Surfrider Foundation San Diego County

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May 19, 2014 *Delivered via Email and U.S. Mail*

Deputy Director Sherilyn Sarb
Coastal Planner Toni Ross
California Coastal Commission
7575 Metropolitan Drive, Ste. 103
San Diego, CA 92108-4421

RE: Goetz/Sylver Seawall
Appeal No. A-6-CII-10-043

Dear Deputy Director Sarb and Planner Ross,

Thank you for providing the report prepared by Geosoils Incorporated (GSI) dated January 27, 2014. Surfrider notes that there has been a dramatic shift on the theory presented by the Mr. Goetz and Sylver regarding the necessity of the seawall. The City of Carlsbad issued an "emergency" permit for a seawall based on a perceived threat to people on the beach, despite the fact that the applicant was seeking an emergency seawall permit five months after a bluff collapse. Now, the applicant is attempting to justify the seawall based on a threat to the principle structures set 43 feet away from the edge of the bluff.

When utilizing the Bishop's method to find the unconstrained Factor of Safety, it appears the principal structures are not threatened. Instead the applicants propose a new method to rationalize development, the "progressive failure" approach, which appears to not meet the standard requirement of a principal structure being imminently threatened in order to consider approval of a seawall. Surfrider further points out that the structures in question should not be entitled to protection because they were permitted in 2002 under the presumption that they would be safe from erosion for 75 years. The San Diego Chapter of the Surfrider Foundation disputes that the modeling performed by Geosoils accurately predicts future erosion rates in light of the previous six geotechnical reports on the property submitted prior to the construction of the residences in 2004.[1]

[1] For the convenience of Commission and Staff, all citations in the electronic (PDF) version contain links to the material cited. Simply click on the link and the cited background material should appear. Left clicking on the cited text return you to this letter.

EXHIBIT NO. 11
APPLICATION NO.
A-6-CII-10-043
Correspondence from the Applicant
California Coastal Commission

1. The Numerous Geotechnical Reports Submitted Prior to Construction of the Residential Structures Warned of the Episodic Nature of Erosion, but Demonstrate a 40 Foot Setback is Sufficient for the Safety of the Structures

Surfrider is concerned with the use of "purchased" geoscience. Throughout the years, we have found repeated examples where geologists claim that a bluff will be stable for 75-100 years without the need of a seawall in order to the justify a minimum setback. Then, shortly after construction, the landowner applies for a seawall claiming the structure is in imminent danger from erosion. For example, in Pismo Beach (aka Shell Beach) a coastal home was permitted to be located only 25 feet away from the bluff edge, and then the very next year the owner applied for a seawall. (Coastal Commission Appeal No. A-3-PSB-02-016.) Certain geologists appear to be ready and willing to justify any erosion rate based on whether the applicant wishes to build a structure or protect a structure. The numerous geotechnical reports contained in the history of the parcels at issue here demonstrates the influence of purchased science.

In February of 1984, in response to concerns about erosion, the landowner submitted a geotechnical report recommending a seawall to protect the empty lots. (Exhibit A (Converse Rep.).)[2] The geotechnical report noted that between August 7 - 9, 1983, 23 - 27 feet of bluff collapsed. (Converse Rep. at p. 5.) Such report was filed with the City of Carlsbad undoubtedly in the hopes of obtaining a seawall despite the lack of existing structures.[3]

In September 1984, the same geologist submitted an updated geotechnical report stating that the "fill" in the cove area had eroded another 6-7 feet. (Exhibit B, Converse Rep. 2 at p.4-5.) The report also noted the lack of "cementation" in the cove area (Converse Rep. 2 at p. 5.)[4] This second report again recommended a seawall to protect the empty lots.

In 1991, another bluff retreat study was prepared and submitted to the City of Carlsbad. (Exhibit C, ICG report dated March 28, 1991.) Five soil borings were analyzed. (ICG report at p. 3.) The report noted "a variety of old fills" scattered across the property. (ICG Report at p.6.) Despite noting the episodic erosion, including the 1983 storm event, and the variety of "old fills", gunite facing and other past alterations, the report estimated an annual retreat rate of just 0.4 feet a year. (ICG Rep. at pp. 10-11, 15.) The report, which used GSTABL5 for modeling, came to the conclusion that a 45 foot setback was sufficient for 75 years of erosion. (ICG Rep. at 15.) The report states:

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[2] For the soil report, two borings and five trenches were analyzed. (Converse Rep. at Introduction.)

[3] It also appears that "fill material" was imported to the site to fill in the slope and reduce erosion in the "cove area". (Converse Rep. at 7.) A gunite seawall was also installed. (Id.) There does not appear to be permits for these past actions.

[4] Three additional soil samples were obtained and analyzed in connection with the Converse 2 report, dated September 1984. (Converse Rep.2 at Introduction.)

> The loss of 45 feet of bluff, back from the existing rim, would still fall short of the structural setback established through slope stability calculations and the 1.5 factor of safety.

(ICG Rep. at p. 15.)

The ICG report was prepared to justify development of the lots with two residential structures, which explains the optimistic erosion evaluations. (ICG Rep. at 2.)

In March of 1997, yet another geotechnical report was prepared. (Exhibit D, SGC Report dated March 10, 1997.) This report supported an application for development of a single family residence set back "a minimum of 25 feet from the edge of the bluff." (SGC Rep. at p. 3.) The report does discuss the episodic erosion and an Army Corps of Engineers report that estimated 28 to 35 feet of erosion at the site. (SGC Rep. at 9.) Nevertheless, based on an analysis of historic photographs, the geologist estimated the upper bluff edge eroded only 6 feet over 25 years, and therefore predicted a "worst-case" or maximum bluff edge retreat of 18 feet in 75 years. (SCG Rep. at 9.) The report later estimates a worst case scenario of 18 to 24.8 feet in 75 years. (SCG Rep. at 10.) Admittedly, this report was probably analyzing the northern portion of the lot. Nevertheless, the lot was not split until the year 2000.[5]

In 1998, a civil engineer submitted an "updated geotechnical study". (Exhibit E, Randle Report dated March 16, 1998.) This report estimated a bluff erosion rate of 1-2 inches a year over a 75 year period. (Randle Rep. at p. 4.) The report estimated a likely amount of bluff retreat of 6 to 12 feet or less. (Randle Rep. at p. 3.) The report recommended a bluff setback of 25 feet for all foundations and 30 feet for all primary structures. (Randle Rep. at p. 4.) The report also claimed that the bluff in the cove area would match the surrounding bluff, because of the additional setback and beach. (Randle Rep. at p. 3.)

In 2001, a sixth geotechnical review was prepared on behalf of Mr. Geotz, who had apparently purchased one of the lots. (Exhibit F, Owen Engineering Group report dated October 10, 2001.) The OEG report is described as an "update" to the 1998 report by Civil Engineer C. J. Randle. The report recommended a setback 40 feet from the top of the bluff (OEG rep. at p. 1.) The report does not explain its recommendation of a 40 foot setback for the properties. The OEG report was submitted in a packet to the City of Carlsbad with the C. J. Randle Report and excerpts from a Coastal Commission report which clearly noted that "there are long periods when there are no failures followed by one or two events where tens of feet of bluff may retreat." (Ex. F, "Excerpt 2".)

In 2002, the Geotz and the Sylver residences were approved by the Planning Commission of Carlsbad. (Exhibit G & H.) The Planning Commission found, "A geotechnical analysis of the project site was prepared. The analysis concluded that the proposed development will have a useful life of at least 75 years and that the development will have no adverse effects on the

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[5] APN 210-120-30 was split into three lots as APN 210-120-32, 210-120-33, 210-120-34.

stability of the coastal slope." (Exs. G & H at p. 2 ¶ 4.) The residences were completed in 2004.

On December 19, 2008, after three days of heavy rains, the bluff retreated 3-5 feet. According to Geosoils, the bluff collapse deposited an estimated 150 cu yards of bluff material. (Exhibit I.) Mr. Goetz requested an emergency permit which was granted on or about April 16, 2009. Unfortunately, Mr. Goetz did not immediately commence construction and the initial emergency permit expired. (Exhibit J.) A second emergency seawall permit was issued on June 10, 2009, and construction commenced during the summer beach season.[6] Surfrider appealed the follow-up regular CDP to the Coastal Commission.

In a letter report dated March 30th, 2010 Geosoils Inc (GSI), estimated a long-term erosion rate for the property of .16 feet per year. (Ex. K, GSI Rep. 1 at 5.) GSI explained the following method for arriving at such a low erosion rate:

> Retreat rates reported in site-specific geotechnical investigations (circa 1991) on nearby properties to the north range from 0.22 ft/yr to 0.4 ft/yr. These retreat rates were used to establish the bluff top setback requirement of new development at those properties. By contrast, however, the USGS report is a recent comprehensive report that covers the coast of California and specifically this location using survey data from as early as the 1890s through the present time. ***The USGS report shows that this particular section of bluff has not retreated over the last 115 years.*** This stability relative to other seacliffs in North San Diego County is likely due to the fact that this section is at the back of the cove and significantly landward of both the mean high tide line and the high water mark…That being said, the bluff did fail in December 2008 prompting the emergency repairs. Using the USGS bluff retreat rate of 0 feet over the period from 1890 to 2006, along with the approximate 6 feet of retreat from the well documented failure in 2008, ***the actual retreat rate is calculated to be 6 ft over 120 years. This translates to an average annual retreat rate of 0.05 feet.*** That being said, the Coastal Commission has used 0.27 feet per year for other recent projects. Although dissimilar to these projects, to be conservative, we have chosen the midway point between the actual rate (0.05) and the Commission's number (0.27) to come up with 0.16 as the annual retreat rate for this site.

(Exhibit K, at pp. 4-5 (emphasis added).)

Thus, according to GSI (the applicants' current geologist), the actual annual retreat rate at this site is .05 feet per year (1/2 inch). Therefore, according to GSI, the in-lieu sand mitigation fee should only be $2,469. (Exhibit K, at p. 6.)

[6] Ironically, the professional engineer who prepared the building plans for the seawall was C. J. Randle, P.E. who opined in 1998 that the structures could have a 25 foot setback and maintain an appropriate factor of safety for 75 years "conservatively". (Exhibit E, Randle Rep. at p. 3 & 4.)

On or about July 2012, Geosoils submitted a new geotechnical report to the Coastal Commission. Despite the previous six geology opinions, and its own low estimate of annual erosion rate, Geosoils now contended that both structures would have a factor of safety below 1.2 and a seismic factor of safety of .7 without the seawall, despite having a setback of more than 40 feet. (Exhibit L, GSI Rep.2 at p. 23.) GSI found that removal of the seawall would place the structures in imminent danger of collapse. (GSI Rep. at 28.)

> The unique geologic and geomorphic factors (i.e. regional and onsite faulting, groundwater, bluff instability, bluff geometry, susceptibility to wave attack and marine erosion, etc.) have significantly contributed to increase erosion on this section of the coastline, and this erosion would only continue, and most likely accelerate, upon remove of the seawall.

(GSI Rep.2 at 28.)

In a letter dated January 8, 2013, Coastal Commission Staff took issue with the methodology of the models used to find that the structures were in imminent danger of collapse. (Exhibit M, CCC corr. at pp. 2-3.) In addition, Coastal Staff found that the soil core borings were questionable:

> [The soil borings] do not appear to be from relatively undisturbed samples taken from the subject site. The cohesion values, in particular, are very low in comparison with similar materials with which the Commission's staff geologist is familiar.

(CCC corr. at p. 2.)

The Coastal Commission letter also pointed out that the report 2012 analysis, did not utilize the unconstrained Factor of Safety to determine the most likely failure planes.

The Applicant spent over a year preparing a response and presented its most recent report on January 24, 2014. (Exhibit N, GSI Rep.3.) Such report still does not appear to analyze the most likely failure plane as requested by the Coastal Commission, but instead determines the modeled FOS 1.0 plane and models progressive failures from such point to derive the FOS of the westward portion of the structures. (GSI Rep.3 at p.2.) Unsurprisingly, the most recent GSI Report claimed that the principal structures are in danger from erosion. (GSI Rep. at p. 3.)

2. The Developer/Owner Goetz Knowingly Took the Risk of Episodic Bluff Erosion

In this case, the seawall applicant, Dean Goetz, was the initial owner of the property when his single family residence was permitted and developed. (See Exhibit F & G.) As such, he had complete control on whether to increase his setback from the bluff edge to provide an extra measure of safety. He cannot claim ignorance of the episodic nature of erosion, especially after

Owen Engineering Group reviewed all the previous engineering studies and submitted excerpts from a Coastal Commission report that clearly explained the episodic nature of erosion of the bluff. (Exihibit F.) He cannot claim ignorance of the fact that the bluff would erode, from time to time, in large chunks

Mr. Sylver also cannot claim ignorance of the geological conditions. A sale of real property in California requires transfer disclosure statements that specifically identify geologic hazards and conditions. (Civ. Code §§ 1102 et. seq., 1102.6.) Ordinarily, all geology reports are submitted to the potential buyer during escrow. The episodic nature of erosion of coastal bluffs in San Diego, especially at this site, was well known and documented. If Mr. Sylver is claiming ignorance, he should be able to demonstrate the lack of disclosure of the geologic conditions by submitting all the documents from escrow for Coastal Commission review. Nevertheless, if Mr. Sylver could demonstrate utter lack of notice of the geologic risks and conditions, he still would not be entitled to a seawall. However, he may be entitled to seek damages against the seller of the property.

3. The Coastal Commission May Rely on Earlier Geology Reports and Erosion Estimates Contained in the Record to Deny the Seawall.

At Mr. Goetz's disposal, were the geology reports from 1984, which identified severe erosion problems and the variety of loosely consolidated artificial fill materials at the site in support of a shoreline armoring project. (Exhibit A, Converse Rep. at p. 8.; Exhibit B, at pp. 4-5.)[7] Later geology reports, also noted the undocumented fill deposits included asphalt and large slabs of concrete to infill gullies. (Exhibit C, ICG Rep. at p. 6.) Nevertheless, Mr. Goetz relied on later reports that emphasized the episodic nature of erosion with "long periods during which little or no erosion occurs, are sporadically interrupted by catastrophic events where tens of feet of beach and bluff may be lost." (Ex. C, ICG Rep. at 7.) Such later reports estimated the average annual rate of erosion between 1-2 inches to .4 feet per year. (ICG Rep. at p.15.)

According to the experts advising Mr. Goetz, a bluff can experience 27 feet of erosion in a single event, but not experience significant erosion over the next 75 years. Being duly informed about the episodic nature of erosion, the loss of 3-5 feet of bluff in 2008 should not have caused any alarm. It was predicted by the six geology reports prepared prior to construction of his home. There is nothing in the record that demonstrates that the previous geology reports are wrong. The bluff has responded as predicted.

Further, there is no reason that the Coastal Commission should assume that GSI's latest report is more accurate than the previous reports. In 2010 GSI stated that except for the more recent collapse, a 2006 USGS report demonstrates little to no erosion historically at the site. (Exhibit K, at pp. 4-5 (emphasis added).) GSI made such assessment in 2010, <u>after</u> the 3-5 feet of bluff collapsed. GSI estimated an average erosion rate of 0.05 ft. per year (1/2 inch). In 2012 and 2014, GSI predicts the structures are now in imminent threat of erosion. There is no reason

[7] The Converse Report was submitted in support of Project No. 83-02299-01.

from the historical record, as confirmed by GSI, that the next 115 years will be different than the previous 115 years.[8]

The Coastal Commission is certainly entitled to rely on earlier geology reports that demonstrate a relatively low annual erosion rate in determining that the structures are not in imminent danger of erosion, and therefore not entitled to a seawall under Public Resources Code section 30235. In addition, the Coastal Commission may rely on the professional experience of Dr. Mark Johnsson and other Coastal Staff who can attest to the fact that 40 feet of erosion in a single year is a very rare event along this portion of the coast. The highest average rate of yearly erosion, as documented by Dr. Gary Griggs, is 9 feet a year at Año Nuevo State Park in Central California. (Griggs, Patch and Savoy, LIVING WITH THE CHANGING CALIFORNIA COAST (2005) at p. 84.) The Coastal Commission should deny the seawall because the structures, located more than 40 feet away from the edge of the bluff, are not in imminent danger from erosion.

4. Another Extreme Erosion Event is Statistically Unlikely in the Near Term.

The GSI report specifically notes that 27 feet of bluff eroded between August 7th and 9th in August of 1983. (Exhibit L, GSI Rep.2 at p. 28.) There are a couple of unique features about such storm event that explains the intense erosion event back in 1983. First, the 1982-83 storm season was during El Niño conditions, and between January and March 1983, the California Coast experienced 8 major storms with waves heights between 16 and 22 feet. (LIVING WITH THE CHANGING CALIFORNIA COAST at p. 25.) This stripped the sand from the beach, leaving cobble and destroyed or severely weakened the gunite seawall at the property. (See also Ex. L, GSI Rep.2 at p. 28.) Then a large southern hemisphere south swell combined with an extreme high tide to cause extremely rapid erosion at the site. (Ex. A, Converse Rep. at 5.) One of the geotechnical reports noted that while the offshore reef dissipates the energy from the northwest swells, the cove is subject to wave attack from the south. (Ex. C, ICG rep. at p. 9.) Thus, the extreme erosion events in this area require large southern hemisphere (long period) swells in conjunction with an extreme high tide event. A similar event was not documented prior to 1983, nor after 1983.

According to the geotechnical reports, the upper bluff edge has apparently experienced relatively little retreat between 1929 and 1989. (Ex. D, SGC Rep. at 9.) Further, the seawall engineer in this case, C.J. Randle, opined in 1998 that the natural setback created by the creation of the cove serves as a buffer from direct wave attack. "Thus, once formed, the top of bluff retreat rate for the cove area becomes similar to that of the adjacent tops of bluff to both sides of the cove." (Ex. E, Randle Rep. at p. 3.) This same sentiment is echoed by GSI in explaining an almost imperceptible long-term annual erosion rate at the site:

> The USGS report shows that this particular section of bluff has not retreated over the last 115 years. This stability relative to other seacliffs in North San Diego

[8] Selective soil sampling may also skew modeling. At least 10 previous soil samples were taken from the site. (Ex. A at B2 and Ex. C at stamp 1067.) The Coastal Commission should compare past samples to determine consistency with GSI's latest soil samples.

> County is likely due to the fact that this section is at the back of cove and significantly landward of both the mean high tide and the high water mark.

(Ex. K, GSI Rep. 1 at pp. 4-5.)

Considering that only one extreme erosion event occurred between 1929 and 2009; that episodic bluff retreat was expected and predicted; and, at least three geotechnical reports support that a 40-45 foot setback is sufficient to keep the structures safe for 75 years, the Coastal Commission should view the most recent GSI reports with extreme skepticism. Installing an emergency seawall after a bluff collapse of 3-5 feet, despite having more than 40 feet of additional setback was an overreaction and appears to have been calculated to install a seawall prior to any actual necessity.

5. The Public Benefits from Additional Erosion at the Cove.

In Solana Beach, 10 miles to the south, all installed seawalls must be designed to be able to be removed in the future. Both GSI and Terracosta[9] have been used as principal engineers on removable seawalls in Solana Beach. It is unlikely that the seawall installed at the site is significantly different that the seawalls installed at Solana Beach.

The GSI report assumes that removing the seawall will cause additional instability and the cove area will experience rapid erosion. First, Goetz and Sylver were informed that the emergency permit only granted a temporary right to a seawall and that the seawall might be ordered to be removed. If the applicants did not install a seawall that could be removed without increasing the instability of the bluff, such harm is truly self-inflicted.

Delaying construction of a permanent seawall until additional erosion occurs has a substantial public benefit. The farther the back end of the beach erodes, the greater the area for the public to recreate. While Surfrider never recommends recreating directly below an eroding bluff, we note that a receding coastline is necessary to ensure that public access is maintained. The primary impact caused by a seawall is "passive erosion" whereby the back end of the beach is fixed in place by the seawall while the high-tide line continues to migrate landward. The beach is lost in front of the seawall. By ordering the removal of the seawall and allowing natural erosion to continue, the beach area and therefore public access is maintained.

6. Permit Must Be Denied Based on the Unresolved Conflicts with the Carlsbad LCP.

Surfrider submitted substantial comments and objections in our previous comments to the Coastal Commission. We do not intend to reiterate our comments here. However, we do want to point out a couple of mandatory requirements that were not imposed on the project. The Carlsbad LCP expressly states:

[9] GSI's "peer reviewer"

> **Developments shall be conditioned to provide the public with the right of access to a minimum of twenty-five feet of dry sandy beach at all times of the year**. The minimum requirement applies to all new developments proposed along the shoreline requiring any type of local permit including a building permit, minor land division or any other type of discretionary or nondiscretionary action.

(Carlsbad Municipal Code (CMC) § 21.204.060(A)(1); See also, CMC § 21.204.060 "[seawalls] shall not obstruct or interfere with the passage of people along the beach at any time"]

The failure to impose such a condition was explained in the report at the Planning Commission dated April 7, 2010, "The existing beach area is and has been subject to tidal action and does not provide twenty-five feet of dry sandy beach at all times of the year. The project is not able to increase the extent of the beach to provide a permanent twenty-five feet of dry sandy beach as the area does not exist within the cove for the creation of such a beach that would not be susceptible to wash and erosion from wave action." The Planning Commission did not impose the condition required by the LCP.

The requirement for a project to maintain 25 feet of dry sandy beach in front of the seawall does not appear to be a requirement that can be waived. Even if the requirement could be waived, it would require a variance, which was not sought nor granted in this case. Surfrider has not identified any provisions within the Coastal Act that permits the Coastal Commission to grant a variance relieving an applicant of mandatory conditions expressly stated in an LCP.

The Coastal Commission's review is limited to finding that the project is consistent with the LCP and the public access and public recreation policies of Chapter 3 of the Coastal Act. (Coastal Act §§ 30604(b) & (c).) The seawall will eventually destroy public use and access of the beach in this location, and is not consistent with the LCP. (Coastal Act §§ 30210, 30211, 30212.) The Coastal Commission must deny the permit.

7. If Approved, the Coastal Commission Should Impose a Requirement to Pay for Public Access Improvements Nearby.

Seawalls eventually destroy the beach through the process of passive erosion. Allowing seawalls to prematurely protect private property favors the few that can afford to build directly on the coast, over the general public that uses the beach. Prior to development, surfers and beach goers would access the beach along the north end of the cove. To mitigate the loss of beach access, stairs were created at the south end of the cove.

The real impact of the seawall will be eventually loss of the beach and beach access at the cove and along Terramar beach. This will cause an irreparable impact to a beach that is popular due to the offshore reefs that produce both surfable lefts and rights during large swells. Surfrider strongly urges the removal of the seawall. Nevertheless, beach access is lacking at the far north end of Carlsbad State Beach. In the past, Surfrider has suggested that paying for State Parks to

build beach access might be appropriate mitigation for the project. After all, the project will have a substantial impact on beach access and ocean dependent recreation in direct violation of Coastal Act §§ 30210, 30211, 20212 and 30220.

8. The Coastal Commission Should Impose a Sand Mitigation Fee Based on 40 Feet of Erosion Over the Next Year.

Section 30235 of the Coastal Act requires the imposition of conditions to mitigate the adverse impacts to shoreline sand supply. Carlsbad LCP imposes a similar condition, which generally requires the payment of an in-lieu sand mitigation fee to be used for large scale beach replenishment projects. In this case, Carlsbad imposed a sand mitigation fee of $2,469, which was allegedly sufficient to purchase the loss of sand at $3 per cubic yard. Such mitigation is palpably inadequate to mitigate the loss of sand created by the construction of the seawall.

GSI is claiming that the emergency installation of the seawall was necessary based on its analysis showing the primary structures are in imminent threat of erosion. This means that erosion is predicted to threaten the structures in the next one to two storm seasons. (Ex. N, GSI Rep.3 at p.3.) Should the Coastal Commission approve the seawall based on GSI's analysis, it should impose a mitigation fee based on 40 feet of erosion during the next year, and a reasonable rate of erosion over the following 75 years. The applicants should not be able to argue that the structures are in danger from catastrophic bluff erosion, while simultaneously arguing that the bluff erodes 0.05 feet per year. Not only should a proper in-lieu mitigation fee be imposed, but a fee to mitigate the recreational impacts should be imposed as well.

