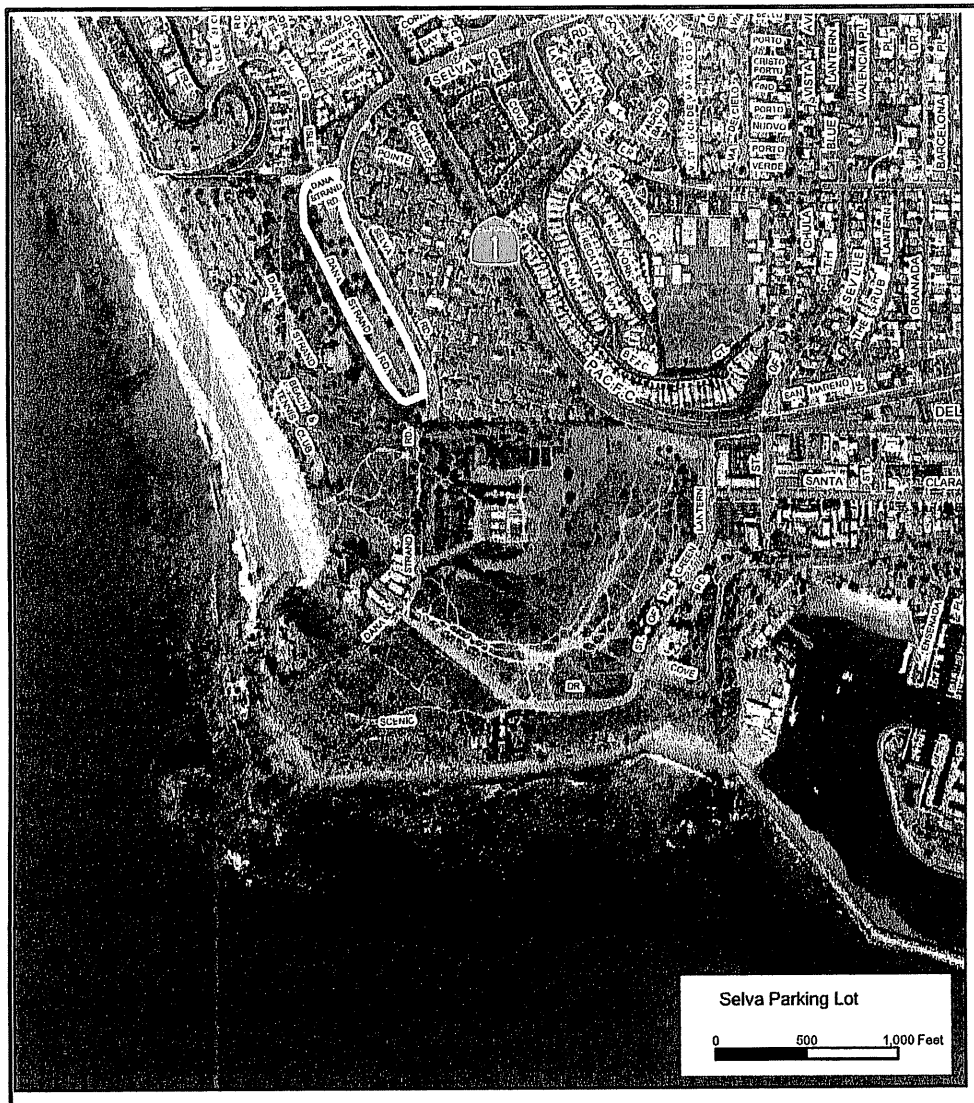
3.3 Geologic Units

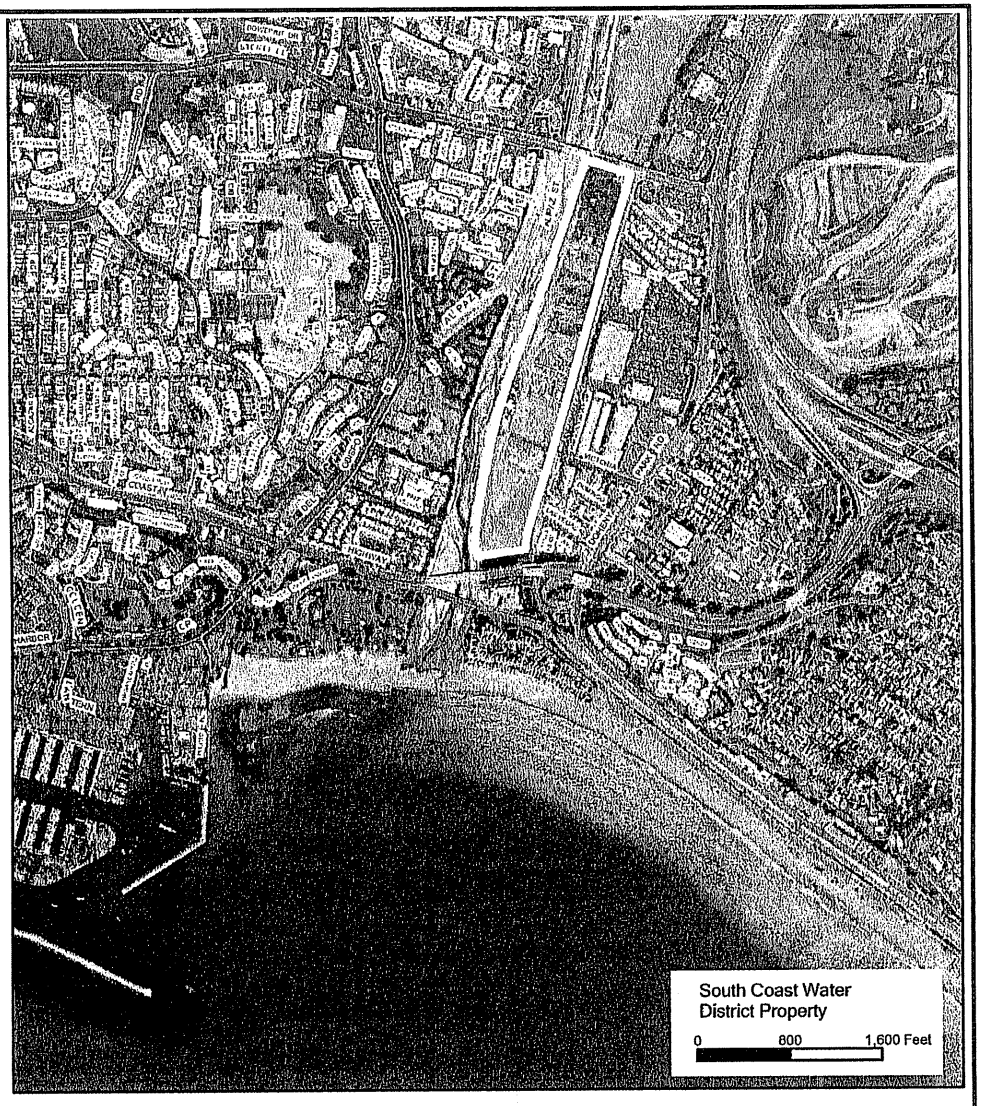
Artificial fill, beach sand, and alluvial deposits underlie Dana Point Harbor. The sea cliffs surrounding the Harbor to the north and west are cut into marine sedimentary rocks. The rocks exposed in the sea cliff are capped by marine and non-marine terrace deposits, and the slopes along the sea cliff are locally mantled by landslide debris and talus deposits (Edgington, 1974 and Zeiser Geotechnical, 1990).

Leighton and Associates (L&A 2002) performed a preliminary geotechnical investigation of a portion of the commercial core revitalization area between Dana Point Harbor Drive and the seawall, east of Casitas Place and west of the existing boat yard area near Embarcadero Place. L&A reported artificial fills in this area typically ranging in thickness from 15 to 20 feet. The fills were reported to consist of sandy clay, clayey sand, and silty clay. A layer of dense to very dense, gravelly sand alluvium was reported at depths of 20 to 22 feet in one of L&A's borings. Fluvial terrace deposits were reported by Environmental Services (1972) to have been exposed at the extreme eastern end of the Harbor. The total depth of soils beneath the site of the Harbor in areas outside the commercial core revitalization area explored by L&A (2002) is not specifically known.

The surficial soils beneath the Harbor revitalization area are likely underlain by the marine Middle Miocene San Onofre Breccia and Upper Miocene to lower Pliocene Capistrano formations that form the sea cliffs north of the Harbor. Dana Point Cove is formed by the San Onofre Breccia's resistant nature to wave erosion and the more erodible Capistrano formation. The San Onofre Breccia formation is exposed at the western end of Dana Point Harbor along the east-facing cliffs where the Dana Point headland is formed by the resistant nature of the San Onofre Breccia, as shown in Figure 5, *San Onofre Breccia Exposures at Dana Point*. The San Onofre Breccia and Capistrano bedrock formations are juxtaposed across the Dana Cove fault, which is located in the northwest corner of the steep coastal bluffs.

The San Onofre Breccia formation consists of very coarse, reddish-brown to blue-gray, massive to poorly bedded breccia with occasional interbeds of coarse-grained, pebbly sandstone and siltstone. The breccia matrix generally consists of poorly to well-cemented silt and sand.





Data Source: Parcels provided Orange County data on March 2003
Aerial Photography By Eagle Aerial Flown June 2002

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10/17/03 10-102529

Source: Dana Point Harbor Revitalization Project
Notice of Preparation (October 2003)

Off-Site Parking Areas

Project No. 03006A	Date: JAN 2004	Project: Dana Point Harbor Revitalization Project	Figure 4

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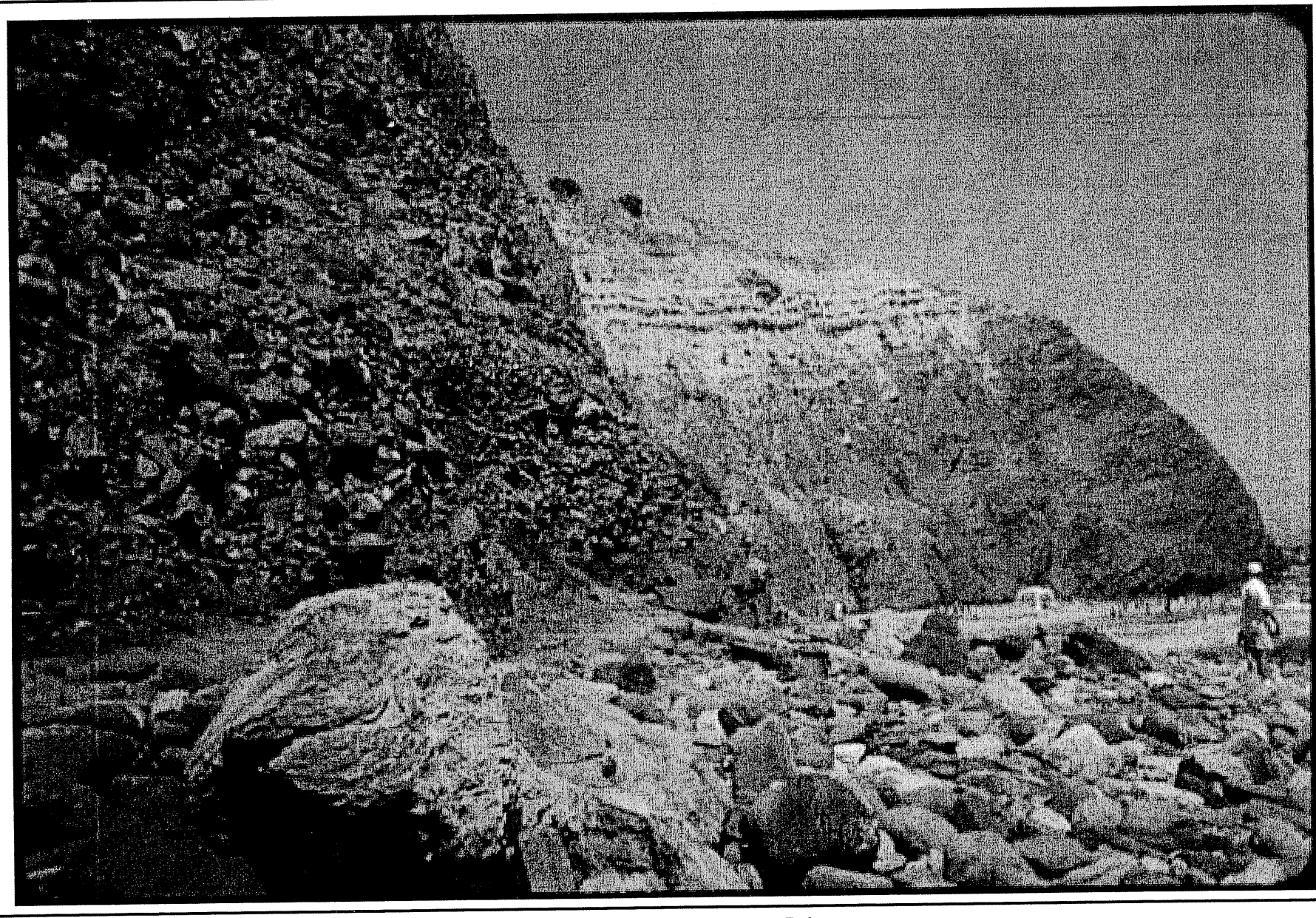

San Onofre Breccia Exposures at Dana Point

Project No. 03006A	Date: JAN 2004	Project: Dana Point Harbor Revitalization Project	Figure 5

T:\03006A\DanaPoint\Figures\Figure5.grf

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The Capistrano formation consists of interbedded conglomerate and sandstone along the western half of the sea cliffs, more predominate siltstone further to the east, and sandstone with occasional siltstone and conglomerate layers in the eastern portion of the Harbor area. Capistrano Formation bedrock is sheared in a zone about 70 to 100 feet wide along the Dana Cove Fault (Kerwin, 1992).

3.4 Groundwater

Due to the proximity of the site to the ocean, the ground water level across the site generally coincides with the sea level. The ground water levels previously encountered (L&A 2002) ranged between approximately 3 feet below ground surface at the southern (seawall) edge of the Commercial Core area and to about 7 feet below ground surface at the northern boundary of the Commercial Core area (in the vicinity of the existing parking and boat storage areas in Planning Areas 1 and 2).

3.5 Local and Regional Faults

The location of the site in relation to known active, potentially active, and conjectural faults is shown on Figure 6, *Regional Fault Map* and Figures 7 through 9, *Location of Model Fault Sources – Model 1, 2 and 3*, respectively. The key faults surrounding the Dana Point Harbor area are summarized below:

Dana Cove Fault

A well-defined fault zone passes diagonally through the Harbor, directly under and nearly parallel to the existing West Basin Pier (bearing approximately 43 degrees west of north). Environmental Sciences (1973), inferred the seaward projection of the fault zone to be about 250 feet wide consisting of sheared breccia and contorted siltstones and sandstones. No seismic activity has been reported along this fault and Zeiser Geotechnical (Zeiser 1990) characterized the fault as inactive.

Newport-Inglewood Fault Zone/ South Coast Offshore Zone of Deformation

Figure 6, *Regional Fault Map*, shows the location of the site relative to mapped active faults in the region, as identified by Jennings (1994).

The closest active fault to the site is the South Coast Offshore Zone of Deformation (SCOZD), which is approximately 3.4 miles (5.5 km) southwest of the site. The identification and characterization of the SCOZD is an ongoing research topic in Southern California. The SCOZD represents the likely offshore connection between the Newport-Inglewood Fault Zone located to the northwest and the Rose Canyon Fault Zone located further to the south forming the Newport-Inglewood/Rose Canyon Fault Zone (Fischer and Mills, 1991 and Fischer and Lee, 1992). The SCOZD was identified on the basis of geophysical investigations conducted by Western Geophysical (1972), as a continuous trace within a zone of en-echelon faults in the acoustic basement. Local northwest to west-trending folds in the shallower horizons are also associated with this zone.

The location of the postulated SCOZD is shown on Figure 7, *Location of Model Fault Sources – Model 1* .The SCOZD extends approximately 42 miles from its northern terminus, located offshore

approximately 5 miles south of Newport Beach to its southern terminus, located offshore southwest of Oceanside.

The SCOZD appear to reflect a tectonic style similar to that of the onshore portion of the Newport-Inglewood Fault, which extends onshore from the east-west Malibu-Santa Monica Fault zone at the southern front of the Transverse Ranges to the northwest, to offshore between Newport Beach and Laguna Beach at the San Joaquin Hills Structural High. The Newport-Inglewood Fault is characterized by short, discontinuous, northwest-trending en-echelon, right-lateral faults, relatively shallow drag fold anticlines, and subsidiary normal and reverse faults. The Newport-Inglewood Fault was the source of the destructive 1933 Long Beach Earthquake (Mw 6.4) (Hauksson and Gross, 1991). Scientists from the US Geological Survey (Sliter and others, 2001) also interpret recent faulting at the base of the slope between Dana Point and Oceanside to be related to a strand of the Newport-Inglewood Fault.

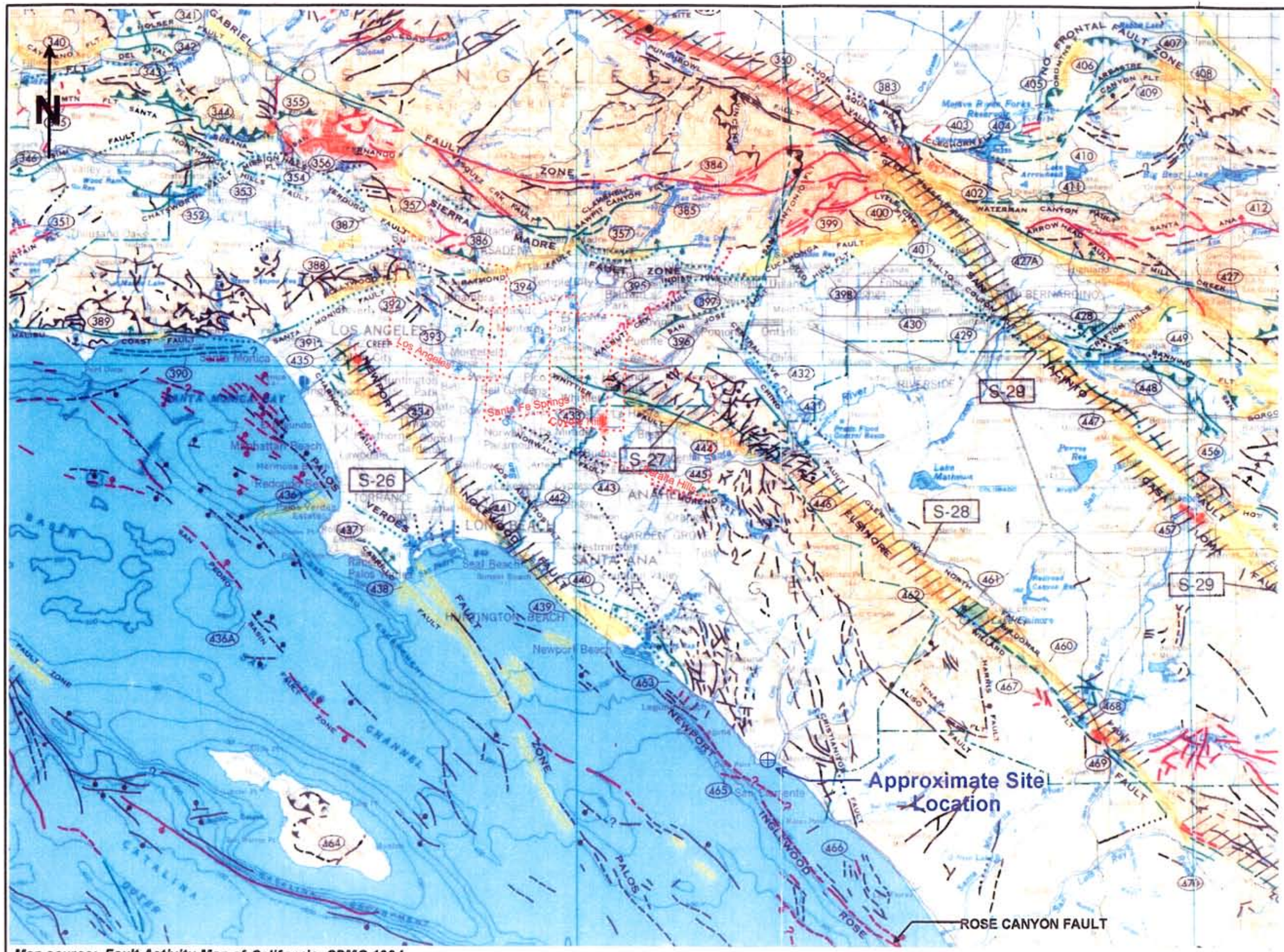
San Joaquin Hills and Oceanside Blind Thrust Faults

In addition to the mapped surface faults shown on Figure 6, *Regional Fault Map,* blind thrust faults are also believed to exist in the region. The identification and characterization of active, seismogenic blind fault seismic sources is an ongoing research topic in Southern California. These blind thrust faults are not expressed at the surface, but are inferred to exist based on indirect information, such as seismicity and folded stratigraphy. Two recently postulated fault sources, the San Joaquin Hills Blind Thrust (SJHB) and the Oceanside Blind Thrust (OBT), are judged to be potentially significant seismic sources for Dana Point Harbor. Possible models for the location of postulated (SJHB and OBT) faults in relation to Dana Point Harbor are shown on Figures 7 through 9, *Location of Model Fault Sources – Model 1, 2 and 3*, respectively.

The uplift of the San Joaquin Hills is produced by a southwest dipping blind thrust fault that extends at least 9 miles from northwestern Huntington Mesa to Dana Point and comes to within 1.2 miles of the ground surface (Grant et. al, 1997; Mueller et. al., 1998). Work by Grant et al. (1997 and 1999) suggests that uplift of the San Joaquin Hills began in the Late Quaternary and continues during the Holocene period. Grant and others (2000) conclude that the San Joaquin Hills are rising in response to a potentially seismogenic, underlying blind fault that should be included in regional seismic hazard models.

Leighton and Associates (2002) characterized the closest active fault to the site as the San Joaquin Hills Blind Thrust, located at about 1.7 miles (2.7 km) from the site and capable of generating a magnitude Mw 6.8 offshore.

Several recent studies have also interpreted offshore seismic data to image a major, east-dipping low-angle normal fault that is responsible for tectonic "footwall uplift" in Miocene time of high-grade metamorphic rocks now exposed on Catalina Island (Crouch and Suppe, 1993; Bohannon and Geist, 1998). Bohannon and Geist (1998) refer to this fault as the Oceanside Detachment. Rivero and others (2000) speculate that large portions of these detachments have been reactivated to form the



LEGEND

Faults; U = upthrown side, D = downthrown side, bar and ball on downthrown side, parallel arrows along fault indicate relative direction of movement, perpendicular arrow on fault indicates direction of dip, low angle fault indicated by barbs on upper plate.

Historic fault; displacement during the past 200 years.

Holocene fault; displacement during past 10,000 years, but not within the past 200 years.

Late Quaternary fault; displacement during past 700,000 years.

Quaternary fault; displacement during past 1.6 million years.

Pre-Quaternary fault; displacement older than past 1.6 million years.

Fault segment associated with a significant linear trend of accurately located earthquake epicenters.

Interpreted surface projection of Puente Hills Blind Thrust segments

0 15 30

APPROXIMATE SCALE, km

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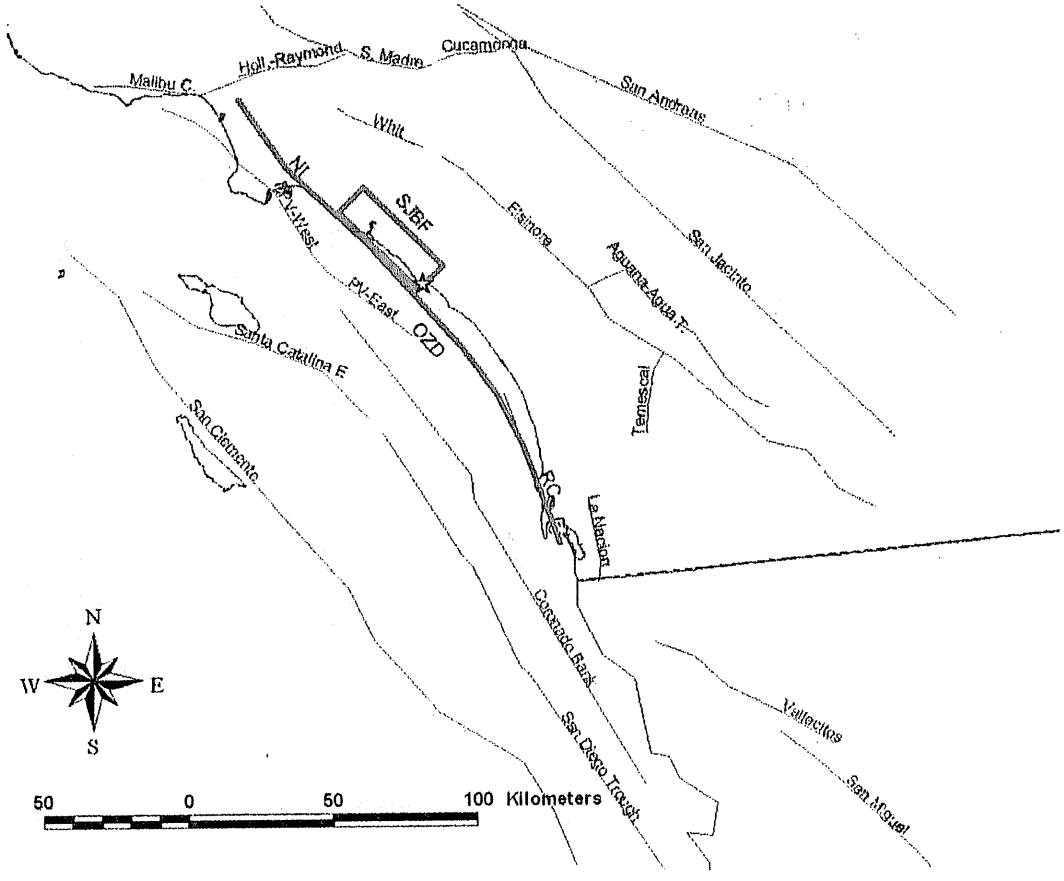
Regional Fault Map

Project: Dana Point Harbor Revitalization Project

Proj. No. 03006A | Figure 6



SJBF = SAN JOAQUIN HILLS BLIND THRUST FAULT
NI = NEWPORT-INGLEWOOD FAULT
OZD = OFFSHORE ZONE OF DEFORMATION
RC = ROSE CANYON FAULT

After: Hanson, et al. (2002)

LOCATION OF MODEL FAULT SOURCES - MODEL 1

Project No. 03006A	Date: JAN 2004	Project: Dana Point Harbor Revitalization Project	Figure 7

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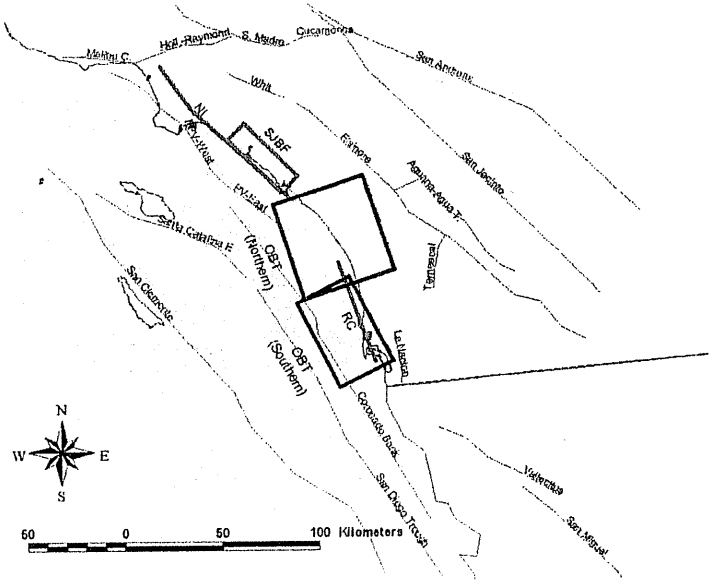

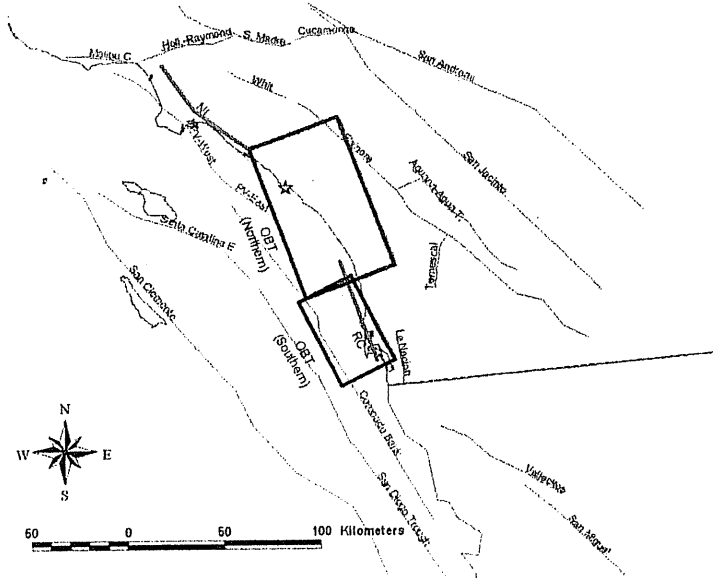

SJBF = SAN JOAQUIN HILLS BLIND THRUST FAULT
NI = NEWPORT-INGLEWOOD FAULT
RC = ROSE CANYON FAULT
OBT = OCEANSIDE BLIND THRUST FAULT

After: Hanson, et al. (2002)

LOCATION OF MODEL FAULT SOURCES - MODEL 2

Project No. 03006A	Date: JAN 2004	Project: Dana Point Harbor Revitalization Project	Figure 8

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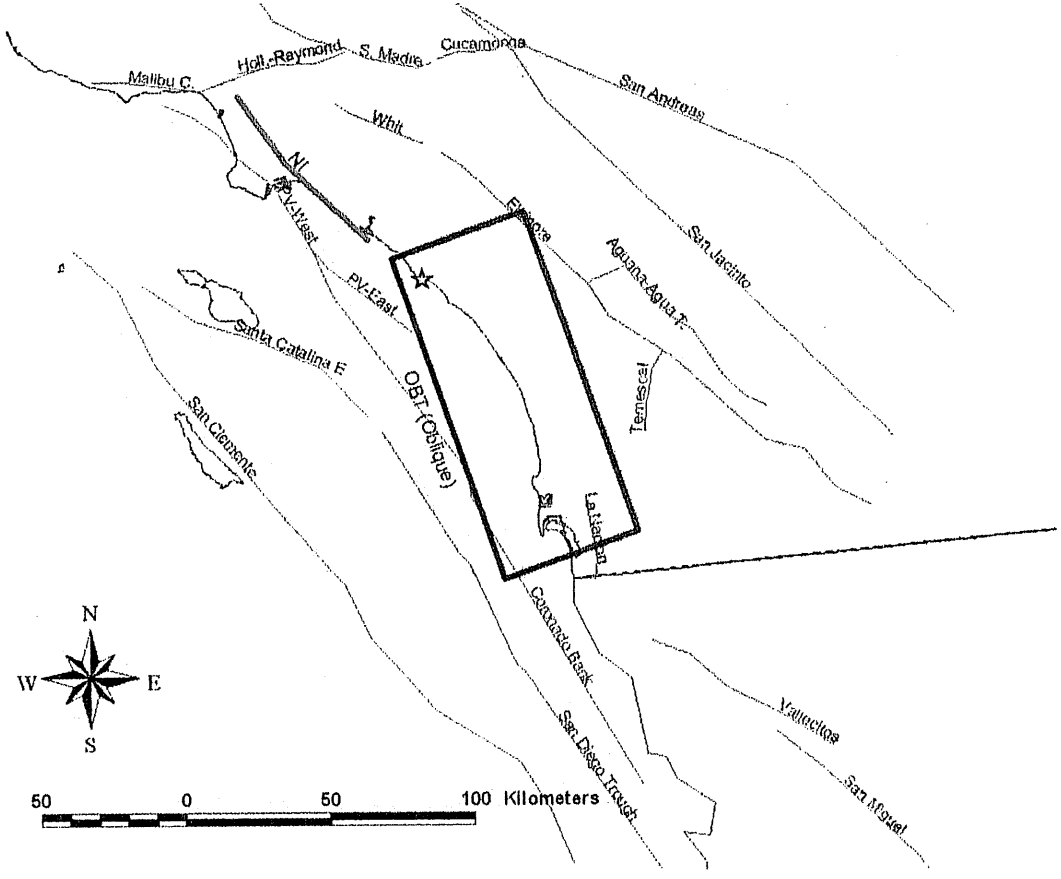

NI = NEWPORT-INGLEWOOD FAULT
OBT= OCEANSIDE BLIND THRUST FAULT

After: Hanson, et al. (2002)

LOCATION OF MODEL FAULT SOURCES - MODEL 3

Project No. 03006A	Date: JAN 2004	Project: Dana Point Harbor Revitalization Project	Figure 9

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Oceanside and Thirtymile Bank Thrusts, which comprise the Inner California Borderlands blind thrust system. They note that seafloor scarps associated with the shallow, east-vergent fold-and thrust belt above the Oceanside Thrust and between Dana Point and Oceanside may reflect recent activity of the underlying thrust and the complex anticline associated with the SCOZD. However, they acknowledge that this is not definitive due to the lack of precise age control on seafloor sediments.

4.0 POTENTIAL GEOLOGIC SEISMIC HAZARDS

Potential geologic hazards include surface fault rupture, seismic shaking, liquefaction, seismically induced settlement, landsliding, acute erosion, subsidence, collapsible/expansive soils, tsunami, seiches, dam inundation and flooding. An evaluation of the potential impacts on the site from these potential geologic hazards is presented in the following sections.

4.1 Fault Surface Rupture

No known active or potentially active faults are mapped through the site and, therefore, the site is not located within a currently designated Alquist-Priolo Earthquake Zone (Hart and Bryant, 1999). Based on this consideration, the potential for surface fault rupture at the site is considered low.

4.2 Seismic Shaking

Southern California is a seismically active region that can be expected to experience strong seismic shaking from future earthquakes generated by various active or potentially active faults. Dana Point Harbor could be subjected to strong ground shaking during a significant earthquake on a nearby or regional fault. Earthquakes that can produce strong shaking at the site may occur on mapped active (e.g., the Newport-Inglewood Fault Zone) or other postulated active faults (e.g., SCOZD) in the region, or on faults with little or no surface expression, such as the San Joaquin Hills Blind Thrust (SJHB) and the Oceanside Blind Thrust (OBT) Faults, discussed above. The location of the site in relation to known active/potentially active faults is shown on Figure 6, *Regional Fault Map.* Possible models for the location of postulated (SCOZD, SJHB, and OBT) faults are shown on Figures 7 through 9, *Location of Model Fault Sources – Model 1, 2 and 3*, respectively.

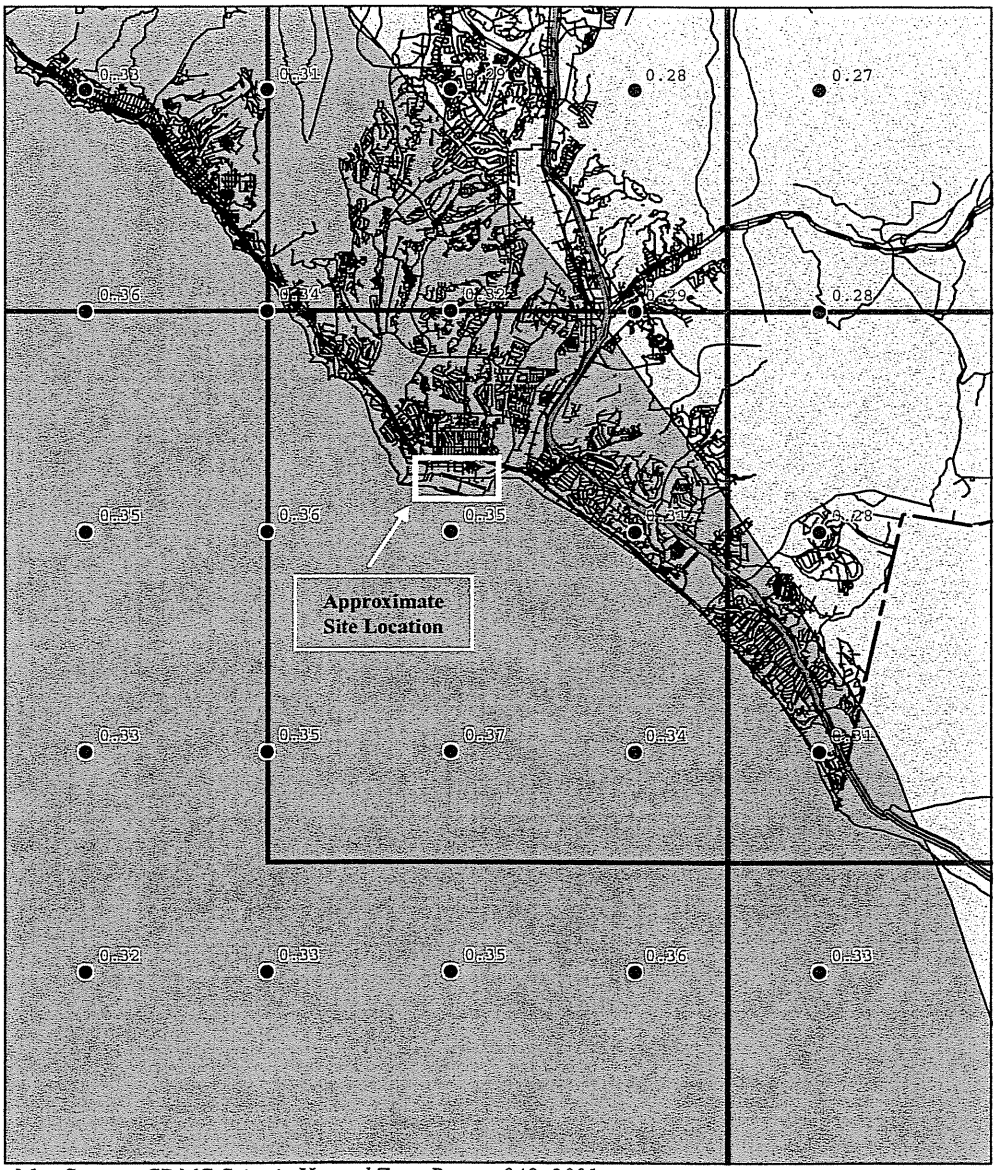
Mapped Peak Ground Accelerations

The estimated peak ground accelerations (PGA) in the Dana Point Harbor area are based on the seismic hazard maps for ground shaking prepared by the California Department of Conservation Division of Mines and Geology (CDMG) and presented in the Seismic Hazard Zone Report 049, (CDMG 2001). The CDMG maps are shown on Figure 10, *Peak Ground Acceleration Corresponding to 475-year Return Period for Soft Rock Conditions* and Figure 11, *Peak Ground Acceleration Corresponding to 475-year Return Period for Alluvium Conditions*. As can be seen in Figures 10 and 11, the mapped peak ground acceleration in the Dana Point Harbor area with an average return period of 475 years is approximately 0.35 g for soft rock conditions and approximately 0.37 g for alluvium conditions.

10% EXCEEDANCE IN 50 YEARS PEAK GROUND ACCELERATION (g)

1998

SOFT ROCK CONDITIONS



Map Source: CDMG Seismic Hazard Zone Report 049, 2001
(Dana Point 7.5 Minute Quadrangle and portions of adjacent quadrangles)

0 1.5 3
Miles

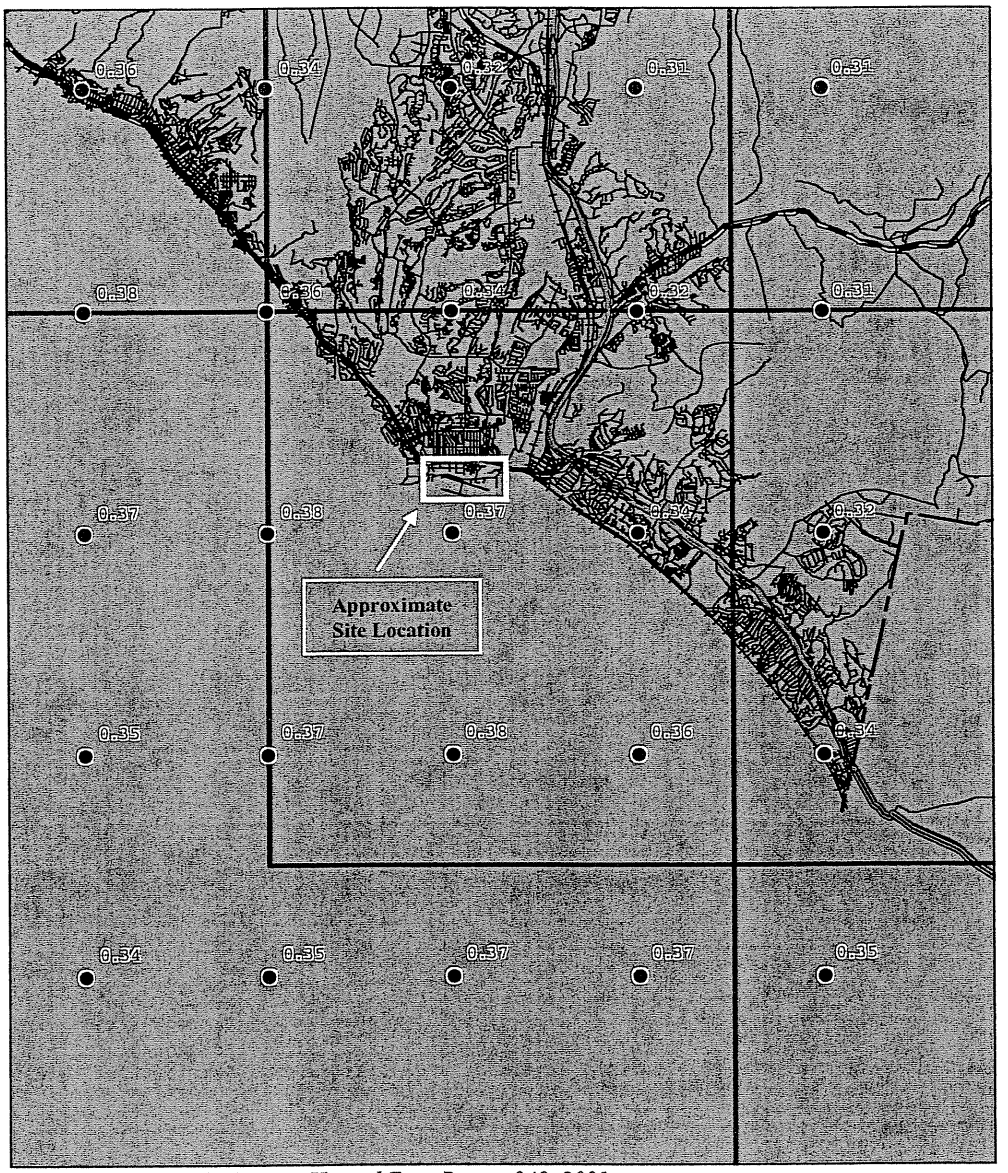
GeoPentech

Peak Ground Accelerations Corresponding to 475-year Return Period for Soft Rock Conditions

PROJECT: **Dana Point Harbor Revitalization Project**

PROJECT #: **030006A**	DATE: JAN **2004**	FIGURE: 10

10% EXCEEDANCE IN 50 YEARS PEAK GROUND ACCELERATION (g)
1998
ALLUVIUM CONDITIONS



Map Source: CDMG Seismic Hazard Zone Report 049, 2001
(Dana Point 7.5 Minute Quadrangle and portions of adjacent quadrangles)

0 1.5 3
Miles

GeoPentech

Peak Ground Accelerations Corresponding to 475-year Return Period for Alluvium Conditions

PROJECT: Dana Point Harbor Revitalization Project

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A ground motion study was not performed as part of this report. However, any ground motion evaluation for the Dana Point Harbor area should be rationally consistent with the peak ground acceleration value indicated on Figures 10 and 11, keeping in mind that, in the area of the Dana Point Harbor, the CDMG ground motion mapped values reflect only the Newport-Inglewood Fault Zone and do not consider all the postulated seismic sources discussed in Section 3.5 of this report.

Site-Specific Probabilistic Seismic Hazard Evaluation

A site-specific Probabilistic Seismic Hazard Analysis (PSHA) for the Dana Point Harbor was performed by Leighton & Associates (L&A 2002). Postulated ground motions at Dana Point Harbor were estimated using the program FRISKSP (Blake 2000). The Peak Horizontal Ground Accelerations (PHGA) for the Design Basis Earthquake (DBE), with a 10% probability of exceedance in 50 years, and the Maximum Credible Earthquake (MCE), with a 10% probability of exceedance in 100 years, were estimated using attenuation relationships based on Abrahamson and Silva (1997), Sadigh et al. (1997), and Campbell (1997 Rev.). The PHGA recommended by L&A was taken as the average of the PHGAs computed using each of the three relationships as 0.59g for the DBE and 0.78g for the MCE.

Alternative Models of Active Nearby Faults

Table 1 summarizes key fault characterization parameters of active nearby faults.

It is noted that the PSHA performed by L&A included the San Joaquin Hills Blind Thrust, Newport-Inglewood, Palos Verdes, Coronado Bank, and Rose Canyon Faults. However, the SCOZD and the Oceanside Blind Thrust, as described in Section 3.5 of this report, were not included. Because of the proximity of Dana Point Harbor to these faults, alternative seismic source characterization models, as listed in Table 1 and discussed below, may be appropriate for future seismic hazard analyses for Dana Point Harbor. In addition, because of its higher postulated slip rate, the SCOZD may represent a greater seismic shaking risk to the Dana Point Harbor.

On the basis of the available information, including Global Positioning System (GPS) data available for the region, possible alternative models of a system of faults consisting of the Newport-Inglewood/SCOZD/Rose Canyon Fault Zones, San Joaquin Hills Blind Thrust, and the Oceanside Blind Thrusts were developed (Hanson, et al, 2002; Moriwaki, et al, 2003). These alternative models are shown on Figures 7 through 9, *Location of Model Fault Sources – Model 1, 2 and 3*, respectively. Any ground motion assessment at the Dana Point Harbor area should reflect these alternative models in some fashion.

4.3 Liquefaction

Liquefaction can be defined as a significant and relatively sudden reduction in stiffness and shear strength of saturated sandy soils caused by a seismically or statically induced increase in pore water pressures. Potential for seismically induced liquefaction exists whenever saturated relatively loose sandy soils exist where the potential for seismic shaking is adequately high. In general the

consequences of liquefaction tend to be greater as the depth of the saturated sandy soils become shallower and its volume becomes larger. Also, the consequences of liquefaction become greater when a site has an open face such as the case in harbors.

As shown in Figure 12, *Seismic Hazard Zones Dana Point Quadrangle*, the site is located in a zone designated as having a potential for liquefaction based on the Seismic Hazard Zones Liquefaction Map for the Dana Point 7.5-Minute Quadrangle prepared by the CDMG (2001). However the Seismic Hazard Zone Report (SHZR) 049, notes that "*in the Dana Point Quadrangle, artificial fill areas large enough to show at the scale of mapping consist of engineered fill for elevated freeways, the Dana Point Harbor, and some of the mass graded areas. Since these fills are considered to be properly engineered, zoning for liquefaction in such areas depends on soils conditions in underlying strata*" (CDMG 2001).

Review of the available documents indicates a wide range of conclusions reached regarding the potential for liquefaction at the Harbor. The reason for these differing opinions/conclusions is primarily due to the characterization of fill placed during the development of the Harbor.

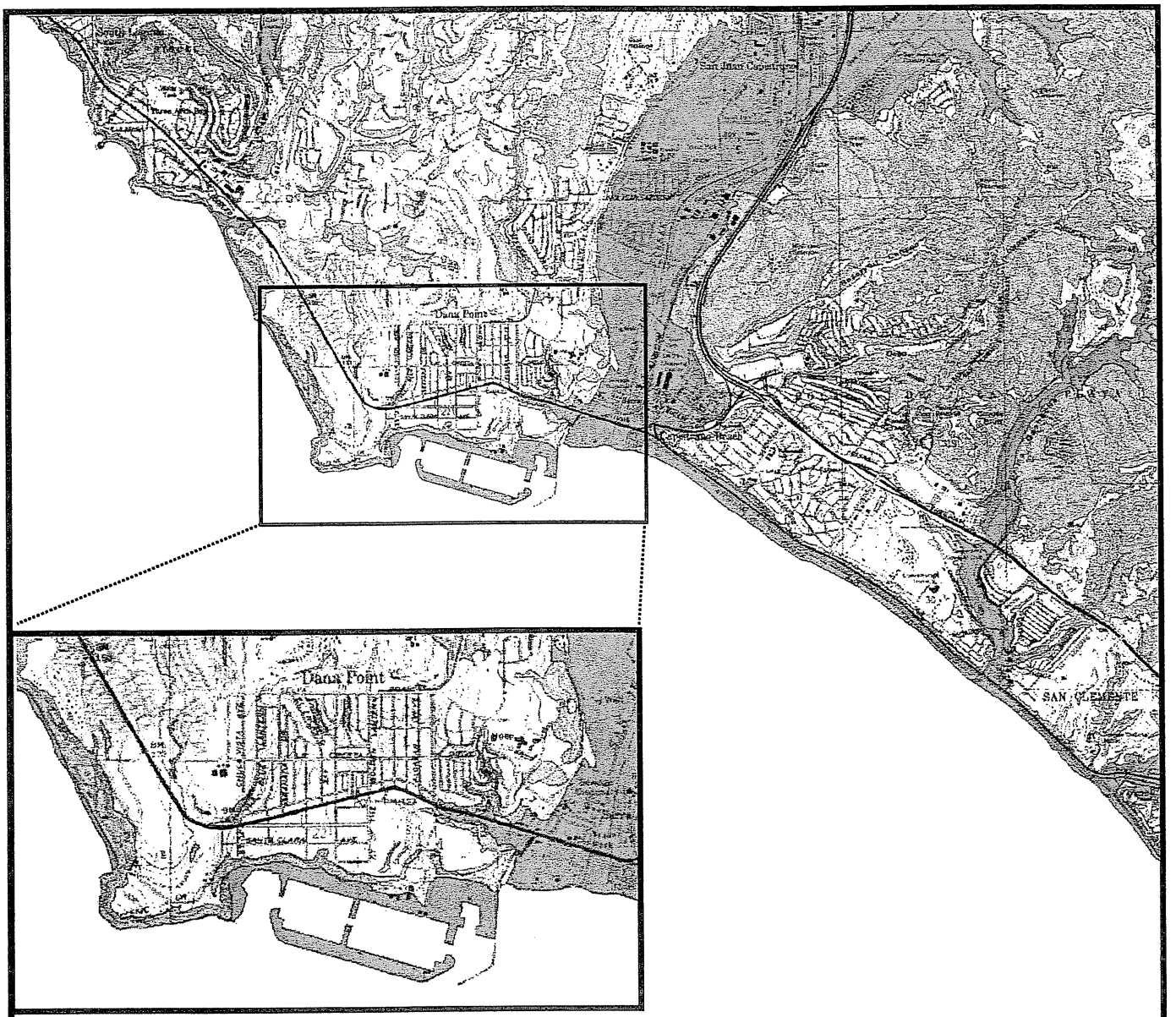
The following is a summary from previous and recent assessments for liquefaction potential:

- The 1973 Final EIR (Environmental Sciences, 1973) states "*the interior harbor was constructed in the dry after breakwater placement, with fill material utilized from adjacent cliffs*". The Final EIR also references reports of fill compaction and documentation and concludes "*Fill areas have been compacted and constructed to standard design values and are suitable for all proposed structures. The materials are dry and have an internal structural makeup not subject to liquefaction in the event of a severe earthquake. The soil materials in the compacted fill areas vary by depth. Coarse granular material is overlain with fine-grained soils above the water level, made up of cut material from the Capistrano Formation*".

- The 2002 Master Plan states that the Harbor was constructed by "*filling the tidelands, dredging the proposed Harbor Basin, and constructing a quay wall to provide a bulkhead for the Harbor Development and Marina areas*". The Master Plan also states that the soils consist of "*undocumented fill*".

- Leighton and Associates, in their 2002 report, characterized the consistency of the 15 to 20 foot thick fills encountered during their recent investigation as varying from "*loose to medium dense and firm to stiff*" and stated that "*Engineering analysis suggested that the potential for localized liquefaction of sandy soils in the fill is high during a major earthquake event. The thickness of these potentially liquefiable soils encountered during our exploration varied from a few inches to up to 5 feet*" (L&A 2002).

The L&A (2002) borings and cone penetration test (CPT) results indicate that the 15 to 20 foot thick fills are predominantly clayey. In general, compacted fill consisting of predominantly clayey soils



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: Seismic hazard Zones identified on this map may include developed land where delineated hazards have already been mitigated to city or county standards. Check with your local building/planning department for information regarding the location of such mitigated areas.

DATA AND METHODOLOGY USED TO DEVELOP THIS MAP ARE PRESENTED IN THE FOLLOWING:

Seismic Hazard Zone Report of the Dana Point 7.5-minute quadrangle, Orange County, California California Division of Mines and Geology, Seismic Hazard Zone Report 045

GeoPentech

SEISMIC HAZARD ZONES
DANA POINT QUADRANGLE

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would not be expected to pose significant liquefaction hazard. However, based on the variability of the fill soils encountered, including the occurrence of some lenses of silty sand and silt several inches thick, and the possibility that some dredged material may have been used in the apparently compacted fills, the possibility of liquefaction and/or lateral spreading at the site cannot be discounted. Further investigation/evaluation is required to assess the likely extent of the potential for liquefaction and lateral spreading at individual sites.

