GEOTECHNICAL REPORT
THE INN AT SUNSET CLIFFS
SAN DIEGO, CALIFORNIA

Prepared for
PROJECT DESIGN CONSULTANTS
San Diego, California

Prepared by
TERRACOSTA CONSULTING GROUP, INC.
San Diego, California

Project No. 2317-01
December 24, 2020

TerraCosta Consulting Group

Geotechnical Engineering
Coastal Engineering
Maritime Engineering

Project No. 2317-01
December 24, 2020

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GEOTECHNICAL REPORT
THE INN AT SUNSET CLIFFS
SAN DIEGO, CALIFORNIA

Dear Mr. Morrow:

TerraCosta Consulting Group, Inc. (TerraCosta) is pleased to submit this geotechnical report addressing the recently proposed tied-back secant pile wall landward of and along the eastern edge of the existing lower deck at The Inn at Sunset Cliffs (Inn) in the Ocean Beach area of San Diego, California.

The accompanying report describes our findings pertaining to the general coastal processes in the area, and the geotechnical conditions that pertain to the proposed seawall. Design criteria for the proposed vertical seawall are also provided.

This report also addresses the impact of tsunamis and also takes into consideration the influence of new MSLR research results summarized by the Ocean Protection Council (OPC 2018), which in turn led the California Coastal Commission (CCC 2018) update of its CCC (2015) guidance.

We appreciate the opportunity to work with you on this project. If you have any questions or require additional information, please give us a call.

Very truly yours,
TERRACOSTA CONSULTING GROUP, INC.

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Attachments

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1 INTRODUCTION AND PROJECT DESCRIPTION

The Inn at Sunset Cliffs (The Inn) is located at the southwest corner of Sunset Cliffs Boulevard and Point Loma Avenue within the community of Ocean Beach in San Diego, California (Vicinity Map, Figure 1). The Inn is further located within the Sunset Cliffs area of San Diego, and specifically within the City's Sunset Cliffs Shoreline and Upper Cliff Stabilization Project; Document No. 76649.

This project has been revised to move the seawall to the eastern, or land side, of the existing lower concrete deck in order to eliminate past concerns over the possible absence of necessary permits for the construction of the original deck, which was likely constructed in the early to mid-1970s. The more landward location of the now proposed project also simplifies the construction of the secant pile wall, eliminating most if not all of the debris that the overlapping drilled piers would extend through, while at the same time potentially providing new low-tide habitat after the 67-year-old seawall and its associated backfill are removed. The currently proposed project has been designed to simplify project approval through both the City of San Diego and the California Coastal Commission.

The proposed tied-back secant pile wall project consists of overlapping drilled piers with alternate piers reinforced with a higher strength concrete to provide structural capacity. Tieback anchors will be centered in every third non-reinforced drilled pier. The secant pile walls would be constructed landward and along the eastern edge of the existing lower deck. The exposed wall surface would be architecturally carved and colored to resemble the adjacent natural geologic exposures and adjacent walls to the north. All existing walls, debris, and concrete stairs, slab and infill seaward of the proposed wall would be removed to potentially create some low-tide habitat at the base of the new wall. Any Bay Point Formation exposed at the base of the wall during construction would be allowed to erode naturally. Additional architectural surface may be applied as this erosion progresses to a critical point.

2 GENERAL SITE CONDITIONS

2.1 Existing Improvements

The subject site is bounded by Sunset Cliffs Boulevard on the east, Point Loma Avenue on the north, private development to the south, and coastal bluffs fronting the Pacific Ocean on the west. Improvements at the subject site include two 2-story commercial buildings, and a swimming pool situated between the buildings, all of which are located at street level, as shown on the Site Map (Figure 2). Additionally, a lower level concrete deck, accessible by stairway, is located west of the buildings and swimming pool, and is bound on the west by an existing aging seawall. The seawall is variable in height and consists of an original masonry block wall supported by a cast-in-place concrete foundation. Several additional foundations and concrete panels have been incorporated as repairs over the past 30+ years.

We have reviewed several historical photos dating back to 1939. The below 1953 aerial photo shows The Inn under construction and the seawall and lower viewing area constructed concurrently (Photo 1). Thus, all of the existing improvements pre-date the California Coastal Act.

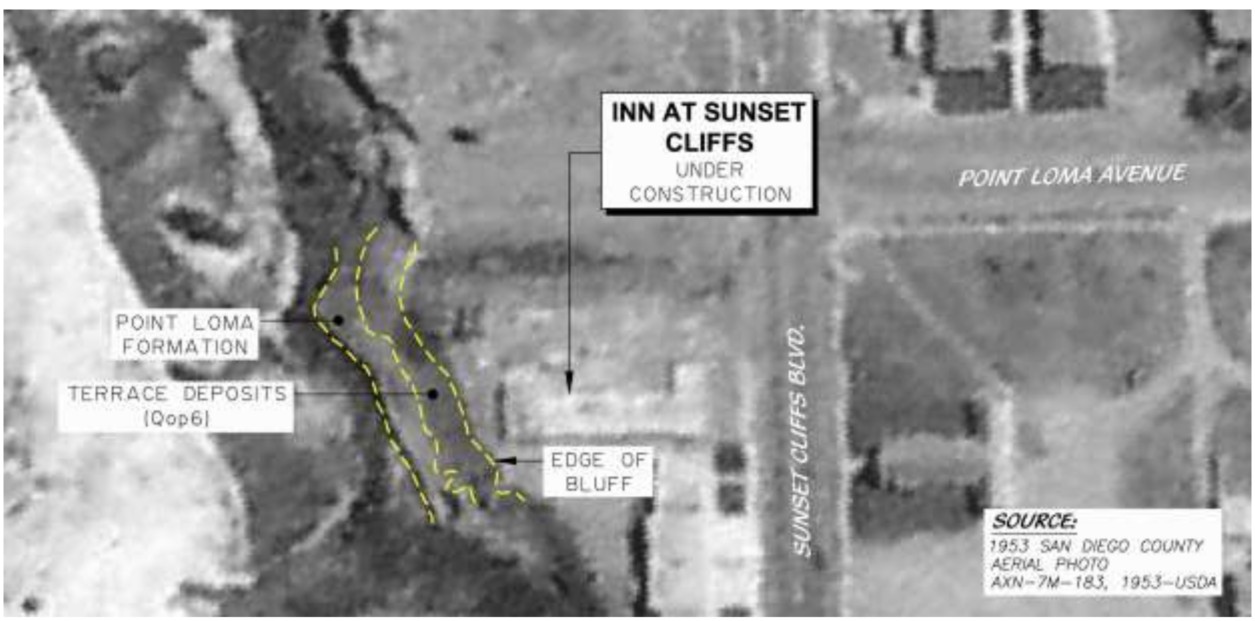


Photo 1 **1953 Aerial**

Photos taken from 1978 through 1982 during the construction of the Sunset Cliffs Shoreline Stabilization Project show the lower cliff-forming bedrock unit (Photos 2 and 3), the top of which is estimated to be near elevation +5 to +12 feet (MSL), on top of which the variable height masonry block wall was constructed in 1953. Although relatively erosion-resistant, marine erosion has continued to cause the lower cliff-forming geologic unit to retreat, undermining the existing wall foundation in 1990, in 2003, 2015, and again in 2018, causing the localized loss of wall backfill and necessitating the previously described foundation/wall maintenance.

Naturally occurring fractures in this lower bedrock unit typically result in differential erosion and the formation of surge channels, which, over time, can undermine existing foundations and eventually breach a seawall, creating a void in the backfill behind the wall. This occurred in 1990, 2003, 2015, and again in 2018, and is typical of many of the older seawalls along the Point Loma coastline that, after several decades, have become undermined, eventually resulting in the loss of wall backfill and, worst case, failure of the wall.

Photo 2 **Photo Date: 1982**

Photo 3 **Photo Date: 1982**

2.2 Geologic Setting

Point Loma is a 6-mile-long peninsular promontory (Photo 4), extending southerly from the low land adjacent to the mouth of the San Diego River. The Point Loma coastal bluffs are bordered by a narrow wave-abraded Quaternary-age terrace or bench, with elevations ranging from 25 to 95 feet MSL. Wave impact erosion has etched out the less resistant rock along faults and fractures in the coastal bluff, creating the shallow coves and sea caves that punctuate the Point Loma coastline. The more resistant rocks of the Point Loma Formation form the lower cliffed section of the coastal bluff and shore platform, which extends seaward. The relatively flat surface of the modern-day abrasion platform is interrupted by isolated remnants of more erosion-resistant rock, which have formed "sea stacks" and topographic highs. Further seaward, the abrasion platform becomes progressively deeper, and is locally incised by surge channels formed along the trends of major joint sets or faults.

Photo 4 **Photo Date: 1974**

Small pocket beaches exist in areas where sufficient sand is available. However, since the storms in 1980, little sand has existed within the pocket beaches adjacent the site, thereby exposing the bedrock shore platform, which comprises the gently seaward sloping sea floor fronting the site.

2.3 Subsurface Conditions

Two geologic formations are present in the general area of The Inn. The Point Loma Formation is a member of the 70 to 80 million year old Cretaceous-age Rosario Group, which extends from southern San Diego County to northern Baja, California. The Quaternary terrace deposits, which forms the upper coastal bluff terrace deposits, consists of both marine and non-marine, poorly consolidated, fine- and medium-grained, red to pale brown, fossiliferous sandstone. Minor deposits of local overburden soils include colluvium and artificial fill soils. The following paragraphs describe these geologic units from oldest to youngest. The local (site) geology is presented in geologic cross section view through the site on Figures 3A through 5B.

<u>Point Loma Formation:</u> The Cretaceous-age Point Loma Formation is an approximately 900-foot-thick sedimentary rock layer that discontinuously crops out in coastal areas of northern Baja, California to as far north as Carlsbad (Kennedy, 1975). At the site, it forms the lower, more resistant parts of the sea cliff. This geologic unit generally dips to the northeast, and also comprises the gently seaward-sloping seafloor adjacent the wall. The foreshore slope in this area is on the order of 1 in 60.

The Point Loma Formation consists of well-indurated marine sediments deposited by an offshore and deep-water submarine fan. Exposures of the Point Loma Formation in the sea cliff generally consist of a massive, well indurated, dark gray siltstone, with coarse to medium “gritty” sandstone and partially-cemented siltstone interbeds.

Although the Point Loma Formation is generally very resistant to wave erosion, some of the highly-fractured shale interbeds, especially those containing significant amounts of clay, have been subjected to accelerated wave erosion, resulting in upwards of 10+ feet of sea cliff retreat at adjacent properties.

<u>Old Paralic Deposits:</u> Previously referred to as the Bay Point Formation, this unit is approximately 13± feet in thickness in the site vicinity, and forms the upper sloping part of the coastal bluff above approximate elevation 12 feet (MSLD). The Bay Point Formation is a Pleistocene-age (approximately 120,000 years old) terrace deposit that consists of nearshore marine, poorly-consolidated, fine- to medium-grained sandstone considerably more susceptible to erosion than the underlying Point Loma Formation. The contact between the Point Loma Formation and the Bay Point Formation is concealed by the existing wall at this location. At adjacent sites, and presumably behind the wall, the Bay Point Formation extends to the bluff top near elevation 25 feet, with slope inclinations generally ranging from 40 to 60 degrees from the horizontal and locally near vertical.

More contemporary mapping has broken the Bay Point Formation into smaller distinct units (Figure 6). The current “Geologic Map of the San Diego 30’ x 60’ Quadrangle” identifies this formational unit as “Old Paralic Deposits” (Q_{OP6}, middle to late Pleistocene), consisting of mostly poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone, and conglomerate, resting on the 22 m to 23 m Nestor Terrace.

The Bay Point Formation may form at least a small part of the sloping portion of the bluff exposed between the lower viewing deck and the bluff-top deck. However, on site, most, if not all, of the slope is covered by a thin mantle of sandy fill soils.

2.4 Geologic Structure

The Point Loma coastline has been affected by regional tectonic forces, by the Rose Canyon fault zone, and by tectonic regimes, which pre-date the Rose Canyon fault zone (Fischer and Mills, 1991; Greene and Kennedy, 1981). Coastal warping associated with the current tectonic regime has gently tilted the bedding and shore platform approximately 2 to 6 degrees to the northeast. Episodes of faulting and long-continued tectonic stresses have resulted in literally thousands of visible joints, fractures, and shear zones having both micro- and large-scale variations in erosion potential that also predominantly trend in a northeast direction.

2.5 Groundwater

Localized seepage was observed nearby in both the sea coves and sea caves. The groundwater typically migrates through the permeable joints and fractures within the Point Loma Formation. As the water migrates through these joints and fractures, the cementing agents within the rock are partially dissolved, further weakening the rock along the joint or fracture.

A contributor to the erosion of coastal bluffs is the flow of groundwater along the contact between the relatively pervious, moderately-consolidated coastal terrace deposits, and the well-consolidated, less pervious, Cretaceous formations that form the lower sea cliffs. The likely sources of this groundwater are: 1) natural groundwater migration from highland areas to the east of the terrace, and 2) infiltration of the terrace surface by rainfall, and by agricultural and residential irrigation water. Typically, the volume of groundwater exiting the bluff face in the site area varies from location to location, and between seasons, even during drought years.

Groundwater seepage exiting the bluff face on top of the Cretaceous-age sediments tends to cause spring sapping within the terrace deposits and the potential for the formation of solution cavities along faults, joints, and bedding planes, locally accelerating marine erosion in these areas. Although a significant concern affecting other parts of San Diego County's coastline, groundwater typically does not play a significant role in destabilizing the Cretaceous-age coastal sediments. However, whenever possible, it is prudent to eliminate this potential source of increased coastal erosion.

2.6 **Geologic Hazards**

2.6.1 *Seismicity/Ground Shaking*

The project area is located in a moderately-active seismic region of Southern California that is subject to moderate to strong shaking from nearby and distant earthquakes. Ground shaking from earthquakes on six major active fault zones could affect the site. These include the Rose Canyon, Coronado Bank, San Diego Trough, San Clemente, Elsinore, and San Jacinto/Superstition Hills Fault Zones. The nearest of these, the Rose Canyon Fault, parallels the shoreline and is located approximately 7 kilometers (about 4.4 miles) east of the site. The maximum credible earthquake for the Rose Canyon Fault is considered to be Magnitude 7. The maximum probable earthquake for this fault has been estimated at Magnitude 6½.

2.6.2 *Ground Rupture*

Our review of currently available published geologic mapping indicates no known faults on or immediately adjacent to the site. Additionally, although a single "fracture" is indicated in the exposed Point Loma Formation bedrock outcrop (please see the "Plot Plan" – GEI February 2004 report – Appendix A), no faults are indicated by GEI in either their report text or their mapping of the site.

Because no faults have been mapped on the site, we believe the potential for ground rupture to be very low.

2.6.3 *Tsunamis*

Although they are relatively rare events, tsunamis and some narrow classifications of seiches should be considered hazards at this open-ocean shoreline site. Our review of the Tsunami

Inundation Map (Figures 7a and 7b) for Emergency Planning, Point Loma Quadrangle, dated June 1, 2009, is described in more detail in Section 4 of this report.

2.6.4 *Liquefaction*

Based on site and subsurface conditions, it is our opinion that the potential for liquefaction of subsurface soils at the site is negligible.

2.6.5 *Slope Stability/Coastal Erosion*

The California Building Code requires that both graded and natural slopes in new development have a minimum factor of safety of 1.5 against slope instability; and, for steeper natural slopes, including coastal bluffs, any new slope-top or bluff-top improvements must be located behind the 1.5 factor of safety line. The stability of coastal bluffs is also affected by marine erosion, which tends to steepen the face of the coastal bluff, reducing its stability. Moreover, bluff-top setbacks usually include annualized erosion rates over a period of 50 to 75 years as another bluff-top setback criteria.

As indicated above in Section 2.3 – Subsurface Conditions, two geologic formations underlie the site vicinity, with the lower cliff-forming unit consisting of the relatively strong and erosion-resistant 70- to 80-million year old Point Loma Formation, which extends up to an estimated elevation +12 feet at the site. This geologic unit is overlain by the Pleistocene-age terrace deposits estimated to be approximately 120,000 years old, which are also reasonably strong, although much less erosion-resistant, extending up to the contemporary ground surface near elevation 27 feet, resulting in about a 15-foot-thick section of terrace deposits.

A stability analysis of the natural geologic slopes (without the protective seawall and backfill) was completed for the site. Our analysis indicated that the slope has factors of safety ranging from 1.4 against a shallow failure within the terrace deposits, to a high of 4.0 against a deep-seated failure for gross stability. Obviously, the existence of the seawall increases the factors of safety. The results of our analysis are presented in Appendix B.

In spite of the relatively stable geologic conditions specific to slope stability, ongoing marine erosion has resulted in efforts to repair and mitigate coastal erosion within the Point Loma area dating back to the early 1900s. Between 1950 and 1960, the City of San Diego Engineering Department investigated cliff erosion along Sunset Cliffs, following numerous failures and requests for assistance by property owners. It was during this period, specifically 1953, that the original seawall was constructed to mitigate marine erosion that was affecting the entire Point Loma shoreline. At the site, even with a relatively high factor of safety against slope instability, in the absence of the seawall, the bluff-top improvements (including the two buildings) are at risk of damage from coastal erosion, with the southerly building at imminent risk (absent the seawall), with a reasonable probability of storm-induced damage occurring within the next two years.

In examining Figure 2, which shows the existing top-of-bluff from both GEI's and TerraCosta's measurements, the minimum distance from the top-of-bluff to the southwestern corner of the southern building is approximately 13 feet measured from GEI's reported top-of bluff, and only 8 feet measured from the 1953 aerial photograph (Photo 1).

In examining Figure 5, and in the absence of the proposed seawall (or the existing seawall, for that matter), the elevation of the Point Loma shelf rock is around +3 feet MSL, which means that on a daily basis, waves will break over the low elevation shelf rock up to the back step, and then up the steeply inclined Bay Point terrace deposits. If all of the existing fill soils shown on Figure 5 were removed (along with the seawall), a severe storm season similar to the 2015-16, 1997-98, or 1982-83 storm seasons could easily cause over 10 feet of bluff-top erosion. With the building only 8 feet back from the top-of-bluff and a low elevation shelf rock that allows waves to run up the upper sloping terrace deposits on a daily basis, the steeply inclined terrace deposits will quickly erode, damaging the southerly building during the first storm season. It is for this reason that the seawall remains critical to the protection of the southern building at the project.

3 COASTAL BLUFF GEOMORPHOLOGY

3.1 Terminology

The geomorphology of a typical Point Loma sea cliff is shown in Figure 8. Depicted on Figure 8 are the shore platform, a lower, near-vertical cliffed surface called the sea cliff, and an upper-bluff slope generally ranging in inclination between 35 and 80 degrees (measured from the horizontal). Little or no flat area is exposed above sea level at the base of the cliff, even at very low tides. The sea cliff is bounded at its landward edge by the coastal terrace deposits.

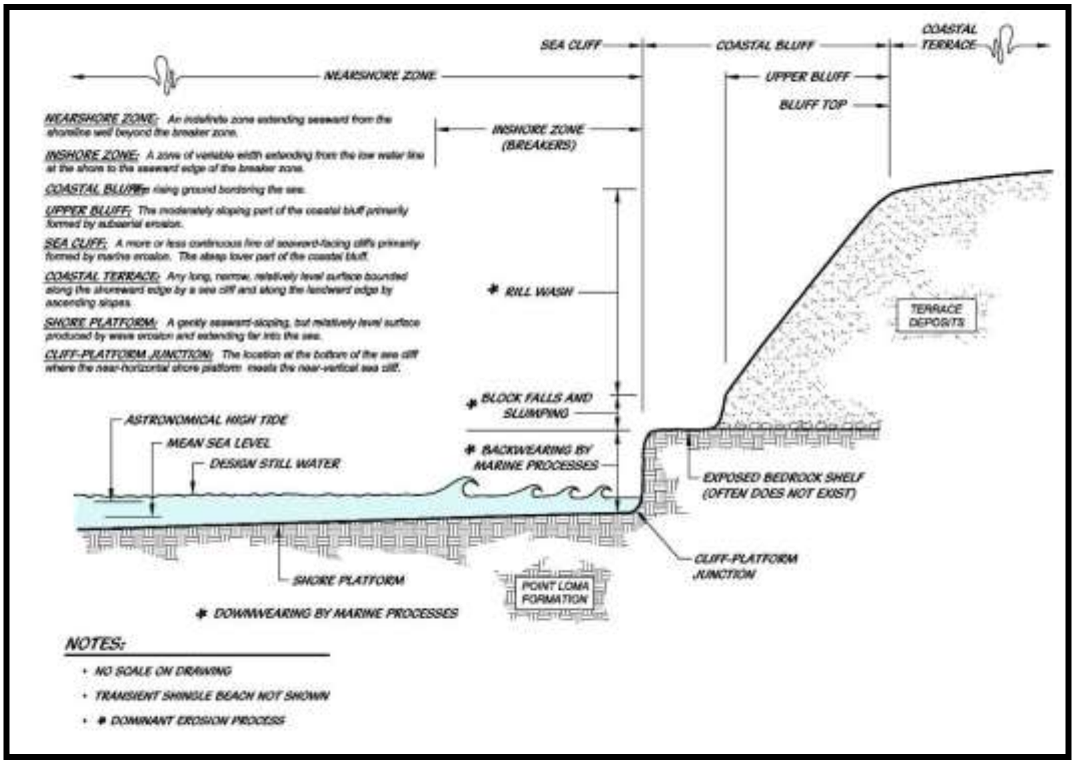


Figure 8. Typical Coastal Bluff Profile (Looking North up the Coast).

The term "bluff top" (or "top-of-bluff") is an important one, being essential to post-Coastal Act structure setback considerations. A simple definition for this term is the boundary between the upper bluff and the coastal terrace. A more rigorous definition of the term, as adopted by the CCC, follows (note that the definition uses the terms "cliff" and "bluff" interchangeably):

> *"A bluff or cliff is a scarp or steep face of rock, decomposed rock, sediment or soil resulting from erosion, faulting, folding or excavation of the land mass. The cliff or bluff may be simple planar or curved surface or it may be steplike in section. For the purposes of these guidelines, "cliff" or "bluff" is limited to those features having vertical relief of ten feet or more, and "seacliff" is a cliff whose toe is or may be subject to marine erosion. "Bluff edge" or "cliff edge" is the upper termination of a bluff, cliff or seacliff. When the top edge of the cliff is rounded away from the face of the cliff as a result of erosional processes related to the presence of the steep cliff face, the edge shall be defined as that point nearest the cliff beyond which the downward gradient of the land surface increases more or less continuously until it reaches the*

> *general gradient of the cliff. In a case where there is a steplike feature at the top of the cliff face, the landward edge of the topmost riser shall be taken to be the cliff edge."*

A geomorphic definition for top-of-bluff more clearly defines that intersection in which contemporary coastal regulators are most interested, as it provides a definition with which most any geologist is familiar and can easily locate. The more appropriate geomorphic definition follows:

> *The boundary between the coastal bluff and the coastal terrace. Specifically, this boundary is represented by the landward extent of increased subaerial erosion due to the presence of the coastal bluff. Subaerial erosion, in its broadest sense, encompasses all of the natural geologic processes and human actions that contribute to erosion, excluding marine erosion. A coastal bluff represents the rising ground bordering the sea, which may include a sea cliff, but is characterized by an upper, moderately-sloping section ending at a coastal terrace. A coastal terrace can be defined as that geologic feature that was formed during a higher stillstand of the sea, and represents a wave-cut abrasion surface often characterized by a long, narrow, relatively level surface, bounded along the shoreward edge by the coastal bluff. Higher relic coastal terraces representing earlier stillstands of the sea commonly extend well inland. However, for the purpose of this guideline, the top of the coastal bluff is defined as that boundary of the coastal terrace that was developed during the last stillstand of the sea, which occurred approximately 125,000 years ago.*

Offshore from the sea cliff is an area of indefinite extent called the nearshore zone (see Figure 8). The bedrock surface in the nearshore zone, which extends out to sea from the base of the sea cliff, is the shore platform. Worldwide, the shore platform may vary in inclination from horizontal to a gradient of 3 horizontal to 1 vertical, or $33^{1/3}$ percent (Trenhaile, 1987). Offshore, the gradient of the shore platform is approximated at 1 to 2 percent. The boundary between the sea cliff (the lower near-vertical section of the bluff) and the shore platform is designated as the cliff-platform junction.

Within the nearshore zone is a subdivision designated as the inshore zone, beginning where the waves begin to break (Figure 8). This boundary varies with time because the point at which waves begin to break changes dramatically with changes in wave size and tidal level. During low tides, large waves will begin to break far out to sea. During high tide, waves may not break at all or they may break directly on the lower cliff. The foreshore represents that portion of the shore lying between the upper limit of wave wash at high tide and the ordinary low water mark. It is absent at the site.

3.2 **Coastal Bluff Edge**

The location of the coastal bluff edge was addressed in some detail by GEI and reported in their February 2, 2004, geotechnical report (Appendix A). As indicated in GEI's February 2004 report, a variable thickness veneer of fill mantles the entire westerly surface of the site, obscuring the geologic boundary between the coastal terrace and the landward extent of increased subaerial erosion due to the presence of the coastal bluff. Five hand-dug test excavations were advanced by GEI within the slope separating the lower and upper concrete slabs in December 2003 to locate the edge of the coastal bluff and to facilitate the mapping of the bluff edge in plan view. The plot plan presented in GEI's February 2004 geotechnical report (Appendix A) has been reproduced below as Figure 9, as it shows the location of the coastal bluff edge, determined by the excavation of five test pits specifically excavated to delineate the location of the coastal bluff edge, along with the toe of the coastal bluff as it existed in February 2004, predominantly coincident with the foundation of the existing seawall. Figure 2 also shows the coastal bluff edge as estimated by our review of 1953 aerial photographs. As noted previously, the formational Point Loma shelf rock on which the seawall was constructed almost 70 years ago is continuing to slowly erode, occasionally breaching the foundation of the wall, necessitating the repairs designed by our predecessor firm, Group Delta Consultants-San Diego, in 1991, and by TerraCosta in 2005, 2015, and 2018.

Three representative geologic cross sections are contained in the attached February 2004 geotechnical report prepared by GEI, which in part form the basis for the location of the coastal bluff edge shown on Figure 9. It is our opinion that the GEI test pits most accurately located the top of the coastal bluff that has been reproduced from GEI's report and shown below as Figure 9.

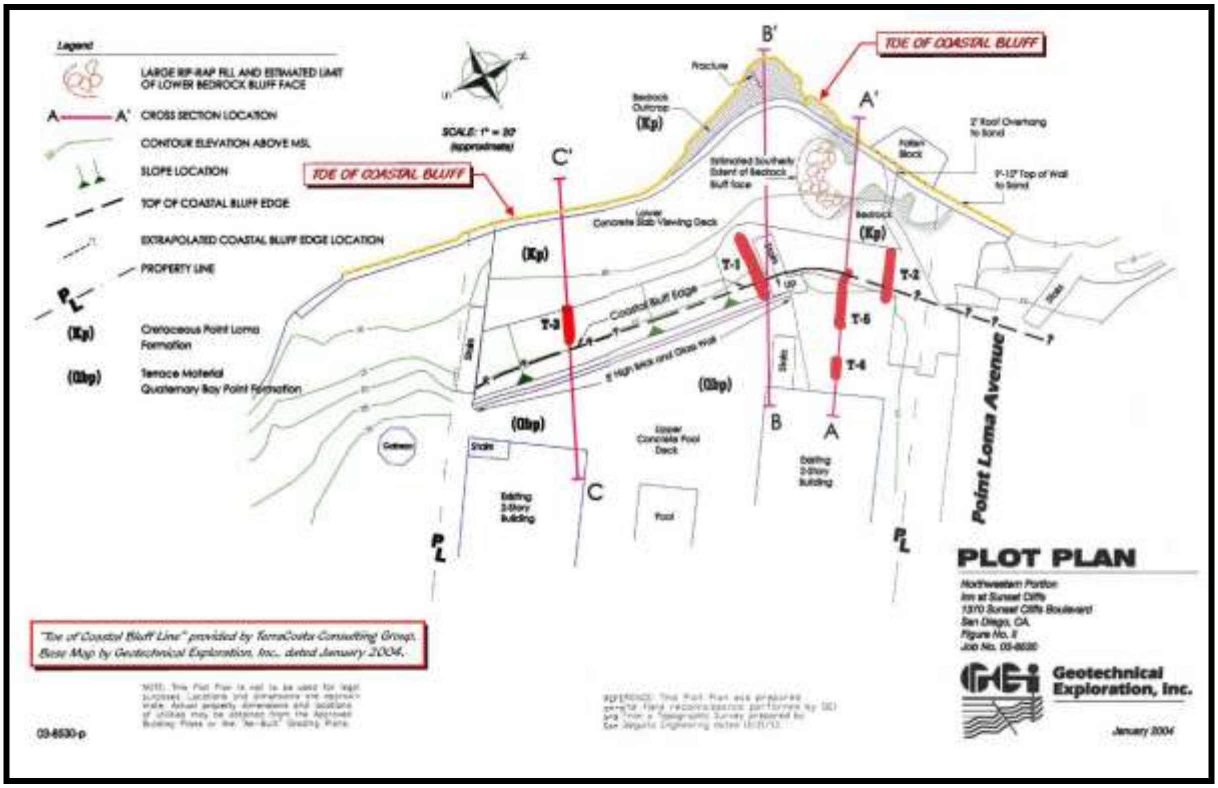


Figure 9. GEI Plot Plan.

GEI also corroborated their bluff edge location by reviewing aerial photographs from 1939, 1950, 1953, 1960, and 1972. While we have also reviewed the available photographs that GEI referenced, we have also independently reviewed additional aerial photographs dated 1953, 1964, 1972, 1980, 1981, 1990, 2003, and 2005 to again corroborate the location of the bluff edge originally determined by GEI.

4 TSUNAMI MAPPING

The University of Southern California Tsunami Research Center, funded through the California Emergency Management Agency, has developed tsunami inundation maps for emergency planning for the entire state of California. The tsunami inundation map for the Point Loma quadrangle is shown on Figure 7A, with an enlargement showing the study area provided on Figure 7B, along with an enlargement of the map text provided on Figure 7C describing the methodology and data sources used in the model. Although the tsunami inundation map provides almost no detailed information on the inundation area along the shoreline, Figure 7B indicates a fairly extensive inundation area throughout the low-lying areas just north of the Ocean Beach Pier. While exact inundation elevations are not available through the University of Southern California Tsunami Research Center, tsunami inundation elevations can be approximated by comparing actual ground surface elevations along the tsunami inundation limits in the vicinity of Ocean Beach, with an estimated inundation elevation, using this admittedly somewhat crude approach, being on the order of 14 feet NGVD 29.