9. The Coastal Commission Should Uphold the Appeal Denying the Seawall on Safety Grounds, Even If It Approves the Seawall Based on the Factor of Safety Analysis Submitted by GSI.

We strongly urge the Coastal Commission to deny the seawall and order the seawall removed. However, if the Coastal Commission finds that a seawall is consistent with Section 30235 and the LCP then it should make the findings based on the safety of the structures, and not on the basis asserted by the City of Carlsbad - - the safety of the people on the beach.

The precedent setting nature of Carlsbad's approval of the permit mandates a denial of the seawall on such basis. If a seawall could be justified based on the safety of the people on the beach, then literally any bluff-top property could justify the installation of a seawall regardless of the setback. Such approval would blow a giant hole in section 30253 (mandating sufficient setback to avoid the need for protective devices), which is already suffering from enforcement problems due to the influence of questionable geotechnical reports. If the seawall is approved, there must be a finding that the seawall is being denied on the basis of the original application, but being granted based on the subsequent geotechnical reports.

Of course, Surfrider objects to the gamesmanship of Goetz, the fabricated nature of the emergency seawall installation, and the manipulation of the science used to justify the seawall for structures that are more than 40 feet away from the edge of the bluff. The seawall should be

denied on that basis. However, if the Coastal Commission finds that the seawall is justified based on a threat to the structures, then, at least, it will not eviscerate all attempts to protect natural beaches from a gluttony of armoring.

CONCLUSION

As outline above, the geotechnical reports and erosion estimates for the site vary significantly depending on whether the report is intended to support construction of new homes or construction of the seawall. In some reports, the erosion event of 1983 is cited as evidence that the structures are in imminent danger from erosion. In other reports, the formation of the cove is alleged to protect the bluff, thereby reducing the possibility of rapid erosion. GSI has cited the cove as both evidence justifying an absurdly low erosion rate (0.05 ft/yr), and evidence that the site is subject to inherent instability justifying the finding of an imminent threat to the structures. The GSI reports are particularly suspect because of they contradict each other.

What is evident is that the property owners in this case had been informed of the episodic nature of erosion at the site, and should not have been surprised when five feet of bluff collapsed. It was predicted. There is no excuse for building a seawall to protect new development 6 years into an alleged 75 year setback. The seawall should be ordered to be removed.

Sincerely,

Todd T. Cardiff, Esq.
Attorney for the Surfrider Foundation

Julia Chunn
Policy Manager
San Diego Chapter of the Surfrider Foundation

EXHIBITS IN SUPPORT OF SURFRIDER FOUNDATION'S COMMENTS IN OPPOSITION TO PROJECT MAY 2014

GOETZ/SYLVER SEAWALL COASTAL DEVELOPMENT PERMIT APPEAL NO. A-6-CII-10-043

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B	Converse Consultants, SUPPLEMENTAL GEOTECHNICAL INVESTIGATION PROPOSED COASTAL PROTECTION MEASURES FOR THE 2± ACRE COASTAL SITE CARLSBAD, CALIFORNIA, PROJECT NO. 83-2299-02	September 20, 1984
C	ICG Inc., GEOTECHNICAL INVESTIGATION AND BLUFF RETREAT STUDY, PARCEL NO. 210-120-30, CARLSBAD CALIFORNIA	March 28, 1991
D	Southland Geotechnical Consultants, GEOTECHNICAL EVALUATION OF COASTAL BLUFF PROPERTY, VACANT PARCEL SOUTH OF 5305 CARLSBAD BOULEVARD, CARLSBAD, CALIFORNIA	March 10, 1997
E	C.J. Randle, P.E. Letter re: ASSUMPTION OF GEOTECHNICAL RESPONSIBILITY, UPDATED GEOTECHNICAL REPORT, ASSESSOR PARCEL NO. 2010-120-30, CARLSBAD, CALIFORNIA	March 16, 1998
F	Owen Engineering Group, Letter re: GEOTECHNICAL REVIEW AND UPDATE OF APPLICABLE GEOLOGY AND GEOTECHNICAL REPORTS FOR 5324 CARLSBAD BLVD. CARLSBAD, CA	October 10, 2001
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L	Geosoils, Inc., GEOTECHNICAL EVALUATION OF COASTAL BLUFF STABILITY 5323 AND 5327 CARLSBAD BOULEVARD, CARLSBAD, SAN DIEGO, CALIFORNIA	July 12, 2012
M	California Coastal Commission, Letter re: COASTAL DEVELOPMENT PERMIT APPEAL # A-6-CII-10-043	January 8, 2013
N	Geosoils Inc, GEOTECHNICAL RESPONSE TO CALIFORNIA COASTAL COMMISSION REVIEW COMMENTS, SEAWALL LOCATED AT 5323 AND 5327 CARLSBAD BOULEVARD, CARLSBAD, SAN DIEGO COUNTY, CALIFORNIA, COASTAL DEVELOPMENT PERMIT APPEAL # A-6-CII-10-043	January 24, 2014

EXHIBIT A

Converse Consultants

Geotechnical Engineering
and Applied Sciences

GEOTECHNICAL INVESTIGATION
PROPOSED COASTAL PROTECTION
MEASURES FOR THE ECKE SITE
CARLSBAD, CALIFORNIA

Conducted For:

R. M. NOBLE & ASSOCIATES
22235 Pacific Coast Highway
Malibu, California 90265

Project No. 83-02299-01
February 1, 1984

Converse Consultants, Inc.
1440 South State College Blvd.
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PLAN CHECK NO. 1 Docket 77

Converse Consultants

Geotechnical Engineering
and Applied Sciences

GEOTECHNICAL INVESTIGATION PROPOSED COASTAL PROTECTION MEASURES FOR THE ECKE SITE CARLSBAD, CALIFORNIA

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Converse Consultants

Geotechnical Engineering
and Applied Sciences

February 1, 1984

R. M. Noble & Associates
22235 Pacific Coast Highway
Malibu, California 90265

Attention: Mr. Ron Noble

Subject: Geotechnical Investigation 1.2± Acre
Ecke Site, Carlsbad, California
(CCI Project No. 83-02299-01)

Gentlemen:

This report presents the results of our geotechnical investigation for the 1.2± acre site located west of Carlsbad Boulevard, south of Shore Drive, and northwest of Cerezo Drive in Carlsbad. This property is adjacent to the Pacific Ocean between an existing residential area on the north side and the La Costa State Beach on the south side. This work was conducted in accordance with our letter of confirmation dated 17 November 1983, authorized orally on 16 November 1983. The results herein have been discussed with you and your Structural Engineer, Mr. Gary Karinen.

Thank you for this opportunity of working with you on this project. Please do not hesitate to call if we can help you further.

Yours very truly,

CONVERSE CONSULTANTS, INC.

Dennis L. Hannan
Managing Vice President

MEU/MEB/DLH:bl

Dist: (7) Addressee

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INTRODUCTION

Problem Defined

During the recent past, the sea cliff area at the referenced site has undergone rapid erosion (see Figure 1, Vicinity Map, and Photographs 1 and 2, following pages). During the severe stormy period of August 1983, significant sea-cliff retreat occurred at the site, as well as at other nearby locations. The property owner had most recently attempted to arrest the erosion by guniting portions of the exposed materials, regrading, placing fill, etc. Reportedly, the owner expressed concern regarding stability of the sea cliff and the need to protect this area from potential future damage.

The purpose of our services were to obtain information relative to the geologic and subsurface conditions, to enable us to provide appropriate geotechnical engineering design parameters, and to assist you in the final design of coastal protection.

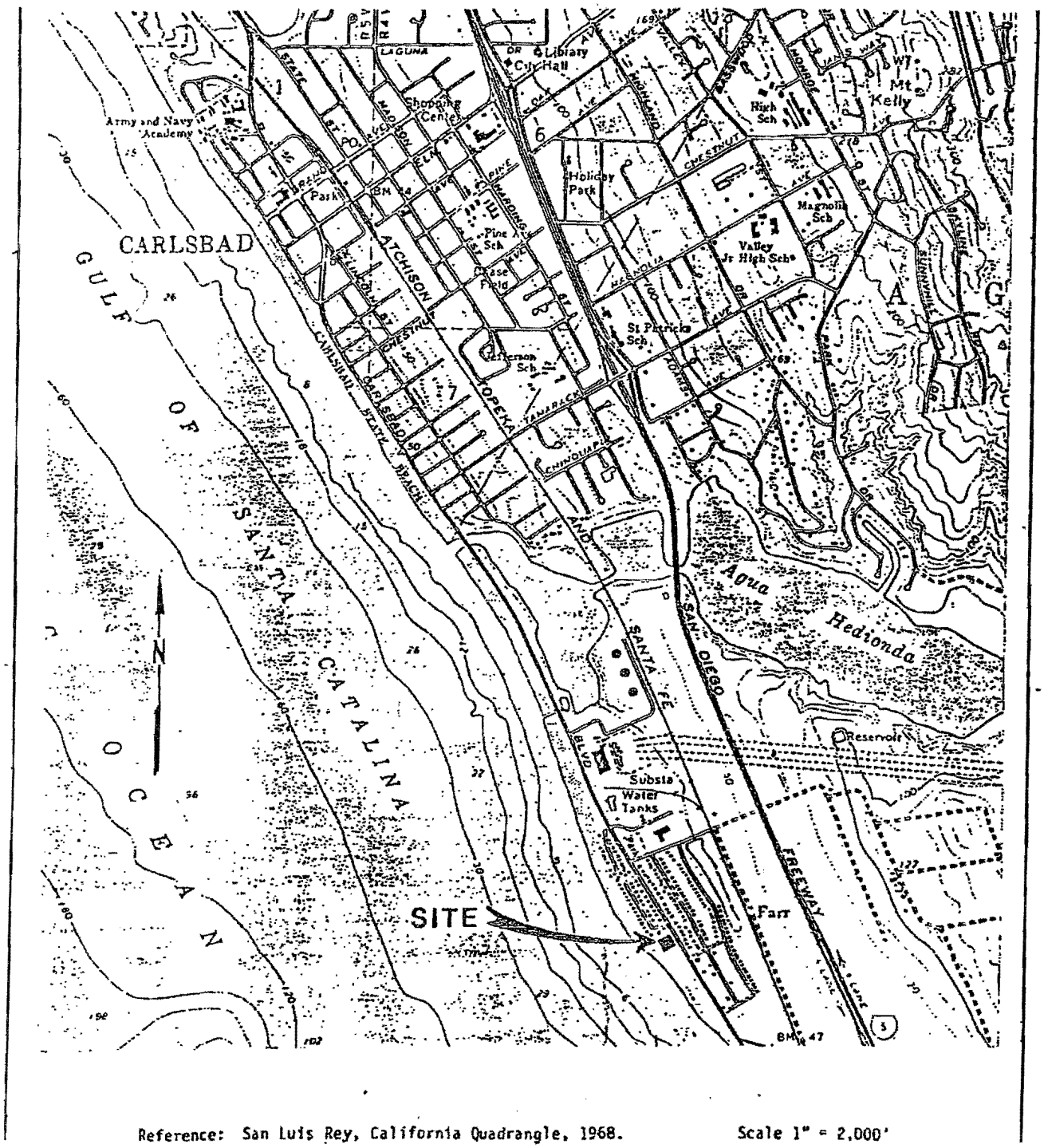
Scope of Work

Our scope of services performed included:

- Obtaining a detailed topographic map and selected profiles of the beach and cliff area at 1" = 20' with 2-foot contour intervals;

- Detailed geologic mapping on the new topographic base map;

- Drilling and logging of two (2) core holes to obtain subsurface information and to extract samples, and excavating and logging of five (5) trenches;



Reference: San Luis Rey, California Quadrangle, 1968. Scale 1" = 2,000'

VICINITY MAP

1.2± ACRE ECKE SITE
Near Cerezo Drive and Carlsbad Blvd.
Carlsbad, California

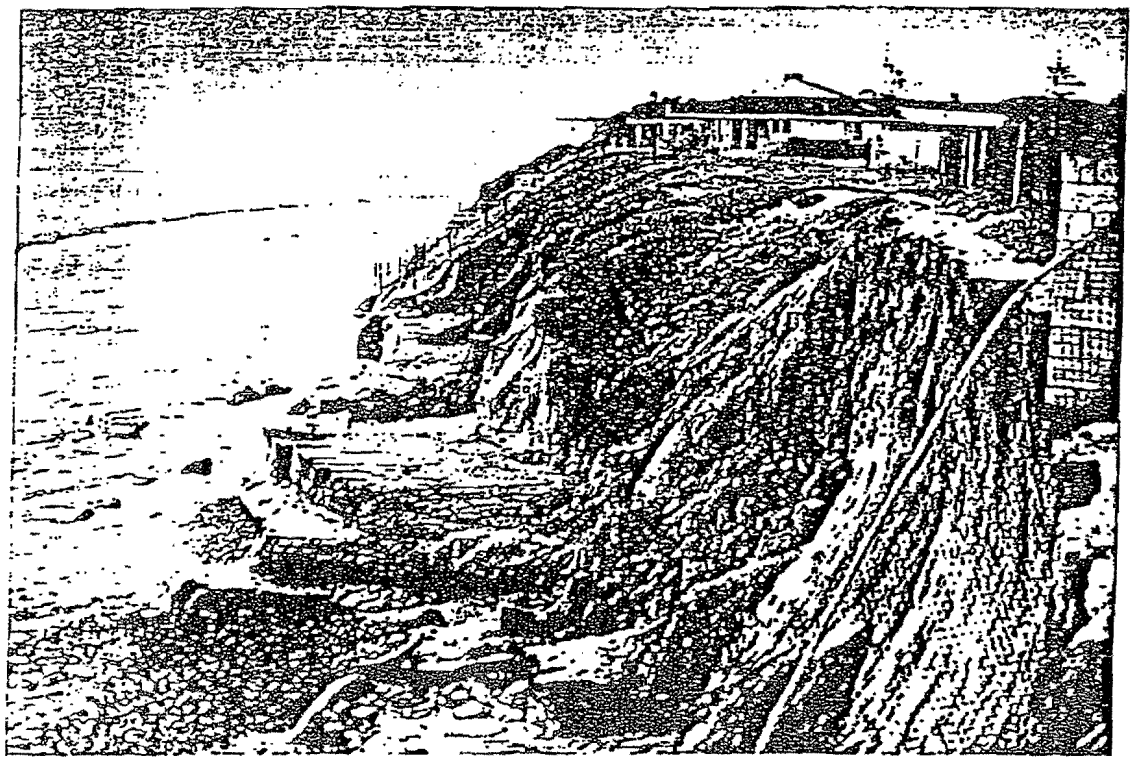
Project No.
83-02299-01

Figure No.
1

Converse Consultants Geotechnical Engineering and Applied Sciences

000949

Photograph 1. View looking northwest at site on September 23, 1978.
Courtesy of Gerry Kuhn.



Photograph 2. View looking northwest at site on August 9, 1983.
Courtesy of Gerry Kuhn.

- Laboratory examination and testing of selected samples, including tests for in-place moisture and density, shear strength, and unconfined compression;

- Retaining a consultant to obtain information relative to past coastal erosion;

- Geologic and engineering evaluations and analyses based on the data obtained;

- Coordination with R. M. Noble & Associates during the various stages of design; and

- Preparation of this report, which includes all field and laboratory test data, drawings, and recommendations for design and construction.

Pre-Existing Conditions and Coastal Erosion

The following paragraphs pertaining to pre-existing conditions and coastal erosion are based on personal accounts from Mr. Gerry Kuhn, a Research Associate at Scripps Institution of Oceanography, or data reviewed by Mr. Kuhn. Prior to 1978, the site consisted of a relatively low, moderately steep, irregular cliff face ascending from a narrow sandy beach to a relatively flat surface. The sandy beach existed from at least 1942 to late 1977, and provided a buffer to direct wave attack between the ocean and the bluff.

During late 1977, severe erosion was caused by a period of moderate wave action coupled with high tides. This erosion was subsequently followed by replacement of the beach sand by cobbles (see Photograph 1). In early 1978, sections of the sea cliff immediately south of the site experienced rockfalls, topples, and block-glide failures. At the site, the gunite covering portions of the cliff face was destroyed by beach

cobble ("artillery") attack during storm periods. A second application of gunite was destroyed when surface water runoff gullied the bluff face behind the gunite covering.

Between 7 and 9 August 1983, storm waves generated from the southern hemisphere combined with the perigean tide of August 1983 caused extremely rapid erosion at the site. Waves to the south of Palomar Airport Road and from Tamarack Avenue to the north (see Figure 1) were reported between 3 and 4 feet in height. Directly offshore from the site, wave heights ranged from 8 to 10 feet because of wave convergence along the axis of Carlsbad Submarine Canyon (see Figure 2, below). After the gunite was broken up and scattered by these waves on 7 August 1983, the upper portion of the sea cliff was eroded approximately 23-27 (horizontal) feet during the following 2-day period (see Photograph 2).

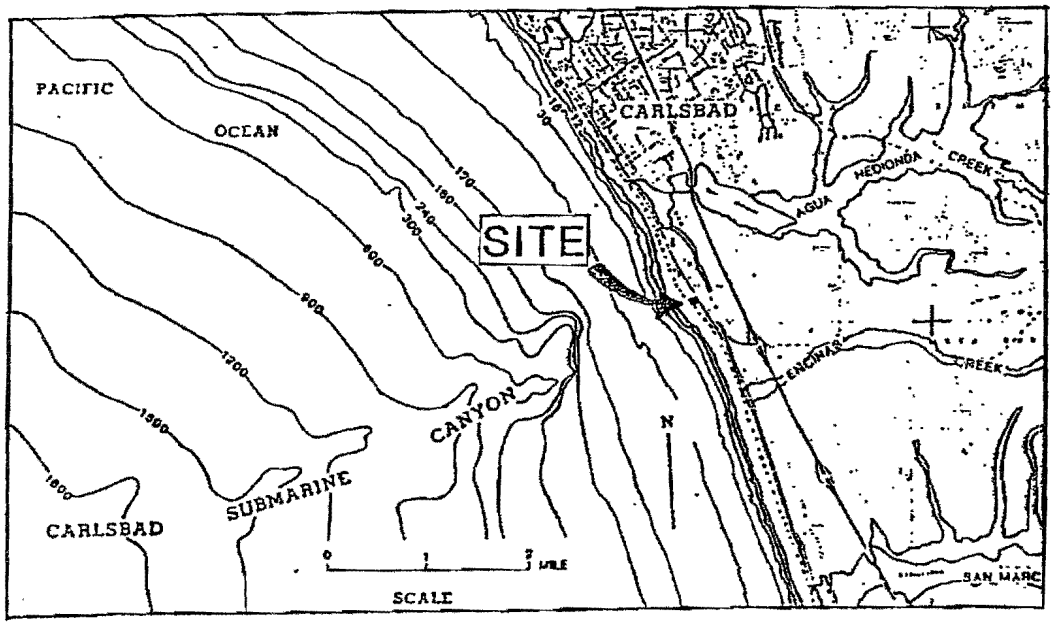


Figure 2. Site shown in relation to
Carlsbad Submarine Canyon.
Courtesy of Gerry Kuhn.

FIELD EXPLORATION AND LABORATORY TESTS

Field Exploration

The field work for this investigation included geologic mapping, trenching and drilling conducted between November 29 and December 19, 1983. Two (2) rotary core holes were drilled to depths of 28 and 33 feet. Core hole logs from these holes are presented in Appendix A. Five (5) backhoe trenches were excavated. Graphic representations of these trenches are shown on summary trench logs in Appendix A. Drill hole and trench locations, and pertinent subsurface data from this exploration are shown on Drawings 1 and 2 (in pocket).

Laboratory Tests

Selected samples were tested in the laboratory to evaluate certain engineering properties of the subsurface materials. Laboratory tests included unit weight and moisture content determinations, direct shear tests, and unconfined compression tests on the most representative materials. A description and results of the laboratory tests are presented in Appendix B.

All samples will be discarded 60 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.

EXISTING SURFACE CONDITIONS

At the time of our field exploration, the site consisted of, in general, a 40-foot high bluff ascending from a sandy, gravelly and cobbly beach to a relatively level area which comprises the majority of the site (see Drawing 1, Geologic Map, and Photograph 2). Most of the beach deposits

consisted of sand, with gravel and cobbles noted adjacent to the more severely eroded areas of the cliff. Elevations at the beach level range from about sea level to +8 feet mean sea level (MSL) in the "cove" area. Scattered bedrock (sandstone) blocks were observed along the northwestern portion of the beach.

A relatively level bedrock platform exists within the northwestern half of the site immediately adjacent to the beach. This platform is about 20 feet wide where the material above the more resistant sandstone had been recently eroded as previously discussed. Elevation of the platform is approximately 13 feet MSL. A nearly vertical irregular face descends to the beach, with an intermediate step at elevation 10 feet MSL along the southwestern portion of the platform. A gunite-covered slope ascends from this platform at about 50°-55° (3/4:1, horizontal to vertical). Tension cracks extend discontinuously 2 to 4 feet landward of the outer edge or ocean side of the platform.

Fill material within the southeasterly half of the cliff area or "cove" was apparently placed to retard further cliff deterioration. This fill slopes up to a relatively flat 25-foot wide surface at approximately elevation 20 feet MSL. The natural material above the fill had been trimmed back to a smooth uniform surface extending from the fill at about 65° (1/2:1) to elevation 38 feet MSL. A ramp (access road) extends from the top of the fill in the "cove" area in a northerly direction to the upper part of the site.

The upper portion of the property consists of a generally uniform surface sloping gently to the southwest between elevations of 50 feet MSL near Carlsbad Boulevard, and 40 feet near the bluff top. Several piles of fill material were noted on this surface. Ice plant covers most of this portion of the site. No structures were observed on the property. Surface drainage is almost exclusively by way of sheet flow across the property from Carlsbad Boulevard to the ocean.

GEOLOGY

The geology of this area is relatively simple. The geologic units encountered at the site include, from youngest to oldest, noncompacted fill (Af), beach deposits (Qbs and Qbg), talus deposits (Qtal), terrace deposits (Linda Vista Formation of Wilson, 1972), channel deposits (Qc), and bedrock assigned to the Santiago Formation (Tsb, Wilson, 1972). These units are delineated on the Geologic Map (see Drawing 1, in pocket).

In general, the noncompacted fill consists of imported material stockpiled at the site and placed in the "cove" area. The beach deposits were found to be medium-grained, light to medium gray, loose to medium dense, biotite-rich sand (Qbs), gravel and cobbles (Qbg). At the time of our field exploration, the beach deposits were underlain by channel deposits at depths of about 4 to 5 feet in the "cove" area, where bedrock was not encountered, and by bedrock at depths of about 1 to 3 feet along the northwestern beach area (see Drawing 1, and Summary Trench Logs in Appendix A). The talus deposits, located on the bedrock platform in the southeastern portion of the study area, consist of locally derived material eroded from the terrace deposits which comprise the steep sea cliff above the platform.

Pleistocene-age terrace deposits, consisting of both nonmarine and marine deposits, cap the underlying bedrock at about the 13-14 foot MSL elevation along the sea cliff and probably at a similar elevation beneath the remainder of the site. Wilson (1972) refers to these deposits as the Linda Vista Formation. The basal 2- to 3-foot thick marine deposits were noted in the southeastern portion of the study area. These deposits are indistinctly stratified, poorly consolidated, medium- to coarse-grained sand, gravel and rounded cobbles. Evidence of marine origin was noted by the presence of pholad (fossil clam) borings within the exposed bedrock platform. The nonmarine terrace deposits are well-stratified, poorly consolidated, medium- to coarse-grained sand (see

Drawing 2, Geologic Sections). Stratification of the nonmarine terrace deposits was found to dip (tilt downward) uniformly at about 2°-3° in a southwestern direction or directly toward the ocean. Based on available data (Ku and Kern, 1974; Kern, 1977; Karrow and Bada, 1980), the marine deposits may correspond to the Nestor terrace and are probably on the order of 80,000 - 120,000 years old.