4.4 Lateral Spreading

Lateral spreading tends to occur when a layer of liquefied soils moves laterally toward a slope or an open face. Lateral spreading can move the ground by a few inches to tens of feet causing ground fissures, grabens, and significant damage to structures on the ground. If the potential for lateral spreading is high, it would be very difficult to design structures to withstand such movements.

As discussed in Section 4.3, the possibility of liquefaction and/or lateral spreading at the Harbor site cannot be discounted. Further investigation/evaluation is required to evaluate the likely extents of potential liquefaction and lateral spreading at individual sites. Further site-specific subsurface investigations would be desirable to quantify the potential for lateral spreading because the variable fill soils appear to be predominantly clayey, and may not be as susceptible to liquefaction and/or lateral spreading as the mapping (Figure 12, *Seismic Hazard Zones Dana Point Quadrangle*) of the site may indicate.

4.5 Seismically Induced Settlement

Seismically induced settlement is caused by loose to medium-dense granular soils densified during ground shaking or following ground shaking as induced excess pore water pressures dissipate. Dry and partially saturated soils as well as saturated granular soils are subject to seismically induced settlement. Leighton and Associates (L&A, 2002) reported that some loose to medium-dense sandy layers were encountered in the fill soils underlying the site during their recent field investigation. Based on their evaluation, L&A concluded that densification of these layers and associated seismically induced settlements may occur during earthquake shaking.

4.6 Landsliding

No significant slopes exist within the proposed project site. Therefore, the potential for onsite landsliding is not considered a potential impact for the proposed development.

The cliffs behind the project area have been historically subject to landsliding (Zeiser 1990). Slope repair and landslide mitigation on the historic landslide area has been accomplished by shotcrete and rock anchors (Kerwin 1989).

Periodic slumping of cliff materials may also be anticipated due to continued erosion of the friable Capistrano Formation sandstones along this sea cliff, leaving massive resistant conglomerate

interbeds precariously undermined (Zeiser 1990). However, the landsliding hazard to the proposed project has been mitigated by positioning Del Obispo Road against the cliff with a landscaped strip acting as a barrier; therefore, these cliffs pose no significant impact to the proposed development.

Factors that have been identified as contributing to the landsliding hazard for these slopes include bluff face instability, seepage, block falls, and adverse bedding (Zeiser1990). The proposed development is not anticipated to impact or contribute to the factors causing a landsliding hazard for these slopes.

4.7 Erosion

The ground surface at the site of the proposed developments is relatively flat and areas surrounding the buildings will be covered with pavements and landscaping. As such, a geological hazard related to surface water erosion is considered unlikely for the proposed development areas.

Construction of the harbor breakwaters effectively shield the bluff face cliffs behind the project area from further marine (storm wave) erosion processes (Zeiser 1990). However, rainfall, irrigation of the properties within the blufftop residential zone, storm drain outfalls, and relatively poor surface-drainage control conditions accelerate erosion of friable Capistrano Formation sandstones along this seacliff (Zeiser 1990). As noted above, the potential hazard to the development has been mitigated by positioning Del Obispo Road against the cliff with a landscaped strip acting as a barrier between the cliffs and the proposed development areas.

The proposed development is not anticipated to impact or contribute to the factors causing erosion on these slopes.

4.8 Subsidence

The site is not within an area of known subsidence associated with fluid withdrawal (groundwater or petroleum), peat oxidation, or hydrocompaction.

4.9 Collapsible/Expansive Soils

Based on the results of the preliminary field investigation and laboratory testing performed by Leighton and Associates (2002), the near surface fill soils within the areas of investigation exhibit a low expansion potential. However, further sampling and testing during the design phase is recommended to confirm the preliminary findings.

4.10 Tsunami

A tsunami is a sea wave generated by a large submarine landslide or an earthquake-related ground deformation beneath the ocean. Historic tsunamis have been observed to produce a run-up on shore of several tens of feet in extreme cases.

The tsunami hazard in Southern California has been qualitatively calculated as moderate south of Palos Verdes to San Diego (Borrero et. al, 2003). McCulloch (1985) predicted a 100-year tsunami event could result in a run up of about 4 feet above mean sea level in the vicinity of Laguna Beach and the Dana Point coast. Dana Point Harbor is partially shielded from tsunami waves by the headlands, which deflect ocean waves approaching the shore from the west. However, the Harbor could incur significant damage in the event that a tsunami generated in the Southern Pacific struck Dana Point. Inundation maps are currently being developed for California under the U.S. National Tsunami Hazard Mitigation Program. Preliminary estimates for tsunami run-up heights on the southern California coast have been predicted to range from 6 to 18 feet for a tectonically triggered tsunami event (Borrero et. al. 2003). However, inundation maps showing site-specific wave run-ups at Dana Point Harbor have not, as yet, been published.

As a result, the tsunami hazard should be considered during the design phase of the development. Further study of the potential impacts of inundation on the existing or proposed building structures along the seawall should be performed during the design phase.

4.11 Seiches

Seiches are large oscillating waves generated in enclosed bodies of water in response to ground shaking. Because of the partially enclosed configuration of the Dana Point Harbor, there is a possibility of seiche occurring onsite. Further study of wave run up near the Harbor during a major seismic event should be performed during the design phase.

4.12 Inundation and Flooding

The potential hazard for dam inundation and seismically induced flooding is generally associated with seismically induced failure of a reservoir located on a drainage upstream of a site. The Project site is not located downstream or downslope of any reservoir or water storage facilities that could adversely affect the Project site in the event of earthquake-induced failures. Therefore, the potential for seismically induced flooding is considered remote.

Dana Point Harbor has been identified as being within the 100-year flood hazard zone of San Juan Creek, as well as within a mapped coastal flood with velocity hazard in accordance with the Federal Emergency Management Agency Flood Insurance Rate Map of Orange County (Cotton/Beland/Associates 1991). Further study of the potential impacts of flooding of the San Juan Creek on the existing or proposed building structures along the seawall should be performed during the design phase.

5.0 SIGNIFICANT GEOLOGICAL/ GEOTECHNICAL IMPACTS

This preliminary report has been prepared to address significant geological/geotechnical impacts for incorporation into the Draft EIR. Potential geological/geotechnical impacts are identified in Table 2.

The significance criteria for potential geological/geotechnical impacts listed in Table 2 have been evaluated in accordance with the California Environmental Quality Act (CEQA) significance criteria for Geology and Soils and Mineral Resources (as listed in *Appendix G – Environmental Checklist Form* of the CEQA Guidelines). Potential environmental issues/impacts listed in Appendix G of the CEQA Guidelines, but not related to geology and soils are not included in Table 2 of this report.

Issues evaluated to have "no impact" as indicated in Table 2 are not addressed further. The following significant impacts are identified in Table 2:

- Strong Seismic Ground Shaking.
- Liquefaction.
- Lateral Spreading.
- Inundation by Tsunami and/or Flooding on the San Juan Creek.

These impacts are further described in Table 3 as they relate to the Planning Areas addressed, the seawall and the proposed construction parking areas.

Other impacts from the project related to soils excavation and geotechnical considerations, not included in Table 2 are those due to construction. Specifically, the following construction impacts may be anticipated:

- **Vibrations**: If piles are used for foundation support of the proposed new structures, it is anticipated that pre-fabricated piles will be driven into the existing fill soils within the harbor (landside) area. Noise, odor, and vibration should be anticipated.
- **Siltation/Sedimentation Due to Erosion of Open Cut Construction Areas**: Due to pre-existing grading and the relative flatness of the site, this impact will most likely be slight.
- **Construction Spoils:** Grading and excavations for new foundations and utilities will generate construction spoils. Construction spoils stockpiled on-site may pose additional dust, erosion, and air quality impacts. If encountered, soils contaminated with fuel and/or other constituents of concern may require special handling and disposal.

These additional construction impacts should be addressed further in the Draft EIR.

6.0 MITIGATION MEASURES

Preliminary mitigation measure recommendations for the potential geological/geotechnical impacts identified (strong seismic ground shaking; liquefaction, lateral spreading, inundation by tsunami and/or flooding) are described in Table 3. In general, mitigation measures include the following:

- Further investigation and characterization of the subsurface conditions on a site-by-site basis for existing and proposed structures.
- Site-specific evaluation of the magnitude of potential impacts on various portions of the site.
- Repair and/or replacement of existing structures and incorporation of mitigation measures into the design of proposed improvements.

Further details of the mitigation measures are provided in Table 3 as they relate to the Planning Areas addressed, the seawall and the proposed construction parking areas.

7.0 GENERAL CONDITIONS

This preliminary report was prepared based on available documents provided by RBF Consulting. GeoPentech has not performed any field exploration, laboratory testing or engineering analyses to supplement the available information.

Professional judgments presented in this report are based on an evaluation of the technical information gathered and GeoPentech's general experience in the field of geotechnical engineering. GeoPentech does not guarantee the performance of the Project in any respect, only that the engineering work and judgment rendered meet the standard of care of the geotechnical profession at this time.

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9.0 GLOSSARY OF GEOLOGIC TERMS

alluvium – Detrital deposits made by streams on river beds, flood plains, and alluvial fans; esp. a deposit of silt or silty clay laid down during time of flood. The term applies to stream deposits of recent time. It does not include subaqueous sediments of seas and lakes.

average return period – (ARP) is the reciprocal of the mean annual rate of exceedance for a particular magnitude earthquake. For example, and a particular magnitude earthquake having a 10% probability of exceedance in 50 years has an average return period of 475-years.

bedded – Arranged or deposited in layers or beds, esp. said of sedimentary rocks; stratification.

bedrock – The solid rock that underlies gravel, soil, or other superficial material.

breccia – A coarse-grained clastic rock, composed of angular broken rock fragments held together by a mineral cement or a fine-grained matrix.

clastic – Pertaining to a rock or sediment composed principally of fragments derived from pre-existing rocks or minerals and transported some distance from their places of origin.

coarse-grained – Said of a sediment or sedimentary rock, and of its texture, in which the individual constituents are easily seen with the unaided eye, i.e. have an average diameter greater than 2 mm (0.08 in). The term is used in a relative sense, and various size limits have been suggested and used.

conglomerate – A coarse-grained clastic sedimentary rock, composed of rounded to subangular fragments larger than 2 mm in diameter set in a fine-grained matrix of sand or silt, and commonly cemented by calcium carbonate, iron oxide, silica, or hardened clay; the consolidated equivalent of gravel.

earthquake magnitude – A measure of the strength of an earthquake, or the strain energy released, as determined by seismographic observations (see also moment magnitude).

fault – A fracture or fracture zone along which there has been displacement of the sides relative to one another parallel to the fracture.

fault contact –Describes when dissimilar geologic units are found in immediate contact due to faulting.

fault zone – A fault that is expressed as a zone of numerous small fractures or of fault breccia or gouge. A fault zone may be hundreds of meters wide.

groundwater level (water table) – The surface between the zone of saturation and the zone of aeration; that surface of a body of unconfined ground water at which the pressure is equal to that of the atmosphere.

interbed – A bed, typically thin, of one kind of rock material occurring between or alternating with beds of another kind.

marine terrace – A wave-cut platform that has been exposed by uplift along a seacoast or by lowering of sea level; an elevated marine-cut bench.

massive –Rocks of any origin that are more or less homogeneous in texture or fabric, displaying an absence of flow-layering, foliation, cleavage, joints, fissility, or thin bedding.

matrix - The finer-grained material enclosing the larger grains in a sediment or sedimentary rock.

moment magnitude (Mw) – related to the physical size of fault rupture and the movement (displacement) across the fault, and as such is a more uniform measure of the strength of an earthquake.

physiographic province – A region of which all parts are similar in geologic structure and climate and which has had a unified geomorphic history; its relief features differ significantly from those of adjacent regions.

sedimentary rock – A layered rock resulting from the consolidation of sediment, e.g. a clastic rock such as sandstone.

shear – A deformation resulting from stresses that cause contiguous parts of a body to slide relatively to each other in a direction parallel to their plane of contact. Also used to refer to surfaces and zones of failure by shear, and to surfaces along which differential movement has taken place.

Richter Scale – A numerical scale of earthquake magnitude, devised in 1935 by the seismologist C.F. Richter.

Reference:

Dictionary of Geological Terms, 3rd edition, American Geological Institute, R.L. Bates & J.A. Jackson, ed.; Anchor Books, 1984.

Table 2

CEQA Guidelines Appendix G Environmental Checklist Form (Excerpt)

VI. GEOLOGY AND SOILS	Potentially Significant Impact	Less Than Significant Impact With Mitigation Incorporation	Less Than Significant Impact	No Impact
Would the project: a) Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:				
i) Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault? Refer to Division of Mines and Geology Special Publication 42.				✓
ii) Strong seismic ground shaking?	✓			
iii) Seismic-related ground failure, including liquefaction?	✓			
iv) Landslides?				✓
b) Result in substantial soil erosion or the loss of topsoil?				✓
c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse?		✓		
d) Be located on expansive soil, as defined in Table 18- 1-B of the Uniform Building Code (1994), creating substantial risks to life or property?				✓
e) Have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water?				✓
VIII. HYDROLOGY AND WATER QUALITY	**Potentially Significant Impact**	**Less than Significant Impact with Mitigation Incorporation**	**Less than Significant Impact**	**No Impact**
Would the project: a) Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:				
Items a) through i) as outlined in Appendix G of the CEQA guidelines are not addressed in this report.	Not addressed by this report. Refer to Draft EIR Document			
j) Inundation by seiche, tsunami, or mudflow?	✓			
X. MINERAL RESOURCES	**Potentially Significant Impact**	**Less than Significant Impact with Mitigation Incorporation**	**Less than Significant Impact**	**No Impact**
Would the project: a) Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:				
a) Result in the loss of availability of a known mineral resource that would be of value to the region and the residents of the state?				✓
b) Result in the loss of availability of a locally-important mineral resource recovery site delineated on a local general plan, specific plan or other land use plan?				✓

Table 3
Potential Impacts and Remedial Measures

Facility/Structure:		Strong Seismic Ground Shaking	Liquefaction	Lateral Spreading	Inundation by Seiche Tsunami and Flooding
Planning Areas 1 through 5 **(Commercial Core , Island, Youth and Group Facility)**	**Potential Impacts**	Impacts of strong ground shaking may include: -Potential architectural and structural damage to existing and proposed buildings and utilities. -Seismically induced ground failures.	Impacts of potential liquefaction within the existing fill may include: -Bearing capacity failures -Settlement and damage to existing and proposed structures, utilities and surface improvements (such as pavements). -Damage due to differential settlements and uneven surfaces. -Damage to utilities and rigid pipes due to dislocation at connections/joints.	Due to the proximity of the proposed and existing structures to the existing bulkheads, new and proposed landside facilities and improvements would be severely impacted in the event of lateral spreading of the supporting and retained fill soils. Damage due to bearing capacity failures, ground fissuring, excessive horizontal and vertical displacements, and slope failures may be anticipated.	Inundation by flooding, and/or the wave run-up from tsunami and earthquake induced seiche could impact existing and proposed structures.
	Mitigation Measures	Seismic ground shaking hazard is common in Southern California, and the effects of ground shaking can be mitigated if the proposed structures are designed in conformance to current building codes and engineering practices. Ground motion assessments for design should reflect the recently developed alternative models for potential faults in the vicinity of the harbor.	Further investigation and detailed characterization of the existing fill conditions is required to define the liquefaction potential extents. Mitigation measures may include: -Supporting proposed structures on deep foundations extending into bedrock. -Stiffened floor slab designs. -Total/partial removal of the potentially liquefiable soils and replacement with compacted fill. -Soil Remediation/Site Improvement.	Further evaluation of lateral spreading potential is required. In the event that the lateral spreading potential is high remedial measures may include: -Repair or replacement of existing seawall for site containment. -Total/partial removal of the potentially liquefiable soils and replacement with compacted fill. -Soil remediation/ Site Improvement.	Site-specific evaluation of the magnitude of potential inundation from flooding, tsunami and/or seiche and the risks involved is required prior to incorporation of mitigation measures into the design of the proposed development.

Table 3, continued
Potential Impacts and Remedial Measures

Facility/Structure:		Strong Seismic Ground Shaking	Liquefaction	Lateral Spreading	Inundation by Seiche Tsunami and Flooding
Planning Areas 9 and 10 **(East and West Marinas, Seawall/bulkhead)**	**Potential Impacts**	Impacts of strong ground shaking may include: -Excessive earth pressures due to dynamic loading on existing bulkhead could cause the existing bulkhead/quay wall to experience significant lateral movement and possibly collapse. -Seismically induced ground failures behind the sea wall.	Impacts of potential liquefaction within the existing fill soils retained by and supporting the seawall may include: -Bearing capacity failure of existing gravity-wall retaining section. -Settlement and damage of the wall. -Slope failures in the existing rip-rap or concrete-slab faced slope.	Significant lateral movement of the existing bulkhead can be expected in the event of lateral spreading of the supporting and retained fill soils. Damage at seawall due to bearing capacity failure, horizontal and vertical displacements, and slope failures at rip-rap and concrete faced slopes may be anticipated.	The inundation due to flooding, wave run-up from a tsunami and earthquake induced seiches could significantly impact the existing seawall. Impacts could include overtopping of the existing seawall, and scouring of the rip-rap and concrete slope faces and undermining of supporting footings.
	Mitigation Measures	Repair/retrofit or replacement of existing bulkhead and slope containment system. Alternatives proposed in the Master Plan include, repair/retrofit of existing bulkhead to provide additional lateral stability during earthquake loading(such as installing a tie-back and wale system), slope reinforcement, soil remediation, and/or complete replacement of existing seawall.	Further evaluation of liquefaction potential of retained soils is required. Mitigation measures may include: - Repair/retrofit of existing bulkhead (including underpinning), slope reinforcement, and/or complete replacement of existing seawall. -Excavation of liquefiable soils and replacement with compacted fill. -Soil Remediation/Site Improvement.	Further evaluation of lateral spreading potential of retained soils is required. In the event that the lateral spreading potential is high remedial measures may include: -Repair/retrofit of existing seawall, slope reinforcement (including sheet piles to improve slope stability and confinement of liquefiable soils), and/or complete replacement of existing sea wall. -Excavation of the potentially liquefiable soils. -Soil remediation/ Site Improvement.	Specific evaluation of the potential risks involved in the above described impacts and incorporation of appropriate design measures.

Table 3, continued
Potential Impacts and Remedial Measures

Facility/Structure:		Strong Seismic Ground Shaking	Liquefaction	Lateral Spreading	Inundation by Seiche Tsunami and Flooding
Planning Areas 9 and 10 **(East and West Marinas, Dock work involving the replacement of docks and moving guest slips)**	**Potential Impacts**	Potential damage to docks, mooring facilities due to strong ground shaking in the event of an earthquake may include damage and potential structural failure of the mooring facilities and docks due to excessive deformations.	Potential for liquefaction within the mooring basin will depend on the condition of existing soils supporting the dock pilings. Consequences of liquefaction may include loss of frictional capacity in soils supporting pilings, excessive lateral loading of pilings and resulting deformations of mooring facilities.	Consequences of lateral spreading within the mooring basins, adjacent to existing pilings may include loss of frictional capacity in soils supporting pilings and excessive lateral loading of pilings and resulting deformations/damage of mooring facilities	Inundation by flooding, and/or the wave run-up from tsunami and earthquake induced seiche could impact existing and proposed docks and dock facilities.
	Mitigation Measures	Evaluation of existing docks and mooring facilities and their replacement should be in conformance to current building codes and engineering practices. Ground motion assessments should reflect the recently developed alternative models for potential faults in the vicinity of the harbor.	Further investigation and detailed characterization of the existing fill conditions along with liquefaction analysis. Structural analysis of the mooring facilities is also required. Supporting the new mooring facilities into bedrock.	Specific evaluation of lateral spreading is required along with evaluation of the stability and integrity of the mooring facilities. Based on the results of the analysis, repairs of the existing facilities and/or replacement may be made incorporating the new findings.	Specific evaluation of the potential risks involved in the above described impacts and incorporation of appropriate design measures.

Table 3, continued
Potential Impacts and Remedial Measures

Facility/Structure:		Strong Seismic Ground Shaking	Liquefaction	Lateral Spreading	Inundation by Seiche Tsunami and Flooding
Construction Parking Areas **South Coast Water District Parcel**	**Potential Impacts**	Impacts of strong ground shaking may include: -Potential damage to existing and proposed structures and utilities. -Seismically induced ground failures.	Based on the CDMG maps potential of liquefaction exists in this area. No specific site investigation was available. Consequences of liquefaction may include bearing capacity failures, and damage to facilities, utilities and pavements due to seismically-induced settlements.	Potential for lateral spreading at the parcel will depend on the site-specific soil conditions and topography. But as the site is not bounded by open-faced slopes the potential for lateral spreading is considered low.	Parcel is located about ½ mile from shore, within 100-year flood hazard zone of San Juan Creek, as well as within a mapped coastal flood and velocity hazard zone. Inundation by flooding and/or the wave runup from a tsunami could impact proposed facilities at the site.
	Mitigation Measures	Seismic ground shaking hazard is common in Southern California, and the effects of ground shaking can be mitigated if the proposed structures are designed in conformance to current building codes and engineering practices. Ground motion assessments for design should reflect the recently developed alternative models for potential faults in the vicinity of the harbor.	Site specific geotechnical investigation is required to characterize the subsurface condition and the potential for liquefaction at the parcel. Remedial measures can be selected once the subsurface conditions and the type of structures are known.	Site specific geotechnical investigation is required to characterize the subsurface condition and the potential for lateral spreading at the parcel.	Specific evaluation of the potential risks involved in the above described impacts and incorporation of appropriate design measures.

Table 3, continued
Potential Impacts and Remedial Measures

Facility/Structure:		Strong Seismic Ground Shaking	Liquefaction	Lateral Spreading	Inundation by Seiche Tsunami and Flooding
Construction Parking Areas **Selva Parking Lot**	**Potential Impacts**	Because not structures or utilities exist on this site and based on the proposed use of this parcel as overflow parking, no significant impact due to ground shaking is anticipated.	The site is not located in an area identified as having the potential for liquefaction, based on the CDMG Seismic Hazard Zones map for the Dana Point Quadrangle. However, site specific geotechnical investigation would be required to confirm the subsurface conditions and potential for liquefaction if major structures are to be placed in this parking lot.	Potential for lateral spreading at this site is remote.	Site is located about 1/4 mile from shore on an elevated bluff therefore there is no potential impact from inundation by seiche or tsunami. In addition, the parcel is not located within a mapped flood plain.
	Mitigation Measures	No remediation is required.	No remediation is required.	No remediation is required.	No remediation is required.

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT FOR THE DANA POINT HARBOR MASTER PLAN, DANA POINT HARBOR, CITY OF DANA POINT, CALIFORNIA

Prepared for:

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1133 Columbia Street, Suite 205
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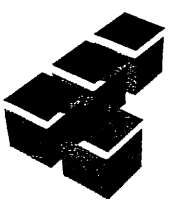
Project No. 600024-001

December 12, 2002



Leighton Consulting, Inc.

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A LEIGHTON GROUP COMPANY

December 12, 2002

Project No. 600024-001

To: Wallace, Roberts & Todd, Inc.
1133 Columbia Street, Suite 205
San Diego, California 92101

Attention: Mr. Richard Barrett

Subject: Preliminary Geotechnical Investigation Report for the Dana Point Harbor Master Plan, Dana Point Harbor, City of Dana Point, California

Leighton Consulting, Inc. has performed a preliminary geotechnical investigation for the proposed Dana Point Harbor Master Plan in the city of Dana Point, California. The purpose of this investigation is to evaluate the subsurface conditions of the site and provide recommendations for the current plan.

The current plan proposes to construct approximately eight new buildings housing retail spaces, two parking decks, and a boat storage building. Based on the results of our investigation and analysis, the proposed development at the site is feasible from a geotechnical standpoint. A summary of key findings of our investigation is presented in the following paragraphs.

The site is generally covered with fill soils overlying bedrock. The thickness of the fill varies from 2 to 20 feet. It is likely that the fills were placed during construction of the harbor facility. The groundwater across the site generally coincides with the sea level. Depending the site topography, depth to groundwater varied from approximately 7 feet below ground surface near the northern end of the site to approximately 3 feet below ground surface at the southern end of the site.

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Results of our exploration indicated that the consistency of the existing fill soils is non-uniform across the site. To minimize the impact on the proposed development, auger cast-in-place piles and driven piles are considered for the project. For minor structures, conventional shallow foundation established on at least 3 feet of compacted fill may be used.

Moderate to strong ground shaking at the site is anticipated should an earthquake occur along one of the regional faults. The potential for localized liquefaction resulting from a strong earthquake is considered high at the site. Also, seiches and tsunamis are design concerns due the configuration of the harbor and close proximity of the site to the Pacific Ocean.

Based on laboratory testing results, the expansive potential of the onsite soil is low. The onsite soil, however, is corrosive to metals and concrete. Special design and construction provisions will be required for protection of buried metals and concrete in contact with the soil.

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,

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1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the results of a geotechnical investigation for the Dana Point Harbor Master Plan in the city of Dana Point, California. The purpose of this study was to evaluate the subsurface conditions and provide preliminary recommendations for the project plan. The scope of our work included the following tasks:

- Review of available geologic reports and maps.
- Notification of Underground Service Alert (USA) of marked boring locations prior to the commencement of our field exploration and coordination of a drilling contractor.
- Subsurface exploration consisting of excavation, logging, and sampling of three hollow-stem auger borings and eight Cone Penetrometer Test (CPT) within the project area.
- Collection of relatively undisturbed ring samples and bulk samples at selected depth intervals from the three soil borings and transportation of the samples to our laboratory for testing.
- Laboratory testing of selected samples to evaluate engineering characteristics of the onsite earth materials within the exploration depths.
- Geotechnical evaluation of collected data and relevant engineering analyses.
- Preparation of this report summarizing our findings, conclusions, and recommendations.

1.2 Site Location and Proposed Development

The project area is bounded by Dana Point Harbor Drive to the north, Puerto Place to the east, Casitas Place to the west, and a sea wall to the south (Figure 1). The site is currently occupied with boat storage parking areas, automotive parking lots, several restaurants and retail shops, and a County of Orange maintenance facility yard.

The preliminary plan indicates that the proposed development will consist of approximately eight new buildings housing retail spaces near the harbor, two parking decks, and a boat storage building as depicted on Figure 2. The exact dimensions, locations, and structural loading information will not be determined until the next phase of the design.

2.0 FIELD AND LABORATORY INVESTIGATION

2.1 Field Investigation

Prior to the field investigation, a site reconnaissance was performed by a geotechnical engineer from our staff to mark the proposed boring and CPT locations and to evaluate the marked locations with respect to access for heavy equipment. USA was then notified of the marked locations.

Three soil borings (B-1 through B-3) were drilled within the project area on November 6, 2002. Depth of soil borings varied between approximately 20 and 50 feet. In addition to the soil borings, a total of eight CPT sounding were performed on November 15, 2002 at selected locations. CPT soundings met with refusal at various depths from 2 to 20 feet. Locations of the soil borings and CPTs are shown on Figure 2, Boring Location Map.

All borings were continuously logged during drilling. Samples were collected at selected intervals using California samplers and/or Standard Penetration Test (SPT) samplers. At each sampling interval, the California or the SPT samplers were driven into the bottom of the boring with successive drops of a 140-pound weight falling 30 inches. Relatively undisturbed soil samples were collected in 1-inch rings placed inside the California sampler. In addition, bulk samples of the soils were collected from selected depths. SPTs were performed in general accordance with the American Standard Test Method (ASTM) D1586-84 Standard Test Method. The blowcounts (i.e. "N-value") for every 6 inches of the 18-inch SPT and California samplers were also included in the boring logs. The soil consistency was evaluated using the N-values and blowcounts of the ring samplers converted to equivalent SPT blowcounts. The boring logs are included in Appendix B.

Each sample collected was classified in the field in general conformance with the Unified Soil Classification System (USCS) and subsequently verified in the laboratory. All samples were sealed and packaged for transportation to our laboratory for testing. Upon completion of the drilling, the boreholes were backfilled with soil cutting.

The CPT soundings were performed in accordance with ASTM D5778 and D3441. The CPT logs are also included in Appendix B.

2.2 Laboratory Testing

Laboratory tests were performed on representative samples to verify the field classification of the recovered samples and to determine the geotechnical properties of the subsurface materials. The following tests were performed:

- Sieve Analysis;
- Direct Shear;
- Consolidation;
- Compaction;
- Atterberg Limits;
- R-value;
- In-situ Moisture/Dry Density;
- Expansion Index; and
- Corrosivity Suite.

All laboratory tests were performed in general conformance with ASTM or State of California Standard Methods. The results of the laboratory tests are presented in Appendix C of this report.

3.0 SITE GEOLOGY AND SEISMICITY

3.1 Regional Geology

3.1.1 Geologic Setting

Dana Point Harbor is located within the northwest trending Peninsular Ranges geomorphic province in southern California. The Peninsular Ranges province is elongate area characterized by parallel fault-bounded mountain ranges and intervening valleys. The province extends southward from the Transverse Ranges at the northern side of the Los Angeles Basin southward into Mexico. The site lies at the southernmost end of The San Joaquin Hills, which are a northwest trending topographically high area that extend southward from Newport Beach to Dana Point.

The harbor is a coastal reentrant or cove protected by the headland at Dana Point. The protected cove owes its existence to the differing resistance to wave erosion of the two bedrock formations exposed along a fault in the steep coastal bluff. Bedrock units include the Capistrano Formation and the San Onofre Breccia. Both the San Onofre Breccia and the Capistrano Formation are exposed in the sea cliffs behind the harbor where, separated by the Dana Cove Fault. However, we only encountered the Capistrano Formation during our subsurface investigation. The weaker Capistrano Formation has been preferentially eroded, creating Dana Cove. More youthful sediments have been deposited in the harbor including colluvium, alluvium, beach deposits, landslide debris, talus, and artificial fill placed during construction of the modern harbor in 1969 and 1970.

3.1.2 Earth Units

3.1.2.1 Surficial Units

The bedrock is mantled by surficial earth units including talus, beach sands, stream deposited alluvium and artificial fill. The CPT soundings and borings drilled for this investigation encountered from 2 to 20 feet of artificial fill over the bedrock. Fill thickness increased toward the ocean with the thickest section of alluvium in Boring B-1. Artificial fill generally consisted of sandy clay, clayey sand, and silty clay. Boring B-3 encountered about 5 feet of alluvium over the bedrock. The alluvium was capped by fill. The alluvium consisted of gravelly sand.

3.1.2.2 Bedrock Units

San Onofre Breccia: The San Onofre Breccia is a Middle Miocene-age (about 11 to 16 million years old) formation of marine origin. It consists of a very coarse, reddish-brown to blue-gray, massive to crudely bedded breccia with interbeds of coarse, pebbly sandstone and siltstone. The matrix is generally an earthy, poorly cemented silt, or a well-cemented angular sand. The San Onofre Breccia is exposed at the western end of Dana Point Harbor along the east-facing sea cliffs where it is in fault contact with the Capistrano Formation. The San Onofre Breccia is a bedrock unit that is resistant to erosion and forms the headland at Dana Point.

Capistrano Formation: The Capistrano Formation is a Late Miocene to Early Pliocene-age (about 3.6 to 11 million years old) formation of marine origin. In the Dana Point area, the Capistrano Formation is widespread with a total thickness of nearly 2,400 feet (Eddington, 1974). This marine (ocean deposited) bedrock formation is divided into a few recognizable subunits: a siltstone facies, a sandstone facies, and sandstone with conglomerate and sedimentary breccia. These three facies of the Capistrano Formation are all exposed in the sea cliffs surrounding the subject site generally dipping into slope (north). The siltstone facies is medium to dark gray and brownish gray to dark greenish gray, fine grained, poorly to moderately consolidated and massive to moderately fissile (Eddington, 1974). The sandstone facies is yellowish brown to pale yellowish brown and medium gray to light gray, fine to medium grained and weakly cemented, and massive to poorly bedded (Eddington, 1974). The sandstone and breccia facies is yellowish brown and coarse grained, weakly cemented to friable, with angular to rounded pebbles and cobbles of multiple origins, massive to poorly bedded, and with interbeds of well-graded sand and silt (Eddington, 1974). The bedrock encountered in the borings are from the siltstone facies of the Capistrano Formation. Capistrano Formation bedrock adjacent to the Dana Cove fault contact is sheared in a zone about 70 to 100 feet wide (Kerwin, 1992).

3.1.3 Geologic Structure

3.1.3.1 Faulting

Two major faults are located in close proximity to the site. A description of these faults are presented below:

Newport-Inglewood Fault Zone: The Newport-Inglewood Fault Zone is a broad zone of left-stepping en echelon faults and folds striking southeastward from near Santa Monica across the Los Angeles basin to Newport Beach. Altogether these various faults constitute a system more than 150 miles long that extends into Baja California, Mexico. Faults having similar trends and projections occur offshore from San Clemente and San Diego (the Rose Canyon and La Nacion Faults). A near-shore portion of the Newport-Inglewood Fault Zone was the source of the destructive 1933 Long Beach earthquake (M_L 6.3) (Hauksson and Gross, 1991). This fault zone is considered a Type B fault (CDMG, Peterson et al., 1996), and the reported recurrence interval for a large event along this fault zone is 1,200 to 1,300 years with an expected slip of 1 meter (Forest et al., 1997).

San Joaquin Hills Blind Thrust Fault: The seismic hazards in Southern California have been further complicated with the recent realization that major earthquakes can occur on large thrust faults that are concealed at depths between 5 to 20 km, referred to as "blind thrusts." The uplift of the San Joaquin Hills is produced by a southwest dipping blind thrust fault that extends at least 14 km from northwestern Huntington Mesa to Dana Point and comes to within 2 km of the ground surface (Grant et al., 1997; Mueller et al., 1998). Work by Grant et al. (1997 and 1999) suggest that uplift of the hills began in the Late Quaternary and continues during the Holocene. Uplift rates have been estimated between 0.25 and 0.5 mm/yr. If the entire length of the fault ruptured, the earthquake generated has been estimated to be M_w 6.8 (Grant et al., 1999).

3.2. Subsurface Soil Conditions

The site is covered by up to 20 feet of fill materials consisting of a mixture of silty sand, clayey sand, sandy clay, and silty clay overlying bedrock. Thickness of fill generally increased from the northwest toward the southeast. Fill materials in proximity to Dana Point Harbor Drive, west of Street of the Golden Lantern, were found to be approximately 2 feet; thickness of fill materials to the south and east of Street of the Golden Lantern were typically 15 to 20 feet thick. The consistency of the fill materials is highly non-uniform across the site. It varies from loose to medium dense and firm to stiff. The fill materials were likely placed during construction of the existing harbor facility.

Below the fills are mostly the bedrock of the Capistrano Formation. The bedrock is comprised of mostly fine-grained cemented sand and silty sand. The Capistrano Formation extends to at least to our maximum depth of exploration of 50 feet at Boring B-3. The consistency of the bedrock is generally very dense. At the maximum exploration depth of 50 feet at Boring B-3, we encountered hard claystone. A thin layer of dense to very dense native alluvium consisting of gravelly sand was encountered at approximately 20 to 22 feet at Boring B-3.

3.3 Soil Expansion Potential

Based on our preliminary field investigation and laboratory testing, the near surface fill material that is present onsite exhibits a low expansion potential. Therefore, no special design provisions will be required for designing concrete members in direct contact with the onsite soil. However, we recommend that further testing be performed during the design phase to confirm the findings.

3.4 Groundwater Conditions

Due to the proximity of the site to the ocean, the groundwater level across the site generally coincides with the sea level. Based on the site topography, the groundwater at the southern end of the site was estimated to be approximately 3 feet below the ground surface. The depth to groundwater is estimated to be approximately 7 feet below existing ground surface near the northern edge of the existing parking lots.

We measured groundwater at all three soil borings (B-1, B-2, and B-3) at depths of 6, 9, and 8 feet, respectively. It is our opinion that if the borings had been allowed to remain open for an extended period of time (it was not practical to do so at the time of exploration) the groundwater would have risen approximately to sea level. Due to the close proximity of the site to the ocean, the groundwater will be subject to tidal fluctuation.

3.5 Seismicity and Ground Motions

Using the computer program EQFAULT (Blake, 2000a), 30 known regional active and potentially active faults were found within approximately 100 km of the site. It is expected the site will be subject to moderate to strong ground shaking from earthquakes generated by these faults. The closest active fault to the site is likely to generate the strongest ground motion. The closest active fault to the site is the San Joaquin Hill Blind Thrust, located at about 2.7 km from the site. The fault is capable of generating a magnitude of M_w 6.8 earthquake. Other faults that may cause slight to considerable damages include both the offshore and Los Angeles Basin segments of the Newport-Inglewood Fault, the Palos Verdes Fault, and the Coronado Bank Fault. A complete list of active and potentially active faults, and the results of the deterministic analysis, are included in Appendix D.

A probabilistic seismic hazard assessment of the major seismic sources was performed to evaluate the likelihood of earthquake events occurring at the site. The ground motions for the 475- and 949-year return period were analyzed using the program FRISKSP (Blake 2000). These two earthquake events are define as the Design Base Earthquake (DBE) and the Maximum Credible Earthquake (MCE), respectively. The peak horizontal ground acceleration (PHGA) for the above two earthquake events was determined for the attenuation relationships based on Abrahamson and Silva (1997), Sadigh et al. (1997), and Campbell (1997 Rev.), and the recommended PHGA was taken as the average of the three relationships. The corresponding probability of exceedance and PHGA are as follows:

Return Period	Probability of Exceedance	Peak Ground Acceleration
475-year	10% in 50 years	0.59g
949-year	10% in 100 years	0.78g

Results of the probabilistic seismic hazard analysis in terms of PHGA vs. return period are included in Appendix D.

3.6 Seismic Hazards

Seismic hazards affecting the site may include fault rupture, soil liquefaction, earthquake-induced settlements, earthquake-induced lateral displacements, landslide, earthquake-induced flooding, seiches, and tsunamis. An evaluation of these effects on the project site is discussed in the following paragraphs.

3.6.1 Fault Rupture

The site is not located within a currently designated Alquist-Priolo Earthquake Zone (Hart and Bryant, 1999). No known active faults are mapped on the site. Based on this consideration, the potential for surface fault rupture at the site is considered to be low.

3.6.2 Liquefaction

Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine, clean sandy soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose and medium dense, near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils exhibit low to negligible liquefaction potential. Bedrock and fine-grained soils (i.e. silt and clay) are generally considered not susceptible to liquefaction.

The proposed project site is located in a State of California-defined Liquefaction Hazard Zone (CDMG, 2001). Engineering analysis suggested that the potential for localized liquefaction of the sandy soils in the fills is high during a major earthquake event. The thickness of these potentially liquefiable soils encountered during our exploration varied from a few inches to up to 5 feet.

3.6.3 Earthquake-Induced Settlements

Earthquake-induced settlements will consist of dynamic settlements (above groundwater) and liquefaction settlements (below groundwater). These settlements occur primarily in loose sandy soils due to reduction in volume during or after an earthquake event. We estimated that localized seismic settlement due

to liquefaction to be up to 2 inches. Dynamic settlements of the sandy soils in the fill was estimated to be on the order of 1 to 2 inches. Earthquake-induced settlements may result in damage to underground utilities and surface improvements such as pavements. The impact on foundation design is discussed in Section 4.5.

3.6.4 Earthquake-Induced Lateral Displacements

Liquefaction of subsurface soils may also cause lateral spreading and flow failure. Lateral spreading generally occurs in areas with relatively small initial ground gradient (less than 6 percent) and is demonstrated predominately by displacement of soil layers above subsurface liquefiable soil layers. Flow failure will involve a block of soil moving along an underlying continuous liquefying soil layers in areas with a steeper ground gradient (more than 6 percent). Flow failure is differed from slope failure in which the soil movement is mainly caused by gravity of the soil mass sliding along a slip plane.

Since the onsite sandy soils are susceptible to liquefaction, lateral spreading or flow failure will be a design concern, especially for areas in the vicinity of the seawall. The lateral extent of the liquefiable soils should be further delineated during the detailed design phase.

3.6.5 Landslide

No significant ground slopes exist within the project site. Therefore, the potential for seismically induced onsite landslides is considered to be nil.

3.6.6 Earthquake-Induced Flooding

This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. There is no major dam or water-retaining structure located near the site. The potential of earthquake-induced flooding that will affect the site is considered to be low.

3.6.7 Seiches

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Because of the partially enclosed configuration of the Dana Point Harbor, the possibility of seiche phenomena occurring onsite could be of concern. Further study of wave runup near the harbor during a major seismic event should be performed during the design phase.

3.6.8 Tsunamis

Tsunamis are waves generated in large bodies of water as a result of change of seafloor topography caused by tectonic displacement.. As a result of the proximity of the site to the ocean, its near-sea level elevation, tsunami hazard should be considered during design. McCulloch (1985) predicted a 100-year tsunami event could result in a runup of about 4 feet at the harbor. When combined with high tide, the wave runup may topple the existing seawall. Further study on its potential impact on the existing or proposed building structures along the seawall should be performed during the design phase.

4.0 RECOMMENDATIONS

4.1 General

Presented herein are our preliminary recommendations for the project plan. It should be noted that these recommendations were based on limited field exploration and our current understanding of the project. A full investigation should be performed during the detailed design phase of this project when more specific site and building information is known.

4.2 Earthwork

4.2.1 Overexcavation and Recompaction

Because of the lack of relative consistency of the existing fills across the site, we recommend overexcavating and recompacting the site subgrade soils within the proposed construction areas to a depth of at least 12 inches below proposed subgrade to reduce the potential for settlements. For areas under slab-on-grade, the overexcavation and recompaction should extend to a depth of 2 feet below the bottom of the slab. The overexcavation should extend a horizontal distance at least equal to the depth of the overexcavation beyond the slab.

If excavation to remove unsuitable soils is required to extend below groundwater, the geotechnical consultant should be consulted to determine the extent of the excavation or other alternative in lieu of excavation.

It should be noted that the potential for post-construction settlements of the existing fill below the overexcavation still exists since complete removal of the existing fill is considered impractical.

4.2.2 Filling and Compacting

Most of the on-site soils are suitable to be used as fill provided it is free of organic materials, debris, and other deleterious materials. An average shrinkage factor of 7 to 10 percent is expected for the on-site soils when removed and replaced as compacted fill. The maximum size of fill materials should not exceed 6 inches. Imported soils, if required, should be non-expansive with an Expansion Index of

50 or less. The geotechnical consultant will request that soil samples from the borrow site be tested to verify of the suitability of the materials before the soil is allowed to be used.

Due to the relatively shallow groundwater encountered over much of the site, special care should be used to avoid the use of saturated soils as backfill. Overly moist or saturated soils should be allowed to dry to within a few percent above optimum before being utilized as fill.

All fill soils should be placed in lifts, moisture-conditioned, and properly compacted.

4.2.3 Temporary Excavation

For temporary excavation above groundwater, the onsite soil may be excavated based on the Occupational Safety and Health Administration (OSHA) requirements for Type B soil, i.e., the excavation may be cut to a 1H:1V (horizontal to vertical) slope.

If excavation of utility trenches or building foundations is required to extend below groundwater, shoring and dewatering will be required. Due to the relatively shallow groundwater encountered over much of the site, a braced sheet pile shoring system is considered more suitable for the project. The shoring should provide adequate penetration below the bottom of excavation to prevent heaving. However, driving sheet piles will probably meet refusal once encountered bedrock. Considering the shallow groundwater and depth to bedrock, excavation of more than 10 feet may be difficult.

For preliminary design purpose, a uniform pressure of $34H_1$ and $16\ H_2$ may be used in designing braced shoring up to 10 feet deep, where H_1 and H_2 is the depth of excavation above and below groundwater, respectively. In addition, hydrostatic pressure should be considered for shoring extends below groundwater. Lateral resistance for sheet piles may be provided by passive pressure below the bottom of the excavation. The allowable passive pressure for sheet piles may be taken as 100 psf on the pile per foot of depth measured below the bottom of the planned excavation.

The shoring design must consider support of the proposed adjacent traffic lanes, parking structures, and/or underground utilities. Also, the effect of shoring deflection on supported structures should be considered.

It is the responsibility of the contractor to design and install the dewatering system. Pump tests should be performed to estimate the inflow quantity anticipated during excavation. Dewatering may cause ground subsidence. Impact of dewatering on adjacent buildings/structures can be evaluated when pump test data is available. The contractor should anticipate continuous pumping of groundwater during the excavation. Disposal of groundwater should comply with all environmental regulations.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

4.4 Seismic Design Parameters

Based upon the California Building Code (CBC, 1998), the site is within Seismic Zone 4 with a Z factor of 0.4, as is the case for most of Southern California. A soil profile type of S_D (i.e., stiff soil profile) should be used, as shown in Table 16-J of the 1998 CBC. Per the 1998 CBC, seismic design can be based on a Seismic Source Type of "B" (i.e., Newport-Inglewood Fault located approximately 4.0 km from the site) with a Near-Source Factors N_a and N_v of 1.1 and 1.33, respectively.

4.5 Foundation Design

Because of the potential for settlement of the existing fill soils, deep foundations are recommended for the building structures. At this preliminary stage of the project, we have evaluated augered cast-in-place (ACIP) and driven piles. Drilled piers are not considered efficient for this project due to presence of high groundwater.

For minor structures that are separated from the building, conventional shallow foundation may be considered. Preliminary recommendations for foundation design are presented in the following paragraphs.

4.5.1 Augered Cast-in-Place Foundations

ACIP piles are installed by rotating a continuous hollow-stem flight auger into the ground until a specified termination criteria is achieved. A sand-cement grout is then pumped through the auger stem under pressure as the auger is slowly withdrawn to fill the drilled hole. Steel reinforcement will be inserted into the drilled hole before the grout is set. ACIP foundation is considered because it offers:

- Low noise and vibration during installation
- Readily adaptable to high groundwater conditions.

Assuming a 10-foot embedment in bedrock, the allowable downward capacity for an 18-inch-diameter ACIP was estimated at 40 tons.

A factor of safety of 2.0 has been incorporated in the pile capacity calculations. Downdrag resulted from both static and earthquake-induced settlements of the existing fill has been considered in the calculation. It should be noted that the pile capacities shown are based on soil strengths alone. The actual capacities may be limited to lesser values by the strength of pile materials and connections. For seismic or other short-term loading, the allowable pile capacities may be increased by one-third. All piles should be spaced at a minimum of three times the pile width on centers.

Lateral load analyses of piles were conducted for a pile-top deflection of ½ inch for fixed-head and a free-head conditions. The estimated lateral capacity for an 18-inch ACIP pile is 30 kips for fixed-head and 12 kips for free-head conditions.

Due to the potential for liquefaction, the pile should be designed to resist buckling due to column action over a potentially unsupported length of 5 feet.

Additional lateral load resistance may be provided by passive soil pressure acting against the vertical portion of foundation elements such as grade beams and pile cap. Recommendations for passive resistance are discussed in Section 4.6.2.

It should be noted that the very dense nature of the underlying bedrock might result in difficulty with advancing the auger flight during installation for larger diameter piles.

4.5.2 Driven-Pile Foundations

As an alternative to ACIP piles, a driven pile foundation may be utilized. For a single 12-inch-square prestressed concrete pile, with a minimum of 5 feet embedment into bedrock, the estimated allowable downward capacity may be taken as 55 tons.

A factor of safety of 2.0 has been incorporated in the pile capacity calculations. Downdrag resulted from both static and earthquake-induced settlements of the existing fill has been considered in the calculations. It should be noted that the pile capacities shown are based on soil strengths alone. The actual capacities may be limited to lesser values by the strength of pile materials and connections. For seismic or other short-term loading, the allowable pile capacities may be increased by one-third. All piles should be spaced at a minimum of three times the pile width on centers.

Lateral load analyses of piles were conducted for a pile-top deflection of ½ inch for fixed-head and a free-head conditions. The estimated lateral capacity for a 12-inch square pile is 19 kips for fixed-head and 8 kips for free-head conditions.

Due to potential for liquefaction, the piles should be designed to resist buckling due to column action over a potentially unsupported length of 5 feet.

Additional lateral load resistance may be provided by passive soil pressure acting against the vertical portion of foundation elements such as grade beams and pile cap. Recommendations for passive resistance are discussed in Section 4.6.2.

An indicator pile-driving program should be performed to verify the pile capacity and driveability included in the design package.

It should be noted that the very dense nature of the underlying sandstone could result in difficult driving. Also, the noise and vibration associated with pile driving should be addressed before the commencement of a pile-driving program.

4.5.3 Conventional Shallow Foundation

Minor structures separated from the building structures may by supported on conventional shallow foundations established on at least 3 feet of compacted fill. Based on the existing site conditions, planing for these minor structures will be limited to the areas where groundwater is located below the bottom of the proposed compacted fill.

The foundation should be a minimum of 24 inches wide and embedded at least 18 inches below lowest adjacent grade. For designing the foundation, an allowable maximum bearing pressure of 2,000 psf may be used. The recommended bearing pressure are for total dead load plus frequently applied live loads. These values may be increased by one-third for wind and seismic loads.

4.5.4 Foundation Settlements

Settlements of piles generally result from the settlement of the supporting soils and elastic compression of piles. Due to the very dense nature of the bedrock, foundation settlement due to the settlement of the supporting soils will likely be negligible. The elastic compression of piles was estimated for a 18-inch ACIP and 12-inch prestressed concrete pile to be less than ½ inch. The elastic compression of the piles is expected to occur during construction as loads are applied.

Settlement analysis can be further evaluated if the pile cap configuration becomes available.

The total settlement of conventional shallow foundation was estimated to be on the order of 1.0 inches. Differential settlements may be taken as half of the total settlements.

4.6 Retaining Structure Design

4.6.1 Lateral Pressure

The design of earth retaining structures, including cantilever retaining wall (i.e., unrestrained) and other underground utility appurtenance structures should consider the lateral earth pressures exerted by surrounding soils. Values presented herein do

not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Cantilevered retaining walls, that are allowed to deflect, retaining a level backfill for a horizontal distance equal to or greater than the height of the wall may be designed for an active equivalent fluid pressure of 38 pcf. For a 2:1 backfill, the walls may be designed using an active equivalent fluid pressure of 49 pcf. Walls that are fixed against rotation (no movement allowed) should be designed using at-rest equivalent fluid pressures of 54 pcf for level backfill and 80 pcf for a 2H:1V backslope. For seismic loading of retaining walls, an inverted triangular pressure distribution of 60 pcf (level backfill) and 90 pcf (2H:1V backfill) may be used in addition to the static earth pressures.

In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads, such as those from truck traffic, adjacent structures, or vehicles, should be added, if the loads fall within a 1H:1V projection from the surcharging structure on the stem of the wall. For an aerial surcharge behind a restrained wall, we recommend utilizing a horizontal load equal to 50 percent of the vertical surcharge load. For a cantilever wall, the lateral surcharge may be taken as 34 percent of the vertical surcharge. The increased lateral load should be uniformly distributed behind the wall.

Earth pressures presented herein assume that sufficient drainage will be provided behind the walls to reduce the build-up of hydrostatic pressures from surface water infiltration. Backfill against subterranean walls and retaining walls should consist of free-draining granular soil with sand equivalent of 30 or greater.

Retaining walls 6 feet or lower in height may be supported on conventional shallow foundations. For retaining wall higher than 6 feet, we recommended that it should be supported on foundation extends below the fill. As an alternative, Mechanically Stabilized Earth (MSE) walls that are more tolerant of deformations may be used. The MSE walls may be supported on conventional shallow foundation. The geotechnical consultant can evaluate the settlement along the wall as the final wall configuration becomes available.

4.6.2 Lateral Load Resistance

The soil resistance available to withstand lateral loads is a function of the frictional resistance along the base of the foundation *or* the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the retaining wall foundations and the subgrade soil may be computed using a coefficient of friction of 0.36. The passive resistance may be computed using an equivalent fluid pressure of 345 pcf, assuming there is constant contact between the footing and undisturbed soil.

The above coefficient of friction and passive resistance are ultimate values without appreciable factors of safety.

4.6.3 Existing Seawall

We understand that the existing seawall is a gravity-type wall retaining. Based on our borings in the vicinity of the seawall and our understanding of the as-built condition, it appears that the seawall is supported on the existing fill. Due to the non-uniform consistency of the existing fill, the seawall might have experienced some differential settlements over the years. With respect to the impact of the seawall under seismic loading, we are concerned that the seawall may not be adequate to withstand the dynamic pressure. In addition, if lateral spreading were to occur in the fill soils supporting the seawall, the seawall may become unstable, adversely impacting its ability to retain backfill behind the wall.

We have also evaluated the proposed improvement alternatives for the seawall in the draft Dana Point Harbor Master Plan. Since the potential for liquefaction is considered high at the site, continue maintenance of the existing system may not be a long-term solution. From a geotechnical standpoint, both the retrofit and replacement alternatives are feasible. Selection of the improvement alternative will depend on the lateral extend of the liquefiable soils in the fill below the seawall.

4.7 Preliminary Pavement Design

Laboratory testing performed on a representative near-surface soil sample indicates an R-value of 46 for the sample tested. Based on the design procedures outlined in the current

Caltrans Highway Design Manual, and using a design R-value of 78 for aggregate base course, preliminary flexible pavement sections may consist of the following for the Traffic Indices indicated. Final pavement design should be based on laboratory testing performed near the completion of grading and the Traffic Index determined by the project civil engineer.

Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	3	4
5.0	3	4
6.0	3	6

Concrete aprons and ramps should have a minimum thickness of 6 inches placed on a minimum of 6 inches of aggregate base. From a geotechnical standpoint, no reinforcement will be required. However, as with any concrete slab, it is important to provide for concrete cracking by constructing weakened plane and/or construction joints at frequent intervals. The minimum joint spacing should be determined by the structural engineer, who should also determine if unreinforced concrete slabs are acceptable from a structural standpoint.

Prior to placement of aggregate base, the subgrade soils should be processed as specified in Section 4.2.3. All aggregate base for pavement section should be placed in thin lifts, moisture conditioned to 2 points above the optimum and compacted to a minimum of 95 percent relative compaction.

All pavement construction should be performed in accordance with the *Standard Specifications for Public Works Construction.* Field inspection and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Since it is not practical to remove all existing fill on the site, some post-construction settlement can be expected in the paved areas. Reinforcing the pavement subgrade to further reduce the potential for post-construction settlement may be considered.

4.8 Slab-On-Grade

Concrete slabs subjected to special loads should be designed by the structural engineer in accordance with the Uniform Building Code (UBC). For this project where conventional light floor loading conditions exists, a minimum slab thickness of 5 inches is recommended from a geotechnical standpoint. A structural engineer should design the slab reinforcement.

It should be noted that potential for settlements of the existing fill when subject to additional loads from the slab is likely to occur since completely removal of the existing fill is not practical.

A moisture barrier consisting of 6-mil Visqueen (or equivalent) placed below slabs where moisture-sensitive floor coverings or equipment is planned. The Visqueen should be placed on top of a minimum of 2 inches of sand and covered with a minimum of 2 inches of sand. Moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slab. We recommend that the floor covering contractors test the moisture vapor flux rate prior to attempting application of moisture-sensitive flooring. "Breathable" floor coverings or special slab sealants should be considered if the vapor flux rates are high. Floor covering manufacturers should be consulted for specific recommendations.

4.9 Soil Corrosivity

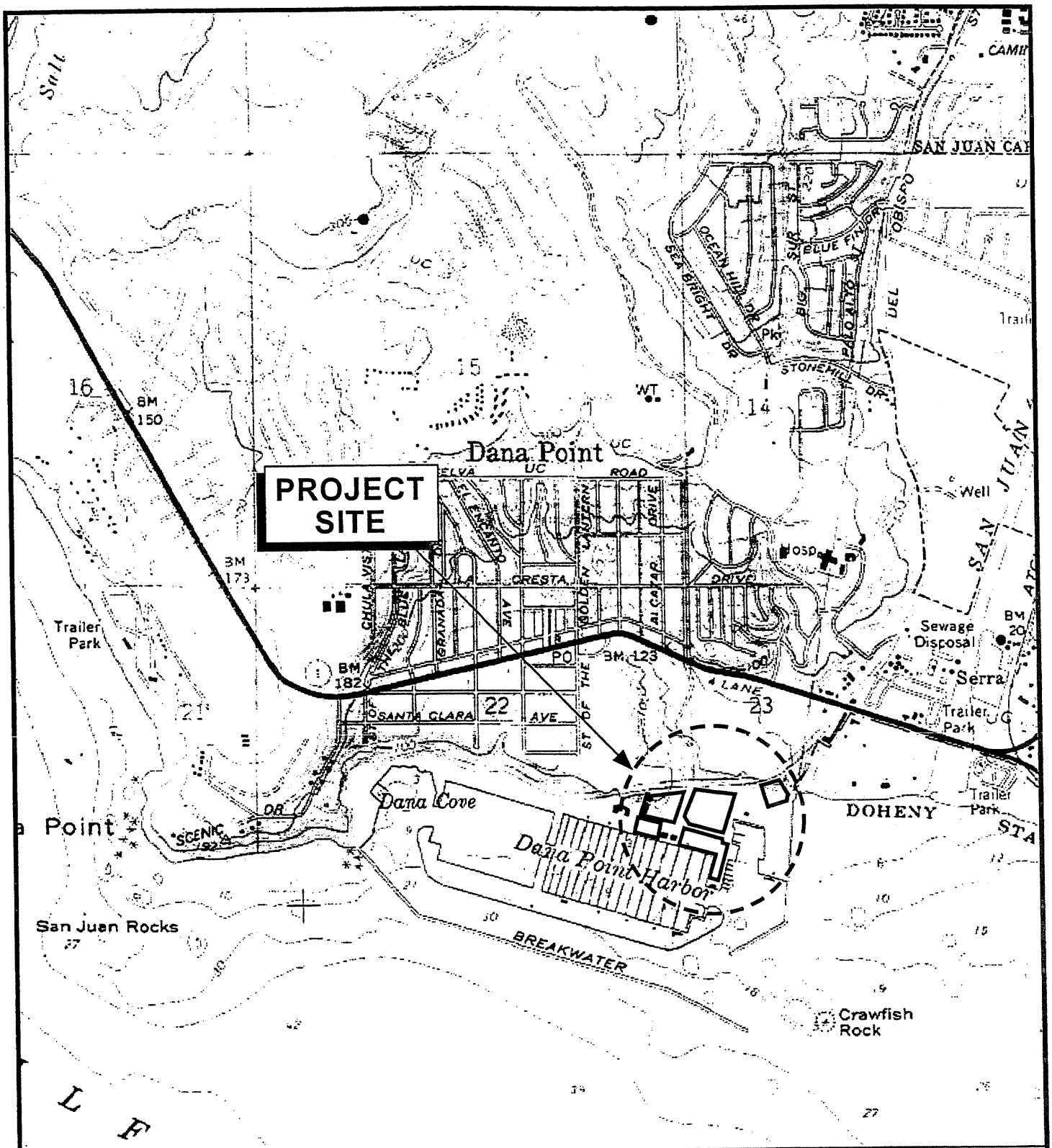
Water-soluble sulfates in soil can react adversely with concrete. However, concrete structures in contact with soil containing sulfate concentrations of less than 0.10 percent are considered to have negligible sulfate exposure (1997 UBC).

A representative sample of the subsurface soil was tested for water-soluble sulfates. Test results indicate a maximum soluble sulfate content of approximately 0.76 percent by weight. Based on this result, the soil sample is considered corrosive to concrete. A special cement type will be required for concrete members in direct contact with the onsite soils.

Soil corrosivity to ferrous metal can be estimated by the soil's chloride content, pH level, and electrical resistivity. In general, soil having a minimum resistivity less than 1,000

ohm-cm is considered severely corrosive. Soil with a chloride content greater than 500 parts per million (ppm) is considered corrosive to ferrous metal.

As a screening for potentially corrosive soil, a representative soil sample was tested to determine minimum resistivity, chloride content, and pH level. The test results indicate a minimum soil resistivity of approximately 363 ohm-cm, a pH level of approximately 7.8, and a chloride content of approximately 733 ppm. Based on these results, the soil sample is considered severely corrosive to ferrous metal. Typical mitigation techniques will include protective coating and thicker sections for corrosion allowance for metals.



SITE LOCATION MAP

Proposed Dana Point Harbor
Revitalization Project
Dana Point, California

Project No.	600024-001
Scale	As shown
Engr./Geol.	VPI/EAS
Drafted By	JMP
Date	December 2002

Leighton Consulting, Inc.

Figure No. 1

LEGEND

B-2 — Approximate Location of Soil Boring

CPT — Approximate Location of CPT Sounding

— — — — Approximate Boundary of Proposed Development

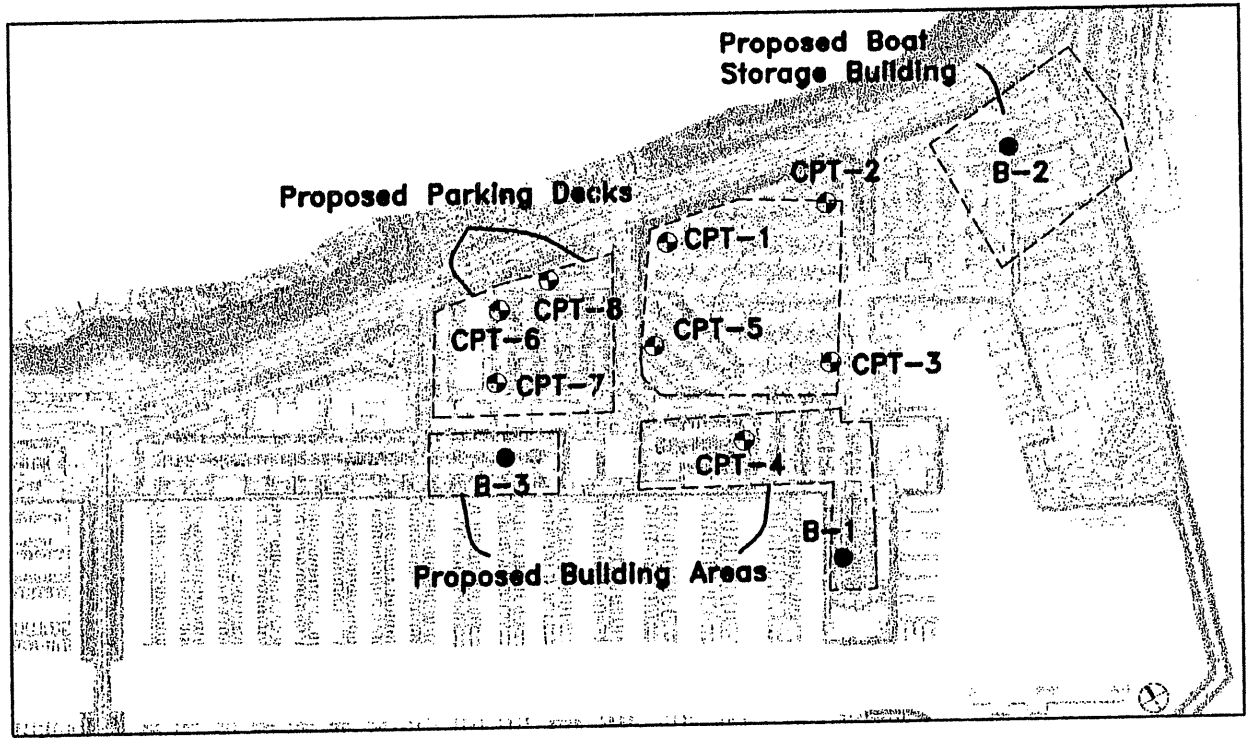


Figure No. 2

BORING LOCATION MAP
PROPOSED DANA POINT HARBOR REVITALIZATION
CITY OF DANA POINT, CALIFORINA

Leighton Consulting, Inc
A LEIGHTON GROUP COMPANY

Proj: 600024-001	Scale: As Shown
Engineer/Geologist VPI/REL	Date: 12/3/02
Path: p:drafting/600024/001/of_12_02/figure_2	

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APPENDIX B

Boring Logs and CPT Soundings

GEOTECHNICAL BORING LOG B-1

Date 11-6-02 | Sheet 1 of 2

Project WRT/Dana Point | Project No. 600024-001

Drilling Co. Martini Drilling Company | Type of Rig CME-75 HSA

Hole Diameter 6" | Drive Weight 140 lbs. | Drop 30"

Elevation Top of Hole ' | Location See Boring Location Map.

Logged By DAG

Sampled By DAG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								@ 0' ASPHALT, about 3"	
									@ 3" FILL CLAYEY SAND, moist, olive-brown, fine- to medium-grained, rounded pebbles to 1/2"	CR,EI
				B-1					@ 3' encoutering occasional rocks to 3"	
	5			R-1	14 13 12			SC CL	@ 5' upper 3": CLAYEY SAND, as above, dark gray, rounded gravels to 1" lower 4": SANDY CLAY, stiff, moist, olive-brown, high plasticity, medium-grained sands, sub-grounded gravels to 1/2" note: partial recovery from R-1	
	10			S-1	2 4 5		17.3	CL-ML	@ 10' SILTY CLAY, stiff, moist (sat?), medium gray, mottled, contains some fine sands, staining,rounded pebbles to 1/2", fractured rock in middle of sampler, fine sands at top 2" of sampler	
	15			R-2	2 3 6			SC	@ 15' CLAYEY SAND, loose, wet, olive-brown, medium-grained, contains silt, minor pebbles, minor amounts of sandy clay in sampler	
	20			S-2	N/A			SM	@ 20' CAPISTRANO FORMATION SILTY SANDSTONE, saturated, olive-brown, medium- to coarse-grained, mottled, stained, some fine sands present, minor amounts of clay	
				S-3	15 40 50/4"			SP	@ 22.5' SANDSTONE, very dense, saturated, brown, medium- to coarse-grained, some fines present, rounded pebbles to 1/2", heavily mottled and stained	
	25			S-4	10 42 50			SP	@ 25' SANDSTONE, very dense, saturated, olive-brown, medium-grained, contains low-plasticity fines, mottling, staining	
	30									

SAMPLE TYPES:

S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

GEOTECHNICAL BORING LOG B-1

Date 11-6-02 | Sheet 2 of 2

Project WRT/Dana Point | Project No. 600024-001

Drilling Co. Martini Drilling Company | Type of Rig CME-75 HSA

Hole Diameter 6" | Drive Weight 140 lbs. | Drop 30"

Elevation Top of Hole ' | Location See Boring Location Map.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION (Logged By DAG; Sampled By DAG)	Type of Tests
	30			S-5	8 16/6"			SP	@ 30' SANDSTONE, as above, minor rounded pebbles, laminated "staining" pattern appears as horizontal bands Note: S-5 driven 12" only	
	35			S-6	45 50/5"			SM	@ 35' SILTY SANDSTONE, very dense, wet, brown, medium- to coarse-grained, some fine sands present	
	40			S-7	32 50/3"			SM	@ 40' SILTY SANDSTONE, as above	
	45								Total Depth of hole : 41 feet below ground surface (bgs). Groundwater was encountered within the hole during drilling at an approximate depth of 6 feet bgs. Backfilled with soil cuttings and surface finished with cold asphalt patch. Note: Sea Level measured at approximately 2.5 feet bgs	
	50									
	55									
	60									

SAMPLE TYPES:
- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- AL ATTERBERG LIMITS
- EI EXPANSION INDEX
- RV R-VALUE

LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-2

Date 11-6-02 | Sheet 1 of 1

Project WRT/Dana Point | Project No. 600024-001

Drilling Co. Martini Drilling Company. | Type of Rig CME-75 HSA

Hole Diameter 6" | Drive Weight 140 lbs. | Drop 30"

Elevation Top of Hole ' | Location See Boring Location Map.

Logged By DAG

Sampled By DAG

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S							@ 0' ASPHALT, approximately 4" asphalt, 6" base	
									@ 10" FILL SILTY SAND, moist, brown, fine-grained, contains sub-rounded gravels to 3", some clay present	RV,EI
				B-1					@ 1.5' becomes olive-brown, gravelly	
	5			R-1	7 7 8		12.4	CL	@ 5' SANDY CLAY, stiff, moist, brown, highly plastic, contains fine-grained sands, about 2" black clay in tip of sampler, sample highly disturbed	
				B-2						
	10			S-1	1 2 3			CL	@ 10' SANDY CLAY, firm, saturated, black, highly plastic, contains fine-grained sands, minor rootlets	
	15			S-2	13 26 34			SP	@ 15' CAPISTRANO FORMATION SANDSTONE, very dense, saturated, light brown, fine-grained, mottled, red staining, contains low-plasticity silt	
	20			R-2	10 56/6"			SP	@ 20' as above	DS
	25								Total Depth of hole : 21 feet below ground surface (bgs). Groundwater was encountered within the hole during drilling at an approximate depth of 9 feet bgs. Water at 9 feet Backfilled with soil cuttings and surface finished with cold asphalt patch.	
	30									

SAMPLE TYPES:

S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

GEOTECHNICAL BORING LOG B-3

Date 11-6-02 | Sheet 1 of 2

Project WRT/Dana Point | Project No. 600024-001

Drilling Co. Martini Drilling Company. | Type of Rig CME-75 HSA

Hole Diameter 6" | Drive Weight 140 lbs. | Drop 30"

Elevation Top of Hole ' | Location See Boring Location Map.

Logged By DAG

Sampled By DAG

Elevation Feet	Depth Feet	Graphic Log (N–S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0								ASPHALT, about 3"	
				B-1					@ 3" FILL CLAYEY SAND, moist, dark brown, fine- to medium-grained, sub-rounded gravels to 1" gravelly, high-plasticity clay, contains some rounded rocks to 3"	RV
									@ 2.5' SANDY CLAY, moist, dark brown, highly plastic, contains fine-grained sands, some gravel to 1/2",less gravel than above	
	5			R-1	10 19 18	112.5	12.6	SC	@ 5' CLAYEY SAND, medium dense, moist, fine-grained, high-plasticity clay, rounded pebbles to 1/4", sample contained one (1) rounded 2" rock, sampler was wet on outside	SA,MD
				B-2						
	10			S-1	2 2 6			SM	@ 10' SILTY SAND, loose, saturated, olive-brown, fine-grained, contains minor high-plasticity clay, 2" black clay at top of sampler (may be sluff)	
	15			S-2	16 25 12			SP	@ 15' ALLUVIUM GRAVELLY SAND, dense, wet, medium gray, medium- to coarse-grained, gravels to 1.5", contains low-plasticity silt	
									@ 16.5' to 18' driller encountered rough drilling	
	20			S-3	48 50/3"			SP	@ 20' CAPISTRANO FORMATION SANDSTONE, very dense, saturated, tan, medium-grained, minor silt, minor pebbles, difficult to remove from sampler	
	25			R-2	25 50/6"	117.3	13.3	SP	@ 27.5' SANDSTONE, very dense, wet, medium gray, medium- to coarse-grained, contains some silt	
	30									

SAMPLE TYPES:

S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

GEOTECHNICAL BORING LOG B-3

Date 11-6-02 | Sheet 2 of 2

Project WRT/Dana Point | Project No. 600024-001

Drilling Co. Martini Drilling Company. | Type of Rig CME-75 HSA

Hole Diameter 6" | Drive Weight 140 lbs. | Drop 30"

Elevation Top of Hole ' | Location See Boring Location Map.

Logged By DAG

Sampled By DAG

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30			S-4	N/A			SP	@ 30' SANDSTONE,wet, brown, fine-grained, some fines present, samples may consist of "heaved" material only	
	35			S-5	34 50/6"			SP	@ 35' SANDSTONE, very dense, wet, olive-brown, medium-grained, contains some silt, may have minor mottling, yellow staining appears as horizontal rings	
	40			S-6	50/6"			SP	@ 40' SANDSTONE, very dense, wet, brown, medium-grained, orange and red staining, "rust" particles, rocks to 1", some coarse sand, 1/2" muscovite fragment	
	45			S-7A S-7	6 25 25/3"			SP	@ 45' SANDSTONE, very dense, saturated, blue-gray, orange and yellow mottling, 3" siltstone fragment in tip of sampler (black clay or claystone)	
	50			R-3A R-3	28 50/3"			CL	@ 50' CLAYSTONE, hard, black, fissile, some mottling, tiny shell fragments, similar to material found in tip of S-7, upper 3" of sample is Sand as above	
	55								Total Depth 51 feet below ground surface (bgs). Groundwater was encountered within the hole during drilling at an approximate depth of 9 feet (bgs). Backfilled with soil cuttings and surface finished with cold asphalt patch.	
	60									

SAMPLE TYPES:

S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AL ATTERBERG LIMITS
EI EXPANSION INDEX
RV R-VALUE

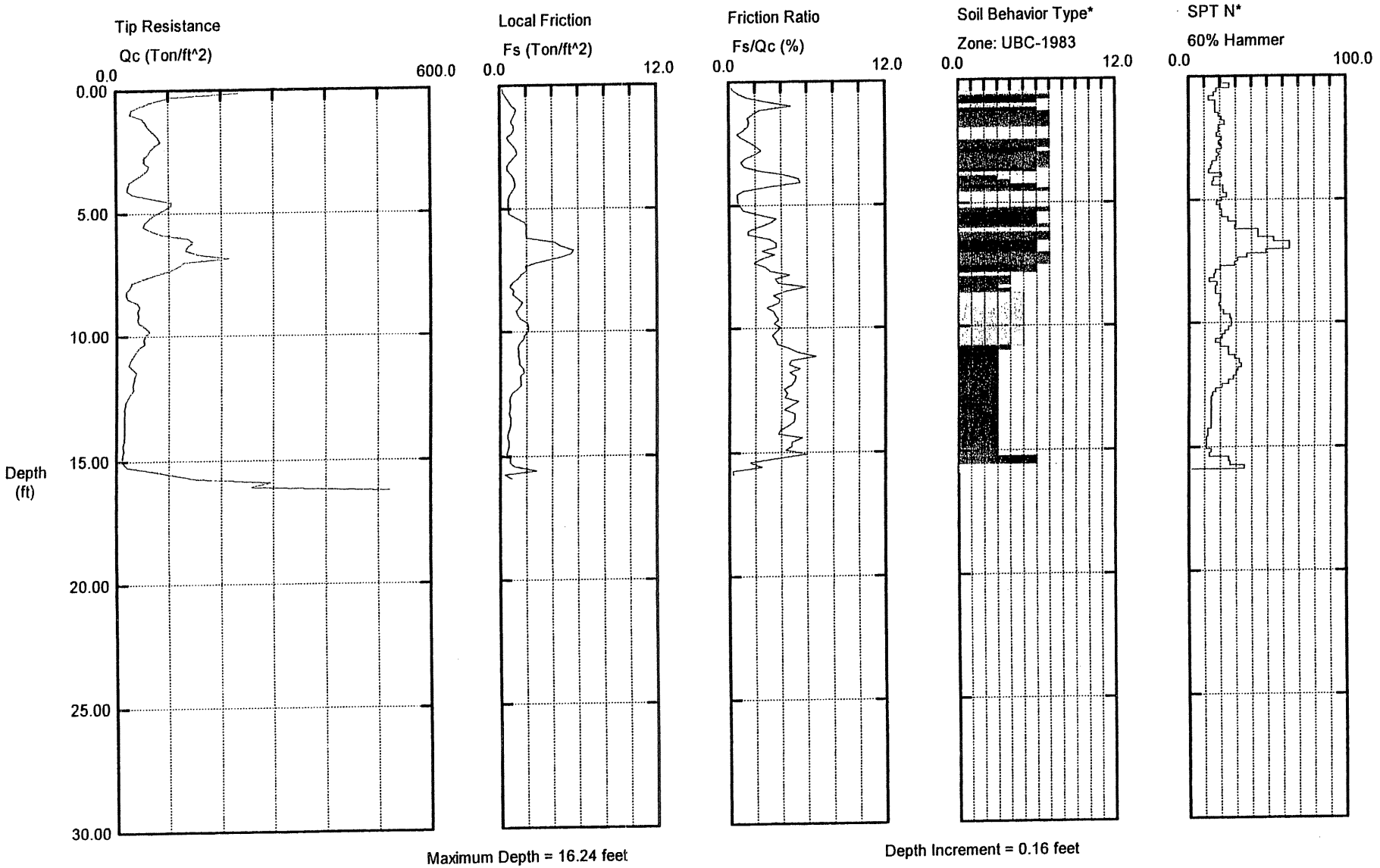
LEIGHTON AND ASSOCIATES, INC.

il behavior type and SPT b[illegible]on data from UBC-1983

Leighton Consu[illegible]ng, Inc.

Operator: Bobby-Victor
Sounding: CPT-01
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 08:31
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 16.24 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-500

CPT INTERPRETATIONS

SOUNDING : CPT-01
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 08:31

PROJECT No.: 600027-001
CONE/RIG : 468/BH-VO/R#3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	69.68	.69	SAND to SILTY SAND	17	28	66		
.300	.98	25.85	4.76	CLAY	26	41		1.5	
.450	1.48	58.64	1.41	SILTY SAND to SANDY SILT	20	31	61		48.5
.600	1.97	80.67	1.07	SAND to SILTY SAND	20	32	70		48.5
.750	2.46	64.03	1.59	SILTY SAND to SANDY SILT	21	34	64		46.5
.900	2.95	53.11	1.94	SILTY SAND to SANDY SILT	18	28	58		45.5
1.050	3.44	48.65	1.16	SILTY SAND to SANDY SILT	16	26	56		44.5
1.200	3.94	21.10	5.31	CLAY	21	34		1.2	
1.350	4.43	68.77	1.08	SILTY SAND to SANDY SILT	23	37	66		44.5
1.500	4.92	87.40	.66	SAND to SILTY SAND	22	35	72		45.5
1.650	5.41	54.62	2.33	SANDY SILT to CLAYEY SILT	22	35		3.6	
1.800	5.91	85.98	2.29	SILTY SAND to SANDY SILT	29	46	72		44.5
1.950	6.40	135.24	3.04	SANDY SILT to CLAYEY SILT	54	87		7.9	
2.100	6.89	214.64	2.50	SILTY SAND to SANDY SILT	72	100	98		47.5
2.250	7.38	100.72	1.88	SILTY SAND to SANDY SILT	34	51	77		44.5
2.400	7.87	29.06	4.59	CLAY to SILTY CLAY	19	29		1.7	
2.550	8.37	17.19	5.84	CLAY	17	25		1.1	
2.700	8.86	43.89	3.83	CLAYEY SILT to SILTY CLAY	22	32		2.6	
2.850	9.35	39.24	3.35	CLAYEY SILT to SILTY CLAY	20	28		2.6	
3.000	9.84	62.35	3.39	CLAYEY SILT to SILTY CLAY	31	44		3.6	
3.150	10.33	53.71	3.26	CLAYEY SILT to SILTY CLAY	27	37		3.1	
3.300	10.83	30.72	4.55	CLAY to SILTY CLAY	20	28		1.8	
3.450	11.32	30.02	4.74	CLAY	30	41		1.7	
3.600	11.81	32.29	4.59	CLAY to SILTY CLAY	22	29		1.9	
3.750	12.30	26.45	4.70	CLAY	26	35		1.5	
3.900	12.80	15.44	4.21	CLAY	15	20		1.0	
4.050	13.29	15.21	4.18	CLAY	15	20		1.0	
4.200	13.78	14.77	4.94	CLAY	15	19		.9	
4.350	14.27	13.40	3.73	CLAY to SILTY CLAY	9	11		.8	
4.500	14.76	11.45	4.77	CLAY	11	14		.7	
4.650	15.26	19.78	3.62	CLAY to SILTY CLAY	13	16		1.3	
4.800	15.75	150.77	.20	SAND	30	37	84		44.0

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

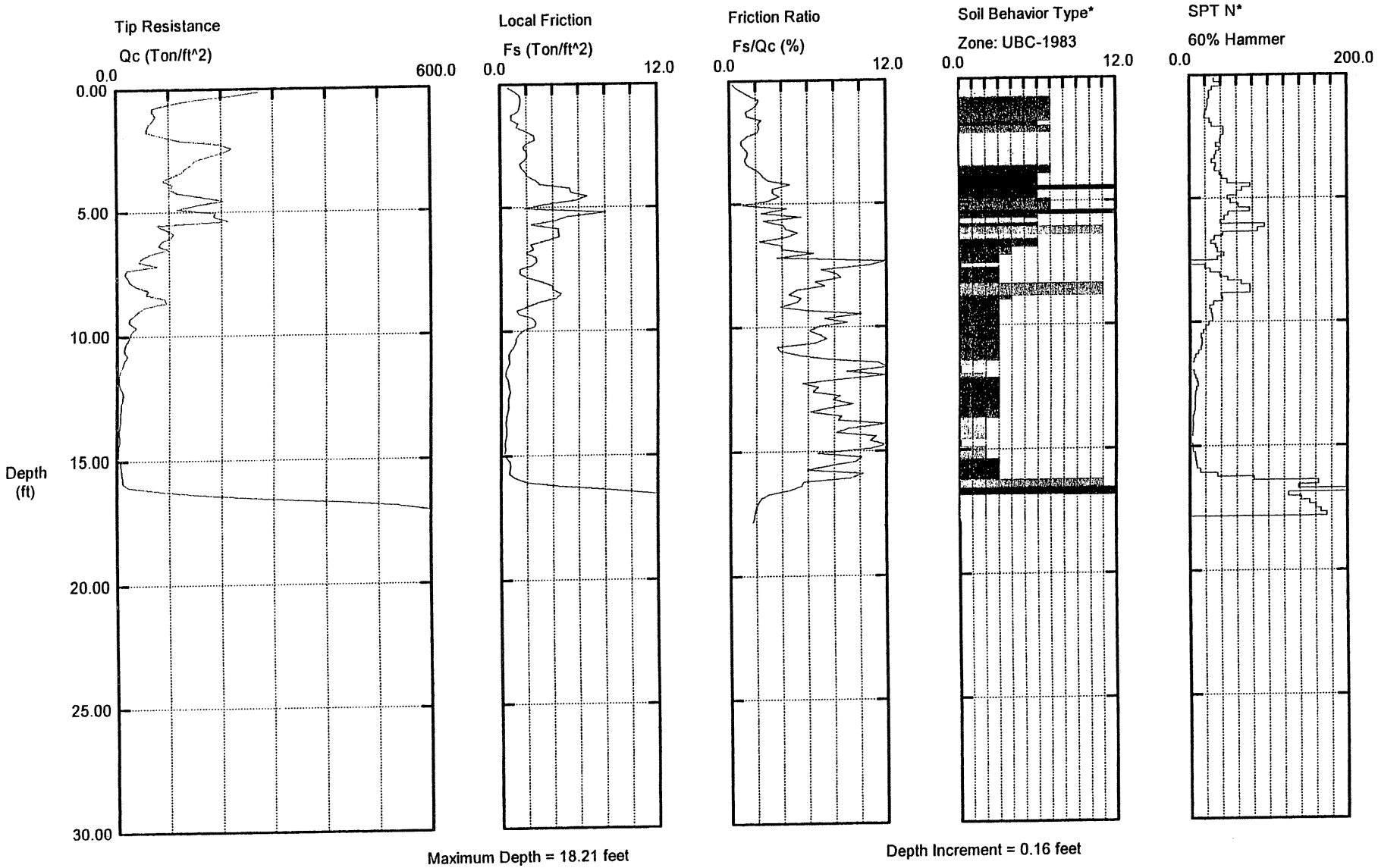
Interpretations based on: Robertson and Campanella, 1989.

l behavior type and SPT b n data from UBC-1983

Leighton Consulting, Inc.

Operator: Bobby-Victor
Sounding: CPT-02
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 09:05
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 18.21 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-501

CPT INTERPRETATIONS

SOUNDING : CPT-02
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 09:05

PROJECT No.: 600027-001
CONE/RIG : 468/BH-VO/R#3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	140.58	1.06	SAND to SILTY SAND	35	56	86		
.300	.98	68.92	2.11	SILTY SAND to SANDY SILT	23	37	66		
.450	1.48	62.97	1.34	SILTY SAND to SANDY SILT	21	34	63		48.5
.600	1.97	88.17	2.28	SILTY SAND to SANDY SILT	29	47	73		49.0
.750	2.46	220.35	.87	SAND	44	70	99		
.900	2.95	149.35	1.38	SAND to SILTY SAND	37	60	88		49.5
1.050	3.44	122.24	1.32	SAND to SILTY SAND	31	49	82		48.0
1.200	3.94	106.35	2.53	SILTY SAND to SANDY SILT	35	57	78		47.0
1.350	4.43	163.84	3.28	SANDY SILT to CLAYEY SILT	66	100		9.6	
1.500	4.92	114.89	2.83	SANDY SILT to CLAYEY SILT	46	73		6.7	
1.650	5.41	213.72	2.38	SILTY SAND to SANDY SILT	71	100	98		48.5
1.800	5.91	108.58	4.05	CLAYEY SILT to SILTY CLAY	54	87		6.4	
1.950	6.40	78.63	4.28	CLAYEY SILT to SILTY CLAY	39	63		4.6	
2.100	6.89	47.44	3.99	CLAYEY SILT to SILTY CLAY	24	37		2.8	
2.250	7.38	18.48	12.85	CLAY	18	28		1.2	
2.400	7.87	24.58	8.15	CLAY	25	37		1.6	
2.550	8.37	54.79	7.25	CLAY	55	81		3.2	
2.700	8.86	54.37	5.41	CLAY	54	79		3.2	
2.850	9.35	22.84	5.80	CLAY	23	33		1.3	
3.000	9.84	28.30	8.99	CLAY	28	40		1.8	
3.150	10.33	16.19	6.89	CLAY	16	23		1.0	
3.300	10.83	19.40	3.56	CLAY to SILTY CLAY	13	18		1.3	
3.450	11.32	8.24	7.44	CLAY	8	11		.5	
3.600	11.81	3.63	8.90	ORGANIC MATERIAL	4	5		.3	
3.750	12.30	10.96	5.51	CLAY	11	14		.7	
3.900	12.80	6.31	8.40	CLAY	6	8		.4	
4.050	13.29	6.40	7.64	CLAY	6	8		.4	
4.200	13.78	3.72	8.25	ORGANIC MATERIAL	4	5		.3	
4.350	14.27	4.38	8.13	ORGANIC MATERIAL	4	6		.4	
4.500	14.76	2.12	15.80	ORGANIC MATERIAL	2	3		.1	
4.650	15.26	6.12	10.07	CLAY	6	8		.3	
4.800	15.75	10.52	5.91	CLAY	11	13		.6	
4.950	16.24	78.54	5.59	*VERY STIFF FINE GRAINED	79	95			
5.100	16.73	431.48	2.94	*SAND to CLAYEY SAND	100	100			
5.250	17.22	635.82	1.99	SAND to SILTY SAND	100	100	100		
5.400	17.72	717.88	1.76	SAND to SILTY SAND	100	100	100		

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

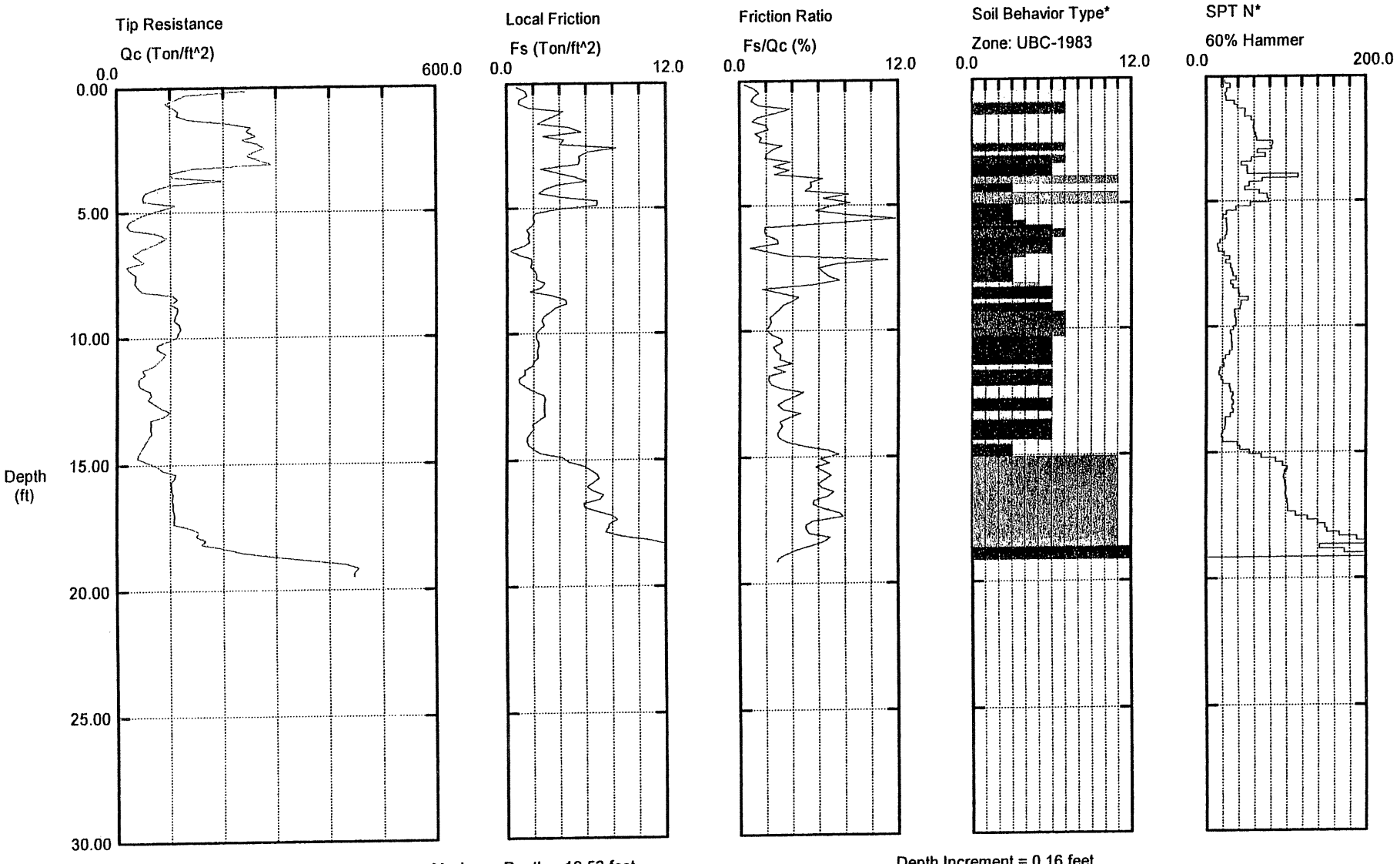
Interpretations based on: Robertson and Campanella, 1989.

il behavior type and SPT b n data from UBC-1983

Leighton Consulting, Inc.

Operator: Bobby-Victor
Sounding: CPT-03
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 09:35
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 19.52 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-502

CPT INTERPRETATIONS

SOUNDING : CPT-03 PROJECT No.: 600027-001
PROJECT : WRT/Dana Point CONE/RIG : 468/BH-VO/R#3
DATE/TIME: 11-15-02 09:35

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	109.92	1.43	SAND to SILTY SAND	27	44	79		
.300	.98	115.47	1.42	SAND to SILTY SAND	29	46	80		
.450	1.48	212.00	1.35	SAND to SILTY SAND	53	85	98		
.600	1.97	260.31	2.16	SILTY SAND to SANDY SILT	87	100	100		
.750	2.46	276.73	1.44	SAND to SILTY SAND	69	100	100		
.900	2.95	267.41	2.02	SAND to SILTY SAND	67	100	100		
1.050	3.44	99.96	2.52	SILTY SAND to SANDY SILT	33	53	76		47.0
1.200	3.94	95.79	6.27	*VERY STIFF FINE GRAINED	96	100			
1.350	4.43	47.99	4.94	CLAY to SILTY CLAY	32	51		2.8	
1.500	4.92	81.26	8.34	*VERY STIFF FINE GRAINED	81	100			
1.650	5.41	22.65	8.32	CLAY	23	36		1.5	
1.800	5.91	76.06	1.88	SILTY SAND to SANDY SILT	25	41	68		44.0
1.950	6.40	61.33	2.78	SANDY SILT to CLAYEY SILT	25	39		4.1	
2.100	6.89	39.39	2.18	SANDY SILT to CLAYEY SILT	16	25		2.6	
2.250	7.38	23.67	7.37	CLAY	24	36		1.4	
2.400	7.87	31.93	6.74	CLAY	32	48		1.9	
2.550	8.37	104.38	1.64	SILTY SAND to SANDY SILT	35	51	78		44.0
2.700	8.86	113.49	3.95	CLAYEY SILT to SILTY CLAY	57	82		6.6	
2.850	9.35	106.82	2.58	SILTY SAND to SANDY SILT	36	51	78		44.0
3.000	9.84	114.47	2.15	SILTY SAND to SANDY SILT	38	54	80		44.0
3.150	10.33	74.87	3.09	SANDY SILT to CLAYEY SILT	30	42		4.4	
3.300	10.83	84.66	2.68	SANDY SILT to CLAYEY SILT	34	47		5.6	
3.450	11.32	46.93	3.97	CLAYEY SILT to SILTY CLAY	23	32		2.7	
3.600	11.81	39.69	2.18	SANDY SILT to CLAYEY SILT	16	21		2.6	
3.750	12.30	63.61	3.09	SANDY SILT to CLAYEY SILT	25	34		3.7	
3.900	12.80	82.22	3.37	SANDY SILT to CLAYEY SILT	33	43		4.8	
4.050	13.29	61.93	4.58	CLAY to SILTY CLAY	41	53		3.6	
4.200	13.78	64.07	3.20	SANDY SILT to CLAYEY SILT	26	33		3.7	
4.350	14.27	49.63	3.00	SANDY SILT to CLAYEY SILT	20	25		3.3	
4.500	14.76	37.43	6.74	CLAY	37	47		2.1	
4.650	15.26	85.49	6.77	*VERY STIFF FINE GRAINED	85	100			
4.800	15.75	100.57	6.94	*VERY STIFF FINE GRAINED	100	100			
4.950	16.24	104.48	6.20	*VERY STIFF FINE GRAINED	100	100			
5.100	16.73	103.48	5.61	*VERY STIFF FINE GRAINED	100	100			
5.250	17.22	105.86	7.52	*VERY STIFF FINE GRAINED	100	100			
5.400	17.72	153.09	5.01	*VERY STIFF FINE GRAINED	100	100			
5.550	18.21	158.61	6.85	*VERY STIFF FINE GRAINED	100	100			
5.700	18.70	293.07	4.33	*VERY STIFF FINE GRAINED	100	100			
5.850	19.19	453.64	2.80	*SAND to CLAYEY SAND	100	100			

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

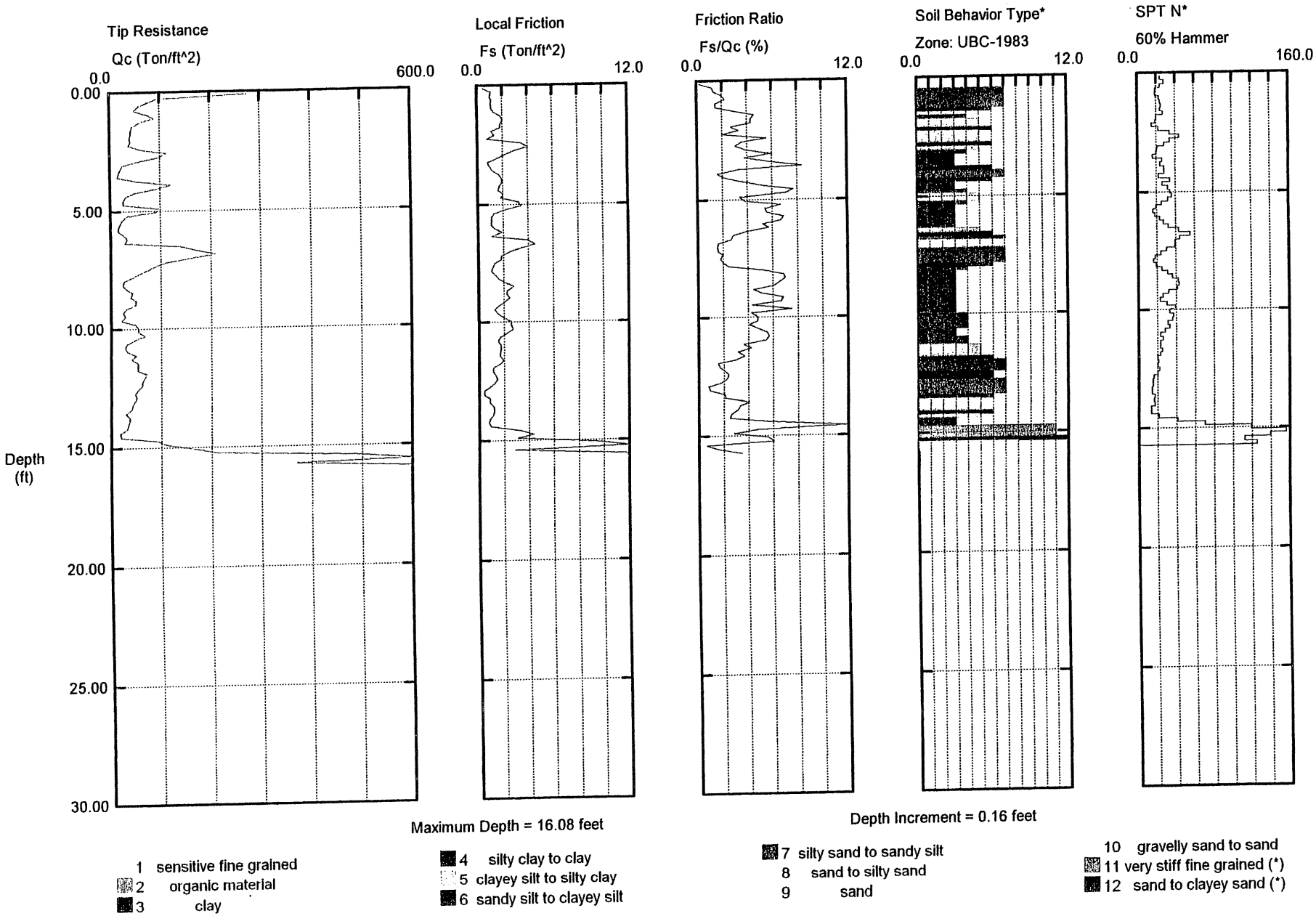
Interpretations based on: Robertson and Campanella, 1989.

l behavior type and SPT b' n data from UBC-1983

Leighton Consu..ng, Inc.

Operator: Bobby-Victor
Sounding: CPT-04A
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 10:28
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 16.08 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-504

CPT INTERPRETATIONS

SOUNDING : CPT-04A
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 10:28

PROJECT No.: 600027-001
CONE/RIG : 468/Bh-VO/R#3

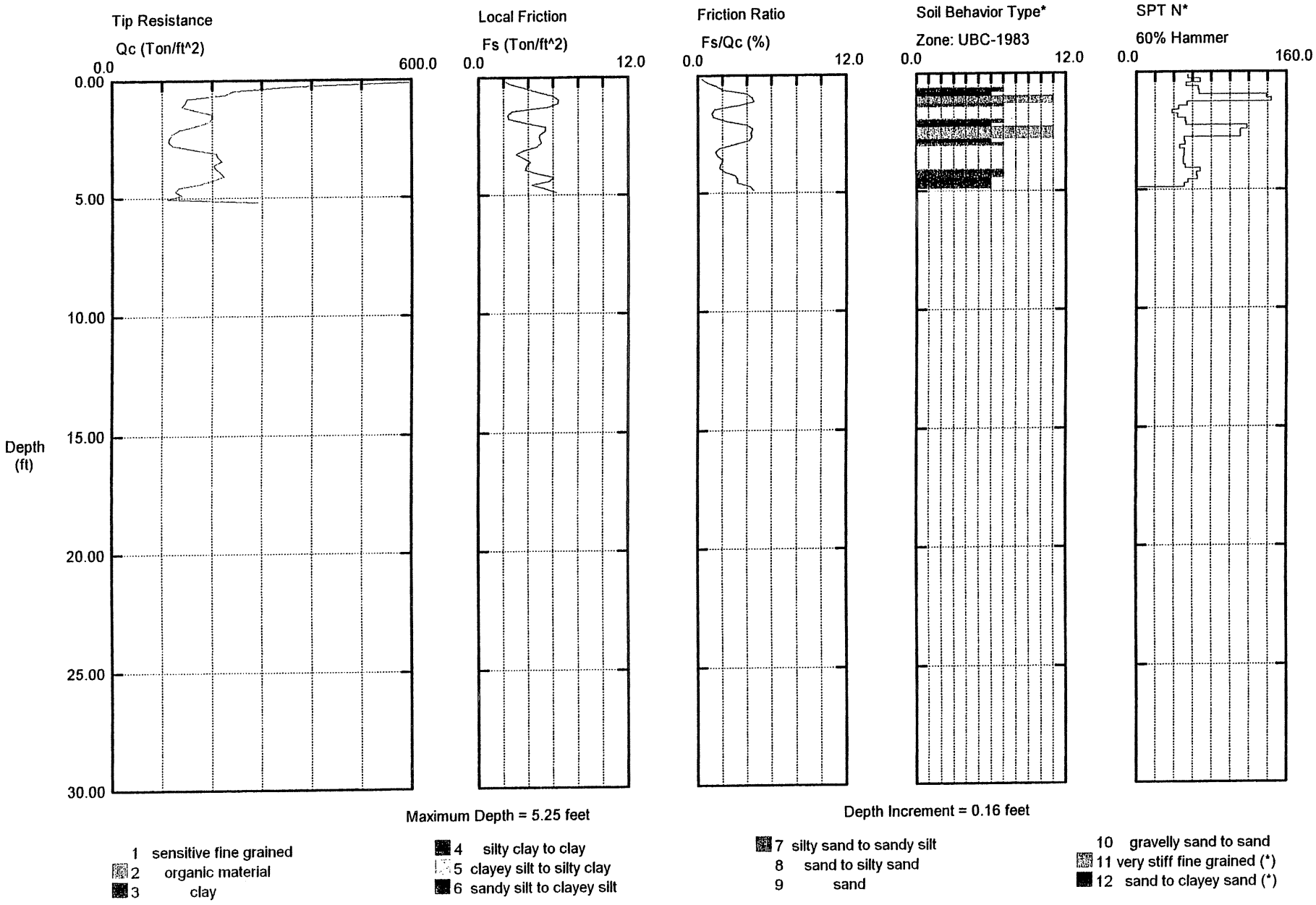
DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	73.04	1.37	SILTY SAND to SANDY SILT	24	39	67		
.300	.98	75.89	1.46	SILTY SAND to SANDY SILT	25	40	68		
.450	1.48	44.55	4.55	CLAY to SILTY CLAY	30	47		2.6	
.600	1.97	39.56	2.69	SANDY SILT to CLAYEY SILT	16	25		2.6	
.750	2.46	62.14	5.52	CLAY	62	99		3.6	
.900	2.95	54.60	3.87	CLAYEY SILT to SILTY CLAY	27	44		3.2	
1.050	3.44	16.42	5.99	CLAY	16	26		1.1	
1.200	3.94	120.33	1.59	SAND to SILTY SAND	30	48	82		47.5
1.350	4.43	30.51	5.28	CLAY	31	49		1.8	
1.500	4.92	97.68	3.35	SANDY SILT to CLAYEY SILT	39	62		5.7	
1.650	5.41	23.71	5.37	CLAY	24	38		1.4	
1.800	5.91	15.38	6.61	CLAY	15	25		1.0	
1.950	6.40	28.00	3.84	CLAY to SILTY CLAY	19	30		1.8	
2.100	6.89	209.09	1.64	SAND to SILTY SAND	52	81	97		47.5
2.250	7.38	90.42	2.00	SILTY SAND to SANDY SILT	30	46	73		44.0
2.400	7.87	40.94	2.44	SANDY SILT to CLAYEY SILT	16	25		2.7	
2.550	8.37	30.59	6.95	CLAY	31	45		1.8	
2.700	8.86	49.56	4.40	CLAY to SILTY CLAY	33	48		2.9	
2.850	9.35	23.48	6.62	CLAY	23	34		1.3	
3.000	9.84	46.65	4.21	CLAYEY SILT to SILTY CLAY	23	33		2.7	
3.150	10.33	67.07	4.05	CLAYEY SILT to SILTY CLAY	34	47		3.9	
3.300	10.83	26.56	5.63	CLAY	27	36		1.5	
3.450	11.32	39.92	4.23	CLAY to SILTY CLAY	27	36		2.3	
3.600	11.81	53.43	3.04	SANDY SILT to CLAYEY SILT	21	29		3.5	
3.750	12.30	61.55	2.09	SILTY SAND to SANDY SILT	21	27	61		40.5
3.900	12.80	47.04	1.78	SILTY SAND to SANDY SILT	16	20	53		39.0
4.050	13.29	42.21	2.04	SANDY SILT to CLAYEY SILT	17	22		2.8	
4.200	13.78	36.46	3.30	CLAYEY SILT to SILTY CLAY	18	23		2.4	
4.350	14.27	29.59	2.47	SANDY SILT to CLAYEY SILT	12	15		1.9	
4.500	14.76	92.86	4.67	*VERY STIFF FINE GRAINED	93	100			
4.650	15.26	210.96	6.01	*VERY STIFF FINE GRAINED	100	100			
4.800	15.75	369.07	3.44	*SAND to CLAYEY SAND	100	100			

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Leighton Consulting, Inc.