5 WAVE CLIMATE

Waves provide nearly all of the energy input that drives shoreline processes along the California coast. As illustrated in Figure 10, incoming waves along the southern California coast fall into three main categories: Longer period northern and southern hemisphere swell, and locally short-period generated seas. North hemisphere swell from the North Pacific Ocean dominate the winter wave conditions off California, while southern hemisphere swell is more important in the summer. Short-period seas are produced by storms sweeping through the area. The offshore islands, shallow banks, submarine canyons and generally complex bathymetry of southern California greatly complicate the wave climate at the coast (Figure 11).

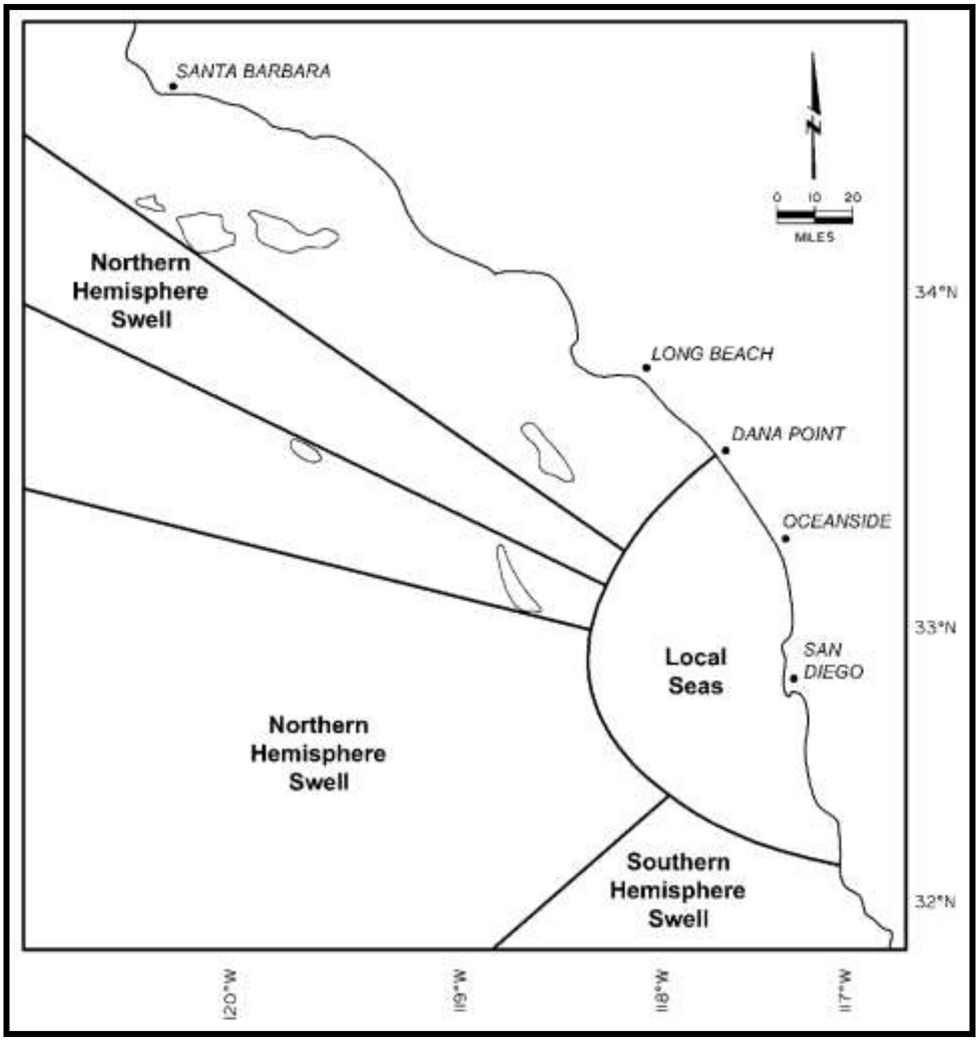


Figure 10. Map Showing Generalized Wave Exposure for Southern California.

Coastal orientation, and the islands and banks greatly influence the swell propagating toward shore by partially sheltering southern California, including Point Loma, especially from directions north of west. Figure 11 shows the approximate directions from which incoming swell is blocked by the islands. The Point Loma coastline faces west and is therefore also relatively exposed to southern hemisphere swell. Because of the complicated effects of bathymetry and island shadowing, the wave height at the shoreline is sensitive to relatively small changes in the incoming direction of the deep ocean waves.

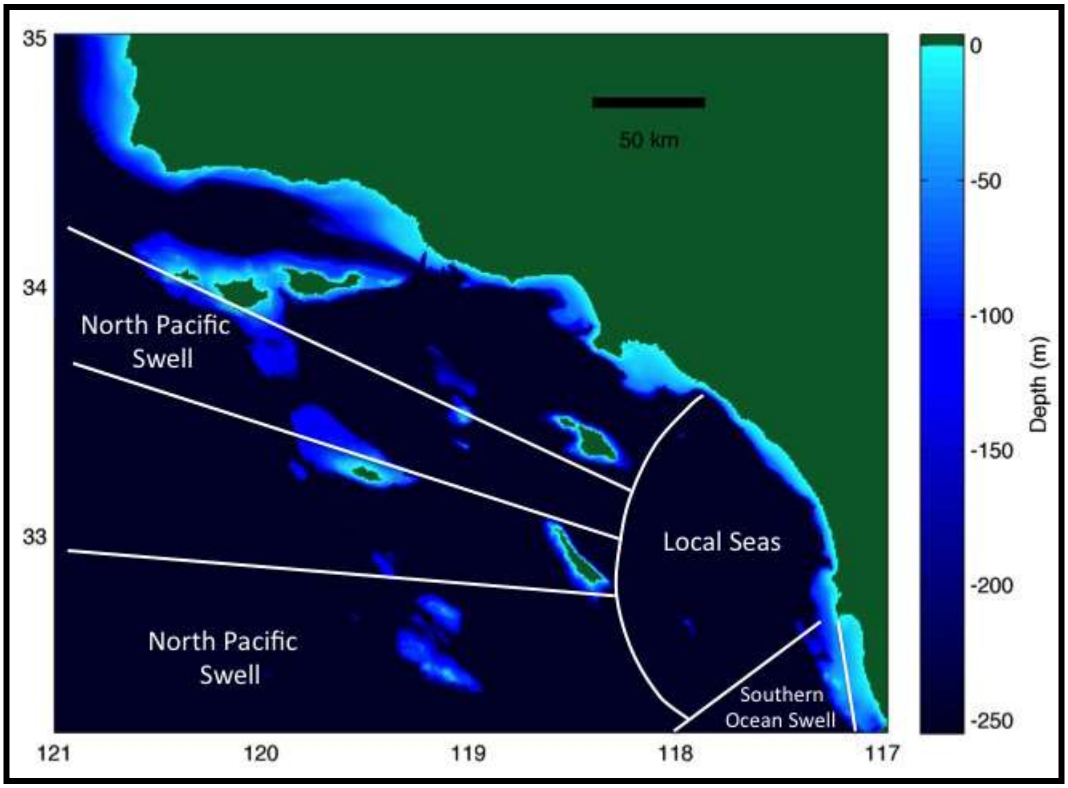


Figure 11. Map Showing Generalized Bathymetry in the Southern California Bight and Wave Exposure Windows at Oceanside.

While waves along the San Diego County shoreline generally range in height from 2 to 5 feet, deep water waves off the coast have been recorded with deep water significant wave heights approaching 10 meters (33 feet).

5.1 Short-Term Sea Level Change

The effect of waves on the coast is highly dependent on the sea level during the wave episode. Large waves at low sea level cause limited erosion, since they break well offshore. When episodes of large waves combine with short-term high sea level from tides and other factors, rapid retreat may occur along vulnerable coastlines.

6 FEMA MAPPING

We conducted a review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map for the study area (Figures 12a and 12b). The Inn falls within a VE Zone (Coastal High Hazard Area), with a base flood elevation (BFE) of 22 feet (NAVD 88) or 19.69 feet (NGVD 29). The VE Zone designation also includes anticipated wave heights at or exceeding 3 feet, however allows construction when the structure is designed to accommodate anticipated wave forces and, notably, when minimum building foundations are 2 feet above the base flood elevation. The proposed top-of-wall elevation of 27.7 feet (NGVD 29) at The Inn is 8 feet higher than the FEMA BFE, with the existing structure foundations near elevation 28 feet (NGVD 29), well above the elevation 22 FEMA BFE + 2 feet.

7 WATER LEVELS

Past water elevations are based on the tide gauge data from La Jolla, which has been collected by NOAA since 1924. These data are applicable to the San Diego County region open-ocean coastline. The tidal and geodetic reference relationships at La Jolla are provided below in Table 1.

Table 1. Tidal Datums (Station 9410230, 1983-2001 Tidal Epoch)

Description	Datum	Elevation (feet, MLLW)
Highest Observed Tide (11/25/2015)	Max Tide	7.81
Highest Astronomical Tide	HAT	7.20
Mean Higher-High Water	MHHW	5.33
Mean High Water	MHW	4.60
Mean Tide Level	MTL	2.75
Mean Sea Level	MSL	2.73
Mean Sea Level	NGVD 29	2.56
Mean Diurnal Tide Level	DTL	2.66
Mean Low Water	MLW	0.91
North American Vertical Datum of 1988	NAVD 88	0.25
Mean Lower-Low Water	MLLW	0.00
Lowest Astronomical Tide	LAT	-1.88
Lowest Observed Tide (12/17/1933)	Min Tide	-2.87
Station Datum	STND	-4.37
Great Diurnal Range	GT	5.33
Mean Range of Tide	MN	3.69

(*Source: NOAA 2018*)

Tide gauges measure total water level outside the breaker zone, which includes contributions from the tide, as well as storm surges and other factors that raise sea level over the short and long term, including the effects of El Niño.

7.1.1 *Tides*

Tides are caused by the gravitational pull of astronomical bodies; primarily the moon, sun, and planets. Tides along the San Diego coast have a semi-diurnal inequality. On an annual average basis, the lowest tide is about 1.7 feet (MLLW datum) and the highest tide is about 7.1 feet, MLLW datum.

Figure 12A. FEMA Map.

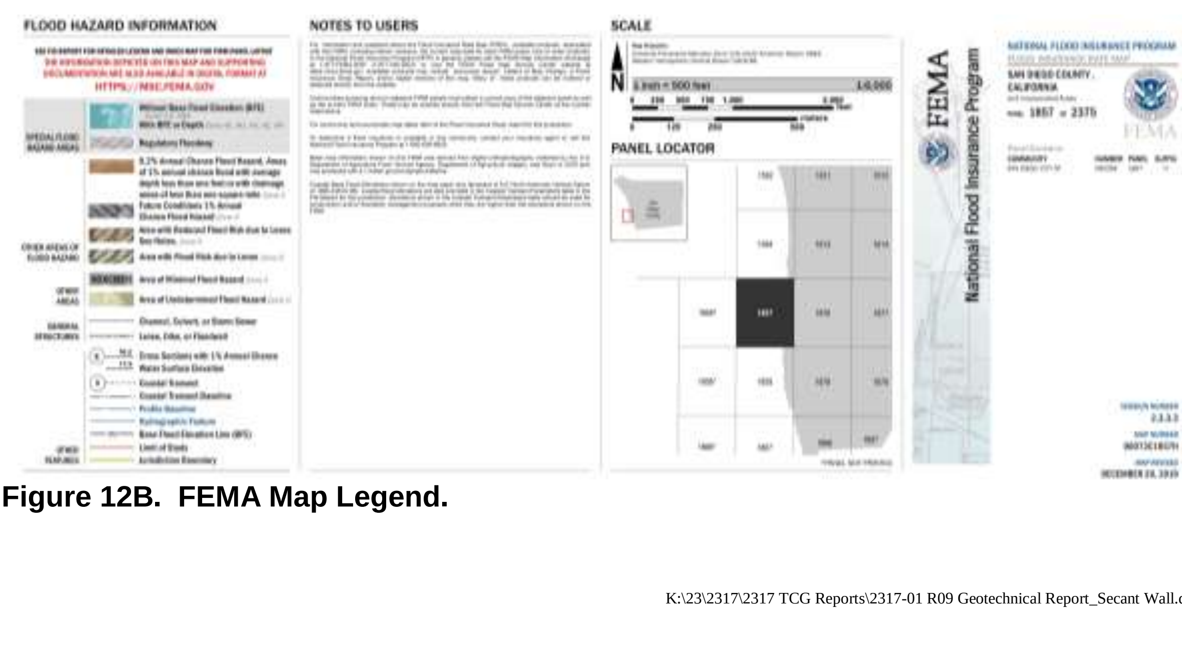


Figure 12B. FEMA Map Legend.

7.1.2 *El Niño*

Large-scale, Pacific Ocean-wide warming periods occur episodically and are related to the El Niño phenomenon. These meteorological anomalies are characterized by low atmospheric pressures and persistent onshore winds. During these events, average sea levels in southern California can rise up to 0.5 foot above normal. Tidal data indicates that seven episodes (1914, 1930 through 1931, 1941, 1957 through 1959, 1982 through 1983, 1997 through 1998, and 2015 through 2016 - mild El Niño-type conditions were also reported in 1988 and 1992) have occurred since 1905. Further analysis suggests that these events have an average return period of 14 years, with 0.2-foot tidal departures lasting for two to three years.

The added probability of experiencing more severe winter storms during El Niño periods increases the likelihood of coincident storm waves and higher storm surge. The record water level of 8.35 feet, MLLW, observed in San Diego Bay in January 1983, includes an estimated 0.8 foot of surge and seasonal level rise (Flick and Cayan, 1984), which set the stage for the wave-induced flooding and erosion that marked that winter season.

7.2 **Sea Level Rise**

Past and possible future changes in mean sea level (MSL) are of interest in design and planning for all coastal cities, as well as for any engineering activities on the coast. Global mean sea level rose at least 300 feet, and perhaps as much as 400 feet, during the past 18,000 years or so (CLIMAP, 1976). Sea level, both globally and along California, rose approximately 0.7 foot over the past century, as shown in Figure 13. Furthermore, evidence suggests that the rate of global mean sea level rise has accelerated since the mid-1800s, or even earlier (Church and White, 2006; Jevrejeva, et al., 2008), and that it has now reached a rate of about 1 foot per century over the past decade or so (Nerem, et al., 2006).

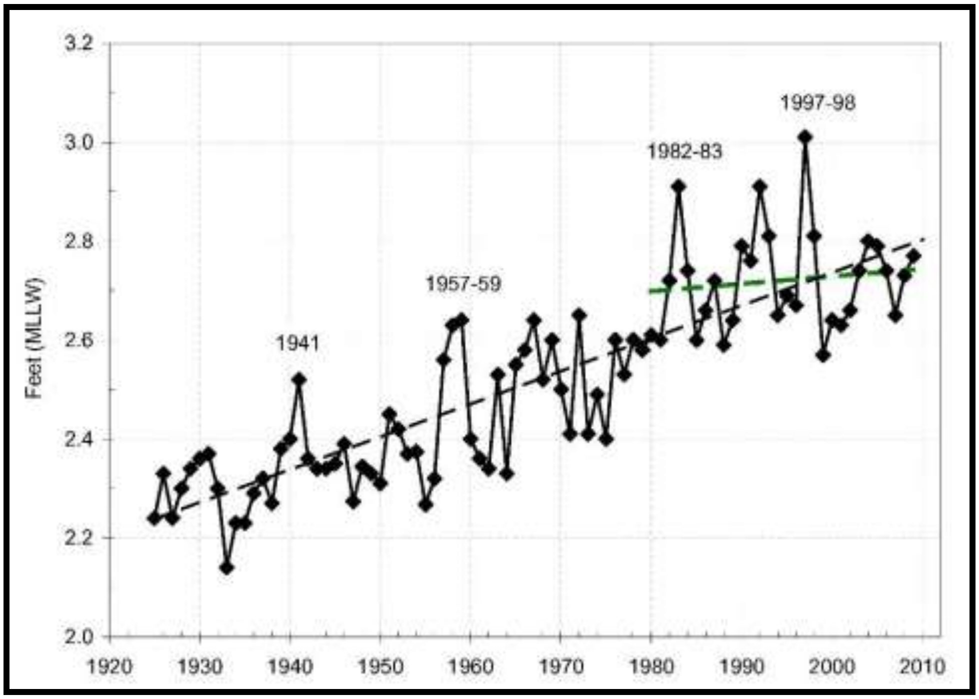


Figure 13. Annual Average Sea Level History at La Jolla, 1925-2007. Broken Line Shows Linear Trend of 0.7 Feet/Century Rise.

Figure 13 is a plot of the annual mean sea levels measured at the La Jolla tide gauge starting in 1925. The linear trend indicates the approximate 0.7 foot per century sea level rise. Also noticeable are the enhanced sea levels during the El Niño episodes of 1941, 1957-59, 1982-83, and 1997-98 (respectively labeled).

A notable feature of the sea level history at La Jolla is the leveling-off of sea level rise since about 1980 (Figure 13). The green broken line shows a much reduced trend of about 0.15 foot per century between 1980 and 2009, or about 4.5 times smaller than the overall trend of 0.67 foot per century. A similar reduction in the rate of sea level rise has been noted at San Francisco, which has a similar overall appearance as the La Jolla record, but is a much longer record extending back to 1856.

Figure 14 shows the global distribution of the rate of sea level change for the period of 1993-2012 (University of Colorado, 2012). Note that warm colors (yellow-orange-red) show areas of sea level rise (positive rates), while cool colors (green- blue) indicate falling sea level (negative rates) over the record. Inspection of the North Pacific reveals that sea levels in the western Pacific, especially in the lower latitudes, have risen at a rate of 3-9 mm/year (equivalent to 30-90 cm per century, or about 1-3 feet per century). Conversely, sea levels in the eastern Pacific, extending from Central America north to Washington State, have fallen at a rate of 0-3 mm per year (0-30 cm per century, or 0-1 foot per century). This may explain the coastal tide gauge observations (La Jolla sea level history; Figure 13) described above.

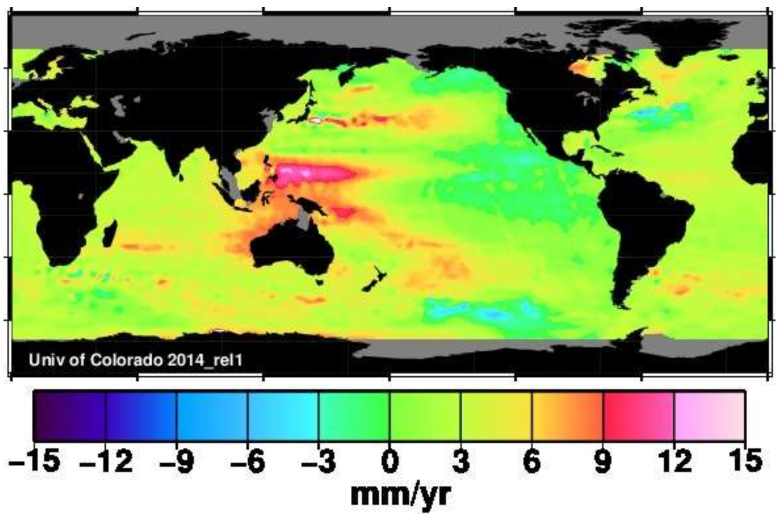


Figure 14. Global Sea Level Change Rates 1993-2012 as derived from satellite altimetry measurements, following University of Colorado (2012).

While the cause of these regional differences undoubtedly lies in the large-scale circulation of the Pacific Ocean and the overlying atmosphere, no detailed explanation is known. However, these observations could be a cause for some concern. If the conditions driving sea level up in the western Pacific and down in the eastern Pacific were to relax or even reverse, sea level along the coast of California could begin to increase at a much higher rate than what has been observed over the past several decades. Future global sea level rise scenarios could further increase the rate of sea level rise.

When considering the effects of future sea level rise, the National Academy of Sciences (NAS, 2012) presents a possible global, west-coast, and state-wide future Mean Sea Level Rise (MSLR) for California, Oregon, and Washington (Figure 15, dots) and its range (Figure 15, bars). These are based on the IPCC (2007) mid-range Green House Gas emissions scenarios for the ocean steric (warming) expansion component added to the results of new research projecting the likely contributions of future ice-melt. The resulting projected *global* MSLR relative to 2000 ranged from 0.08-0.23 m (0.26-0.75 ft) by 2030; 0.18-0.48 m (0.59-1.6 ft) by 2050; and 0.50-1.4 m (1.6-4.6 ft) by 2100 (Figure 15, red bars). The global estimates were adjusted for vertical crustal movement (uplift north of Cape Mendocino and down-drop in the south) resulting in the orange bars, also shown in Figure 15. The State of California (2013) used these results of NAS (2012) shown as the updated MSLR guidance in Table 2.

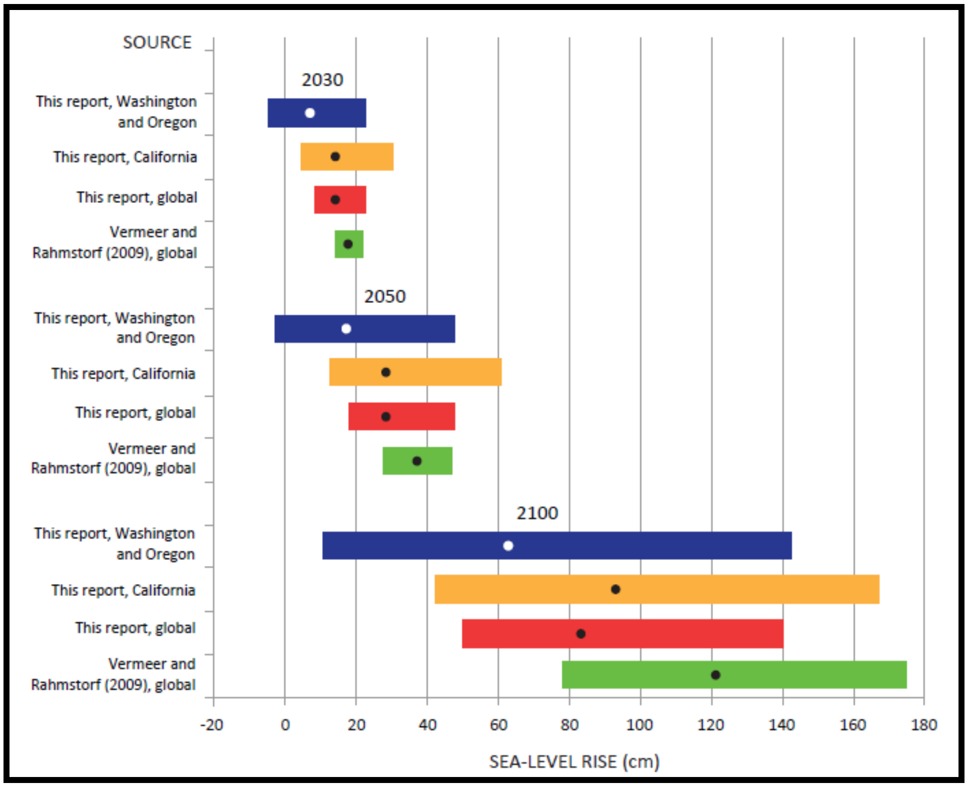


Figure 15. NAS (2012) summary of global, Washington, Oregon, and California (south of Cape Mendocino) MSLR projections for 2030, 2050, and 2100 relative to 2000.

Table 2. Updated MSLR Guidance from State of California (2013)

Time Period	North of Cape Mendocino[3]	South of Cape Mendocino
2000 - 2030	-4 to 23 cm (-0.13 to 0.75 ft)	4 to 30 cm (0.13 to 0.98 ft)
2000 – 2050	-3 to 48 cm (-0.1 to 1.57 ft)	12 to 61 cm (0.39 to 2.0 ft)
2000 – 2100	10 to 143 cm (0.3 to 4.69 ft)	42 to 167 cm (1.38 to 5.48 ft)

While many sea-level rise scenarios have been published, the California Coastal Commission, on August 12, 2015, adopted their Sea Level Rise Policy Guidance document, which provides sea level rise projections from the Third National Climate Assessment (NCA; Melillo, et al.), released in 2014, providing a set of four global sea level rise scenarios ranging from 8 inches to 7 feet by the year 2100, reflecting different amounts of future greenhouse gas emissions, ocean warming, and ice sheet loss.

The OPC (2018) update offered a new strategy by presenting MSLR trajectories as functions of emission scenarios as well as probability of occurrence. An extreme trajectory with unknown probability was also added. For example, the low-emissions 2100 endpoint value of 1.7 feet of MSLR has a 50 percent chance of being exceeded, while the corresponding high-emissions 2100 endpoint is 2.6 feet. In another way to look at it, the low-emissions 2100 MSLR value has a 66 percent chance of lying between 1.1 and 2.5 feet, while the high-emissions range is 1.8 to 3.6 feet. There is a 5 percent chance that MSLR will be 3.3 feet (low) or 4.5 feet (high) by 2100, and a 0.5 percent chance of 5.8 feet (low) or 7.0 feet (high). Finally, the extreme scenario postulates 10.2 feet MSLR by 2100 in case of rapid Antarctic ice loss.

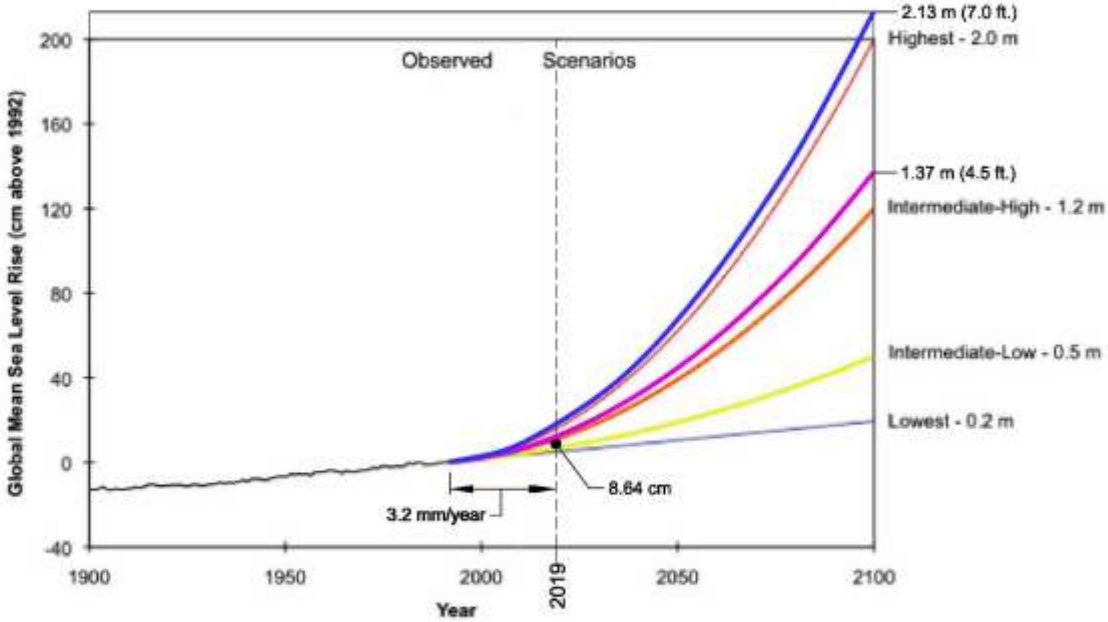
OPC (2018) contains a description of the best available science to support planning; MSLR projections; guidance on how to select projections; and recommendations for planning and adaptation. Projections for two greenhouse gas emissions scenarios are provided for 12 locations with long-term tide-gauge data in California, from Crescent City south to San Diego. OPC (2018) employs the highest and lowest of the four emissions scenarios used by the Intergovernmental Panel on Climate Change's Fifth Assessment Report: RCP 8.5 and RCP 2.6, respectively.

Each RCP (representative concentration pathway) denotes a family of possible underlying socioeconomic conditions, policy options, and technological considerations that span from the low-end RCP 2.6, which requires significant emissions reductions, to the high-end, "business-as usual" fossil-fuel-intensive evolution, RCP 8.5. For further details, see IPCC (2014). These two high and low-end pathways were chosen by OPC (2018) to bracket the current best-estimate of the range of possible futures. However, even the "high" probabilistic projections may underestimate the chances of extreme MSLR, resulting, for example, from loss of the West Antarctic ice sheet. Therefore, OPC (2018) includes an extreme scenario designated "H++." The probability of this scenario is currently unknown, but presumably very small.

OPC (2018) presents results for each location in a series of tables that specify several time sequences of MSL from 2030-2150, where each series has a specified probability or range of probabilities of occurrence associated with it. The MSLR projections assume 2000 as the base year and project MSLR in specified future years relative to MSL in 2000. There is a table for each scenario, low and high. The OPC (2018) MSL elevation projections for San Diego from 2000-2150 are reproduced in Table 3.

7.3 **Design SLR Scenario**

We have reproduced as Figure 16 the Coastal Commission's four suggested sea level rise scenarios through the year 2100 from the Coastal Commission's 2015 Sea Level Rise Policy Guidance Document, ranging from the Lowest at 0.2 meter, to the Highest at 2.0 meters, measured from the 1992 baseline. Based on recent discussions with Dr. Reinhard Flick, the State Oceanographer with the California Department of Boating and Waterways and a Research Scientist at Scripps Institution of Oceanography, global sea level has risen from 1992 through 2019 at a relatively uniform rate of 32 centimeters per century, or at the same trajectory as previously reported by Nerem (2005) and illustrated below in Figure 17. While Nerem's data extended from 1993 to 2005, the more recent recorded global sea level elevation change from 1993 to 2019 provides essentially the same data. This information is also shown on Figure 16, which from 1992 through 2019 has resulted in 8.64 centimeters of relatively uniform sea level rise in the past 27 years. If this uniform rate of sea level rise (consistent with that shown on Figure 16) were to extend out to the year 2100, this would be equivalent to a future mean sea level of 0.35 meter (1.13 feet) above the 1992 datum, and slightly above the Coastal Commission's 2015 suggested Lowest SLR scenario. Notably, and while we appreciate the Coastal Commission's 2018 Sea Level Rise Policy Guidance Update requiring adaptive strategies to accommodate worst-case SLR scenarios, the OPC's 2018 Sea-Level Rise Guidance also acknowledges that there is a 17 percent probability of sea level rise by the year 2100 being lower than 1.8 feet, as shown on Table 3.