The Pleistocene-age channel deposits encountered buried beneath the beach deposits in the "cove" area, are massive, medium gray to olive brown, medium- to coarse-grained, poorly consolidated (uncemented), sand with occasional very thin silt layers (see Drawing 2, Geologic Section B-B'). These deposits are quite possibly related to either a former (ancestral) Agua Hedionda Creek or Encinas Creek and deposition of fluvial material near the head of the Carlsbad Submarine Canyon prior to, or similtaneous with, early terrace deposition. It should be noted that the more extensive landward erosion in the "cove" area has probably occurred because of the erodible channel deposits, rather than the more resistant sandstone bedrock, present at that location.

The Eocene-age Santiago Formation (sandstone member of Wilson, 1972) observed during the geologic mapping and exploratory drilling consisted of generally fine- to medium-grained, gray and greenish gray to light yellow brown, massive to thinly bedded, soft to moderately hard, slightly to moderately weathered, slightly to moderately fractured (jointed), slightly cemented sandstone with occasional discontinuous clayey siltstone interbeds and lenses (see Drawing 1, Geologic Map). As previously indicated, this sandstone bedrock forms the platform and lower sea cliff up to elevation 12 to 14 feet MSL within the northwestern half of the site (see Drawing 2, Geologic Section A-A'). Ground water was encountered in Drill Hole D-1 at a depth of 13 feet, or just slightly above mean sea level. Bedding in this area, as determined by orientation of exposed siltstone beds, was found to dip generally at low angles of 2° to 4° in various directions. Core samples obtained from the drill holes revealed dips of 2° to 5°.

In general, the sandstone exposed was considered massive and uninterrupted by discontinuities such as faults, fractures, joints, etc. However, a few joint surfaces were observed trending essentially perpendicular to the beach or shore direction. The joint attitudes shown on Drawing 1 represent joint sets with spacing between observed joint surfaces of about 1 to 2 feet. More significantly, a series of closely spaced tension cracks were observed on the bedrock platform between 2 and 4 feet from the platform edge (see Drawing 1). Based on the noted conditions, it is our opinion that these cracks are related to stress release in the bedrock, particularly due to removal (erosion) of the overlying terrace deposits. During the recent past and periods of storm activity, portions of the bedrock have been displaced onto the beach. Many such blocks were evident. Additional bedrock degradation should be anticipated following similar evolution of cracking, separating, and dislodging along joints and other planar features, unless this area is protected. The present tension cracks are probably truncated at or above underlying siltstone beds where they are present.

The siltstone lenses within the Santiago Formation at the site vary between about 6 and 12 inches in thickness. They have been relatively easily eroded and are presently marked by cavities and narrow shelves within the sandstone exposed below the platform. Concretions were noted at the more resistant parts of the sandstone exposures.

DISCUSSION

Based on our investigation, and the engineering and design studies performed by R. M. Noble & Associates, alternative coastal protection measures for the site were considered and included a rock revetment along the subject shoreline or a retaining (sea) wall along portions of this shoreline. Adequate wave protection and minimal alterations to the existing site conditions were considered in the selection of the coastal protection measures. Ultimately, a retaining wall along the northern half of the shoreline founded on the existing bedrock platform in conjunction with stone revetment protection along the base of the

exposed sandstone bedrock and along the remaining southern half of the shoreline of the site were selected. A plan view of the proposed coastal protection measures is shown in Drawing 3 (following the text). Cross-sectional views of the proposed retaining wall and stone revetment sections are shown in Drawings 4 and 5, respectively.

As shown in Drawings 3 and 4, the proposed retaining wall will essentially consist of a single row of drilled cast-in-place concrete piles extending from the top of wall to an appropriate depth for which sufficient bearing support and lateral resistance are developed for the wall. The piles would be spaced 8 to 10 feet on centers. The section between the piles and between the top and 4 feet below the base of the proposed wall on the bedrock platform will consist of a reinforced concrete retaining wall section structurally tied to the adjacent piles. In order to account for the possibility of scouring and degradation of the existing bedrock platform due to wave forces, bearing and lateral support of the wall from the piles are to be developed below mean sea level in bedrock. The design of the wall will consist of a continuous stem section extending about 11 to 12 feet horizontally from the base of wall and beneath the wall backfill. Anticipated maximum downward bearing loads on the piles are on the order of about 130 to 140 kips per pile, and lateral loads per pile are on the order of about 100 kips. The backfill above the top of the proposed retaining wall at elevation 24 feet MSL will generally consist of a 2:1 (horizontal to vertical) slope. The existing bedrock platform below the base of the proposed wall at about elevation 13 feet MSL is planned to be protected by a stone revetment extending to the present beach level.

The southern half of the shoreline will consist of stone revetment protection as shown in Drawings 3 and 5. This revetment would essentially extend from below the present beach level to elevation 24 feet MSL. Above elevation 24 feet, a 2:1 cut and/or fill slope is proposed.

For the selected proposed coastal protection measures, our geotechnical recommendations for design and construction are presented in the following section of this report. It should be realized, however, that the proposed structures are planned in areas with a relatively high probability of change, particularly as a result of existing conditions and anticipated potentially destructive storm waves this season. Some on-going distress to the cliff area should be expected prior to the proposed construction. If conditions change, our recommendations presented in this report may have to be modified.

RECOMMENDATIONS

Retaining Wall

We recommend that the proposed retaining wall be designed for an active earth pressure of 45 psf/ft of depth below the top of the wall supporting a 2:1 (horizontal to vertical) sloping backfill. For portions of the wall supporting intact bedrock, a minimum active earth pressure of 30 psf/ft below the top of wall may be used in conformance with UBC code requirements.

The retaining wall should be well-drained with a minimum 18-inch thick layer of filter gravel placed over the horizontal stem at the base of the wall and beneath the backfill, and relieved by weep holes and/or a drainage pipe placed along the base of the wall (see Drawing 4). The filter gravel should conform to a Class I, Type A filter drainage material per Caltrans Specifications (1981), Section 68-1.025, and should be protected from siltation by the overlying wall backfill material by a suitable high-strength, tear-resistant, woven or nonwoven filter fabric having an Equivalent Opening Sieve size (EOS) of 70-100. A subdrain pipe placed behind the wall should consist of a minimum 4-inch diameter, noncorrosive drainage pipe having maximum 3/8-inch diameter hole perforations. The pipe should be surrounded by at least 6 inches of the recommended filter gravel. A minimum 12-inch thick layer

of well-graded washed concrete sand or a suitable commercially available drainage panel placed at the wall backface and extending to the top of the recommended filter gravel drain should be included.

The backfill for the retaining wall may consist of the onsite soils, provided that they are free of any deleterious substances and particles greater than 6 inches in size. If imported backfill material is used, the import material should have an Expansion Index less than 30 and approved by the Soil Engineer. All wall backfill should be properly compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D1557-78. Additional recommendations for placement of the wall backfill is given under "Site Grading and Earthwork".

The support of the retaining wall, both vertically and laterally, will be accomplished with the use of piles. Although lateral resistance for the wall could probably be obtained to a certain extent by frictional resistance at the base of wall against the underlying bedrock, such support would be severely reduced if the bedrock beneath the wall was subjected to scouring by waves. Recommendations for the design of piles and for lateral resistance design are given in the following sections.

Pile Design for Retaining Wall

Drilled, cast-in-place, concrete friction piles are recommended for the support of the retaining wall. Based on our investigation, the piles will develop their designed capacity in the underlying bedrock. A maximum skin friction value of 4,000 psf may be used in design; however, it is expected that the requirements for lateral load resistance will govern the design pile lengths. The portion of the piles above mean sea level should not be considered for any available vertical and lateral load support in light of possible scouring within this zone. The piles should be spaced not less than 3 pile diameters apart, center-to-center.

Resistance to the anticipated loads on the proposed retaining wall may be obtained from the portion of the piles below mean sea level supporting the wall. The design data for 30-inch and 36-inch diameter drilled piles are given in Table I (below). No applied moments and lateral restraint of the top of pile at mean sea level (free-head conditions) are assumed.

TABLE I

LATERAL PILE DESIGN CRITERIA[1]

	Pile Diameter	
	30 Inches	36 Inches
Allowable Lateral Capacity, P, Kips	100	130
Maximum Moment[2]	4.2 P	4.9 P
Depth to Maximum Moment, ft.[3]	7.0	8.5
Depth to Zero Moment, ft.[3]	18.0	21.0
Minimum Pile Embedment Length, ft.[3]	18.0	21.0
Computed Lateral Deflection at Sea Level, in.	1/2 to 1	1/2 to 3/4

Notes:

1. Pile design data include consideration of scouring above mean sea level.

2. Moment due to lateral load, P, applied at mean sea level of pile; if lateral load is in kips, the moment will be in foot-kips.

3. Depths measured from mean sea level.

In our opinion, the drilling of proposed piles may be accomplished with a conventional drilling machine equipped for drilling in moderately hard rock. Provisions in drilling, however, should be provided for penetrating occasional harder rock layers. Severe caving is not expected in the drilled pile excavations, provided that a positive hydrostatic load

is maintained in the hole. As indicated in our drill holes, ground water is expected in the drilled pile excavations extending below mean sea level. Much of the placing of steel and concrete for the piles will have to be performed below the ground-water level. The placement of concrete below water should be performed by forced pumping of concrete through a tremie. It is recommended that drilling of the pile excavations be performed under the observation of the Soil Engineer.

Site Grading and Earthwork

In general, areas that are to be graded for the proposed site improvements should be cleared of debris, trash and rocks greater than 6 inches in size. In all areas to receive compacted fill, the ground surface should be properly compacted in the top 12 inches to provide a suitable surface to place fill. Compacted fill to be placed in areas sloped more than 5:1 (horizontal to vertical), should be properly benched and keyed into competent material. Fill should be placed in layers not exceeding 8 inches in thickness when loose. All fill should be compacted to at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557-78 (see Appendix C).

In our opinion, the on-site soils and bedrock material may be excavated with conventional grading equipment. Materials obtained from on-site excavations may be used as compacted fill material, provided they are free of any deleterious substance and particles greater than 6 inches in size. Any material to be imported to the site for use in compacted fills should be approved by the Soil Engineer. The placement and compaction of fills should be performed under the observation and testing of the Soil Engineer.

Finished Slopes and Slope Protection

We recommend that finished fill and cut slopes be no steeper than 2:1 (horizontal to vertical). All permanent finished slopes should be protected from erosion, particularly from wave effects. Finished slopes

not directly exposed to wave forces may be protected from erosion, as appropriate, using deep rooting ground cover plantings, surface pavings, soil cement, or a combination of these procedures. Surface paving may consist of gunite, asphalt concrete, Portland cement concrete, or suitable masonary or rock veneers. Soil cement, consisting of at least 5 percent of Portland cement by weight mixed with the on- site soils, may be used to provide a stabilized ground surface to resist erosion; the zone of soil cement protection should be at least 12 inches thick. Positive slope drainage should be provided.

As discussed with R. M. Noble & Associates, a combination of slope face protection and ground cover protection is intended for the anticipated cut and fill slopes above elevation 24 feet MSL. An erosion mat is planned for the splash zone portion of the slope between elevation 24 feet and 34 feet MSL. Above elevation 34 feet MSL, ground cover plantings are planned. In our opinion, this protective measure should provide suitable protection against occasional splashes from waves and adequate control of surface runoff on the slope faces. Positive slope drainage, including drains (i.e., interceptor drain at the top-of-slope, "V" ditch at the toe-of-slope and top of the wall, etc.), if needed, should be provided.

Temporary Excavations

Temporary construction excavations in the undisturbed terrace deposits and bedrock, in our opinion, may be made to a vertical depth of about 5 feet without shoring. For deeper excavations, the cut surface should be sloped no steeper than 1/4:1 (horizontal to vertical), or should be properly shored. Such cuts are not expected to exceed about 15 feet in depth. Excavations in the beach sand and gravel deposits above the groundwater level, in our opinion, will generally have to be sloped no steeper than 1:1 or flatter, and excavations extending below the ground water level will require relatively flat cut slopes and/or sheeting as appropriate. All excavations and shoring systems should meet the minimum requirements given in Article 6 of the State of California Safety and Health Standards.

Excavations in bedrock extending below the ground water level generally may be dewatered by sumps and pumps. For excavations in beach deposits, dewatering by sumps and pumps may be feasible to a limited depth of about 2 feet below the ground-water level. If a dewatered excavation and a relatively stable excavation bottom are required for construction, the ground-water level should be lowered to a sufficient depth, possibly the planned toe depth, until sufficient structural constraint is placed on the excavation bottom, for which the potential for heaving and/or boiling of the excavation bottom is eliminated. Well-points may be used for deep dewatering. Alternatively, ground freezing techniques may be used to provide a stable excavation in the beach deposits below the water level.

In general, excavation of the on-site soils and bedrock may be accomplished with conventional excavating equipment. Excavation of beach deposits below the ground-water level, however, may require backhoes or draglines.

Design and Construction Review

This report has been prepared to assist the Engineer in the design of the proposed coastal protection. Prior to construction, it is recommended that this firm be provided the opportunity to review the design drawings and specifications in order to ascertain that the recommendations contained in this report and used in the final design are applicable to the site conditions.

It is also recommended that this firm be retained to provide services during construction of the structure(s). This is to observe compliance with the design concepts, specifications, and recommendations, and to provide advice regarding design changes in the event that the site and subsurface conditions differ from those anticipated prior to the start of construction.

LIMITATIONS

The findings and recommendations of this report are based on the results of the field exploration and laboratory tests combined with interpolation and extrapolation of soil and bedrock conditions between and beyond borings. There may be subsurface conditions not indicated by the exploration. However, in our opinion, the exploration is adequate to define the subsurface conditions for the purpose of this study. The recommendations represent our best professional judgment as to the procedures to be followed in design and construction based on the data obtained and the planned construction.

Professional services in connection with this geotechnical report have complied with generally accepted practice in the fields of soil mechanics, foundation engineering, and engineering geology. We make no other warranty, either express or implied. If conditions change prior to the actual construction, or if conditions are encountered during construction that appear to be different from those found during our investigation, we should be notified immediately so that appropriate modifications of recommendations can be provided, if needed.

Bidding contractors are urged to study the boring logs, trench logs, maps and specifications, and to determine to their own satisfaction the subsurface conditions that may be encountered by appropriate independent pits, borings, or other methods prior to submitting bids.

This report has been prepared exclusively for R. M. Noble & Associates for specific application to the proposed site coastal improvements described in this report. No other warranty, express or implied, is made.

Respectfully submitted,

CONVERSE CONSULTANTS, INC.

Mark E. Unruh, C.E.G. 1176
Staff Geologist

Franklin Fong, R.C.E. 24179
Project Engineer

Reviewed by:

Mark E. Bryant, C.E.G. 1046
Senior Geologist

Thomas J. Scheil, R.C.E. 31150
Principal Engineer

MEU/FF/MEB/TJS:bl

DATA REVIEWED

Aerial photographs (stereo), October 9, 1970, San Diego County, VTN, Photos 2-2 and 3-5, scale approximately 1"=2,000', (color, vertical).

Aerial photographs (stereo), December 5, 1983, Robert Bein, William Frost & Associates, Photos #1 and 2, scale 1"=170', (black and white, vertical).

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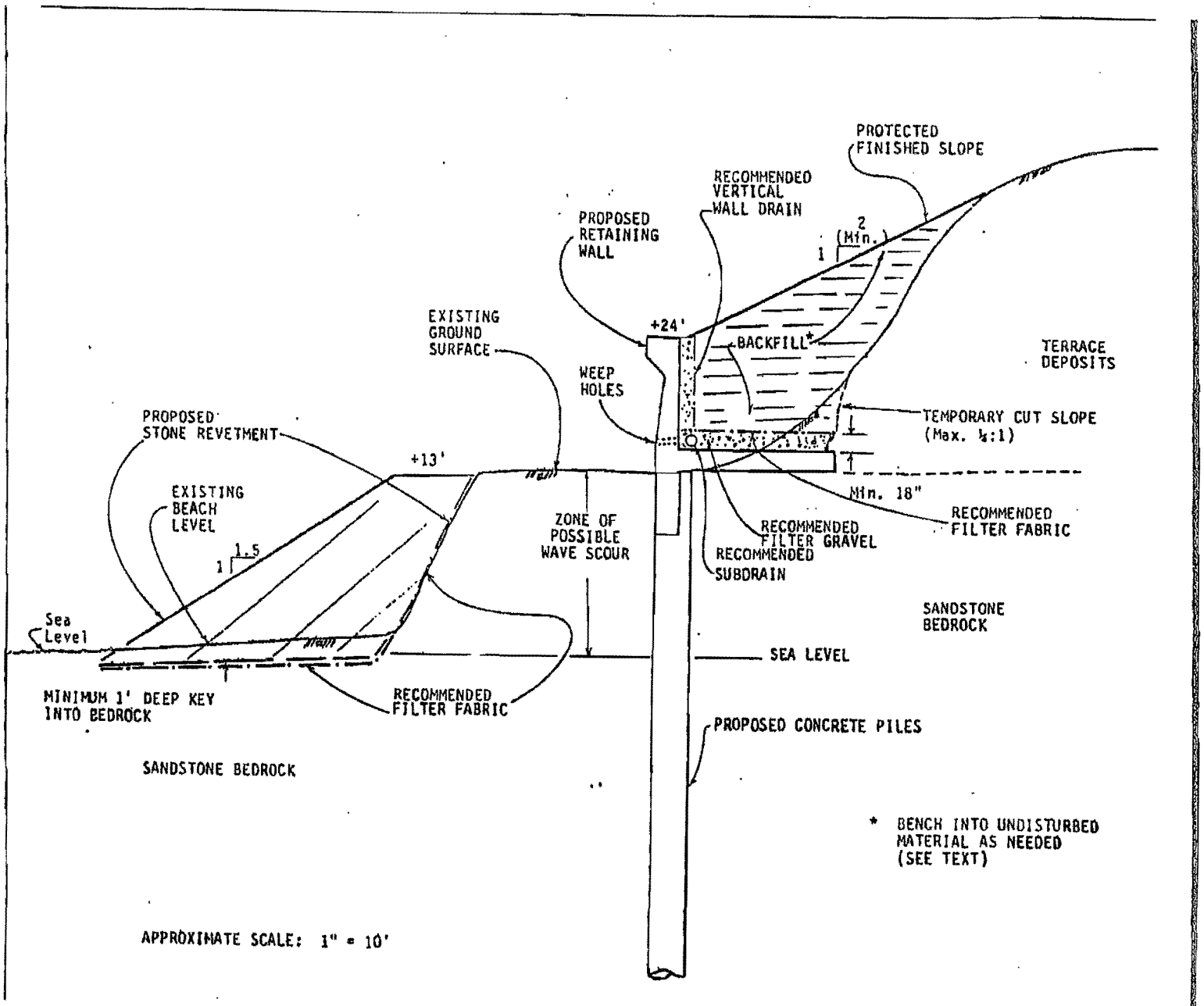
Sheet ___ Calc ___ by ___



PROPOSED RETAINING WALL/REVETMENT - TYPICAL DETAIL

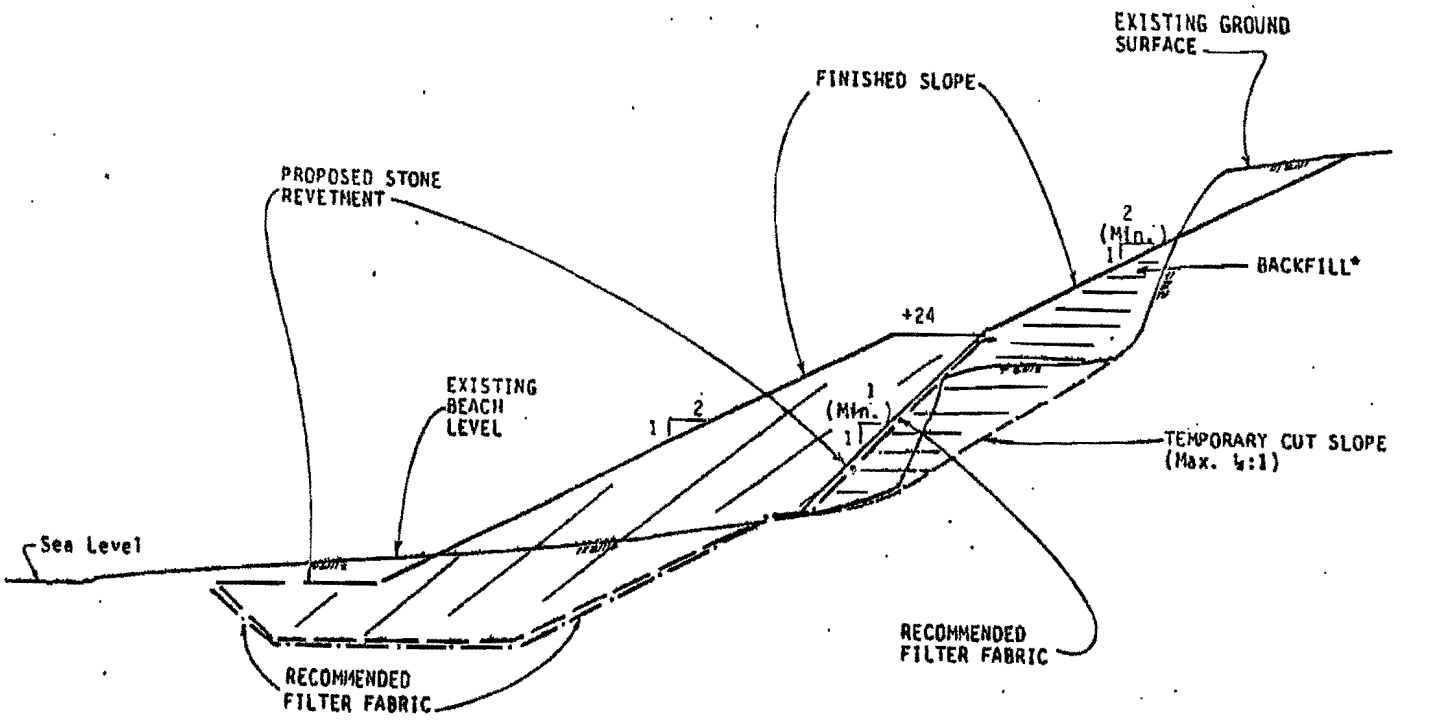
1.2± ACRE ECKE SITE
Near Cerezo Drive and Carlsbad Blvd.
Carlsbad, California

Converse Consultants
Geotechnical Engineering and Applied Sciences

Project No. 83-02299-01

Drawing No. 4

000968



* BENCH INTO UNDISTURBED MATERIAL AS NEEDED (SEE TEXT)

APPROXIMATE SCALE: 1" = 15'

PROPOSED STONE REVETMENT - TYPICAL DETAIL

1.2± ACRE ECKE SITE
Near Cerezo Drive and Carlsbad Blvd.
Carlsbad, California

Project No. 83-02299-01

Drawing No. 5

Converse Consultants
Geotechnical Engineering and Applied Sciences

000069

APPENDIX A

EXPLORATORY BORINGS

EXPLORATORY TRENCHES

TERMS USED FOR THE PHYSICAL DESCRIPTION OF BEDROCK

HARDNESS	SOFT	- Can be dug by hand and crushed by fingers
	MODERATELY HARD	- Friable, can be gouged deeply with knife and will crumble readily under light hammer blows
	HARD	- Knife scratch leaves dust trace, will withstand a few hammer blows before breaking
	VERY HARD	- Scratched with knife with difficulty, difficult to break with hammer blows
STRATIFICATION	THINLY LAMINATED	- less than 1/10"
	LAMINATED	- 1/10" to 1/2"
	VERY THINLY BEDDED	- 1/2" to 2"
	THINLY BEDDED	- 2" to 2 feet
	THICKLY BEDDED	- more than 2 feet
FRACTURING	INTENSELY FRACTURED	- less than 1" spacing
	VERY FRACTURED	- 1" to 6" spacing
	MODERATELY FRACTURED	- 6" to 12" spacing
	SLIGHTLY FRACTURED	- 12" to 36" spacing
WEATHERING	VERY WEATHERED	- Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., through discoloration, rock disintegration, mineral decomposition
	MODERATELY WEATHERED	- Some fracture coating, moderate or localized discoloration, little to no affect on cementation, slight mineral decomposition
	SLIGHTLY WEATHERED	- A few stained fractures, slight discoloration, little to no affect on cementation, no mineral decomposition
	FRESH	- Unaffected by weathering agents, no appreciabl change with depth

KEY TO SOIL SYMBOLS AND TERMS

TERMS USED IN THIS REPORT FOR DESCRIBING SOILS ACCORDING TO THEIR TEXTURE OR GRAIN SIZE DISTRIBUTIONS ARE GENERALLY IN ACCORDANCE WITH THE UNIFIED SOIL CLASSIFICATION SYSTEM.