Operator: Bobby-Victor
Sounding: CPT-05A
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 11:16
Location: WRT/Dana Point
Job Number: 600027-001



sdf-506

CPT INTERPRETATIONS

SOUNDING : CPT-05A
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 11:16

PROJECT No.: 00027-001
CONE/RIG : 468/BH-VO/R#3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	243.76	1.52	SAND to SILTY SAND	61	97	100		
.300	.98	145.80	4.42	*VERY STIFF FINE GRAINED	100	100			
.450	1.48	199.15	1.33	SAND to SILTY SAND	50	80	96		
.600	1.97	163.88	2.43	SILTY SAND to SANDY SILT	55	87	90		
.750	2.46	113.81	4.32	*VERY STIFF FINE GRAINED	100	100			
.900	2.95	164.07	2.86	SILTY SAND to SANDY SILT	55	87	91		
1.050	3.44	219.03	1.65	SAND to SILTY SAND	55	88	99		
1.200	3.94	215.85	1.74	SAND to SILTY SAND	54	86	98		
1.350	4.43	175.63	3.20	SANDY SILT to CLAYEY SILT	70	100		10.3	
1.500	4.92	138.32	4.52	*VERY STIFF FINE GRAINED	100	100			

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

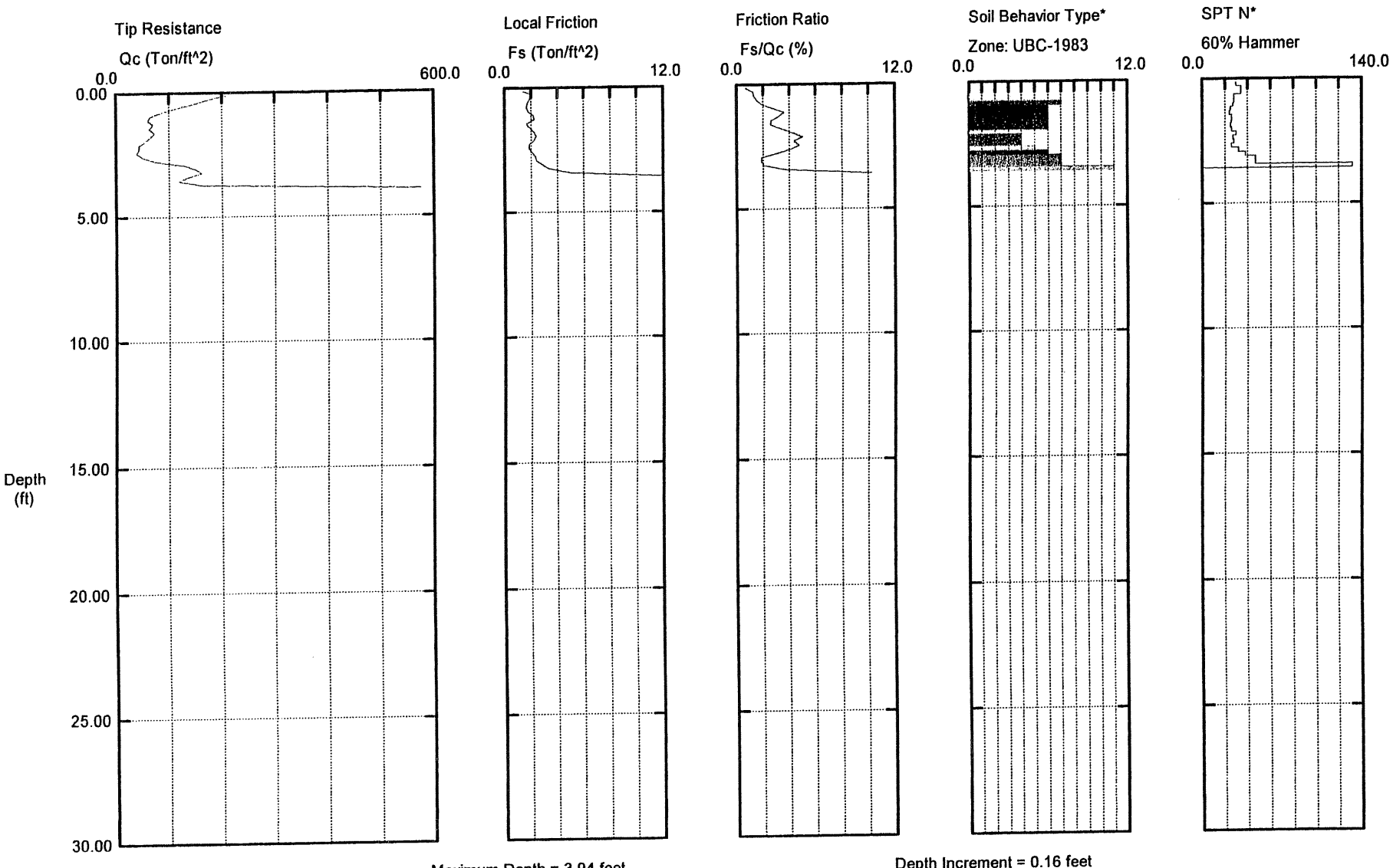
Interpretations based on: Robertson and Campanella, 1989.

il behavior type and SPT h ...n data from UBC-1983

Leighton Consulting, Inc.

Operator: Bobby-Victor
Sounding: CPT-06
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 11:34
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 3.94 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-507

CPT INTERPRETATIONS

SOUNDING : CPT-06
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 11:34

PROJECT No.: 600027-001
CONE/RIG : 468/BH-VO/R#3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	145.23	1.28	SAND to SILTY SAND	36	58	87		
.300	.98	64.61	2.76	SANDY SILT to CLAYEY SILT	26	41		4.3	
.450	1.48	62.93	2.60	SANDY SILT to CLAYEY SILT	25	40		4.2	
.600	1.97	55.30	4.28	CLAYEY SILT to SILTY CLAY	28	44		3.2	
.750	2.46	39.28	4.75	CLAY to SILTY CLAY	26	42		2.3	
.900	2.95	129.00	1.88	SILTY SAND to SANDY SILT	43	69	84		49.0
1.050	3.44	133.97	3.72	SANDY SILT to CLAYEY SILT	54	86		7.9	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

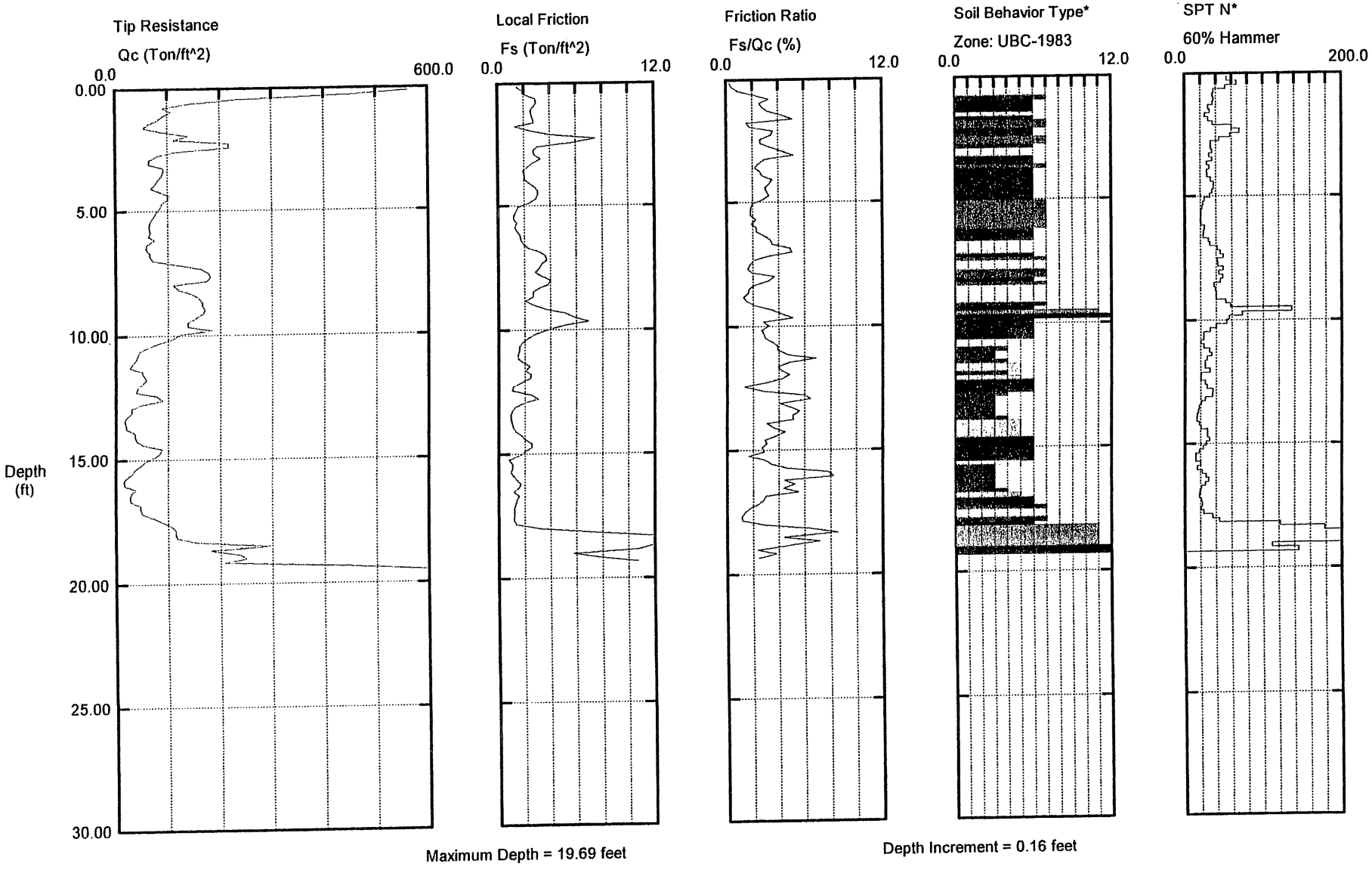
Interpretations based on: Robertson and Campanella, 1989.

il behavior type and SPT h n data from UBC-1983

Leighton Consulting, Inc.

Operator: Bobby-Victor
Sounding: CPT-07
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 11:44
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 19.69 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-508

CPT INTERPRETATIONS

SOUNDING : CPT-07
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 11:44

PROJECT No.: 600027-001
CONE/RIG : 468/BH-VO/R#3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	228.51	.92	SAND	46	73	100		
.300	.98	106.56	2.55	SILTY SAND to SANDY SILT	36	57	78		
.450	1.48	62.46	4.31	CLAYEY SILT to SILTY CLAY	31	50		3.7	
.600	1.97	138.81	1.78	SILTY SAND to SANDY SILT	46	74	86		
.750	2.46	217.08	2.58	SILTY SAND to SANDY SILT	72	100	99		
.900	2.95	64.37	4.35	CLAYEY SILT to SILTY CLAY	32	51		3.8	
1.050	3.44	90.93	2.45	SILTY SAND to SANDY SILT	30	48	74		47.0
1.200	3.94	74.48	2.70	SANDY SILT to CLAYEY SILT	30	48		4.9	
1.350	4.43	104.23	2.94	SANDY SILT to CLAYEY SILT	42	67		6.1	
1.500	4.92	83.98	2.76	SANDY SILT to CLAYEY SILT	34	54		4.9	
1.650	5.41	67.11	1.78	SILTY SAND to SANDY SILT	22	36	65		44.0
1.800	5.91	65.99	1.85	SILTY SAND to SANDY SILT	22	35	64		43.5
1.950	6.40	59.83	2.88	SANDY SILT to CLAYEY SILT	24	38		4.0	
2.100	6.89	65.16	4.83	*VERY STIFF FINE GRAINED	65	100			
2.250	7.38	162.42	2.05	SILTY SAND to SANDY SILT	54	82	90		46.0
2.400	7.87	172.98	1.90	SILTY SAND to SANDY SILT	58	86	92		46.5
2.550	8.37	143.38	2.12	SILTY SAND to SANDY SILT	48	71	87		45.5
2.700	8.86	166.24	1.23	SAND to SILTY SAND	42	60	91		46.0
2.850	9.35	152.41	3.35	SANDY SILT to CLAYEY SILT	61	87		8.9	
3.000	9.84	183.17	2.75	SILTY SAND to SANDY SILT	61	86	94		46.0
3.150	10.33	81.35	2.81	SANDY SILT to CLAYEY SILT	33	45		4.7	
3.300	10.83	41.11	3.89	CLAYEY SILT to SILTY CLAY	21	28		2.4	
3.450	11.32	26.49	6.81	CLAY	26	36		1.5	
3.600	11.81	58.17	4.21	CLAYEY SILT to SILTY CLAY	29	39		3.4	
3.750	12.30	38.01	2.90	CLAYEY SILT to SILTY CLAY	19	25		2.5	
3.900	12.80	50.67	5.98	CLAY	51	66		2.9	
4.050	13.29	20.20	4.89	CLAY	20	26		1.3	
4.200	13.78	19.72	5.07	CLAY	20	25		1.3	
4.350	14.27	39.47	4.44	CLAY to SILTY CLAY	26	33		2.3	
4.500	14.76	82.77	3.02	SANDY SILT to CLAYEY SILT	33	41		4.8	
4.650	15.26	47.59	1.58	SILTY SAND to SANDY SILT	16	20	52		38.5
4.800	15.75	19.38	4.59	CLAY	19	24		1.2	
4.950	16.24	37.67	4.40	CLAY to SILTY CLAY	25	30		2.2	
5.100	16.73	27.87	5.48	CLAY	28	33		1.6	
5.250	17.22	50.33	2.32	SANDY SILT to CLAYEY SILT	20	24		3.3	
5.400	17.72	105.29	1.08	SAND to SILTY SAND	26	31	73		42.5
5.550	18.21	115.78	6.42	*VERY STIFF FINE GRAINED	100	100			
5.700	18.70	180.52	7.10	*VERY STIFF FINE GRAINED	100	100			
5.850	19.19	208.11	3.74	*SAND to CLAYEY SAND	100	100			
6.000	19.69	757.74	*****		0	0			.0

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

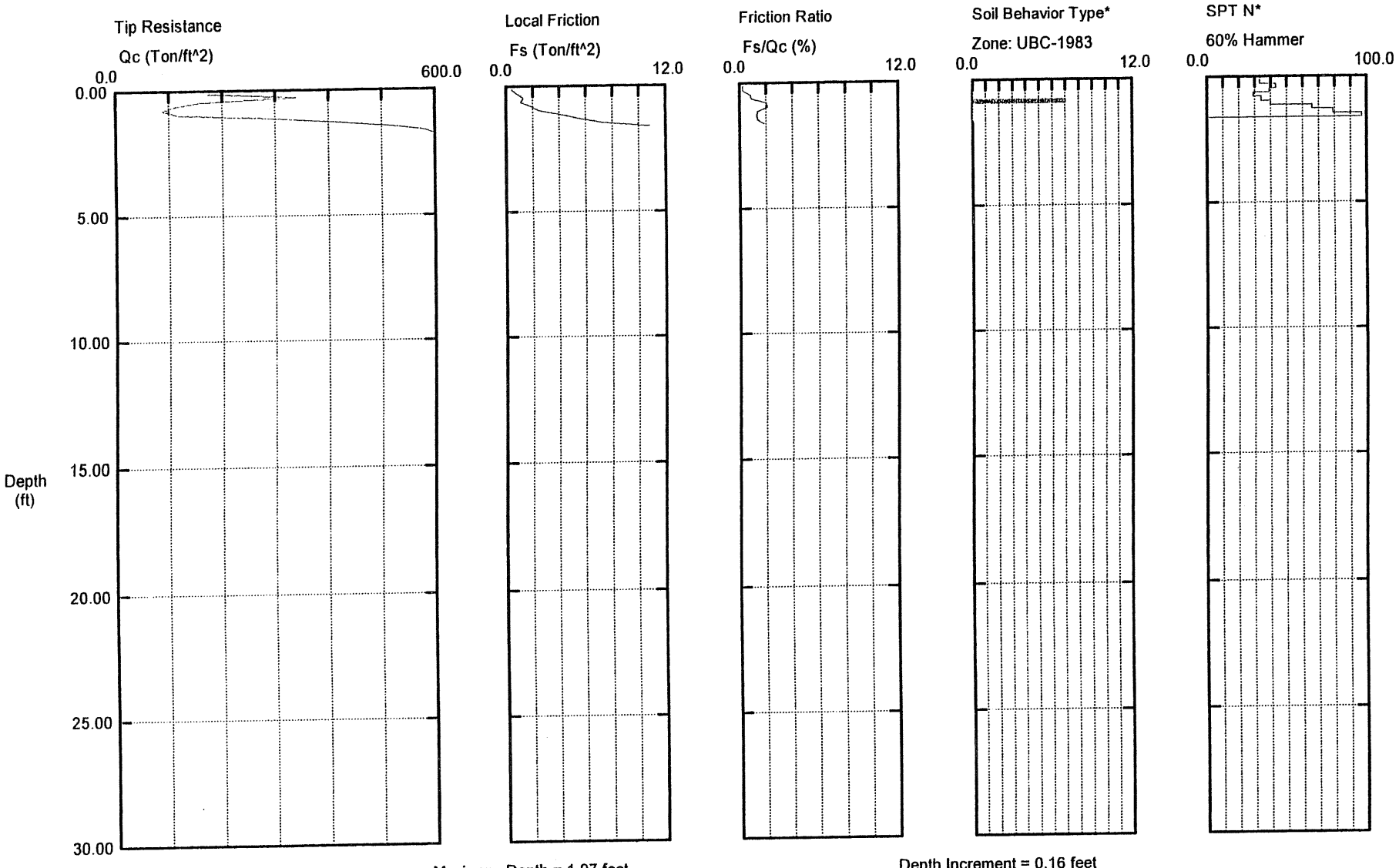
HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Leighton Consulting, Inc.

Operator: Bobby-Victor
Sounding: CPT-08
Cone Used: 468/BH-VO/R#3

CPT Date/Time: 11-15-02 12:25
Location: WRT/Dana Point
Job Number: 600027-001



Maximum Depth = 1.97 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

sdf-509

CPT INTERPRETATIONS

SOUNDING : CPT-08
PROJECT : WRT/Dana Point
DATE/TIME: 11-15-02 12:25

PROJECT No.: 600027-001
CONE/RIG : 468/BH-VO/R#3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	157.79	.82	SAND	32	50	89		
.300	.98	115.53	2.13	SILTY SAND to SANDY SILT	39	62	80		
.450	1.48	522.81	1.38	SAND	100	100	100		

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 120 pcf
ASSUMED DEPTH OF WATER TABLE = 7.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

APPENDIX C

Laboratory Test Results

Leighton Consulting, Inc.

SOIL RESISTIVITY TEST
DOT CA TEST 532 / 643

Project Name: WRT / Dana Point
Project No. : 600024-001
Boring No.: B-1
Sample No. : B-1
Visual Soil Identification: Olive Clayey Sand (SC)

Tested By : VJ Date: 11/13/02
Data Input By: FT Date: 11/18/02
Checked By: FT Date: 11/18/02
Depth (ft.) : 1-6

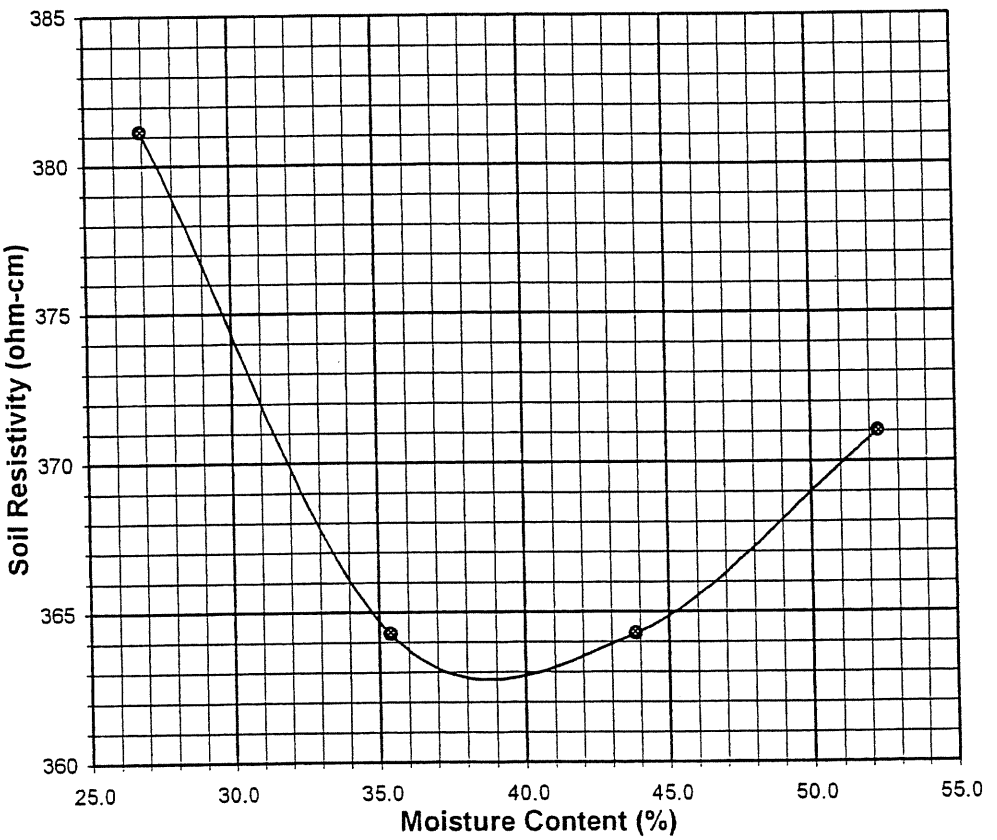
Initial Moisture Content (%)

Wet Wt. of Soil + Cont. (gm.	199.50
Dry Wt. of Soil + Cont. (gm.)	187.46
Wt. of Container (gm	67.01
Moisture Content (%) (MC	10.00

Initial Soil Weight (gm)(Wt	1300.00
Box Constant:	6.7460

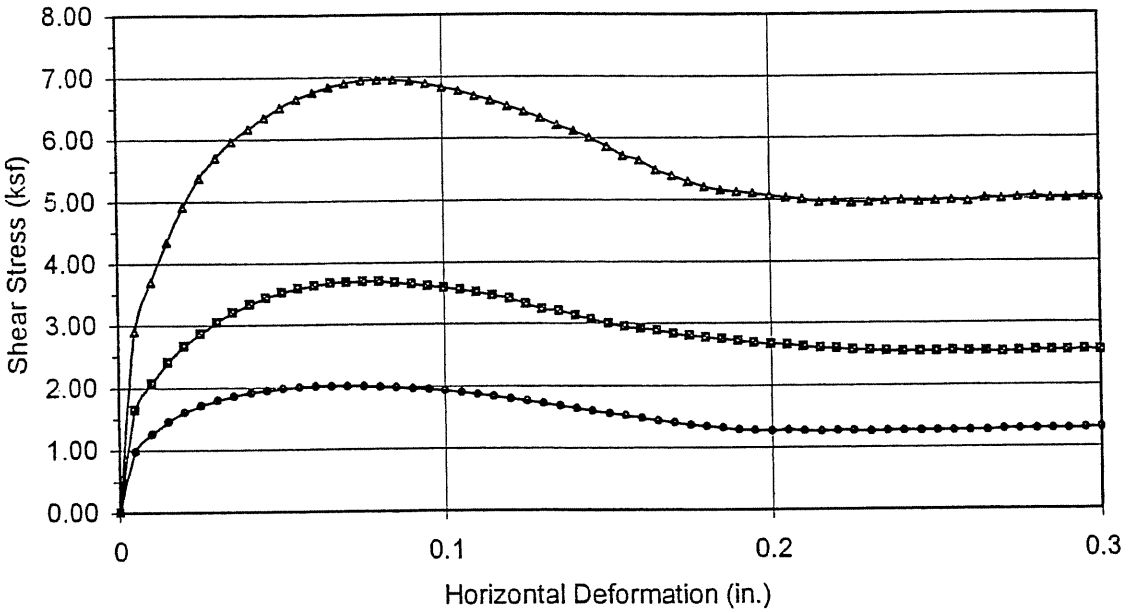
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

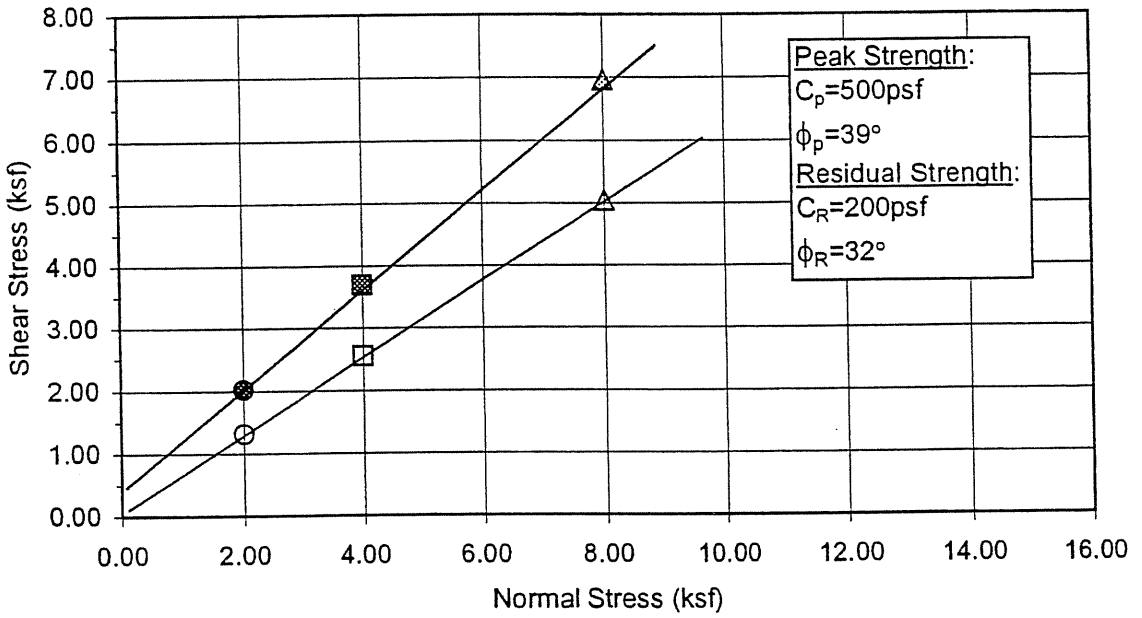
Remolded Specimen	Moisture Adjustments				
Water Added (ml) (Wa)	200	300	400	500	
Adj. Moisture Content (MC)	26.92	35.38	43.84	52.30	
Resistance Rdg. (ohm)	57	54	54	55	
Soil Resistivity (ohm-cm)	381	364	364	371	



Minimum Resistivity	Moisture Content	Sulfate Content	Chloride Content	Soil pH
DOT CA Test 532 / 643		DOT CA Test 417 Part	DOT CA Test	DOT CA Test 532/643
363	**38.5**	**764**	**733**	**7.76 @ 21.3 °C**

Figure C-1





Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 2.011	▩ 3.691	▨ 6.959
Shear Stress @ End of Test (ksf)	○ 1.313	□ 2.547	△ 5.035
Deformation Rate (in./min.)	0.050	0.050	0.050
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	20.08	20.08	20.08
Dry Density (pcf)	110.0	109.7	110.7
Saturation (%)	101.8	101.0	103.8
Soil Height Before Shearing (in.)	0.9895	0.9852	0.9767
Final Moisture Content (%)	20.6	20.5	20.1

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Leighton Consulting, Inc.

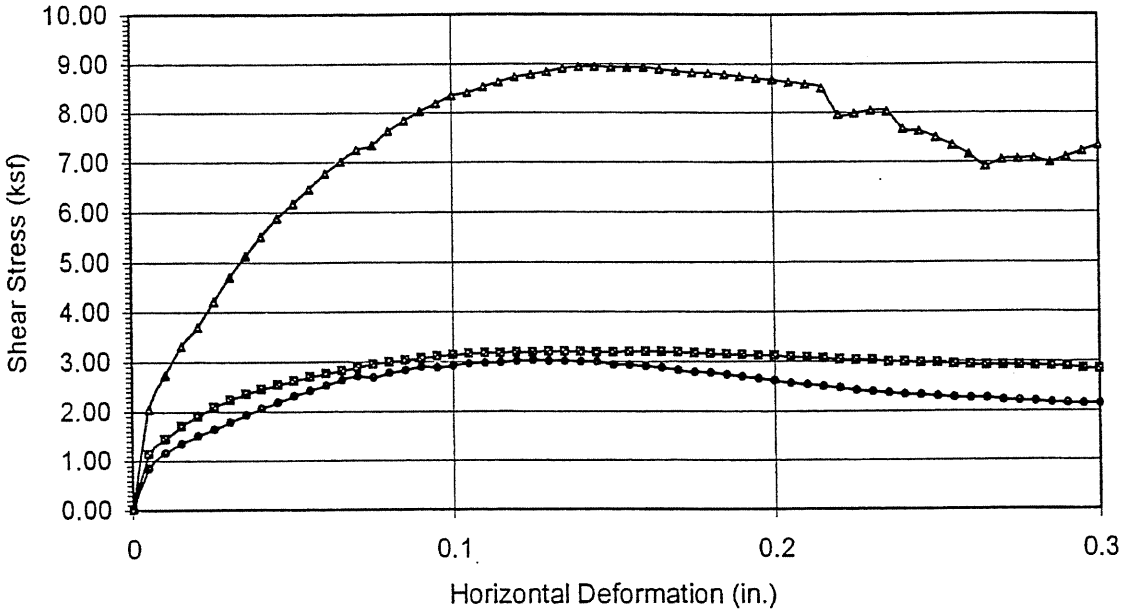
Boring No.: B-2
Sample No.: R-2
Depth (ft) 20
Soil Description: Yellowish Greyish Brown Silty Sand (SM)

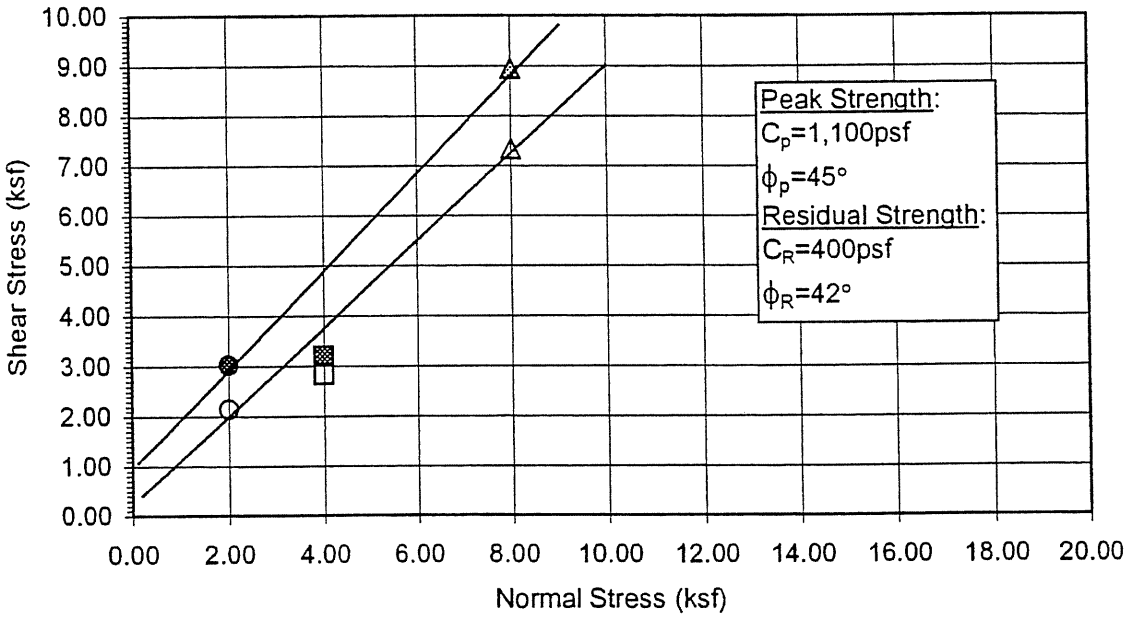
Project No.: 600024-001

WRT / Dana Point

Figure C-2a 12-02

DS B-2,R-2@20





Normal Stress (kip/ft²)	2.000	4.000	8.000
Peak Shear Stress (kip/ft²)	● 3.021	▩ 3.196	◬ 8.948
Shear Stress @ End of Test (ksf)	○ 2.139	□ 2.834	△ 7.342
Deformation Rate (in./min.)	0.050	0.050	0.050
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	12.59	12.59	12.59
Dry Density (pcf)	120.2	120.3	123.3
Saturation (%)	84.5	84.6	92.6
Soil Height Before Shearing (in.)	0.9893	0.9839	0.9740
Final Moisture Content (%)	13.9	13.8	12.4

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Leighton Consulting, Inc.

Boring No.: B-3
Sample No.: R-1
Depth (ft) 5
Soil Description: Greyish Olive Brown Sandy Lean Clay (SC) with 1/2 to 3/4" Sandstone

Project No.: 600024-001
WRT / Dana Point
Figure C-2b 12-02

DS B-3, R-1@5

Leighton Consulting, Inc.

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	WRT / Dana Point	Tested By:	JHW	Date:	11/15/02
Project No. :	600024-001	Checked By:	FT	Date:	11/19/02
Boring No.:	B-2	Depth (ft.)	1-6		
Sample No. :	B-1				

Visual Sample Description: Brown Sandy Clay (CLS)

Dry Wt. of Soil + Cont. (gm.)	1000.00
Wt. of Container No. (gm.)	0.00
Dry Wt. of Soil (gm.)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0185
Wt. Comp. Soil + Mold (gm.)	629.90	442.00
Wt. of Mold (gm.)	209.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (gm.)	839.50	651.10
Dry Wt. of Soil + Cont. (gm.)	776.60	598.40
Wt. of Container (gm.)	0.00	209.10
Moisture Content (%)	8.10	13.54
Wet Density (pcf)	126.9	130.9
Dry Density (pcf)	117.4	115.3
Void Ratio	0.436	0.462
Total Porosity	0.303	0.316
Pore Volume (cc)	62.8	66.6
Degree of Saturation (%) [S_{meas}]	**50.2**	79.1

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h.

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
11/15/02	8:56	1.0	0	0.1105
11/15/02	9:06	1.0	0	0.1100
Add Distilled Water to the Specimen				
11/18/02	14:59	1.0	0	0.1290
11/18/02	16:22	1.0	0	0.1290

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	19.0
Expansion Index (EI)$_{50}$ = EI_{meas} - (50 -S_{meas})x((65+EI_{meas}) / (220-S_{meas}))	**19**

Figure C-3a

Leighton Consulting, Inc.

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	WRT / Dana Point	Tested By:	JHW	Date:	11/14/02
Project No. :	600024-001	Checked By:	FT	Date:	11/18/02
Boring No.:	B-1	Depth (ft.)	1-6		
Sample No. :	B-1				

Visual Sample Description: Brown Clayey Sand (SC)

Dry Wt. of Soil + Cont. (gm.)	1000.00
Wt. of Container No. (gm.)	0.00
Dry Wt. of Soil (gm.)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0205
Wt. Comp. Soil + Mold (gm.)	624.80	441.80
Wt. of Mold (gm.)	206.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (gm.)	843.70	647.90
Dry Wt. of Soil + Cont. (gm.)	781.90	594.10
Wt. of Container (gm.)	0.00	206.10
Moisture Content (%)	7.90	13.87
Wet Density (pcf)	126.3	130.6
Dry Density (pcf)	117.0	114.7
Void Ratio	0.440	0.470
Total Porosity	0.306	0.320
Pore Volume (cc)	63.3	67.5
Degree of Saturation (%) [S_{meas}]	**48.5**	79.7

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h.

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
11/14/02	16:37	1.0	0	0.0785
11/14/02	16:47	1.0	0	0.0780
Add Distilled Water to the Specimen				
11/15/02	8:22	1.0	0	0.0990
11/15/02	10:54	1.0	0	0.0990

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	21.0
Expansion Index (EI)$_{50}$ = EI_{meas} - (50 -S_{meas})x((65+EI_{meas}) / (220-S_{meas}))	**20**

Figure C-3b

Leighton Consulting, Inc.

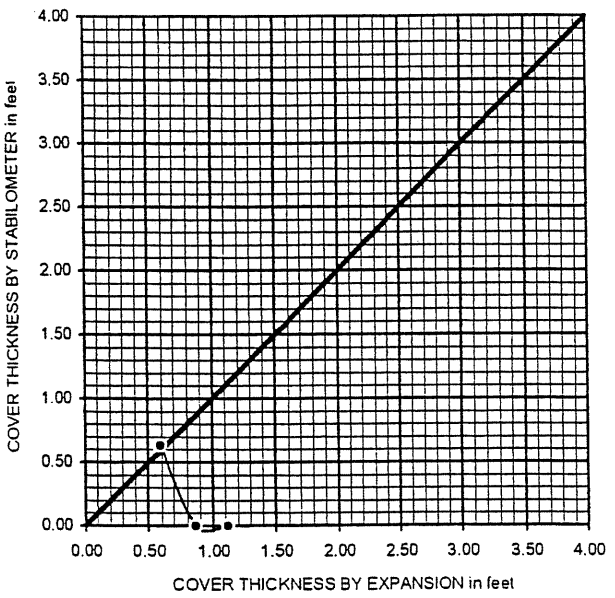
R-VALUE TEST RESULTS

PROJECT NAME:	WRT / Dana Point	PROJECT NUMBER:	600024-001
SAMPLE NUMBER:	B-1	SAMPLE LOCATION:	B-2 1-6'
SAMPLE DESCRIPTION:	Sandy Clay (CLS)	TECHNICIAN:	SCF
		DATE SAMPLED	11/6/2002

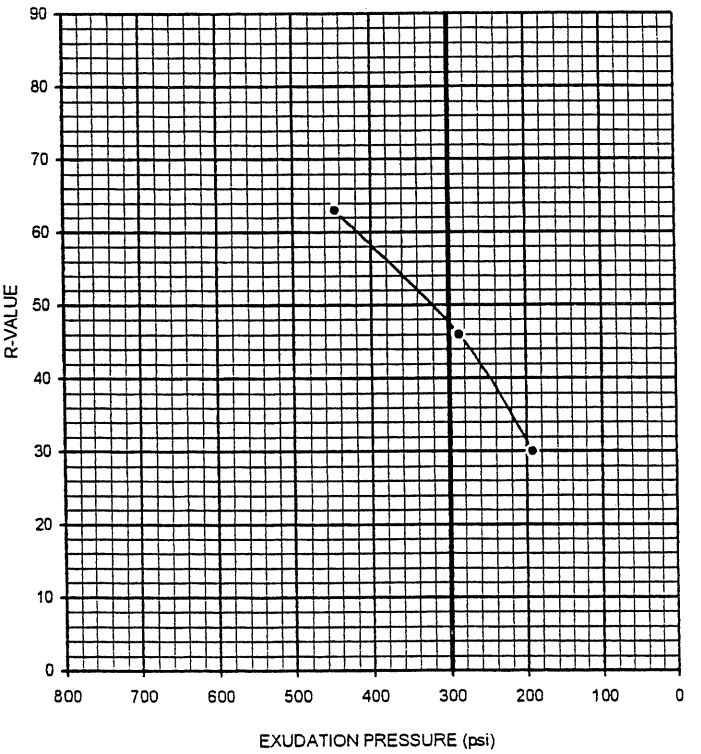
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	9.5	10.0	10.4
HEIGHT OF SAMPLE, Inches	2.41	2.50	2.60
DRY DENSITY, pcf	126.5	125.7	125.4
COMPACTOR AIR PRESSURE, psf	200	100	75
EXUDATION PRESSURE, psf	446	287	193
EXPANSION, Inches x 10exp-4	19	0	0
STABILITY Ph 2,000 lbs (160 psi)	41	67	97
TURNS DISPLACEMENT	3.72	4.00	4.27
R-VALUE UNCORRECTED	66	46	28
R-VALUE CORRECTED	63	46	30

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.59	0.86	1.12
EXPANSION PRESSURE THICKNESS, ft.	0.63	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	63
R-VALUE BY EXUDATION:	47
EQUILIBRIUM R-VALUE:	47

Figure C-4a

Leighton Consulting, Inc.

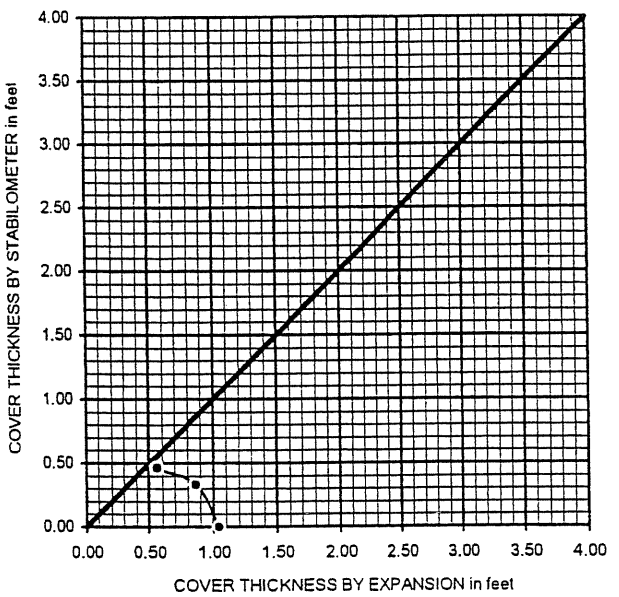
R-VALUE TEST RESULTS

PROJECT NAME:	WRT / Dana Point	PROJECT NUMBER:	600024-001
SAMPLE NUMBER:	B-1	SAMPLE LOCATION:	B-3 1-6'
SAMPLE DESCRIPTION:	Sandy Clay (CLS)	TECHNICIAN:	SCF
		DATE SAMPLED	11/6/2002

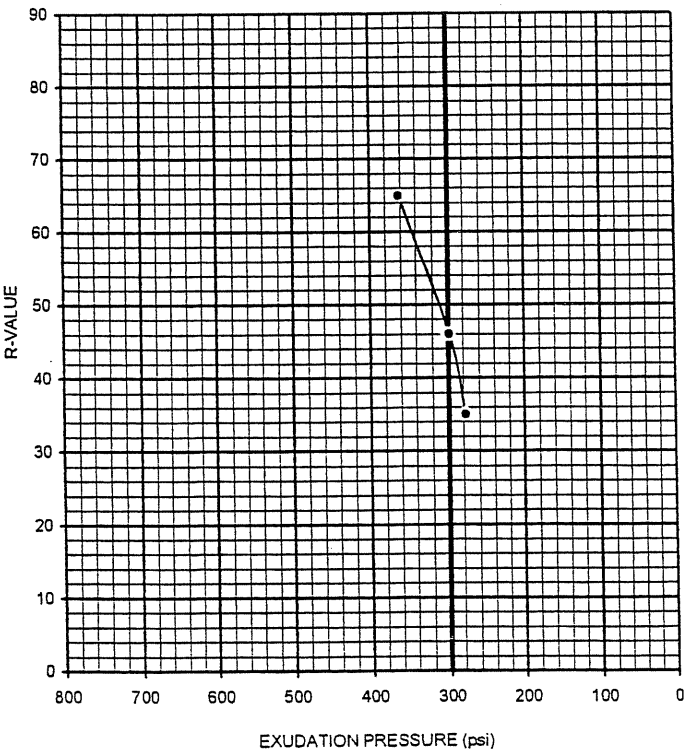
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.0	11.5	11.7
HEIGHT OF SAMPLE, Inches	2.47	2.49	2.47
DRY DENSITY, pcf	123.1	122.3	122.3
COMPACTOR AIR PRESSURE, psf	150	100	75
EXUDATION PRESSURE, psf	362	299	278
EXPANSION, Inches x 10exp-4	14	10	0
STABILITY Ph 2,000 lbs (160 psi)	43	68	86
TURNS DISPLACEMENT	3.70	3.91	4.02
R-VALUE UNCORRECTED	65	46	35
R-VALUE CORRECTED	65	46	35

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.56	0.86	1.04
EXPANSION PRESSURE THICKNESS, ft.	0.47	0.33	0.00

EXPANSION PRESSURE CHART

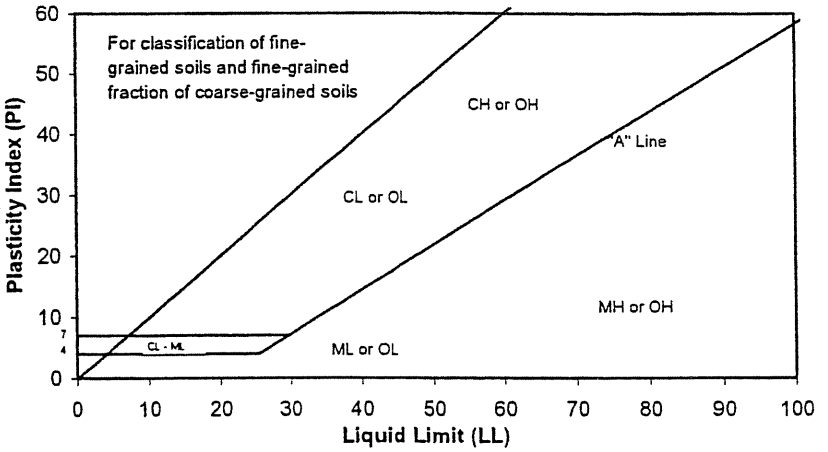


EXUDATION PRESSURE CHART



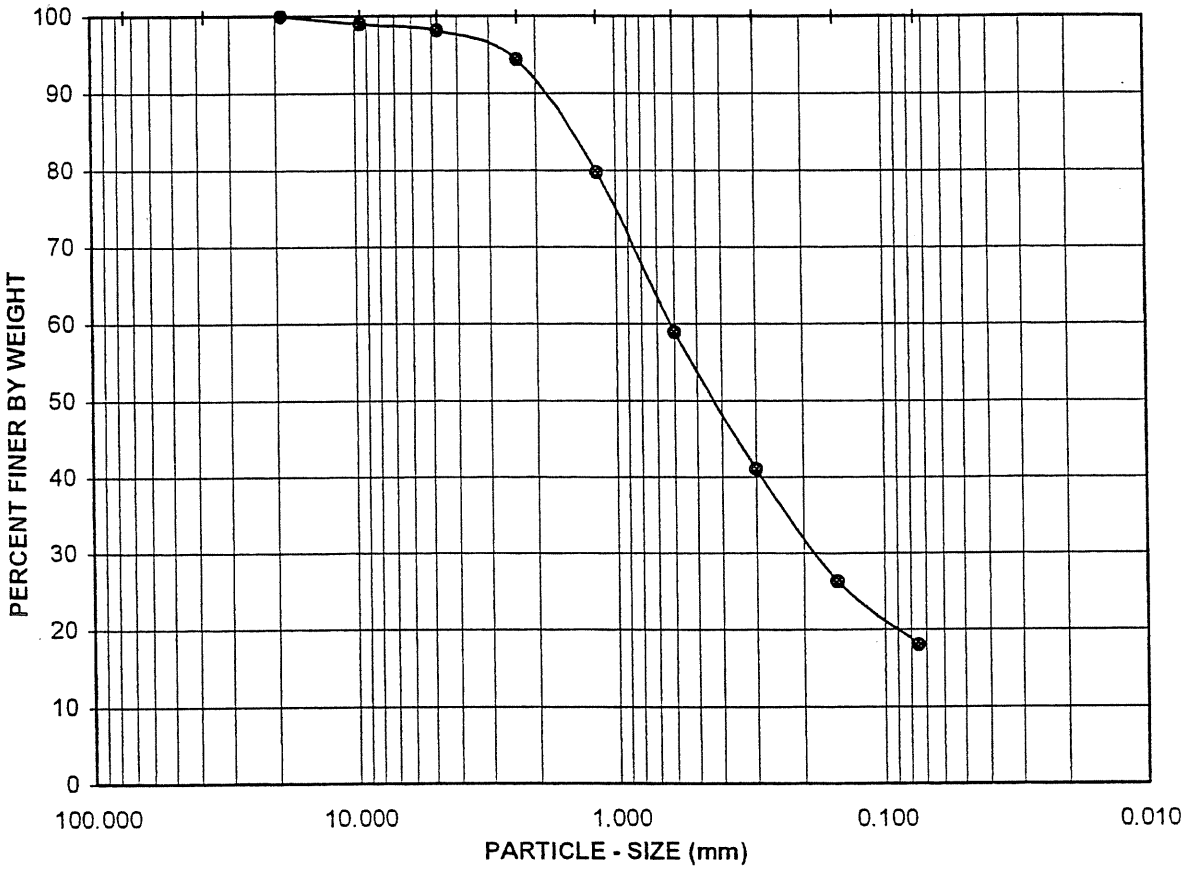
R-VALUE BY EXPANSION:	69
R-VALUE BY EXUDATION:	46
EQUILIBRIUM R-VALUE:	46

Figure C-4b



GRAVEL		SAND			FINES
COARSE	FINE	CRSE	MEDIUM	FINE	SILT

U.S. STANDARD SIEVE OPENING U.S. STANDARD SIEVE NUMBER HYDROMETER



Boring No.:	Sample No.:	Depth (ft.):	Soil Type	GR:SA:FI	LL,PL,PI
B-3	R-2	6-10	(SC)	2 : 80 : 18	N/A

Visual Sample Description:
Clayey Sand (SC)

Leighton Consulting, Inc.

Project No.: 600024-001

WRT / Dana Point

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Figure C-5 12-02

Sieve B-3, B-2@ 6_10

Leighton Consulting, Inc.

COMPACTION TEST

ASTM D 1557

Project Name: WRT/Dana Point | Tested By : MTR | Date: 12/01/02
Project No.: 600024-001 | Calculated By : | Date: 12/02/02
Boring No.: B-3 | Depth (ft.) 6-10
Sample No. : B-2
Visual Sample Description: Olive Clayey Sand

Preparation Methoc

[ ] Moist
[X] Dry

[X] Mechanical Ram
[ ] Manual Ram

Mold Volume (ft ³) 0.03322 *Ram Weight 10 LBS Drop 18 inches*

-2.5 0 2.5 5

TEST NO.	1	2	3	4	5	6
Wt. Comp. Soil + Mold (gm.)	3655.0	3735.0	3811.0	3807.0		
Wt. of Mold (gm.)	1707.0	1707.0	1707.0	1707.0		
Net Wt. of Soil (gm.)	1948.0	2028.0	2104.0	2100.0		
Wet Wt. of Soil + Cont. (gm.)	447.70	470.90	395.30	483.50		
Dry Wt. of Soil + Cont. (gm.)	432.60	447.40	368.50	440.80		
Wt. of Container (gm.)	54.70	52.00	50.40	53.20		
Moisture Content (%)	4.00	5.94	8.43	11.02		
Wet Density (pcf)	129.3	134.6	139.6	139.4		
Dry Density (pcf)	124.3	127.0	128.8	125.5		

Maximum Dry Density (pcf) 129.0 **Optimum Moisture Content (%** 8.0

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if No.4 retained < 20%

[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if + #4 > 20% and + 3/8 " < 20%

[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if + 3/8 in >20% and + ¾ in <30%

Particle-Size Distribution:
GR:SA:FI

Atterberg Limits:
LL,PL,PI

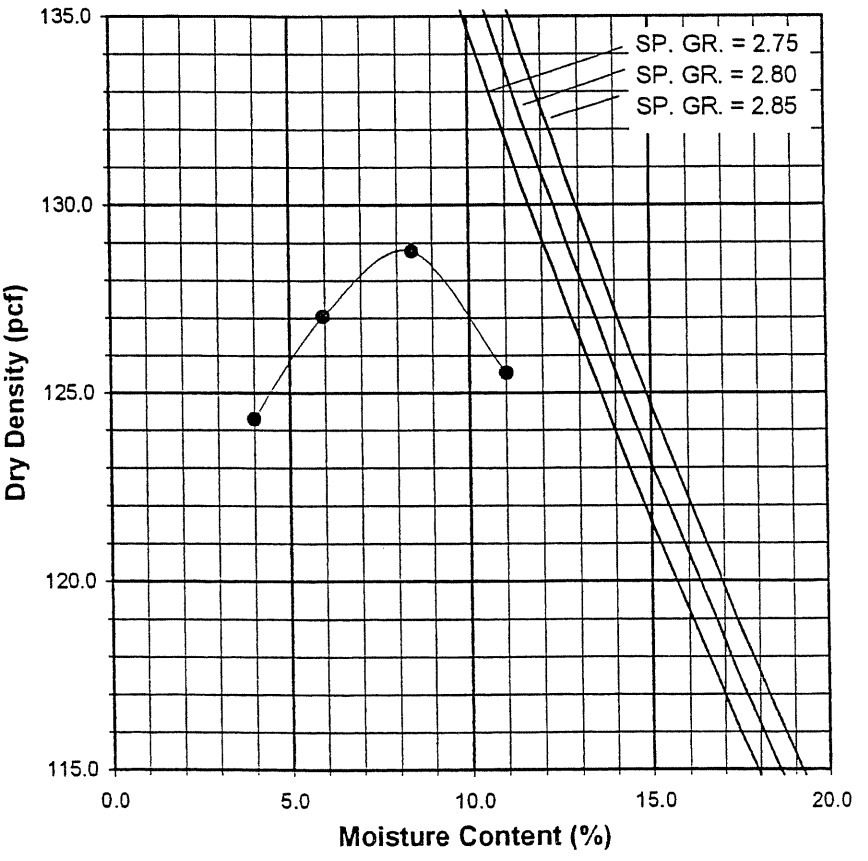


Figure C-6

Leighton Consulting, Inc.