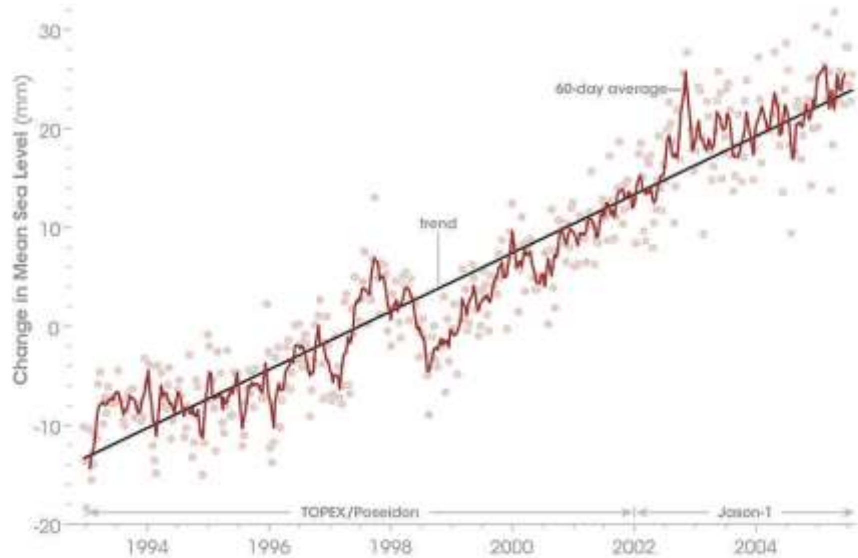


Modified from Figure 5 of the California Coastal Commission Sea Level Rise Policy Guidance document adopted August 12, 2015.

FIGURE 16

As indicated in the previous section, the real significance of the various SLR scenarios is the volume of overtopping and the amount beyond which overtopping becomes objectionable. Regardless of the assumed SLR scenario, future overtopping rates can be reduced by simply increasing the height of the seawall, with several feet of future increased wall height relatively easy to accommodate. More importantly, the wall as currently designed can safely accommodate even the highest suggested SLR scenario.

Global mean sea level rise rate over the past two decades has increased over the rate observed for the past century, and has reached about 1 foot per century (32 cm per century). This is indicated from satellite data reporting and trend analysis shown in Figure 17 (Nerem, 2005).



Satellite-derived global sea level elevation change from 1993-2005 after Nerem (2005). Slope of the trend line is about 1 foot per century (32 cm per century).

FIGURE 17

8 WAVE DESIGN CRITERIA

8.1 Design Stillwater

The maximum design storm still-water level (SWL) is critical to any wave analyses, as it determines the wave energy that can be propagated into the shoreline, eventually impacting and overtopping structures. It is the deep-water wave height superimposed upon the extreme SWL that defines the joint probability of the design storm condition, creating the largest wave forces on structures, along with a maximum runup and overtopping volume. In addition to tidal fluctuation, water levels at the shoreline are influenced by storm surge, wave setup, and surf beat. These influences, combined with the astronomical high tide, allow offshore storm waves to impact coastal structures. In 1953, in the Point Loma area, a reasonable design stillwater level might have been on the order of 5 to 6 feet MSL. Fifty years later, in the early 2000s, when including storm surge, wave setup, and possibly a foot of long-term sea level rise for a 50- to 75-year design life of a structure, the likely maximum design stillwater level would be 7.5 feet MSL. In 2020, and given the possibility of even more extreme sea level rise scenarios (see OPC 2018), one might look to a maximum design stillwater level on the order of 10 to 12 feet MSL, and possibly more depending upon the criticality of the structure.

8.2 Design Wave Height

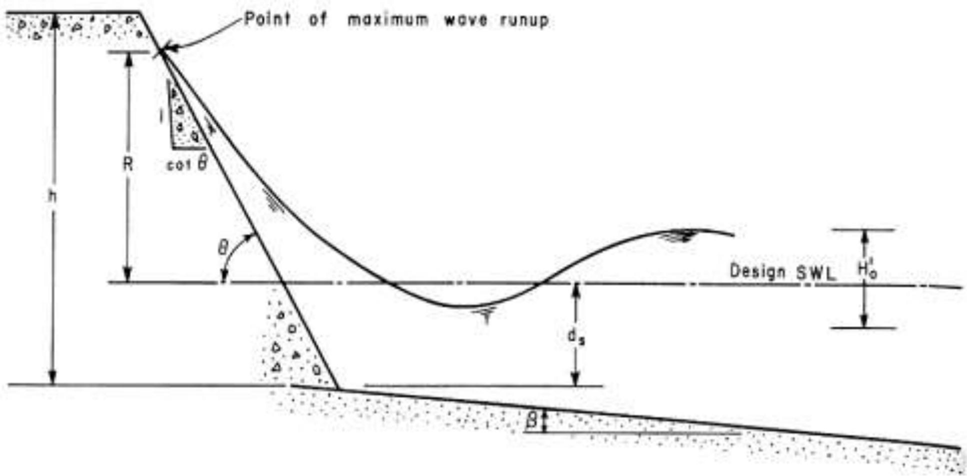
The maximum wave height that can reach the structure occurs during the period when the maximum depth of standing water exists in front of the structure, which includes both the maximum SWL, combined with the maximum scour at the base of the structure. The maximum water depth at the base of the structure, d_s, would then be the maximum historic stillwater level of 5.25 feet (NGVD 29) (Table 1), plus the design scour depth of -5 feet (NGVD 29) plus the design sea level rise value obtained from Table 3 (OPC, 2018). The resultant maximum breaking wave height occurs when a specific deep-water wave is allowed to shoal and break directly upon the structure.

Our evaluation of the maximum design wave for the subject structure is based on criteria set forth in the U.S. Army Corps of Engineers Shore Protection Manual (1984 Edition). For purposes of computing the maximum wave height, we have assumed a design scour elevation in front of the structure of -5 feet (NGVD 29), and a foreshore slope of 1 to 60. Three design still water levels were selected.

8.3 Wave Runup and Overtopping Analysis

Wave runup is defined as the rush of water up a beach or coastal structure that is caused by, or associated with, breaking waves. The maximum runup is the highest vertical elevation that the runup will reach above the still water level. If the maximum runup is higher than the top of a coastal structure, the excess represents overtopping. Runup elevation depends on the incident wave characteristics, the beach profile including profile elevation, and other factors. Most wave runup and overtopping analyses are based upon equations and nomographs provided in the U.S. Army Corps of Engineers Shore Protection Manual (SPM, USACE, 1984), and the more recent Internet-based Coastal Engineering Manual (Part VI-Chapter 5, 2006).

The following definition sketch for both wave runup and overtopping, reproduced from the 1984 SPM, graphically illustrates the point of maximum wave runup for a particular design condition.



Definition sketch: wave runup and overtopping

It should also be clear from the sketch that any wave runup exceeding the height of the structure then represents overtopping.

We evaluated both the maximum height of runup and volume of overtopping based on the U.S. Army Corps of Engineers 2006 Coastal Engineering Manual (CEM) for a total of three design conditions. We assumed a design scour elevation of -5 feet, a foreshore slope of 1 on 60, and wave periods ranging from 6 to 18 seconds.

The three design conditions are described below:

Case 1 represents the 1982-83 storms, with an estimated design still water level of 5.25 feet (NGVD 29).

Case 2 assumes 3.5 feet of sea level rise by the year 2100, with a 17 percent probability of sea level rise exceeding this height by the year 2100.

Case 3 assumes 7 feet of sea level rise by the year 2100, with a 1/2 percent probability of sea level rise exceeding this value by the year 2100.

The following table lists the calculated design wave runup elevation for the three design conditions, along with the calculated volume of overtopping, the latter measured in both liters per second per meter (l/s/m) and cubic feet per second per foot ($ft^3/s/f$). Calculations are attached to this report.

Design Condition	Maximum Design Wave Runup Elevation (feet)	Overtopping Volume (l/s/m)	Overtopping Volume ($ft^3/s/f$)
Case 1	24	0	0
Case 2	28	13	0.14
Case 3	36	50	0.54

The seawall has a design top-of-wall elevation of 27.7 feet (NGVD 29), with Case 1 not overtopping the wall; Case 2 resulting in only minor overtopping; and, assuming 7 feet of sea level rise (Case 3), the maximum design wave runup elevation is over 8 feet above the top-of-wall elevation. Notably, with a top-of-wall elevation of 27.7 feet (NGVD 29), this height is above the elevation of most seawalls in San Diego County, and although the Case 2 and Case 3 design wave conditions will result in overtopping, it is important to recognize that wave overtopping currently occurs on an almost monthly basis, often with rather spectacular views, and the property owners understandably keep both their guests and the public away during these storm events.

9 DESIGN CONSIDERATIONS

The proposed shoreline stabilization project shown on Figure 18 is necessary to prevent the continued erosion of the lower bluff threatening the bluff-top structures and to prevent flanking of the adjacent walls to the north and south. Cross Sections A, B, and C (Figures 3, 4, and 5) show the existing and proposed improvements. Absent the proposed improvements, both the bluff-top structures and the adjacent improvements would be compromised.

The design of the proposed tied-back secant pile wall incorporates 30-inch-diameter cast-in-drilled-hole (CIDH) reinforced concrete piers on 22-inch centers, resulting in 8-inch overlaps between each sequential concrete shaft to create a continuous structural concrete wall. Alternating drilled piers would be poured with a slightly weaker concrete, with the sequential alternating drilled piers then reinforced with high strength concrete to provide the structural capacity for the wall derived from a series of reinforced CIDH concrete piers on 44-inch spacing with tiebacks centered in every third non-reinforced drilled pier at 11 feet on center. The overlapping concrete piers create a structural wall, with the exposed surface then architecturally carved and colored to blend in with the adjacent natural geologic exposures and adjacent walls to the north.

9.1 **Wall Loading Conditions**

The secant pile wall is designed as a tied-back wall drilled into the bedrock formational shelf rock, the elevation of which is estimated to range from 5 to 12 feet, with the design scoured shore face seaward of the wall at around elevation -5 feet NGVD 29. The wall should be designed to resist an equivalent fluid pressure of 70 pounds per cubic foot (pcf) from -5 feet up to +12 feet, or 2 feet above the geologic contact, whichever is higher. Above the saturated zone, the wall should be designed to resist an equivalent fluid pressure of 45 pcf up to the top of the wall. Hydroaugers are recommended at +10 feet, or at the elevation of the geologic contact, whichever is higher, spaced at 88 inches to prevent continuous saturation of the wall backfill. The lower portion of the secant pile wall below elevation -5 feet is embedded in very dense, massive, relatively impermeable sandstone, cemented enough to not impose any lateral loads. Based on our calculations of passive resistance for the secant pile wall embedded in the bedrock formational shelf rock, we recommend a passive equivalent fluid pressure of 600 pcf for design.

9.2 **Wall Drainage**

Provisions for wall drainage above elevation +10 feet or at the geologic contact, whichever is higher, will consist of 10-foot-long hydroaugers spaced at 88 inches on center.

9.3 **Site Access**

Site access will be limited for the proposed shoreline stabilization. Equipment necessary to drill holes for construction and to break up and move debris may require lifting over the existing buildings via crane from the Point Loma Avenue street-end.

10 LIMITATIONS

Coastal engineering and the earth sciences are characterized by uncertainty. Professional judgments represented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet the current professional standards; we do not guarantee the performance of the project in any respect. This warranty is in lieu of all other warranties, expressed or implied.

We have observed only a small portion of the pertinent soil and groundwater conditions at the proposed project site. The recommendations made herein are based on the assumption that soil conditions do not deviate appreciably from those found during our field investigation. If the plans for site development are changed, or if variations or undesirable geotechnical conditions are encountered during construction, TerraCosta Consulting Group, Inc. should be consulted for further recommendations.

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Table 3. Projected Sea-Level Rise (in feet) for San Diego

Probabilistic projections for the height of sea-level rise shown below, along with the H++ scenario (depicted in blue in the far right column), as seen in the Rising Seas Report. The H++ projection is a single scenario and does not have an associated likelihood of occurrence as do the probabilistic projections. Probabilistic projections are with respect to a baseline of the year 2000, or more specifically the average relative sea level over 1991 - 2009. High emissions represents RCP 8.5; low emissions represents RCP 2.6. ***Recommended projections for use in low, medium-high and extreme risk aversion decisions are outlined in blue boxes below.***

		Probabilistic Projections (in feet) (based on Kopp et al. 2014)						H++ scenario (Sweet et al. 2017) *Single scenario
		MEDIAN	LIKELY RANGE			1-IN-20 CHANCE	1-IN-200 CHANCE	
		50% probability sea-level rise meets or exceeds...	66% probability sea-level rise is between...			5% probability sea-level rise meets or exceeds...	0.5% probability sea-level rise meets or exceeds...	
					Low Risk Aversion		Medium - High Risk Aversion	Extreme Risk Aversion
High emissions	2030		0.4	-	0.6	0.7	0.9	1.1
	2040	0.7	0.5	-	0.9	1.0	1.3	1.8
	2050	0.9	0.7	-	1.2	1.4	2.0	2.8
Low emissions	2060	1.0	0.7	-	1.3	1.7	2.5	
High emissions	2060	1.2	0.9	-	1.6	1.9	2.7	3.9
Low emissions	2070	1.2	0.9	-	1.6	2.0	3.1	
High emissions	2070	1.5	1.1	-	2.0	2.5	3.6	5.2
Low emissions	2080	1.4	1.0	-	1.9	2.4	3.9	
High emissions	2080	1.9	1.3	-	2.5	3.3	4.6	6.7
Low emissions	2090	1.6	1.0	-	2.2	2.9	4.8	
High emissions	2090	2.2	1.6	-	3.0	3.7	5.7	8.3
Low emissions	2100	1.7	1.1	-	2.5	3.3	5.8	
High emissions	2100	2.6	1.8	-	3.6	4.5	7.0	10.2
Low emissions	2110*	1.9	1.3	-	2.7	3.5	6.4	
High emissions	2110*	2.8	2.0	-	3.7	4.7	7.5	12.0
Low emissions	2120	2.0	1.3	-	3.0	4.1	7.6	
High emissions	2120	3.1	2.3	-	4.3	5.5	8.8	14.3
Low emissions	2130	2.2	1.4	-	3.3	4.6	8.6	
High emissions	2130	3.5	2.6	-	4.9	6.3	10.2	16.6
Low emissions	2140	2.4	1.5	-	3.6	5.1	9.8	
High emissions	2140	3.9	2.8	-	5.4	7.1	11.7	19.2
Low emissions	2150	2.5	1.5	-	3.9	5.7	11.1	
High emissions	2150	4.3	3.0	-	6.1	7.9	13.5	22.0

**Most of the available climate model experiments do not extend beyond 2100. The resulting reduction in model availability causes a small dip in projections between 2100 and 2110, as well as a shift in uncertainty estimates (see Kopp et al. 2014). Use of 2110 projections should be done with caution and with acknowledgement of increased uncertainty around these projections.*

Source: Ocean Protection Council, 2018

Project Name: Inn at Sunset Cliffs - Project No.: 2317-01 - Figure 1

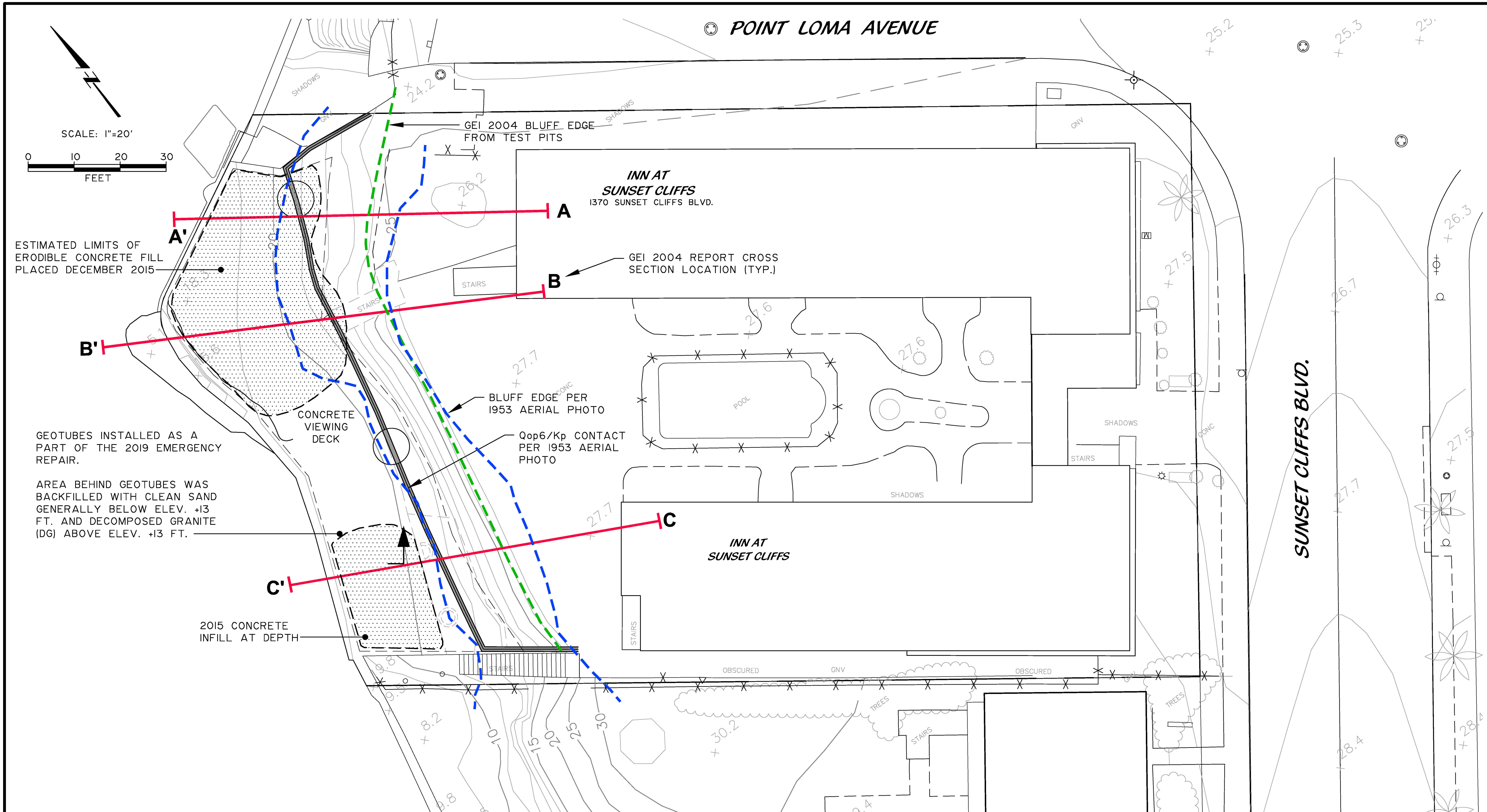

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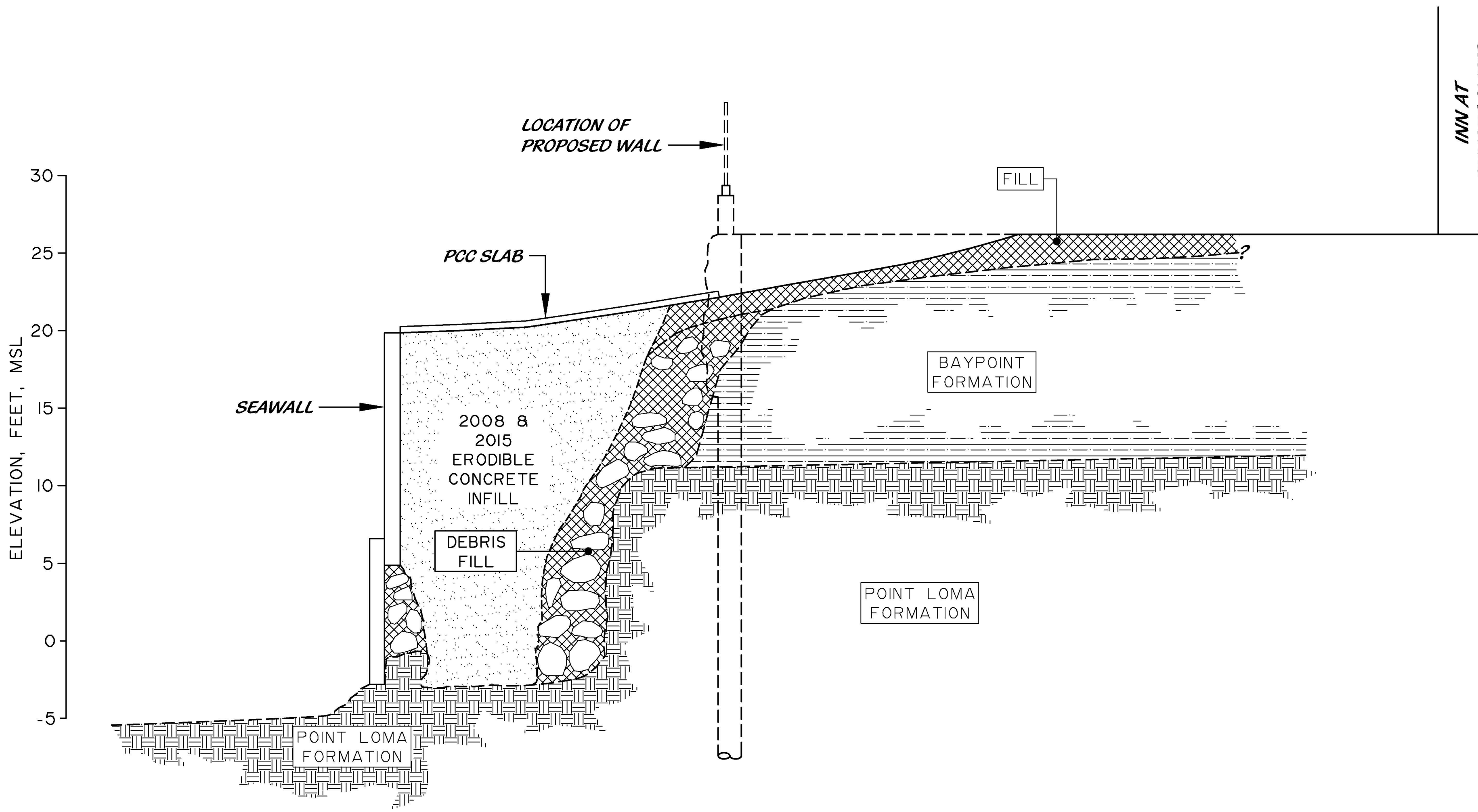
FIGURE NUMBER	PROJECT NAME	PROJECT NUMBER
2	INN AT SUNSET CLIFFS	2317-01

SITE MAP



EXISTING CROSS SECTION

SCALE: 1"=10'

A-1
-

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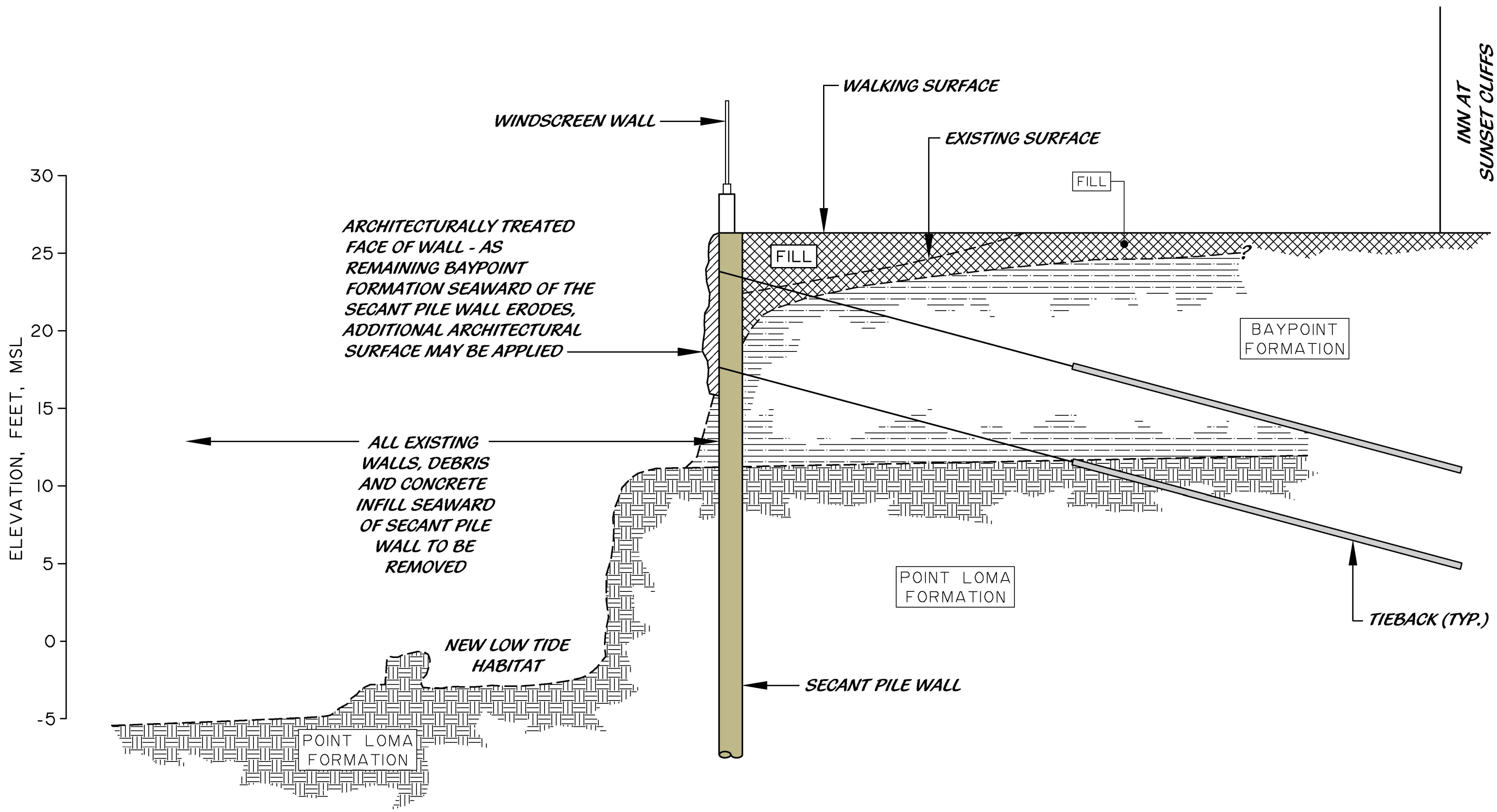
FIGURE NUMBER
3A

PROJECT NAME
INN AT SUNSET CLIFFS

PROJECT NUMBER
2317-01

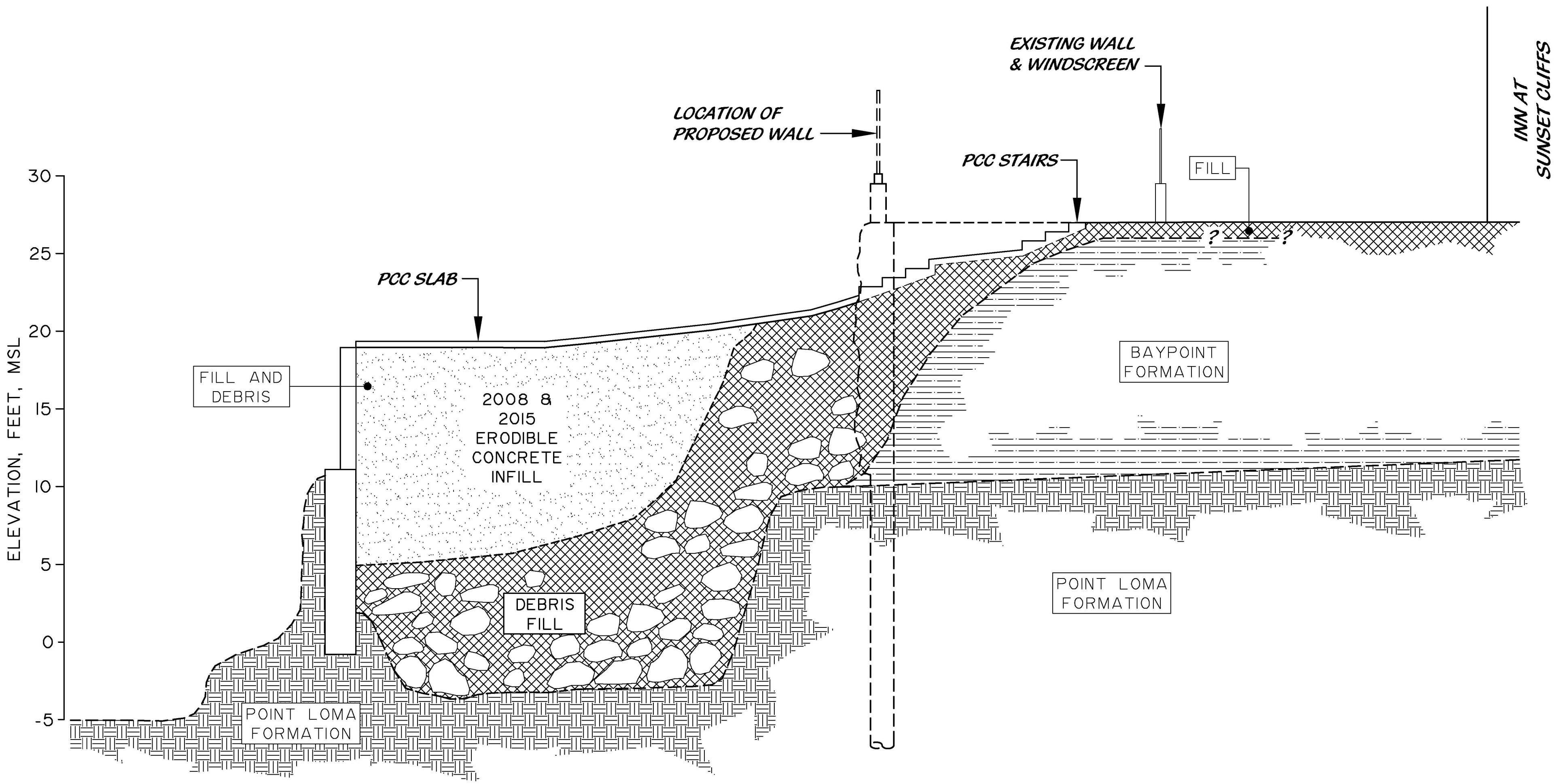
EXISTING CROSS SECTION



PROPOSED CROSS SECTION A-2

SCALE: 1"=10'

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 3B
	PROJECT NAME INN AT SUNSET CLIFFS	PROJECT NUMBER 2317-01
	PROPOSED CROSS SECTION	



EXISTING CROSS SECTION

SCALE: 1"=10'