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (more than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (more than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sands, little or no fines
		Sands with fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-grained soils (more than half of material is smaller than No. 200 sieve)	Silts and clays (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and clays (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
	Highly organic soils		Pt	Peat and other highly organic soils

TERMS DESCRIBING CONDITION, CONSISTENCY AND HARDNESS

COARSE GRAINED SOILS (major portion retained on No. 200 sieve): Includes (1) clean gravels, (2) silty or clayey gravels, and (3) silty, clayey or gravelly sands. Consistency is rated according to relative density, as determined by laboratory tests.

Descriptive Term	Relative Density
Very loose	0 to 15%
Loose	15 to 40%
Medium dense	40 to 70%
Dense	70 to 85%
Very dense	85 to 100%

FINE GRAINED SOILS (major portion passing No. 200 sieve): Includes (1) inorganic and organic silts and clays, (2) gravelly, sandy or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by direct shear tests.

Descriptive Term	Shear Strength (ksf)
Very soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very stiff	2.00 to 4.00
Hard	4.00 and higher

ROCK: Includes gravels, cobbles, rock, caliche and bedrock materials. Hardness is related to field identification procedures described below.

Descriptive Term	Field Identification Test
Soft	Can be cut by hand and crushed by fingers
Moderately hard	Friable, can be gouged deeply with knife and will crumble readily under light hammer blows
Hard	Knife scratch leaves dust trace, will withstand a few hammer blows before breaking
Very hard	Scratched with knife with difficulty, difficult to break with hammer blows

SOIL MOISTURE

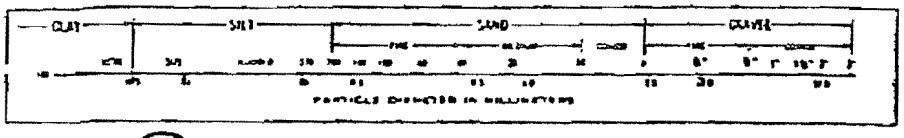
From low to high the soil moisture is indicated by:

Dry
Slightly moist
Moist
Very moist
Wet

SIZE PROPORTIONS

Designation	Percent by Weight
Trace	0 to 10
Little	10 to 20
Some	20 to 35
And	35 to 50

GRAIN-SIZE DISTRIBUTION



PROJECT NO. 83-02299-01

CORE HOLE LOG

Converse Consultants

SHEET 1 NO. 1

HOLE NO. D-1

PROJECT Ecke FEATURE Proposed Sea/Retaining Wall LOCATION 40' from fence

INCLINATION 90° BEARING STARTED 11/29 10:30 am COMPLETED 11/29 1:55 pm

TOTAL DEPTH 28.0' GROUND ELEV. 13.7'± HOLE SIZE NX Core

ELEV. WATER TABLE +0.7' WATER PRESSURE TESTS TO NO. CORE BOXES 1

LOGGED BY KEU CONTRACTOR Cal Testing DRILL FOREMAN

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Elev.	RQD	Depth	Core Rec. %	Graphic Log	Classification and Physical Condition	Drilling Information
					BEDROCK-SANDSTONE very light gray (N7), fine to medium grained, soft to moderately hard, mostly quartz, some feldspar minor biotite, surface shows holes from clams, slightly cemented, massive.	
	60	5	95			
	70		75		2-5° dip, moderately fractured, predominantly fine grained (7-8'), very thinly bedded	
	25	10	30		medium grained, light yellow brown to gray thinly bedded, moderately weathered, moderately hard	
▽		15	N/R		cuttings appear to be similar to material above	
		20				
	0	25	8			
		30			End of hole at 28.0' Ground water encountered at 13'	

PROJECT NO. 83-02299-01

CORE HOLE LOG

Converse Consultants

SHEET 1 NO.

HOLE NO. D-2

PROJECT Ecke FEATURE Proposed Sea/Retaining Wall LOCATION 90' from fence

INCLINATION 90° BEARING -0- STARTED 11/29 2:30 pm COMPLETED 11/30 9:00 a

TOTAL DEPTH 33.0'± GROUND ELEV. 13.5'± HOLE SIZE NX Core

ELEV. WATER TABLE WATER PRESSURE TESTS TO NO. CORE BOXES 2

LOGGED BY MEU CONTRACTOR Cal Testing DRILL FOREMAN

THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Elev.	RQD	Depth	Core Rec. %	Graphic Log	Classification and Physical Condition	Drilling Information
					BEDROCK-SANDSTONE very light gray (N7), medium grained, clays on fracture surface, soft to moderately hard, closed fractures, quartz, feldspar & micas	
	20	5	25			
	30		40		occasional iron stained bed sub horizontal, indistinct bedding, 2-3° dip (laminated), moderately hard, slightly to moderately weathered	
	0	10	0			
	8	15	20		soft zone	
	0	20	0		slightly weathered, moderately fractured, thinly bedded	
	100	25	100		soft to moderately hard, medium grained, (coarser than above), massive, light olive gray (5Y 5/2)	
	28	30	80		soft zone fine to medium grained, light olive gray (5Y 6/1)	
		35			End of hole at 33.0'	

SUMMARY TRENCH LOG – TRENCH NO. 1

Project Name: ECKE

Project No 83-02299-01

Date: 12-19-83

Logged By: KEU

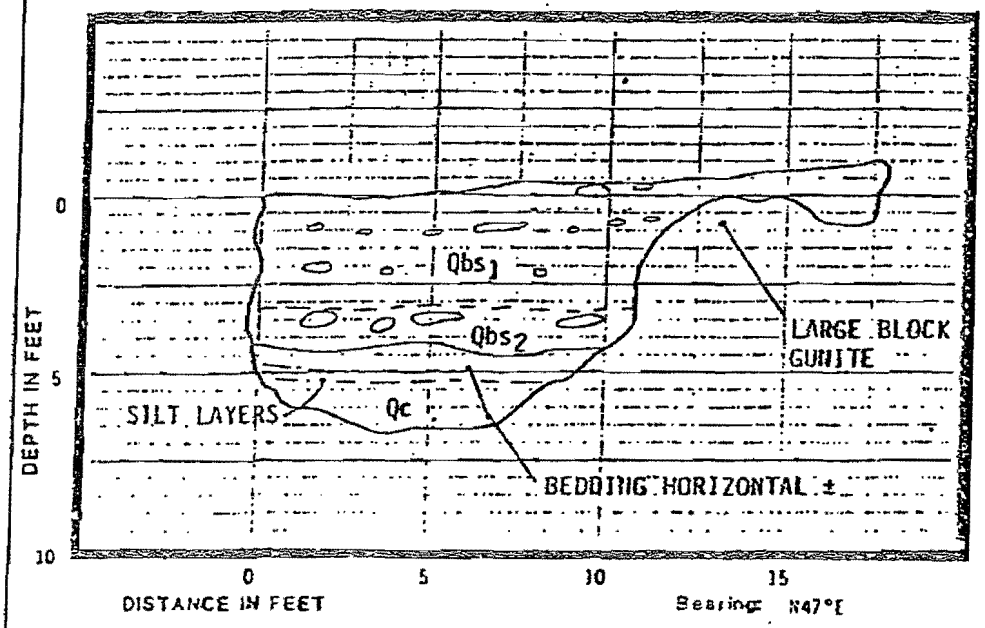
Elevation: 4'±

Trenching Equipment: INTERNATIONAL 100 BACKHOE

Ground Water Depth: 3.2'±

DEPTH IN FEET Min.	Max.	IDENTIFICATION
0.0	3.2	BEACH DEPOSITS (SAND) (Qbs_1): Mixture of cobbles and medium to coarse sand, more cobbles (to 1') near contact
3.2	4.5	BEACH DEPOSITS (SAND) (Qbs_2): Medium to coarse sand, zone of oxidized material, black
4.3	6.8	CHANNEL DEPOSITS (Qc): Massive, medium gray, medium grained, poorly cemented, sand, occasional thin silt layer
		Note: Ground water inflow immediately filled pit to top of black zone.

Northwest Wall



BEDDING ORIENTATION

NO.	STRIKE	DIP

Converse Consultants

THIS SUMMARY APPLIES ONLY AT THE LOCATION AND TIME OF EXCAVAT SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHA AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

SUMMARY TRENCH LOG – TRENCH NO. 2

Project Name: Ecke

Project No 83-02299-01

Date: 12-19-83

Logged By: HEU

Elevation: 5'±

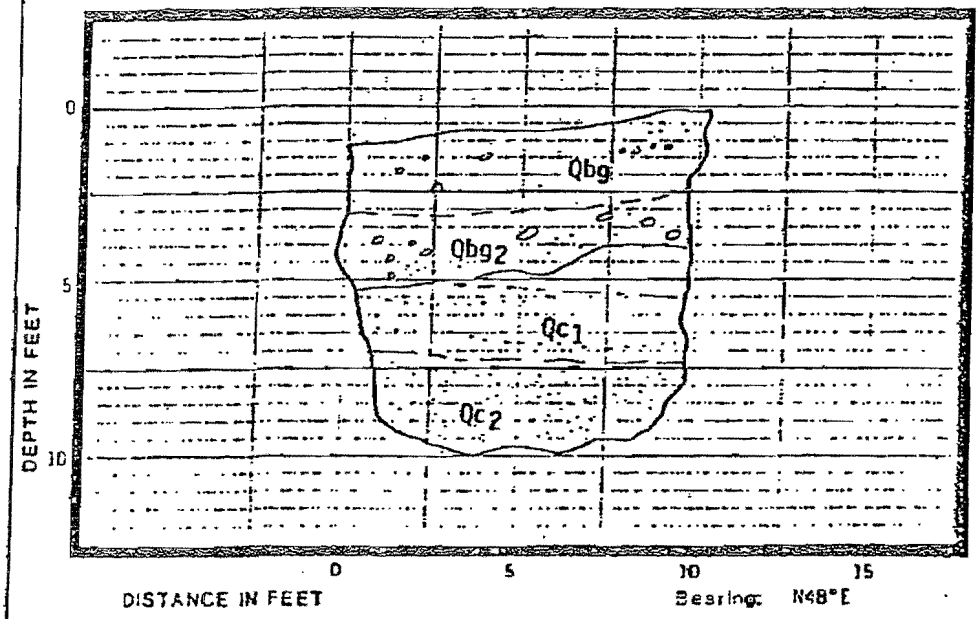
Trenching Equipment: INTERNATIONAL 100 BACKHOE

Ground Water Depth: 2.5'

DEPTH IN FEET Min.	Max.	IDENTIFICATION
0.0	2.5	BEACH DEPOSITS (GRAVEL) (Qbg_1): Mixture of medium grained sand and cobbles
2.5	4.3	BEACH DEPOSITS (GRAVEL) (Qbg_2): Mixture of medium grained sand and 1" to 3" diameter gravel, zone of stained black with organics
3.8	7.4	CHANNEL DEPOSITS (Qc_1): Massive sand, medium gray, medium grained, medium dense, with occasional silt layer
6.0	9.0	CHANNEL DEPOSITS (Qc_2): Massive sand, moderate yellowish brown, medium to coarse grained, loose, subrounded to rounded grains

Note: Ground water inflow immediately filled pit to top of black zone.

Northwest Wall



Bearing: N48°E

BEDDING ORIENTATION

NO.	STRIKE	DIP

Converse Consultants

THIS SUMMARY APPLIES ONLY AT THE LOCATION AND TIME OF EXCAVATIO SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANG AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

SUMMARY TRENCH LOG - TRENCH NO. 3

Project Name: Ecke

Project No 83-02299-01

Date: 12-19-83

Logged By: HEU

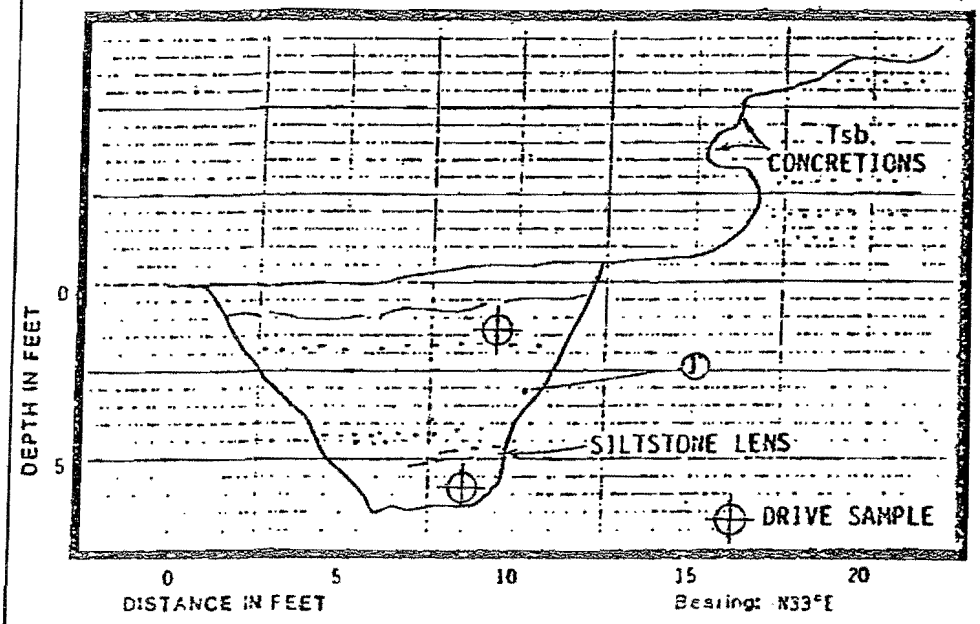
Elevation: 0.5'±

Trenching Equipment: INTERNATIONAL 100 BACKHOE

Ground Water Depth: 1.0'

DEPTH IN FEET Min.	Max.	IDENTIFICATION
0.0	1.5	BEACH DEPOSITS (Qbs): Medium gray, medium grained, sand, loose, some crossbeds, many dark minerals.
1.0	7.0	SANTIAGO FORMATION (Tsb): Sandstone, medium gray, fine to medium grained, soft to moderately hard, very moist to wet
		Note: Ground water flowed at Qbs/Tsb contact

Northwest Wall



BEDDING ORIENTATION

NO.	STRIKE	DIP
1	Horizontal	

Converse Consultants

THIS SUMMARY APPLIES ONLY AT THE LOCATION AND TIME OF EXCAVATI SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHA AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

SUMMARY TRENCH LOG – TRENCH NO. 4

Project Name: Ecke

Project No 83-02299-01

Date: 12-19-83

Logged By: HEU

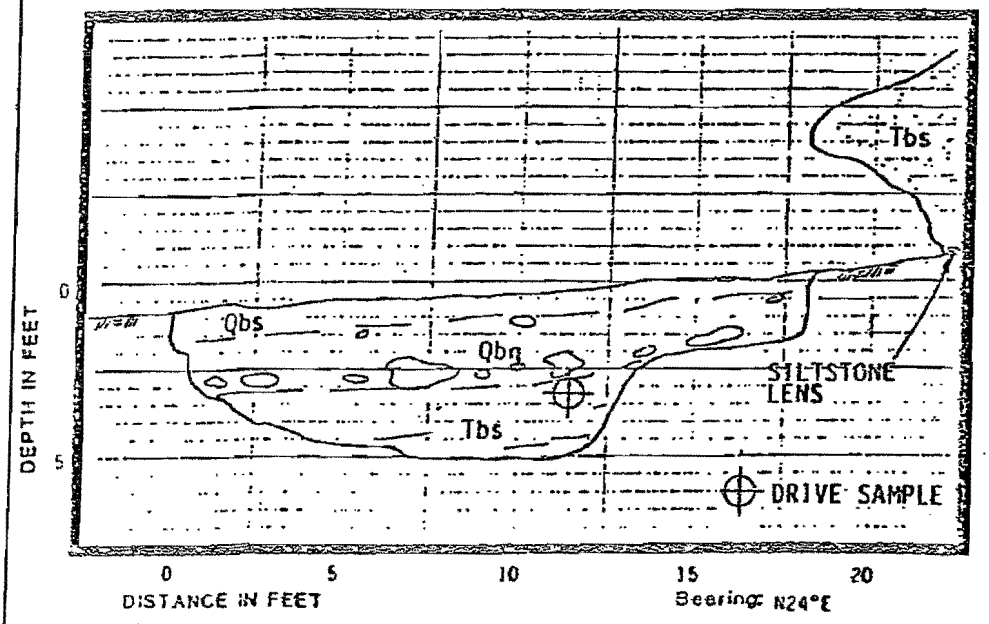
Elevation: 1.0'±

Trenching Equipment: INTERNATIONAL 100 BACKHOE

Ground Water Depth: 0'

DEPTH IN FEET Min.	Max.	IDENTIFICATION
0.0	1.5	BEACH DEPOSITS (Qbs): Sand, medium gray, medium grained, loose, with some gravel (1"-3")
0.5	3.5	BEACH DEPOSITS (Qbg): Sand, medium gray, medium grained, with 50% gravel to 80% at bottom
2.5	5.5	SANTIAGO FORMATION (Tsb): Massive sandstone, medium gray, medium grained, moderately hard, with with occasional siltstone lens.

Northwest Wall



BEDDING ORIENTATION

NO.	STRIKE	DIP

Converse Consultants

THIS SUMMARY APPLIES ONLY AT THE LOCATION AND TIME OF EXCAVA SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CH AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

SUMMARY TRENCH LOG – TRENCH NO. 5

Project Name: Ecke

Project No 83-02299-01

Date: 12-19-83

Logged By: HEU

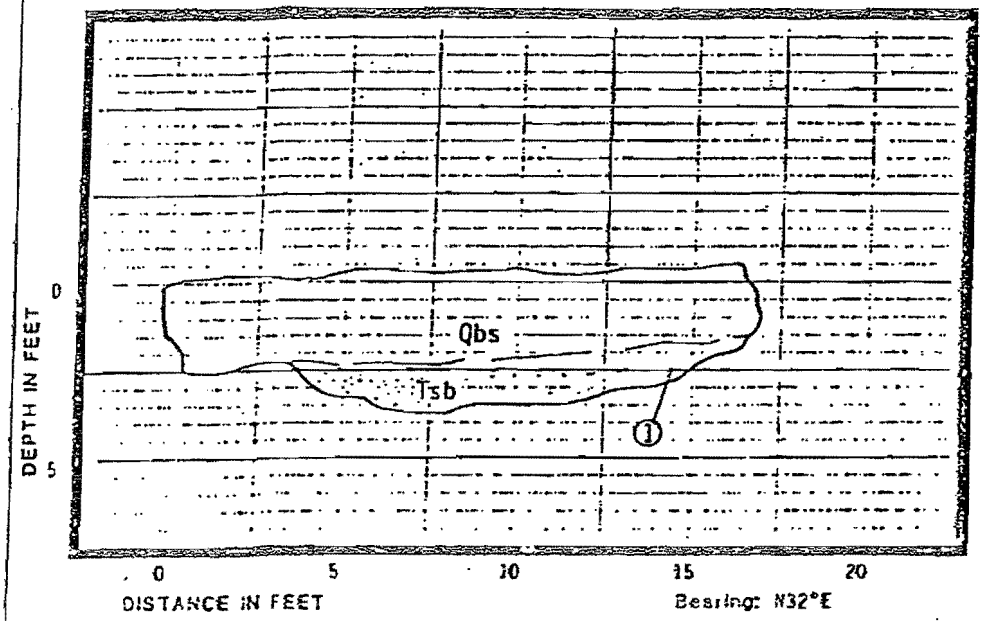
Elevation: -2.0'

Trenching Equipment: INTERNATIONAL 100 BACKHOE

Ground Water Depth: 2.0'

DEPTH IN FEET Min.	Max.	IDENTIFICATION
0.0	2.6	BEACH DEPOSITS (Qbs): Sand, medium gray, medium grained, loose to medium dense, some cross bedding
2.2	4.0	SANTIAGO FORMATION (Tsb): Sandstone, medium to light gray, medium grained, massive, soft

Northwest Wall



BEDDING ORIENTATION

NO.	STRIKE	DIP
1	Horizontal	

Converse Consultants

THIS SUMMARY APPLIES ONLY AT THE LOCATION AND TIME OF EXCAVATION. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

APPENDIX B

LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

Laboratory tests were conducted on representative soil and rock samples for the purpose of classification and for the determination of their physical properties and engineering characteristics. A brief description of the various laboratory tests conducted for this study is presented in the following paragraphs. Results of the laboratory tests performed for this study are presented on the boring summary sheets within this Appendix.

The soil samples presently stored at our laboratory will be discarded 90 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.

Moisture Content and Dry Density

Moisture content and dry density tests were performed on relatively undisturbed samples obtained from the field in accordance with ASTM Test Method D2216-80 to determine their dry densities and moisture contents, and to provide qualitative information regarding soil strength and compressibility. The results of the moisture and density tests are presented in Table B-1.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed or remolded samples in accordance with ASTM Test Method D3080-72 to determine shear strength. The tests were performed on samples at field moisture content and/or under an increased moisture content. The samples tested were sheared under a constant rate of strain under given normal loads applied to the samples. The results of the direct shear tests are presented in Table B-1.

Unconfined Compression Tests

Unconfined compression tests were performed on selected, relatively undisturbed samples in accordance with ASTM Test Method D2166-66 to determine their unconfined compressive strength. The results of the tests are summarized below.

TABLE B-1

SUMMARY OF LABORATORY TEST RESULTS

Trench or Boring No.	Sample Depth ft.	Moisture Content %	Dry Density pcf	Shear Strength C,psf	Shear Strength Ø, degrees	Unconfined Compressive Strength psf
D-1	3	12.1	127	-	-	24,320
D-1	8	7.7	125	-	-	54,520
D-1	12	5.1	123	-	-	30,480
D-2	6	4.2	124	-	-	120,770
D-2	23	2.6	126	-	-	57,600
TR-3	2	14.4	114	35	44 (2437) 2900	-
TR-3	6	14.8	115	880	32 (2146) 2400	-
TR-4	3.5	15.5	112	380	40 (2351) 2900	-

Average 2300

APPENDIX C

SUGGESTED SPECIFICATIONS FOR THE COMPACTION OF FILL

APPENDIX C

SUGGESTED SPECIFICATIONS FOR THE COMPACTION OF FILL

1. All areas to receive compacted fill shall be stripped of all vegetation, debris, existing fill, and soft or disturbed soils. The excavated areas shall be observed by the Soils Engineer prior to placing compacted fill.

2. The exposed ground surface shall then be scarified to a depth of six inches and the scarified ground shall be compacted to at least 90 percent of the maximum laboratory density as determined by the ASTM compaction method described below. Benching into firm natural ground shall be performed as the compacted fill is brought to final grade.

3. Fill, consisting of soil approved by the Soils Engineer, shall be placed in compacted layers with approved compaction equipment. Nonorganic on-site materials are considered satisfactory for reuse in the compacted fills. All imported fill shall be examined by the Soils Engineer prior to use in fill areas. Rock larger than six inches in diameter shall not be used in the compacted fills.

4. The fill shall be compacted to at least 90 percent of the maximum laboratory dry density for the material used. The maximum dry density shall be determined by the ASTM D1557-78 compaction method.

5. Observation and field tests shall be performed during grading by the Soils Engineer to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction of less than 90 percent is indicated, additional compactive effort shall be made with adjustment of the moisture content as necessary until 90 percent compaction is obtained.

6. Wherever, in the opinion of the Soils Engineer, or Engineering Geologist, an unstable condition is being created, either by cutting or filling, the work shall not proceed in that area until an investigation has been made and the grading plan revised, as necessary.