ATTERBERG LIMITS

ASTM D 4318

Project Name:	WRT / Dana Point	Tested By:	ACS	Date:	11/21/02
Project No. :	600024-001	Input By:	FT	Date:	12/02/02
Boring No.:	B-2	Checked By:	FT	Date:	11/22/02
Sample No.:	B-2	Depth (ft.)	6-10		

Visual Sample Description: Olive Lean Clay (CL)

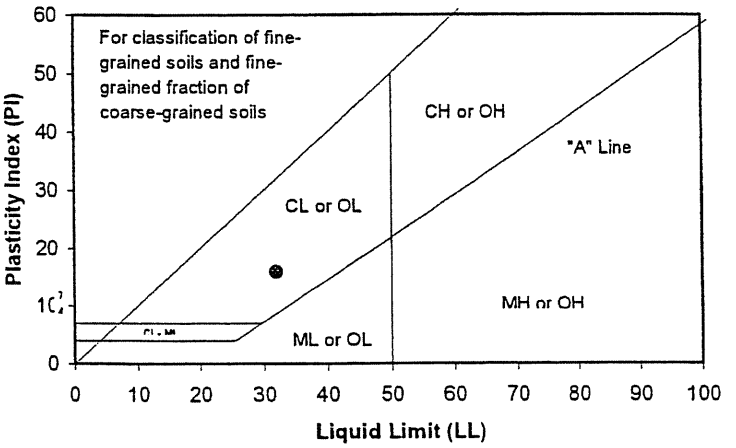
	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	
Number of Blows [N]			30	22	16	
Wet Wt. of Soil + Cont. (gm)	9.72	9.61	17.21	17.53	16.41	
Dry Wt. of Soil + Cont. (gm)	8.52	8.42	13.36	13.44	12.44	
Wt. of Container (gm)	1.10	1.12	1.04	1.07	1.00	
Moisture Content (%) [Wn]	16.17	16.30	31.25	33.06	34.70	

Liquid Limit	**32**
Plastic Limit	**16**
Plasticity Index	**16**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) = 8.76

One - Point Liquid Limit Calculation

LL =Wn(N/25) $^{0.121}$



PROCEDURES USED

- ☐ Wet Preparation
  Multipoint - Wet
- ☒ Dry Preparation
  Multipoint - Dry
- ☒ Procedure A
  Multipoint Test
- ☐ Procedure B
  One-point Test

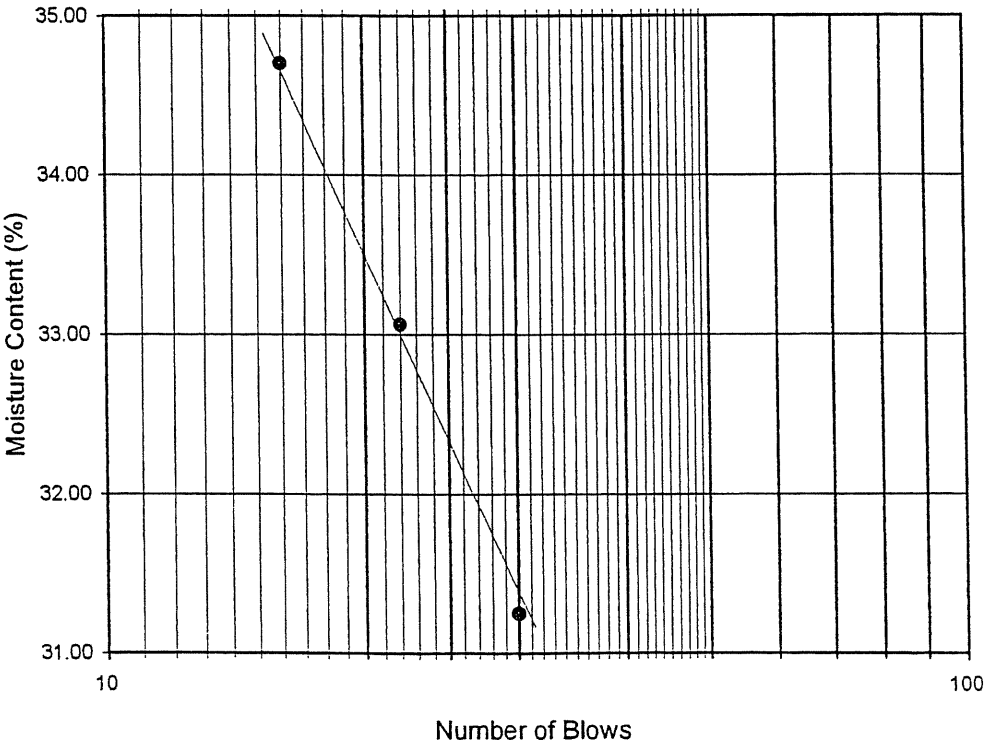
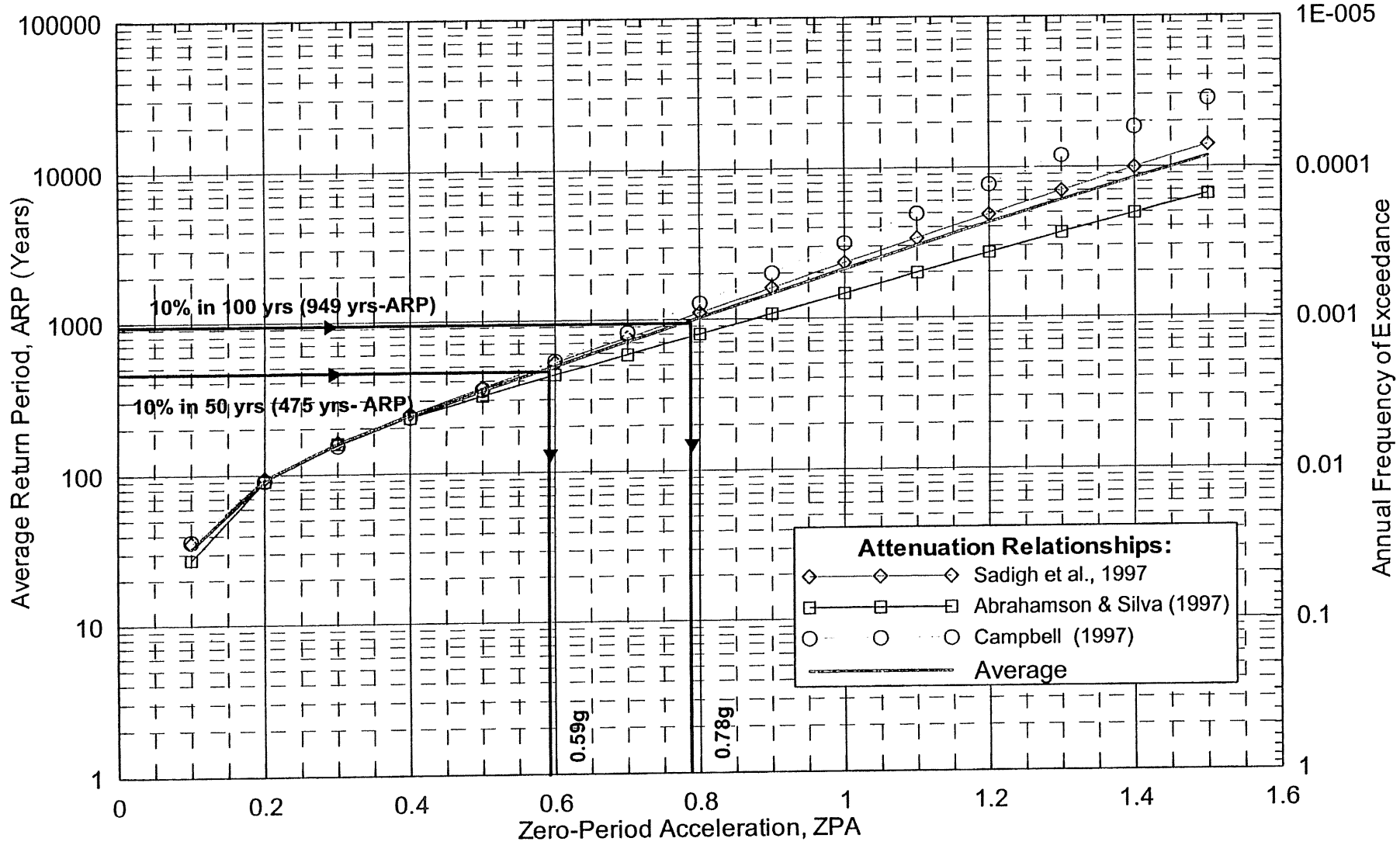


Figure C-7



WRT - DANA POINT	
600024-001	
Leighton Consulting A LEIGHTON GROUP COMPANY	

Total Hazard ZPA for WRT- Dana Point

Figure D-1

ZPA-ARP1.xls

Date: December 2002

Figure D -1.grf

APPENDIX D

Seismic Hazard Analysis

FAULT LIST
FOR
WRT/DANA POINT

Project No. 600024-001

SITE COORDINATES:
SITE LATITUDE: 33.4614
SITE LONGITUDE: 117.6931
SEARCH RADIUS: 100 mi

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT		
		MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN JOAQUIN HILLS	1.7(2.7)	6.8	0.882	XI
NEWPORT-INGLEWOOD (Offshore)	2.5(4.0)	6.9	0.621	X
NEWPORT-INGLEWOOD (L.A.Basin)	16.7(26.8)	6.9	0.144	VIII
PALOS VERDES	18.5(29.7)	7.1	0.141	VIII
CORONADO BANK	19.0(30.5)	7.4	0.157	VIII
ELSINORE-GLEN IVY	22.6(36.3)	6.8	0.100	VII
ELSINORE-TEMECULA	23.5(37.8)	6.8	0.096	VII
CHINO-CENTRAL AVE. (Elsinore)	24.8(39.9)	6.7	0.112	VII
COMPTON THRUST	26.5(42.7)	6.8	0.110	VII
WHITTIER	27.3(44.0)	6.8	0.082	VII
ROSE CANYON	27.7(44.5)	6.9	0.085	VII
ELYSIAN PARK THRUST	30.3(48.7)	6.7	0.091	VII
ELSINORE-JULIAN	39.6(63.8)	7.1	0.065	VI
SAN JOSE	41.3(66.5)	6.5	0.058	VI
SAN JACINTO-SAN JACINTO VALLEY	45.7(73.5)	6.9	0.050	VI
SIERRA MADRE	45.8(73.7)	7.0	0.069	VI
CUCAMONGA	45.9(73.9)	7.0	0.069	VI
SAN JACINTO-SAN BERNARDINO	46.1(74.2)	6.7	0.044	VI
SAN JACINTO-ANZA	48.7(78.4)	7.2	0.056	VI
RAYMOND	51.6(83.1)	6.5	0.045	VI
CLAMSHELL-SAWPIT	52.5(84.5)	6.5	0.044	VI
VERDUGO	53.4(85.9)	6.7	0.049	VI
HOLLYWOOD	55.0(88.5)	6.4	0.039	V
SAN ANDREAS - Southern	55.9(89.9)	7.4	0.055	VI
SAN ANDREAS - San Bernardino	55.9(89.9)	7.3	0.052	VI
SAN ANDREAS - 1857 Rupture	59.4(95.6)	7.8	0.067	VI
SAN ANDREAS - Mojave	59.4(95.6)	7.1	0.043	VI
SANTA MONICA	59.5(95.7)	6.6	0.041	V
CLEGHORN	59.6(95.9)	6.5	0.029	V
NORTH FRONTAL FAULT ZONE (West)	61.9(99.6)	7.0	0.050	VI
MALIBU COAST	62.4(100.4)	6.7	0.041	V
SIERRA MADRE (San Fernando)	66.2(106.6)	6.7	0.039	V
EARTHQUAKE VALLEY	66.9(107.7)	6.5	0.026	V
NORTHRIDGE (E. Oak Ridge)	67.1(108.0)	6.9	0.043	VI
ANACAPA-DUME	68.1(109.6)	7.3	0.056	VI
SAN GABRIEL	68.2(109.7)	7.0	0.035	V
SAN JACINTO-COYOTE CREEK	68.3(109.9)	6.8	0.031	V
PINTO MOUNTAIN	69.5(111.8)	7.0	0.034	V
NORTH FRONTAL FAULT ZONE (East)	73.6(118.5)	6.7	0.034	V
SANTA SUSANA	75.4(121.4)	6.6	0.031	V

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	ESTIMATED MAX. EARTHQUAKE EVENT: PEAK SITE ACCEL. g	ESTIMATED MAX. EARTHQUAKE EVENT: EST. SITE INTENSITY MOD.MERC.
SAN ANDREAS - Coachella	77.4(124.5)	7.1	0.033	V
HELENDALE - S. LOCKHARDT	78.7(126.7)	7.1	0.032	V
HOLSER	81.0(130.3)	6.5	0.027	V
BURNT MTN.	83.0(133.5)	6.4	0.019	IV
ELSINORE-COYOTE MOUNTAIN	84.1(135.3)	6.8	0.025	V
SIMI-SANTA ROSA	85.6(137.8)	6.7	0.029	V
EUREKA PEAK	85.7(138.0)	6.4	0.018	IV
OAK RIDGE (Onshore)	85.7(138.0)	6.9	0.034	V
LENWOOD-LOCKHART-OLD WOMAN SPRGS	86.7(139.6)	7.3	0.034	V
LANDERS	88.1(141.8)	7.3	0.033	V
SAN JACINTO - BORREGO	88.3(142.1)	6.6	0.020	IV
SAN CAYETANO	91.3(146.9)	6.8	0.029	V
JOHNSON VALLEY (Northern)	92.8(149.4)	6.7	0.021	IV
OAK RIDGE(Blind Thrust Offshore)	96.7(155.7)	6.9	0.029	V
CHANNEL IS. THRUST (Eastern)	97.4(156.8)	7.4	0.042	VI
SAN ANDREAS - Carrizo	97.6(157.1)	7.2	0.028	V
EMERSON So. - COPPER MTN.	97.8(157.4)	6.9	0.022	IV

-END OF SEARCH-

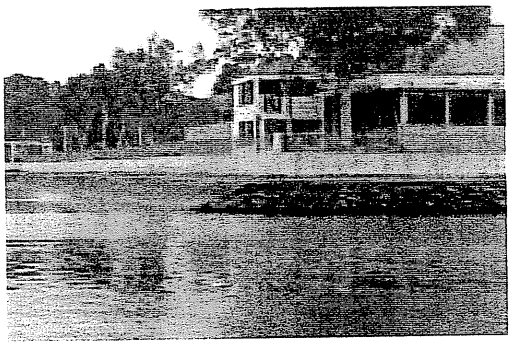
57 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

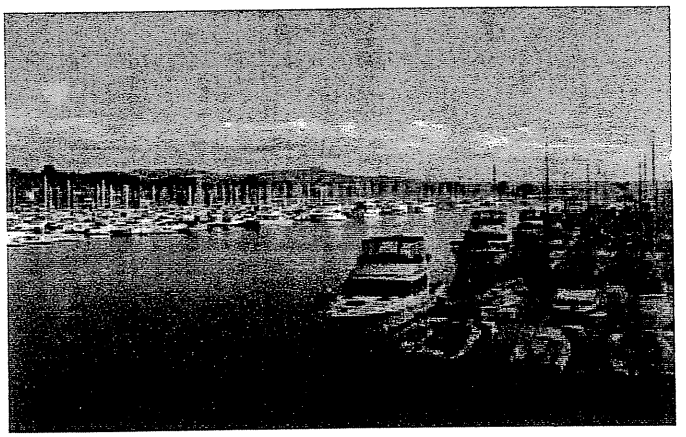
THE SAN JOAQUIN HILLS FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.7 MILES (2.7 km) AWAY.

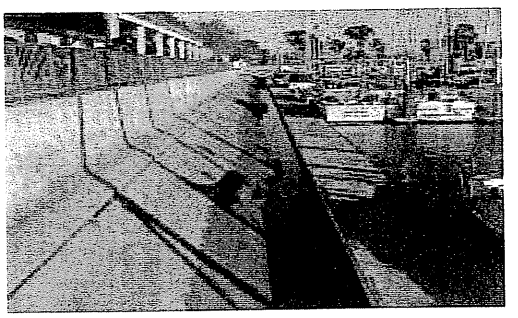
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8820 g

APPENDIX B

Site Photos







Dana Point Marina Redevelopment

Bulkhead Structural Evaluation

Prepared for
County of Orange

December 2003

Bulkhead Structural Evaluation

Dana Point Marina Redevelopment

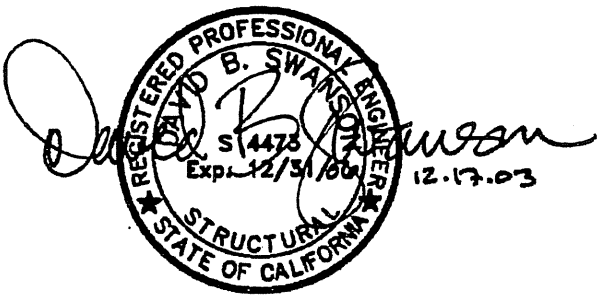
December 17, 2003

Prepared for:

County of Orange
Public Facilities and Resources Department

and

A.C. Martin Partners, Inc.



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Principal

Prepared by:

BLUEWater Design Group

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Hugh Townsend, P.E., S.E

Doug Wilson, P.E.

Corbin Hammer

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Executive Summary

The Dana Point Marina, constructed in 1968, is a 1,400 plus slip marina located in the City of Dana Point, California. Within the marina basin, the uplands soil is retained by a reinforced concrete quay wall at the upper 5 feet with reinforced concrete revetment panels over 15 feet of fill soil sloped at 1.5H:1V. The quay wall extending from the basin around to the boat launch is similar to the system in the basin except a stone rubble revetment was utilized in place of the concrete revetment panels.

The overall structural condition of the exposed portions of the quay wall and concrete revetment in the basin area appears to be in relatively good condition considering its approximate 35-year age and exposure to salt water, the elements, and tides. Select areas of the quay wall and the concrete revetment panels show signs of deterioration and distress, which will continue in the future. It is not anticipated that the existing 35-year-old quay wall system has a remaining design life of 35 to 50 years for the proposed development; therefore, we expect that some repair, retrofit, or replacement may be warranted.

Site investigations were performed by Mr. Tim Bazley of BLUEWater Design Group on October 26 and again with Mr. David Swanson and Mr. Doug Wilson on November 23, 24, and 25, 2003. The purpose of the investigation was to assess the general structural condition and performance of the existing bulkhead and review future bulkhead performance needs based on the Master Plan. Observations include:

- There has been overall vertical settlement of 1 to 2 inches throughout the site.
- There has been relative movement between the quay wall precast stem panels.
- Both the footing and revetment panel corners are cracked where bearing contact has been made.
- There is spalling of concrete on the wall face.
- Voids are visible on the surface of the concrete stem wall and the larger aggregate is visible.
- There are areas of significant concrete slab-on-grade grinding where differential settlement across joints has resulted in vertical edges appearing.
- Concrete is spalling along the edge of the footing, particularly at the riprap slope.
- The riprap has settled under the quay wall up to 6-10 inches in some areas.
- Dislodged concrete revetment panels were observed in a number of locations.
- The riprap stone revetment shows variable signs of settlement and movement since its construction.

To better understand the capability of the existing wall systems, the existing design was evaluated based on the design values provided in the current project geotechnical report. Based on this preliminary analysis, the system is currently structurally adequate under dead load. The system is under designed for the additional loads due to surcharge live loads, such as from

traffic, as well as seismic loading. Due to these deficiencies, the expected continued degradation of the existing wall, and the magnitude of uplands development, repair or replacement is recommended. Estimated costs for repair range from $200 to $1,000 per linear foot. Estimated costs for replacement may be as low as $1,000 per linear foot to as much as $3,000 per linear foot. Prior to establishing the final solution, additional investigation and evaluation is recommended.

1.0 Introduction

1.1 Project Overview

The Dana Point Marina is a 1,400 plus slip marina located in the City of Dana Point, California. Constructed around 1968 by the County of Orange and the U.S. Corps of Engineers, the facility serves both commercial and pleasure vessels along with providing uplands retail, restaurant, and hotel services. Maritime facilities include several boatyards for small boat repair and storage as well as boat, kayak, and jet ski rental. Recreational facilities include picnic areas and a beach.

The marina is configured into four quadrants as delineated by uplands facilities on the north and an artificially created island to the south, both of which are bisected by an access bridge. A rubble breakwater to the south provides wave protection from the Pacific Ocean. See Sketch 1 for a map of the overall marina facility. Based on available record drawings, the marina was created by depositing soil fill in the wet for a bulkhead with sections of steel sheet piling to create a cofferdam. The marina area was then dewatered and the basin excavated and existing land masses created with fill in the dry. The uplands fill area at the edge of the marina is at approximate elevation +10 feet (MLLW). The bottom of the basins are at approximately –10 to –12 feet, which generally coincides with the bedrock elevation on the site. Thus, the uplands fill is roughly 20 feet thick.

We understand that the current $90 million project includes redevelopment of the uplands facilities along with select float reconfiguration and is limited to the northeast corner of the marina. The scope of work for this assignment was to visually examine and investigate the structural adequacy of the existing shoreline quay wall and revetment within the area of proposed redevelopment. This encompasses the area from approximately Dock "F" on the north side of the east basin easterly to the southwest corner of the pier at the easterly marina entrance channel. Visual observation of the wall from that point easterly to the Anchor Marine Center was also conducted. Refer to Sketch 2 for a delineation of the area examined as well as the location and extent of each wall type. See also Photos 1 through 6.

1.2 Existing Structural System

Within the basin, the uplands soil is retained by a reinforced concrete quay wall at the upper 5 feet with reinforced concrete revetment panels over 15 feet of fill soil sloped at 1.5H:1V. Refer to Sketch 3 for a typical cross section. A continuous cast-in-place reinforced concrete thrust beam is located at the toe of the revetment panels to resist the tendency for the panels to slide down the slope. The panels, which are 10 feet wide with a 1 ½-inch gap between, are tied together by the thrust block. The quay wall itself appears to be a combination of cast-in-place and precast concrete elements. The footings are cast-in place concrete while the stems are precast panels. A concrete grouted fillet is situated between the two elements at the footing to stem.

The quay wall is a gravity wall system, meaning that it relies on its own weight, the weight of the soil over the heel, and the friction between the supporting soil and the footing to resist the

tendency of the wall to slide and overturn. Since no soil is provided over the toe of the wall, no passive pressure is utilized to resist the forces of the retained soil. This is a relatively unique structure by today's design standards. A 1 ½-inch gap as-designed between the toe of the wall and the revetment panels also precludes force transfer to the revetment panels. The gaps between revetment panels, gaps between quay wall stems, and weep holes in the revetment panels and quay wall stems provide a route for drainage of water from behind the wall during tidal fluctuations.

The quay wall extending from the basin around to the boat launch ramp is similar to the system in the basin except a stone rubble revetment was utilized in place of the concrete revetment panels. See Sketch 4. A reinforced concrete cantilever retaining wall occurs along the south side of the boat launch ramp and then continues on the north side of the launch ramp in an easterly direction to the approximate location of the proposed dry stack facility. See Sketch 5. The stone revetment system then continues.

1.3 Project Approach

To determine the best course of action for repair, rehabilitation, or replacement of the aging structure, a condition assessment and structural evaluation was performed. In addition, we understand that Leighton Consulting, Inc. performed subsurface borings and a preliminary geotechnical investigation in December 2002. The results of the geotechnical investigation are presented in Reference 1. Information contained in this report along with site investigation, record drawings, and other information provided by the BLUEWater Design Group was utilized in this study.

Included in this report are: a brief overview of the condition of the existing bulkhead, summary descriptions of each repair or replacement alternative, and a discussion of general replacement considerations. A summary of these items with respect to each alternative is summarized in Table 4.

2.0 Existing Conditions

Site investigations were performed by Mr. Tim Bazley of BLUEWater Design Group on October 26 and again with Mr. David Swanson and Mr. Doug Wilson on November 23, 24, and 25, 2003. The purpose of the investigations was to assess the general structural condition and performance of the existing bulkhead and review future bulkhead performance needs based on the Master Plan. The site visit included a visual examination of the bulkhead at low tide and discussions with the project geotechnical engineer. The site investigation did not include materials sampling or in-situ testing of the existing bulkhead structure.

2.1 Overall Structural Condition

The overall structural condition of the exposed portions of the quay wall and concrete revetment in the basin area appears to be in relatively good condition considering its approximate 35-year age and exposure to salt water, the elements, and tides. Select areas of the quay wall and the concrete revetment panels show signs of deterioration and distress, which will continue in the future. Since the rate of deterioration of reinforced concrete generally increases with time, the observed deterioration is expected to progress more quickly than it has historically. The overall alignment of the quay wall, the revetment panels, and riprap slope shows some movement but is not of a magnitude that suggests structural concerns. A brief visual examination of the quay wall and revetment panels in the other areas of the marina indicated similar deterioration and overall movement. Although beyond the scope of the current project, the issues and expected service life in the project area is also likely to be applicable to the rest of the marina walls. It is not anticipated that the existing 35-year-old quay wall system has a remaining design life of 35 to 50 years for the proposed development; therefore, we expect that some repair, retrofit, or replacement may be warranted.

Overall aerial settlement of 1 to 2 inches was observed throughout the site (Photos 7 and 8). This includes areas significantly beyond the wall, such as throughout parking areas and in concrete drainage swales. Based on the observed settlement throughout the site, settlement has likely occurred globally throughout the site rather than just within the local proximity of the wall. This is suspected because of the use of loose unconsolidated fill material that was subsequently flooded after construction (when the cofferdams were breached). There is also a low velocity flow of water during tidal fluctuations that may cause some movement of soil. It is not clear, based on field observations, when the settlement occurred over the life of the current facilities. There is evidence of grinding and concrete transition strips on the concrete sidewalks where differential movement has occurred. Additional research into the timeframe of these repairs may shed some light on whether this is a continuing maintenance issue or whether these repairs occurred early in the life of the marina.

2.2 Quay Wall Condition

The quay wall shows signs of its age. Observed items are described below. Referenced photographs can be found in Appendix B.

1. There is relative movement between the precast panels (Photos 9 and 10). The as-built drawings indicate that a ¾-inch gap was provided between each of the 20-foot-long precast panels. Joints in the cast-in-place footing did not include gaps. Currently, there are variable gaps in the panel joints without an associated change in the footings. It is possible that some of the variation occurred during construction due to tolerances and the need to match finish grades; however, some is likely due to movement of the wall system over time.
2. Both the footing and revetment panel corners are cracked where bearing contact has been made (Photo 11). It appears that movement of the wall and panels has caused localized

damage to the panels in some locations. This damage does not result in a global reduction in capacity; however, deterioration of the wall system in these localized areas will be accelerated.

3. There is spalling of concrete on the wall face (Photos 12 and 13). This appears to be the result of rusting of reinforcing steel in the wall. As the reinforcement oxidizes, its volume increases, which in turn places tensile stresses in the concrete cover. These stresses are relieved through spalling of the concrete. This condition is expected to accelerate over time. There is some evidence of previous attempts to patch these areas, particularly at the areas where the wall section is supported by the stone revetment. These efforts have had limited success as much of the patch material has subsequently spalled off (Photo 14).
4. Voids are visible on the surface of the concrete stem wall and the larger aggregate is visible (Photos 15 and 16). This is indicative of a loss of the concrete paste and small aggregate (fines) that occurs on the surface of a concrete object. The loss of fines may be the result of poorly consolidated areas that were sacked during construction and the slurry has slowly been worn away. These areas may be at an increased risk for concrete corrosion and subsequent spalling.
5. Surface rust stains exist on top of the toe of the concrete footing (Photos 17 and 18). Based on limited observation of areas where this has led to spalling, the rusting appears to be the result of aggregate or other items in the concrete oxidizing rather than deterioration of the reinforcing steel in the wall.
6. Tree root systems are causing uplift of the sidewalk, pavements, and curbs of several inches throughout the facility (Photos 19 and 20). This is more of a serviceability problem than a structural issue.
7. Some of the joints between revetment panels and between the panels and the toe of the wall are partially grouted (Photo 21). Note that grouting is not shown on the record drawings.
8. There are many areas along the sidewalk adjacent to the bulkhead where significant differential settlement across sidewalk joints has resulted in raised vertical edges causing potential tripping hazard to pedestrians. At these locations there has been significant grinding of the raised portion of the concrete joint to smooth out the joint and remove the tripping hazards.
9. Elastomeric sealants have been installed at select locations along the sidewalk where the sidewalk is lower than the stem wall to close the gap that has formed due to backfill settlement and stem the inflow of surface drain water.
10. Concrete is spalling along the edge of the footing, particularly at the riprap slope (Photos 22 through 25). The spalled areas are up to 2-3 feet long by 4-6 inches deep. There are no signs of reinforcement, although the record drawings indicate that reinforcement should be observed. The degradation appears to be more an issue of poor construction quality than from excessive loading. There are early signs of similar deterioration in the basin area in the vicinity to the boat launch ramp.

11. The riprap has settled under the quay wall up to 6-10 inches in some areas (Photos 26 and 27). The riprap continues to be unstable and raises concerns as to the stability of the quay wall it is supporting.
12. The top of the quay wall is fractured in one location (Photo 28). This may be the result of differential expansion and contraction of the guardrail, a localized overload of the guardrail, or a localized defect in the wall.

2.3 Concrete Slope Revetment Panel Condition

Dislodged panels were observed in a number of locations (Photos 29 through 31). We understand that for the basin area with a design depth of -10 MLLW, three concrete slope panels were installed in the dry. There were various slope failures during the construction phase. If the resulting void was larger than 2 feet by 2 feet by 2 feet then the panel was removed, the void filled and the panel replaced. If the void was smaller, the void was backfilled in place without removing the panels. In the 1970s there were additional panel failures. At that time the failed panels were cored and mud jacked back into place. Weep holes were installed in the failed panels as can be seen today. Per the original design, each joint was to be protected with an 18-inch-wide strip of filter fabric to protect against the migration of material from behind the panels. In many locations the filter fabric has either deteriorated, or has been washed away, allowing for the potential continuing loss of material from behind the slope panels. There is concern that these voids are returning or expanding today and this is contributing to the cause of the movement of the panels exhibited in the photos. However, based on annual survey data from the County of Orange, minimal movement has occurred since 1991 when data collection began. The geotechnical engineer indicates that the likely geotechnical mechanism was migration of the fines from the backfill through tidal fluctuations. Attempts to repair or replace sections of the concrete revetment could be difficult. Removal of panel sections could result in sloughing of soil from behind the panels, particularly in the intertidal zone. Additional field investigation including possibly coring select panels and probing for voids is recommended as part of future work.

2.4 Riprap Revetment Condition

The riprap stone revetment shows variable signs of settlement and movement since its construction. (Photo 32) We observed void areas of the stone revetment under the cast-in-place concrete quay wall footing. Since the footing for the quay wall was cast-in-place concrete, the top layers of stones were partially cast into the footing. This cast-in-place footing in connection with the stem wall is capable of acting as a deep beam section to bridge over voids caused by the settlement of the stone rubble. The extent of bearing is difficult to ascertain in the field but has a major impact on the potential performance of the wall. Reduced bearing of the quay wall footing could result in uneven bearing and accelerated settlement as well as reduced friction with which to resist sliding forces for both gravity and seismic loads.

2.5 Cantilever Retaining Wall Condition

The cantilever wall section adjacent to the boat launch ramp appears to be in good condition with limited areas of visual degradation at the face of the concrete wall. The as-built drawings indicate that the walls were specified to have been constructed with a batter, but the tallest section of wall to the south of the boat launch appears to be vertical or out of plumb in areas, suggesting some wall deflection over time. We would expect this type of behavior for a cantilever wall of this size constructed in an area of loosely filled soil.

3.0 Structural Analysis

To better understand the capability of the existing wall systems, the existing design was evaluated based on the design values provided in the current project geotechnical report. Note that the design values used may be greater than the original design values so an "inadequate" factor of safety does not necessarily indicate a problem with the original design. The walls were checked for global sliding and overturning stability as well as whether the allowable soil bearing pressure was exceeded. The wall stem and footing were both checked for adequate structural capacity.

The factor of safety is the ratio of the calculated capacity to the calculated demand. Typically a factor of safety of 1.5 is considered adequate for retaining wall sliding and overturning mechanisms. With respect to bearing pressure and the structural capacity of the wall elements, an element is considered to be overstressed if the demand is greater than the capacity. In these cases, an overstress condition does not necessarily indicate that a failure will occur, since factors of safety can also be implicitly included in other aspects of the design.

3.1 5-foot Tall Quay Wall

The cross-section evaluated is shown in Sketches 6 and 8. This cross-section occurs over both compacted fill and stone riprap. Currently, there is no differentiation of the geotechnical values for these two support conditions. Future evaluation should include additional investigation of these geotechnical design values.

Table 1 – 5-foot Tall Quay Wall Type Structural Evaluation Parameters

Factor of Safety / Overstress	Gravity	Seismic
Sliding	1.0 ≤ 1.5 (No Good)	0.7 ≤ 1.5(No Good)
Overturning	4.0 ≥ 1.5 OK	2.0 ≥ 1.5 OK
Bearing Pressure	OK	OK
Stem Bending	OK	OK
Footing Bending	OK	OK

3.2 9-foot Tall Quay Wall

The cross-section of the 9-foot tall quay wall is shown in Sketch 7. This cross-section occurs over the revetted slope in the dry boat storage launch area. Future evaluation should include additional investigation of these geotechnical design values to ensure that they are adequate for the stone riprap.

Table 2 – 9-foot Tall Quay Wall Type Structural Evaluation Parameters

Factor of Safety / Overstress	Gravity	Seismic
Sliding	1.3 ≤ 1.5 (No Good)	0.7 ≤ 1.5 (No Good)
Overturning	5.4 ≥ 1.5 OK	2.1 ≥ 1.5 OK
Bearing Pressure	OK	OK
Stem Bending	OK	20% Overstressed
Footing Bending	OK	OK

3.3 Concrete Cantilever Wall

The cross-section of the cantilever retaining wall is shown in Sketch 9. The dimensions and reinforcement vary depending on the height of the wall. The current analysis is based on the cross section at the maximum height.

Table 3 – Concrete Cantilever Wall Type Structural Evaluation Parameters

Factor of Safety / Overstress	Gravity	Seismic
Sliding	1.5 ≥ 1.5 OK	0.68 ≤ 1.5 (No Good)
Overturning	4.9 ≥ 1.5 OK	1.5 ≥ 1.5 OK
Bearing Pressure	90% Overstressed	230% Overstressed
Stem Bending @ Base	OK	110% Overstressed
Stem Bending @ End of Taper	OK	85% Overstressed
Footing Bending	OK	60% Overstressed

4.0 Identified Structural Issues

4.1 Basic Issues for the Quay and Retaining Walls

The following are issues that should be addressed in any plan to extend the life of the wall:

1. Lateral Stability. The lateral stability of the quay wall, particularly with respect to sliding, under seismic and vertical surcharge loads is marginal.

2. Soil Bearing Pressure. The cantilever retaining wall imposes excessive bearing pressures on the supporting subgrade material. This could lead to excessive deformation and possible global stability failure.

3. Structural Capacity. Some elements of the walls are overstressed under load. This is a particular issue with the cantilever retaining wall under seismic loading.

4. Condition of Concrete Structure in the Marine Environment. The marine environment can be harsh on structures. The existing wall shows signs of deterioration, which will accelerate over time.

5. Future Configuration of Walkway. The redevelopment offers the opportunity to enhance the aesthetic and functional features of the walkway. These could include bump outs or a curvilinear plan to allow pedestrians to stop along the walkway without obstructing the flow of others. Architectural features, such as a relief or design, could also be incorporated into both the horizontal and vertical concrete elements.

6. Overall Performance for Vertical Settlement. Vertical settlement appears to be a primary cause of maintenance efforts at the facility. Accommodation of future settlement should be considered.

4.2 Slope/Revetment Issues

The following are issues that should be addressed in any plan to extend the life of the wall with respect to the revetment slope:

1. Movement of Concrete Revetment Panels. Repair or replacement may cause more harm but based on the historical issues surrounding voids and panel movement, additional field investigation is recommended.
2. Stability During a Seismic Event. There is a potential for the entire slope to become unstable during a seismic event. We strongly recommend that this issue be evaluated further due to its potential implications. Repair, rehabilitation, or replacement of the quay wall could be of limited value if a larger global failure of the entire slope occurs. The likelihood and magnitude of such an event should also be evaluated relative to the uplands and marina uses.

5.0 Bulkhead Alternatives

5.1 General Considerations

The purpose of the current quay wall and revetment slope is to retain the earth on the upland side and provide a transition from the uplands area to the marina. There are two conditions that should be considered with respect to retaining the upland soil. Lateral load is imposed by the wedge of soil being retained as well as surcharge from transient or live loads on the surface (such as vehicles). During a design level earthquake, there is also the potential lateral surcharge

loading condition caused by the inertial mass of the upland soil acting against the free face of the retaining wall.

5.1.1 Seismic Considerations

The behavior of the site soils during an earthquake was evaluated by Leighton Consulting as part of their initial geotechnical investigation. There are two items of concern with respect to soil behavior during seismic events. An additional lateral load is placed on the retaining wall by the inertial mass of the retained soil. Another concern during an earthquake is the potential liquefaction of the soil. When liquefaction occurs, the soil acts like a liquid and it looses its ability to laterally support itself. The result is a large increase in lateral pressures and loss of vertical and lateral support for bulkheads, piling, and deadman anchors. This causes vertical settlement at the ground surface and lateral spread of the soils at the free face of the bulkhead structure. Global stability under seismic ground shaking of both the revetment slope and the quay wall soil-structure system is another concern for this type of waterfront facility.

There is some flexibility in the selection of seismic design criteria for the design of the bulkhead rehabilitation. Design to the California Building Code magnitude seismic event may neither be necessary nor required for a waterfront bulkhead if the bulkhead's seismic performance does not impact occupied structures. The stated intent of the building code is to provide life-safety for occupied structures. Structurally, this entails providing a structure that does not collapse onto occupants, and following a seismic event, remains standing long enough to allow occupants of the structure to exit safely. Since the anticipated vertical and lateral displacements, assuming the wall is ineffective, will likely meet this life-safety performance objective, it may not be necessary to design the bulkhead to resist this level of seismic event. However, the areas where more stringent seismic design criteria and considerations may be warranted would be at locations where buildings are in close proximity to the bulkhead. Depending on the potential seismic failure mechanism under consideration, recommended building setback distances from the quay wall could range from 2 to 3 times the height of the bulkhead wall (10 to 20 feet) for localized liquefaction and lateral spreading failure to several times the height of the revetment slope and bulkhead system for global seismic instability (20 to 60 feet). These parameters should be considered during the master planning and conceptual design phase of the project and investigated more fully during design development to determine their potential impact. Better seismic performance of the quay wall in these areas will help the buildings they support also meet the life-safety performance objective.

From a service perspective, new development on the uplands portion of the marina will likely be pile supported in order to provide adequate resistance to long-term settlement. These pile-supported structures will likely be less susceptible to damage or reduced performance caused by the anticipated bulkhead displacements. Therefore, depending on the anticipated development, additional bulkhead seismic performance improvements may not be required. This should be further investigated during the design development phase of the project.

5.1.2 Construction Impact Considerations

One of the challenges that the design team and contractor will face is keeping the marina operational during quay wall repairs or upgrades. On the upland side, there will be some impact on parking, but the more complex issue will be the impact on existing facilities in close

proximity to the bulkhead. Since the uplands portion of the marina provides landside access to the marina, temporary access will need to be provided throughout the docks during the construction period. This may necessitate limiting construction activity to select segments of the bulkhead at a time. Structural repair or upgrade options that allow construction to occur from the uplands side will generally be less costly and will likely have less impact on marina operations.

The determination of land side versus water side construction may be partially driven by the contractor's ability to gain access from the uplands side and in timing the repairs with the landside construction. In particular, care must be exercised to not overload the bulkhead during construction operations. Cranes and pile driving equipment can place high loads on adjacent retaining walls. Keeping the equipment back from the edge can minimize the impact, but this requires larger and more costly equipment in order to accommodate the longer reach required.

5.2 Concept-Level Design Alternatives

A number of alternatives were evaluated for repair, rehabilitation, and replacement of the existing bulkhead. The basic alternatives are to maintain the status quo, repair the quay wall, or replace it. The concepts are also summarized in Table 4.

5.2.1 Basis of Design

Conceptual design of the various alternatives was performed to allow for development of a budgetary-level opinion of probable cost. The following criteria were used in the design effort:

1. A uniform 250-psf live load surcharge was considered on the uplands side of the bulkhead to account for vehicular traffic. If traffic was limited to pedestrians only, the design live surcharge load could be reduced to 100 psf. The project's geotechnical engineer can provide refined surcharge design criteria based on proposed site use.
2. The static lateral earth pressures provided in the geotechnical report were utilized for design of the bulkheads. The active pressure is a triangular-shaped pressure of approximately 38H (psf). The "H" refers to the distance from the surface. A uniform lateral surcharge of approximately 85 psf is imposed to reflect the lateral pressures from the 250-psf live load on the surface. An active pressure in an inverted triangular-shape of 60H (psf) was used to reflect the lateral pressure during a design seismic event.
3. A minimum factor of safety of 1.5 for static loads was utilized for global sliding and overturning mechanisms.
4. Earth pressures and expected impacts to the quay wall and revetment structure from soil liquefaction was neglected for this structural evaluation. Future phases of design should examine the influence of liquefaction and global stability in more detail.

5.2.2 Opinion of Probable Construction Cost

A planning level opinion of probable construction cost is included with the discussion of the various alternatives. These costs reflect the bare structural costs and do not necessarily reflect premiums that may be required due to tidal work, utility conflicts, phasing requirements, etc. Please note that these estimates are very preliminary and should only be used for establishing an

overall budget for the element of the project. More refined cost estimates can be prepared as the design is developed further.

5.2.3 Alternative 1 – Status Quo

This design concept reflects the lowest level of repair proposed. Areas of localized damage could be patched in an attempt to increase the service life of the system. Typical costs for such work are between $20 and $50 per square foot of repair area. This repair does not address any structural performance issues for either static or seismic loading.

5.2.4 Alternative 2 – Repair

This design concept increases the service life and expected performance of the system without removing the existing quay wall. Refer to Sketch 10. Possible features include:

1. Add a new 4- to 6-inch-thick reinforced concrete facing to the exposed side of the quay wall. This provides a new surface where the wall is visible and when tied to the existing will act compositely to increase the strength of the wall. It addresses the concrete wall face spalling and reinforcing steel corrosion on the wall face and top of footing at a cost of approximately $20 to $30 per square foot.
2. Install a deadman anchor with a tie rod to increase the sliding and overturning resistance of the wall system. Due to the uplands improvements including the addition of a sanitary sewer line, the area where the deadman would be added will likely already be excavated. The anchor system will add approximately $400 to $600 per linear foot of wall.
3. Install a structural walkway slab to replace the existing deteriorated and worn concrete walkway. This structural slab could be configured to undulate along the length of the quay wall to provide a more aesthetically pleasing marina design feature and locations for pedestrians to stop and mingle without impeding the movement of pedestrians along the main walkway. A structural walkway slab has the benefit of having the ability to bridge voids that may occur in the underlying fill material and help avoid the long-term settlement in the pavements that have plagued the marina for the last 35 years. A structural slab will add approximately $20 to $30 per square foot to the project cost.

The above repair is not feasible in the area of the stone revetment slope where the existing upland facilities are to remain. In that area a less invasive repair would be needed. See Sketch 11. Possible features in the area of the stone revetment include:

1. Injection grouting under the footing to improve bearing at a cost of about $80 to $100 per linear foot of wall.
2. Addition of riprap over the toe of the wall. This provides an increased measure of stability. The cost is approximately $30 to $50 per linear foot of wall.

Note that these measures may not extend the life of the wall by 30 to 50 years without interim maintenance. Additionally, significant wall deformations should be expected for a large earthquake due to liquefaction and lateral spreading of site soils. However, these measures help improve the existing condition and help minimize risk of accelerated wall deterioration. When those structures are redeveloped, a longer-term solution can be considered.

5.2.5 Alternative 3 – Replacement

This design concept considers replacement of the existing quay wall. The primary benefit of such a solution is the ability to provide a more robust design solution with respect to the expected design forces and global slope seismic stability.

One option is to eliminate the wall segment altogether and replace it with a pier-like structure. See Sketch 12. The concrete revetment panel could be extended to the underside of the deck to provide slope stabilization and protection. The primary benefit of this option is the ability to pile support the deck near the water. This provides the greatest degree of protection in the event of liquefaction even if there is a slope stability failure. It is also likely the most costly option with a cost ranging from $1,000 to $3,000 per linear foot depending on the final configuration.

Another replacement alternative could utilize passive pressure on the toe of the wall to better resist sliding and overturning forces. Refer to Sketch 13. The structural system is similar to the current system, but the embedded section of plank provides additional lateral resistance, provided that the soil is effective even during a seismic event. Depending on the final configuration, such a system could cost between $1,000 and $3,000 per linear foot. The primary drawback to both replacement concepts is the difficulty in constructing the new wall within the intertidal zone.

Table 4 – Concept-Level Design Alternatives Comparison

Alternative Comparison					
	Alternative 1 Status Quo	Alternative 2 In-situ Repair		Alternative 3 Replacement	
Description	Status Quo	Repair – Facing and Deadman	Repair – Patching and Riprap @ Toe	Alternative 3A Pier Replacement	Alternative 3B Wall Replacement
Sketch Number	N/A	SK-10	SK-11	SK-12	SK-13
Cost	Low	Moderate	Low-Moderate	High	High
Service Life	Short	Moderate	Short - Moderate	Long	Long
Maintenance Cost	Moderate-High	Low-Moderate	High	Low	Low
Construction Risk	Low	Low	Low-Moderate	High	High
Construction Impact on Utilities	Low	Low-Moderate	Low	High	High
Construction Impact on Uplands	Low	Low-Moderate	Low	High	High
Construction Impact on Marina	Low	Moderate	Low-Moderate	High	High
Aesthetic Value	Low	Moderate	Low	High	High

5.2.6 Discussion of Comparison Criteria

The criteria contained in Table 4 above are intended to provide a comparison of different aspects of each alternative. Further discussion of each criterion is provided below:

1. Cost: The planning level opinion of probable cost is based on the planning level conceptual level of design. Please note that this estimate is very preliminary and should only be used for establishing an overall budget for the element of the project. More refined cost estimates can be prepared as the design is developed further.

2. Service Life: The service life of the bulkhead is largely a function of the materials used. Patchwork repair of the existing structure has the shortest service life due to its age and susceptibility to deterioration. Concrete structures have the longest service life when properly detailed and constructed.

3. Maintenance Cost: Recurring maintenance costs will be highest for the temporary repairs. The repair alternative will have significantly longer life but it still utilizes a structure that is over 30 years old. All new construction has the potentially lowest recurring maintenance cost.

4. Construction Risk: Increased risk for the contractor in the form of potential subsurface obstructions, utility interferences, and other conflicts will be reflected in higher bid prices.

5. Construction Impact on Utilities: Utilities are located below the existing walkway on the upland side of the bulkhead. All alternatives will have some impact on utilities. Alternatives with work landward of the bulkhead have the highest impact.

6. Construction Impact on Uplands: The construction impact on the uplands side of the marina is a function of the expected extent of work to be performed in conjunction with the uplands area.

7. Construction Impact on Marina: The construction impact on the marina is a function of the expected extent of work to be performed from the water as well as how many access gangways will be unavailable at any given time. The impacts will tend to be localized to a few slips on each dock closest to the bulkhead along the length of the bulkhead that is under construction.

8. Aesthetics: Visual presentation of the finished project will have long-lasting impressions on the users of the marina and walkway area. Cantilevering the new walkway over the wall can hide the wall from above. Providing a curvilinear contour along the length of the bulkhead can break up a less visually appealing straight-line bulkhead. Bump outs may also be considered in order to break up the look of a long, straight bulkhead. Concrete can be provided with etchings or embedded items to provide interest.

6.0 Recommendations for Future Work

Prior to finalizing the design solution, additional investigation and analysis should be undertaken.

1. Verify that voids do not exist behind the displaced revetment panels.
2. Investigate potential for liquefaction.
3. Investigate stability of slope.
4. Finalize design level earthquake and performance objectives.
5. Determine whether the wall and revetment slope are needed for lateral support of the proposed uplands development.
6. Evaluate expected vertical settlement and lateral spread for comparison to acceptable limits.

7.0 Conclusion and Recommendations

The quay wall and revetment are in reasonably good condition considering their 35 plus year age. The system is currently structurally adequate under dead load. The system is under designed for the additional loads due to surcharge live loads, such as from traffic, as well as seismic loading. Due to these deficiencies, the expected continued degradation of the existing wall, and the magnitude of uplands development, repair or replacement is recommended. Estimated costs for repair range from $200 to $1,000 per linear foot. Estimated costs for replacement may be as low as $1,000 per linear foot to as much as $3,000 per linear foot. Prior to establishing the final solution, additional investigation and evaluation is recommended.

sjs\24wf\03\018\report\dana point report.doc\drw

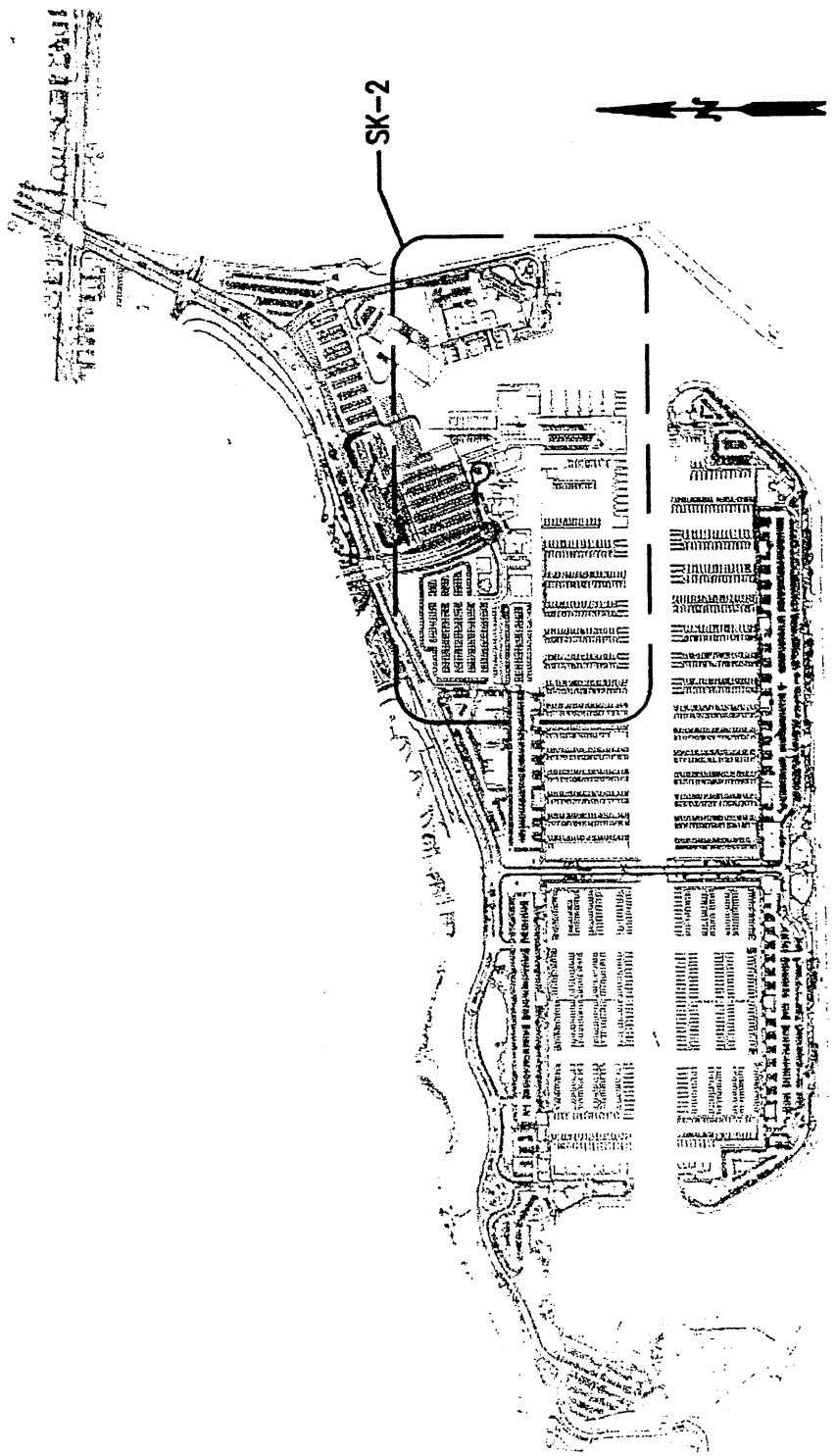
References

Reference 1: *Preliminary Geotechnical Report for the Dana Point Harbor Master Plan*, Leighton Consulting, Inc., December 12, 2002.