B-1

-

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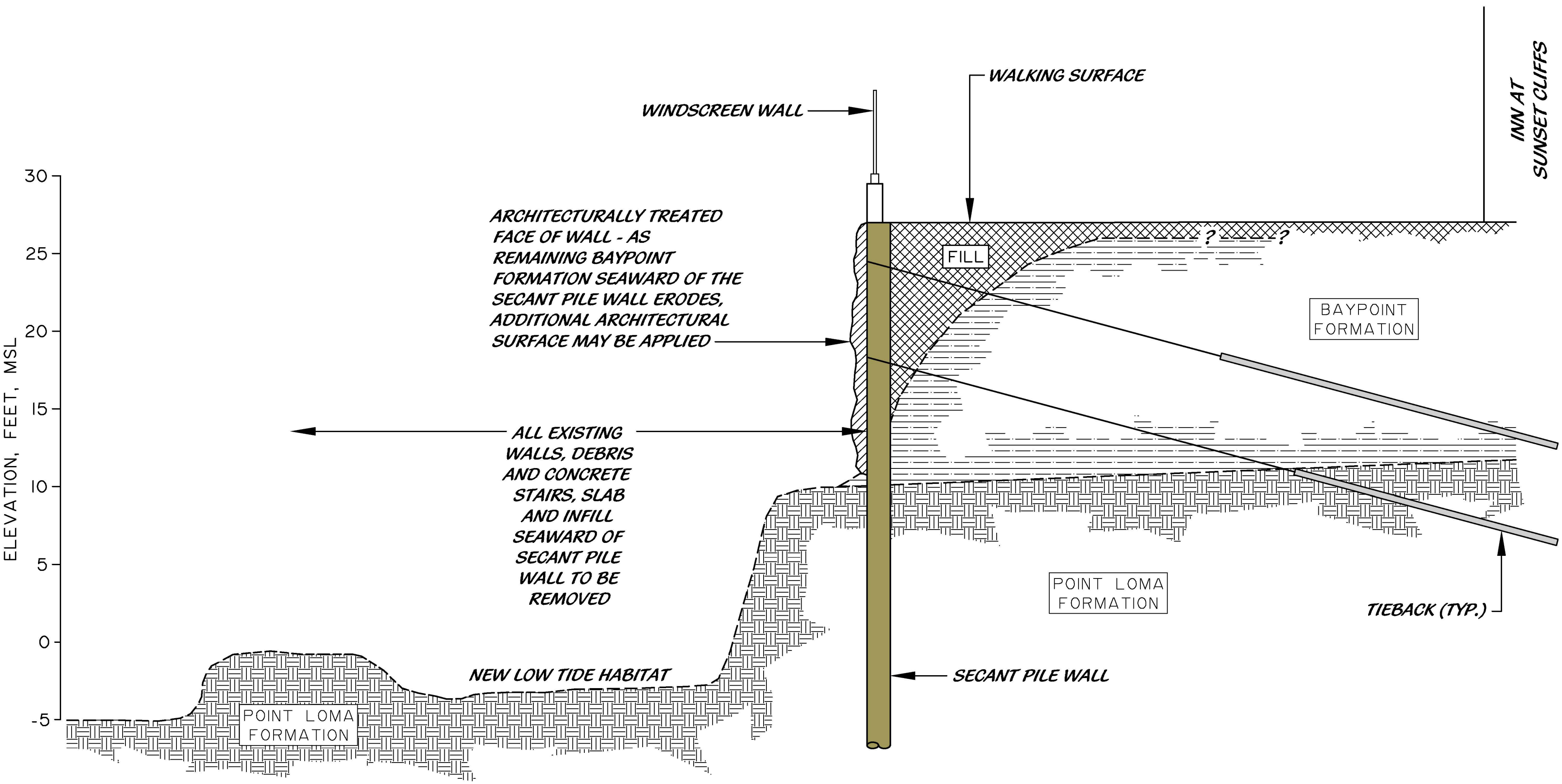
TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 4A
PROJECT NAME INN AT SUNSET CLIFFS	PROJECT NUMBER 2317-01
EXISTING CROSS SECTION	

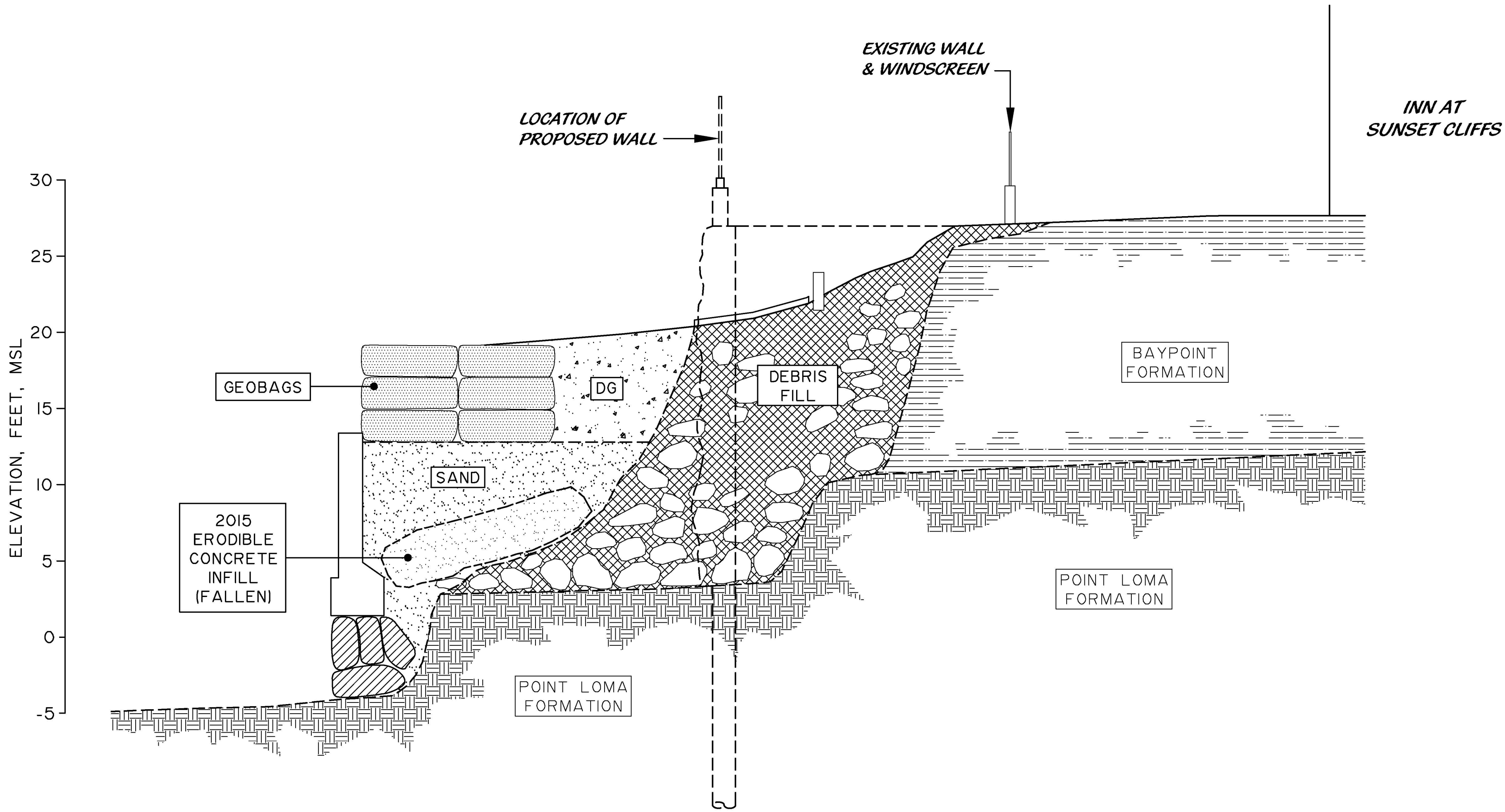


PROPOSED CROSS SECTION B-2

SCALE: 1"=10'

-

TerraCosta Consulting Group	**TERRACOSTA CONSULTING GROUP** ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER **4B**
	PROJECT NAME **INN AT SUNSET CLIFFS**	PROJECT NUMBER **2317-01**
	PROPOSED CROSS SECTION	



EXISTING CROSS SECTION

SCALE: 1"=10'

C-1

-

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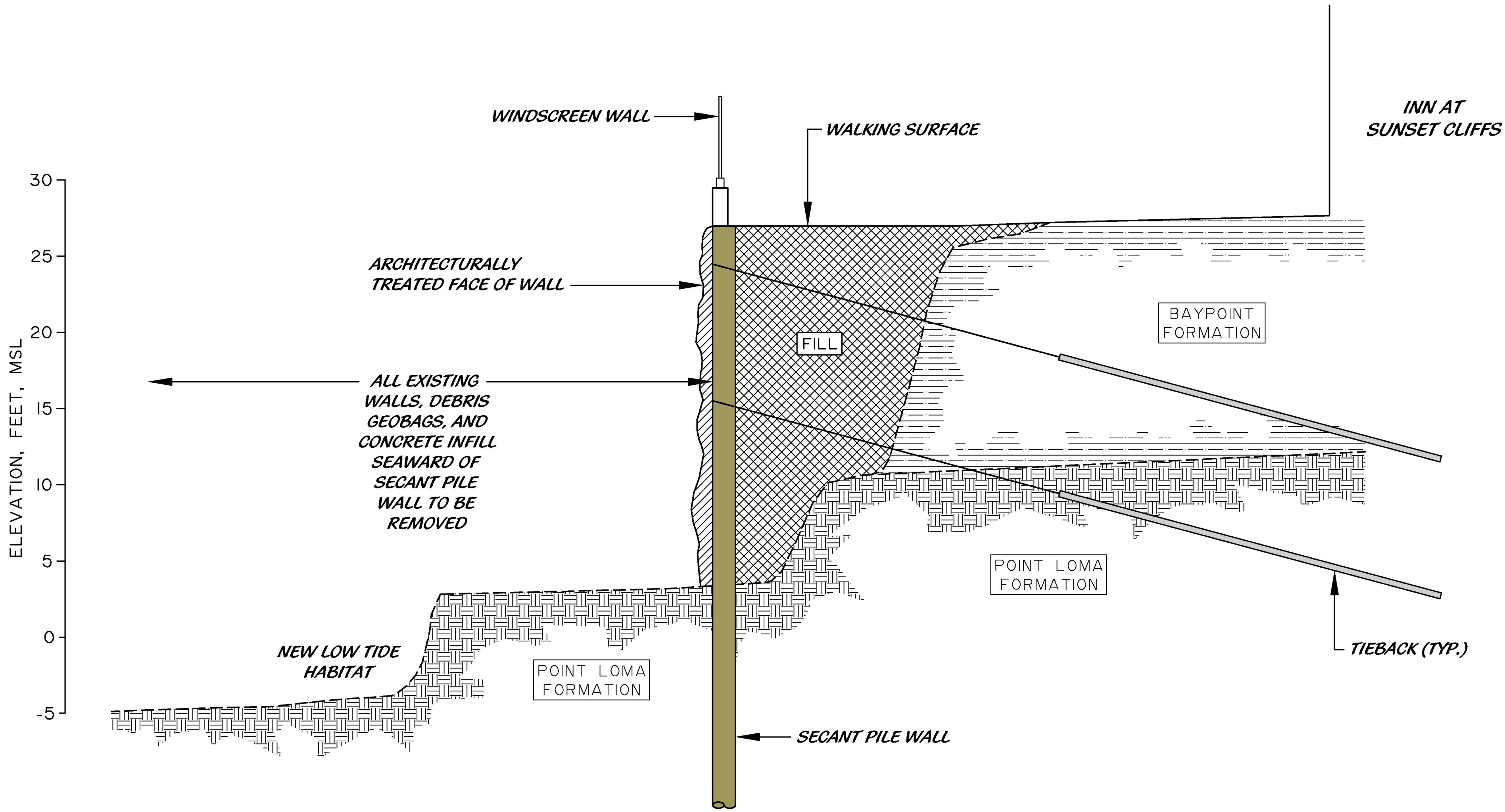
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER	
5A	
PROJECT NAME	PROJECT NUMBER
INN AT SUNSET CLIFFS	2317-01

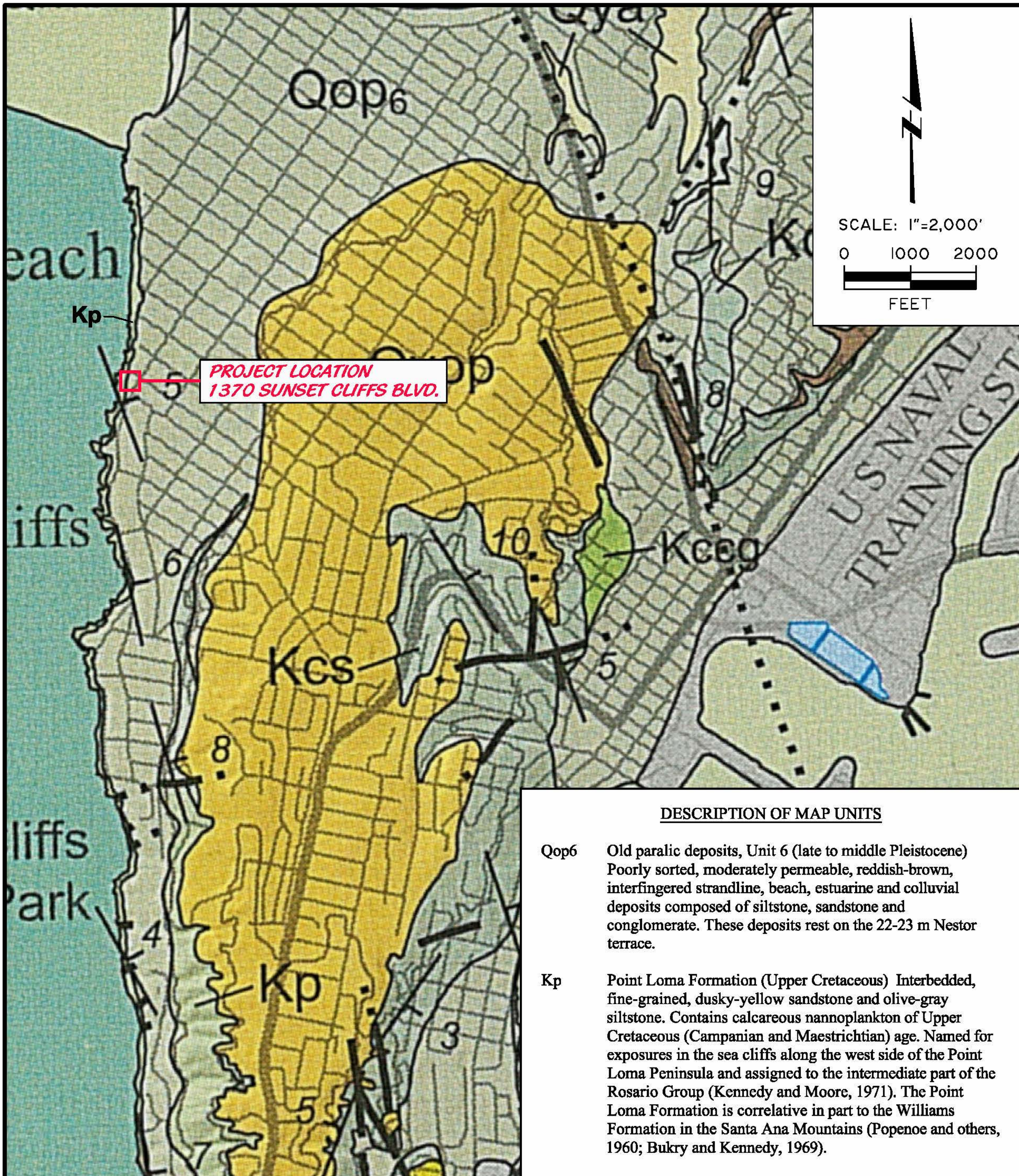
EXISTING CROSS SECTION



PROPOSED CROSS SECTION C-2

SCALE: 1"=10'

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 5B
	PROJECT NAME INN AT SUNSET CLIFFS	PROJECT NUMBER 2317-01
	PROPOSED CROSS SECTION	



DESCRIPTION OF MAP UNITS

Qop6	Old paralic deposits, Unit 6 (late to middle Pleistocene) Poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate. These deposits rest on the 22-23 m Nestor terrace.
Kp	Point Loma Formation (Upper Cretaceous) Interbedded, fine-grained, dusky-yellow sandstone and olive-gray siltstone. Contains calcareous nannoplankton of Upper Cretaceous (Campanian and Maestrichtian) age. Named for exposures in the sea cliffs along the west side of the Point Loma Peninsula and assigned to the intermediate part of the Rosario Group (Kennedy and Moore, 1971). The Point Loma Formation is correlative in part to the Williams Formation in the Santa Ana Mountains (Popenoe and others, 1960; Bukry and Kennedy, 1969).

BASE MAP SOURCE:

Adapted from a portion of "Geologic Map of the San Diego 30'x60' Quadrangle, California," compiled by Michael P. Kennedy and Siang S. Tan, 2008. California Geological Survey, Regional Geologic Map No. 3.

TerraCosta Consulting Group

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SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER **6**

INN AT SUNSET CLIFFS

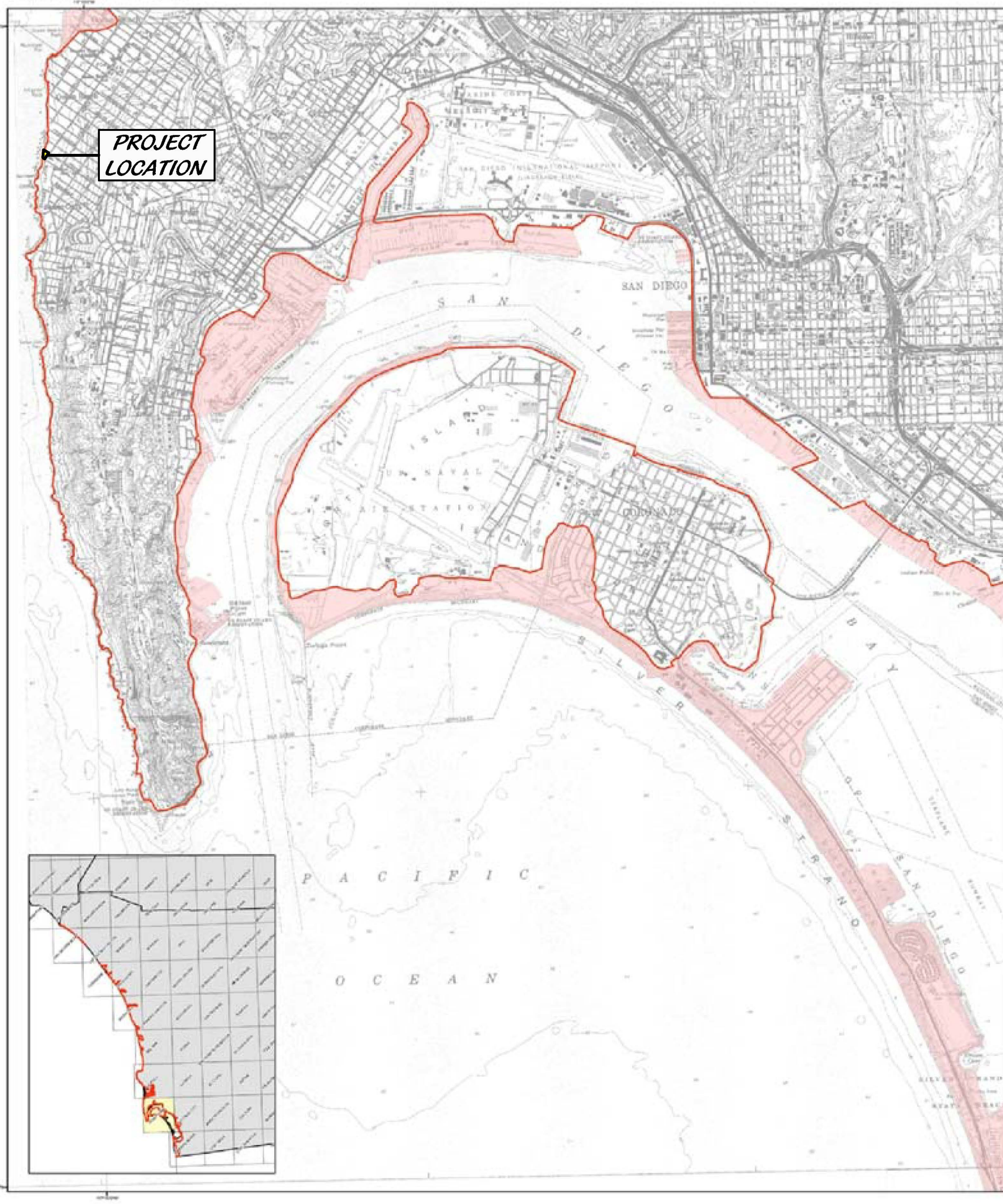
PROJECT NUMBER **2317-01**

GENERALIZED GEOLOGIC MAP

California Emergency Management Agency
California Geological Survey
University of Southern California

Tsunami Inundation Map for Emergency Planning
Point Loma Quadrangle

State of California
County of San Diego



METHOD OF PREPARATION

Initial tsunami modeling was performed by the University of Southern California (USC) Tsunami Research Center funded through the California Emergency Management Agency (CalEMA) by the National Tsunami Hazard Mitigation Program. The tsunami modeling process utilized the MOST (Method of Splitting Tsunamis) computational program (Version 0), which allows for wave evolution over a variable bathymetry and topography used for the inundation mapping (Titov and Gonzalez, 1997; Titov and Synolakis, 1998).

The bathymetric/topographic data that were used in the tsunami models consist of a series of nested grids. Near-shore grids with a 3 arc-second (75- to 90-meters) resolution or higher, were adjusted to "Mean High Water" sea-level conditions, representing a conservative sea level for the intended use of the tsunami modeling and mapping.

A suite of tsunami source events was selected for modeling, representing realistic local and distant earthquakes and hypothetical extreme undersea, near-shore landslides (Table 1). Local tsunami sources that were considered include offshore reverse-thrust faults, restraining bends on strike-slip fault zones and large submarine landslides capable of significant seafloor displacement and tsunami generation. Distant tsunami sources that were considered include great subduction zone events that are known to have occurred historically (1960 Chile and 1964 Alaska earthquakes) and others which can occur around the Pacific Ocean "Ring of Fire."

In order to enhance the result from the 75- to 90-meter inundation grid data, a method was developed utilizing higher-resolution digital topographic data (3- to 10-meters resolution) that better defines the location of the maximum inundation line (U.S. Geological Survey, 1993; Intermap, 2003; NOAA, 2004). The location of the enhanced inundation line was determined by using digital imagery and terrain data on a GIS platform with consideration given to historic inundation information (Lander, et al., 1993). This information was verified, where possible, by field work coordinated with local county personnel.

The accuracy of the inundation line shown on these maps is subject to limitations in the accuracy and completeness of available terrain and tsunami source information, and the current understanding of tsunami generation and propagation phenomena as expressed in the models. Thus, although an attempt has been made to identify a credible upper bound to inundation at any location along the coastline, it remains possible that actual inundation could be greater in a major tsunami event.

This map does not represent inundation from a single scenario event. It was created by combining inundation results for an ensemble of source events affecting a given region (Table 1). For this reason, all of the inundation region in a particular area will not likely be inundated during a single tsunami event.

References:

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Lander, J.F., Lockridge, P.A., and Kozuch, M.J., 1993, Tsunamis Affecting the West Coast of the United States 1806-1992: National Geophysical Data Center Key to Geophysical Record Documentation No. 29, NOAA, NESDIS, NGDC, 242 p.

National Atmospheric and Oceanic Administration (NOAA), 2004, Interferometric Synthetic Aperture Radar (IfSAR) Digital Elevation Models from GeoSAR platform (EarthData): 3-meter resolution data.

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U.S. Geological Survey, 1993, Digital Elevation Models: National Mapping Program, Technical Instructions, Data Users Guide 5, 48 p.

TSUNAMI INUNDATION MAP FOR EMERGENCY PLANNING

State of California ~ County of San Diego

POINT LOMA QUADRANGLE

June 1, 2009

SCALE 1:24,000

Table 1: Tsunami sources modeled for the San Diego County coastline.

Sources (M = moment magnitude used in modeled event)		Areas of Inundation Map Coverage and Sources Used: Dana Point	Oceanside	San Diego
Local Sources	Carlsbad Thrust Fault		X	X
	Catalina Fault	X	X	X
	Coronado Bank Fault			X
	Lasuen Knoll Fault	X		X
	San Clemente Fault Bend Region			X
	San Clemente Island Fault			X
	San Mateo Thrust Fault	X	X	
	Coronado Canyon Landslide #1			X
Distant Sources	Cascadia Subduction Zone #3 (M9.2)	X		X
	Central Aleutians Subduction Zone#1(M8.9)	X	X	X
	Central Aleutians Subduction Zone#2(M8.9)	X		X
	Central Aleutians Subduction Zone#3(M9.2)	X	X	X
	Chile North Subduction Zone (M9.4)	X		X
	1960 Chile Earthquake (M9.3)	X		X
	1952 Kamchatka Earthquake (M9.0)	X		
	1964 Alaska Earthquake (M9.2)	X	X	X
	Japan Subduction Zone #2 (M8.8)	X		X
	Kuril Islands Subduction Zone #2 (M8.8)	X		X
	Kuril Islands Subduction Zone #3 (M8.8)	X		X
	Kuril Islands Subduction Zone #4 (M8.8)	X		X

Cal EMA

USC
UNIVERSITY OF SOUTHERN CALIFORNIA

MAP EXPLANATION

Tsunami Inundation Line

Tsunami Inundation Area

PURPOSE OF THIS MAP

This tsunami inundation map was prepared to assist cities and counties in identifying their tsunami hazard. It is intended for local jurisdictional, coastal evacuation planning uses only. This map, and the information presented herein, is not a legal document and does not meet disclosure requirements for real estate transactions nor for any other regulatory purpose.

The inundation map has been compiled with best currently available scientific information. The inundation line represents the maximum considered tsunami runup from a number of extreme, yet realistic, tsunami sources. Tsunamis are rare events; due to a lack of known occurrences in the historical record, this map includes no information about the probability of any tsunami affecting any area within a specific period of time.

Please refer to the following websites for additional information on the construction and/or intended use of the tsunami inundation map:

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http://www.oes.ca.gov/WebPage/oeswebsite.nsf/Content/B1EC
51BA215931768825741F005E8D80?OpenDocument

University of Southern California – Tsunami Research Center:
http://www.usc.edu/dept/tsunamis/2005/index.php

State of California Geological Survey Tsunami Information:
http://www.conservation.ca.gov/cgs/geologic_hazards/Tsunami/index.htm

National Oceanic and Atmospheric Agency Center for Tsunami Research (MOST model):
http://nctr.pmel.noaa.gov/time/background/models.html

MAP BASE

Topographic base maps prepared by U.S. Geological Survey as part of the 7.5-minute Quadrangle Map Series (originally 1:24,000 scale). Tsunami inundation line boundaries may reflect updated digital orthophotographic and topographic data that can differ significantly from contours shown on the base map.

DISCLAIMER

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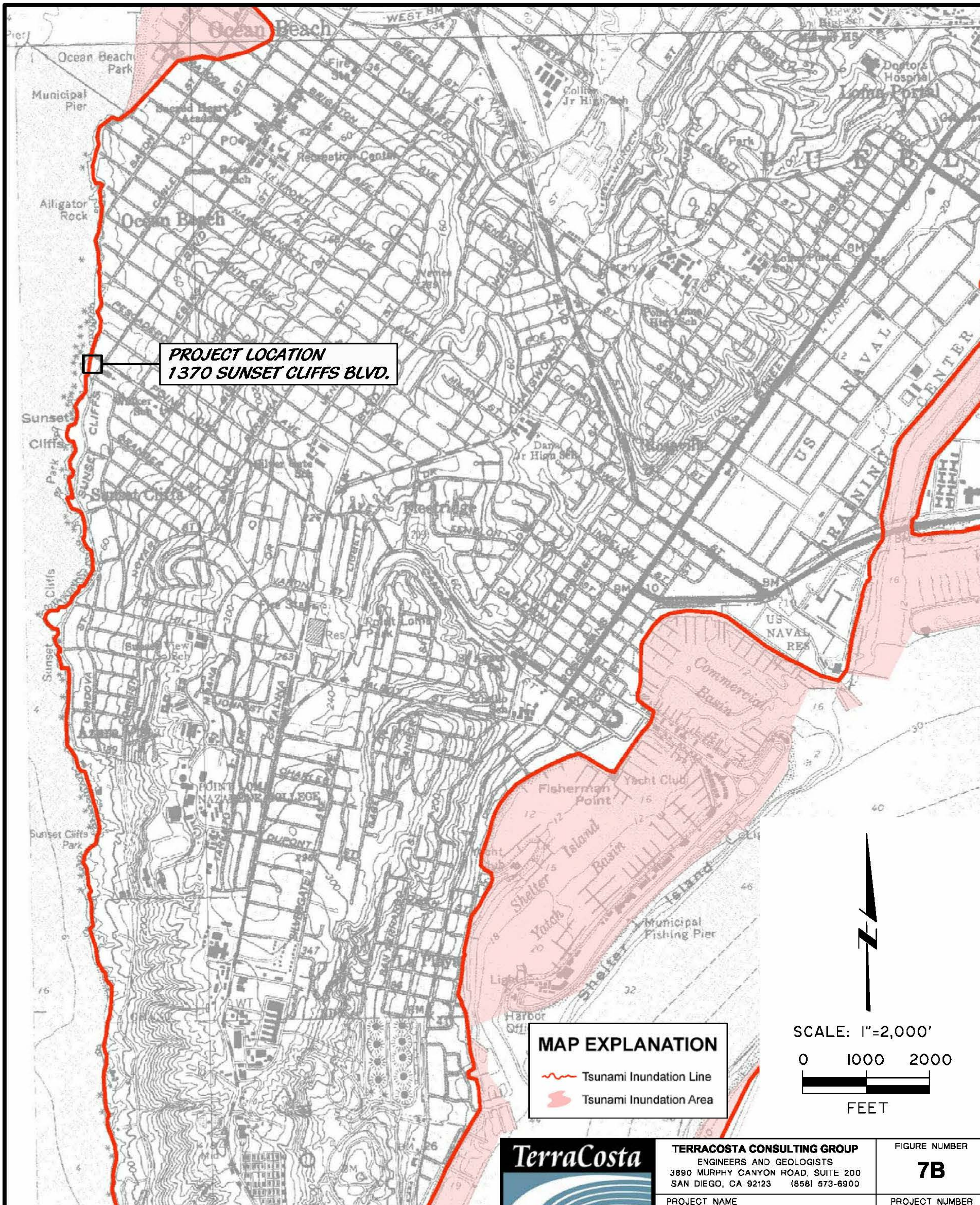
TerraCosta Consulting Group

FIGURE NUMBER
7A

PROJECT NUMBER
2317-01

TSUNAMI INUNDATION MAP
POINT LOMA QUADRANGLE



Reproduced from: "Tsunami Inundation Map for Emergency Planning, Point Loma Quadrangle," by the State of California, County of San Diego, dated June 1, 2009.