EXHIBIT B

Converse Consultants

Geotechnical Engineering
and Applied Sciences

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED COASTAL PROTECTION MEASURES
FOR THE 2± ACRE COASTAL SITE
CARLSBAD, CALIFORNIA

Conducted For:

R. M. NOBLE & ASSOCIATES
22235 Pacific Coast Highway
Malibu, California 90265

These orig are all included in the copy set. Use to verify copies are complete when scanning 7/6/10

Project No. 83-2299-02
September 20, 1984

Converse Consultants, Inc.
2855 Pullman Street
Santa Ana, California 92705
Telephone (714) 261-2414

PLAN CHECK NO. 1 60667

Converse Consultants

Geotechnical Engineering
and Applied Sciences

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION PROPOSED COASTAL PROTECTION MEASURES FOR THE 2± ACRE COASTAL SITE CARLSBAD, CALIFORNIA

Conducted For:

R. M. NOBLE & ASSOCIATES
22235 Pacific Coast Highway
Malibu, California 90265

Project No. 83-2299-02
September 20, 1984

Converse Consultants, Inc.
2855 Pullman Street
Santa Ana, California 92705
Telephone (714) 261-2414

Converse Consultants

Geotechnical Engineering
and Applied Sciences

September 20, 1984

R. M. Noble & Associates
22235 Pacific Coast Highway
Malibu, California 90265

Attention: Mr. Ron Noble

Subject: Supplemental Geotechnical Investigation
2± Acre Coastal Site, Carlsbad, California
(CCI Project No. 83-2299-02)

Gentlemen:

This report presents the results of our supplemental geotechnical investigation for the 2± acre site located west of Carlsbad Boulevard, south of Shore Drive, and northwest of Cerezo Drive in Carlsbad. This property is adjacent to the Pacific Ocean between an existing residential area on the north side and the La Costa State Beach on the south side. This work was conducted in accordance with your oral authorization on August 10, 1984, and our confirmation letter dated 22 August 1984.

Thank you for this opportunity of working with you on this project. Please do not hesitate to call if we can help you further.

Yours very truly,

CONVERSE CONSULTANTS, INC.

Dennis L. Hannan, C.E.G. 953
Managing Vice President

RWR/JBR/MEB/DLH:bl

Dist: (5) Addressee

Converse Consultants, Inc.
2855 Pullman Street
Santa Ana, California 92705
Telephone (714) 261-2414

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CLOSURE	8
DRAWING 1 - Geologic Map	Follows Text
DRAWING 2 - Geologic Sections	Follows Text
FIGURE 1, Proposed Retaining Wall - Typical Detail	Follows Text
APPENDIX A - Field Exploration	Follows Text

INTRODUCTION

Background Information and Purpose

In our geotechnical investigation dated February 1, 1984 (CCI Project No. 83-2299-01), we recommended the use of a stone revetment within the "cove" area for protection of this segment of the coast. We understand that the Coastal Commission suggested that you design an alternate structure (i.e., retaining wall) for this area in order to minimize encroachment onto the beach area.

It is proposed to construct a concrete retaining wall to protect the unconsolidated sediments against sea wave erosion, and to support a 2:1 (horizontal:vertical) slope. The top of the wall will be at about elevation +24 feet, and the base at elevation zero. Tiebacks anchored in the bedrock will be used for lateral earth pressure support at the top of the wall and cast-in-place concrete piles (caissons) will support the wall and resist lateral earth pressure at the base of the wall (see Figure 1, following text).

The purposes of our supplemental geotechnical investigation were to obtain additional information relative to the geologic and subsurface conditions within the "cove" area, to provide geotechnical engineering design parameters, and to assist you in alternate design schemes for coastal protection.

SCOPE OF SERVICES

Our scope of services performed included:

- Drilling and logging of 3 borings ranging in depth from 7.5 to 48.5 feet;
- Laboratory observation and testing of selected samples, including tests for in-place moisture and density, and shear strength;
- Geologic and engineering evaluations and analyses based on the data obtained;

- Coordination with R.M. Noble & Associates during the various stages of design; and
- Preparation of this report, which included field and laboratory test data, drawings, and recommendations for design and construction.

FIELD EXPLORATION

This additional field exploration included the drilling of three borings ranging between 7.5 and 48.5 feet in depth. One boring was drilled using a 6-inch diameter auger provided by Geodrill from San Diego on August 20, 1984. The other borings were drilled using 8-inch diameter hollow stem drill rigs provided by California Testing from Long Beach on September 1, 1984, and by Geodrill on September 5, 1984.

Driven and relatively undisturbed soil samples were obtained in the borings. To obtain samples, the sampler (2.5 inch inside diameter, 3 inch outside diameter), lined with sample rings, was driven into the bottom of the borehole with successive drops of a 140-lb hammer falling 30 inches. Standard penetration tests (SPT) were also taken to evaluate the relative density of the materials.

During drilling, the soils were continuously logged by a geologist from our office, and visually classified essentially in accordance with the Unified Soil Classification System. The central portion of each driven sample was retained in water-proof plastic containers for shipment to the laboratory.

Summaries of the soil conditions encountered are presented in the Boring Summary Sheets in the Appendix. These summaries include the SPT blow count for one foot of sample penetration, moisture content, dry density, shear resistance, and other pertinent field and laboratory data. The boring logs show subsurface conditions at the date and location indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

LABORATORY TESTING

Laboratory tests were conducted on representative soil samples for the purpose of classification and for determining the material engineering properties. Moisture content, density, shear, grain-size, and corrosivity tests were performed.

Moisture content and dry density tests were performed essentially in accordance with ASTM Test Method D2216 to provide qualitative information regarding soil strength and compressibility. Quick direct shear tests were performed on relatively undisturbed samples. The samples were kept at field moisture content and sheared under a constant rate of strain of 0.05 inch/minute under normal loads ranging between 0.5 and 2.5 kips/sq. ft.

The pH, resistivity, and the soluble sulfate and chloride concentrations were determined on one soil sample to evaluate the corrosion potential of common construction material in contact with soil. Anaheim Test Laboratory of Santa Ana performed the tests using essentially the Caltrans procedures: Resistivity - Caltrans 643; Sulfates - Caltrans 417; and Chlorides - Caltrans 422.

The results of moisture content, density, and shear tests are presented on the Boring Summary Sheets, and the grain-size test results are in the Appendix. The soil samples presently stored in our laboratory will be discarded 60 days after the date of this report unless a request to retain the samples for a longer period is received.

EXISTING SURFACE CONDITIONS

For general conditions of the site, please refer to our geotechnical investigation of the site dated February 1, 1984 (CCI Project No. 83-2299-01). During the period from this initial investigation (December 1983) up to the conclusion of our supplemental investigation conducted between August 20 and September 5, 1984, surface conditions have essentially remained the same except within the "cove" area. Fill material had been placed within this "cove" area apparently as a means to retard further cliff deterioration.

At the time of our first visit to the site on 20 August 1984, the 25-foot wide fill platform had been eroded back by approximately 5-7 feet in isolated areas. Tension cracks, indicating potential failure, were also recognized at various locations along the remaining fill platform. It was decided at this time that a smaller "skid" rig would have to be used to drill holes on this fill surface, as it would be too dangerous for a larger rig to gain access. One hole was drilled to a depth of 45 feet within the fenced area at the top of the cliff. A smaller "skid" rig was scheduled for a latter date (September 1, 1984).

On 1 September 1984, we discovered that the fill platform had been eroded even further back, as much as 6-7 feet in one area, making access for drilling impossible. An attempt was made to drill at the base of the moderately eroded access road near the remnant of the eroded fill platform. Due to numerous large cobbles encountered and the danger of losing the drill rig, the hole was terminated at 7.5 feet. With the failure of this rig to obtain samples for testing and with no possibility of drilling on the old fill platform, we drilled one more hole on September 5, 1984 above the "cove" area at the closest point we could near the cliff edge. Adequate samples were obtained at this time for completion of our field work.

SUBSURFACE CONDITIONS

For general subsurface conditions at the site, please refer to the Geology section of our Geotechnical Investation report dated February 1, 1984 (CCI Project No. 83-2299-01). Our latest investigation, consisting of three borings, revealed that bedrock underlies the terrace deposits in the "cove" area and the channel deposits are confined to a limited area near the base of the bluff and below the beach deposits (see Section B-B' of Drawing 2).

Erosion of major portions of the fill has exposed sections of the sandstone bedrock platform which previously could only be seen at the northwest and southeast corners of the site. Bedrock samples obtained from the "cove" area are typical of the sandstone observed elsewhere at the site, with the exception of its apparent lack of cementation. This could explain, in part, the more severe erosion which has taken place in this area.

Ground water was encountered at depths of 39 feet in Boring D-3 and 38.5 feet in Boring D-5, or at about elevations +3 feet and +1.5 feet, respectively. These levels may vary somewhat depending on time of year and in response to tidal fluctuations.

CONCLUSIONS

Based on the results of our soil exploration, laboratory testing, and engineering analyses, it is our conclusion that the wall may be constructed as planned. All loose fill material at the base and behind the proposed retaining wall should be removed prior to the wall erection. The ground surface should be properly benched and compacted to provide a suitable surface to place the fill. Specific recommendations for retaining walls, tiebacks, and pile design follow.

Retaining Wall

We recommend that the proposed retaining wall in the "Cove" area be designed for an active earth pressure of 50 psf/ft of depth below the top of the wall supporting a 2:1 (horizontal to vertical) sloping backfill. For portions of the wall supporting intact bedrock, a minimum active earth pressure of 35 psf/ft below the top of wall may be used in conformance with UBC code requirements. The recommendations presented in our February 1, 1984 report for drainage design, material, and backfill are applicable also to this wall (see Figure 1, attached).

Tiebacks

The earth lateral pressure against the retaining wall will be resisted by tiebacks and pile lateral resistance. The tiebacks, which would be on the order of 50 to 60 feet long, would be embedded in the bedrock.

We recommend that the tieback inclination be at least 10 degrees from horizontal because of the poorly cemented and weak bedrock zones; the tieback angle should not exceed 25 degrees. The first 25 feet (unbonded length) of the tieback should not be considered for the calculation of shear resistance. The skin friction resistance of the bedrock for estimating the tieback length and design load is 2000 psf. The tieback capacities must be based on load tests. The minimum spacing recommended between tiebacks is 8 feet, and the maximum allowable load is 200 kips per tieback.

All tiebacks should be tested to at least 2 times their design loads. Because of the corrosive environment of the project, tiebacks should be well protected against corrosion. To minimize caving of the poorly cemented bedrock zone during construction, it is advisable to keep the tieback hole diameter smaller than 20 inches.

Pile Design For Retaining Wall

Drilled, cast-in-place, concrete friction piles may be used for the support of the retaining wall. The recommendations for spacing, skin friction value, and lateral pile design criteria presented in our February report are applicable for this portion of the wall. The portion of the piles above elevation -5 (five feet below sea level) should not be considered for vertical and lateral load supported in light of possible scouring. The piles should be embedded at least 25 feet below sea level. To minimize the possible scouring affects near the base of the wall, where less than 10 feet (horizontal distance) of bedrock will be exposed between the wall and beach deposits, buried stone protection is recommended (see Figure 1).

Construction Limitations

The flat area at the base of the cliff is relatively small for large construction equipment. It is expected that the contractor will need to build a temporary construction platform/fill to support the equipment.

During our foundation exploration, we experienced "caved-in" or "belling" conditions in soft or poorly cemented zones of the bedrock encountered in the borings. Similar cavings should be expected in some of the pile and anchor borings during construction. Temporary casing may be required to keep some of the pile borings open during placement of concrete.

Any large excavation, particularly for installation of armor stone, should be expected to encounter ground water below about elevation +2 feet. Dewatering and temporary ground support may be needed during construction.

Soil Corrosivity

One sample from Boring 3 at a depth of 39 feet was tested for corrosivity potential. The test indicated a sulfate concentration of 170 ppm, a chloride concentration of 95 ppm, a pH of 7.5, and a minimum electrical resistivity of 5100 ohm/cm.

Soils with sulfate concentrations less than 1000 ppm are generally reported to have a low corrosive effect on foundation concrete. Soils with less than 200 ppm of chloride generally have been observed to have a low corrosive effect on mild grade steel exposed to the subsoil. The results of the resistivity test indicated that the soils may induce moderate corrosion to metallic pipes.

CLOSURE

This report should be read in conjunction with our previous geotechnical investigation report dated February 1, 1984 (CCI Project No. 84-2299-01). The findings of this report are based upon our evaluation and interpretation of the subsurface conditions encountered during both explorations and the results of this and the previous laboratory testing program. The soil/bedrock conditions have been determined at the specific test hole locations only. Conditions between or beyond the borings may vary, and interpolation or extrapolation of the results may not be warranted. In our opinion, the exploration performed has yielded information sufficient for design of the proposed construction.

The findings and recommendations of this report were prepared in accordance with generally accepted professional principles and practice in geotechnical engineering. We make no other warranty, either express or

implied. It is assumed that recommended soil and foundation construction observation will be provided. If conditions are encountered during construction that appear to be different that these indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

Respectfully submitted,

CONVERSE CONSULTANTS, INC.

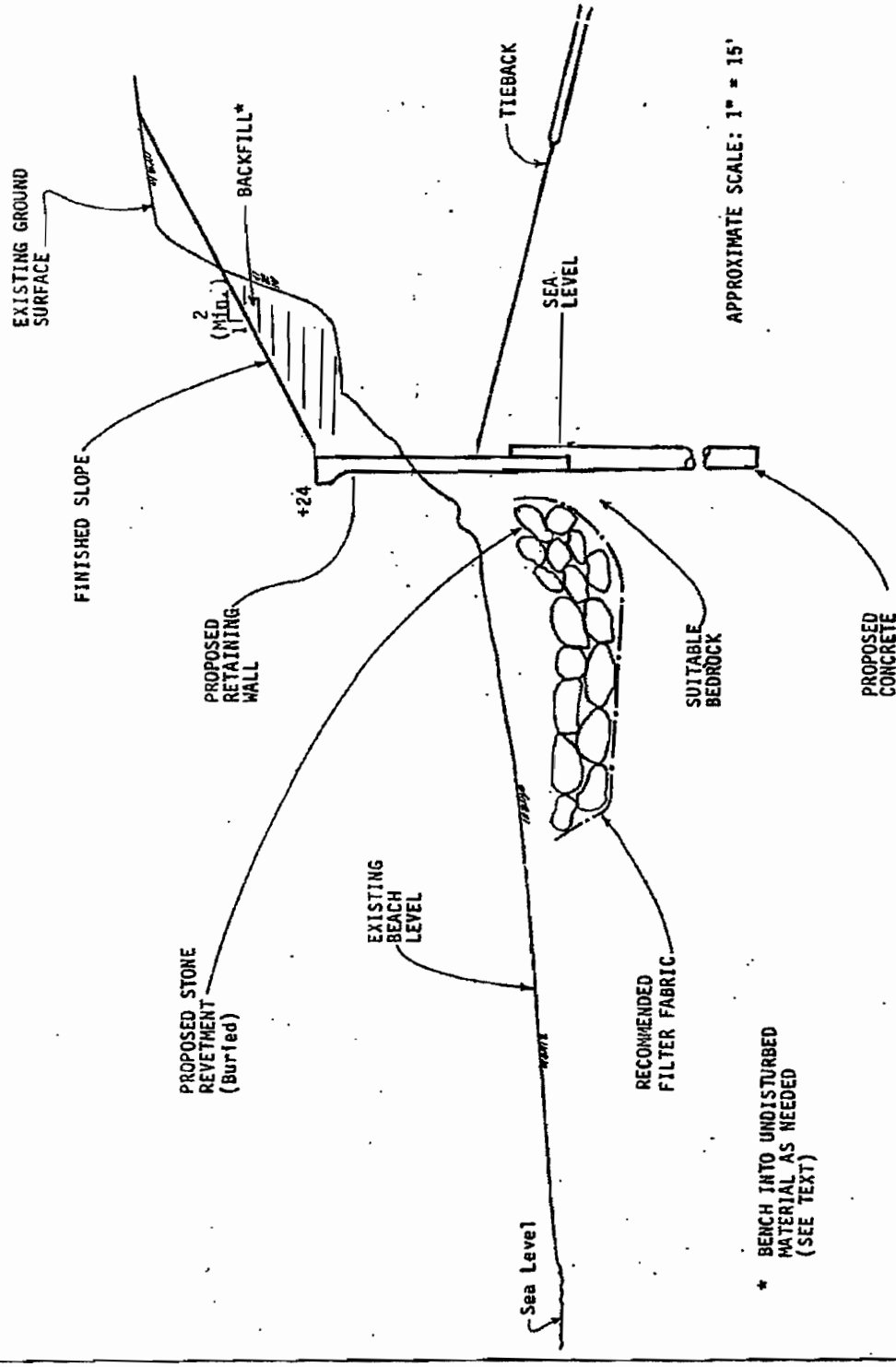
Jacques Roy

Jacques B. Roy, R.C.E. 34386
Senior Engineer

Mark E. Bryant

Mark E. Bryant, C.E.G. 1046
Senior Geologist

JBR/RWR/MEB:bl



PROPOSED RETAINING WALL - TYPICAL DETAIL

2± ACRE COASTAL SITE
Near Cerezo Drive and Carlsbad Blvd.
Carlsbad, California

Project No
83-2299-02

Figure No.
1

Converse Consultants Geotechnical Engineering and Applied Sciences

APPENDIX

FIELD EXPLORATION
GRAIN-SIZE DISTRIBUTION CHART

DEGREE OF WEATHERING

DIAGNOSTIC FEATURE

DESCRIPTIVE TERM	DISCOLORATION EXTENT	FRACTURE CONDITION	SURFACE CHARACTERISTICS	ORIGINAL TEXTURE	GRAIN BOUNDARY CONDITION
UNWEATHERED	NONE	CLOSED OR DISCOLORED	UNCHANGED	PRESERVED	TIGHT
SLIGHTLY WEATHERED	LESS THAN 20% OF FRACTURE SPACING ON BOTH SIDES OF FRACTURE.	DISCOLORED, MAY CONTAIN THIN FILLING	PARTIAL DISCOLORATION	PRESERVED	TIGHT
MEDIUM WEATHERED	GREATER THAN 20% OF FRACTURE SPACING ON BOTH SIDES OF FRACTURE.	DISCOLORED, MAY CONTAIN THICK FILLING	PARTIAL TO COMPLETE DISCOLORATION, NOT FRIABLE EXCEPT POORLY CEMENTED ROCKS.	PRESERVED	PARTIAL OPENING
HIGHLY WEATHERED	THROUGHOUT	----------	FRIABLE AND POSSIBLY PITTED	MAINLY PRESERVED	PARTIAL SEPARATION
COMPLETELY WEATHERED	THROUGHOUT	----------	RESEMBLES A SOIL	PARTLY PRESERVED	COMPLETE SEPARATION

DISCONTINUITY SPACING

DESCRIPTION FOR STRUCTURAL FEATURES: BEDDING, FOLIATION, OR FLOW BANDING	SPACING		DESCRIPTION FOR JOINTS, FAULTS OR OTHER FRACTURES
VERY THICKLY (BEDDED, FOLIATED OR BANDED)	MORE THAN 2 METERS	MORE THAN 6 FEET	VERY WIDELY (FRACTURED OR JOINTED)
THICKLY	60 CM - 2 METERS	2 - 6 FEET	WIDELY
MEDIUM	200 MM - 60 CM	8 - 24 INCHES	MEDIUM
THINLY	60 - 200 MM	2½ - 8 INCHES	CLOSELY
VERY THINLY	20 - 60 MM	3/4 - 2½ INCHES	VERY CLOSELY
DESCRIPTION FOR MICROSTRUCTURAL FEATURES: LAMINATION, FOLIATIONS, OR CLEAVAGE			
INTENSELY (LAMINATED, FOLIATED OR CLEAVED)	6 - 20 MM	1/4 - 3/4 INCH	EXTREMELY CLOSE
VERY INTENSELY	LESS THAN 6 MM	LESS THAN 1/4 INCH	

SEPARATION OF FRACTURE WALLS

DESCRIPTION	SEPARATION OF WALLS IN MM
CLOSED	0
VERY NARROW	0 - 0.1
NARROW	0.1 - 1
WIDE	1 - 5
VERY WIDE	5 - 25+

FRACTURE FILLING

DESCRIPTION	DEFINITION
CLEAN	NO FRACTURE FILLING MATERIAL
STAINED	DISCOLORATION OF ROCK ONLY. NO RECOGNIZABLE FILLING MATERIAL.
FILLED	FRACTURE FILLED WITH RECOGNIZABLE FILLING MATERIAL.

SURFACE ROUGHNESS

CLASSIFICATION	DESCRIPTION
SMOOTH	APPEARS SMOOTH AND IS ESSENTIALLY SMOOTH TO THE TOUCH. MAY BE SLICKENSIDED. *
SLIGHTLY ROUGH	ASPERITIES ON THE FRACTURE SURFACES ARE VISIBLE AND CAN BE DISTINCTLY FELT
MEDIUM ROUGH	ASPERITIES ARE CLEARLY VISIBLE AND FRACTURE SURFACE FEELS ABRASIVE TO TOUCH.
ROUGH	LARGE ANGULAR ASPERITIES CAN BE SEEN. SOME RIDGE AND HIGH-SIDE ANGLE STEPS EVIDENT.
VERY ROUGH	NEAR VERTICAL STEPS AND RIDGES OCCUR ON THE FRACTURE SURFACE.

* WHERE SLICKENSIDES ARE OBSERVED, THE DIRECTION OF THE SLICKENSIDES SHOULD BE RECORDED AFTER THE STANDARD DISCONTINUITY SURFACE DESCRIPTION.

ROCK HARDNESS

CLASSIFICATION	FIELD TEST
VERY SOFT	CAN BE PEELED WITH A KNIFE. MATERIAL CRUMBLES UNDER FIRM BLOWS WITH THE SHARP END OF A GEOLOGIC PICK.
SOFT	CAN JUST BE SCRAPED WITH A KNIFE. INDENTATIONS OF 2 TO 4 MM WITH FIRM BLOWS OF THE PICK POINT.
MEDIUM HARD	CANNOT BE SCRAPED OR PEELED WITH A KNIFE POINT. HAND HELD SPECIMEN BREAKS WITH FIRM BLOWS OF THE PICK.
HARD	DIFFICULT TO SCRATCH WITH KNIFE POINT. CANNOT BREAK HAND HELD SPECIMEN.

TERMS USED FOR THE PHYSICAL DESCRIPTION OF BEDROCK

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No.
83-2299-02

Converse Consultants Geotechnical Engineering and Applied Sciences

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KEY TO SOIL SYMBOLS AND TERMS

TERMS USED IN THIS REPORT FOR DESCRIBING SOILS ACCORDING TO THEIR TEXTURE OR GRAIN SIZE DISTRIBUTIONS ARE GENERALLY IN ACCORDANCE WITH THE UNIFIED SOIL CLASSIFICATION SYSTEM.

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sands, little or no fines
		Sands with fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
	Highly organic soils		Pt	Peat and other highly organic soils

TERMS DESCRIBING CONDITION, CONSISTENCY AND HARDNESS

COARSE GRAINED SOILS (major portion retained on No. 200 sieve): Includes (1) clean gravels, (2) silty or clayey gravels, and (3) silty, clayey or gravelly sands. Consistency is rated according to relative density, as determined by laboratory tests.

Descriptive Term	Relative Density
Very loose	0 to 15%
Loose	15 to 40%
Medium dense	40 to 70%
Dense	70 to 85%
Very dense	85 to 100%

FINE GRAINED SOILS (major portion passing No. 200 sieve): Includes (1) inorganic and organic silts and clays, (2) gravelly, sandy or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by direct shear tests.

Descriptive Term	Shear Strength (tsf)
Very soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very stiff	2.00 to 4.00
Hard	4.00 and higher

ROCK: Includes gravels, cobbles, rock, caliche and bedrock materials. Hardness is related to field identification procedures described below.