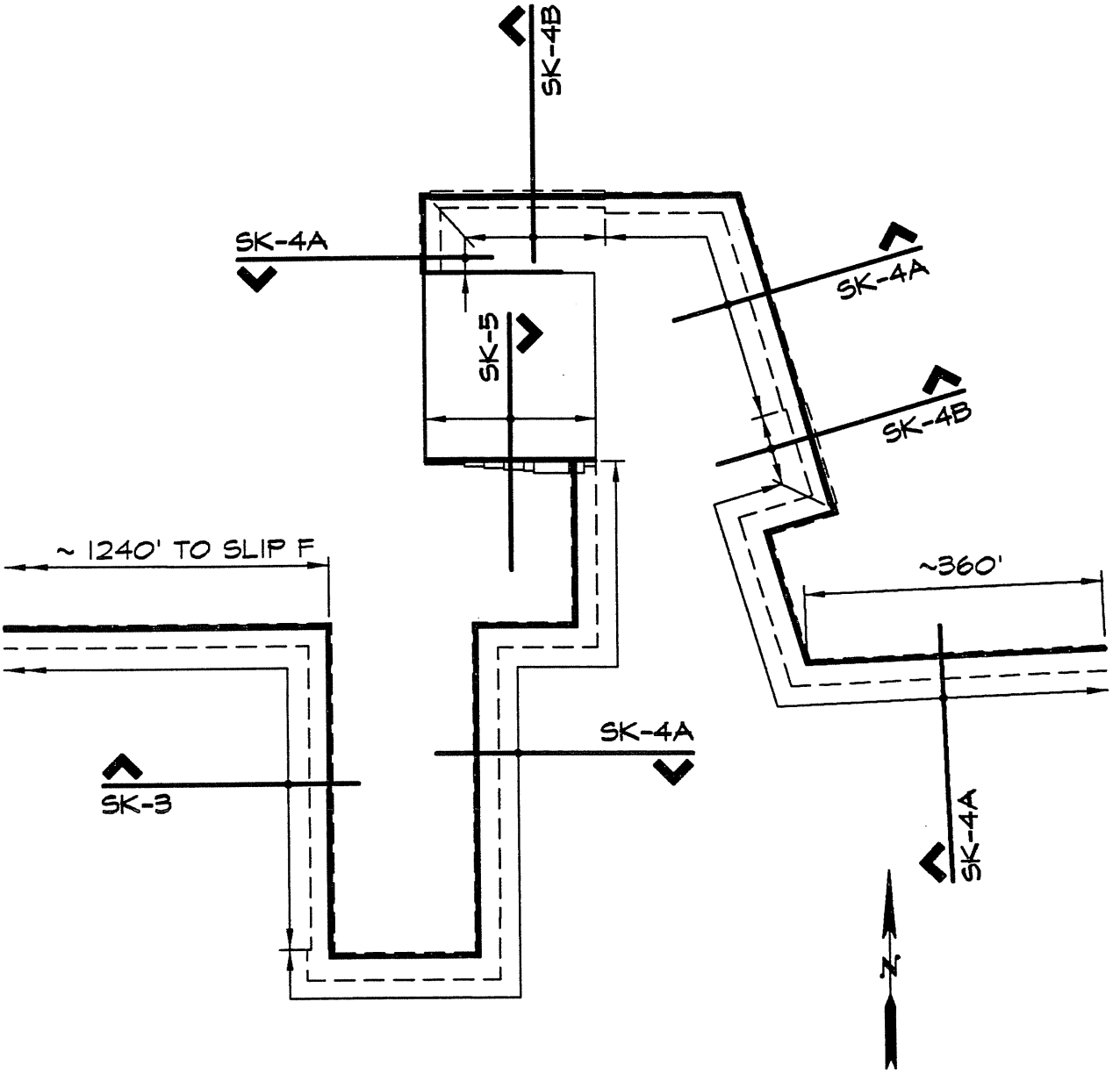
APPENDICES

APPENDIX A

Sketches



	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES. DRW	SCALE: NTS	SHEET NO.
		DR. GSV	DATE: 12-17-09	SK-1
	SITE PLAN	CHK. DBS	FILE NO.: SK-1.dwg	1 OF 13 SHEETS



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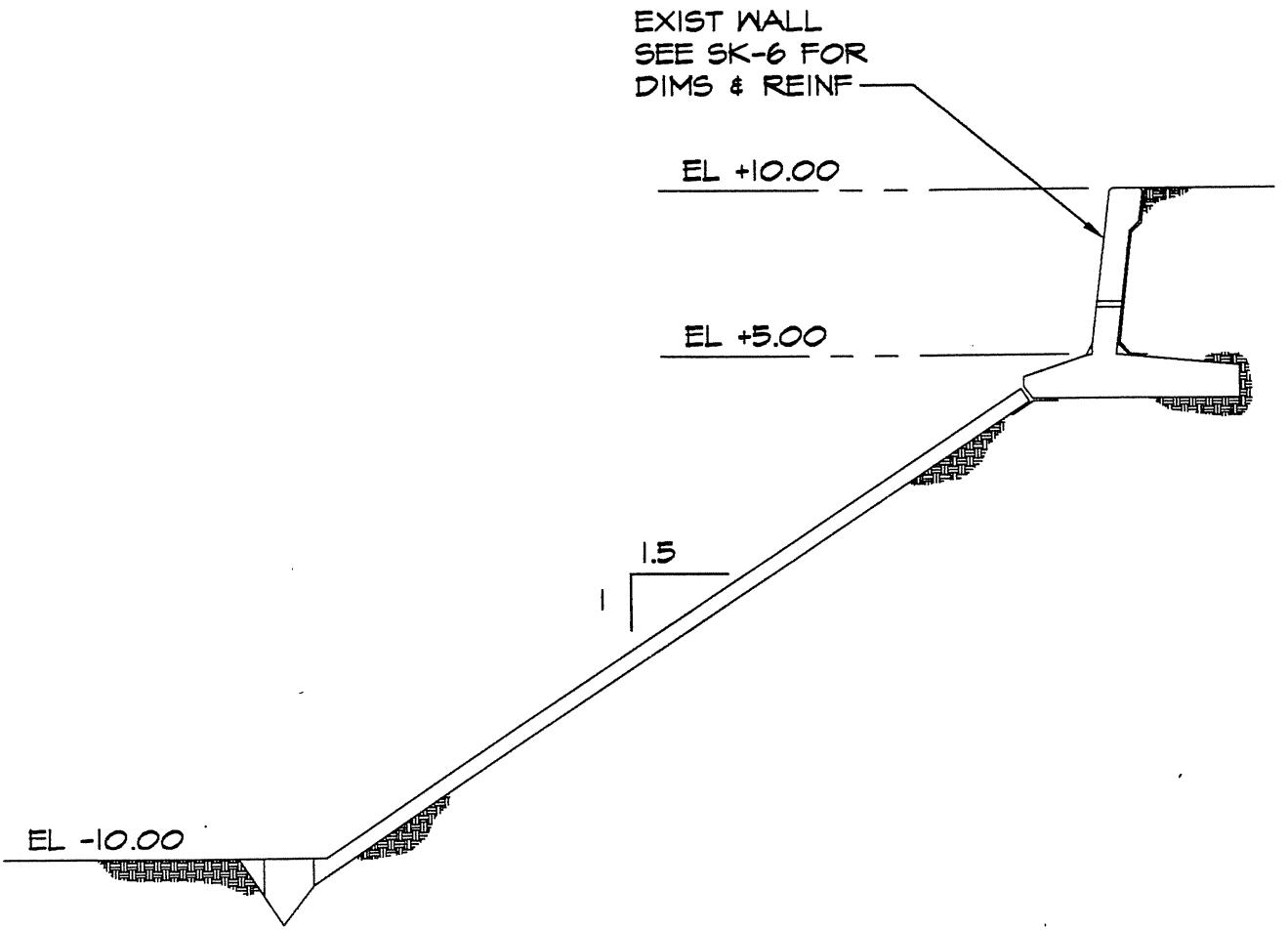
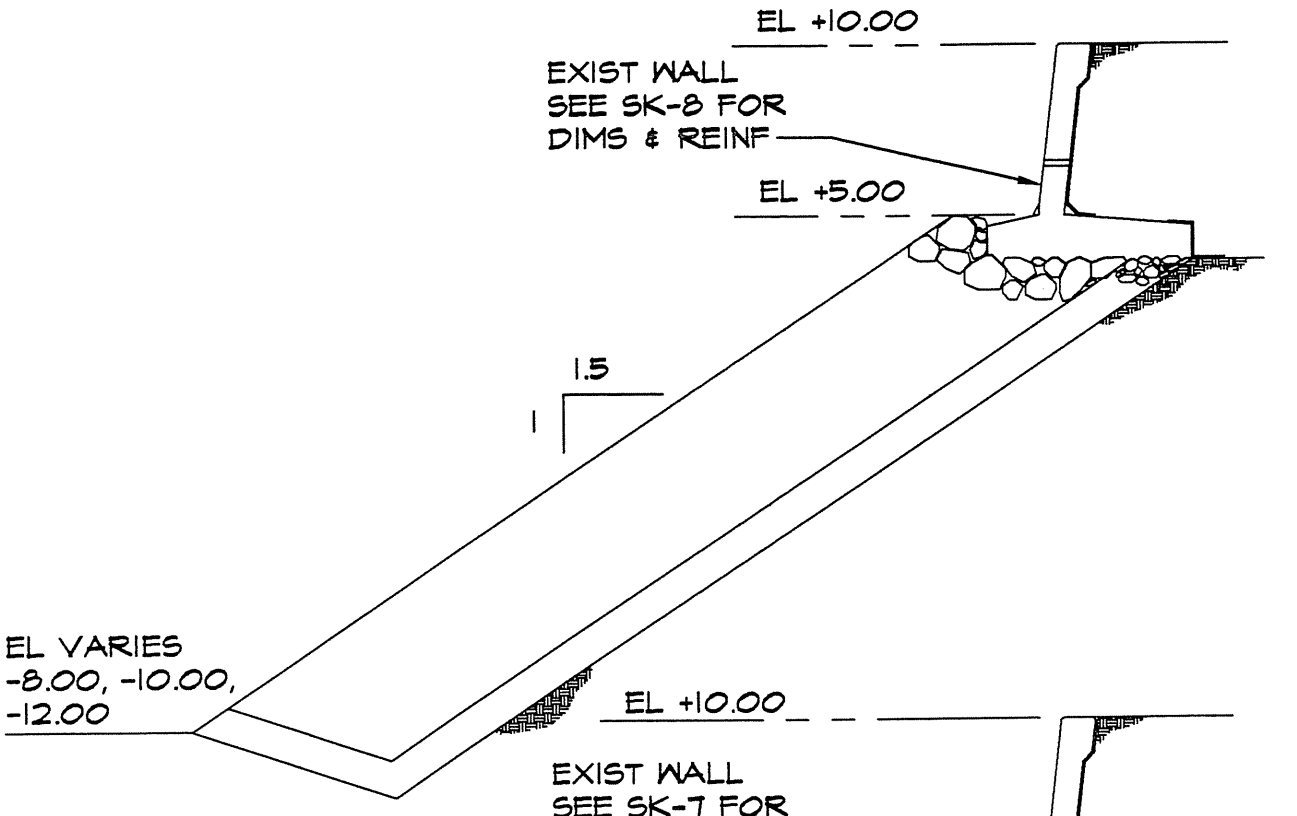
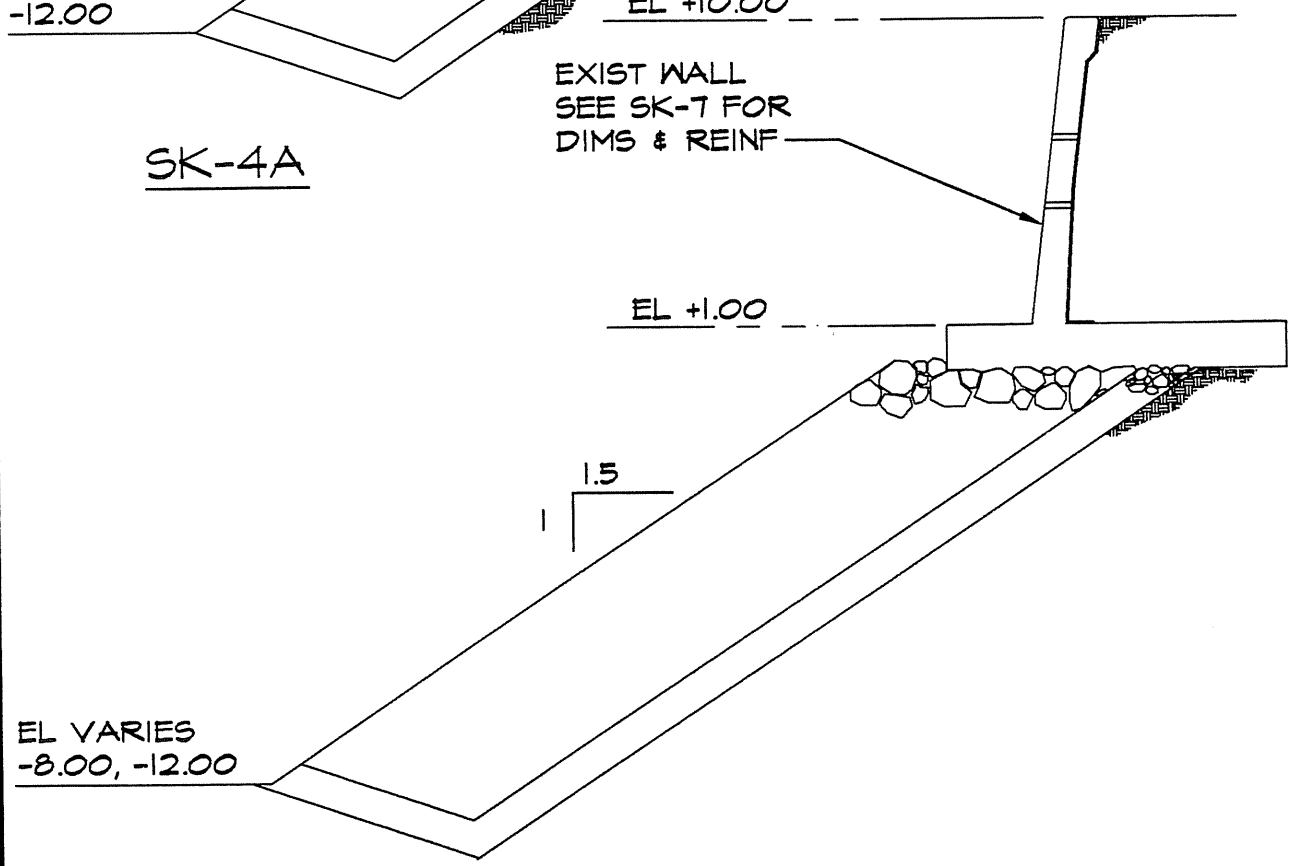
		DR.: GSV	DATE: 12-17-03	SK-2
	PLAN DELINEATION OF WALL TYPES	CHK.: DBS	FILE NO.: SK-2.dwg	2 OF 13 SHEETS



	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRW	SCALE: 3/16"=1'-0"	SHEET NO.
		DR.: GSV	DATE: 12-17-03	SK-3
	TYPICAL CROSS SECTION AT CONCRETE REVETMENT	CHK.: DBS	FILE NO.: SK-3.dwg	3 OF 13 SHEETS



SK-4A



SK-4B

	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRW	SCALE: 3/16"=1'-0"	SHEET NO.
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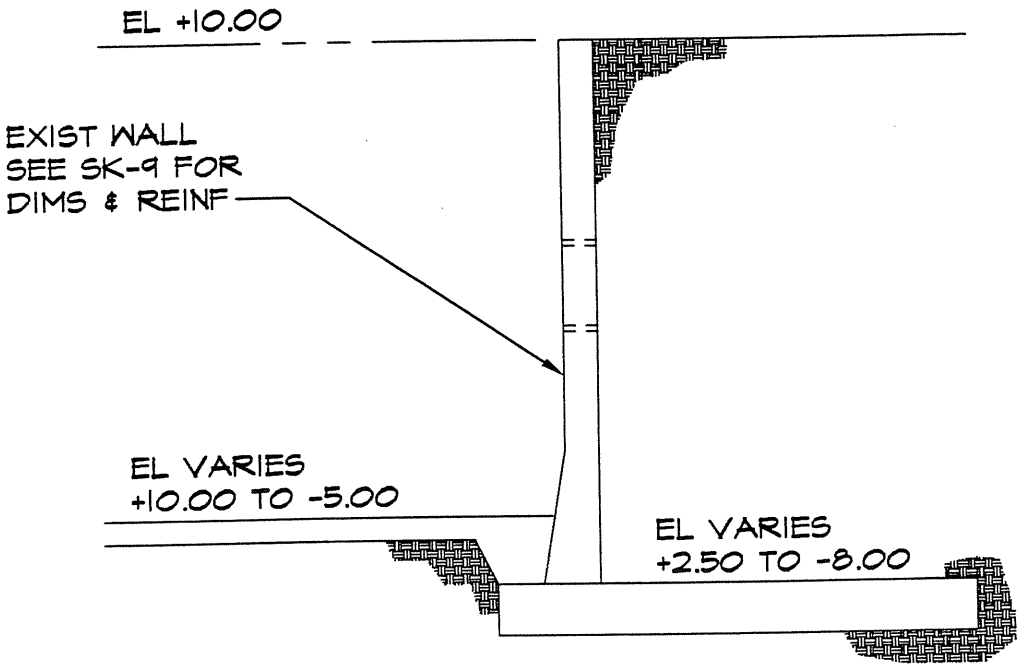
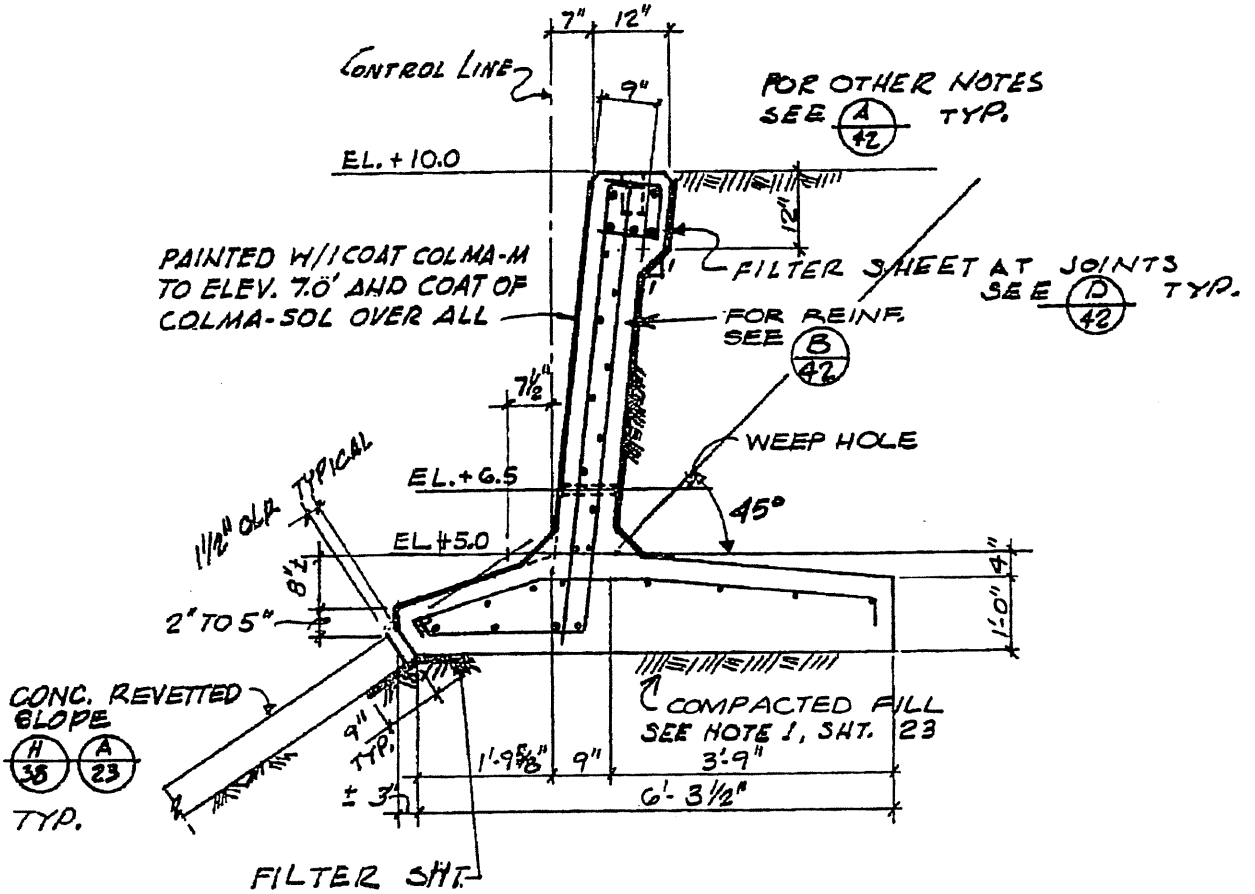
	TYPICAL CROSS SECTION AT STONE REVETMENT	CHK.: DBS	FILE NO.: SK-4.dwg	4 OF 13 SHEETS



BULKHEAD STRUCTURAL EVALUATION
DANA POINT MARINA REDEVELOPMENT

TYPICAL CROSS SECTION
AT CONCRETE CANTILEVER WALL

DES.: DRW	SCALE: 3/16"=1'-0"	SHEET NO.	
DR.: GSV	DATE: 12-17-03	SK-5	
CHK.: DBS	FILE NO.: SK-5.dwg	5 OF 13 SHEETS	



SECTION C/42

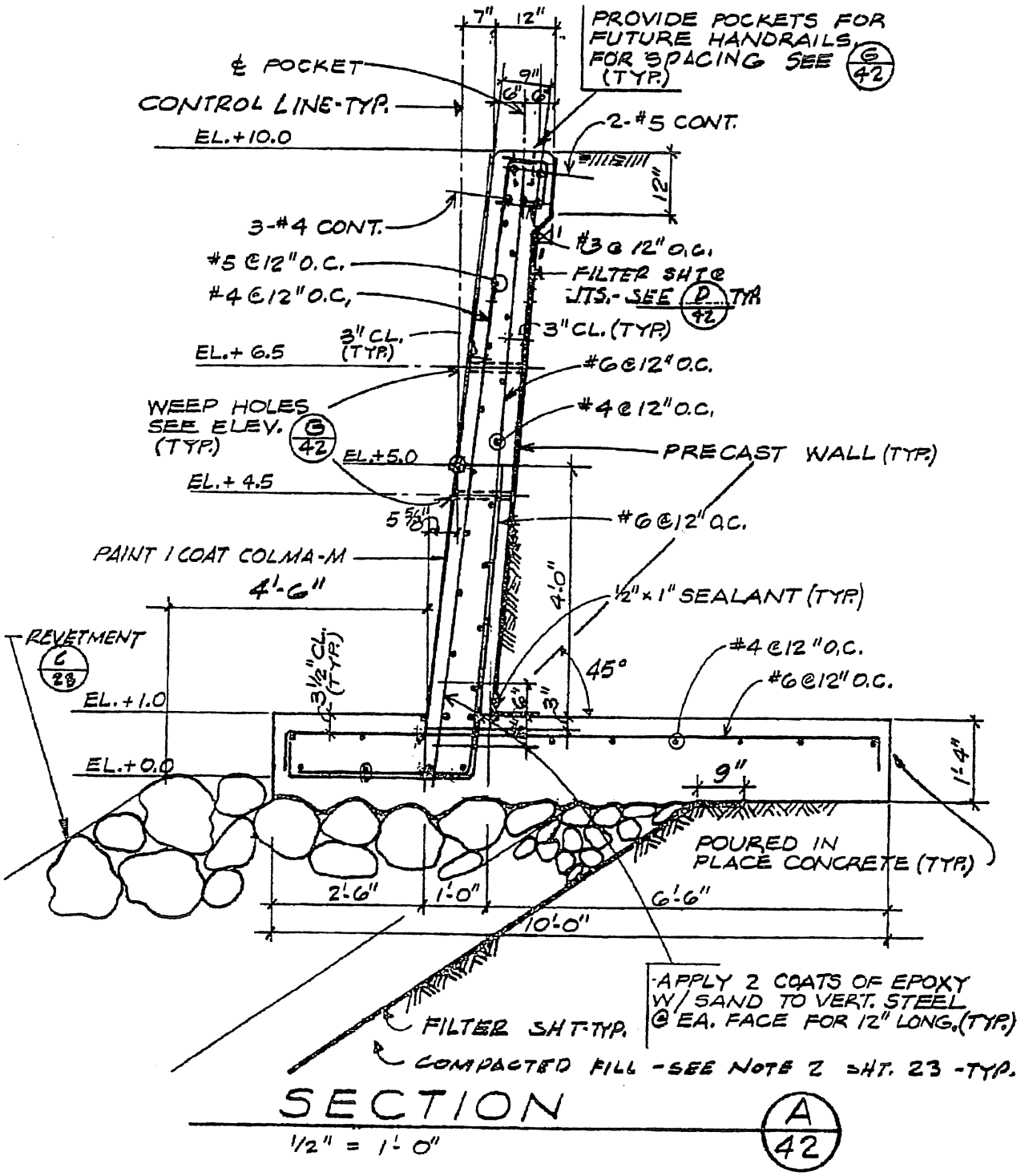
1/2" = 1'-0"

NOTES:

1. DETAIL FROM DANA POINT HARBOR AS BUILTS PREPARED BY KOEBIG & KOEGIG, INC 1968, SHEET 42 OF 50, D10.5-1.

2. DETAIL IS NOT TO SCALE.

	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRW	SCALE: NTS	SHEET NO.
		DR.: GSV	DATE: 12-17-03	SK-6
	EXISTING WALL SECTION	CHK.: DBS	FILE NO.: SK-6.dwg	6 OF 13 SHEETS



SECTION A/42

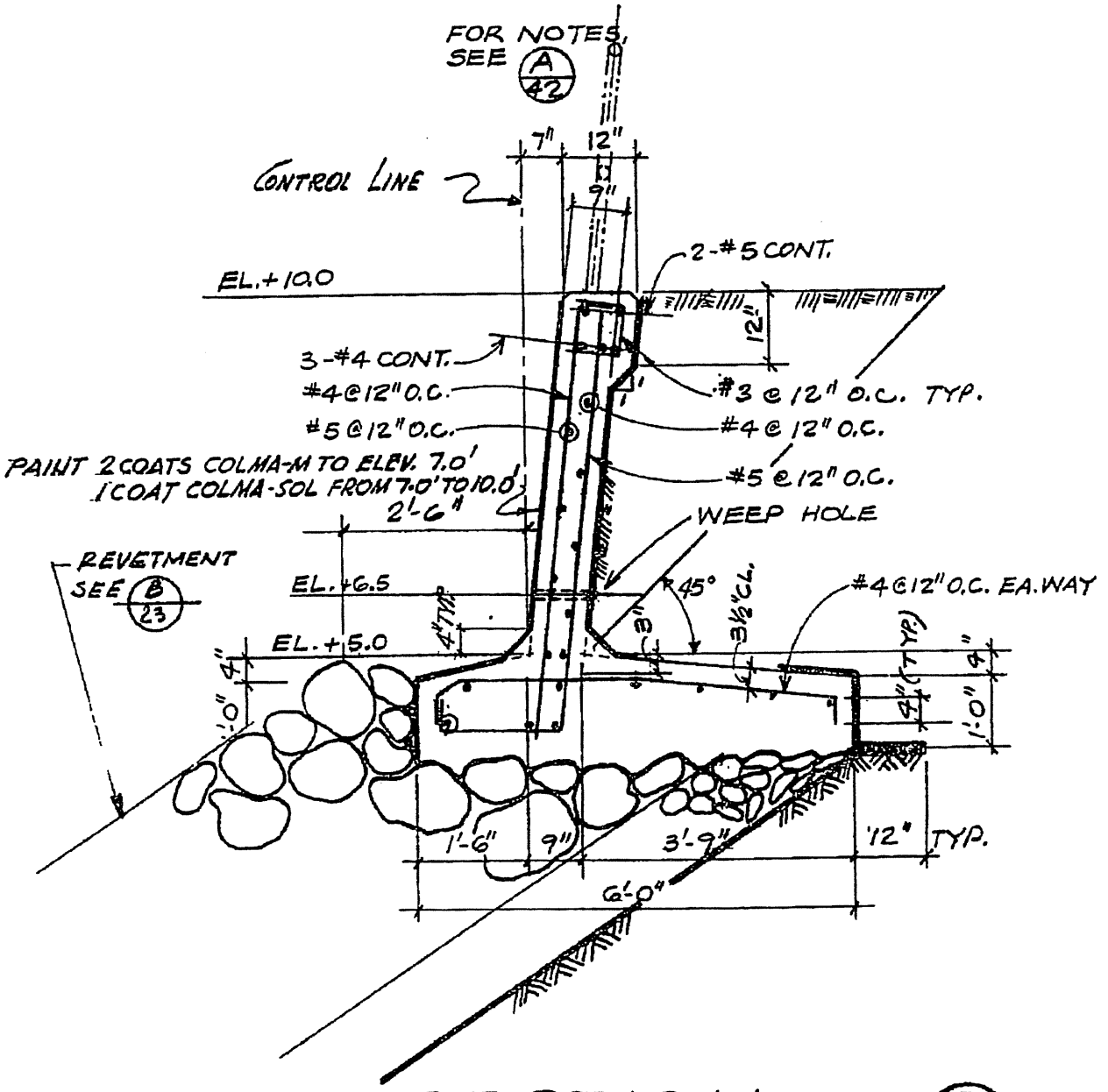
1/2" = 1'-0"

NOTES:

1. DETAIL FROM DANA POINT HARBOR AS BUILTS PREPARED BY KOEBIG & KOEGIG, INC 1968, SHEET 42 OF 50, D10.5-1.

2. DETAIL IS NOT TO SCALE.

	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRW	SCALE: NTS	SHEET NO.
		DR.: GSV	DATE: 12-17-03	SK-7
	EXISTING WALL SECTION	CHK.: DBS	FILE NO.: SK-7.dwg	7 OF 13 SHEETS



SECTION B/42

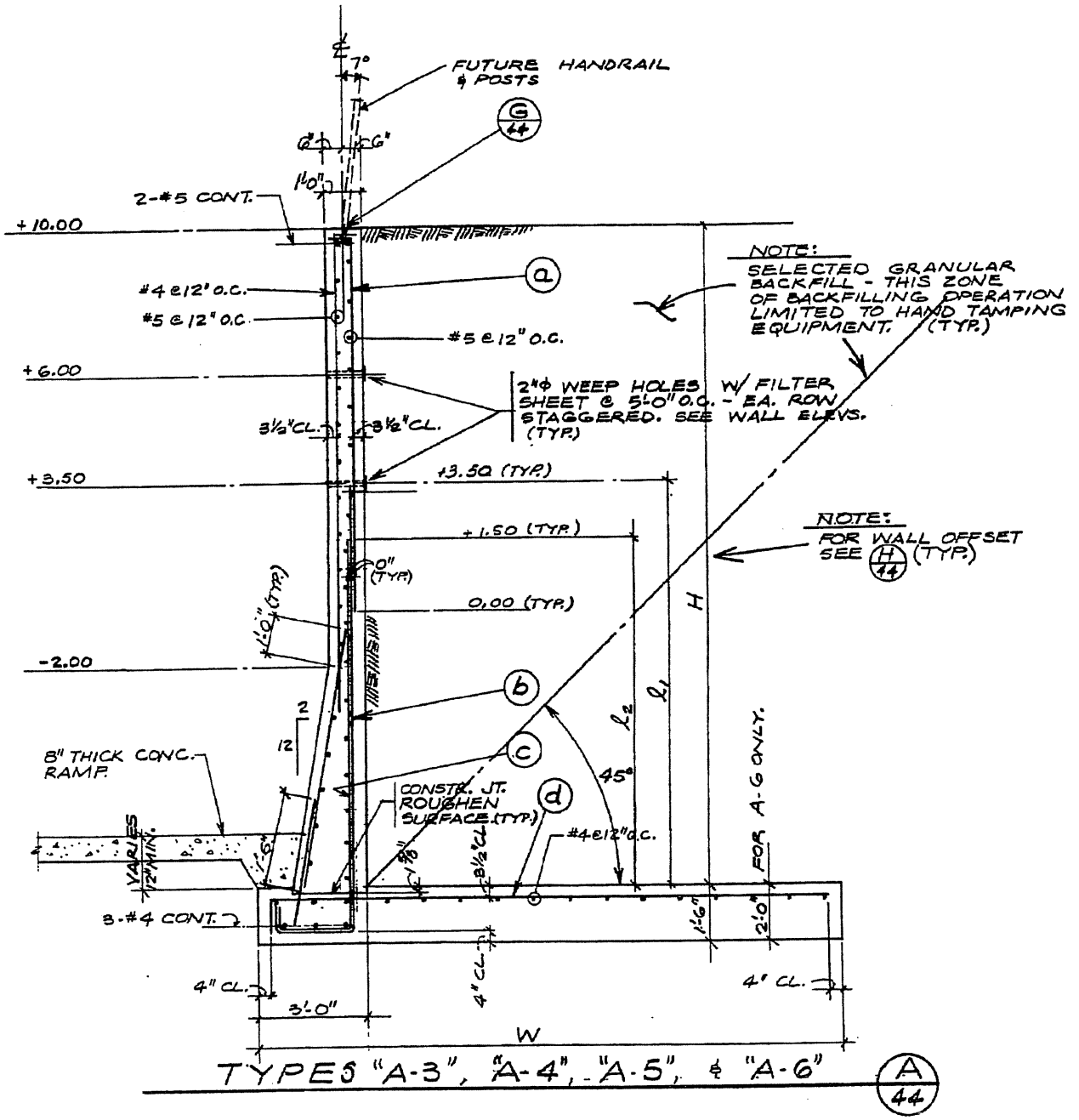
1/2" = 1'-0"

NOTES:

1. DETAIL FROM DANA POINT HARBOR AS BUILTS PREPARED BY KOEBIG & KOEGIG, INC 1968, SHEET 42 OF 50, D10.5-1.

2. DETAIL IS NOT TO SCALE.

BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES. DRW	SCALE: NTS	SHEET NO.
	DR. GSV	DATE: 12-17-03	SK-8
EXISTING WALL SECTION	CHK. DBS	FILE NO.: SK-8.dwg	8 OF 13 SHEETS



SOUTH WALL (ELEV. @ TOP OF WALL - CONSTANT)

TYPE	DIMENSIONS		REINFORCING STEEL				LENGTHS		REMARKS
	H	W	(a)	(b)	(c)	(d)	l_1	l_2	
A-6	20'-0"	18'-0"	#7@12	#9@12	#9@12	#10@6	13'-6"	11'-6"	" " " FTG. THICKNESS = 24"

NOTES:

1. DETAIL FROM DANA POINT HARBOR AS BUILTS PREPARED BY KOEBIG & KOEGIG, INC 1968, SHEET 44 OF 50, D10.5-1.
2. DETAIL IS NOT TO SCALE.

	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRW	SCALE: NTS	SHEET NO. SK-9
		DR.: GSV	DATE: 12-17-03	
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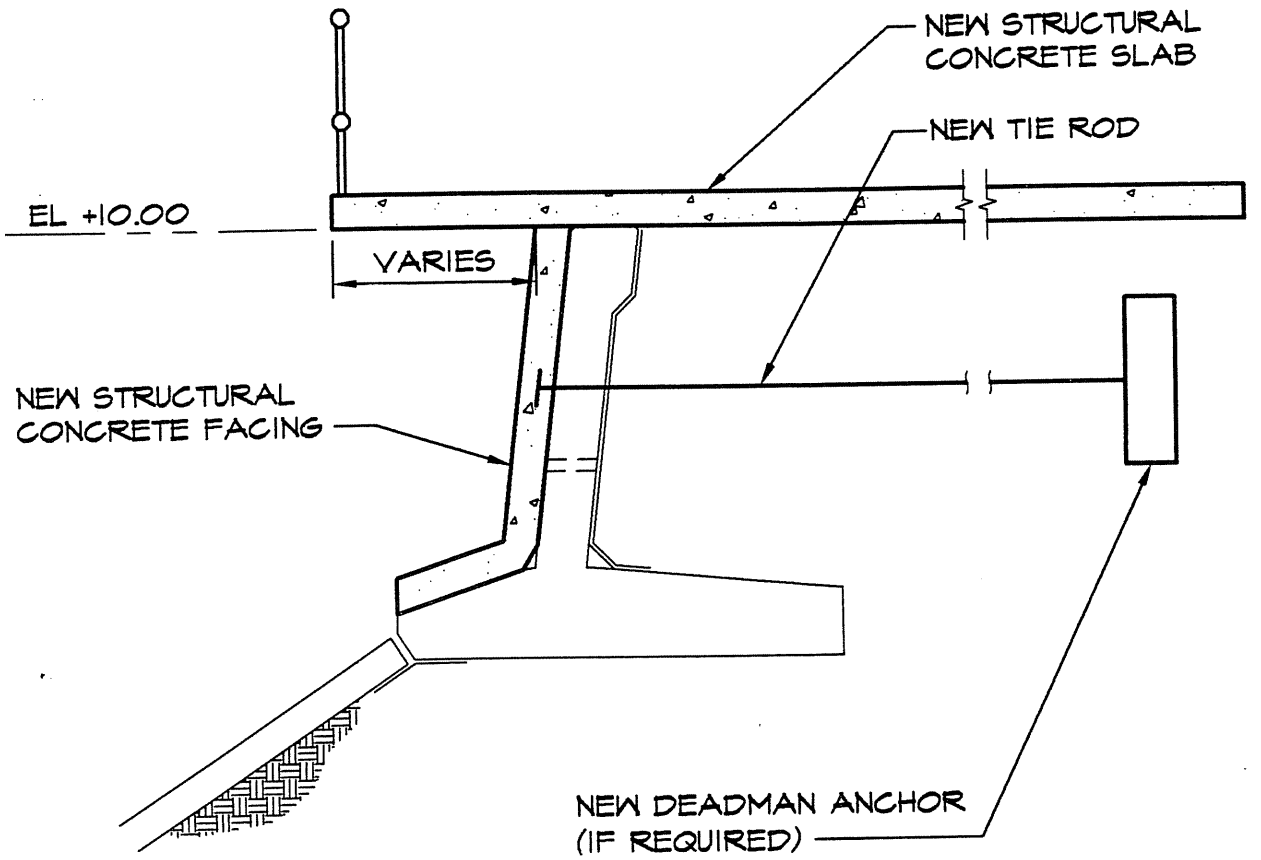

ALTERNATIVE 2 - IN-SITU REPAIR

BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRW	SCALE: 3/8"=1'-0"	SHEET NO.
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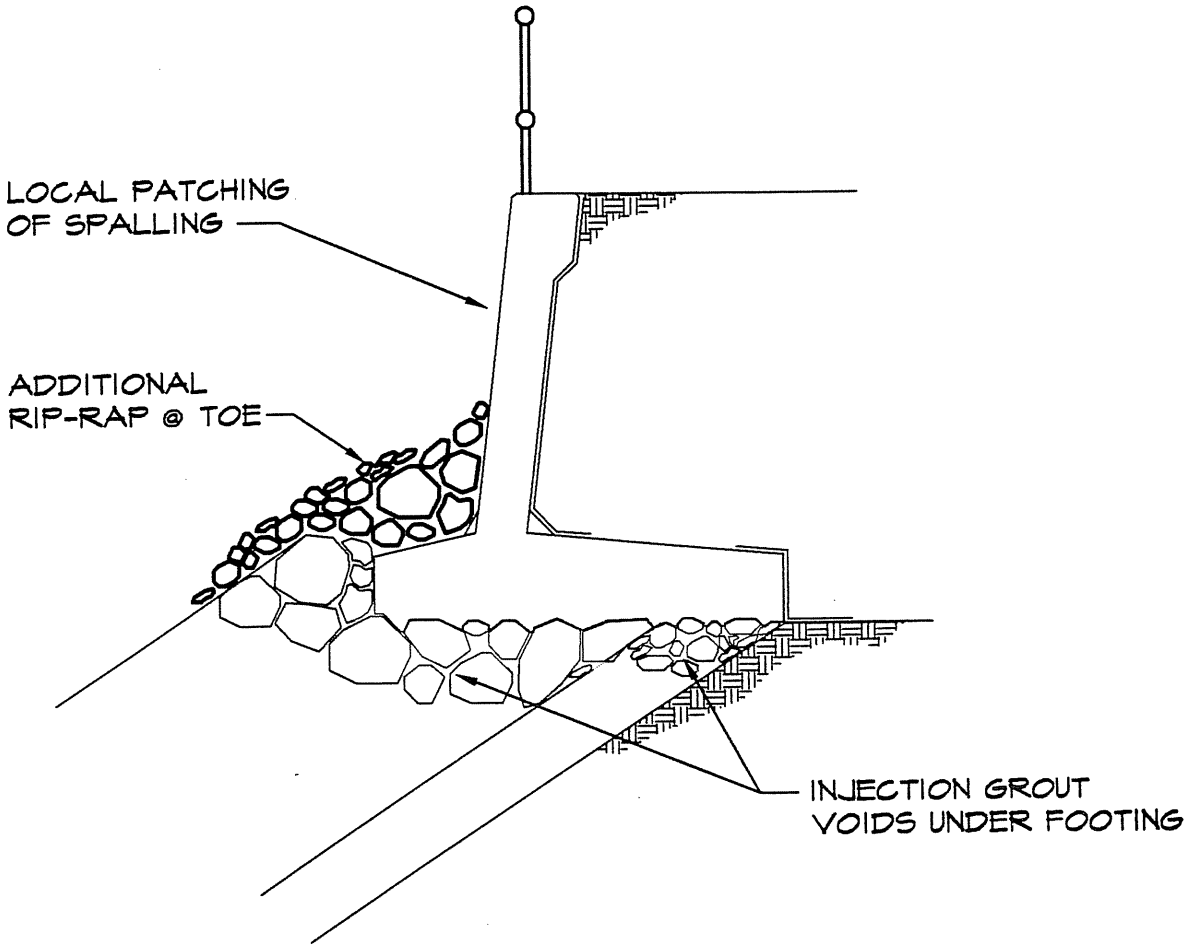
	CHK.: DBS	FILE NO.: SK-10.dwg	10 OF 13 SHEETS



ALTERNATIVE 2 - IN-SITU REPAIR

	BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES. DRM	SCALE: 3/8"=1'-0"	SHEET NO.
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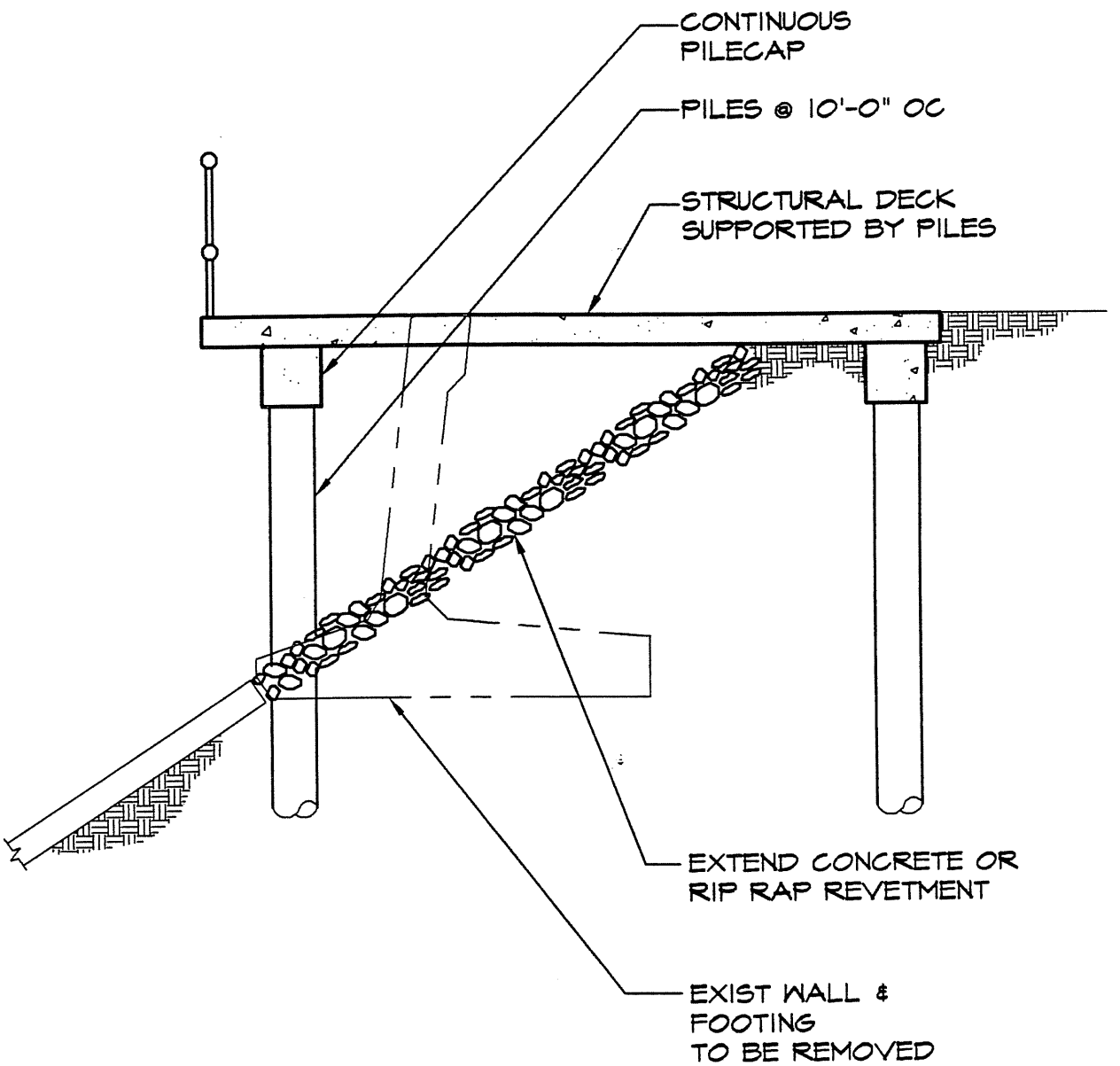

ALTERNATIVE 3 - REPLACEMENT

BULKHEAD STRUCTURAL EVALUATION DANA POINT MARINA REDEVELOPMENT	DES.: DRM	SCALE: 3/8"=1'-0"	SHEET NO.	
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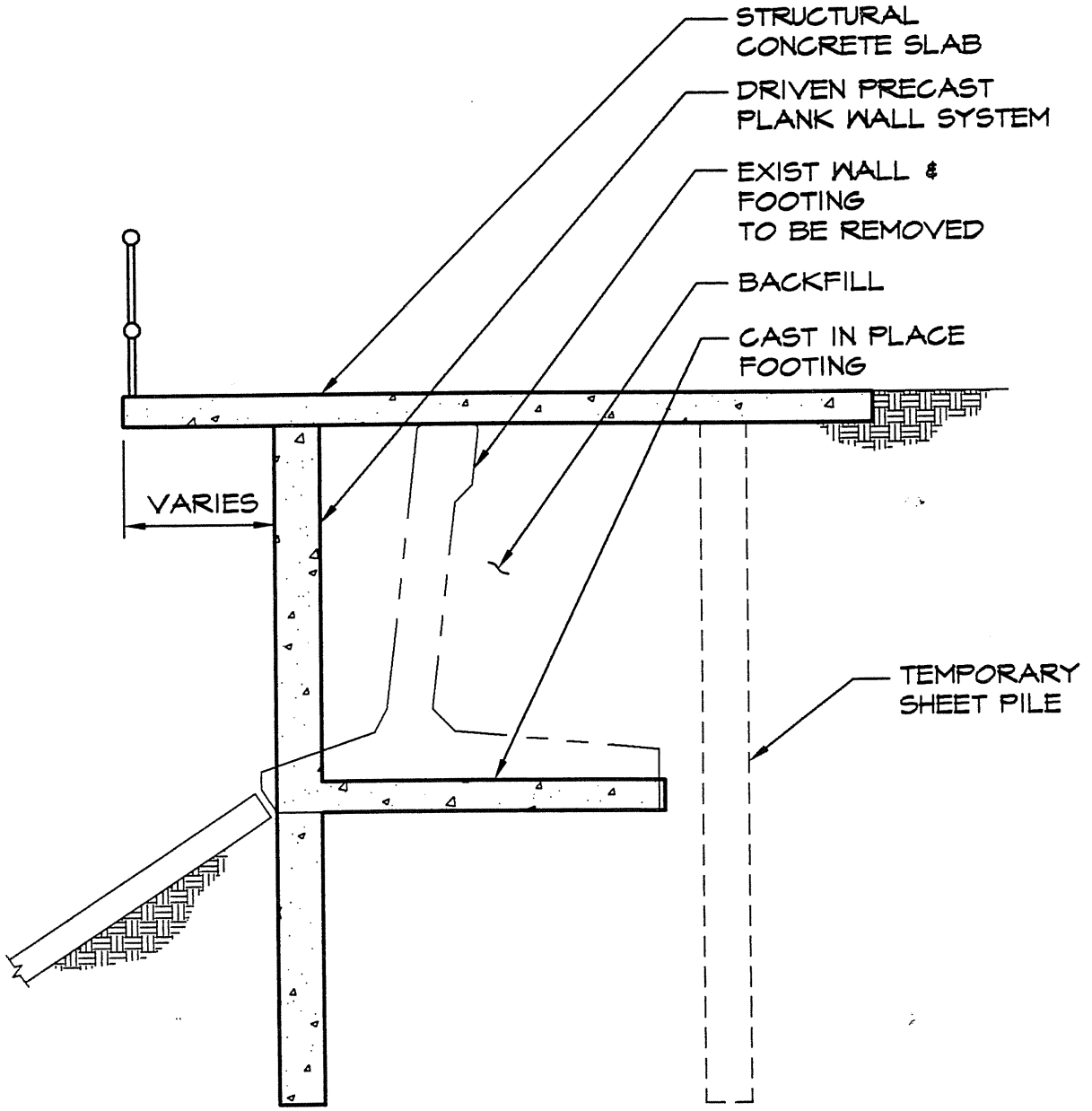
PIER REPLACEMENT CONCEPT	CHK.: DBS	FILE NO.: SK-12.dwg	12 OF 13 SHEETS	



ALTERNATIVE 3 - REPLACEMENT

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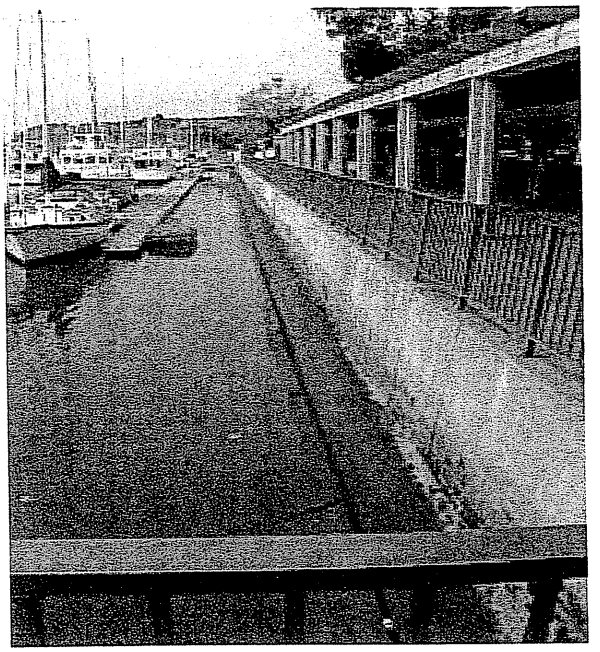

Photo 1 - Quay wall and concrete revetment on north side of basin.

Photo 2 - Quay wall and concrete revetment on east side of basin.

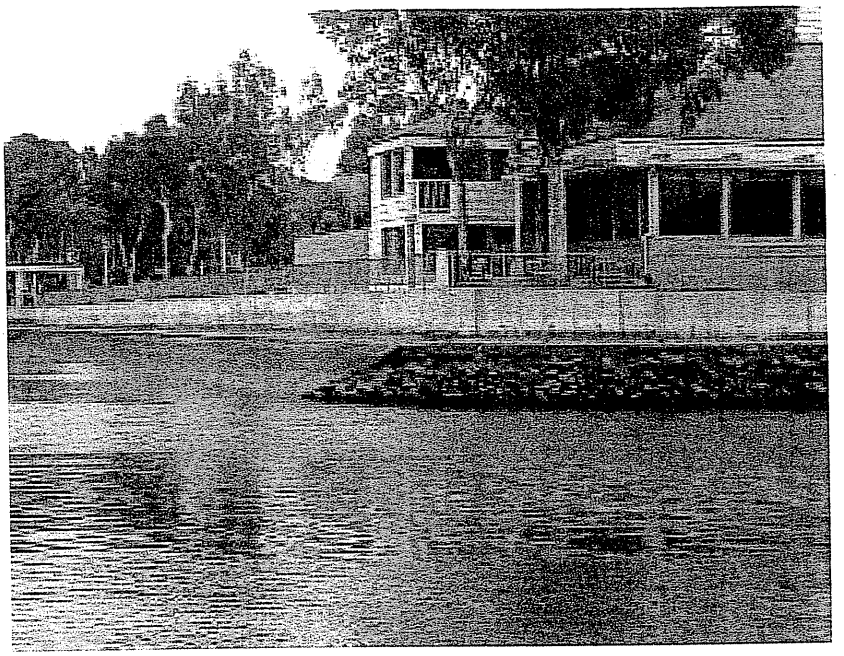

Photo 3 - View of both concrete and riprap revetment

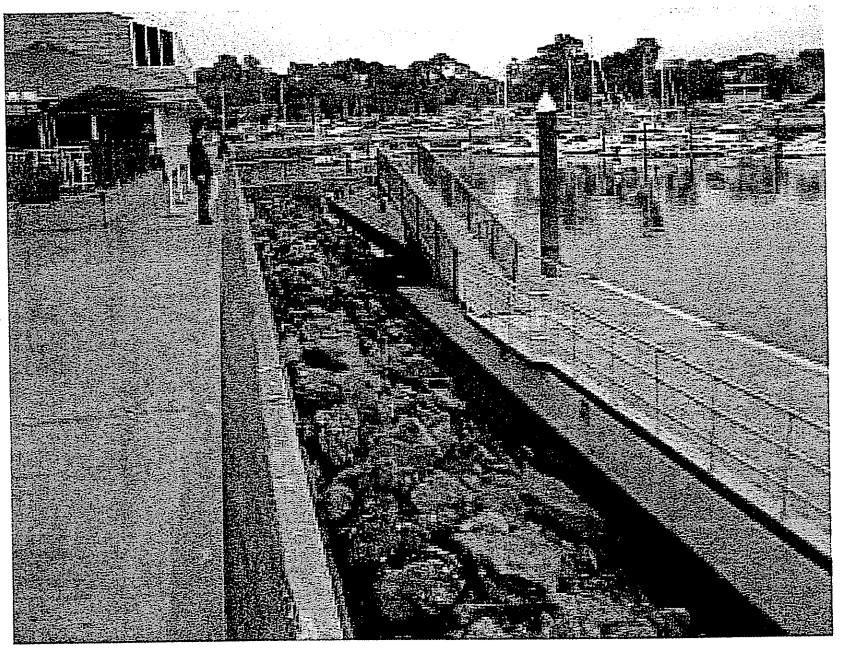

Photo 4 - View of riprap revetment with boat launch area beyond.