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER **7B**
	PROJECT NAME **INN AT SUNSET CLIFFS**	PROJECT NUMBER **2317-01**
	TSUNAMI INUNDATION MAP	

METHOD OF PREPARATION

Initial tsunami modeling was performed by the University of Southern California (USC) Tsunami Research Center funded through the California Emergency Management Agency (CalEMA) by the National Tsunami Hazard Mitigation Program. The tsunami modeling process utilized the MOST (Method of Splitting Tsunamis) computational program (Version 0), which allows for wave evolution over a variable bathymetry and topography used for the inundation mapping (Titov and Gonzalez, 1997; Titov and Synolakis, 1998).

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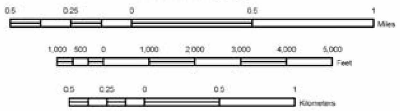
TSUNAMI INUNDATION MAP FOR EMERGENCY PLANNING

State of California ~ County of San Diego

POINT LOMA QUADRANGLE

June 1, 2009

SCALE 1:24,000



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	San Clemente Fault Bend Region			X
	San Clemente Island Fault			X
	San Mateo Thrust Fault	X	X	
	Coronado Canyon Landslide #1			X
Distant Sources	Cascadia Subduction Zone #3 (M9.2)	X		X
	Central Aleutians Subduction Zone#1(M8.9)	X	X	X
	Central Aleutians Subduction Zone#2(M8.9)	X		X
	Central Aleutians Subduction Zone#3(M9.2)	X	X	X
	Chile North Subduction Zone (M9.4)	X		X
	1960 Chile Earthquake (M9.3)	X		X
	1952 Kamchatka Earthquake (M9.0)	X		
	1964 Alaska Earthquake (M9.2)	X	X	X
	Japan Subduction Zone #2 (M8.8)	X		X
	Kuril Islands Subduction Zone #2 (M8.8)	X		X
	Kuril Islands Subduction Zone #3 (M8.8)	X		X
	Kuril Islands Subduction Zone #4 (M8.8)	X		X

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CALIFORNIA EMERGENCY MANAGEMENT AGENCY

CGS
CALIFORNIA GEOLOGICAL SURVEY

USC
UNIVERSITY OF SOUTHERN CALIFORNIA

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State of California Geological Survey Tsunami Information:
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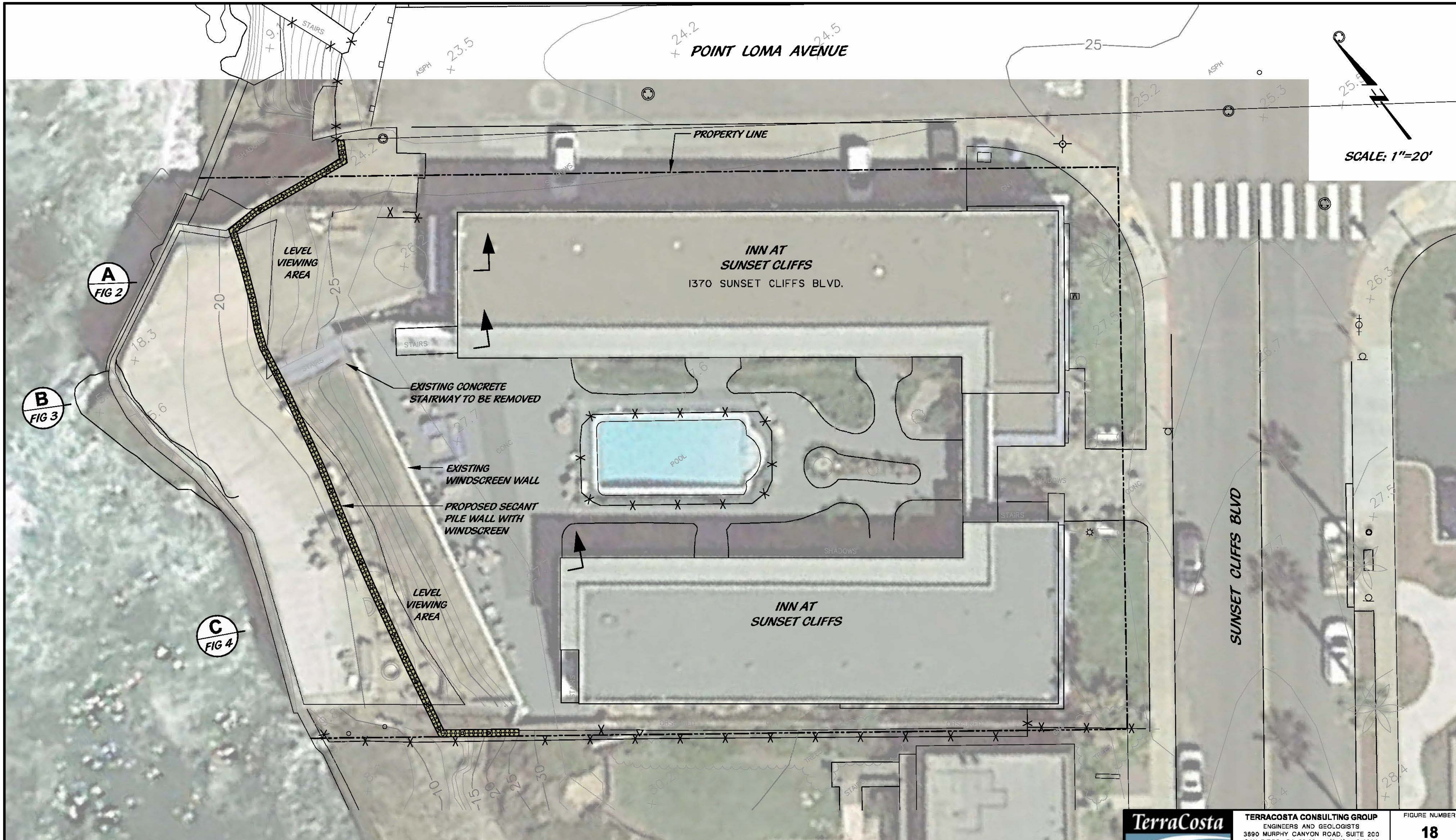
TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
3890 MURPHY CANYON ROAD, SUITE 200
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER	PROJECT NAME	PROJECT NUMBER
7C	INN AT SUNSET CLIFFS	2317-01

TSUNAMI INUNDATION MAP LEGEND



APPROXIMATE AREA OF WESTERLY VIEWING AREA: 2,120 SQUARE FEET

TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
3890 MURPHY CANYON ROAD, SUITE 200
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER	PROJECT NAME	PROJECT NUMBER
18	INN AT SUNSET CLIFFS	2317

CONCEPT PLAN

APPENDIX A

GEI FEBRUARY 2004 REPORT

REPORT OF SEA CLIFF EDGE EVALUATION AND DECK SUPPORT RECOMMENDATIONS

Inn at Sunset Cliffs

1370 Sunset Cliffs Boulevard

San Diego, California

JOB NO. 03-8530

02 February 2004

Prepared for:

INN at SUNSET CLIFFS, L.L.C.

GEOTECHNICAL EXPLORATION, INC.

SOIL & FOUNDATION ENGINEERING • GROUNDWATER
HAZARDOUS MATERIALS MANAGEMENT • ENGINEERING GEOLOGY

02 February 2004

INN at SUNSET CLIFFS, L.L.C.
1529 West Seldon Lane
Phoenix, AZ 85021
Attn: Mr. Dan Fischer

Job No. 03-8530

INN at SUNSET CLIFFS
1370 Sunset Cliffs Blvd.
San Diego, CA 92107
Attn: Ms. Crystal Petersen and Mr. Marc Boyea

Subject: **Report of Sea Cliff Edge Evaluation and Deck Support Recommendations**
Inn at Sunset Cliffs
1370 Sunset Cliffs Blvd.
San Diego, California

Dear Mr. Fischer, Ms. Petersen and Mr. Boyea:

In accordance with your request, and per our proposal of October 6, 2003, ***Geotechnical Exploration, Inc.*** has prepared this report of sea cliff edge evaluation and deck support recommendations for the subject site. The field work was performed in November and December 2003.

In our opinion, the location of the sea cliff edge has been identified and the deck area can be improved as planned providing the recommendations herein are followed.

This opportunity to be of service is appreciated. If you have any questions regarding this matter, please contact our office. Reference to our **Job No. 03-8530** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Leslie D. Reed, President
C.E.G. 999[exp. 3-31-05]/R.G. 3391

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REPORT OF SEA CLIFF EDGE EVALUATION AND DECK SUPPORT RECOMMENDATIONS

Inn at Sunset Cliffs
1370 Sunset Cliffs Boulevard
San Diego, California

JOB NO. 03-8530

The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the Inn at Sunset Cliffs project located at 1370 Sunset Cliffs Boulevard, San Diego, California.

I. EXECUTIVE SUMMARY

The Inn at Sunset Cliffs located at 1370 Sunset Cliffs Boulevard, consists of 2, two-story structures separated by a swimming pool. The two structures containing guest rooms and the reception office are joined by a breezeway fronting on Sunset Cliffs Boulevard. The guest entry, building, and rectangular pool are oriented in a west-northwesterly direction, paralleling Point Loma Avenue, which provides access to lower-level garage entries along the north side of the northern building. For ease of reference, we will, throughout this report, refer to the seaward side of the property as being west, the entry side off Sunset Cliffs Boulevard as being east, and the buildings as being either north (fronting on Point Loma Avenue) or south.

In addition to the building and pool area improvements, a stairway provides access from the upper pool and decking recreation area to lower-level concrete decking. The lower deck and concrete improvements extend across the full north-south width of the property and are bounded on the west (seaward) side by a sea wall structure. Based on aerial photo review, the Inn was constructed between 1950 and 1953. For reference, we have included in Appendix A 1939, 1950 and 1953 aerial photo stereo pairs and oblique aerial photos (1960± and 1972) that clearly show the primary structures, pool and upper recreation deck, and westerly sea wall. The lower recreation area appears, in the Appendix A 1960± photo, to consist of imported beach sand suggesting that at the time of the photograph, the lower concrete decking may not have been placed.

It is our understanding, based on communications with Mr. Paul Benton, Project Engineer/Architect, that it is proposed to extend a second-story deck from the west end of the northern structure. In addition, future improvements may be made to the deck area around the west end of the existing swimming pool. Although the two existing buildings and rectangular swimming pool are actually oriented in a west northwesterly direction, we will refer to the seaward edge of the property as being west.

Our investigation revealed that the sea cliff edge is located at an approximate elevation of 25 feet above Mean Sea Level (MSL) and located 30 to 33 feet west of the western end of the northern building that fronts on Point Loma Avenue. Fill soils varying from 12 to 15 inches in depth were revealed by trenches T-1, T-2, T-4 and T-5 to be underlain by marine terrace materials correlated with the Bay Point Formation (Qbp). The Bay Point formational materials are primarily medium dense, slightly silty, fine to medium sands. They are dark reddish brown in color and display bioturbation characteristic of near-shore burrowing fauna. The top edge of the sea cliff is clearly defined by a sharp break in slope and westerly increasing thicknesses of fill soils containing large chunks of concrete debris. It appears that, especially in test trench T-5, debris-laden fill soils were end dumped over the low sea cliff prior to or during original construction between 1950 and 1953. Several photographs are presented in Appendix B.

The upper deck and pool area of the Inn and Sunset Cliffs may eventually receive elevated deck improvements. Based on our exploratory trenches, we believe the upper deck to be underlain at shallow depth by medium dense sands of the Bay Point Formation. In our opinion, these materials will provide adequate support for new spread footings, if designed in accordance with the criteria presented in this report. An alternative foundation system consisting of drilled caissons placed into the dense Point Loma Formation bedrock materials may also be considered.

With the above in mind, the Scope of Work is briefly outlined as follows:

1. Define the location of the natural sea cliff edge as it existed prior to development of the property, including review of historical topographic maps and aerial photographs.

2. Identify and classify the surface and subsurface soils to depths, in conformance with the Unified Soil Classification System (refer to Appendix A).

3. Evaluate the geotechnical aspects of the site relative to potential deck improvements.

4. Recommend geotechnical design criteria for alternative deck foundation systems.

Undermined sea wall geotechnical criteria are not included in this report. Exploratory work and consultation have been performed in direct cooperation with the sea wall project engineer, Mr. Paul Benton, to expedite processing for remedial work. A letter report and graphics will be prepared to document the findings utilized by Mr. Benton in his wall remediation design.

II. SITE DESCRIPTION

The property is defined as Assessor's Parcel No. 448-341-01-00, Lot 1, Block 27, per Recorded Map 1889. The property consists of 0.6-acre along Sunset Cliffs

Boulevard, in the Sunset Cliffs/ Point Loma area of the City of San Diego (refer to Figure No. I). The property is bounded to the north and west by sea wall-protected natural sea cliffs, beach and the Pacific Ocean, to the south by commercial property at the same elevation, to the north by Point Loma Avenue, and to the east by Sunset Cliffs Boulevard (see Figure No. II).

The property is currently terraced with an upper, relatively level building pad area along the eastern property boundary along Sunset Cliffs Boulevard. The north side of the upper pad steps down to garages along Point Loma Avenue, and a west-facing slope descends approximately 4 to 5 feet to a lower terraced, deck area at an elevation of approximately 20 feet above MSL.

Prior to construction of the existing sea wall, a sea cliff descended from between the upper and lower decks to the rocky and sandy beach below the property. The top of the sea cliff lies below the existing elevation contour of 25 feet above mean sea level (MSL). Refer to Figure No. II for the location of the 25-foot MSL contour and underlying top of sea cliff.

Two, 2-story residential hotel structures, a swimming pool, and associated improvements currently exist on the property. The building pad elevation is approximately 27 feet above MSL (refer to Figure No. II). The lower western deck area, between the short west-facing slope and the lower face of sea cliff, is retained by an up to 25-foot-high sea wall founded in bedrock materials of the lower sea cliff. The wall is approximately 210 feet long. Portions of the north end of the wall have been undermined. The wall foundation and sea cave evaluation, as stated previously, are not part of this evaluation.

Vegetation on the site consists of lawn grass, groundcover, small trees and ornamental shrubs.

III. FIELD INVESTIGATION

Five hand-dug excavations were placed in the western portion of the site in December 2003 to locate the top edge of the sea cliff and to allow mapping of the top edge in plan view. The excavations were placed where feasible due to existing improvements. The soils were logged by our engineering geologists and samples were taken of the predominant soils throughout the field operation. (For the excavation locations and mapping of the top edge of the sea cliff, refer to Figure No. II.) Laboratory test results are presented on Figure No. IIIa-c.

Excavation logs have been prepared on the basis of our observations and the results have been summarized on Figure Nos. IVa-c, Cross-sections A-A′, B-B′, and C-C′. Individual trench logs T-1 through T-5 have been included on the three cross sections. The predominant soils have been classified in conformance with the Unified Soil Classification System (refer to Appendix C).

IV. SOIL AND GENERAL GEOLOGIC DESCRIPTION

The exploratory excavations exposed minor thicknesses of debris-laden silty sand and clayey sand fill soils overlying natural formational materials. Formational material consisted of a 4- to 7-foot thickness of Quaternary Bay Point Formation marine terrace deposits overlying the Cretaceous Point Loma Formation. The Point Loma formational materials are exposed at the western base of the sea wall, forming natural near-vertical and benchlike (sub-tidal) outcrops. These materials are identified as the Upper Cretaceous-age Point Loma Formation (Kp) on geologic maps of the site (Bulletin 200 of the California Division of Mines and Geology). Point Loma Formation outcrops were also observed beneath the lower deck during sea wall/sea cave evaluation (Appendix B, Photo 1). The primary purpose of placing the exploratory excavations was to observe the nature of the contact between the fill soils and natural materials such that the relief of the contact could be directly observed for criteria associated with defining the edge or top of the sea cliff.

A. *Stratigraphy*

Artificial Fill Soils (Qaf): In general, relatively shallow artificial fill soils underlie the decks and gravel surface of the western edge of the upper terrace on the site (Appendix B, Photo 2). The fill soils contain minor to significant amounts of concrete and construction-related debris mixed with silty, fine to medium sands (Appendix B, Photo 3). The fill soils are loose and not suitable for support of improvements without proper cleaning and recompaction. The fill soils directly overlie the sands of the Quaternary Bay Point Formation (Qbp). The fills rapidly thicken and concrete debris increases beyond the western edge of the top of the sea cliff.

Bay Point Formation (Qt/Qbp): As exposed in the exploratory excavations, the Bay Point Formation consists of dark brown, dark orange/orange-brown, slightly silty, fine to medium sands grading downward into tan, slightly silty sand and tan/olive sand that appears to have been derived from the Point Loma Formation. Despite the variations in color, the materials are uniform in texture and grain size (refer to Figure Nos. IIIa and IIIc) and display features characteristic of natural formational material, such as bioturbation (Appendix B, Photo 4) and sublinear, near-vertical iron/manganese accumulations (Appendix B, Photos 5 and 6). The color change to light material is believed to be due to the derivation of basal Bay Point sediments from the directly underlying sands of the Cretaceous Point Loma Formation.

Point Loma Formation (Kp): The Upper Cretaceous Point Loma Formation was observed at the base of the sea wall, beneath and behind the sea wall and in exploratory excavation T-2. As stated above, the lighter colored sands comprising the lower Bay Point Formation are believed to be derived from the underlying Point Loma Formation. The Point Loma Formation comprises most of the coastal sea cliff and is visible as outcrops north and south of the site. It consists of interbedded fine to medium-grained, yellowish to reddish brown, silty sand/sandstone and olive-gray sandy silt that occur in variable-thickness beds up to 1 foot thick. It is relatively well indurated (dense).

B. Geologic Structure

The Quaternary Bay Point Formation at the location of the subject property appears to be a regressive marine sand deposited on the planated surface of the Upper Cretaceous Point Loma Formation. Although bedding was not present in the massively bedded material exposed in exploratory trenches nearby, coastal exposures suggest the materials are flat-lying and have not been disturbed to a detectable degree by faulting or tectonic activity. The underlying Point Loma Formation, as mapped by Kennedy (1975), generally strikes northwest-southeast with shallow easterly dips 4 to 9 degrees in the vicinity of the site. Small, east-west to northeast-southwest trending faults within the Point Loma Formation are mapped by Kennedy (1975) south of the subject property. Relatively high-angle joints, trending northwest-southeast, are also mapped in this unit.

C. Geologic Hazard Designation

A review of the City of San Diego Seismic Safety Study -- Geologic Hazards, Sheet 16, indicates that the site is located within low to moderate risk Geologic Hazard Category 43, which refers to the coastal bluffs (sea cliffs) on the site as "*generally unstable and/or unfavorable jointing, local high erosion.*" The Rose Canyon Fault and associated faults designated as an active fault zone are over 2 miles to the east, northeast and southeast.

V. GROUNDWATER

No free groundwater was encountered in our exploratory excavations during the course of our field investigation and significant groundwater problems are not expected to develop in the future -- if proper drainage and subdrainage is maintained on the property. The site is at the western margin of the marine terrace, which slopes toward the Pacific Ocean, and groundwater is commonly encountered or develops in the terrace materials following regional development.

It must be understood, however, that unless discovered during initial site exploration or encountered during construction operations, it is difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

Water conditions, where suspected or encountered during construction operations, should be evaluated and remedied by the project civil and geotechnical consultants. The project builder and property owner, however, must realize that post-construction appearances of groundwater may have to be dealt with on a site-specific basis.

VI. LABORATORY TESTS AND SOIL INFORMATION

Laboratory tests were performed on the predominant underlying soil materials in order to evaluate the physical and mechanical properties of the marine terrace materials and their ability to support potential future deck structure support systems. The following tests were conducted on the sampled soils:

1. *Moisture Content (ASTM D2216-98)*
2. *Moisture/Density Relations (ASTM D1557-98, Method A)*
3. *Density Measurements (ASTM D1188-90 and D1556-98)*
4. *Mechanical Analysis (ASTM D422-98)*

The relationship between the moisture and density of soil samples gives qualitative information regarding soil strength characteristics and soil conditions.

The Mechanical Analysis Test was used to aid in the classification of the soils according to the Unified Soil Classification System.

Based upon our experience with the Bay Point and Point Loma formational materials in this area of San Diego, our observations of the primary soil types on the project, our laboratory test data, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has utilized conservative values for friction angle and cohesion for those soils that will have significant lateral support or bearing functions on the project. These values have been utilized in recommending the allowable bearing value as well as the active and passive earth pressures for footing and caisson designs. Laboratory test results are presented on Figure No. IIIa-c.

VII. SEA CLIFF EDGE EVALUATION AND DISCUSSION

We have researched and reviewed historical photographs, topographic maps and other available reference materials and site evidence that document the historic sea cliff conditions on the western portion of the property.

Sources of information reviewed by ***Geotechnical Exploration, Inc.*** include the following aerial photographs:

Date	Description/Type	Source
1939	Stereo pair high angle	US Army Corps of Engineers
11/1/50	Stereo pair high angle	National Oceanic & Atmospheric Association (NOAA)
5/2/53	Stereo pair high angle	USDA
1960±	Low angle oblique view of lower deck	Provided by client
1972	Low angle oblique (color)	S.D. Historical Society

The following map sources of information were also utilized in our analysis:

Date	Description/Type	Source
1954	Topographic Map Lambert Coordinates 210-1689 (1"=200')	City of San Diego Maps & Records
1978	Orthophotographic Map Lambert Coordinates 210-1689 (1"=200')	City of San Diego Maps & Records
10/21/03	Topographic Survey	San Dieguito Engineering, Inc.

The primary geologic unit underlying the site, and forming most of the west end coastal sea cliff, consists of the Upper Cretaceous Point Loma Formation (Kp). It also forms the foreshore area of the coast along which a seasonal sand and/or cobble beach exists, as well as offshore intertidal and subtidal ledges. The Point Loma Formation is overlain by relatively shallow thicknesses of the Quaternary Bay Point Formation (Qbp).

Based on the five exploratory trenches placed by our firm, as well as review of historic photographs, we have located the top of the sea cliff defined by the upper rim of the Bay Point Formation. The actual break in slope, as shown on the Plot Plan (Figure No. II) and in cross sections A-A' and B-B', occurs below the "current" elevation contour of 25 feet MSL. The actual top of sea cliff elevatin is approximately 23 feet MSL. The location of the top of sea cliff at the south end of the property was based primarily on the 1939, 1950 and 1970s photographs. Past placement of concrete debris over the face of the upper portion of the sea cliff aided in field identification of the top edge of the Bay Point Formation.

The Shoreline Erosion Assessment and Atlas of the San Diego Region, Volume II, prepared by the California Department of Boating and Waterways and San Diego Association of Governments (1994) profiles this area of the Sunset Cliffs coastline as having "*inadequate setback*" and "*moderate risk.*" The document states:

> "High rocky, nearly vertical cliffs with many rocky coves and narrow, sandy pocket beaches are backed by road and residential area. Cliff erosion in this area is critical. Cliffs are undercut by wave action forming many sea caves. Subaerial erosion from surface runoff, overwatering, and animal burrowing cause numerous rock falls and surficial slope failures along the base, face, and top of cliffs. Houses and apartments on the face of the cliffs in the northern portion of this section are subject to damage or loss from further erosion. Buildings have been condemned and others are poised on the rim of the cliff. Overall, documented rates of erosion along this section average about 3 inches per year (Kennedy, 1973), but site-specific erosion up to 75 feet between 1952 and 1976 has been observed (Kuhn and Shepard, 1984).
>
> The cliff bedrock is composed of the Late Cretaceous, 63- to 90-million-year-old Point Loma Formation, a dense, olive-gray, clay shale interbedded with dusky-yellow sandstone. It typically extends seaward several thousand feet as a submerged shelf. This unit is overlain by the Mid-Pleistocene, 120,000-year-old Bay Point Formation, a poorly to moderately consolidated, fossiliferous silty sandstone (Crampton and Forrest, 1981). The Bay Point erodes at an

> accelerated rate compared to the Point Loma Formation. In the northern portion of the section, the contact between the Point Loma and the Bay Point formations is low in elevation, so wave action has eroded the upper Bay Point farther, producing a wide bedrock bench. This area is a popular place to observe the tide pools, but can be hazardous during periods of high waves and high tides.
>
> As the Point Loma Formation bedrock rises to the south, the bench narrows and marine erosion of the Bay Point becomes less of a factor, and subaerial erosion dominates. At the south end of the section, the bedrock erosion rate is about equal to the terrace material erosion rate, as evidenced by the nearly vertical cliff and the absence of the bedrock terrace at the contact. The small headlands that are prevalent along this section are developed by the collapse of sea caves and by differential erosion along areas with higher occurrences of joints and fractures. Sea caves are generally developed along discontinuities in the bedrock unit, such as along faults or open joints."

The upper Cretaceous Point Loma Formation was encountered in exploratory excavation T-2. It was also observed as outcrop along the base of the sea wall and within the void beneath the lower (westerly) deck. As noted, it comprises the lower portion of the sea cliff and is visible as outcrops on the coast to the north and south along this area of Point Loma. It consists of interbedded fine to medium-grained, yellowish brown, massively bedded sandstone and olive-gray clay shale that occurs in variable-thickness beds up to 1 foot thick. It is well indurated in both its lower silty/shale portion and upper sandier portion. As mapped by Kennedy (1975), these materials generally strike northwest-southeast with shallow northeasterly dips of 4 to 9 degrees in the vicinity of the site.

Rates of erosion of the Cretaceous sandstone have been examined by various researchers. Emery (1941) determined the rate of Cretaceous sandstone erosion to be about 0.02-foot/year for sites along the northern La Jolla shoreline, and Kennedy (1973) determined rates of erosion in the Sunset Cliffs area to be 3 to 4 feet/century, or 0.03 to 0.04 feet/year. Recession of the lower sea cliff at the site controls the rate of recession of the Quaternary Bay Point Formation/terrace deposits, which form the upper portion of the property. Pocket cove sea cliff exposures a short distance south of the property display near-vertical cliffs with no setback bench below the Bay Point Formation. However, headlands, which receive more intense wave erosion, do display a top of Point Loma Formation bench and a setback to the toe of the Bay Point Formation.

It is well known that block fall or mass wastage is usually the controlling factor in sea cliff recession along most of the San Diego County coastline. Undercut and blockfall retreat rates were not readily available as this site seems to have experienced little mass wastage. The 1939 and 1950 aerial photos reveal the westerly end of the property to consist of a relatively resistant headland that has been further protected by and continues to be protected by the sea wall that was constructed in the 1950s.

We have addressed rock strength characteristics for the Point Loma Formation described above and their influence on site stability. The well-indurated interbeds of sandstone and shale possess relatively good strength characteristics. Our Geotechnical Engineer has assigned an angle of internal friction of 32 degrees and a conservative cohesion of 500 psf for these materials based on our experience with testing of these rock/soil properties on other projects.

It is our opinion that the sea cliff face and site should be stable inland of the 25-foot setback for a period of at least 75 years. The current "sea cliff edge" is depicted on Figure No. II. Improvements located at the 25-foot setback or greater are considered to be located over stable bedrock conditions.

VIII. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the field investigation conducted by this firm in conjunction with our knowledge and experience with the soils and sea cliffs along the Sunset Cliffs/Point Loma area of the City of San Diego.

A. Top of Sea Cliff Location

Our investigation revealed that the western portion of the lot is underlain by medium dense to dense formational materials of the Cretaceous-age Point Loma Formation (Kp) and the Quaternary Bay Point Formation/Terrace Materials (Qbp). Overlying these materials are minor thicknesses of artificial fill soils. The encountered artificial fill soils are of variable loose to medium dense consistency and, in our opinion, are not suitable as bearing materials and are not suitable for support of new structural loads. We recommend that the new foundations for the deck addition extend through the fill soils and topsoils to bear directly on the underlying formational materials.

As has been described previously, the top of the sea cliff edge has been defined by placing hand-excavated test trenches. The trenches were oriented in an east-west direction and were excavated from east to west until the sharp break in slope of the Bay Point Formation was encountered. The top edge of the sea cliff, as encountered and as mapped based on historic aerial photos, is presented in plan view on Figure No. II and in cross section on Figure Nos. IVa-c. Photographs presented in Appendix B document some of the physical features utilized in identification of the Bay Point Formation.

B. Design Parameters for Proposed Deck Foundations Using Shallow Footings

1. For preliminary foundation design of new footings, based on the assumption that new footings will be placed at least 18 inches into medium dense to dense natural materials (i.e., the Bay Point Formation), we provide an allowable soil bearing capacity equal to 2,500 pounds per square foot (psf). This applies to footings at least 18 inches into the bearing soils and at least 12 inches in width. For wider and/or deeper footings, the allowable soil bearing capacity may be calculated based on the following equation:

$$Qa = 1200D+700W$$

where

"Qa" is the allowable soil bearing capacity (in psf);

"D" is the depth of the footing (in feet) as measured from the lowest adjacent grade; and

"W" is the width of the footing (in feet).

The allowable soil bearing capacity may be increased one-third for analysis including wind or earthquake loads. Up to 4,000 psf may be allowed for total vertical bearing capacity for foundations in medium dense to dense, sound formation. All foundations shall be in dense natural formation.

2. The passive earth pressure of the encountered medium dense, natural-ground soils (to be used for design of shallow foundations and footings to resist the lateral forces) may be based on an Equivalent Fluid Weight of 300 pounds per cubic foot (pcf) for formational soils. This passive earth pressure

shall only be considered valid for design if the ground adjacent to the foundation structure is essentially level for a distance of at least three times the total depth of the foundation and is properly compacted or dense native soil.

3. An allowable Coefficient of Friction of 0.40 times the dead load may be used between the bearing soils and concrete foundations.

4. The following table summarizes site-specific seismic design criteria to calculate the base shear needed for the design of future deck structures. The design criteria was obtained from the California Building Code (2001 edition) based on the soil characteristics and distance to the closest fault (4.7 miles from the Rose Canyon Fault).

Parameter	Value	Reference
Seismic Zone Factor, Z	0.40	Table 16-I
Soil Profile Type	S_c	Table 16-J
Seismic Coefficient, C_a	$0.40N_a$	Table 16-Q
Seismic Coefficient, C_v	$0.56N_v$	Table 16-R
Near-Source Factor, N_a	1.0	Table 16-S
Near-Source Factor, N_v	1.0	Table 16-T
Seismic Source Type	B	Table 16-U

Based upon our laboratory test results and our experience with the soil types on the subject site, the underlying formational materials should experience a total settlement of less than 1 inch and a differential settlement in the magnitude of approximately 1 inch, under a structural load within the allowable bearing capacity. The angular rotation due to differential settlement is anticipated to be less than 1/240.