Descriptive Term	Field Identification Test
Soft	Can be dug by hand and crushed by fingers
Moderately hard	Friable, can be gouged deeply with knife and will crumble readily under light hammer blows
Hard	Knife scratch leaves dust trace, will withstand a few hammer blows before breaking
Very hard	Scratched with knife with difficulty, difficult to break with hammer blows

SOIL MOISTURE

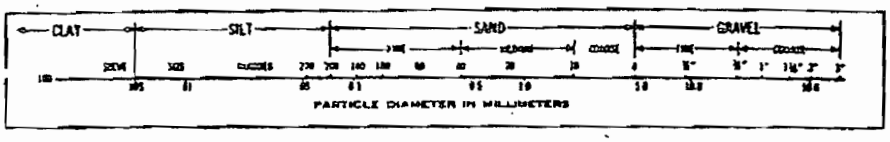
From low to high the soil moisture is indicated by:

Dry
Slightly moist
Moist
Very moist
Wet

SIZE PROPORTIONS

Designation	Percent by Weight
Trace	0 to 10
Little	10 to 20
Some	20 to 35
And	35 to 50

GRAIN SIZE DISTRIBUTION



SUMMARY
BORING NO. D-3

DATE DRILLED: 8-20-84

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY OFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

ELEVATION: 42'±

DEPTH IN FEET	SAMPLES	SYMBOL					DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.
0		SM	slightly moist	medium dense	dark yellow brown	TERRACE DEPOSITS SILTY SAND fine to medium slightly porous slightly cemented				
5	1						22.9	4.0	109	0.48 0.61** 0.95
		SW/SP			light olive brown to dark yellow brown	SAND fine to medium trace silt slightly cemented				
10	2						22.1	3.4	102	0.38 0.61** 0.99
					light olive brown	fine-medium-coarse				
15	1*					lack of cementation	30*			
20	3	SW/GW		medium dense to dense	light yellow brown to pale olive	SAND & SANDY GRAVEL fine to coarse sand with cobbles (3"-5") occasional trace silt lack of cementation	28.5	2.0	103	0.81 0.96* 1.37
25	2*	SW				SAND fine to coarse	46*			

* Standard Penetration Test (SPT) Resistance in blows/ft.

(Continued)

** Sample soaked before shearing at 0.5, 1.0 and 1.5 ksf.

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No.
83-2299-02

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Approved for publication _____ by _____

SUMMARY
BORING NO. D-3 (Continued)

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLES	SYMBOL					DRIVE ENERGY FT.-KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.
25		SW	slightly moist to moist	medium dense to dense	light yellow brown to pale olive	TERRACE DEPOSITS SAND fine to coarse lack of cementation trace to little gravel				
30	4						22.2	2.1	111	0.36 0.79* 1.17
			moist	soft	olive gray to light olive gray	BEDROCK SILTY SANDSTONE & CLAYEY SANDSTONE fine to coarse sand few cleaner sandstone layers				
35	3*						70*			
					yellow brown to pale olive	SANDSTONE fine-medium-coarse trace to little silt				
			very moist			with sandy claystone interlayers				
40	5		wet very moist							0.96 1.29 1.59* 2.44
	4*					CLAYSTONE	40*			
45			ground water encountered at 39'			end of boring at 45'				
50										

* Standard Penetration Test (SPT) Resistance in blows/ft. ** Sample soaked before shearing at 0.5, 1.0 and 1.5 ksf. *** Sample soaked before shearing at 0.5, 1.0, 1.5 and 2.5 ksf.

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No.
83-2299-02

Converse Consultants

Approved for publication _____ by _____

Bond NO. _____

SUMMARY
BORING NO. D-5 (Continued)

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

DEPTH IN FEET	SAMPLES	SYMBOL					DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.
25		SW	slightly moist	dense to very dense	pale olive	TERRACE DEPOSITS SAND fine to coarse very intensely laminated				
30	1*							40*		
			moist	very soft	olive to light olive gray to olive	BEDROCK CLAYEY SANDSTONE fine to medium some orange staining				
	2*							20*		
35						SILTY SANDSTONE & SANDSTONE with thin sandy claystone interlayers fine-medium-coarse some orange staining				
	1		to very moist					38.6		
▽			wet							
	3*							12*		
40				dense						
			very moist	very soft	light olive brown	CLAYSTONE				
	4*		wet	soft	yellow brown to orange brown	SANDSTONE fine to coarse friable		52*		
45										
	2							35.1		
					light olive	fine-medium-coarse				
			ground water encountered at 38.5'			end of boring at 48.5'				
50										

* Standard Penetration Test (SPT) Resistance in blows/ft.

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No. 83-2299-02

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FORM NO 17 Approved for publication _____ by _____

SUMMARY
BORING NO. B-5

DATE DRILLED: 9-5-84

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

ELEVATION: 40'±

DEPTH IN FEET	SAMPLES	SYMBOL					DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.
0			slightly moist	medium dense	dark yellow brown	**TERRACE CEPOSITS** SILTY SAND fine to medium porous slightly cemented				
5						increase in sand				
10		SW			olive to light olive brown	SAND fine to meidum some coarse grains				
15										
20		SW/GW		medium dense to dense	light olive brown to yellow brown	SANDY GRAVEL & SAND with some cobbles				
25		SW		dense to v. dense	pale olive	SAND fine to coarse very intensely laminated				

(Continued)

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No.
83-2299-02

Converse Consultants

Approved for publication _____ by _____

SUMMARY
BORING NO. D-4

DATE DRILLED: 9-1-84

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF ACTUAL CONDITIONS ENCOUNTERED.

ELEVATION: 23'±

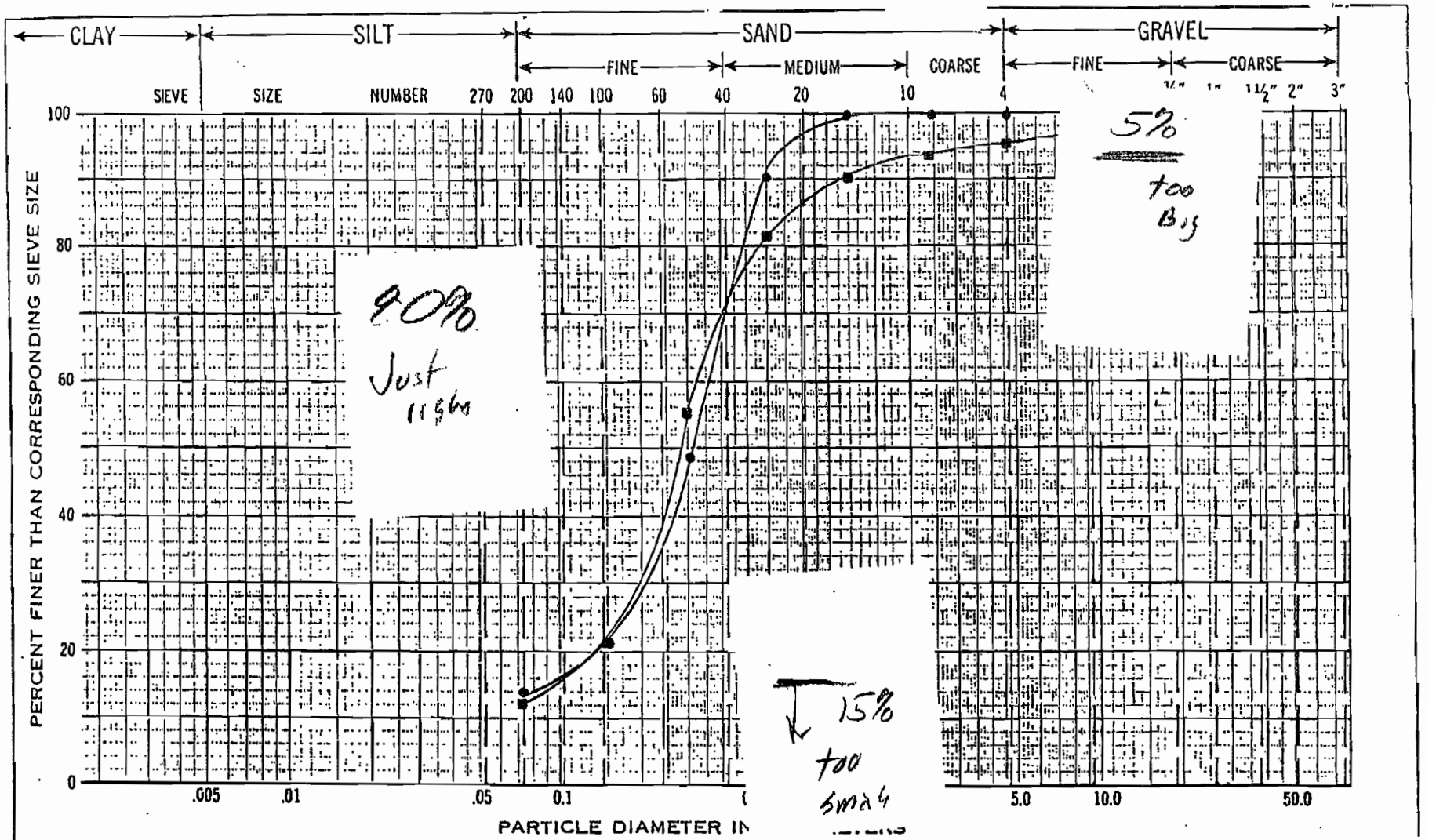
DEPTH IN FEET	SAMPLES	SYMBOL					DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.
0		SW	slightly moist	medium dense	light yellow brown	TERRACE DEPOSITS SAND fine to medium little coarse				
5	1							10.6		
						gravel & some cobbles				
			no ground water encountered			end of boring at 7.5' Refusal on large cobbles.				
10										
15										
20										
25										

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No.
83-2299-02

Converse Consultants

FORM N 111 Approved for publication _____ by _____



● BORING 3, SAMPLE 2

■ BORING 3, SAMPLE 4

GRAIN-SIZE DISTRIBUTION CHART

2± ACRE COASTAL SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Project No.
83-2299-02

Converse Consultants Geotechnical Engineering and Applied Sciences

001009

LEGAL DESCRIPTION:

THAT PORTION OF LOT "H" OF RANCHO AGUA HEDIONDA, IN THE CITY OF CARLSBAD, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO PARTITION MAP THEREOF NO. 1823, FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, NOVEMBER 16, 1896, TOGETHER WITH LOT 1 OF POINSETTIA BEACH UNIT NO. 1, IN THE CITY OF CARLSBAD, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO MAP THEREOF NO. 3897, FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY, MAY 26, 1958.

BASIS OF BEARINGS:

THE BASIS OF BEARINGS FOR THIS PLAT IS THE WESTERLY RIGHT-OF-WAY LINE OF CARLSBAD BOULEVARD AS SHOWN ON MAP NO. 2696, I.E. S30°38'50"E

THIS ADJUSTMENT PLAT WAS PREPARED BY ME OR UNDER MY DIRECTION.

P.N. O'Day 3-10-87

P.N. O'DAY R.C.E. 27214 DATE
O'DAY CONSULTANTS
7750-2H EL CAMINO REAL
CARLSBAD, CA 92008
PHONE: (619) 944-1010

REGISTERED PROFESSIONAL ENGINEER
PATRICK N. O'DAY
No. 27214
EXP. 3/31/87
CIVIL
STATE OF CALIFORNIA

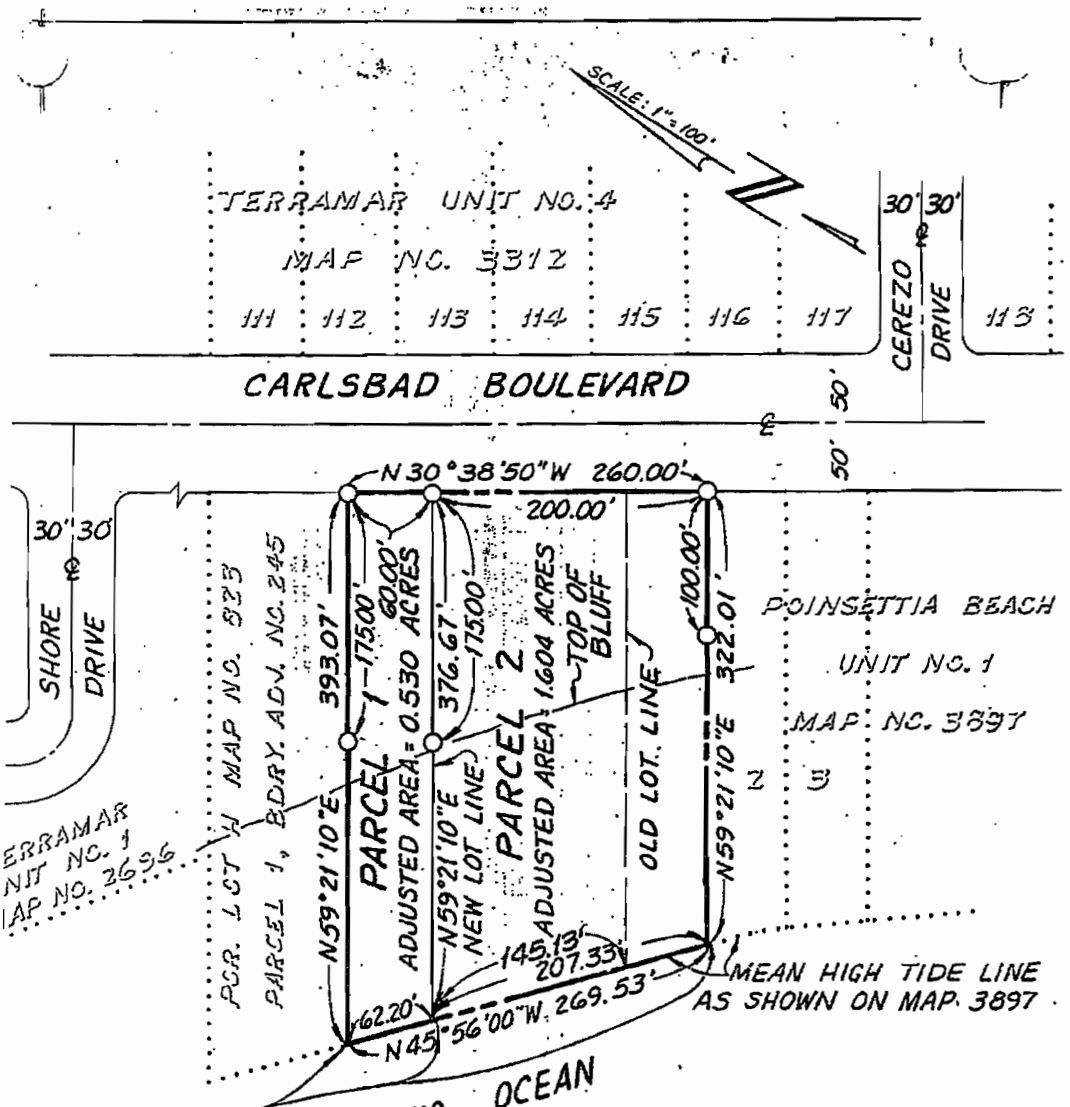
OWNER/APPLICANT:

PAUL ECKE, SR., TRUSTEE
C/O CHRISTOPHER CALKINS
4401 MANCHESTER AVE., SUITE 206
ENCINITAS, CA 92024
PHONE: (619)944-4090

Paul Ecke Sr. March 6, 1987

PAUL ECKE, SR. DATE

ADJUSTMENT PLAT CITY OF CARLSBAD			SHT. 1 OF 2 SHTS.
REQUESTED BY: PAUL ECKE, SR. CHRISTOPHER CALKINS	PREPARED BY: P.N. O'Day	APPROVED BY: Lloyd Rudolph	A.P.N. 210-120-01 E 210-120-7A



REGISTERED PROFESSIONAL ENGINEER
PATRICK N. O'DAY
No. 27214
CIVIL
STATE OF CALIFORNIA

ADJUSTMENT PLAT NO. 342
CITY OF CARLSBAD

SHT. 2 OF 2 SHTS.

REQUESTED BY:
PAUL ECKE, SR.
JO CHRISTOPHER CALKINS

PREPARED BY: P.N. O'Day

APPROVED BY: Lloyd Fulks

A.P.N. 210-120-01 & 210-120-28

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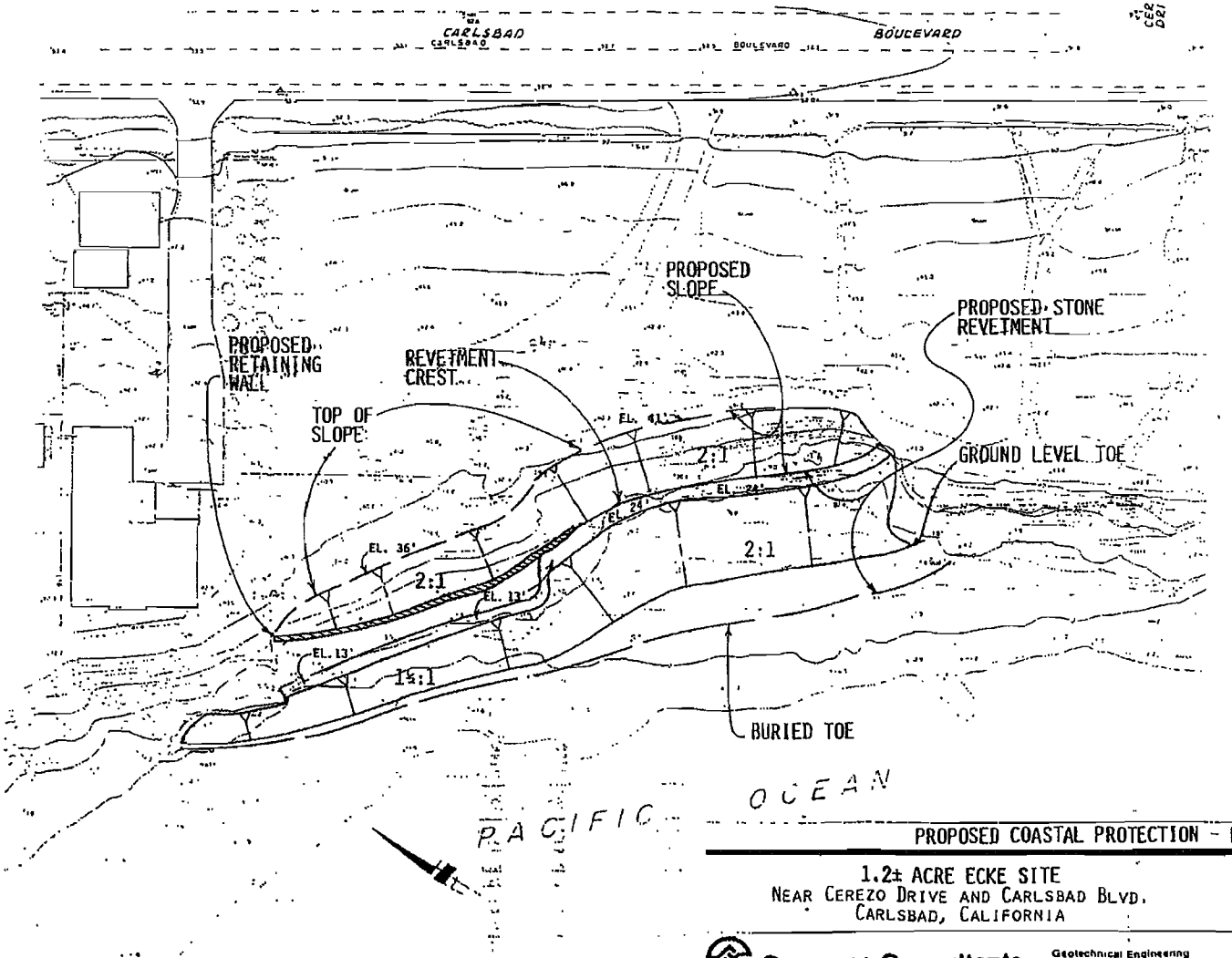


NOTES:

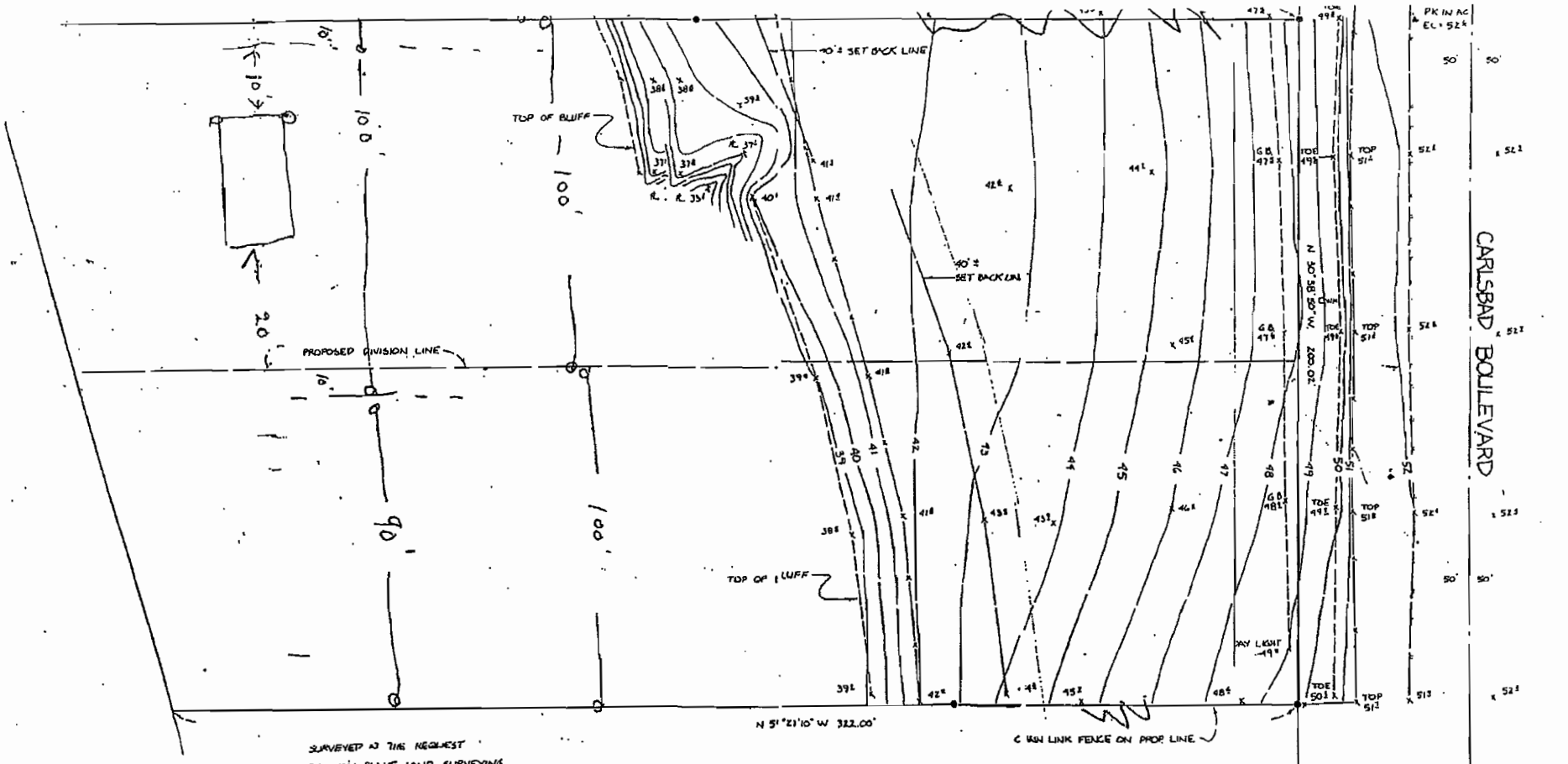
1. PROPOSED COASTAL PROTECTION BASED ON PRELIMINARY DESIGN LAYOUT PREPARED BY R. M. NOBLE & ASSOCIATES.
2. REFER TO DRAWINGS 4 AND 5 FOR TYPICAL SECTIONS AND DETAILS.
3. THIS DRAWING IS PART OF CONVERSE CONSULTANTS REPORT NO. 83-02299-01 AND SHOULD BE READ TOGETHER WITH THE REPORT FOR COMPLETE EVALUATION.