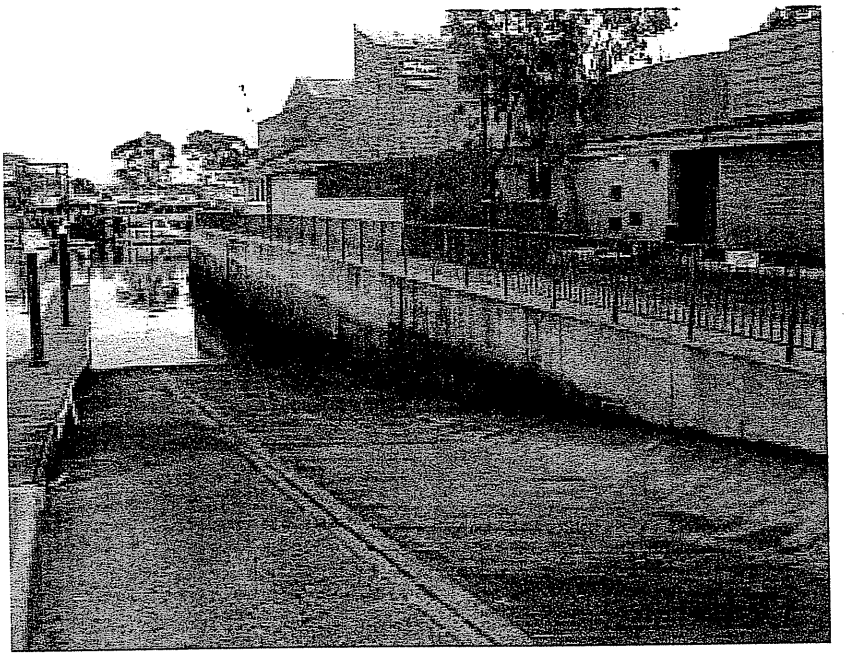

Photo 5 - Boat launch with cantilever retaining wall to the right.

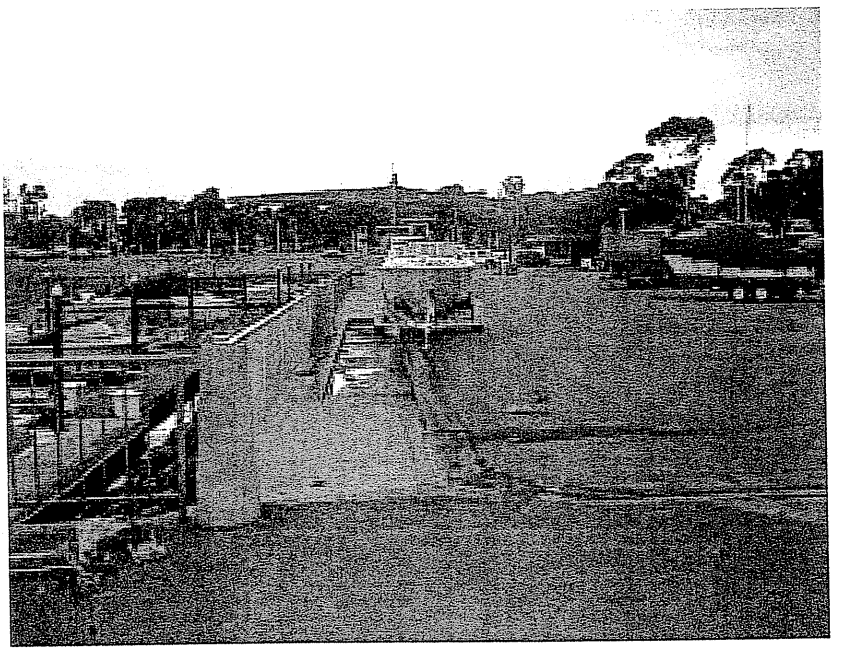

Photo 6 - View of dry storage area with boat basin to the left.

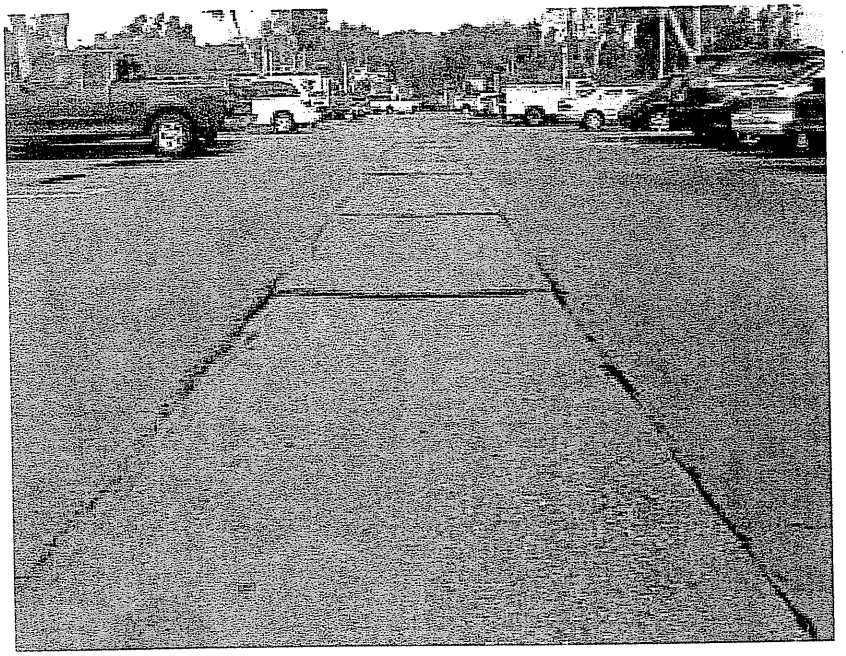

Photo 7 - Differential settlement in parking lot.

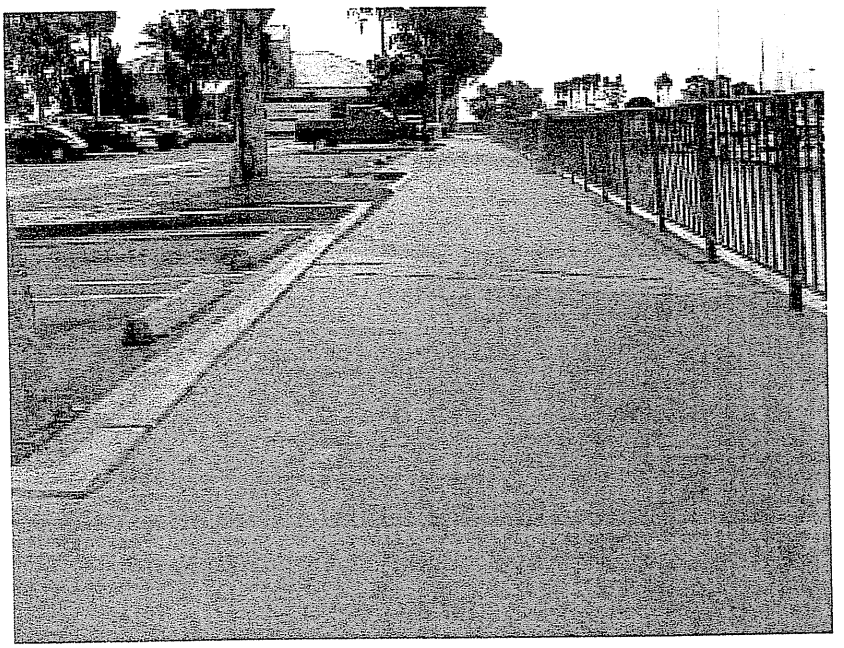

Photo 8 - Differential settlement along walkway.

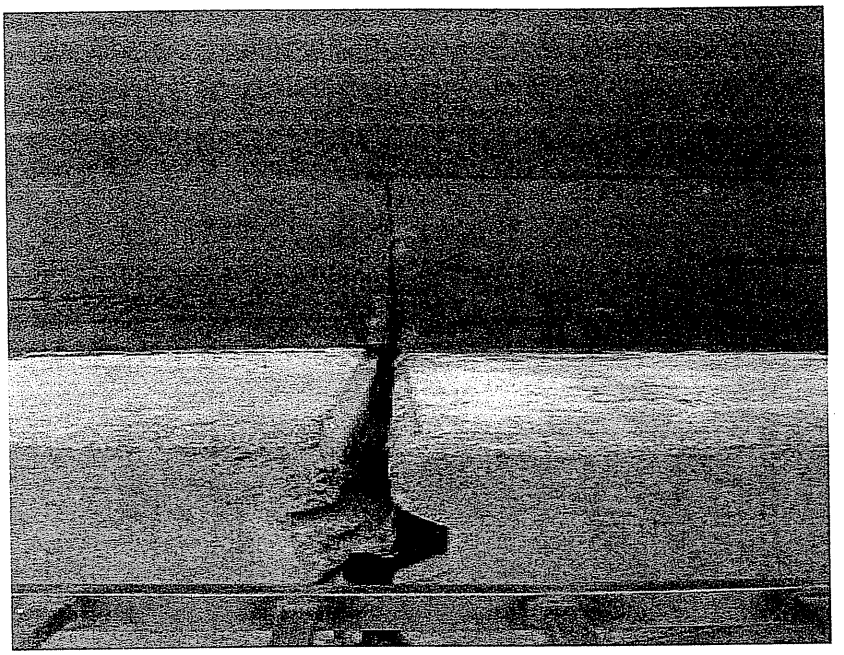

Photo 9 - Precast wall joint with less than the design gap.

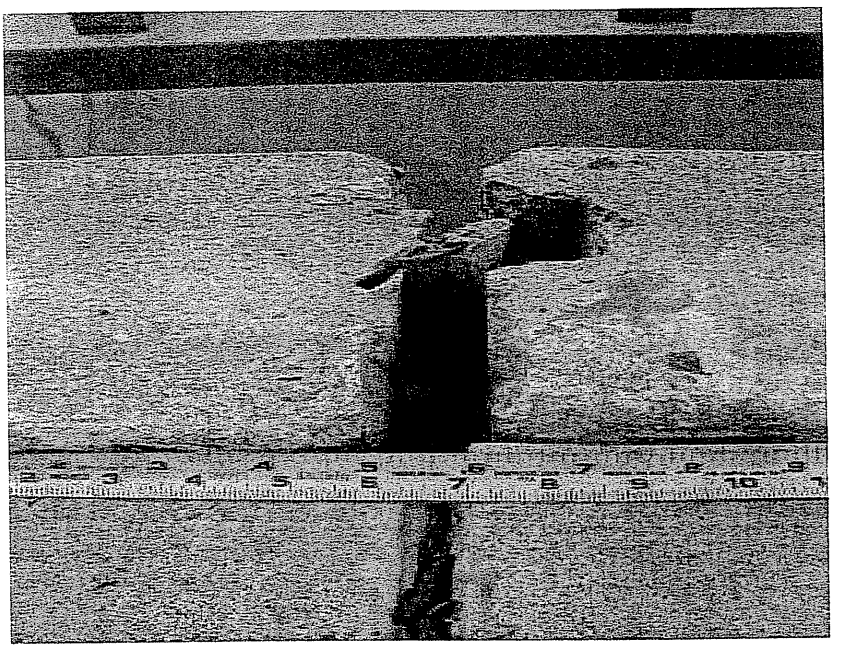

Photo 10 - Precast wall joint with larger than the design gap.

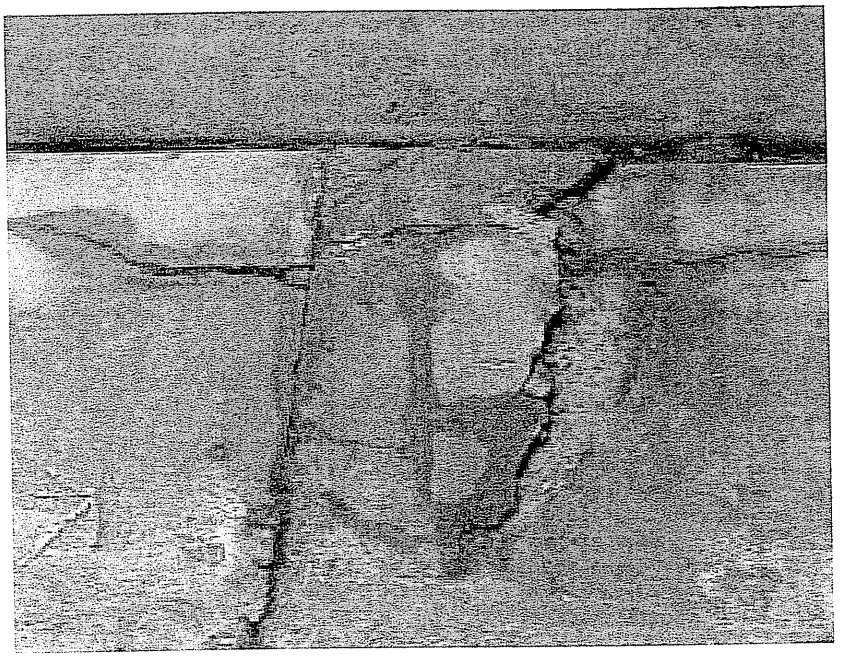

Photo 11 - Damage at toe of the quay wall footting.

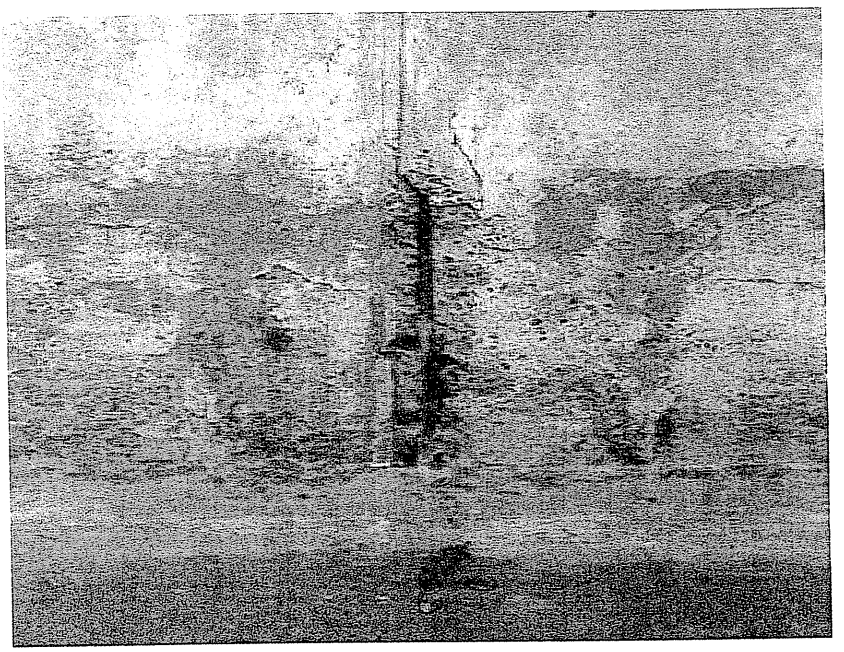

Photo 12 - Spalled concrete with exposed reinforcing bar.

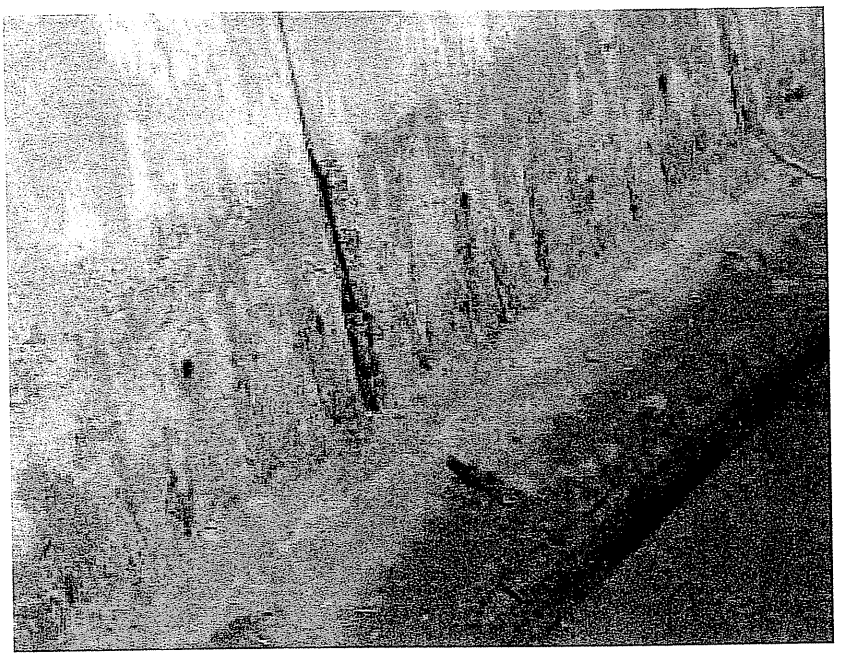

Photo 13 - Expanded view of area with spalled concrete.

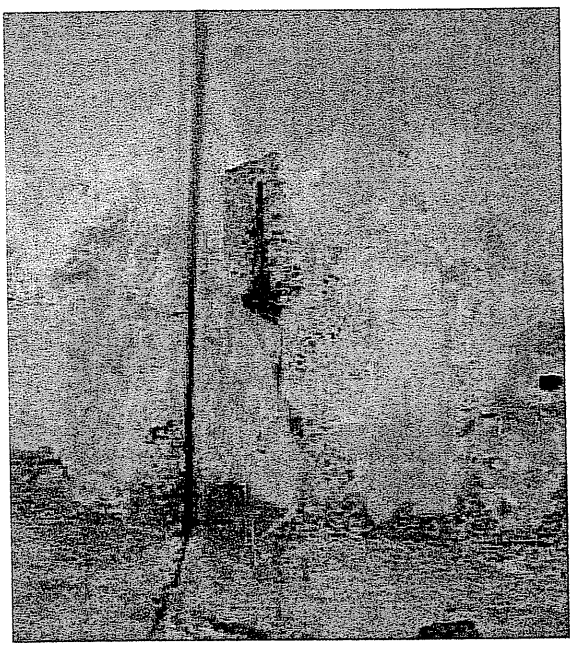

Photo 14 - Failure of previous patch repair.

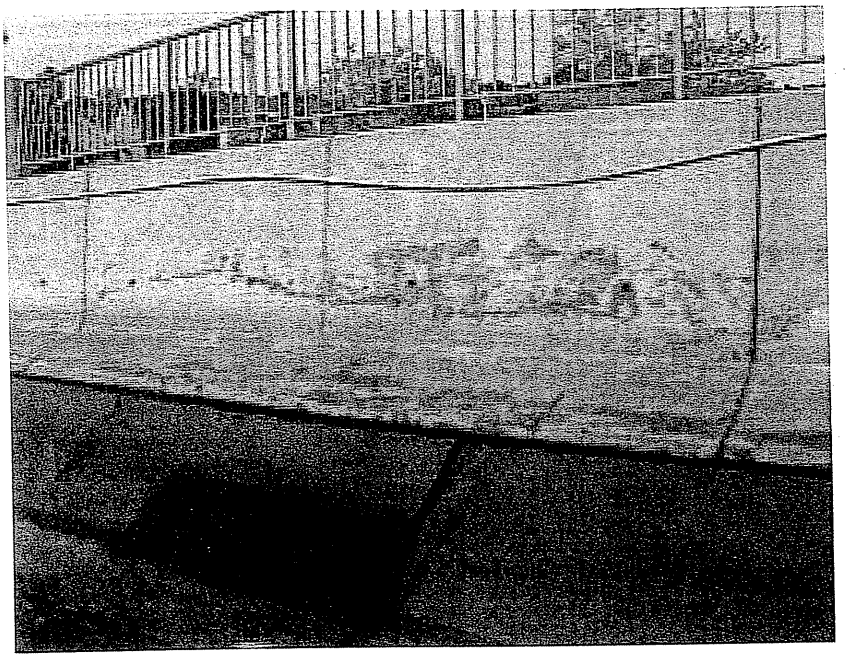

Photo 15 - Area of wall with loss of surface concrete. The horizontal line visible near the weep holes may be a cold joint created during construction.

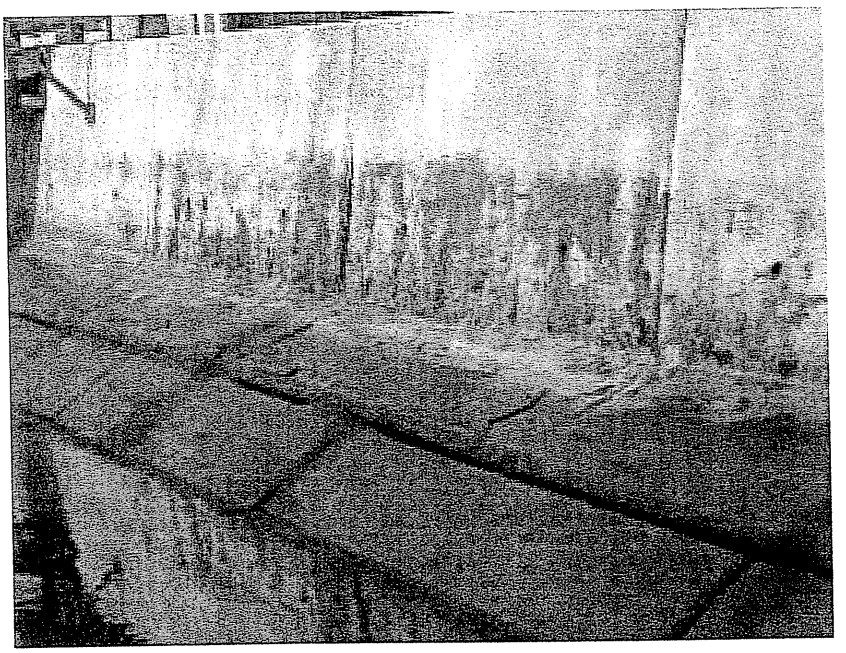

Photo 16 - Area of loss of surface concrete and general deterioration.

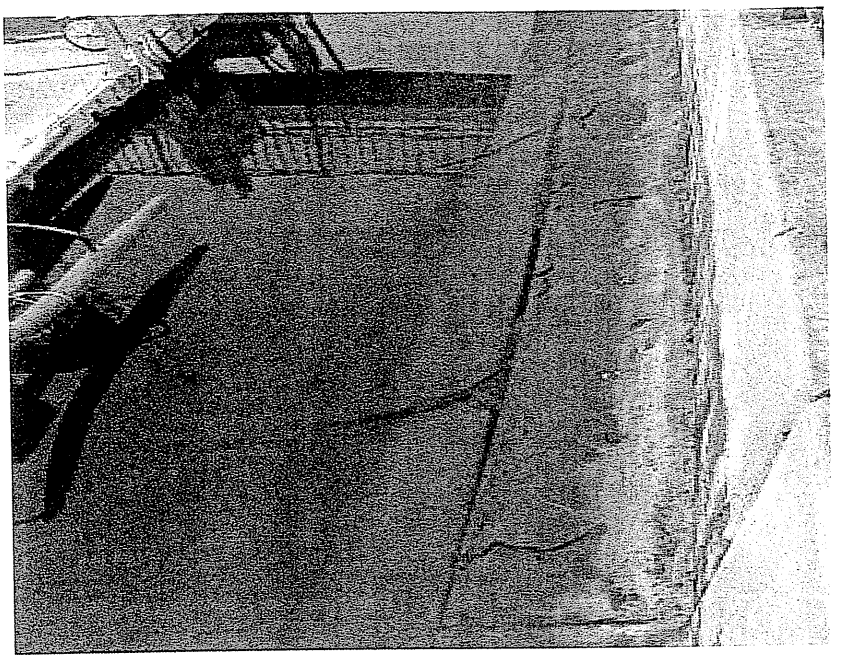

Photo 17 - Surface rust stains on toe of footing.

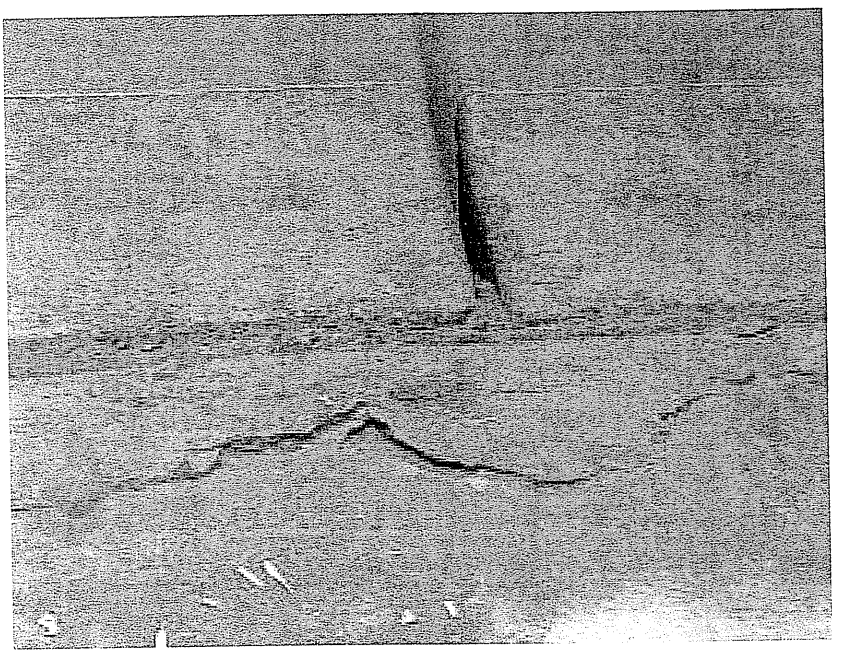

Photo 18 - Close-up of surface rust stains.

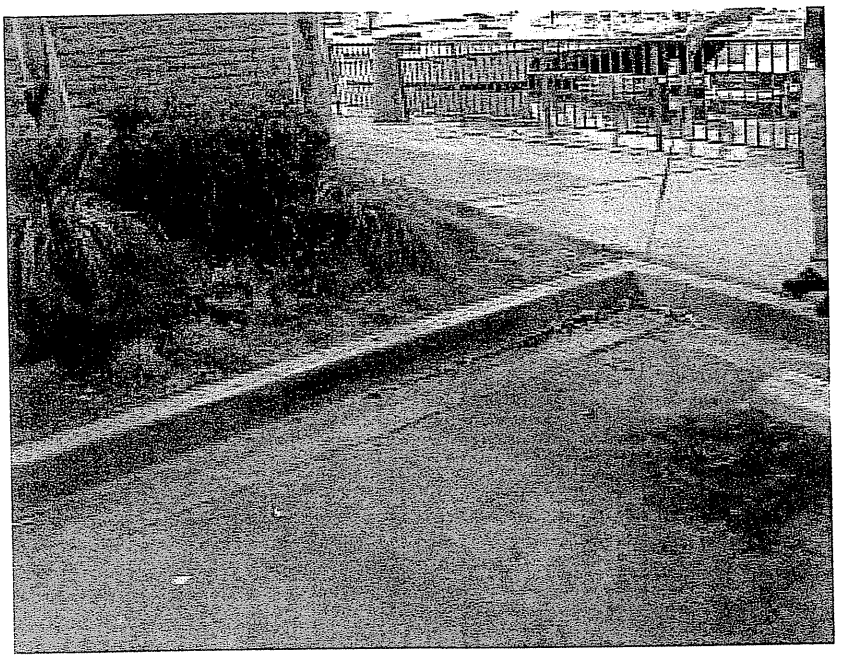

Photo 19 - Localized uplift of sidewalk in proximity to a tree root.

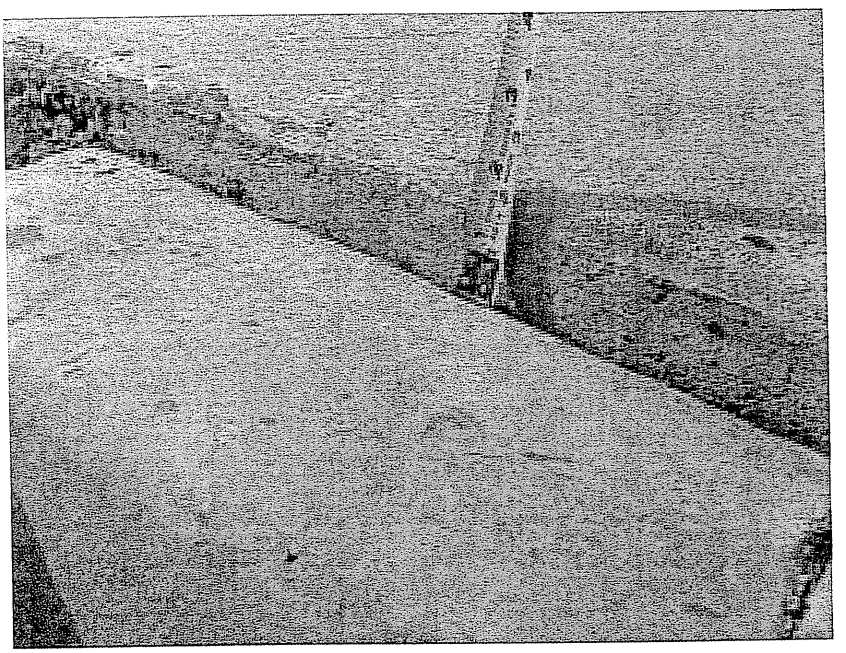

Photo 20 - Close-up view of sidewalk uplift.

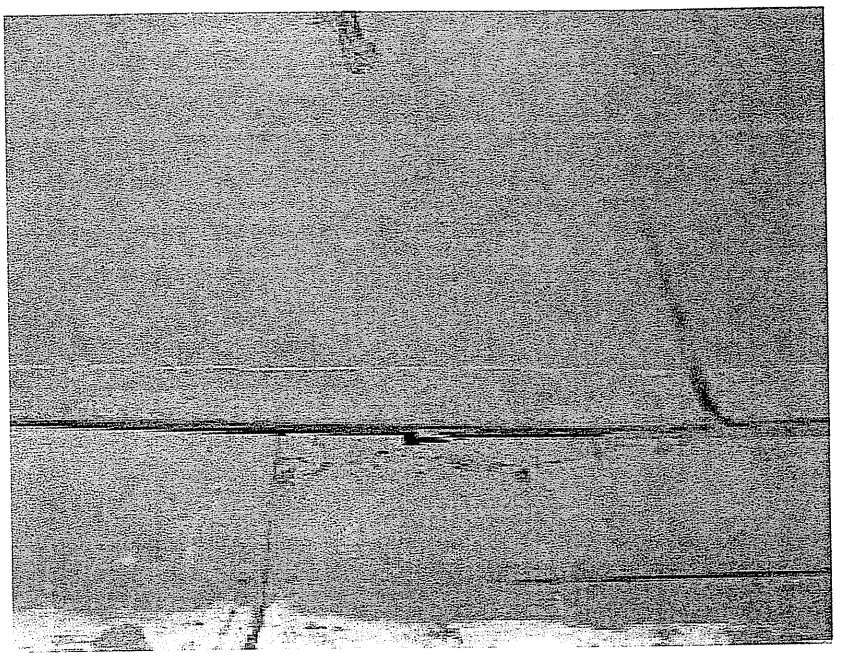

Photo 21 - Grout in gaps between revetment panels and betweeen toe of footing and panels.

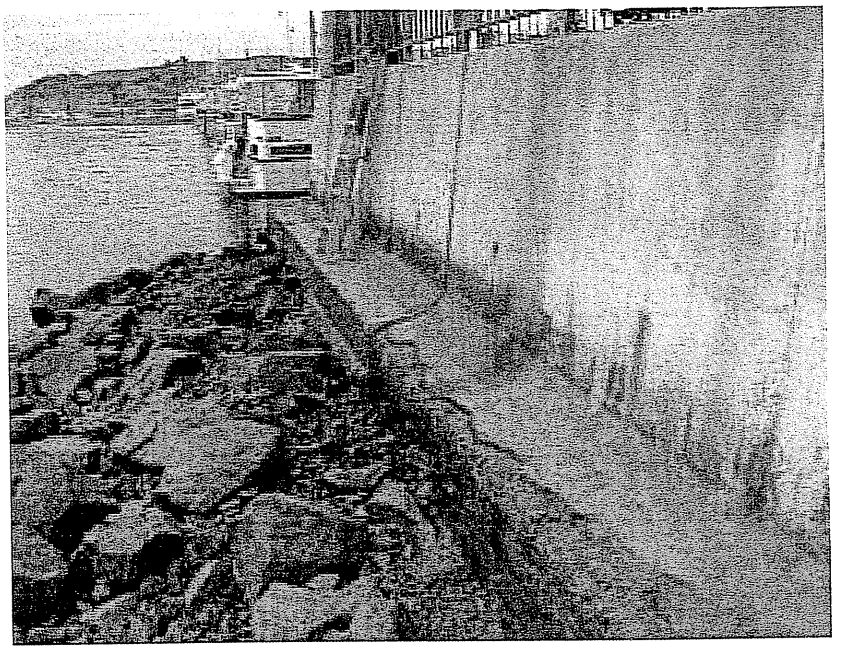

Photo 22 - Spalled concrete and riprap settlement.

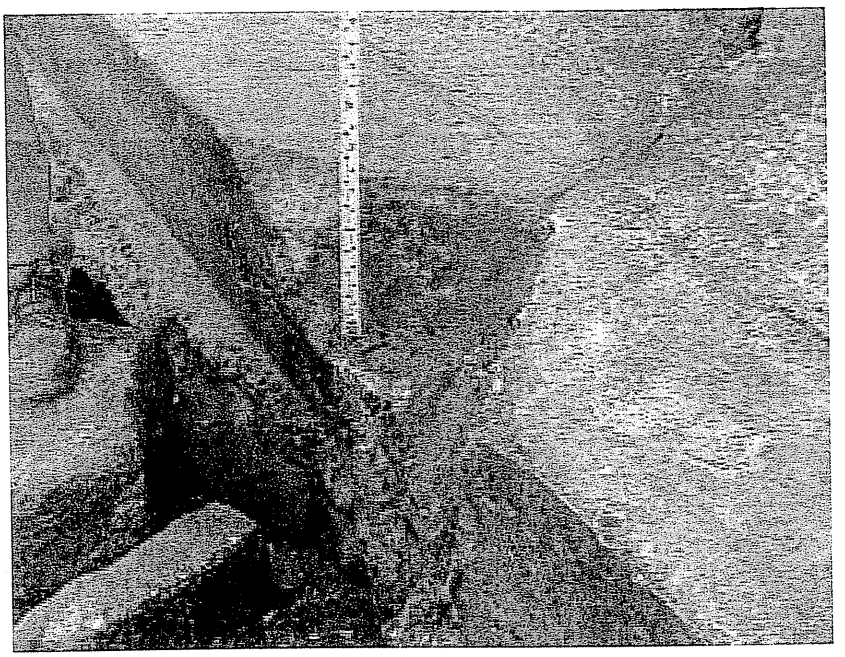

Photo 23 - Note absence of reinforcing bars in toe of footing.

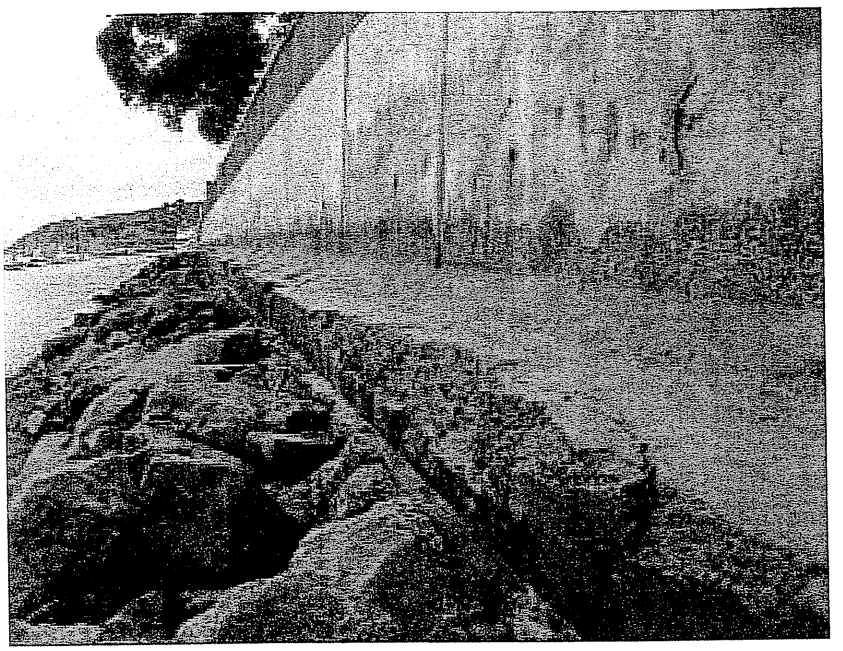

Photo 24 - Spalled concrete at toe of footing.

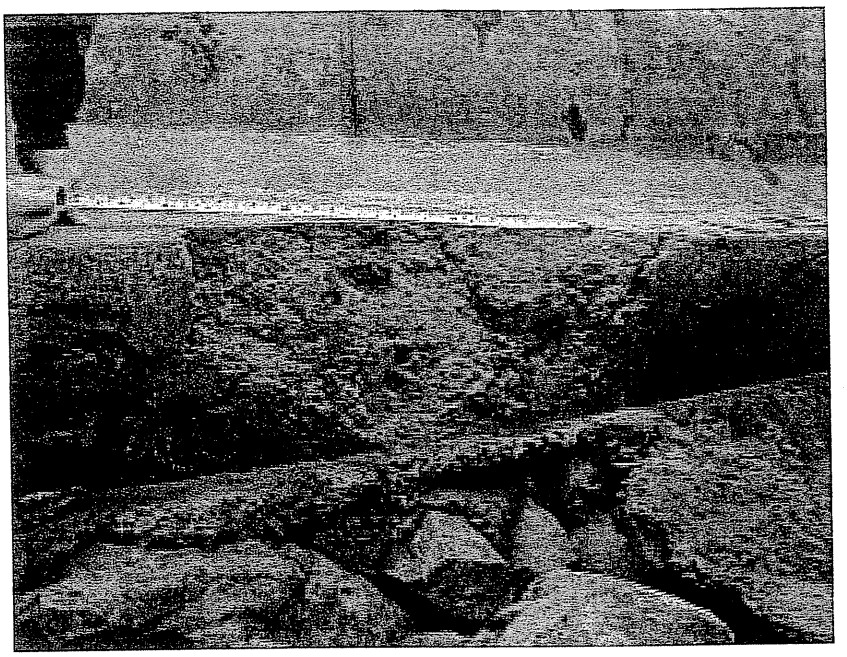

Photo 25 - Large spall at toe of footing.

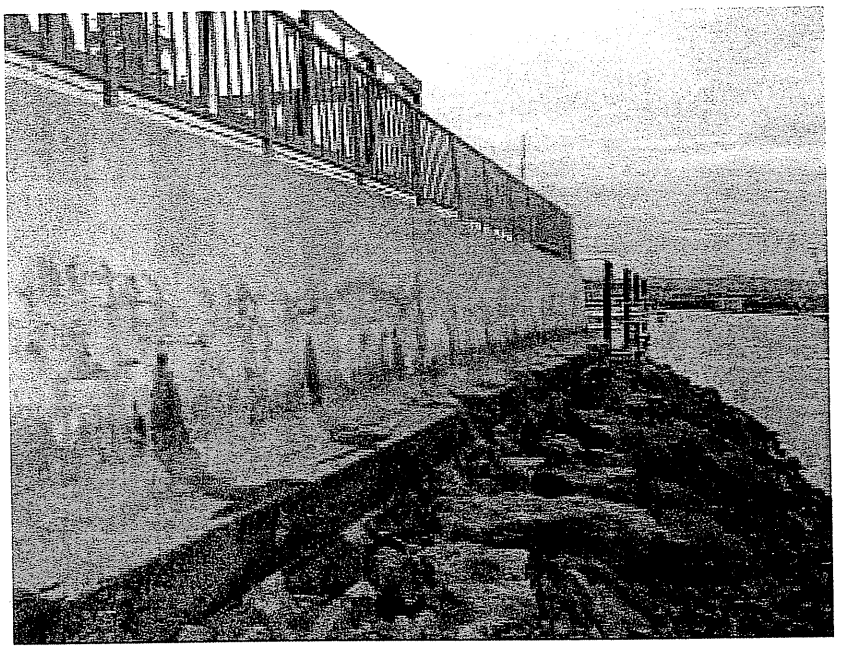

Photo 26 - Settlement of riprap.

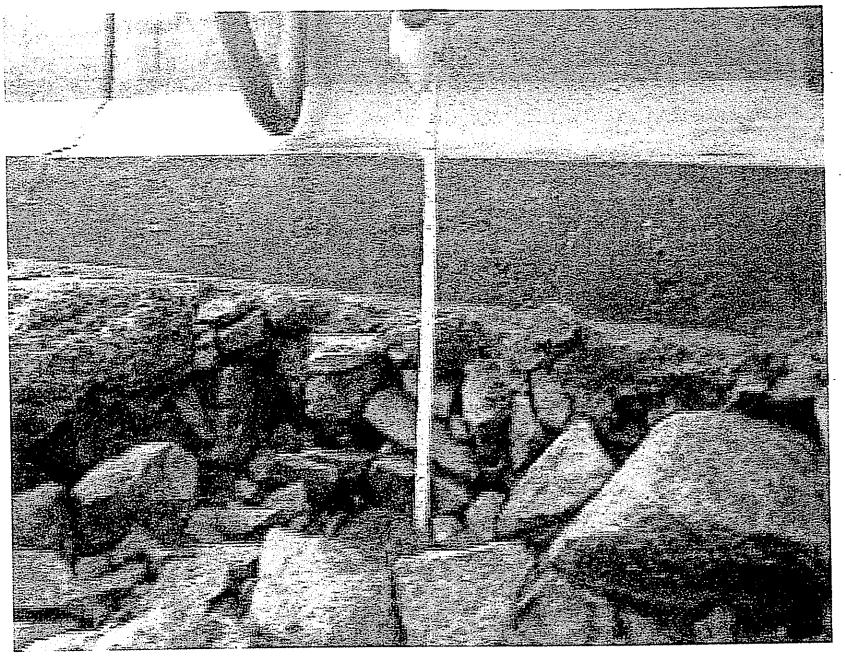

Photo 27 - Area of large loss of riprap in front of wall.

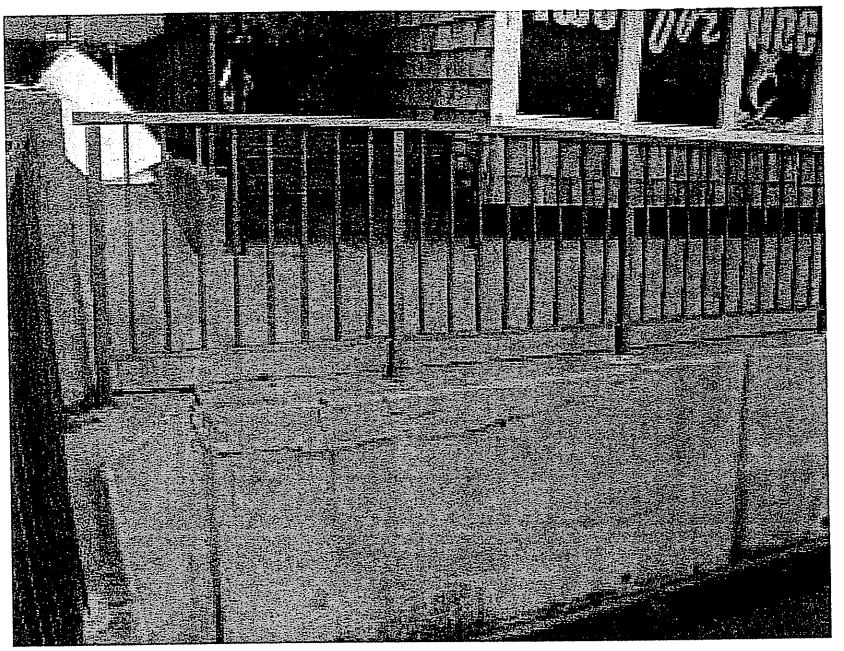

Photo 28 - Fractured stem wall.

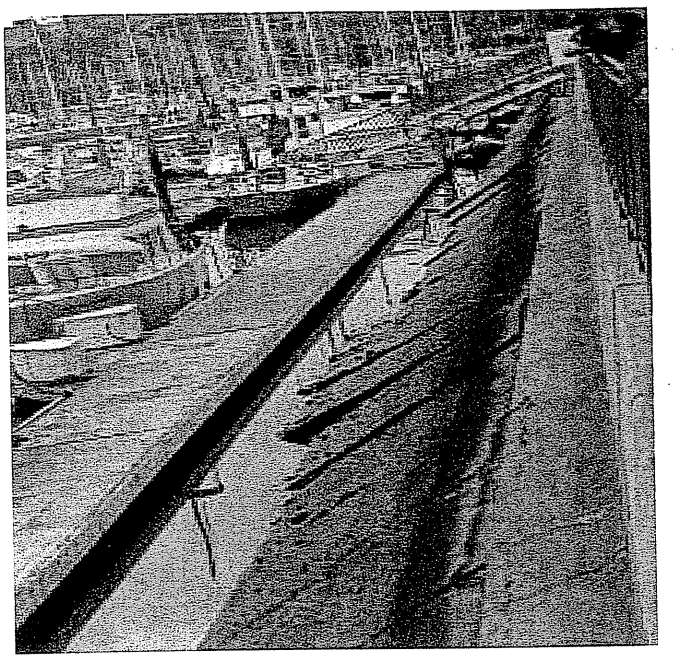

Photo 29 - Dislodged concrete revetment panels.

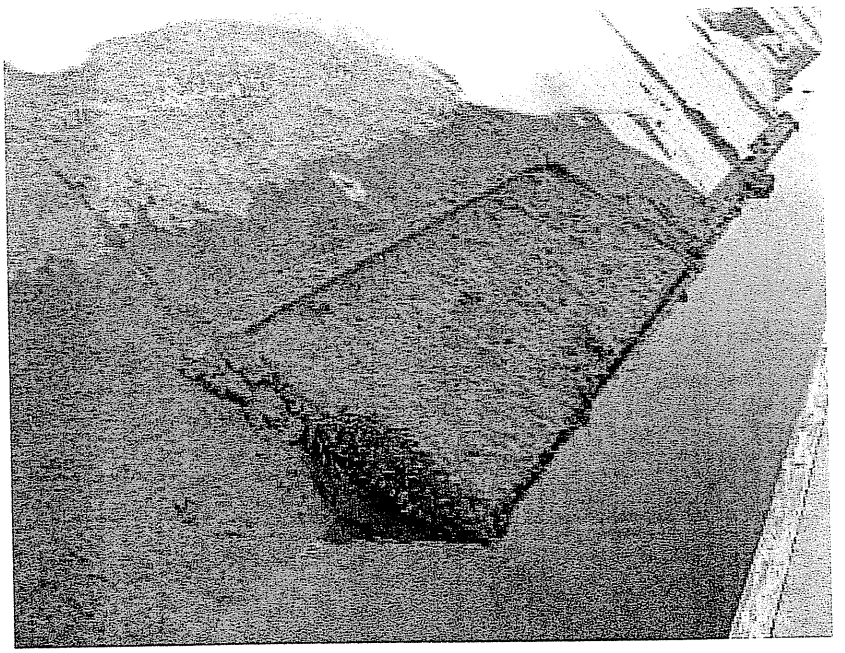

Photo 30 - Dislodged panel with a horizontal break, likely at a control and joint.

Photo 31 - Close-up of dislodged panel.

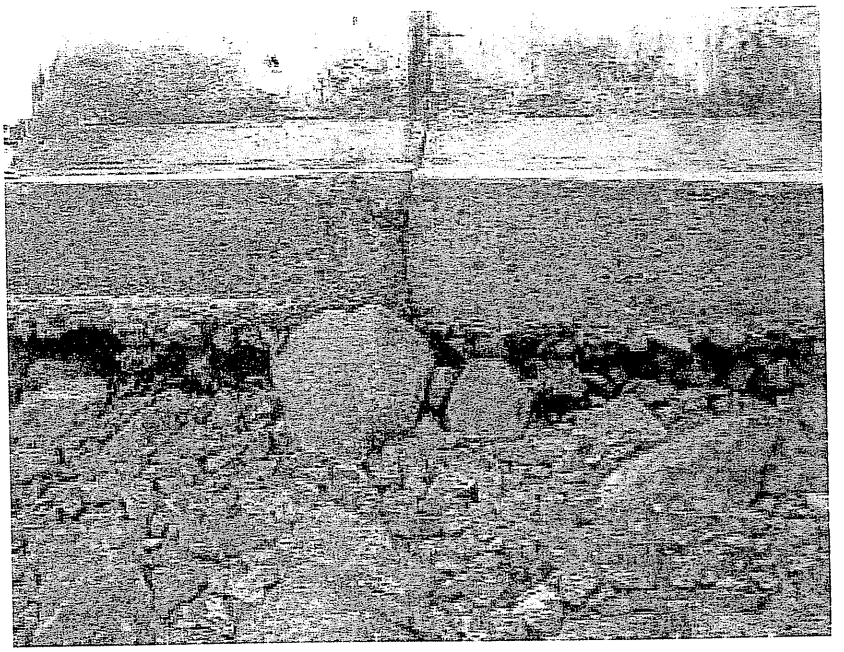

Photo 32 - Settlement and movement of riprap revetment.

1.0 BULKHEAD CONDITION ASSESSMENT

The purpose of this bulkhead condition assessment appendix is to evaluate the existing state of the bulkhead fronting the proposed Dana Point Harbor Revitalization Master Plan infrastructure, structural and hardscape improvements. The proposed Phase I and Phase II Commercial Core Improvements, as illustrated in the Concept Plan Update furnished by Wallace, Roberts and Todd, are primarily located to the north and northeast of the East Basin. As a result, this bulkhead condition assessment focuses on the northern bulkhead segments of Dana Point Harbor bordering the East Basin, the East Channel and the Mooring Basin. The evaluation of the bulkhead will utilize previous field data and investigations to identify specific areas of structural instability and will document areas of potential concern pending the implementation of the Dana Point Harbor Master Plan. Upon completion of the investigation, a cumulative assessment will be formulated regarding the overall condition of the structure and preliminary recommendations will be prepared.

2.0 BULKHEAD DATA COLLECTION

The information presented herein is based on existing construction design drawings, bulkhead maintenance repair and monitoring records, several prior geotechnical investigations, and all relevant engineering materials related to the Dana Point Harbor bulkhead.

3.0 INTRODUCTION

Dana Point Harbor, located within the City of Dana Point and presented in Figure 1, is a full-service small-craft harbor facility consisting of over 250 acres of developed uplands and protected boat storage basins and waterways. Fully-sheltered from the open coast by almost 8,000 lineal feet of Federal breakwater, the interior East and West basins provide berthing for more then 2,500 small-craft. A variety of public, commercial and educational ancillary facilities, and recreational amenities occupy the areas adjacent to the mooring areas and waterways. The harbor was completed in 1971 and has been heavily used for active and passive forms of aquatic and related forms of recreation since its inception over 30 years ago.

The County of Orange, recognizing the need for facility renovation, additional traffic and parking considerations, and economic revitalization, is now under an ambitious program to revitalize the Harbor. As part of the proposed plan, two new parking structures as well as several new buildings will be built and existing commercial buildings, private facilities, utilities, and general site improvements will be renovated and updated to current building codes for modern seismic standards, health and safety requirements and disabled accessibility standards. In 1998, the capital expenditures anticipated over the next fifty years for the Dana Point Harbor capital improvements were estimated to be between $35 and $50 million (in 1998 dollars). Though this number will be refined further through the Master Plan planning process, the expenditures for the harbor revitalization Master Plan will be substantial.

The proposed commercial core improvements developed by Wallace, Roberts and Todd, Inc. are almost all solely located either directly adjacent to, or in the vicinity of, the northern sections of the Dana Point Harbor bulkhead bordering the East Basin, the East Channel, and the Mooring Basin. Therefore, it is important to assess the structural stability of this portion of the bulkhead prior to plan implementation and construction. Figure 2 presents the Dana Point Harbor plan

layout illustrating the bulkhead condition assessment area of interest as it pertains to the proposed Master Plan.

4.0 BULKHEAD DESCRIPTION

The existing bulkhead located within the project area of interest is a cast in place concrete gravity quay wall with either riprap and concrete panel slope protection depending on the bulkhead location within the harbor. The total bulkhead length within the project area is approximately 5,100 linear feet, including an approximate 230-foot boat ramp section. Approximately 2,300 linear feet of the bulkhead is protected by concrete panels while the remaining 2,570 linear feet is stabilized with riprap. Figure 3 presents the typical cross-section index for the bulkhead area of interest.

4.1 Concrete Panel Slope Protection

Figure 4 presents the typical cross-section of the bulkhead sections consisting of concrete panel slope protection located along both the northern and northeastern border of the East Basin. The concrete quay wall, detailed in Figure 5 and located atop the panel slope protection, was poured in place and is one-foot thick with a 6-foot 6-inch by 1-foot 4-inch wide footing. Weep holes are located at an elevation of +6.5 feet and +1.0 feet, Mean Lower Low Water (MLLW) along the entire bulkhead length at a distance of 5 feet on center and control joints were placed about every 8 feet along the span of the panel. The design elevation at the top of the quay wall is +10.0 feet, MLLW. Cast in place concrete panels that are 5.5 inches thick and 10 feet wide protect the 1V to 1.5H (one vertical foot to 1.5 horizontal feet) slope at the base of the retaining wall for a horizontal distance of approximately 20.5 feet. The design compressive strength of all poured in place concrete was 3,500 psi at 28 days. The finished design grade elevation of the East Basin is −10.0 feet, MLLW. The toe of the panel protection consists of a concrete triangular thrust footing with a design depth of 2 feet below finished grade and founded in natural bedrock. Open joints from 1 to 2 inches wide were established in the area between the top of the panels and the base of the quay wall footing and along the panel slope length spaced approximately every 10 feet. Prior to the concrete pour, a filter fabric (approximately 18 inches wide) was placed on approximately 6 feet of compacted fill, composed of select native granular sandstone excavated from the site, and cut slopes at the control joints and weep hole areas. The purpose of the filter fabric was to allow the free flow of water, while impeding the movement of the fill material.