5. A minimum of steel for spread footings embedded a minimum of 18 inches into medium dense to dense formational soils should include at least four No. 5 steel bars continuous, with two bars 3 inches from the bottom of the footing and two bars 2 inches from the top. More steel would be required for larger footings. Reinforcing shall be provided per the structural engineer's drawings.

   Isolated square footings should contain, as a minimum, a grid of No. 4 steel bars on 12-inch centers, in both directions, with no less than three bars each way.

   NOTE: The project Civil/Structural Engineer shall review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but primarily as minimum safeguards to reduce possible crack separations. The actual reinforcing schedule shall be as per the direction of the Civil/Structural Engineer.

C. <u>Design Parameters for Proposed Deck Foundations Using Caisson Foundation Systems</u>

6. Soil design parameters and caisson-related recommendations are provided in case the proposed elevated decks and related improvements are to be founded on drilled piers (caissons). Drilled piers or caissons shall be embedded in the firmer Point Loma Formation a depth of not less than 5 feet. In addition, drilled caissons when drilled closed to the bluff, they shall be provided with a minimum 7-foot setback from which the effective depth of embedment for vertical and lateral resistance shall be calculated.

7. We recommend that caissons be drilled with a diameter no smaller than 24 inches. The bottom of the excavations shall be cleaned by the contractor to leave no more than 1 inch of loose (slough) or muddy soils at the bottom of the drilled excavation. The contractor shall provide an adequate cleaning tool to comply with the above requirement. Furthermore, if drilling in areas of existing loose fills, shoring shall be provided to reduce the soil cave-in potential.

8. Caissons shall be designed by the project structural engineer to properly support the vertical and lateral loads transmitted by the columns or improvements to be supported, and transmit those loads to the bearing soils. The controlling depth of caissons shall be based on the resistance needed to support the vertical and lateral loads.

9. For vertical capacity, caissons shall be embedded not less than 10 feet from the surface, and at least 5 feet into Point Loma formational soils. Variations in soil stratigraphy may require deeper drilling at some locations to ensure proper bearing on dense sandstone. In addition, in areas close to the sea cliff face, the caissons shall start counting passive resistance when the lateral daylight distance is at least 7 feet, or at least 3 times the diameter of the caisson, whichever is larger. Continued observations by a representative of our firm should help confirm the proper depth into Point Loma formational soils.

10. For vertical loading, the minimum center-to-center spacing of caissons shall be 3 diameters. To calculate the total lateral load resistance of isolated caissons, the calculations may consider the passive resistance of one caisson diameter times 2.5 the projected passive resisting length (2.5 diameter times passive resistance length). Allowable soil passive resistance is 150 pcf for

existing loose soils; 275 for Bay Point formation and any properly compacted soils above formational soils; and 350 pcf for dense formational Point Loma formation soils. A soil friction coefficient of 0.4 may also be used, if applicable at the bottom of caisson caps or grade beams.

Caissons aligned in the same direction of the lateral load shall consider the shadow effect by reducing the calculated allowable load by a reduction factor that depends on the spacing between caissons, as follows: If the center to center caisson spacing in the direction of the lateral load is 3B, 4B, 5B, 6B, 7B, of 8B, the group reduction factor shall be 3, 2.6, 2.2, 1.8, 1.4, and 1.0, respectively.

11. The recommended allowable end bearing vertical capacity of caissons drilled at least 5 feet into Point Loma Formation is 10,000 psf. The compressible vertical frictional resistance may be calculated by using an average shaft friction 500 psf of shaft surface, in dense Point Loma formational soils. The required caisson length and embedment into formational soils shall be established by the structural engineer based on the length needed to adequately support the total vertical and lateral loads included in the design. For uplift loads, the allowable frictional resistance may be calculated by using an average 250 psf in Point Loma Formational soils plus the weight of the caisson.

12. If the pole equation is used for the minimum depth of lateral load resistance, the maximum lateral bearing of soils is 4,200 psf for Point Loma formational soils. If the fixity concept is used to calculate the maximum moment, then depth of fixity is 7 feet below the surface , or at least one foot in Point Loma formational soils.

13. Cured frictional caissons should experience soil settlement in the order of less than 1 inch.

14. The design and construction of caissons shall be in accordance with the recommendations presented herein, the current UBC requirements accepted by the City of San Diego, and also in accordance with ACI 336, 3R-93 Design and Construction of Drilled Piers.

15. Caisson excavation shall be filled with concrete within 2 days after excavations are completed, to help reduce the risk of soil caving, mud or slough intrusion, etc.

16. If collapsible soils are encountered during drilling, shoring or slurry shall be used to keep the excavation open. If groundwater is encountered, the tremie method shall be used.

17. The contractor shall follow Cal-OSHA safety guidelines and regulations to help prevent personal injury.

D. General Recommendations

18. Appropriate erosion-control measures shall be taken at all time during construction to prevent surface runoff waters from entering footing excavations.

19. Where not superseded by specific recommendations presented in this report, trenches, excavations and temporary slopes at the subject site shall be constructed in accordance with Title 8, Construction Safety Orders, issued by

OSHA. This office should be contacted for additional recommendations if shoring or steep temporary slopes are required.

20. In order to reduce any work delays at the subject site during site development, this firm should be contacted 24 hours prior to any need for observation of footing excavations, temporary unshored excavation slopes, or field density testing of compacted fill soils. Placement of formwork and steel reinforcement in footing excavations should not occur prior to our observation of the excavations; in the event that our observations reveal the need for deepening or redesigning foundation structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

21. Any required grading operations such as for any new slabs on-grade (patios, walkways, etc.) shall be performed in accordance with the General Earthwork Specifications (Appendix D) and the requirements of the City of San Diego Grading Ordinance.

22. ***Geotechnical Exploration, Inc.*** recommends that we be asked to verify the actual soil conditions revealed during footing excavations to be as anticipated in this report. In addition, the compaction of any fill soils placed during site construction must be tested.

23. It is the responsibility of the owner to ensure that the recommendations summarized in the report are carried out in the field operations and that our recommendations for design of the project are incorporated in the construction plans. It is recommended that we review the foundation plans

prior to construction operations to verify that the intent of our recommendations are incorporated in the plans, and to verify that any additional or modified recommendations that are warranted are included in the plans.

24. This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein to be unsafe.

IX. LIMITATIONS

Our conclusions and recommendations have been based upon all available data obtained from the field investigation and laboratory analysis, as well as our experience with the soils and native materials located in the Sunset Cliffs/Point Loma area of the City of San Diego. Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. *It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time construction operations begin, when temporary slopes are excavated, or when footing excavations are placed. In the event discrepancies are noted, additional recommendations may be issued, if required.* The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the County of San Diego. This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report, or any work done without our observations and testing.

Should you have any questions, please feel free to contact our office. Reference to our **Job No. 03-8530** will help expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Leslie D. Reed, President
C.E.G. 999[exp. 3-31-05]/R.G. 3391

REGISTERED GEOLOGIST
LESLIE D REED NO 999
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JAIME A. CERROS
No. 002007
Exp. 9-30-05
GEOTECHNICAL
STATE OF CALIFORNIA

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JOB NO. 03-8530
February 2004

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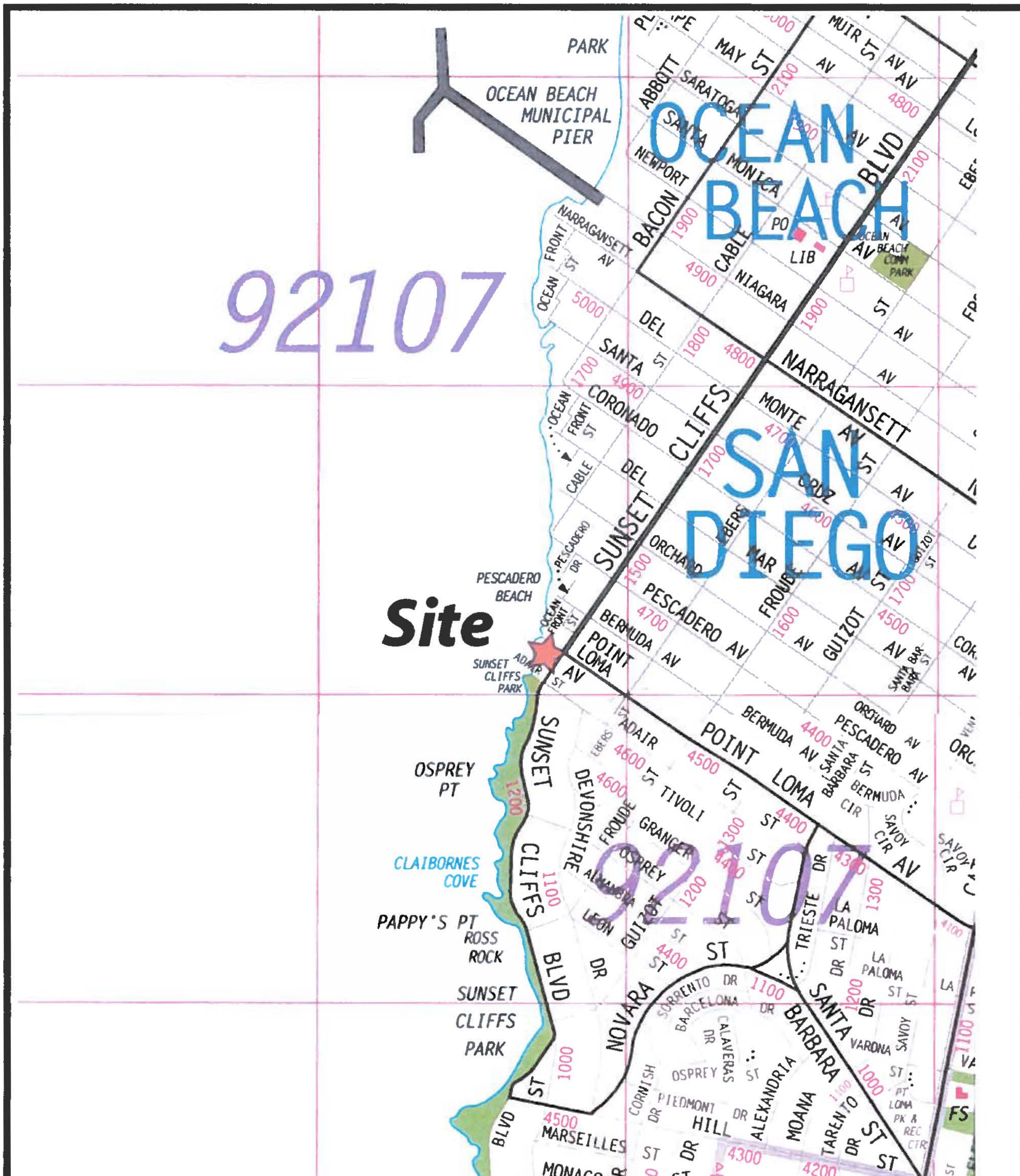
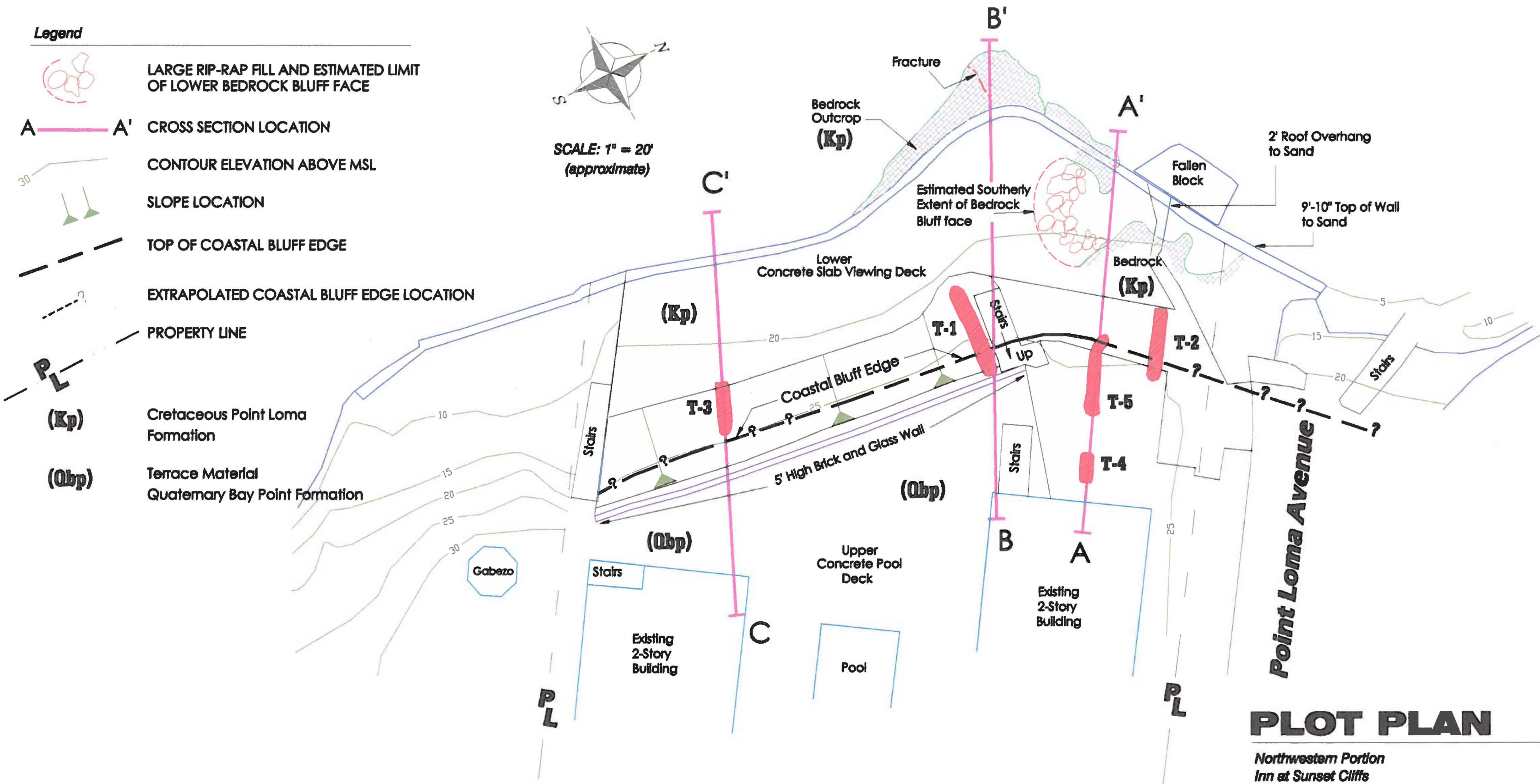
VICINITY MAP



Thomas Bros Guide - San Diego County pg. 1267

Inn at Sunset Cliffs
1370 Sunset Cliffs Boulevard
San Diego, CA.

Figure No. I
Job No. 03-8530



PLOT PLAN

Northwestern Portion
Inn at Sunset Cliffs
1370 Sunset Cliffs Boulevard
San Diego, CA.
Figure No. II
Job No. 03-8530

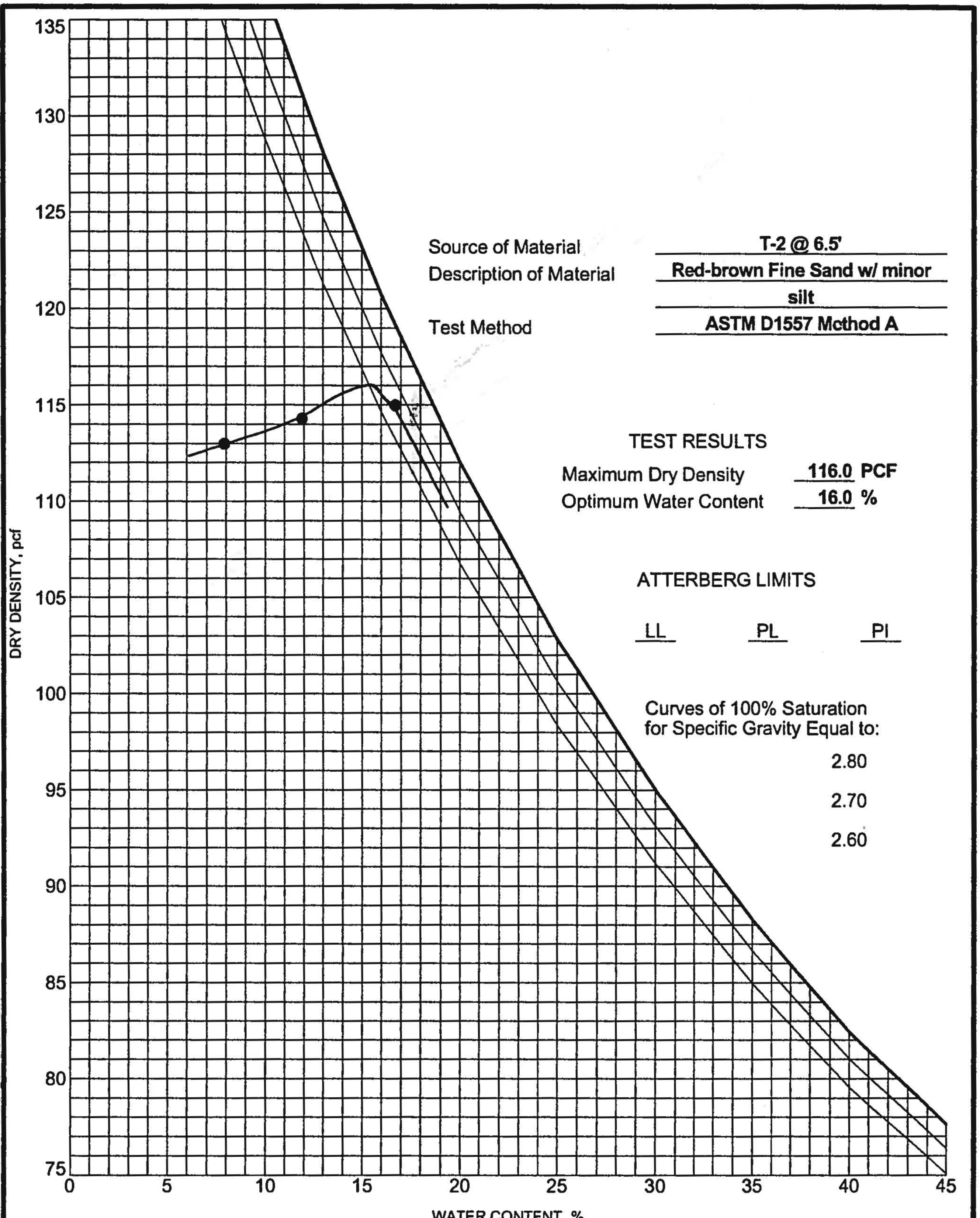
Geotechnical Exploration, Inc.

January 2004

NOTE: This Plot Plan is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

REFERENCE: This Plot Plan was prepared on-site field reconnaissance performed by GEI and from a Topographic Survey prepared by San Dieguito Engineering dated 10/21/03

03-8530-p

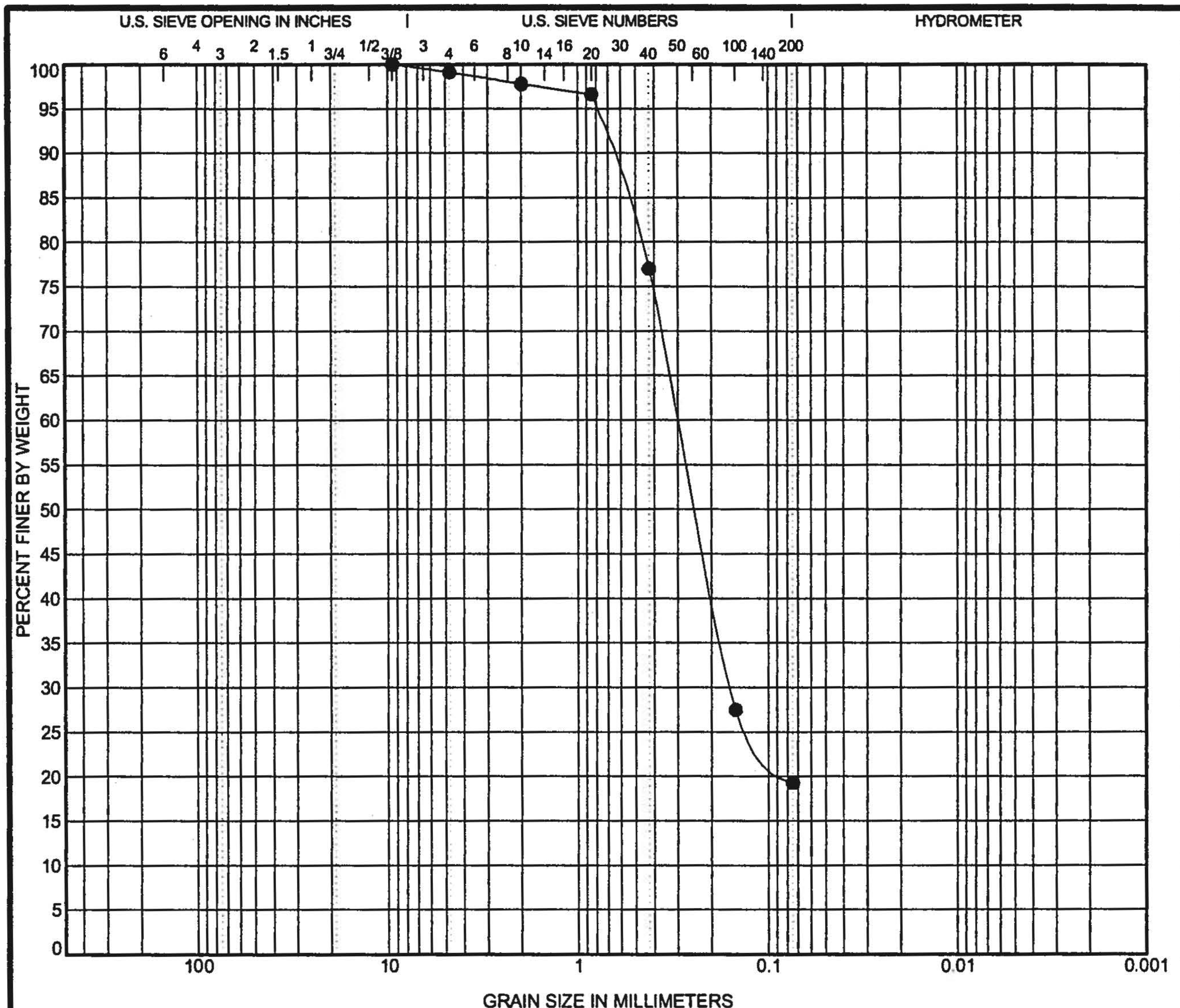


US COMPACTION 8530 SUNSET CLIFFS.GPJ GEO_EXPL.GDT 2/5/04

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: IIIa
Job Name: The Inn at Sunset Cliffs
Site Location: 1370 Sunset Cliffs Blvd., San Diego, Califo
Job Number: 03-8530



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	T-1 @ 3.5'	Red-brown Fine Sand w/ some silt					

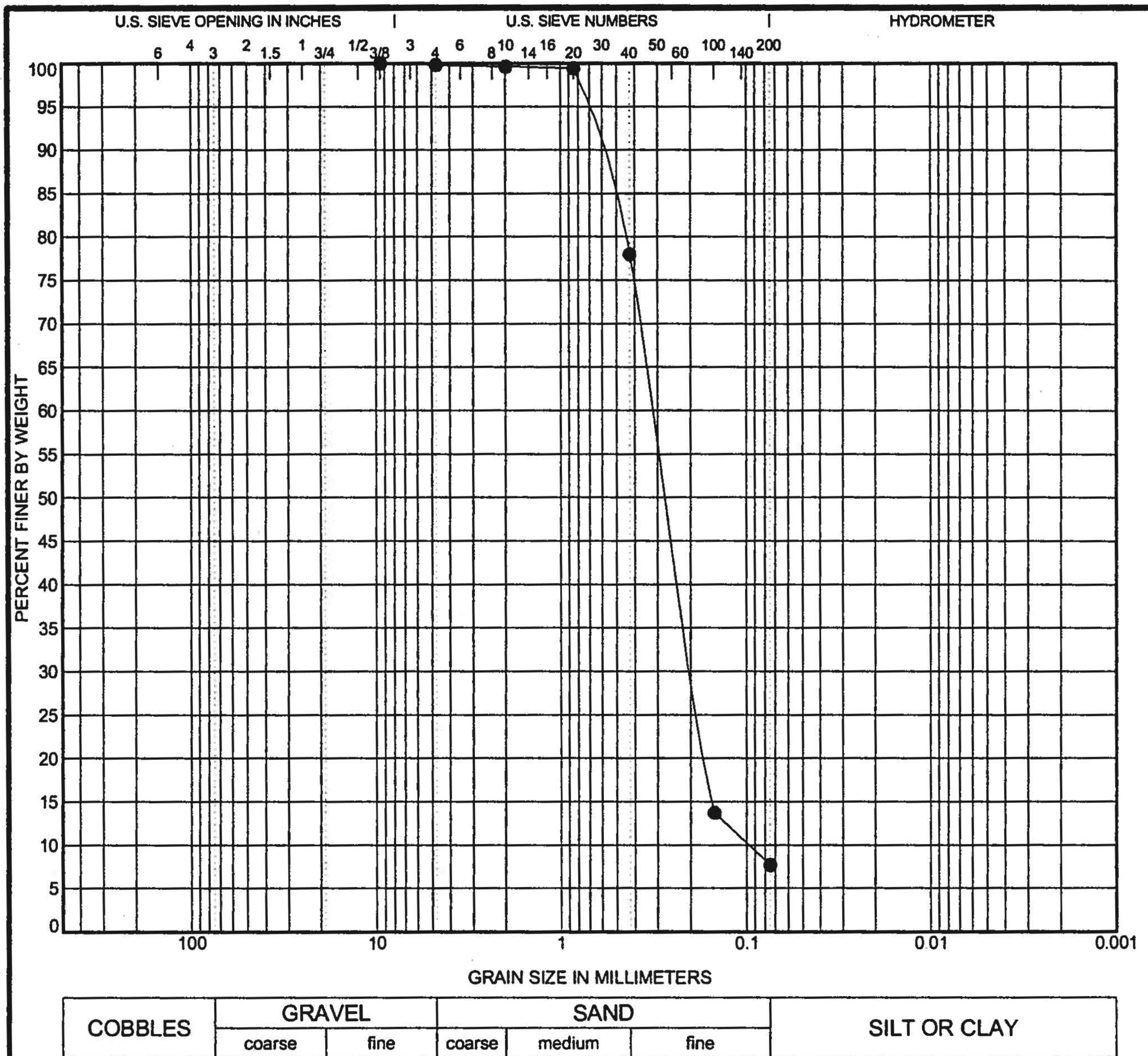
	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	T-1 @ 3.5'	9.5	0.297	0.158		0.9	79.8	19.3	

US_GRAIN_SIZE 8530 SUNSET CLIFFS.GPJ GEO_EXPL.GDT 2/5/04

GEI Geotechnical Exploration, Inc.

GRAIN SIZE DISTRIBUTION

Figure Number: IIIb
Job Name: The Inn at Sunset Cliffs
Site Location: 1370 Sunset Cliffs Blvd., San Diego, Califo[rnia]
Job Number: 03-8530



	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	**T-2 @ 6.5'**	**Red-brown Fine Sand w/ minor silt**				**1.23**	**3.25**

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**T-2 @ 6.5'**	**9.5**	**0.318**	**0.195**	**0.098**	**0.2**	**92.1**	**7.7**	

Geotechnical Exploration, Inc.