PROPOSED COASTAL PROTECTION - PLAN VIEW

1.2± ACRE ECKE SITE
NEAR CEREZO DRIVE AND CARLSBAD BLVD.
CARLSBAD, CALIFORNIA

Converse Consultants — Geotechnical Engineering and Applied Sciences

Scale	1" = 50'
Date	2-1-84
Prepared by	KLF
Checked by	MEB
Approved by	MEB
Project No	83-02299-01
Drawing No	3



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Jon Blake 3/7/91
JON BLAKE L 4368 EXP. 9/30/93

L 91-659

EXHIBIT C

GEOTECHNICAL INVESTIGATION
AND BLUFF RETREAT STUDY
PARCEL NO. 210-120-30
CARLSBAD, CALIFORNIA

PREPARED FOR

M.B. SHORES CORPORATION
P.O. BOX 2571
LEUCADIA, CALIFORNIA 92024

PREPARED BY

ICG INCORPORATED
9240 TRADE PLACE, SUITE 100
SAN DIEGO, CALIFORNIA 92126

MARCH 28, 1991

JOB NO. 04-8529-001-00-00
LOG NO. 1-1418

GEOTECHNICAL INVESTIGATION
AND BLUFF RETREAT STUDY
PARCEL NO. 210-120-30
CARLSBAD, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation of the subject site. The purpose of our investigation was to provide geologic and soils engineering data as they relate to site development, evaluate sea bluff stability, and to recommend a structural set-back from the ocean bluff along the west side of the site.

For use in our investigation, we were provided with a 20-scale topographic map of the site prepared by Jon Blake Land Surveying, dated March 7, 1991. This served as the base for our Geotechnical Map, Plate 1. Because this most recent survey did not include the lower bluff and beach, another 20-scale topographic map was obtained for use as a base for our geologic mapping of the site. This older map (Plate 2) was prepared by Robert Bein, William Frost and Associates, and is dated December 5, 1983.

1.1 Authorization

This investigation was conducted in accordance with the authorization of Michael Shores. The scope of services performed was consistent with our proposal number SDP1-5801 dated February 8, 1991.

1.2 Scope of Services

Our scope of services for this investigation included the following:

1.2.1 Review of existing geotechnical reports and literature pertinent to the project area (Appendix A);

1.2.2 Subsurface exploration utilizing a truck-mounted drill rig, with sampling of the typical soil and rock materials encountered;

1.2.3 Geologic mapping of the bluff, at and adjacent to the site;

1.2.4 Laboratory testing of selected samples to evaluate the pertinent engineering characteristics of the prevailing soils;

1.2.5 Evaluation of ground-shaking potential resulting from seismic events occurring on significant faults in the area;

1.2.6 Development of recommendations for site preparation, construction setbacks, and geotechnical criteria for foundation design,

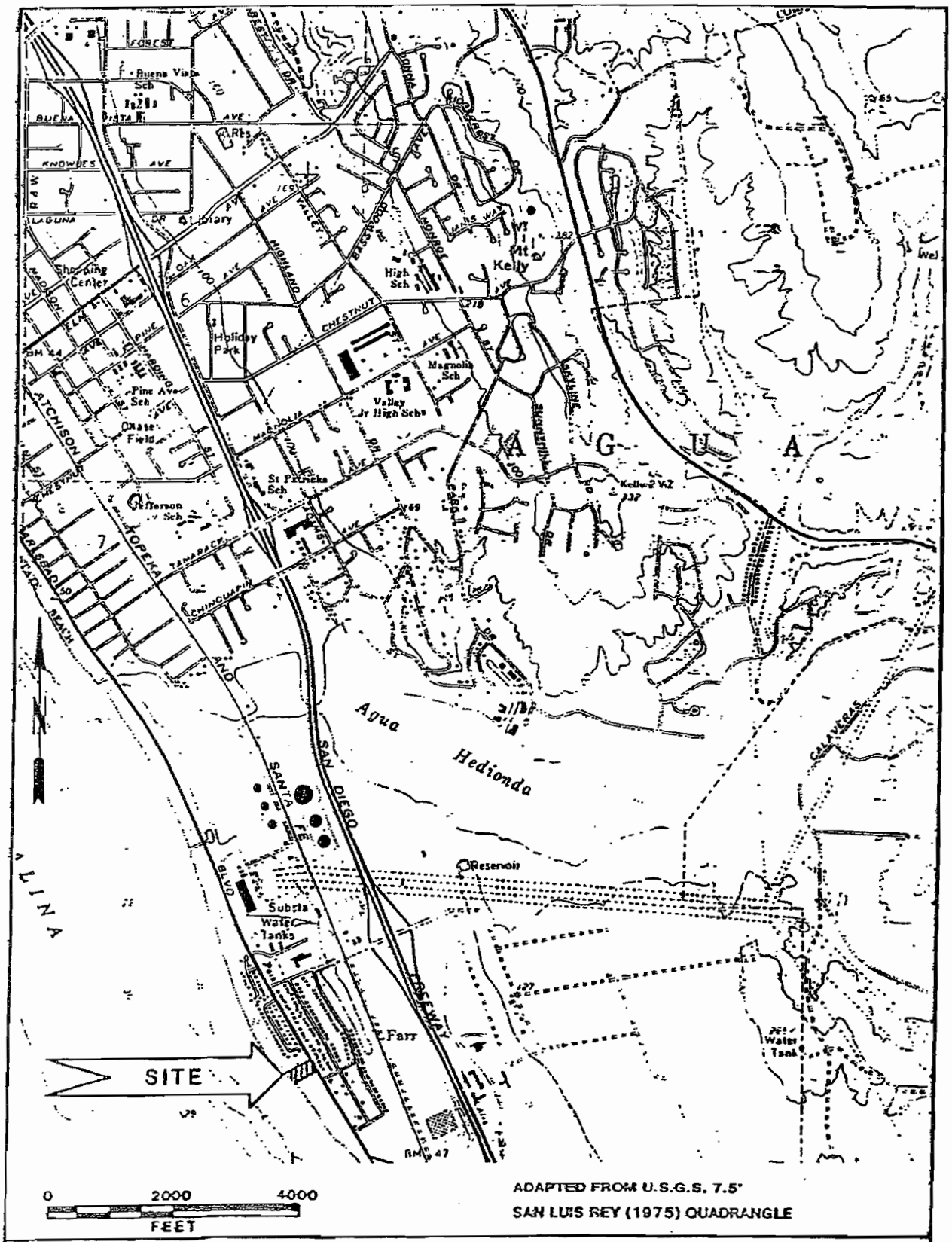
1.2.7 Preparation of this report and the accompanying graphics presenting our findings, conclusions, and recommendations.

2.0 PROPOSED DEVELOPMENT

At the time of this report, specific development plans had not yet been established. It is our understanding that subdivision of the property into two lots and construction of custom, single family homes is being considered. The home(s) will be built using typical wood-frame construction, similar to other houses in the area.

3.0 SITE DESCRIPTION

The project site is located in the Terramar neighborhood of Carlsbad, California, and consists of a 200' wide strip of undeveloped ocean-front property (Figure 1). The site is bounded on the east by the Carlsbad Boulevard right-of-way, on the south by South Carlsbad Beach State Park, and the north by another undeveloped lot. Topographically, it consists of a relatively uniform surface that slopes gently from an elevation of approximately 47 feet, down to 39 feet on the west, where it gives way to a steep-faced bluff descending to the beach. Along the southern portion of the property, the bluff steps inland roughly 25 feet to accommodate a small "cove" along the beach. Drainage across the property is uncontrolled sheet flow towards the beach. Flow over the bluff is concentrated down a beach access path on the northern edge of the cove, and into erosional gullies along the bluff rim above the cove. Vegetation at the site consists of ice plant and low grasses.



LOCATION MAP

JOB NO.:	DATE:	FIGURE:
04-8529-001-00-00	MARCH 1991	1

ICG Incorporated

4.0 SITE INVESTIGATION

4.1 General

Before starting the field work, we reviewed previous reports and available geotechnical literature covering the project area. The resulting information, together with our field exploration, laboratory test results, and previous experience in the area forms the basis for our conclusions and recommendations in this report. The methods used for our work conform to generally accepted standards of practice for geotechnical investigations in southern California.

4.2 Field Exploration

Our field investigation was performed on February 13 and 15, 1991. It began with geologic mapping of the cliff face at the site, and a reconnaissance of the bluffs to the north and south. Five exploratory borings were then drilled in the area between the bluffs and the street to obtain an indication of subsurface conditions across the site.

Borings were drilled with a truck-mounted drill rig, utilizing eight-inch diameter, continuous flight, hollow-stem auger. The borings ranged in depth from approximately 21, to 50.5 feet below existing grade. Samples were obtained using a standard split tube sampler (ASTM D1586-84), a modified California sampler with an inside diameter of 2.5 inches (ASTM D3550-84), and by collecting auger cuttings. Logs describing the subsurface conditions encountered are presented in Appendix B. Lines defining the change between soil or rock types on the boring logs were determined by interpolation between sample locations and are, therefore, approximations. The transition between soil types may be abrupt or gradual.

The approximate boring locations are shown on Plate 1. The borings were located in the field by inspection of the available site map, and pacing; further accuracy of their locations is not implied.

4.3 Laboratory Testing Program

Laboratory tests were performed on selected samples considered to be representative of the foundation soils. Tests were performed in accordance with the methods of ASTM or other accepted standards. Appendix C contains descriptions of the test methods and summaries of the results.

5.0 GEOLOGY

The subject site is situated in the coastal plain section of the Peninsular Ranges geomorphic province. The geologic units which form the bluffs at the specific site consist of Eocene age bedrock of the Santiago Formation, and overlying Quaternary terrace deposits. The unconformable contact between these two units is an irregular wave-abrasion platform cut into the Santiago Formation, on which the terrace deposits accumulated. The elevation of this platform drops from approximately 13.5 feet on the northern portion of the site, to 9 feet or lower in the cove area on the south. Surficial soils mantling these units at the site include a poorly developed sandy topsoil on the plateau above the bluff, talus and slope wash on the face of the bluff, and a mixture of beach sand and cobbles at the base. Scattered old fills are also present. The Geologic Map, Plate 2, shows the approximate distribution of these units. The primary units are described in detail below.

5.1 Geologic Units

5.1.1 Santiago Formation (map symbol Tsa)

Bedrock of the Santiago Formation is the oldest unit exposed in the bluff face and encountered in our borings. As observed at the site the Santiago Formation consists of white to olive gray, fine- to medium-grained, dense to very dense sandstone. The sandstone is generally moderately cemented, and contains some well cemented nodules (concretions) up to several feet in diameter. Internally it is predominantly massive, but also contains some broadly cross bedded channel deposits. These channel deposits are noticeably

more friable than the enclosing massive sands. Also present in the Santiago Formation at the at the site are laterally discontinuous beds and associated rip-up clasts of greenish-gray clayey siltstone.

The massive sandstone of the Santiago Formation is the most erosion resistent material at the site, forming the broad platform at the base of the bluff on either side of the cove. The friable channel deposits included in the bedrock were observed at the base of the cliff along the cove, and were encountered in boring B-2. The presence of these more easily eroded channel deposits, combined with the lower level of the bedrock platform, were probably the primary factors contributing to the formation of the recessed cove area. Two clayey siltstone lenses, up to 6-inches thick, are present along the north side of the cove. These have resulted in the formation of steps in the bedrock platform though erosional undercutting, and providing basal surfaces of weakness for small block failures.

5.1.2 Terrace Deposits (map symbol Qt)

Terrace deposits compose the predominant geologic unit at the site, encountered in all of our exploratory borings and exposed across the upper face of the bluff. They are divided into two subunits based on differing lithologies. As observed along the bluff, the contact between the differing terrace deposits drops gently to the northwest, from an elevation of approximately 18 feet at the southern site boundary, to approximately 16 feet at the northern boundary.

The lower subunit consists of light to dark gray, well stratified, fine- to coarse-grained sand and cobbles. These materials are very friable, and consequently, especially susceptible to erosion.

The upper subunit forms the plateau at the site and consists of fine- to medium-grained silty sand. Near the surface, the sand is stained to a reddish

brown color, is moderately cemented, and very dense. With depth, the reddish brown staining and associated cementation diminishes. Approximately five to ten feet below the surface, the sand is a yellow-gray color and nearly as friable as the lower subunit.

5.1.3 Old Fills (map symbol Fill)

A variety of old fills were observed during our field investigation of the site. Over the eastern portion of the property, up to several feet of dry to damp, loose to medium dense, silty sand was encountered in our exploratory borings. Small mounds of fill were also observed scattered along the bluff face and rim. These are apparently erosional remnants of larger fills placed to infill gullies, and along the bluff face to slow retreat. Some of these fills include large slabs of concrete, asphalt, and other debris.

5.1 Geologic Structure

No faults or other regionally penetrating discontinuities appear to cut the bedrock at the site. Tension cracks in the bedrock platform were observed extending subparallel to, and several feet inward from its outer edge. Spaced sets of near vertical fractures are also present, trending roughly perpendicular to the coastline. Most of the fractures can be observed to die out vertically, or be truncated by siltstone interbeds. These fractures are believed to be localized features related to the stress release produced by unloading of the bedrock through erosion of the overlying terrace deposits.

5.2 Groundwater

At the time of our field reconnaissance, no groundwater seepage was observed along the face of the bluff within the project boundaries. Considerable groundwater seepage was observed along the bluffs to the south of the cove, perched above the Santiago Formation contact, and exiting along fractures in the bedrock. Perched

groundwater was also encountered in our exploratory borings at elevations of between 3 and 6 feet MSL. The level of groundwater at the site will fluctuate with seasonal precipitation, local irrigation, and ocean tides.

6.0 COASTAL EROSION

The southern California coastline can be classified as "submergent", where the land is eroding faster than rivers, longshore deposition, or tectonic uplift can build new land. The rate and manner of coastal bluff degradation is dependent upon a combination of factors, including shoreline and offshore topography, wave, tide, and meteorological conditions, longshore sediment supply, groundwater and surface runoff, and the structure and induration of bluff-forming geologic units.

Studies of coastal erosion along San Diego county indicate that in most areas, bluff retreat is best viewed as an episodic phenomenon (Kuhn et al, 1979). Long periods during which little or no erosion occurs, are sporadically interrupted by catastrophic events where tens of feet of beach and bluff may be lost. This appears to be the pattern of historical bluff retreat at the subject site.

6.1 Bluff Geomorphology

The bluff face along the cove drops off nearly vertically to the beach. A narrow ledge of old fill clings to the cliff face at an elevation of approximately 20 feet, along the southern portion of the cove. The northern portion of the cliff overhangs slightly where the poorly consolidated lower terrace deposits have been eroded back. At the base of the cliff, a wedge of old fill and talus covers a narrow bedrock bench approximately 2 to 4 feet above the level of the beach. The beach inside the cove slopes gently seaward from an elevation of approximately 8 feet, and is composed sand and cobbles. The bluff rim above the cove is dissected by numerous erosional gullies, which extend inward by as much as 8 feet.

North of the cove, a steep slope composed of terrace deposits descends from the bluff rim, down to a bedrock platform at an elevation of roughly 13.5 feet. A fairly continuous talus apron and a few small block topples have accumulated along the toe of the slope. The platform of resistant sandstone extends out from the more easily eroded terrace deposits by as much as 20 feet. At the northern end of the site the bedrock platform drops sharply off to the beach. Rounding the corner into the cove, the bedrock steps down to the beach with benches where two siltstone lenses are present, at approximately 10 and 7.5 feet MSL. Wave action has locally undercut the bedrock platform. The beach elevation here is only about 2 to 3 feet, so that it is completely submerged during high tides.

6.2 Pre-Existing Conditions

To asses the history of bluff retreat at the site, a variety of Aerial photographs and maps were examined (Appendix A), dating back to 1929. The scale of the older photographs and maps are too large to measure distances with great precision. However, they do contain sufficient details and fixed landmarks to suggest that other than the addition of some man made fills, relatively little change in the bluffs occurred until 1977.

Converse Consultants (February, 1984) discusses observations made by Gerald Kuhn, a Research Associate at Scripps Institute of Oceanography. Up until 1977, the bluff face along both the cove and the area to the north consisted of a moderately steep slope descending to a sandy beach. The beach was then sufficiently wide to provide a buffer against direct wave attack on the bluffs. During the winter of 1977/1978, a period of high tides coupled with strong storms caused severe erosion of the beach, replacing the sand with cobbles. A protective gunite covering that had been placed over the bluff face to the north of the cove was severely damaged by wave-thrown cobbles. A second application of gunite was undermined by runoff from the bluff rim. In August of 1983, storm waves generated in the southern hemisphere combined with high tides to break up the remaining gunite, and attack the easily eroded terrace deposits above the bedrock platform. The upper bluff face was rapidly eroded back

by as much as 27 feet, leaving the wide bedrock platform present today. Apparently as a means to retard bluff deterioration in the cove area, fill material was placed against the face of the cliff forming a bench approximately 25-foot wide, at roughly 20 feet above sea level. By September of 1984, portions of this fill bench had reportedly been eroded back by 6 to 7 feet. At the time of our field investigation, this fill bench had been almost completely eroded away.

Plate 3 depicts the bluff retreat along two cross sections though the site, spanning the periods of accelerated erosion during 1977, and 1983. The approximate bluff profiles shown are based on: 1) a 1975, 200-scale map; 2) the 1983, 20-scale map used as the base for our geologic map (Plate 2); and 3) existing site conditions, as determined by tape and hand level in conjunction with the available base maps.

6.3 Modes of Failure

Without preventive measures, the bluffs at the site will continue to retreat. The most significant threat to bluff stability comes from the ocean. The site is situated in the Oceanside littoral cell, where beach-forming sand migrates southward along the coast. The amount of sediment entering this system through rivers has been reduced by the construction of dams and other projects. Furthermore, the sediment's southward transport by longshore currents is interrupted by coastal jetties. This makes the supply to the beach at the site somewhat dependent on the dredging activities performed to move sand around the jetties. The sand beach that had provided some buffer against wave attack, and was eroded away during the storms of 1977-78 and 1983, has not returned.

A shallow rock reef extending out from the beach provides some protection against violent wave attack on the bluffs at the site. The reef causes a northwest swell to break well offshore, thereby dissipating much of the wave energy before it reaches the base of the bluffs. Because of its position just to the north of the site, this reef offers less protection against storm waves generated out of the south.

The bedrock platform to the north of the cove acts as a natural seawall to protect the poorly consolidated terrace deposits that form the base of the upper bluff. Erosion of the upper bluff would be concentrated during periods when high tides and storm surge allow waves to break over the platform and directly attack the terrace deposits. Although relatively resistant to wave attack, the bedrock platform will also retreat. Block-glide failures and topples will occur through a gradual process of undermining along plains of weakness, fracturing, and separating.

In the cove, the lower elevation and the relatively less resistant character of the bedrock, combined with the presence of the highly erodible sand and gravel immediately above it, make this area particularly susceptible to erosion by wave action. The present steepness of the bluff here makes it susceptible to collapse, even without additional erosion at its base. Slope stability is addressed further in Section 8.2 of this report.

The other major factors in bluff retreat are surface water runoff, and the effects of groundwater. Uncontrolled surface water draining is concentrated into gullies incised into the bluff rim. Over time the gullies become deeper, coalesce, and migrate inland, effectively moving the top of the bluff backwards. An increase in groundwater levels could contribute to bluff degradation in a variety of ways including: weakening the bedrock though dissolution along fractures and claystone beds; perched seepage causing erosion at the bedrock/terrace deposit contact; and effectively reducing the shear strength of the lower terrace deposits.

6.3 Bluff Retreat Rates

In quantifying historical bluff retreat at the site, several factors were taken into consideration. The amount of bluff retreat that has occurred since the first useful photographs were taken in 1929, varies along the subject property. Some uncertainty is added by the effects on retreat rates, both positive and negative, that may have resulted from old fills, gunite facing, and other past alterations to native conditions at the site. There are also considerable differences in the amount of erosion,

depending on whether it is being measured at the bluff rim, or at some lower level. Recognizing the many variables, an empirical figure of 25 feet has been extrapolated over the 62 years of documentation in estimating an average retreat rate of 0.4 feet per year.

7.0 SEISMICITY

7.1 General

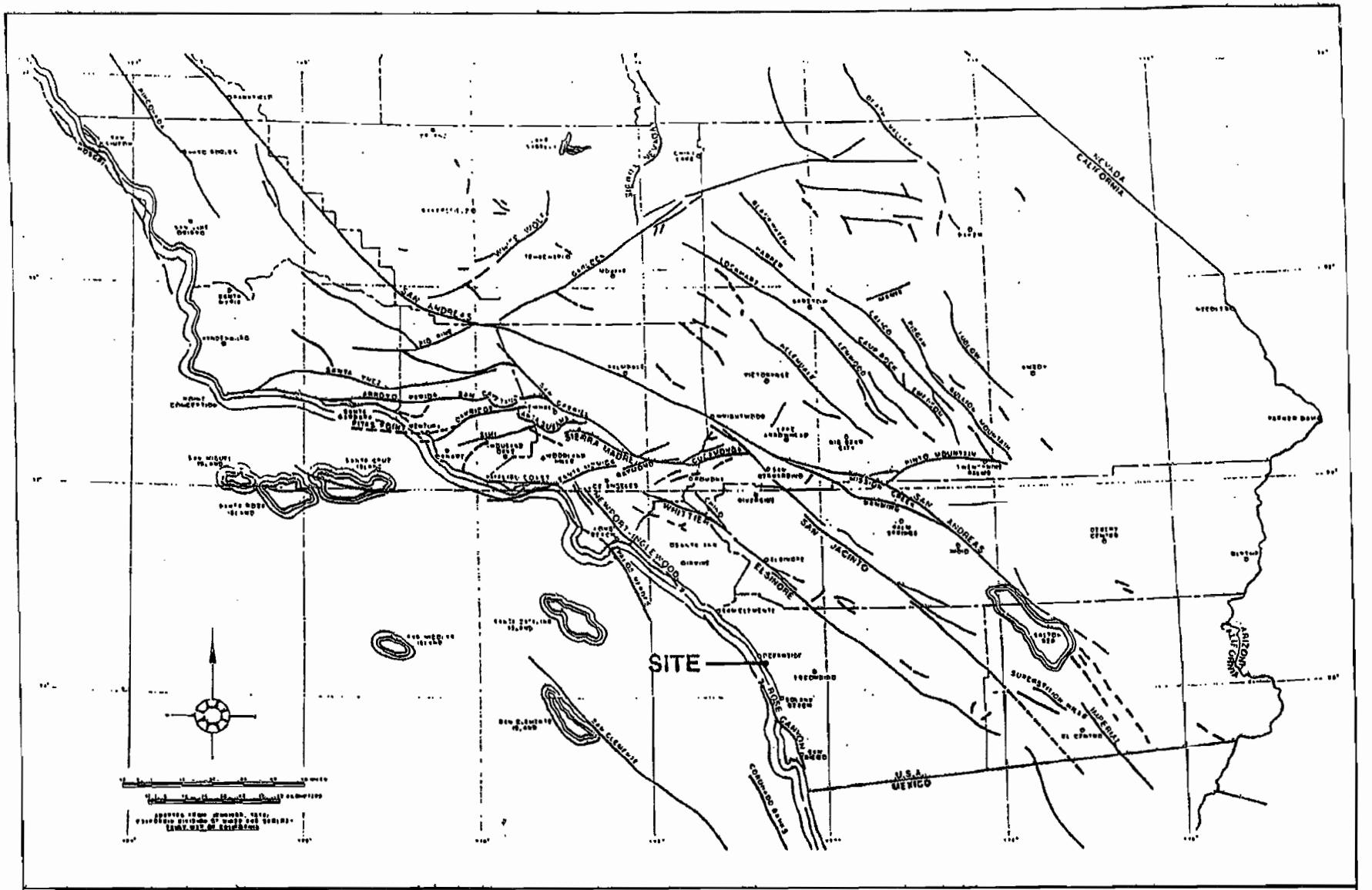
The site is considered to be a seismically active area, as can all of southern California. There are, however, no known active faults either on or adjacent to the project site. Figure 2 shows the known active faults and major earthquake epicenters in the region and their geographic relationship to the site. Because these active faults are at a substantial distance, the seismic risk at this site is considered to be low to moderate in comparison to many parts of southern California.

Most seismic hazards at the site are a consequence of ground shaking caused by events on distant, active faults. The hazard level is sufficient to place the area in seismic risk zone 3 as defined in the Uniform Building Code. In addition to the information on Figure 2, Table 1 lists the active faults within 63 miles (100 kilometers) of the site and the maximum probable earthquakes on those faults. By definition, the maximum probable earthquake for a given fault is the largest earthquake likely to occur within a 100 year interval.

7.2 Earthquake Effects

7.2.1 Surface Fault Rupture

In our opinion, no credible risk of surface rupture exists at the project site. No known active faults or potentially active faults cross the site.