4.2 Stone Revetment Slope Protection

Figure 6 presents the typical cross-section of the remaining 2,570 linear feet of bulkhead sections consisting of stone riprap slope protection. The poured in place concrete quay wall was built to similar design specifications and elevations as described above (See Figure 5); however, the block footing at the base of the wall is 6 feet by 1-foot 4 inches wide and minor modifications to the rebar reinforcement were made as well. The stone revetment protecting the 1V to 1.5H slope consists of a one-foot layer of filter stone placed atop a filter sheet with an outer 4-foot layer of class "B" revetment stone. Although detailed specifications of the class "B" revetment stone were not available during the course of this initial assessment, based on visual evidence the revetment stone is estimated to consist of approximately one-quarter to one-half ton quarry stone. The design depth of the revetment is 2 feet below the finished grade elevation, which varies from −8 to −12 feet, MLLW depending on the location of the bulkhead within the project area. During construction the outer 12 feet of compacted fill under the stone revetment, measured normal to

the slope, was composed of select rocky, granular material derived from the excavation of material from the site.

4.3 Representative Bulkhead Sections Through Proposed Commercial Core Improvements

As part of Phase 2 of the Dana Point Harbor Revitalization Master Plan, a number of commercial core improvements are proposed in the existing parking lot located between the East Basin and the East Channel and Mooring Basin. These improvements include a number of new buildings as well as several structural and logistical upgrades to existing buildings. Accordingly, Figure 7 presents the typical cross-section extending through the northeastern bulkhead section of the East Basin across the existing parking lot and into the East Channel and Mooring Basin. As is illustrated in the figure, both riprap and panel slope protection sections are represented. The distance from bulkhead to bulkhead is approximately 180 feet with a maximum design elevation of +12.5 feet, MLLW. In addition, Figure 8 presents a typical cross-section through the 230-foot boat ramp segment of the project area.

5.0 FOUNDATION ASSESSMENT

The extent, character, and elevation of the subsurface bedrock horizon are critical components in assessing the structural stability of the bulkhead. Several foundation investigations that have been performed within Dana Point Harbor, dating back to before the Harbor was constructed, were reviewed during this bulkhead condition assessment.

5.1 Prior Foundation Investigations

Dames and Moore Consultants conducted a seismic refraction survey at the proposed harbor site for the Orange County Harbor Commission in February 1957. Seismic data collected consisted of 13 seismic lines, which were correlated to additional field information obtained from a marine geologist and a professional diver. In April 1964, Moore and Taber Consultants conducted an investigation and prepared a report entitled "Onshore Geology and Materials Study" for Koebig and Koebig, Inc. and the Orange County Harbor District. Data obtained for this investigation included five 3-inch rotary wash borings drilled to depths ranging from –16 to –21 feet, MLLW. An investigation of the pre-dredge conditions in the Entrance Channel, the East Channel, and the West Channel were conducted in April 1968 by Moore and Taber for the Corps of Engineers resulting in eight 3-inch NX rotary borings with intermittent standard penetration depths ranging from –16 to –22 feet, MLLW. In addition, Lejman and Lee, Inc. collected sub-bottom geophysical profiles in 1987 prior to the 1990 harbor maintenance dredging effort.

5.2 Physical Properties of Bedrock Formation

Based on these pertinent foundation studies, it has been determined that the bedrock foundations throughout Dana Point Harbor consist of tertiary age, hard, lithified rock covered with thin pockets of recent marine sands. The bedrock is part of the Capistrano, Monterey, and San Onofre formations and dips about 10 to 30 degrees to the east-northeast (Moore and Taber, 1968). The Capistrano formation visible in the exposed cliffs behind the harbor extends approximately 1,850 feet along the eastern breakwater. The Capistrano formation is the underlying bedrock foundation for approximately 96 percent of the project area. The remaining 4 percent, which is an approximate 200-foot stretch extending east from the northwestern corner of the East Basin, is

founded in the Monterey formation. The Capistrano formation is characterized by coarse-grained sand and gravel along with cobble sized conglomerates. The Monterey formation consists of soft, dark grey, organic shales interbedded with very hard, tan colored chert beds (U.S. Army Corps of Engineers, 1991). Analysis of the geophysical profiles and sediment samples throughout the harbor indicates that the recent sands overlaying the bedrock range from approximately 2 to 5 feet in thickness and are fine-grained silty sands.

5.3 Relative Bedrock Elevation

The dip in the Capistrano formation, underlying the majority of the bulkhead condition assessment project area, is approximately 22 degrees towards the east-northeast. As such, the elevation of the bedrock is at a minimum along the western portion of the study area and rises gently as one moves east along the bulkhead line through the East Basin and into the East Channel and Mooring Basin, respectively. The determination of the exact bedrock elevation throughout the bulkhead condition assessment project area is somewhat complicated by the fact that field borings acquired in the vicinity of the bulkhead line did not penetrate into bedrock. However, based on the interpretation of the geotechnical reports cited above, it is estimated that the elevation of the bedrock is approximately –11.5 feet, MLLW at the northwestern corner of the East Basin and rises to a maximum value of approximately +0.5 feet, MLLW along the northern bulkhead at the terminus of the East Channel.

6.0 SEISMIC INFLUENCE AND LIQUEFACTION POTENTIAL

The possibility of a primary surface rupture or deformation at the project site is considered to be rather low since no active faults are known to cross the site. However, the close proximity of several major active faults could cause ground shaking (Leighton and Associates, 1999). The known active faults capable of producing strong ground shaking at the site include the Newport-Inglewood (3 kilometers offshore), Palos Verdes, Coronado Bank-Agua Blanco, and Whittier-Elsinore Faults (Leighton and Associates, 1999). In addition, the San Andreas Fault, which is located approximately 87 kilometers from Dana Point Harbor, represents the boundary between two tectonic plates that are slowly moving past each other at a rate of approximately 2 inches per year. This movement occurs in spasmodic episodes about every 150 years (U.S. Army Corps of Engineers, 1991).

In 1999, Leighton and Associates performed a deterministic ground motion analysis utilizing the EQFAULT and EQSEARCH computer programs. These programs effectively calculated the potential ground motion that could occur at the project site as a result of a maximum credible earthquake (MCE) occurring along any of the known active faults within a 100-kilometer radius. The results of the model simulations indicated that the largest maximum probable peak horizontal ground acceleration at the site is approximately 0.83g (where g is the acceleration due to gravity). This acceleration rate was postulated to be the result of a MCE event of magnitude 6.9 on the Richter scale centered on the Newport-Inglewood Fault. An earthquake of magnitude 6.9, centered along the Newport-Inglewood Fault, is believed to have taken place in 1812 (U.S. Army Corps of Engineers, 1991). In addition, the results of the probabilistic analysis indicated that the peak horizontal ground acceleration corresponding to a 10 percent probability of exceedance in 50 and 100 years are 0.34g and 0.48g, respectively. Based on this information, Leighton and Associates postulate that it is highly probable that Dana Point Harbor will experience strong ground shaking within the next 50 years.

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils adopt the physical properties of a fluid when subjected to high intensity ground shaking. In general, the three conditions that need to exist in order for liquefaction to occur include a shallow groundwater table, low-density silty or fine sandy soils, and high intensity ground motion (Leighton and Associates, 1999). The project site has been proven to be within the radius of active faults capable of producing ground motion within the harbor, and the subsurface physical conditions exhibit both a shallow water table and sandy soils. These physical properties indicate that localized zones within the Dana Point Harbor Revitalization area may have the potential to liquefy, which could induce significant damages to harbor facilities.

7.0 BULKHEAD MONITORING INVESTIGATIONS AND MAINTENANCE REPAIR WORK

In an effort to monitor and assess the relative movement and deterioration of the concrete panel and riprap stone slope protection, a number of field level investigations and maintenance repairs have been performed throughout the harbor basin areas.

7.1 Damage Assessment History

Damage of the concrete panel revetment slabs was first noted in February 1971, eight months after the filling of the harbor. A subsequent underwater inspection performed in 1974 revealed that the maximum displacement of the panels had occurred within the intertidal zone adjacent to the base of the quay wall (Moore and Taber, 1980).

7.2 Concrete Panel Slope Protection Monitoring and Maintenance

In 1980, Moore and Taber conducted a detailed investigation of the distressed slope panels. The field investigation performed during this study included mapping of the displaced panels, observation of the panels at varying tidal levels, a close underwater examination, and in-situ sampling of the bearing soil under the slope panels.

Upon analyzing the field data collected over the course of the investigation, Moore and Taber hypothesized that the damages were primarily due to problems with the drainage conditions. They found that most of the filter fabric installed to control soil piping at panel joints and weep holes was clogged or disturbed. Consequently, free drainage was substantially impeded at some locations, while seepage velocities in other areas were sufficient enough to pipe fines from beneath the panels. Engineering calculations indicated that a hydraulic head of 5.5 feet acting behind a given panel would exert enough force to lift the panel. Assuming that the filter fabric is clogged with debris, this head differential can easily develop for short periods of time during tidal cycles that exhibit a tidal elevation greater than 5.5 feet, MLLW. Examination of the filter fabric during the 1980 investigation confirmed that the pores were partially clogged.

Once a given panel begins to lift, the rapid establishment of the drainage flow forces the fill material to migrate down slope where it settles in voids created by the uplift of the panel. The displaced sediment then supports the uplifted panel after the hydraulic forces have dissipated. Over many tidal cycles the continual migration of the underlying soils produces a void beneath the footing of the quay wall resulting in an accumulation of sediment near the mean lower low water vertical datum (Moore and Taber, 1980). The loss of support at the upper portions of the panel then cause the slab to break at the upper control joint where it then settles in the "sunken" position illustrated in Figure 9. As a result of this analysis, remedial measures in the form of

additional lateral joint drainage provisions and grouting techniques were employed to stabilize the distressed panels and prevent piping of backfill.

In addition to these prior investigations and maintenance repairs, the County of Orange has been monitoring the distressed slope panels' movements twice per year since October 1991. These surveys are performed by setting three pins (denoted as top, middle, and bottom on Figure 9) along a given distressed panel at the base of the quay wall and measuring their respective horizontal and lateral movements. These measurements can then be compared to prior surveys to determine the relative movement of the panels over time. The results of the monitoring effort to date indicate that the upper portions of the concrete panels, above the upper control joint, have moved slightly. However, these movements can be considered to be minimal. The monitoring and repair work locations conducted within the project area of interest to date are illustrated in Figure 10.

7.3 Stone Revetment Slope Protection Monitoring

Certain areas of the stone revetment slope protection sections within the Dana Point Harbor Revitalization Plan area have experienced some degradation. Deterioration was limited to areas of dislodged riprap (Noble Consultants, Inc., 1993). The current degree to which the riprap regions of the project area have been compromised has not been fully ascertained as a detailed underwater investigation of the project area was beyond the scope of this assessment.

8.0 BULKHEAD CONDITION EVALUATION

The Dana Point Harbor bulkhead system is a composite structure consisting of a relatively short height of vertical gravity-type retaining wall and a lower section of slope protection that extends from the intertidal zone to the basin bottom.

Because of the hard bottom conditions that characterize the local geology, the inner harbor boat basin areas were designed to be constructed in the dry. The earthen fill moles that forms the east, west, and southern boundary of the marina were used as dikes so that the interior basins could be dewatered and excavated (U.S. Army Corps of Engineers, 1986). The land areas adjacent to the boat basins were raised to their present elevation by placing over 12 feet of fill on top of the existing low lands and beach area. To protect the fill and form the outline of the waterfront, the entire perimeter of the inner harbor was also constructed in the dry. This enabled the construction of the unique bulkhead design cross section that was used. Figure 11 illustrates the Dana Point Harbor area in 1964 showing the conditions prior to construction of the harbor and the then proposed improvements.

8.1 Bulkhead Performance

Since the completion of the harbor improvements in 1970, the bulkhead system has performed reasonably well. The primary purpose of the quay wall/ slope protection system is to retain the fill soils and provide the necessary physical and visual separation requirements between the waterside improvements and the landward development and ancillary facilities. In its most simple terms, the bulkhead system was designed to allow elevated and developable land close to the waters edge. In order to accomplish this intended purpose, several critical design elements had to be implemented. As a result, the logistical constraints on the bulkhead system mandated that the bulkhead prevent leakage of backfill into the marina, provided the necessary lateral

resistance to lateral earth pressures, and stabilized the on grade foundations of the adjacent buildings and infrastructure.

As previously discussed, problems with the slope protection component of the bulkhead were experienced shortly after the completion of the harbor. Hydrostatic loads due to elevation differences between groundwater elevation and low tide conditions exacerbated soil seepage problems and movement of some of the rigid concrete slope protection slabs. The County has implemented a repair and monitoring program that has been effective in responding to repair and maintenance needs.

As with any marine structure, the continual exposure of the concrete quay wall to seawater will result in corrosion of embedded reinforcement steel, and cracking and spalling of concrete (See Figure 5). A conventional program of monitoring, repair, and maintenance will generally address these issues.

It is estimated that the bulkhead will function as intended provided that all weep holes are functional, the seal between the quay wall footing and slope protection is maintained, and the slope protection soil retention system is not compromised. As the existing improvements age with time, maintenance efforts will continue to be necessary to monitor for avenues of backfill leakage, slope protection deterioration, and weep hole clogging. Additionally, the marine exposure of the quay wall will require regular repair and maintenance to address wall cracks, spall areas, and sections of corroded reinforcement steel. As mentioned, a vigilant repair and maintenance system, similar to that currently practiced by the County, can be continued to prolong bulkhead life and preserve function use for the new improvements. In this manner, it is estimated that the bulkhead system can be preserved to prolong its service life.

8.2 Seismic Stability Assessment

It is estimated that the existing bulkhead system may be vulnerable to damages resulting from a maximum credible earthquake (MCE) event. The quay wall is a slab on grade retaining wall that is supported by artificial undocumented fill. Consequently, the lateral stability of the quay wall and the integrity of slope protection is dependent upon the underlying soil strengths and their ability to remain stable when subjected to dynamic loads. Based upon a previous assessment by Leighton and Associates (1999), it is estimated that the potential for liquefaction to occur within localized zones of the Dana Point Harbor fill soils may exist.

The sensitivity of the bulkhead system on its foundation support may be illustrated by the recent failure that occurred at a section of bulkhead at the Orange County Marine Institute (OCMI) (Leighton and Associates, 2001). Construction of a deep excavation immediately landward of the bulkhead resulted in a near catastrophic loss of foundation soils and the subsequent potential collapse of the quay wall when seawater and underlying soils flowed into the excavation pit. Emergency remedial measures were implemented to save approximately 600 feet of the threatened quay wall from collapse. This experience illustrates the vulnerability of the bulkhead integrity if excess hydrostatic pressures build and the soil retention features of the bulkhead system are compromised.

Preliminary calculations using seismic design criteria recommended by Leighton and Associates (1999) for the OCMI improvements, indicate that the existing quay wall may have insufficient resistance to sliding when exposed to earthquake soil loading. Furthermore, if the underlying soils that support the quay wall and slope protection components liquefy during a significant earthquake, substantial soil deformation and structural movement could occur. Figure 12

provides a schematic illustration of possible damage scenarios that could occur as a result of an extreme MCE event.

9.0 SUMMARY AND RECOMMENDATIONS

Based upon the review of available information and preliminary analysis, the following conclusions are made with respect to the assessment of the existing bulkhead system:

1. Under normal static load conditions, the bulkhead system performs as an adequate earth retention system.

2. Presence of groundwater and the bulkhead's exposure to the continual tidal cycles has resulted in problems with differential hydrostatic head conditions and resultant soil leakage in some areas. Loss of backfill from behind or beneath waterfront bulkheads can be a chronic problem. Because the problem can develop slowly over time and can be hidden from view, persistent paths of soil seepage and the extent of subsurface voids that exist, or are forming, can go unnoticed. Therefore, efforts to determine the integrity and existing condition of the backfill should be considered.

3. It is estimated that under static load conditions, the existing bulkhead system can be preserved in place for many years using conventional repair and maintenance practices.

4. When viewed from a dynamic load perspective, the bulkhead is estimated to be vulnerable to significant damages as a result of ground motions induced by an earthquake. The existing structure is considered to be deficient in its ability to withstand the increased soil pressures caused by ground shaking. Furthermore, the slab on grade construction is not reliable when exposed to a MCE event if significant loss of soil strengths occurs as a result of soil liquefaction. Under such conditions significant bulkhead movement, substantial loss of backfill, and/or slope failures could occur to the severe detriment of the landside facilities.

5. At areas where significant earthquake damages could be accepted and repairs to the bulkhead system and adjacent land areas tolerated, maintenance of the existing bulkhead system may be appropriate. However, where new development is being contemplated adjacent to water areas, it is recommended that the existing structure be analyzed in more detail for seismic stability. Where deficiencies are concluded to exist, replacement structures may be appropriate. At a minimum, seismic retrofit methods to prevent loss of backfill and ensure foundation support should be considered.

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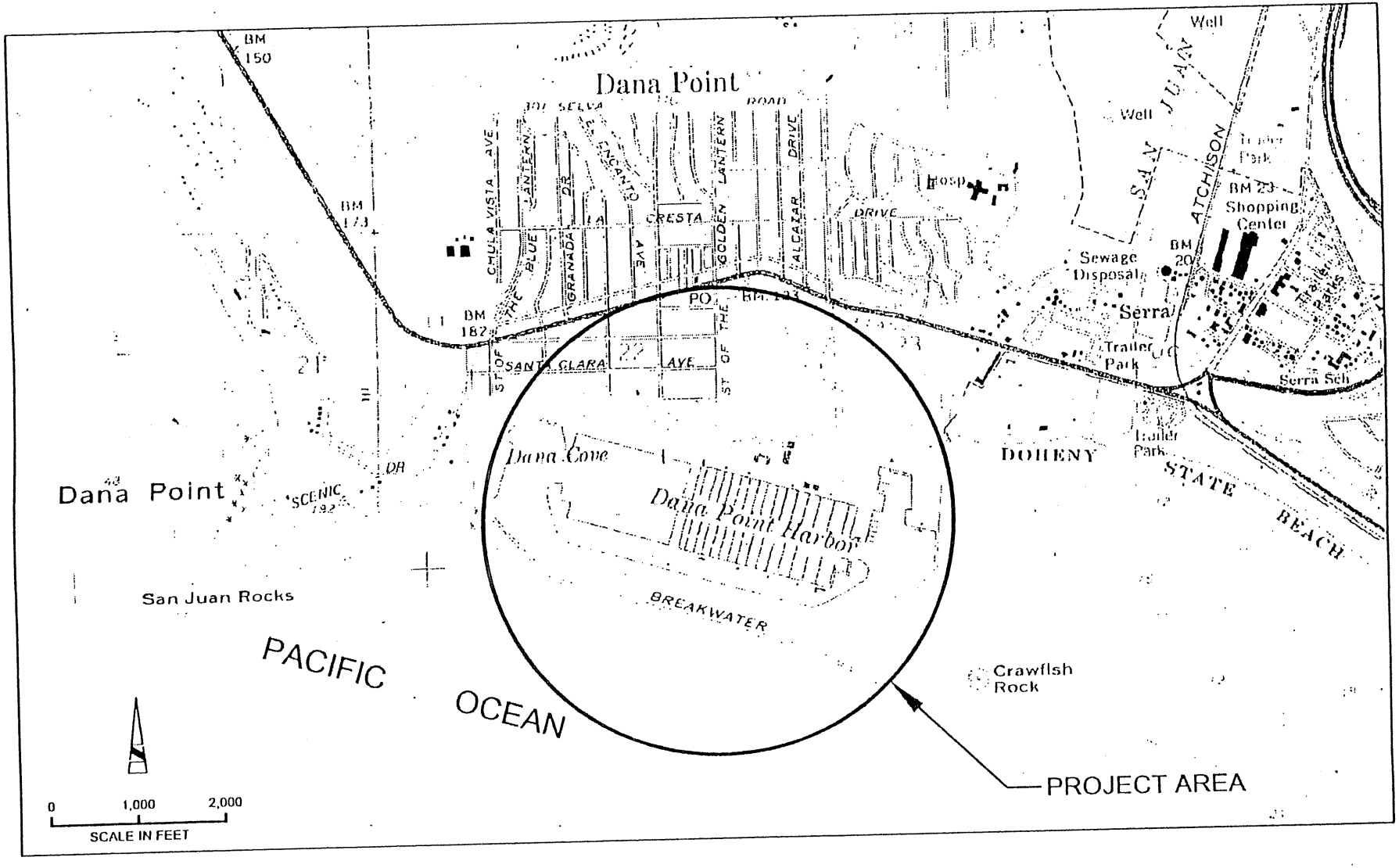

Dana Point Harbor Vicinity Map

Source: USGS Dana Point Quadrangle, photorevised 1975

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Figure 1

AREA OF INTEREST - EAST BASIN NORTH BULKHEAD

WEST BASIN

EAST BASIN

WEST CHANNEL & MOORING

EAST CHANNEL & MOORING BASIN

DANA ISLAND

MAIN CHANNEL

ENTRANCE CHANNEL

EAST BREAKWATER

WEST BREAKWATER

PACIFIC OCEAN

400 0 400 800

SCALE FEET

Dana Point Harbor Plan Layout

NOBLE CONSULTANTS, INC.

Figure 2

QUAY WALL

BOAT RAMP (230 FEET)

A/8

-8

-10

QUAY WALL

A/6

A/4

SLOPE PROTECTION

A/7

A/6

CONCRETE PANEL
SLOPE PROTECTION
(~2,300 FEET)

EAST CHANNEL &
MOORING BASIN

STONE REVETMENT
SLOPE PROTECTION
(~2,570 FEET)

-12

-15

-16

EAST BASIN

BREAKWATER

N

200 0 200 400

SCALE FEET

Bulkhead Plan

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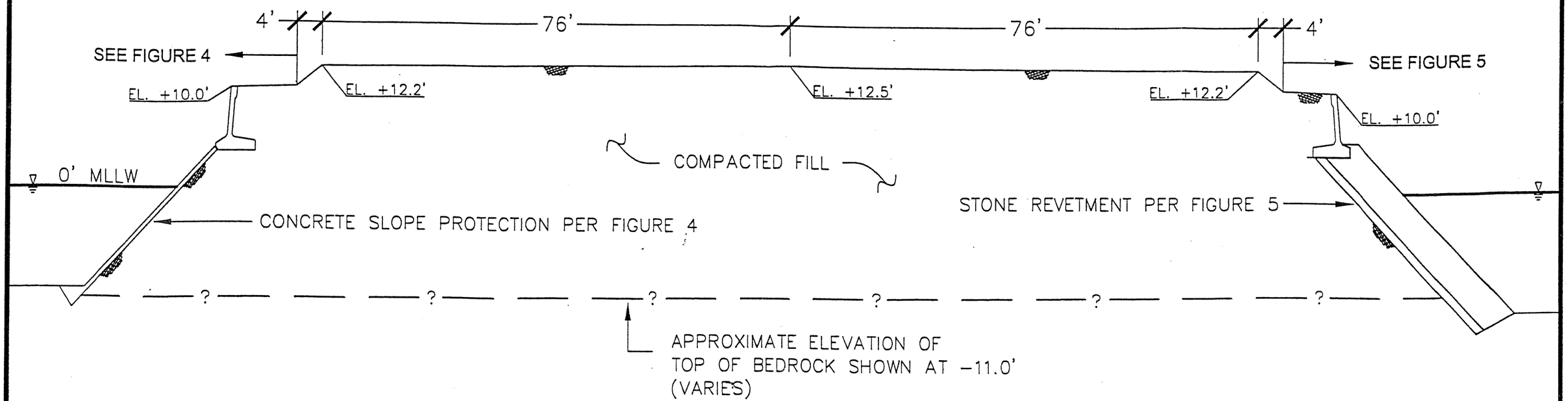
Figure 3



A/7 SECTION DETAIL

SCALE: HORIZONTAL 1" = 16'; VERTICAL 1" = 10'

Typical Section Through Phase 2 Plaza Commercial Core Improvements

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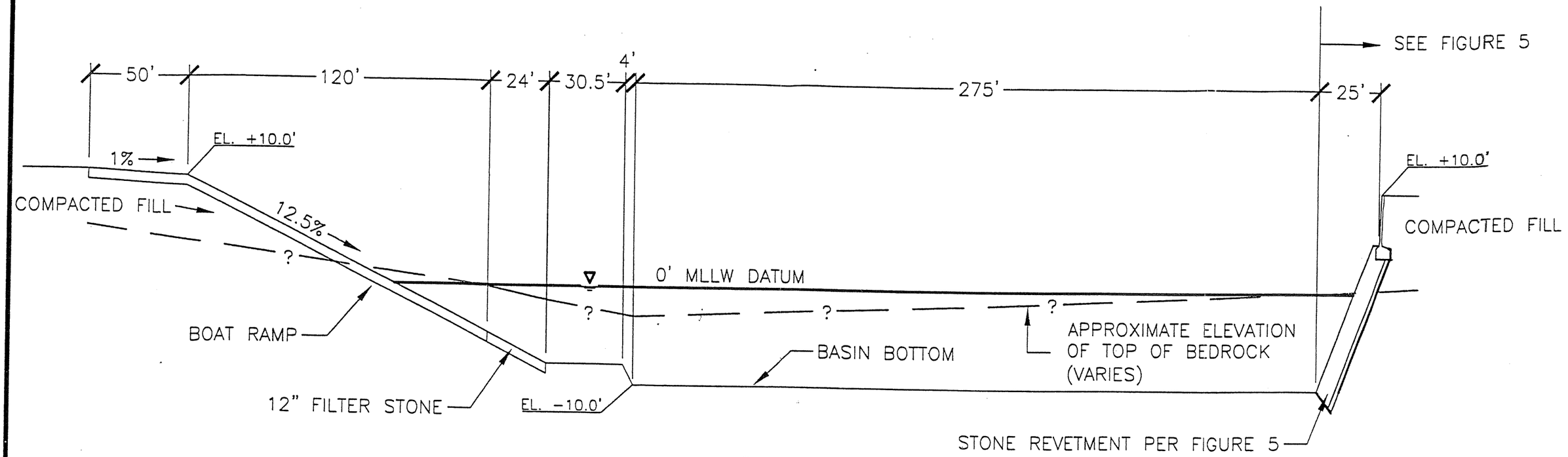
Figure 7



A/8 SECTION DETAIL

SCALE: HORIZONETAL 1" = 40'; VERTICAL 1" = 10'

Typical Section at Boat Ramp

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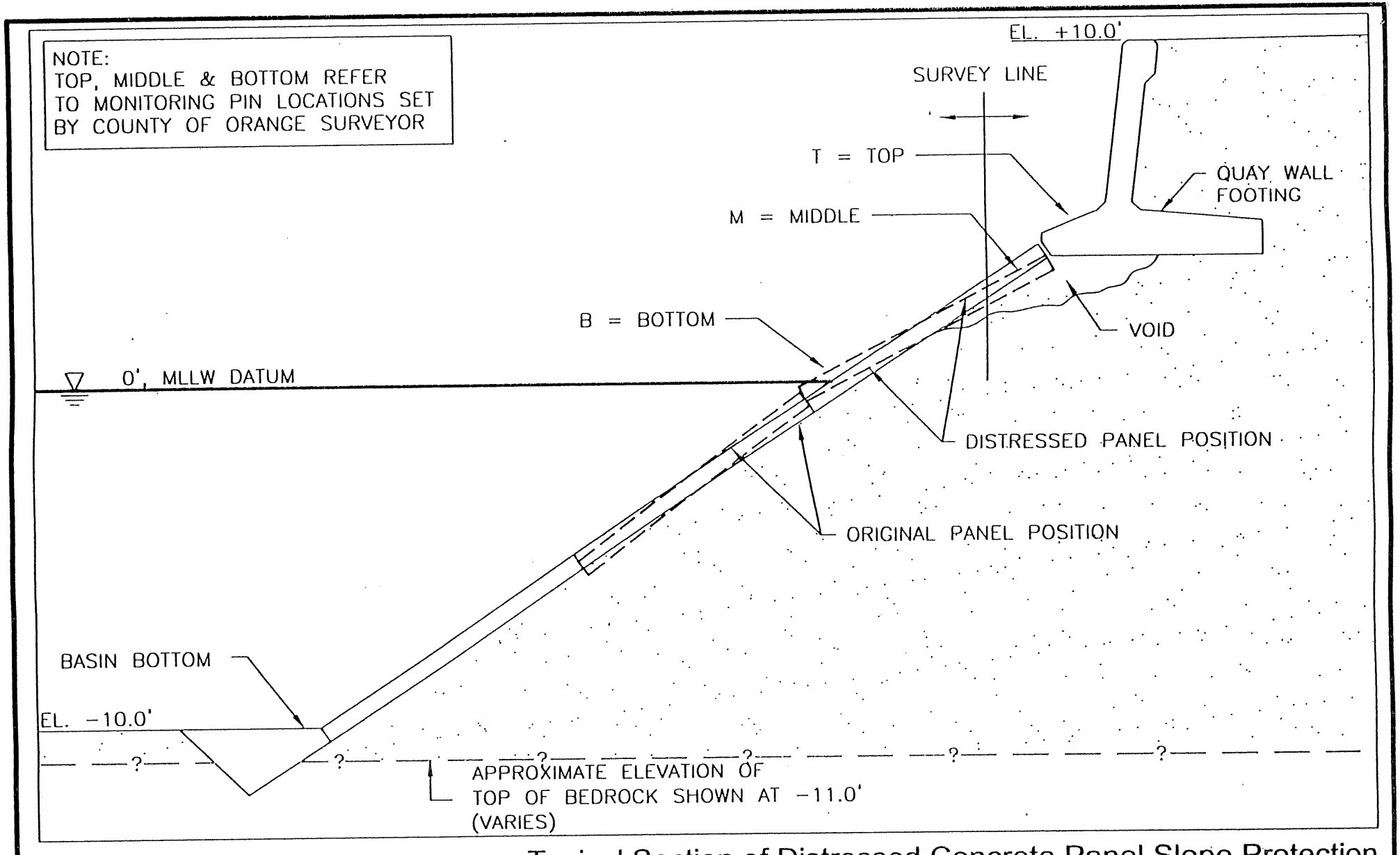
Figure 8



Typical Section of Distressed Concrete Panel Slope Protection

SCALE: 1 INCH = 4 FEET

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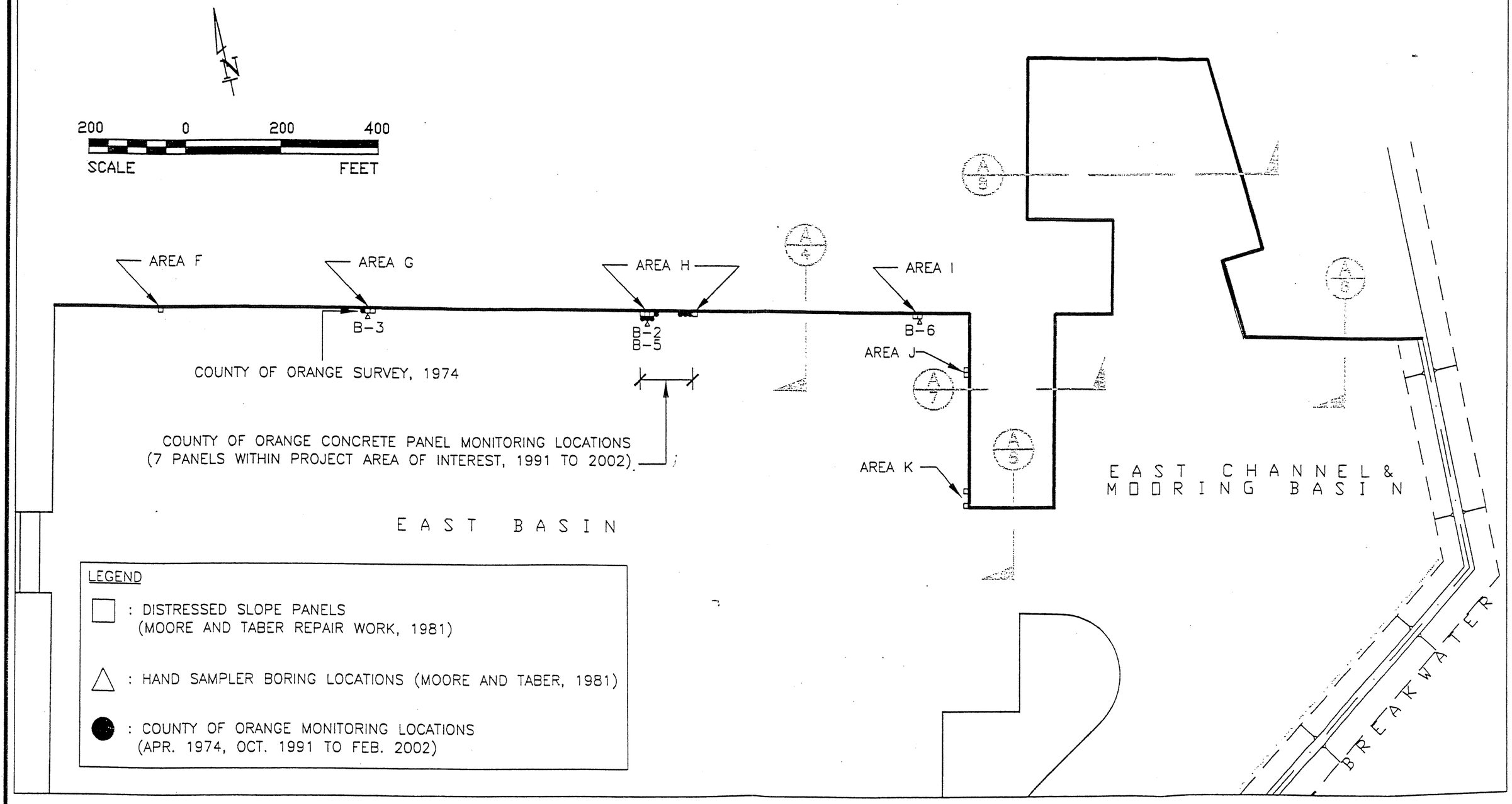
Figure 9



East Basin Concrete Panel Slope Protection Maintenance Repairs and Monitoring Locations

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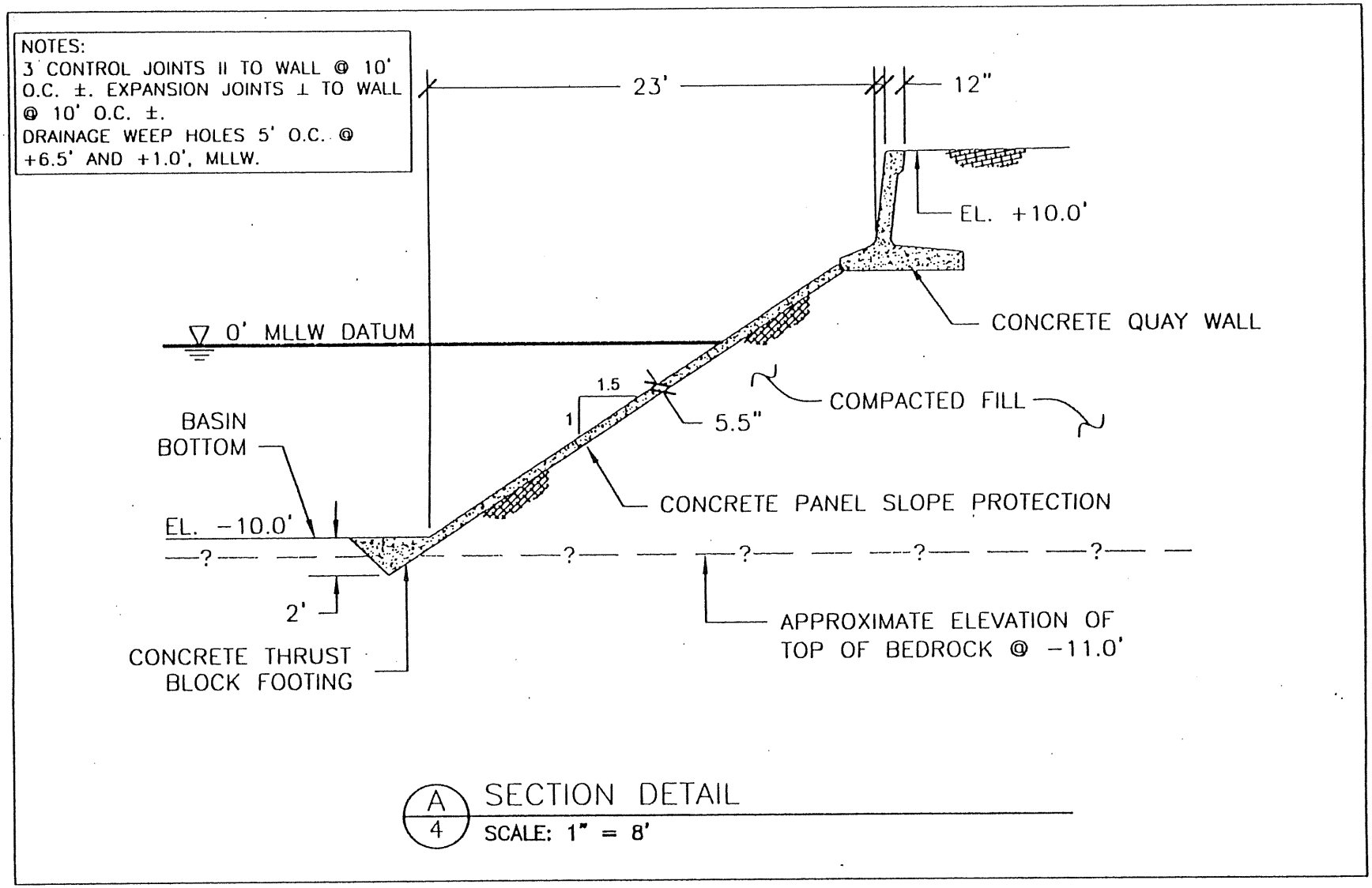
Figure 10



Typical Bulkhead Section with Concrete Panel Slope Protection

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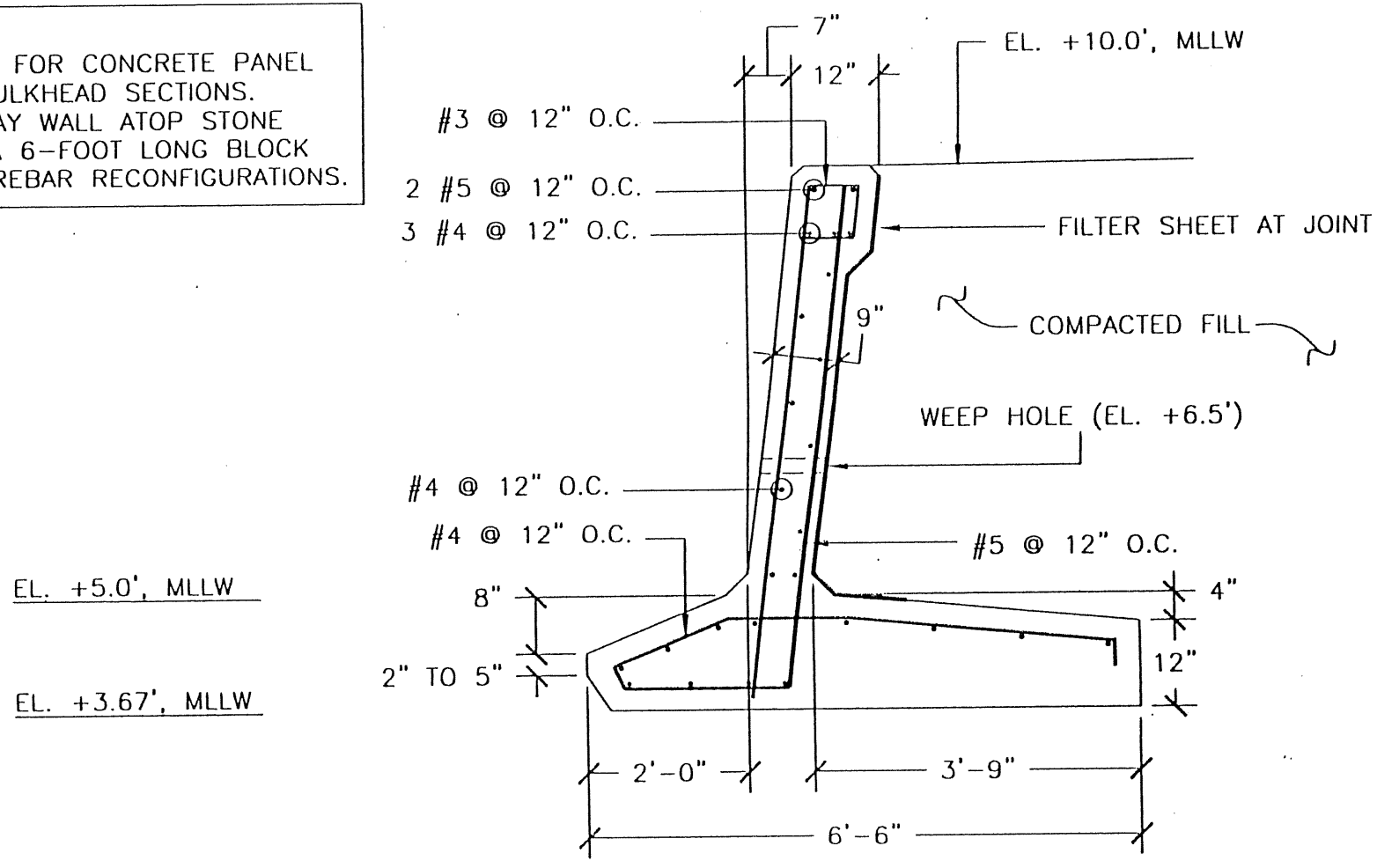
Figure 4

NOTE:
QUAY WALL SHOWN IS FOR CONCRETE PANEL SLOPE PROTECTION BULKHEAD SECTIONS. MODIFICATIONS TO QUAY WALL ATOP STONE REVETMENT INCLUDE A 6-FOOT LONG BLOCK FOOTING AND MINOR REBAR RECONFIGURATIONS.



A/5 QUAY WALL DETAIL
SCALE: 1" ≅ 2'

Typical Concrete Quay Wall Atop Concrete Panel Slope Protection

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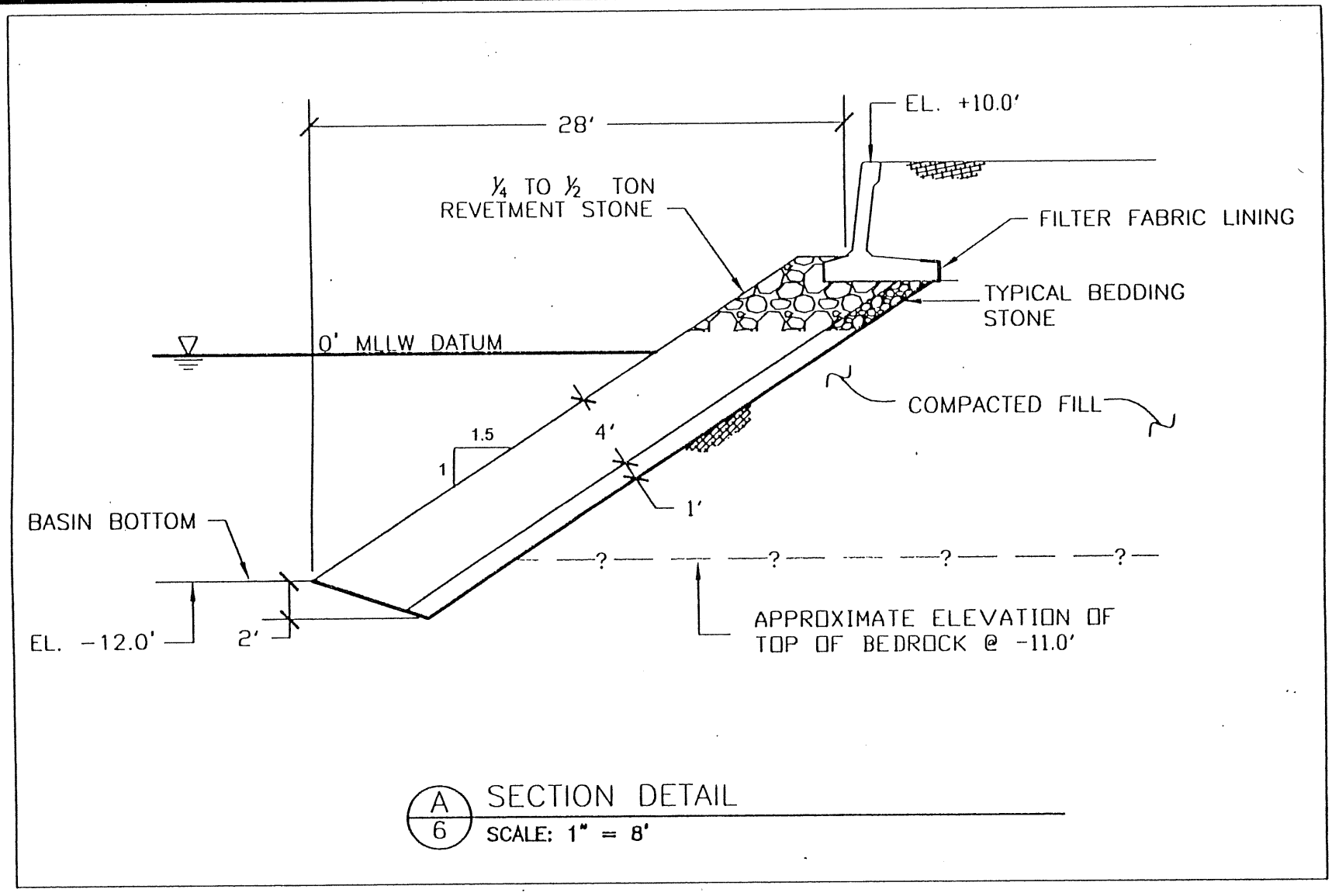
Figure 5



Typical Bulkhead Section with Stone Revetment Slope Protection

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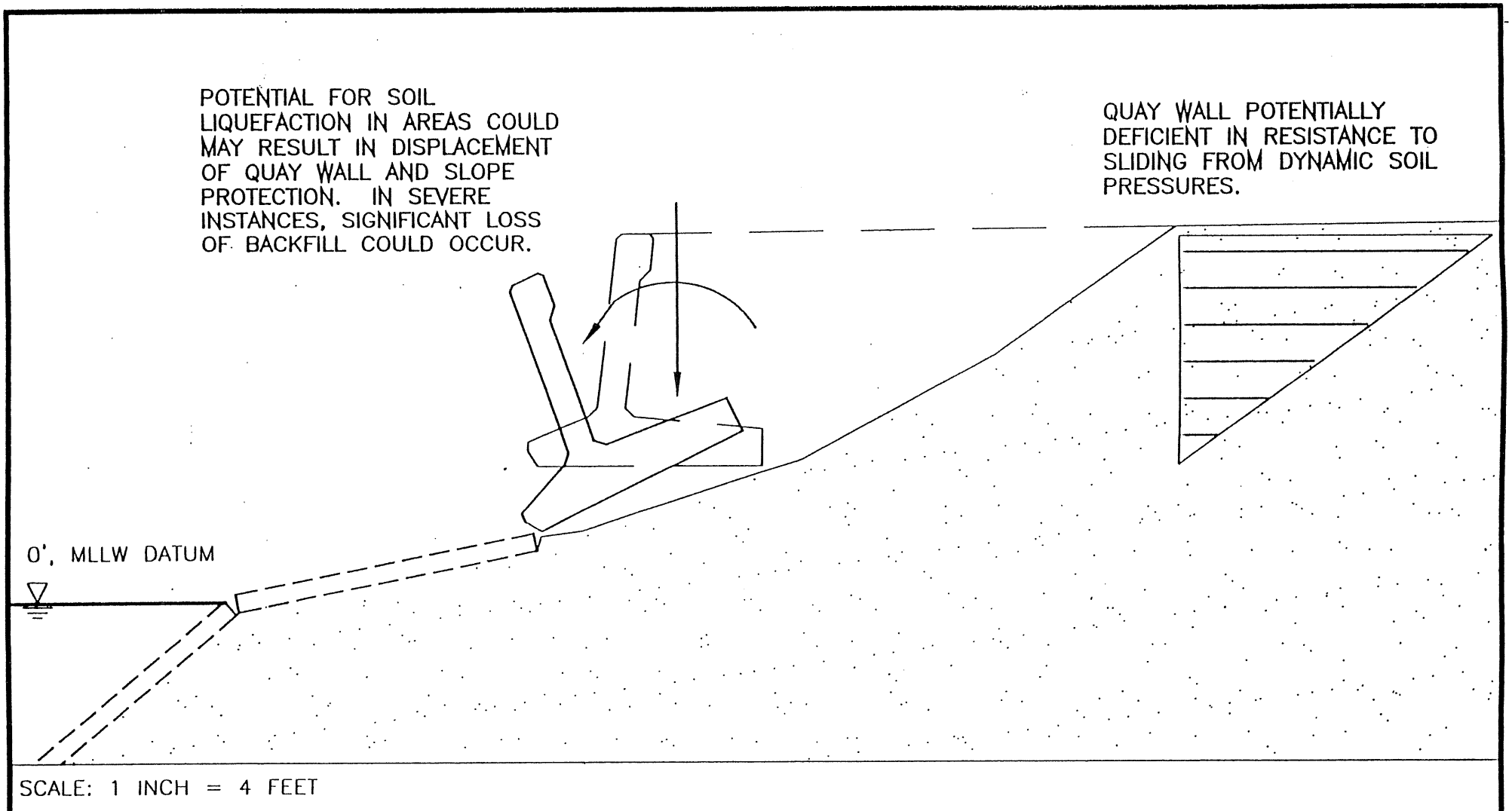
Figure 6



Siesmic Stability Issues Recommended for Further Study to Confirm Applicability

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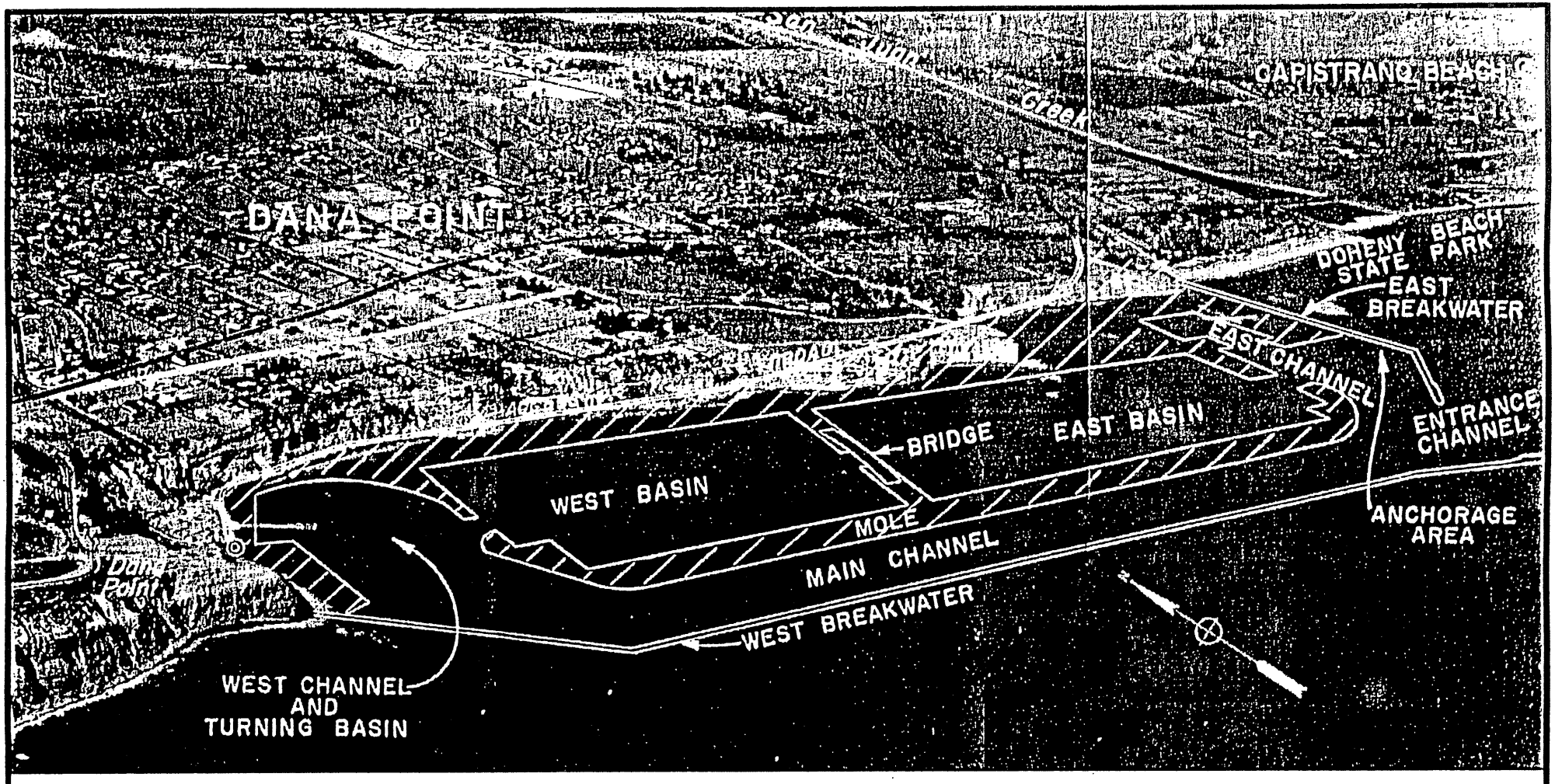
Figure 12



Dana Point Harbor Area in 1964

Source: U.S. Army, 1966

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Figure 11