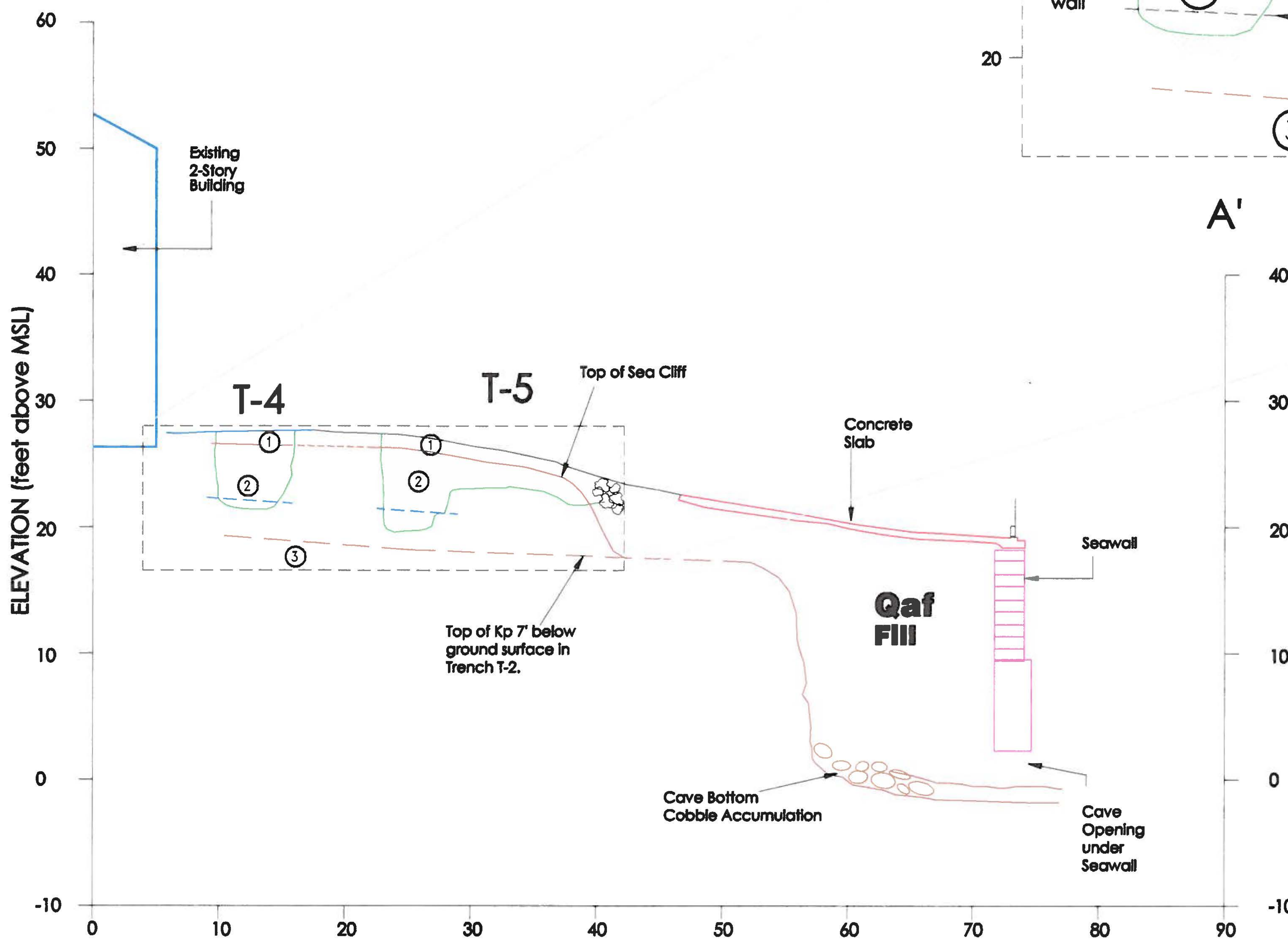
GRAIN SIZE DISTRIBUTION

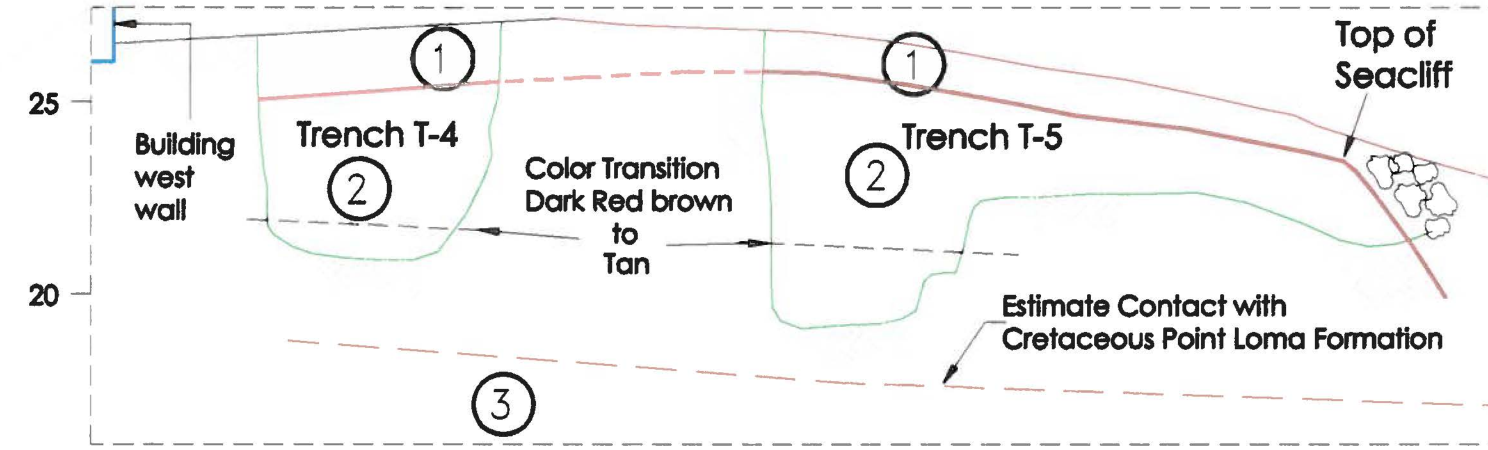
Figure Number: IIIc
Job Name: The Inn at Sunset Cliffs
Site Location: 1370 Sunset Cliffs Blvd., San Diego, Califo
Job Number: 03-8530

US_GRAIN_SIZE 8530 SUNSET CLIFFS.GPJ GEO_EXPL.GDT 2/5/04

CROSS SECTION A-A'

Inn @ Sunset Cliffs





SCALE 1" = 5'

UNIT #	LOCATION	UNIT DESCRIPTION
	Fill	**ARTIFICAL FILL (Qaf)**
	0 to 4"/6"	Silty Sand and Gravel. Loose, dry, very light brown
1	4"/6" to 12"/15"	Silty Fine to Medium Sand and Mixed Angular Cobble-Boulder size Concrete Debris. Loose, damp, brown Buried Concrete Rubble Debris placed on sea cliff face. Angular, cobble to boulder size.
2	**Bay Point Formation** 12"/15" to 5'-6"	**Terrace Materials (Qbp)** Slightly Silty Fine to Medium Sand. Highly bioturbated. Medium dense, damp-moist, dark reddish brown, subvertical black linear manganese/ /iron mineral accumulations. 7" to 9" long. Color transition to tan at approximate elev: 21' to 22' MSL.
3	**Point Loma Formation**	**Cretaceous Point Loma Formation Kp** Estimated contact between Qbp/Kp 18' to 19' feet above Mean Sea Level.

RELATIVE HORIZONTAL DISTANCE (Feet)
SCALE 1" = 10' (Horizontal and Vertical)

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Reference: This Plot Plan was prepared from on-site field reconnaissance performed by GEI and from a Topographic Survey perpared by San Dieguito Engineering dated 10/21/03.

Inn at Sunset Cliffs
1370 Sunset Cliffs Boulevard
San Diego, CA.
Figure No. IVa
Job No. 03-8530

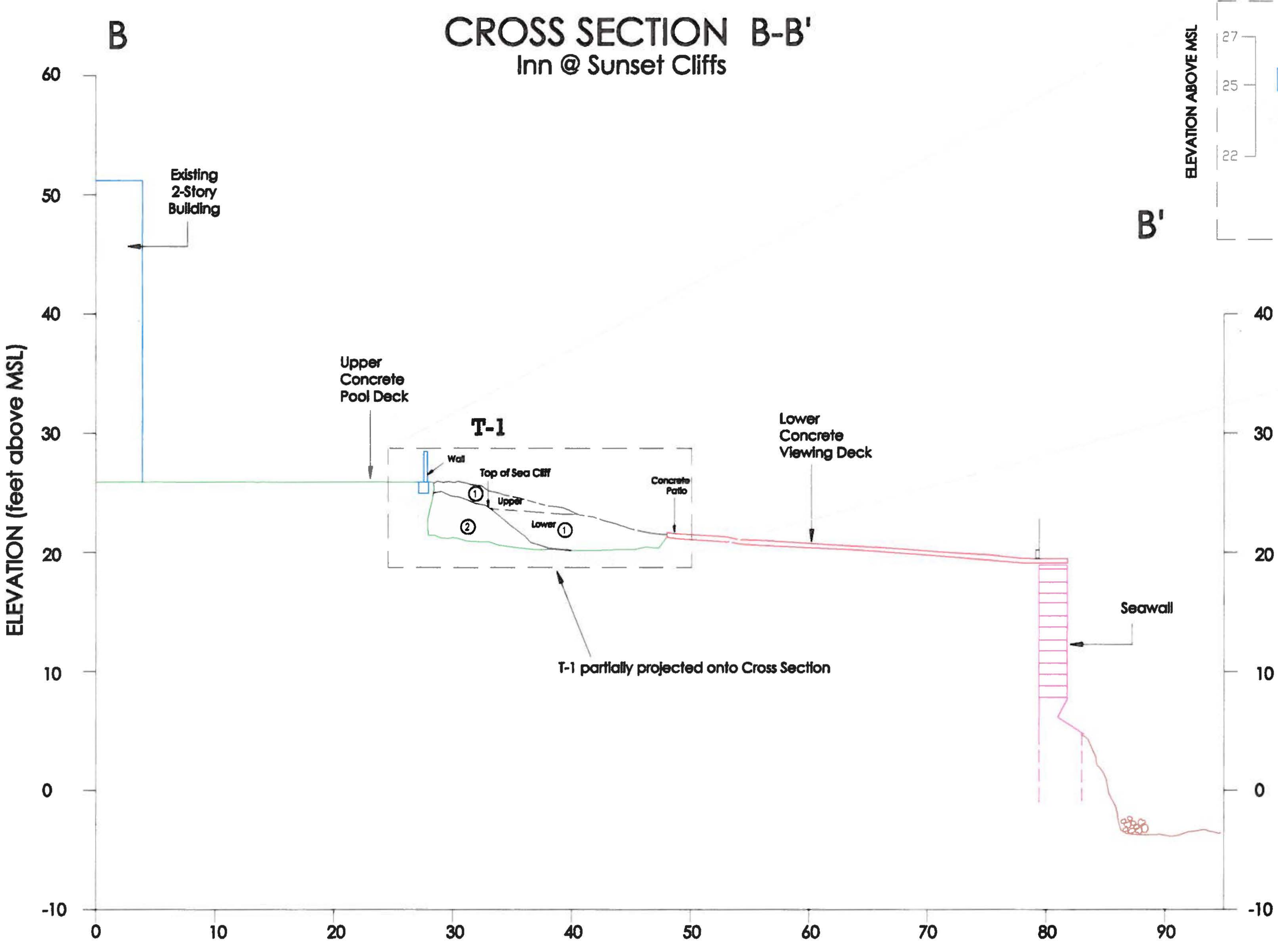
Geotechnical Exploration, Inc.

January 2004

03-8530-DD

CROSS SECTION B-B'

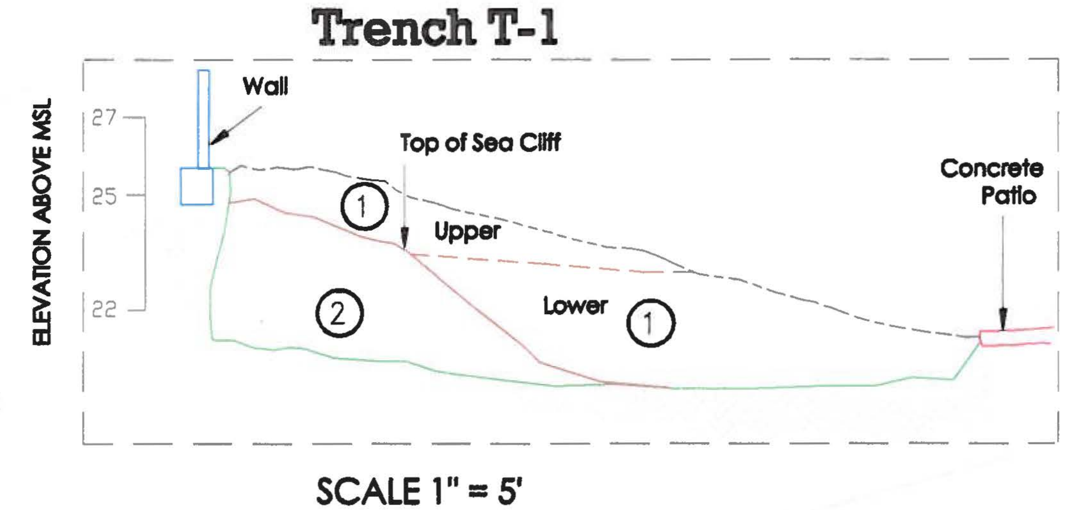
Inn @ Sunset Cliffs



RELATIVE HORIZONTAL DISTANCE (Feet)

SCALE 1" = 10' (Horizontal and Vertical)

Trench T-1



SCALE 1" = 5'

UNIT #	LOCATION	UNIT DESCRIPTION
		ARTIFICAL FILL (Qaf)
1	**Upper Fill**	Gravelly Sand, loose, dry-damp, gray/brown/tan with mixed cobble-size augular concrete debris.
	Lower Fill	Fine to Medium Sand. Loose-medium dense, damp-moist, buff/pale brown/yellow white. "beach sand derived"
		TERRACE MATERIALS (Qbp)
2	**Bay Point Formation**	Slightly Silty fine to medium sand. Medium dense, moist, brown/dark brown.

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

REFERENCE: This Cross Section was prepared from on-site field reconnaissance performed by GEI and from a Topographic Survey prepared by San Dieguito Engineering dated 10/21/03.

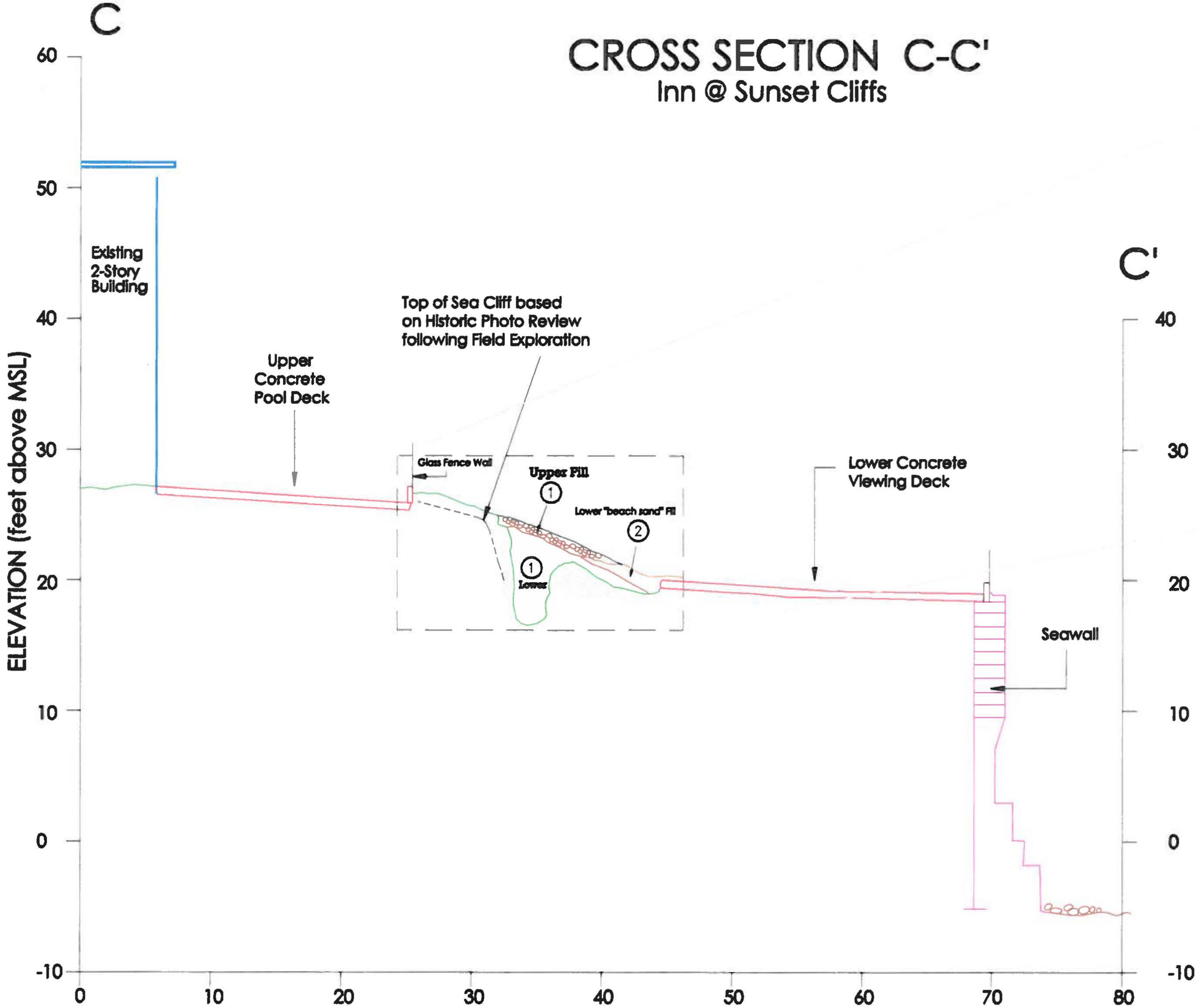
Inn at Sunset Cliffs
1370 Sunset Cliffs Boulevard
San Diego, CA.
Figure No. IVb
Job No. 03-8530

Geotechnical Exploration, Inc.

January 2004

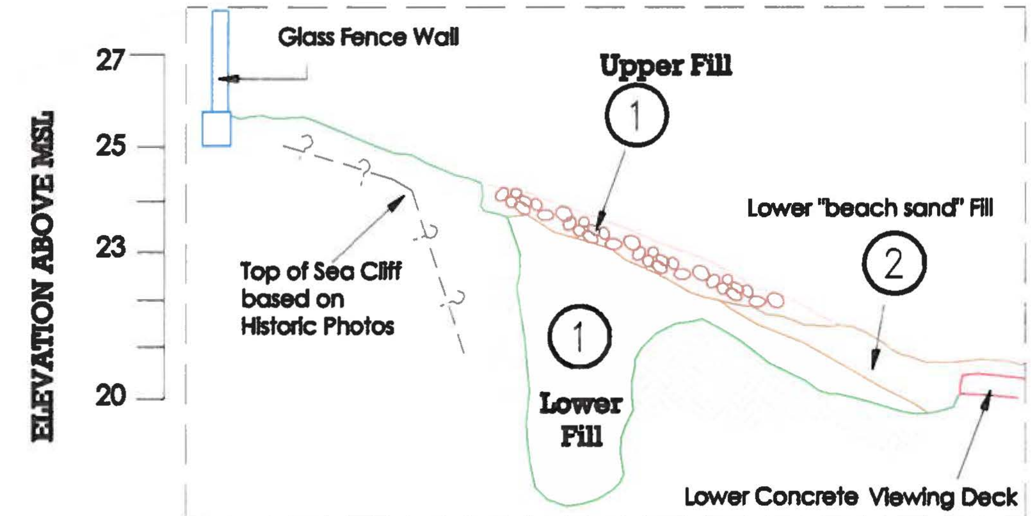
03-8530-EE

CROSS SECTION C-C'
Inn @ Sunset Cliffs



RELATIVE HORIZONTAL DISTANCE (Feet)

SCALE 1" = 10' (Horizontal and Vertical)



SCALE 1" = 5'

UNIT #	LOCATION	UNIT DESCRIPTION
		ARTIFICIAL FILL (Qaf)
1	**Upper Fill**	Silty Fine to Medium Sand. Loose-medium dense, damp, tan/brown/gray with angular gravel and angular cobble-boulder size concrete debris.
2	**Lower Fill**	Fine to medium sand. Loose-medium dense, damp, brown with scattered angular concrete and brick debris.
	Lower "beach sand fill"	Fine to Medium Sand, Loose-medium dense, damp, pale tan/white/gold brown. "beach sand derived".

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

REFERENCE: This Cross Section was prepared from on-site field reconnaissance performed by GEI and from a Topographic Survey prepared by San Dieguito Engineeering dated 10/21/03.

Inn at Sunset Cliffs
1370 Sunset Cliffs Boulevard
San Diego, CA.
Figure No. IVc
Job No. 03-8530

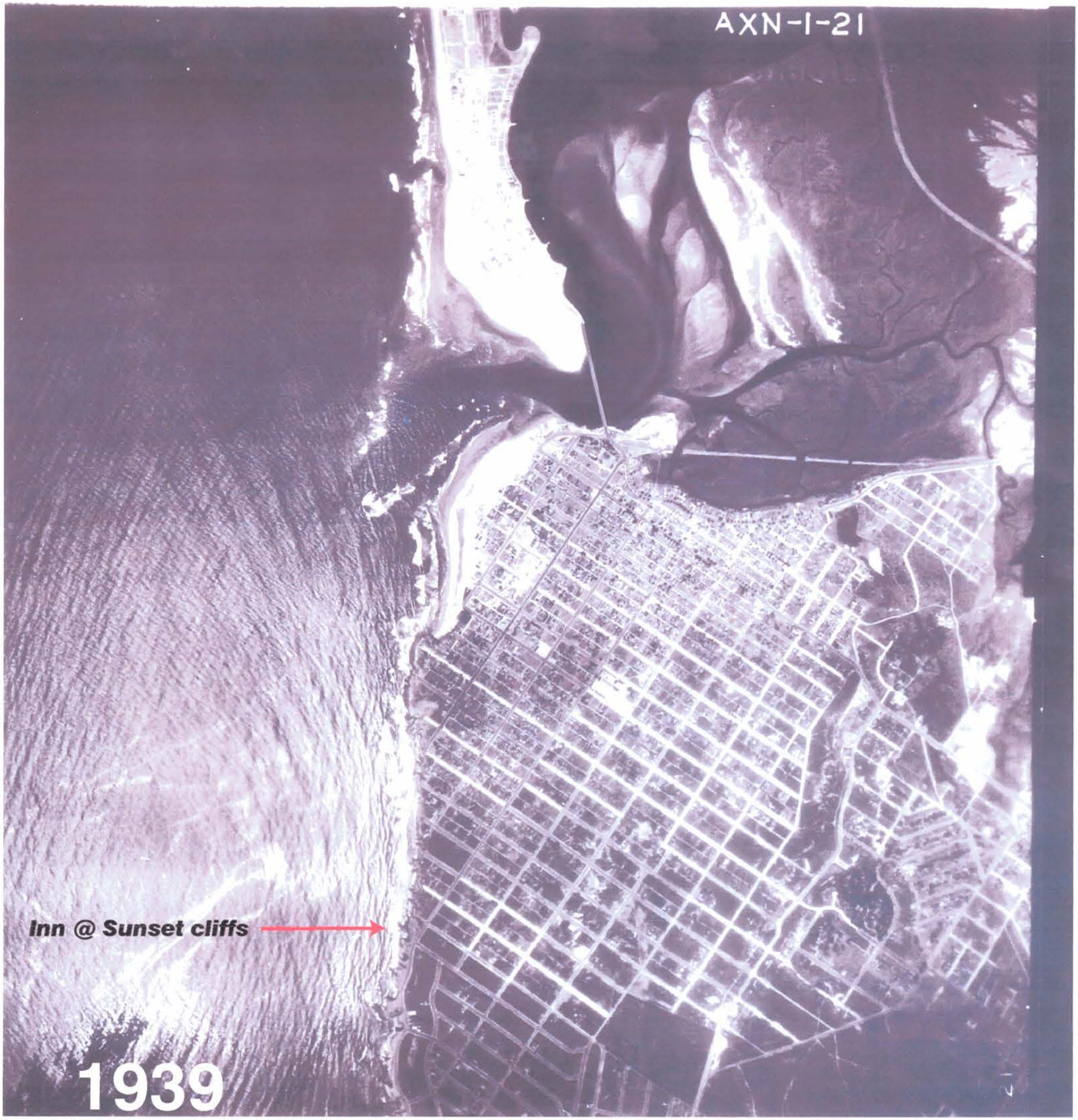
Geotechnical Exploration, Inc.

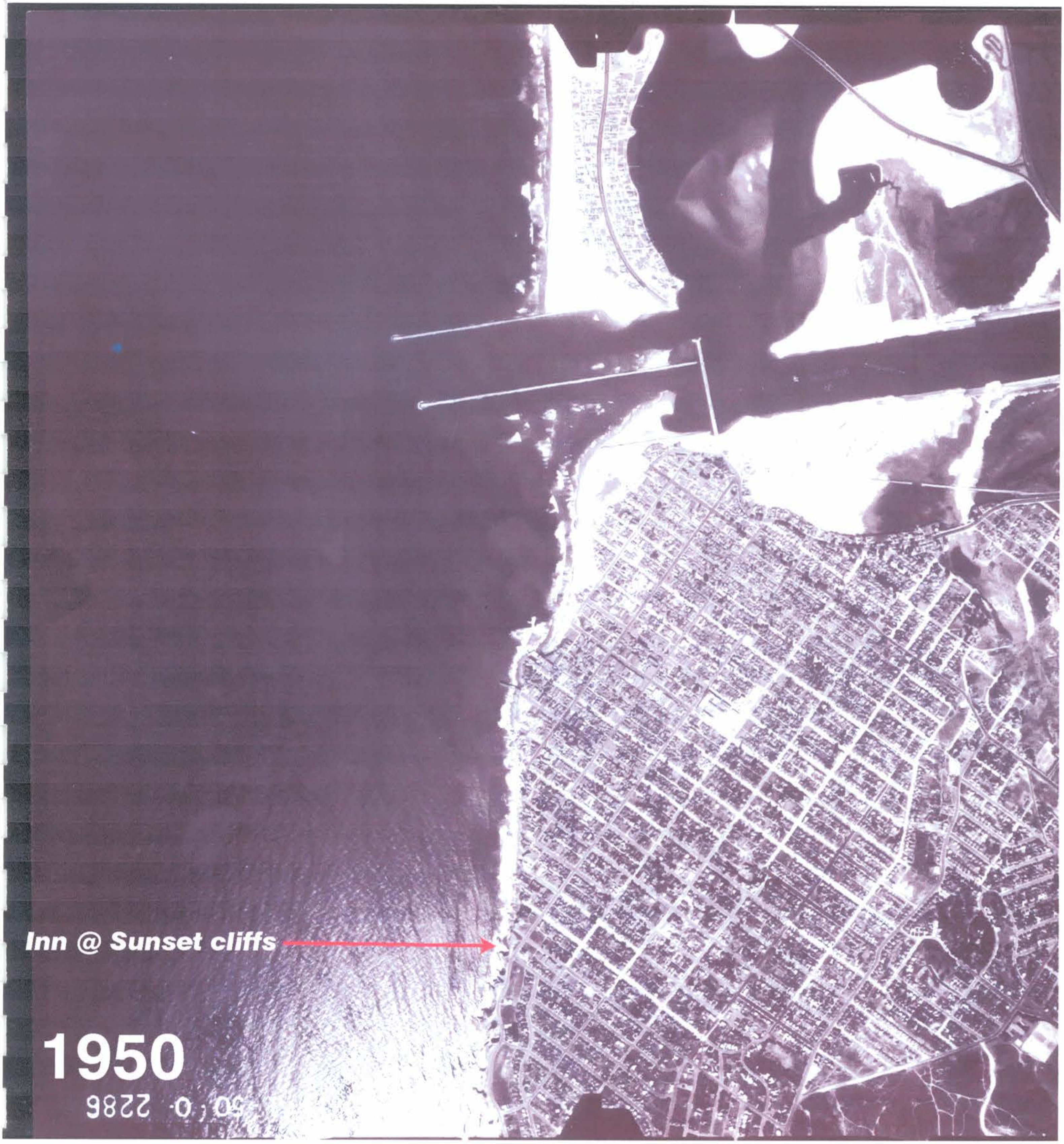
January 2004

03-8530-FF

APPENDIX A

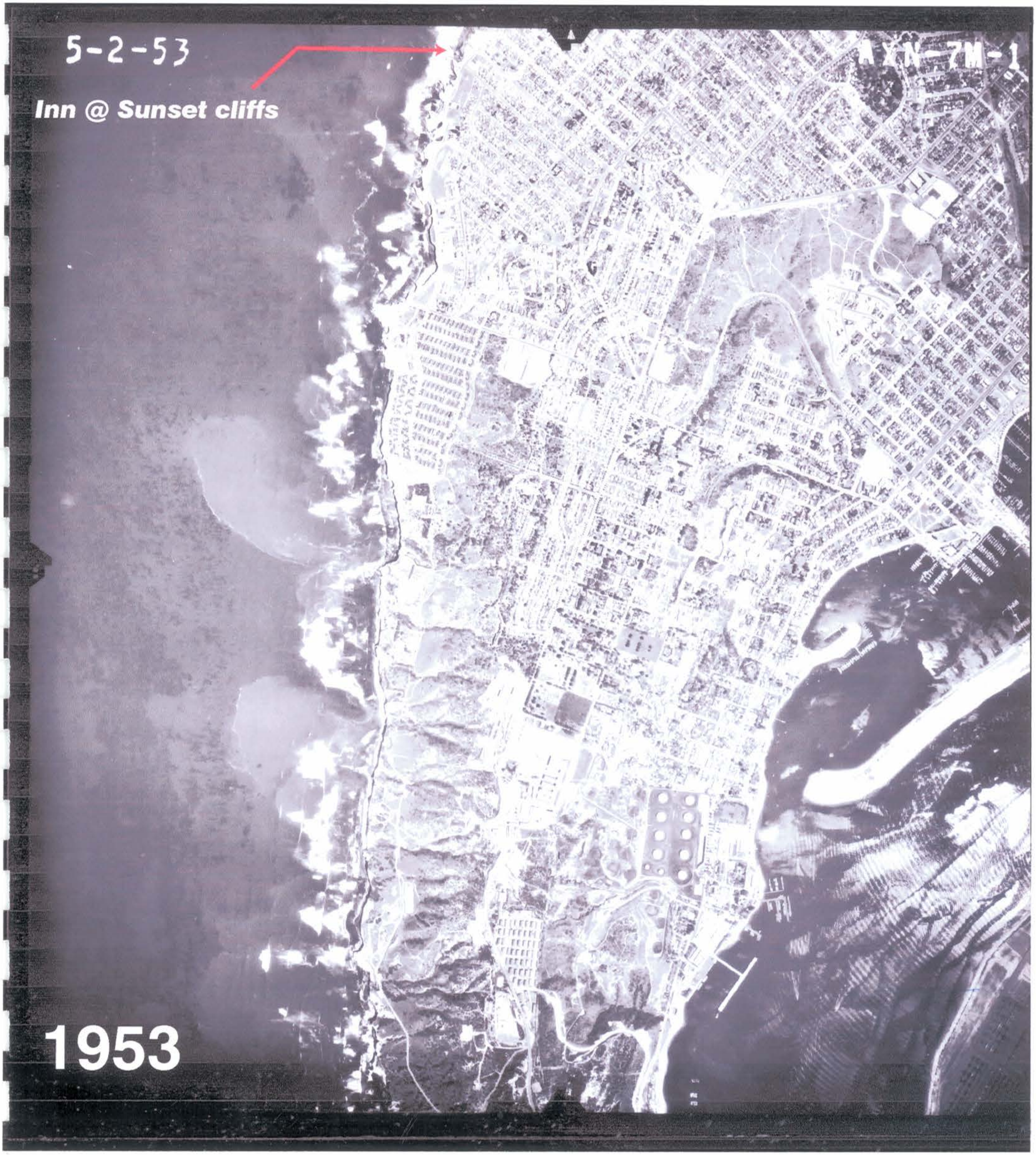

AXN-1-22
Inn @ Sunset cliffs
1939

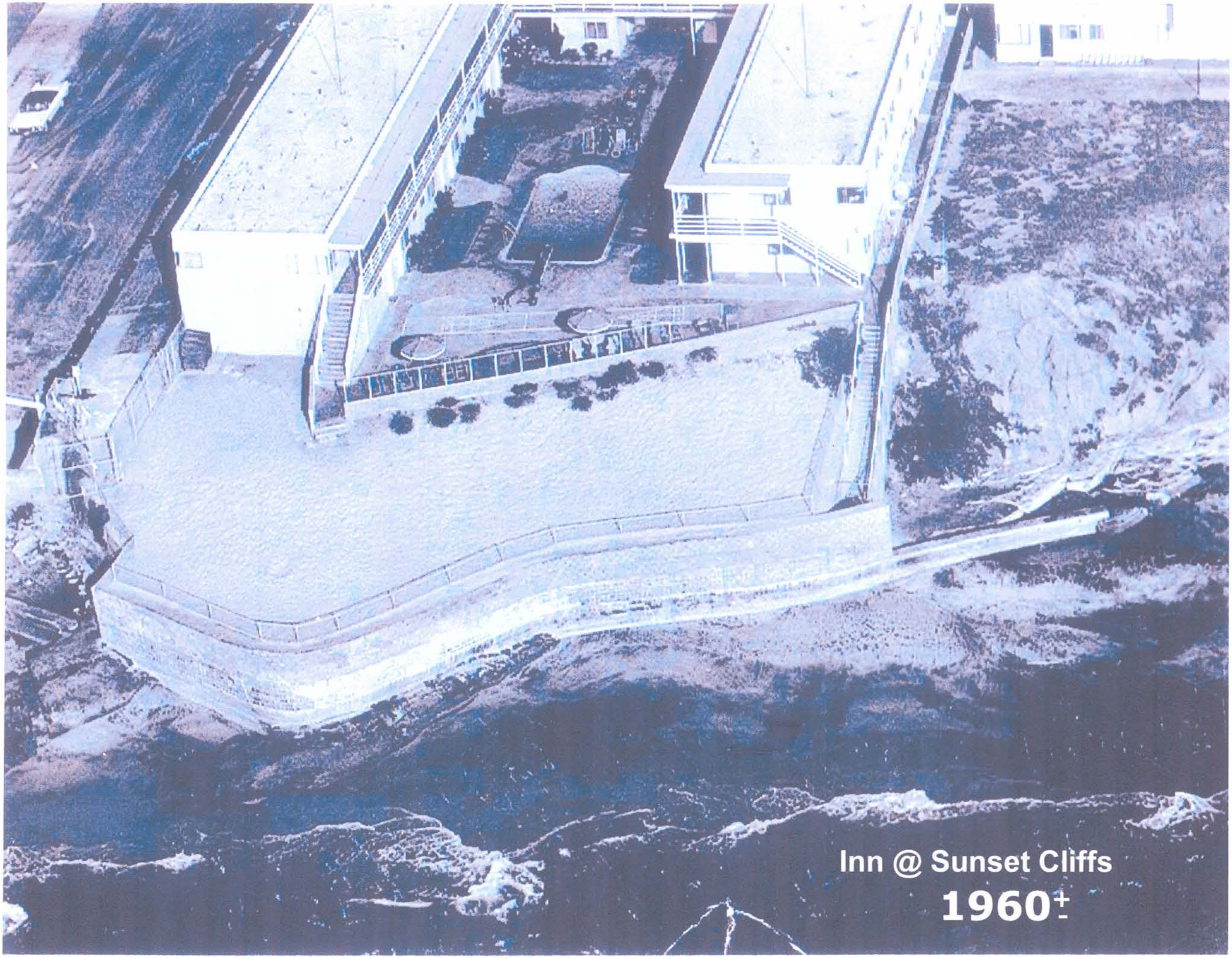

AXN-1-21
Inn @ Sunset cliffs
1939


Inn @ Sunset cliffs
1950


Inn @ Sunset cliffs
1950


5-2-53
N-7M-1
Inn @ Sunset cliffs
1953


5-2-53
Inn @ Sunset cliffs
AXN-7M-1
1953


Inn @ Sunset Cliffs
1960±

Inn @ Sunset Cliffs

San Diego Historical Society
Eberhardt Collection
1972

APPENDIX B

PHOTO LOG

1. Point Loma Formation and rock infill observed below lower deck.

2. Shallow fill soils over Bay Point Formation in Trench T-5.

3. Concrete debris in fill soils, east end of Trench T-3.

4. Bioturbation in Bay Point Formation in Trench T-5.

5 & 6 Sublinear iron/manganese accumulations in Bay Point Formation in Trench T-5.

1

2

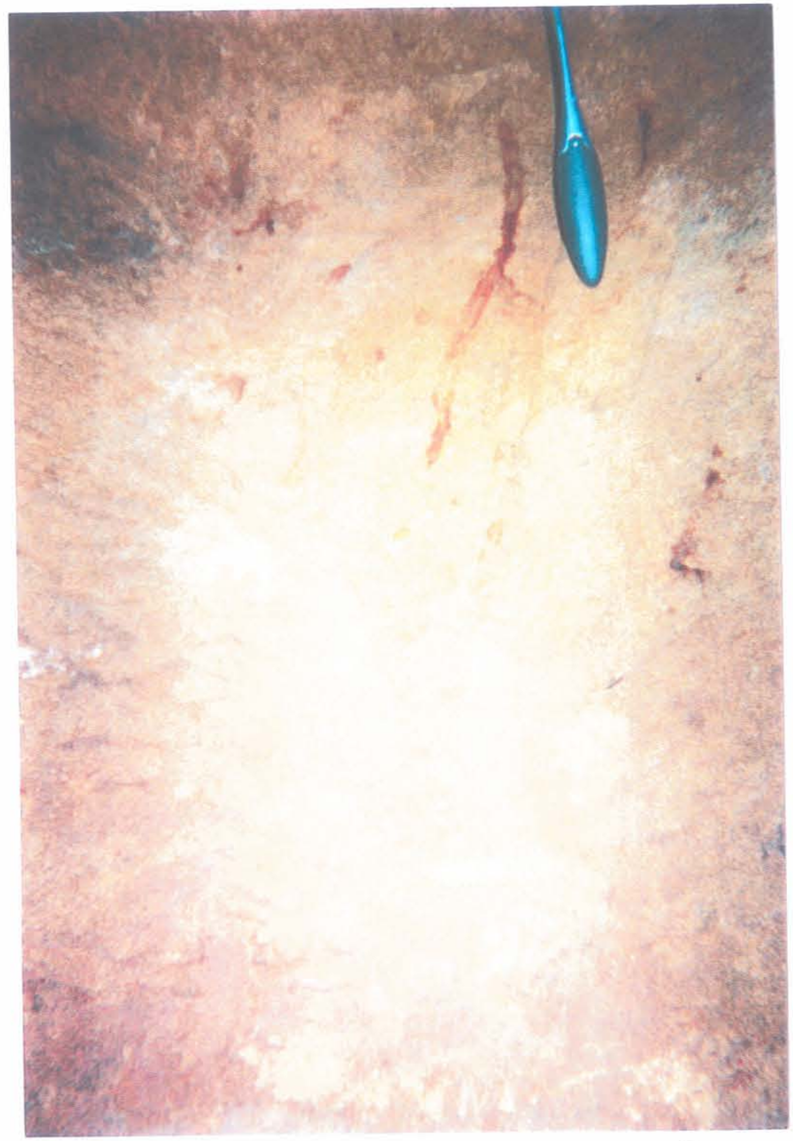
Job No. 03-8530

3

4

Job No. 03-8530

5



6

APPENDIX C
UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION

Coarse-grained (More than half of material is larger than a No. 200 sieve)

GRAVELS, CLEAN GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size, but smaller than 3")	GW	Well-graded gravels, gravel and sand mixtures, little or no fines.
	GP	Poorly graded gravels, gravel and sand mixtures, little or no fines.
GRAVELS WITH FINES (Appreciable amount)	GC	Clay gravels, poorly graded gravel-sand-silt mixtures
SANDS, CLEAN SANDS (More than half of coarse fraction is smaller than a No. 4 sieve)	SW	Well-graded sand, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines.
SANDS WITH FINES (Appreciable amount)	SM	Silty sands, poorly graded sand and silty mixtures.
	SC	Clayey sands, poorly graded sand and clay mixtures.

FINE-GRAINED (More than half of material is smaller than a No. 200 sieve)

SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour, sandy silt and clayey-silt sand mixtures with a slight plasticity.
Liquid Limit Less than 50	CL	Inorganic clays of low to medium plasticity, gravelly clays, silty clays, clean clays.
	OL	Organic silts and organic silty clays of low plasticity.
	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
Liquid Limit Greater than 50	CH	Inorganic clays of high plasticity, fat clays.
	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

APPENDIX D
GENERAL EARTHWORK SPECIFICATIONS

General

The objective of these specifications is to properly establish procedures for the clearing and preparation of the existing natural ground or properly compacted fill to receive new fill; for the selection of the fill material; and for the fill compaction and testing methods to be used.

Scope of Work

The earthwork includes all the activities and resources provided by the contractor to construct in a good workmanlike manner all the grades of the filled areas shown in the plans. The major items of work covered in this section include all clearing and grubbing, removing and disposing of materials, preparing areas to be filled, compacting of fill, compacting of backfills, subdrain installations, and all other work necessary to complete the grading of the filled areas.

Site Visit and Site Investigation

1. The contractor shall visit the site and carefully study it, and make all inspections necessary in order to determine the full extent of the work required to complete all grading in conformance with the drawings and specifications. The contractor shall satisfy himself as to the nature, location, and extent of the work conditions, the conformation and condition of the existing ground surface; and the type of equipment, labor, and facilities needed prior to and during prosecution of the work. The contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications, must be brought to the engineer's attention in order to clarify the exact nature of the work to be performed.

2. A soils investigation report has been prepared for this project by GEI. It is available for review and should be used as a reference to the surface and subsurface soil and bedrock conditions on this project. Any recommendations made in the report of the soil investigation or subsequent reports shall become an addendum to these specifications.

Authority of the Soils Engineer and Engineering Geologist

The soils engineer shall be the owner's representative to observe and test the construction of fills. Excavation and the placing of fill shall be under the observation of the soils engineer and his/her representative, and he/she shall give a written opinion regarding conformance with the specifications upon completion of grading. The soils engineer shall have the authority to cause the removal and replacement of porous topsoils, uncompacted or improperly compacted fills, disturbed bedrock materials, and soft alluvium, and shall have the authority to approve or reject materials proposed for use in the compacted fill areas.

The soils engineer shall have, in conjunction with the engineering geologist, the authority to approve the preparation of natural ground and toe-of-fill benches to receive fill material. The engineering geologist shall have the authority to evaluate the stability of the existing or proposed slopes, and to evaluate the necessity of remedial measures. If any unstable condition is being created by cutting or filling, the engineering geologist and/or soils engineer shall advise the contractor and owner immediately, and prohibit grading in the affected area until such time as corrective measures are taken.

The owner shall decide all questions regarding: (1) the interpretation of the drawings and specifications, (2) the acceptable fulfillment of the contract on the part of the contractor, and (3) the matter of compensation.

Clearing and Grubbing

1. Clearing and grubbing shall consist of the removal from all areas to be graded of all surface trash, abandoned improvements, paving, culverts, pipe, and vegetation (including -- but not limited to -- heavy weed growth, trees, stumps, logs and roots larger than 1-inch in diameter).

2. All organic and inorganic materials resulting from the clearing and grubbing operations shall be collected, piled, and disposed of by the contractor to give the cleared areas a neat and finished appearance. Burning of combustible materials on-site shall not be permitted unless allowed by local regulations, and at such times and in such a manner to prevent the fire from spreading to areas adjoining the property or cleared area.

3. It is understood that minor amounts of organic materials may remain in the fill soils due to the near impossibility of complete removal. The amount remaining, however, must be considered negligible, and in no case can be allowed to occur in concentrations or total quantities sufficient to contribute to settlement upon decomposition.

Preparation of Areas to be Filled

1. After clearing and grubbing, all uncompacted or improperly compacted fills, soft or loose soils, or unsuitable materials, shall be removed to expose competent natural ground, undisturbed bedrock, or properly compacted fill as indicated in the soils investigation report or by our field representative. Where the unsuitable materials are exposed in final graded areas, they shall be removed and replaced as compacted fill.

2. The ground surface exposed after removal of unsuitable soils shall be scarified to a depth of at least 6 inches, brought to the specified moisture content, and then the scarified ground compacted to at least the specified density. Where undisturbed bedrock is exposed at the surface, scarification and recompaction shall not be required.

3. All areas to receive compacted fill, including all removal areas and toe-of-fill benches, shall be observed and approved by the soils engineer and/or engineering geologist prior to placing compacted fill.

4. Where fills are made on hillsides or exposed slope areas with gradients greater than 20 percent, horizontal benches shall be cut into firm, undisturbed, natural ground in order to provide both lateral and vertical stability. This is to provide a horizontal base so that each layer is placed and compacted on a horizontal plane. The initial bench at the toe of the fill shall be at least 10 feet in width on firm, undisturbed, natural ground at the elevation of the toe stake placed at the bottom of the design slope. The engineer shall determine the width and frequency of all succeeding benches, which will vary with the soil conditions and the steepness of the slope. Ground slopes flatter than 20 percent (5.0:1.0) shall be benched when considered necessary by the soils engineer.

Fill and Backfill Material

Unless otherwise specified, the on-site material obtained from the project excavations may be used as fill or backfill, provided that all organic material, rubbish, debris, and other objectionable material contained therein is first removed. In the event that expansive materials are encountered during foundation excavations within 3 feet of finished grade and they have not been properly processed, they shall be entirely removed or thoroughly mixed with good, granular material before incorporating them in fills. No footing shall be allowed to bear on soils which, in the opinion of the soils engineer, are detrimentally expansive -- unless designed for this clayey condition.

However, rocks, boulders, broken Portland cement concrete, and bituminous-type pavement obtained from the project excavations may be permitted in the backfill or fill with the following limitations:

1. The maximum dimension of any piece used in the top 10 feet shall be no larger than 6 inches.

2 Clods or hard lumps of earth of 6 inches in greatest dimension shall be broken up before compacting the material in fill.

3. If the fill material originating from the project excavation contains large rocks, boulders, or hard lumps that cannot be broken readily, pieces ranging from 6 inches in diameter to 2 feet in maximum dimension may be used in fills below final subgrade if all pieces are placed in such a manner (such as windrows) as to eliminate nesting or voids between them. No rocks over 4 feet will be allowed in the fill.

4. Pieces larger than 6 inches shall not be placed within 12 inches of any structure.

5. Pieces larger than 3 inches shall not be placed within 12 inches of the subgrade for paving.

6. Rockfills containing less than 40 percent of soil passing 3/4-inch sieve may be permitted in designated areas. Specific recommendations shall be made by the soils engineer and be subject to approval by the city engineer.

7. Continuous observation by the soils engineer is required during rock placement.

8. Special and/or additional recommendations may be provided in writing by the soils engineer to modify, clarify, or amplify these specifications.

9. During grading operations, soil types other than those analyzed in the soil investigation report may be encountered by the contractor. The soils engineer shall be consulted to evaluate the suitability of these soils as fill materials.

<u>Placing and Compacting Fill Material</u>

1. After preparing the areas to be filled, the approved fill material shall be placed in approximately horizontal layers, with lift thickness compatible to the material being placed and the type of equipment being used. Unless otherwise approved by the soils engineer, each layer spread for compaction shall not exceed 8 inches of loose thickness. Adequate drainage of the fill shall be provided at all times during the construction period.

2. When the moisture content of the fill material is below that specified by the engineer, water shall be added to it until the moisture content is as specified.

3. When the moisture content of the fill material is above that specified by the engineer, resulting in inadequate compaction or unstable fill, the fill material shall be aerated by blading and scarifying or other satisfactory methods until the moisture content is as specified.

4. After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than the density set forth in the specifications. Compaction shall be accomplished with sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other approved types of acceptable compaction equipment. Equipment shall be of such design that it will be able to compact the fill to the specified relative compaction. Compaction shall cover the entire fill area, and the equipment shall make sufficient trips to ensure that the desired density has been obtained throughout the entire fill. At locations where it would be impractical due to inaccessibility of rolling compacting equipment, fill layers shall be compacted to the specified requirements by hand-directed compaction equipment.

5. When soil types or combination of soil types are encountered which tend to develop densely packed surfaces as a result of spreading or compacting operations, the surface of each layer of fill shall be sufficiently roughened after compaction to ensure bond to the succeeding layer.

6. Unless otherwise specified, fill slopes shall not be steeper than 2.0 horizontal to 1.0 vertical. In general, fill slopes shall be finished in conformance with the lines and grades shown on the plans. The surface of fill slopes shall be overfilled to a distance from finished slopes such that it will allow compaction equipment to operate freely within the zone of the finished slope, and then cut back to the finished grade to expose the compacted core. Alternate compaction procedures include the backrolling of slopes with sheepsfoot rollers in increments of 3 to 5 feet in elevation gain. Alternate methods may be used by the contractor, but they shall be evaluated for approval by the soils engineer.

7. Unless otherwise specified, all allowed expansive fill material shall be compacted to a moisture content of approximately 2 to 4 percent above the optimum moisture content. Nonexpansive fill shall be compacted at near-optimum moisture content. All fill shall be compacted, unless otherwise specified, to a relative compaction not less than 95 percent for fill in the upper 12 inches of subgrades under areas to be paved with asphalt concrete or Portland concrete, and not less than 90 percent for other fill. The relative compaction is the ratio of the dry unit weight of the compacted fill to the laboratory maximum dry unit weight of a sample of the same soil, obtained in accordance with A.S.T.M. D-1557 test method.

8. The observation and periodic testing by the soils engineer are intended to provide the contractor with an ongoing measure of the quality of the fill compaction operation. It is the responsibility of the grading contractor to utilize this information to establish the degrees of compactive effort required on the project. More importantly, it is the responsibility of the grading contractor to ensure that proper compactive effort is applied at all times during the grading operation, including during the absence of soils engineering representatives.

__Trench Backfill__

1. Trench excavations which extend under graded lots, paved areas, areas under the influence of structural loading, in slopes or close to slope areas, shall be backfilled under the observations and testing of the soils engineer. All trenches not falling within the aforementioned locations shall be backfilled in accordance with the City or County regulating agency specifications.

2. Unless otherwise specified, the minimum degree of compaction shall be 90 percent of the laboratory maximum dry density.

3. Any soft, spongy, unstable, or other similar material encountered in the trench excavation upon which the bedding material or pipe is to be placed, shall be removed to a depth recommended by the soils engineer and replaced with bedding materials suitably densified.

   Bedding material shall first be placed so that the pipe is supported for the full length of the barrel with full bearing on the bottom segment. After the needed testing of the pipe is accomplished, the bedding shall be completed to at least 1 foot on top of the pipe. The bedding shall be properly densified before backfill is placed. Bedding shall consist of granular material with a sand equivalent not less than 30, or other material approved by the engineer.

4. No rocks greater than 6 inches in diameter will be allowed in the backfill placed between 1 foot above the pipe and 1 foot below finished subgrade. Rocks greater than 2.5 inches in any dimension will not be allowed in the backfill placed within 1 foot of pavement subgrade.

5. Material for mechanically compacted backfill shall be placed in lifts of horizontal layers and properly moistened prior to compaction. In addition, the layers shall have a thickness compatible with the material being placed and the type of equipment being used. Each layer shall be evenly spread, moistened or dried, and then tamped or rolled until the specified relative compaction has been attained.

6. Backfill shall be mechanically compacted by means of tamping rollers, sheepsfoot rollers, pneumatic tire rollers, vibratory rollers, or other mechanical tampers. Impact-type pavement breakers (stompers) will not be permitted over clay, asbestos cement, plastic, cast iron, or nonreinforced concrete pipe. Permission to use specific compaction equipment shall not be construed as guaranteeing or implying that the use of such equipment will not result in damage to adjacent ground, existing improvements, or improvements installed under the contract. The contractor shall make his/her own determination in this regard.

7. Jetting shall not be permitted as a compaction method unless the soils engineer allows it in writing.

8. Clean granular material shall not be used as backfill or bedding in trenches located in slope areas or within a distance of 10 feet of the top of slopes unless provisions are made for a drainage system to mitigate the potential buildup of seepage forces into the slope mass.

<u>Observations and Testing</u>

1. The soils engineers or their representatives shall sufficiently observe and test the grading operations so that they can state their opinion as to whether or not the fill was constructed in accordance with the specifications.

2. The soils engineers or their representatives shall take sufficient density tests during the placement of compacted fill. The contractor should assist the soils engineer and/or his/her representative by digging test pits for removal determinations and/or for testing compacted fill. In addition, the contractor should cooperate with the soils engineer by removing or shutting down equipment from the area being tested.

3. Fill shall be tested for compliance with the recommended relative compaction and moisture conditions. Field density testing should be performed by using approved methods by A.S.T.M., such as A.S.T.M. D1556, D2922, and/or D2937. Tests to evaluate density of compacted fill should be provided on the basis of not less than one test for each 2-foot vertical lift of the fill, but not less than one test for each 1,000 cubic yards of fill placed. Actual test intervals may vary as field conditions dictate. In fill slopes, approximately half of the tests shall be made at the fill slope, except that not more than one test needs to be made for each 50 horizontal feet of slope in each 2-foot vertical lift. Actual test intervals may vary as field conditions dictate.

4. Fill found not to be in conformance with the grading recommendations should be removed or otherwise handled as recommended by the soils engineer.

<u>Site Protection</u>

It shall be the grading contractor's obligation to take all measures deemed necessary during grading to maintain adequate safety measures and working conditions, and to provide erosion-control devices for the protection of excavated areas, slope areas, finished work on the site and adjoining properties, from storm damage and flood hazard originating on the project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with the job specifications, all berms and benches have been properly constructed, and all associated drainage devices have been installed and meet the requirements of the specifications.

All observations, testing services, and approvals given by the soils engineer and/or geologist shall not relieve the contractor of his/her responsibilities of performing the work in accordance with these specifications.

After grading is completed and the soils engineer has finished his/her observations and/or testing of the work, no further excavation or filling shall be done except under his/her observations.

Adverse Weather Conditions

1. Precautions shall be taken by the contractor during the performance of site clearing, excavations, and grading to protect the worksite from flooding, ponding, or inundation by poor or improper surface drainage. Temporary provisions shall be made during the rainy season to adequately direct surface drainage away from and off the worksite. Where low areas cannot be avoided, pumps should be kept on hand to continually remove water during periods of rainfall.

2. During periods of rainfall, plastic sheeting shall be kept reasonably accessible to prevent unprotected slopes from becoming saturated. Where necessary during periods of rainfall, the contractor shall install checkdams, desilting basins, rip-rap, sandbags, or other devices or methods necessary to control erosion and provide safe conditions.

3. During periods of rainfall, the soils engineer should be kept informed by the contractor as to the nature of remedial or preventative work being performed (e.g. pumping, placement of sandbags or plastic sheeting, other labor, dozing, etc.).

4. Following periods of rainfall, the contractor shall contact the soils engineer and arrange a walk-over of the site in order to visually assess rain-related damage. The soils engineer may also recommend excavations and testing in order to aid in his/her assessments. At the request of the soils engineer, the contractor shall make excavations in order to evaluate the extent of rain-related damage.

5. Rain-related damage shall be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress, and other adverse conditions identified by the soils engineer. Soil adversely affected shall be classified as Unsuitable Materials, and shall be subject to overexcavation and replacement with compacted fill or other remedial grading, as recommended by the soils engineer.

6. Relatively level areas, where saturated soils and/or erosion gullies exist to depths of greater than 1.0 foot, shall be overexcavated to unaffected, competent material. Where less than 1.0 foot in depth, unsuitable materials may be processed in place to achieve near-optimum moisture conditions, then thoroughly recompacted in accordance with the applicable specifications. If the desired results are not achieved, the affected materials shall be over-excavated, then replaced in accordance with the applicable specifications.

7. In slope areas, where saturated soils and/or erosion gullies exist to depths of greater than 1.0 foot, they shall be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where affected materials exist to depths of 1.0 foot or less below proposed finished grade, remedial grading by moisture-conditioning in place, followed by thorough recompaction in accordance with the applicable grading guidelines herein presented may be attempted. If materials shall be overexcavated and replaced as compacted fill, it shall be done in accordance with the slope-repair recommendations herein. As field conditions dictate, other slope-repair procedures may be recommended by the soils engineer.

APPENDIX B

SUMMARY SLOPE STABILITY ANALYSES

TerraCosta Consulting Group

Inn at Sunset Cliffs PN 2317
Overtopping Analyses w/TOW = 27.7 ft December 20, 2020

d_s	T	d_s/gT^2	H_b/d_s	H_b	h*	R_c	h^*R_c/H_b	q/-----	q - cfs/ft	25% q, cfs/ft	q - gpm/ft	- liters/s per	q, leters/sec per m	
10.25	6	0.0088	0.86	8.8	0.065	22.45	0.165	0.04	0.031	0.008	14	2.9	0.7	
10.25	10	0.0032	0.91	9.3	0.022	22.45	0.053	1.36	0.122	0.031	55	11.4	2.8	1982-83 Design Storms
10.25	14	0.0016	0.93	9.5	0.011	22.45	0.026	12.53	0.281	0.070	126	26.1	6.5	
13.75	6	0.0119	0.85	11.7	0.088	18.95	0.142	0.06	0.141	0.035	63	13.1	3.3	
13.75	10	0.0043	0.91	12.5	0.029	18.95	0.045	2.30	0.578	0.145	260	53.7	13.4	3.5 ft SLR (17% probability of exceedance)
13.75	14	0.0022	0.92	12.7	0.015	18.95	0.022	19.81	1.269	0.317	570	117.9	29.5	
17.25	6	0.0149	0.85	14.7	0.110	15.45	0.116	0.12	0.588	0.147	264	54.6	13.7	
17.25	10	0.0054	0.90	15.5	0.037	15.45	0.037	4.04	2.299	0.575	1,032	213.6	53.4	7 ft SLR (0.5% probability of exceedance)
17.25	14	0.0027	0.91	15.7	0.019	15.45	0.019	34.87	5.049	1.262	2,266	469.2	117.3	Use 50 for design

H_b/d_s ---> fig. 7-4 SPM assuming 60:1 offshore slope
h* = $(d_s/H_b)(2*3.14159*d_s/g*T^2)$ ---> eq. 16.1 Handbook of Coastal and Ocean Engineering by Kim (2010)
R_c = freeboard. Given design SWL = 5.25 ft. For a 27.7 ft top of wall, Rc = 22.45 ft. With 3.5 ft of SLR, Rc = 18.95 ft……………
q/------- ---> fig. 16.10 Handbook of Coastal and Ocean Engineering by Kim (2010)
q - cfs/ft --> eq. 16.4 Handbook of Coastal and Ocean Engineering by Kim (2010)
25% q refers to the benefit of the wave deflector
0.5% probability scenerio refers to the propability of sea level in the year 2100 being greater than 7.0 ft above the 2000 baseline value

Inn at Sunset Cliffs PN 2317
Runup Analyses w/TOW = 27.7 ft December 20, 2020

d_s	T	d_s/gT^2	H_b/d_s	H_b	H_b/gT^2	H_b/H_o'	H_o'	H_o'/gT^2	d_s/H_o'	R/H_o'	R	fig 7-13	R'	R*	Goda R	R/Ho'	Goda R*
2019 design criteria																	
10.53	6	0.0091	0.86	9.1	0.00781	1.18	7.67	0.00662	1.37	2.0	15.3	1.210	18.6	24.1	13.6	1.77	19.1
10.53	10	0.0033	0.91	9.6	0.00298	1.61	5.95	0.00185	1.77	2.1	12.5	1.210	15.1	20.7	14.4	2.42	19.9
10.53	14	0.0017	0.93	9.8	0.00155	2.04	4.80	0.00076	2.19	2.5	12.0	1.210	14.5	20.1	14.7	3.06	20.2
10.53	18	0.0010	0.95	10.0	0.00096	2.48	4.03	0.00039	2.61	3.0	12.1	1.210	14.6	20.2	15.0	3.72	20.5
1.8 ft SLR design criteria																	
12.13	6	0.0105	0.85	10.3	0.00889	1.14	9.04	0.00780	1.34	1.8	16.3	1.210	19.7	26.8	15.5	1.71	22.6
12.13	10	0.0038	0.91	11.0	0.00343	1.52	7.26	0.00226	1.67	2.5	18.2	1.210	22.0	29.1	16.6	2.28	23.7
12.13	14	0.0019	0.93	11.3	0.00179	1.94	5.81	0.00092	2.09	2.8	16.3	1.210	19.7	26.8	16.9	2.91	24.1
12.13	18	0.0012	0.93	11.3	0.00108	2.40	4.70	0.00045	2.58	3.0	14.1	1.210	17.1	24.2	16.9	3.60	24.1
3.5 ft SLR design criteria																	
13.83	6	0.0119	0.87	12.0	0.01038	1.10	10.94	0.00944	1.26	1.5	16.4	1.210	19.9	28.7	18.0	1.65	26.9
13.83	10	0.0043	0.90	12.4	0.00387	1.46	8.53	0.00265	1.62	2.5	21.3	1.210	25.8	34.6	18.7	2.19	27.5
13.83	14	0.0022	0.91	12.6	0.00199	1.84	6.84	0.00108	2.02	2.6	17.8	1.210	21.5	30.3	18.9	2.76	27.7
13.83	18	0.0013	0.93	12.9	0.00123	2.22	5.79	0.00056	2.39	3.0	17.4	1.210	21.0	29.9	19.3	3.33	28.1
4.5 ft SLR design criteria																	
14.83	6	0.0128	0.84	12.5	0.01075	1.09	11.43	0.00986	1.30	1.5	17.1	1.210	20.7	30.6	18.7	1.64	28.5
14.83	10	0.0046	0.90	13.3	0.00415	1.42	9.40	0.00292	1.58	2.4	22.6	1.210	27.3	37.1	20.0	2.13	29.9
14.83	14	0.0023	0.91	13.5	0.00214	1.80	7.50	0.00119	1.98	2.5	18.7	1.210	22.7	32.5	20.2	2.70	30.1
14.83	18	0.0014	0.93	13.8	0.00132	2.16	6.39	0.00061	2.32	3.0	19.2	1.210	23.2	33.0	20.7	3.24	30.5
eq.																	
17.33	6	0.0149	0.84	14.6	0.01256	1.05	13.86	0.01196	1.25	1.5	20.8	1.210	25.2	37.5	21.8	1.58	34.2
17.33	10	0.0054	0.90	15.6	0.00484	1.37	11.38	0.00354	1.52	2.4	27.3	1.210	33.1	45.4	23.4	2.06	35.7
17.33	14	0.0027	0.91	15.8	0.00250	1.70	9.28	0.00147	1.87	2.5	23.2	1.210	28.1	40.4	23.7	2.55	36.0
17.33	18	0.0017	0.92	15.9	0.00153	2.03	7.85	0.00075	2.21	3.0	23.6	1.210	28.5	40.8	23.9	3.05	36.2

H_b/d_s ---> fig. 7-4 SPM assuming 60:1 offshore slope

H_b/H_o'---> fig. 7-5 SPM

R/H_o'---> fig. 7-14 SPM

fig 7-13 from the SPM corrects for scale effects

R* = R + SWL ---> this is the computed height of runup using the SPM

Goda R is from 18.1 of the Handbook of Coastal and Ocean Engineering by Kim (2010)

R/Ho' in column Q calculates for comparison of SPM fig 7-14