REGIONAL FAULT MAP

JOB NO.:	DATE:	FIGURE:
04-8529-001-00-00	MARCH 1991	2

ICG Incorporated

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TABLE 1

SEISMICITY FOR MAJOR FAULTS WITHIN 100 KILOMETERS OF SITE

FAULT	DISTANCE	Maximum Credible Earthquake[1]		Maximum Probable Earthquake[1]	
		M_L[2]	Peak Bedrock Acceleration[3]	M_L[4]	Peak Bedrock Acceleration[3]
Rose Canyon	6 miles SW	7.0	0.44g	6.5	0.38g
Elsinore	25 miles NE	7.5	0.23g	7.0	0.17g
San Jacinto	49 miles NE	7.5	0.10g	7.5	0.10g
San Clemente	55 miles SW	7.5	0.08g	7.3	0.07g
San Andreas	65 miles NE	7.5	0.05g	7.5	0.05g
Newport-Inglewood	50 miles NW	7.0	0.06g	6.5	0.04g
Coronado Banks	24 miles S	6.5	0.12g	6.0	0.08g
La Nacion[5]	29 miles S	6.8	0.12g	6.0	0.06g

1 The maximum credible earthquake is the largest earthquake that appears capable of occurring under the presently known tectonic framework. The maximum probable earthquake is the largest earthquake that is likely to occur during a 100-year interval.

2 Values are local magnitudes, taken from Jennings (1975), and Greensfelder (1974).

3 From attenuation chart in Seed and Idriss (1982).

4 Values are local magnitudes, generally taken from Seismic Safety Study for City of San Diego (1974).

5 The earthquake capability of the La Nacion Fault has not been established. Although it is classed as only potentially active, it is included for information purposes due to its proximity to the site.

7.2.2 Ground Accelerations

In our opinion, based on information now available, the most significant event likely to affect the project will be a 6.5 magnitude event on the Rose Canyon Fault. For the Rose Canyon Fault event, we estimate a peak bedrock acceleration within the project area of about 0.38g. Design of structures should conform to the requirements of the governing agencies, as well as to the standard practices of the Structural Engineers Association of California.

Recent work on the Rose Canyon fault zone has shown strands within the zone to offset holocene (11,000 years and younger) deposits, making them "active" as defined by the California Division of Mines and Geology. The age of the most recent movement, the fault's recurrence interval (expected period between major earthquake events), and the relationship between the active trace, and other faults within the fault zone, have not yet been established. As a result, the impact on seismic safety is not known. The degree of the hazard may not be determined for years.

We have reviewed the existing information available regarding the fault and conclude that a magnitude 7.0 earthquake is an appropriate maximum credible event for a 20 mile rupture length (offshore La Jolla to Coronado Bridge). A maximum probable event of magnitude 6.5 is hypothesized for the same fault.

7.2.3 Seismically Induced Settlement and Liquefaction

Because of the high relative densities of both the native materials underlying the site and suitably compacted fill that may be placed during development, liquefaction or seismically induced settlements are not considered a hazard.

7.2.4 Other Hazards

Seismically induced slope failures along the bluff are a possibility, however, they should not pose a significant threat provided the structural setback recommended in this report are followed. Due to the proximity of the site to the ocean, tsunamis are always a possibility, however no seismic sea-waves higher than normal tides have ever been recorded in San Diego County.

8.0 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

8.1 General Discussion

No geotechnical conditions were apparent during our investigation which would preclude the site development as planned. The site condition which should most severely impact the development is the potential for future erosion and slope stability of the bluff area. The on-site soils are considered non-expansive and should be suitable to support the proposed structure once the existing fills and loose soils have been removed or recompacted.

The following recommendations are intended to provide a suitable bearing pad for foundations. We recommend that all loose soils or undocumented fill soils within the building area be removed and replaced as compacted fill. Once these soils have been prepared, the structure can be supported on a conventional shallow foundation system bearing entirely on bedrock or compacted fill.

The remainder of Section 8.0 presents our recommendations for foundation design and construction in detail. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations appear not to cover any specific feature of the project, please contact our office for additions or revisions to our recommendations.

8.2 Structural Setback and Slope Stability

A structural setback from the bluff rim is recommended to protect dwellings from collapse within the economic life of the residence. The setback depicted on the Geotechnical Maps (Plates 1 and 3) is based on slope stability analysis along two cross sections through the site (Plate 2). Engineering practice uses a factor of safety of 1.5 as the minimum criteria for a stable slope (a factor of safety of less than 1.0 indicates that a slope should fail). Slope stability calculations indicate that the trial failure surfaces behind the setback line exhibit a factor of safety greater than 1.5. Trial

failure surfaces west of the setback line had factors of safety less than 1.5. Our analysis was conducted using the STABL5 computer program. Failure surfaces were generated for circular type failures. Circular failures were used in the analysis because of the absence of continuous places of weakness which are used in block glide type failures. Soil strengths parameters used in our calculations were based on the residual strength of the soil as calculated from direct shear tests in our laboratory. It is our understanding that no new slopes are currently planned for the site. If future plans include slopes or the placement of fill beyond the setback line, ICG Inc. should be contacted to analyze the slope stability.

The major factor affecting the future stability of the existing bluff is the erosion potential of the bluff soils. As discussed in Section 6.4, the bluff retreat rate is estimated to be 0.4 feet per year. A factor of safety of 1.5 is applied, and the result is multiplied by 75 years, to arrive at an estimated 45 feet of bluff retreat over a 75 year economic life for the proposed residence. The loss of 45 feet of bluff, back from the existing rim, would still fall well short of the structural setback establish through slope stability calculations and the 1.5 factor of safety. Appurtenant structures, temporary structures, decks and landscaping constructed beyond the setback will incur a high risk of potential damage over time. The risk for damage increases as the bluff top is approached. It is recommended that measures be taken to prevent the erosion of the bluff face. Such measures could include the channeling of surface drainage away from the bluff face, construction of a sea wall and/or covering the bluff face with an erosion resistant material, ie. rip rap, or gunite.

8.3 Grading and Earthwork

8.3.1 Geotechnical Observation

During grading, ICG Inc. should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by the preliminary investigations, to adjust designs to actual field conditions, and to determine that the grading

is in general accordance with the recommendations of this report. Our personnel should perform sufficient testing of any fill placed to support our opinion as to whether compaction recommendations have been complied with.

8.3.2 Site Preparation

Site preparation should begin with the removal of all vegetation and deleterious materials within the area to be graded. In addition all loose soils and undocumented fill should be completely removed down to firm bedrock materials. The exposed bedrock should be scarified to a depth of 8 to 12 inches and recompacted to 90 percent of maximum density, ASTM D-1557. Native on-site soils can be used as fill material, provided all deleterious materials have been completely removed.

If during grading a cut/fill transition is created, and deep foundations are not used in the fill portion, the following recommendation should be implemented. The cut portion of the pad should be overexcavated to a depth of 5 feet below finish grade and replaced as a compacted fill. The lateral extent of the excavation should be 5 feet out side of the building perimeter. All fill soils should be compacted as recommended in Section 8.3.3 Fill Compaction.

8.3.3 Fill Compaction

All fill and backfill to be placed in association with site development should be accomplished at slightly over optimum moisture conditions and using equipment that is capable of producing a uniformly compacted product. The minimum relative compaction recommended for fill is 90 percent of maximum density based on ASTM D1557 (modified Proctor). Sufficient observation and testing should be performed by the geotechnical consultant so that an opinion can be rendered as to the degree of compaction achieved.

Representative samples of imported materials and on site soils should be tested by the geotechnical consultant in order to evaluate the maximum density, optimum moisture content, and where appropriate, shear strength, consolidation, and expansion characteristics of the soil.

During grading operations, soil types other than those analyzed in the geotechnical reports may be encountered by the contractor. The geotechnical consultant should be notified to evaluate the suitability of these soils for use as fill and as finish grade soils.

8.3.4 Trench Backfill

All trench backfill should be compacted by mechanical means in uniform lifts of 8 to 12 inches. The backfill should be uniformly compacted to at least 90 percent of ASTM D1557.

8.4 Site Drainage

Foundation and slab performance depends greatly on how well the runoff waters drain from the site. This is true both during construction and over the entire life of the structure. The ground surface around structures should be graded so that water flows rapidly away from the structures without ponding. The surface gradient needed to achieve this depends on the prevailing landscape. In general, we recommend that pavement and lawn areas within five feet of buildings slope away at gradients of at least two percent. Densely vegetated areas should have minimum gradients of at least five percent away from buildings in the first five feet. Densely vegetated areas are considered those in which the planting type and spacing is such that the flow of water is impeded.

Planters should be built so that water from them will not seep into the foundation, slab, or pavement areas. Site irrigation should be limited to the minimum necessary to sustain landscaping plants. Should excessive irrigation, water line breaks, or

unusually high rainfall occur, saturated zones or "perched" groundwater may develop in fill soils.

Because of the highly erosion prone nature of the on-site soils additional care should be taken to direct runoff water away from the top of the bluff. Drainage channels and swales should be constructed to help channel water away from erosion prone areas. We recommend that drainage water be carried in a pipe or conduit to the base of the bedrock bench in the lower portion of the bluff, rather than be allowed to flow freely over the bluff face.

8.5 Foundation Recommendations

8.5.1 General

Special design considerations for soil expansion are not considered necessary. All foundations should bear entirely on either uniformly compacted fill or on bedrock. The following paragraphs in Section 8.6 will present our recommendations for foundation design. Our recommendations are considered to be generally consistent with the standards of practice. They are based on both analytical methods and empirical methods derived from experience with similar geotechnical conditions. Reinforcement recommendations are considered the minimum necessary for the likely soil conditions and are not intended to supersede the design of the Structural Engineer or criteria of governing agencies.

8.5.2 Foundations on Bedrock

If the building area is prepared such that all footings will bear entirely on bedrock the following foundation design parameters should be applicable.

Allowable Soil Bearing:	3000 psf (allow a one-third increase for short-term wind or seismic loads)
Minimum Footing Width:	12 inches
Minimum Footing Depth:	18 inches
Minimum Reinforcement:	1 no. 4 bar at both top and bottom in continuous footings, or design as simply supported beam capable of supporting the applied loads over a span of 4 feet, whichever is greater. Equal resistance to positive and negative moments should be provided.

8.5.3 Foundations on Fill

If all footings will bear entirely on compacted fill soils, the following foundation design parameters should be applicable.

Allowable Soil Bearing:	2000 psf (allow a one-third increase for short-term wind or seismic loads)
Minimum Footing Width:	12 inches
Minimum Footing Depth:	18 inches
Minimum Reinforcement:	1 no. 5 bar at both top and bottom in continuous footings, or design as simply supported beam capable of supporting the applied loads over a span of 6 feet, whichever is greater. Equal resistance to positive and negative moments should be provided.

8.5.4 Settlement

The anticipated total and differential settlement for the proposed structure should be within tolerable limits provided that the recommendations of this report are followed. In general, total settlements are estimated to be less than 1 inch, and differential settlement is expected to be less than ½ inch. It is recommended that we review the actual foundation plans to evaluate the footing configurations and loading conditions.

8.5.5 Lateral Load Resistance

Lateral loads against structures may be resisted by friction between the bottoms of footings and the supporting soil. A coefficient of friction of 0.4 is recommended for both fill and bedrock materials. Alternatively, a passive pressure of 350 pcf and 250 pcf is recommended for undisturbed bedrock material and compacted fill, respectively. If friction and passive pressure are combined, the passive pressure value should be reduced by one-third.

8.5.6 On-Grade Slabs

On-Grade slabs should be at least 4 inches thick and reinforced by no. 3 reinforcing bars placed on 24 inch centers placed at mid-height in the slab. Crack control joints should be provided in all slabs, spaced on 15 to 20 foot centers.

Concrete slabs constructed on soil ultimately cause the moisture content to rise in the underlying soil. This results from continued capillary rise and the termination of normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab unless some protection is provided. This may cause mildewed carpets, lifting or discoloration of floor tile, or similar problems.

To minimize these problems, suitable moisture protection measures should be used. Various alternatives exist, such as concrete toppings or additives, or synthetic moisture-resistant membranes. Information on the usage, installation, and warranty should be obtained from the manufacturer if these products are used. The effectiveness of such measures can be improved by installing a capillary break under the membrane or damp-proofed slab.

If waterproofing membranes are installed beneath concrete slabs, at least one inch of sand should be placed between the membrane and the slab to decrease the likelihood of curing problems in the concrete.

8.5.7 Foundation Observation

All foundation excavations should be observed by the geotechnical consultant prior to placement of forms, reinforcement, or concrete. The observation will confirm that the soil conditions are as anticipated and that the intent of our recommendations have been complied with. The excavations should be trimmed to design dimensions and should be clear of all loose slough.

8.6 Earth Retaining Structures

Cantilever retaining walls backfilled with on-site soil should be designed for an active earth pressure approximated by an equivalent fluid pressure of 40 lbs/ft^3. The active pressure should be used for walls free to yield at the top at least 1 percent of the wall height. For walls restrained so that such movement is not permitted, an equivalent fluid pressure of 60 lbs/ft^3 should be used, based on at-rest soil conditions. The above pressures do not consider any sloping backfill, surcharge loads, or hydrostatic pressures. If these are applicable, they will increase the lateral pressures on the wall and we should be contacted for additional recommendations.

Retaining wall backfill should be compacted to at least 90 percent relative compaction, based on ASTM D1557. Backfill should not be placed until walls have achieved adequate structural strength. Heavy compaction equipment which could cause distress to walls should not be used.

8.7 Reactive Soils

Our testing program indicated that the soils on-site contain sulphate contents high enough to be detrimental to type I portland cement. Therefore, we recommend that Type II cement be used in all concrete which will be in contact with soil.

8.8 Review of Plans

When the grading plans and foundation plans are developed, they should be forwarded to the geotechnical consultant review. The recommendations of this report are based on assumptions regarding the proposed development. Our review will confirm these assumptions and evaluate if the intent of the recommendations of this report have been complied with.

90 LIMITATIONS OF INVESTIGATION

Our investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional opinions included in this report.

The samples taken and used for testing and the observations made are believed representative of the project site; however, soil and geologic conditions can vary significantly between borings.

As in most projects, conditions revealed by excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations made, if warranted.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated

into the plans, and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

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 other than our own personnel on the site.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

***

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APPENDIX A

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APPENDIX B

Field Exploration

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 43' LOCATION: See Map

LOG OF BORING NO. 1

Sheet 1 of 1

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0							TERRACE DEPOSITS (Qt): Reddish brown silty fine to medium SAND, dry to damp, very dense, some organics in upper foot	PARTICLE SIZE ANALYSIS
		56						
		76						
5								
		28					Becoming lighter color, friable, damp, dense	
10								
		26						
15								
		22					Light to dark gray salt and pepper fine SAND, damp, medium dense, very friable	
20							@21', Refusal on cobbles	
							Total Depth 21' No Groundwater Refusal on Cobbles Backfilled 2/15/91	
25								
30								
35								

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-2

TE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

OGGED BY: MD GROUND ELEVATION: 47' LOCATION: See Map

LOG OF BORING NO. 2

Sheet 1 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		8					FILL/TOPSOIL: Light reddish brown silty fine SAND, damp, loose	
5		21					TERRACE DEPOSITES (Qt): Dark reddish brown silty fine to medium SAND, damp, medium dense, moderately friable	
10		17						
15		27					Yellowish gray fine to medium SAND, damp, medium dense, friable	
20		23					Light to dark gray fine SAND, damp, medium dense, slightly silty, micaceous, friable	
25		30						
30		26					Light to dark gray fine SAND, slightly moist, dense, micaceous, very friable, laminated/ cross bedded	
35		16						
		42					SANTIAGO FORMATION (Tsa): Olive gray with orange mottling silty claystone, slightly moist, very stiff	DIRECT SHEAR PARTICLE SIZE ANALYSIS

JOB NO.: 04-8529-001-00-00 | ICG Incorporated | FIGURE: B-3

ATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

OGGED BY: MD GROUND ELEVATION: 47' LOCATION: See Map

LOG OF BORING NO. 2

Sheet 2 of 2

DEPTH (FEET)	CLASSIF-ICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
40							Light orange brown fine to medium SANDSTONE, moist, dense, trace gravel	
							▽	
		12	■				Hole sloughing	
45								
		31	■					
50							Greenish gray silty fine SANDSTONE, wet, dense	
							Total Depth 50.5' Groundwater at 44' Backfilled 2/15/91	
55								
60								
65								
70								
75								

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-4

TE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

OGGED BY: MD GROUND ELEVATION: 44' LOCATION: See Map

LOG OF BORING NO. 3

Sheet 1 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		10					FILL/TOPSOIL: Light reddish brown silty fine SAND, dry to damp, loose to medium dense	
5		40					TERRACE DEPOSITES (Qt): Reddish brown silty fine to medium SAND, damp, dense, moderately cemented	
10		34						
15		16					@15', Becoming light gray fine SAND, slightly silty, damp, medium dense, friable	
20							@19.5', some cobbles, no sampling	
25		25					Light gray fine to medium SAND, trace gravel, damp, medium dense, friable	
30		38						
35		26					@32', Cobble bed	
		25					Yellow-orange fine to medium SAND, slightly silty, moist, medium dense, water added during drilling	

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-4

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 44' LOCATION: See Map

LOG OF BORING NO. 3

Sheet 2 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
40								
							Total Depth 41' Backfilled 2/15/91 Groundwater at 38'	
45								
50								
55								
60								
65								
70								
75								

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-5

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 41' LOCATION: See Map

LOG OF BORING NO. 4

Sheet 1 of 1

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		11					FILL/TOPSOIL: Reddish brown silty fine SAND, damp, loose to medium dense	
5		86					TERRACE DEPOSITES (Qt): Dark reddish brown silty fine to medium SAND, damp, very dense, cemented	MAXIMUM DENSITY SULFATE pH RESISTIVITY
10		40					Becoming light gray, less cemented	
15		20						
20		34					Light to dark gray fine SAND, slightly moist, dense, friable, laminated/ cross bedded @22.5'-26', Cobble beds	
25								
30		49					SANTIAGO FORMATION (Tsa): Olive gray silty fine SANDSTONE, with clayey laminations and inclusions, damp, dense to very dense	
35		35/5"					@34', Becoming moist	INSITU MOISTURE DENSITY PARTICLE SIZE ANALYSIS
							Total 34.5' No Groundwater Backfilled 2/15/91	

JOB NO.: 04-8529-001-00-00 ICG Incorporated FIGURE: B-5

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 46' LOCATION: See Map

LOG OF BORING NO. 5

Sheet 1 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		16					FILL/TOPSOIL: Light reddish brown silty fine SAND, damp, loose to medium dense	
5		30					TERRACE DEPOSITES (Qt): Reddish brown fine SAND, slightly silty, damp, dense, lightly cemented	
10		44						
15		17					@15', Becoming dark gray, friable	
20		21						
25		43						
30		30					Gray with orange staining fine to medium SAND @31', Cobbles	
35		31					Light to dark gray, fine to medium SAND, moist, dense, laminated very friable	
		28						

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-6

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 46' LOCATION: See Map

LOG OF BORING NO. 5

Sheet 2 of 2

DEPTH (FEET)	CLASSIF-ICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
40							SANTIAGO FORMATION (Tsa): Olive gray silty to clayey fine SANDSTONE, wet, dense to very dense	
45							Total Depth 41' Groundwater at 39' Backfilled 2/15/91	
50								
55								
60								
65								
70								
75								

JOB NO.: 04-8529-001-00-00

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FIGURE: B-7

APPENDIX C

Laboratory Testing Program

APPENDIX C

LABORATORY TESTING

Selected representative samples of soils encountered were tested using test methods of the American Society for Testing and Materials, or other generally accepted standards. A brief description of the tests performed follows:

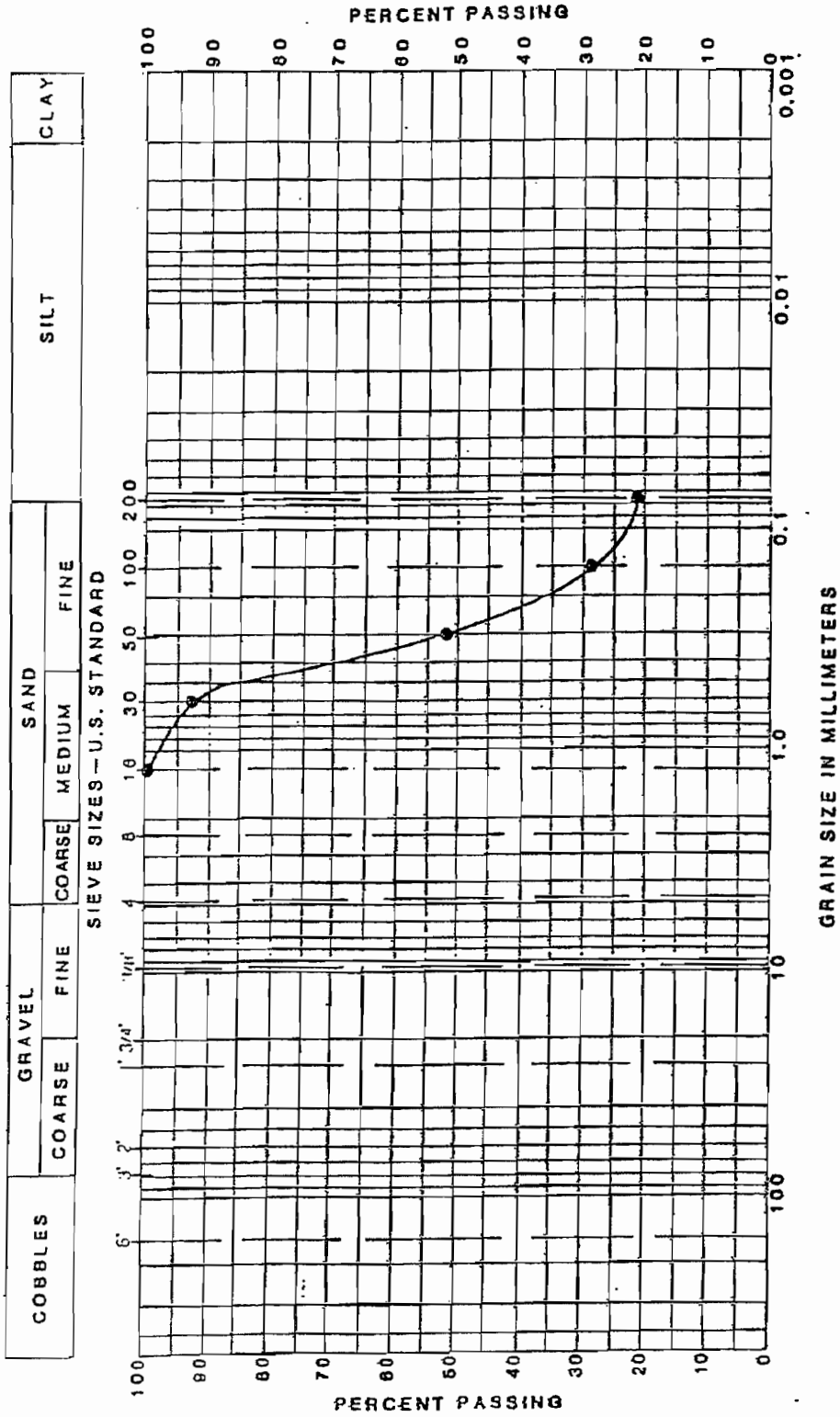
Classification: Soils were classified visually according to the Unified Soil Classification System. Visual classification was supplemented by laboratory testing of selected samples and classification in accordance with ASTM D2487. The soil classifications are shown on the Boring Logs.

Particle Size Analysis: A particle size analysis was performed in accordance with ASTM D422. The grain size distribution was used to determine presumptive strength parameters used to develop foundation design criteria. The results are provided on the following Figures C-1 to C-3.

Direct Shear Tests: Unconsolidated, undrained direct shear tests were performed in accordance with ASTM D3080. The samples were remolded to 90 percent of the modified proctor density and tested in a saturated condition using normal loads of 1 ksf, 2 ksf, and 4 ksf. The results of the tests are presented in the attached Figure C-4.

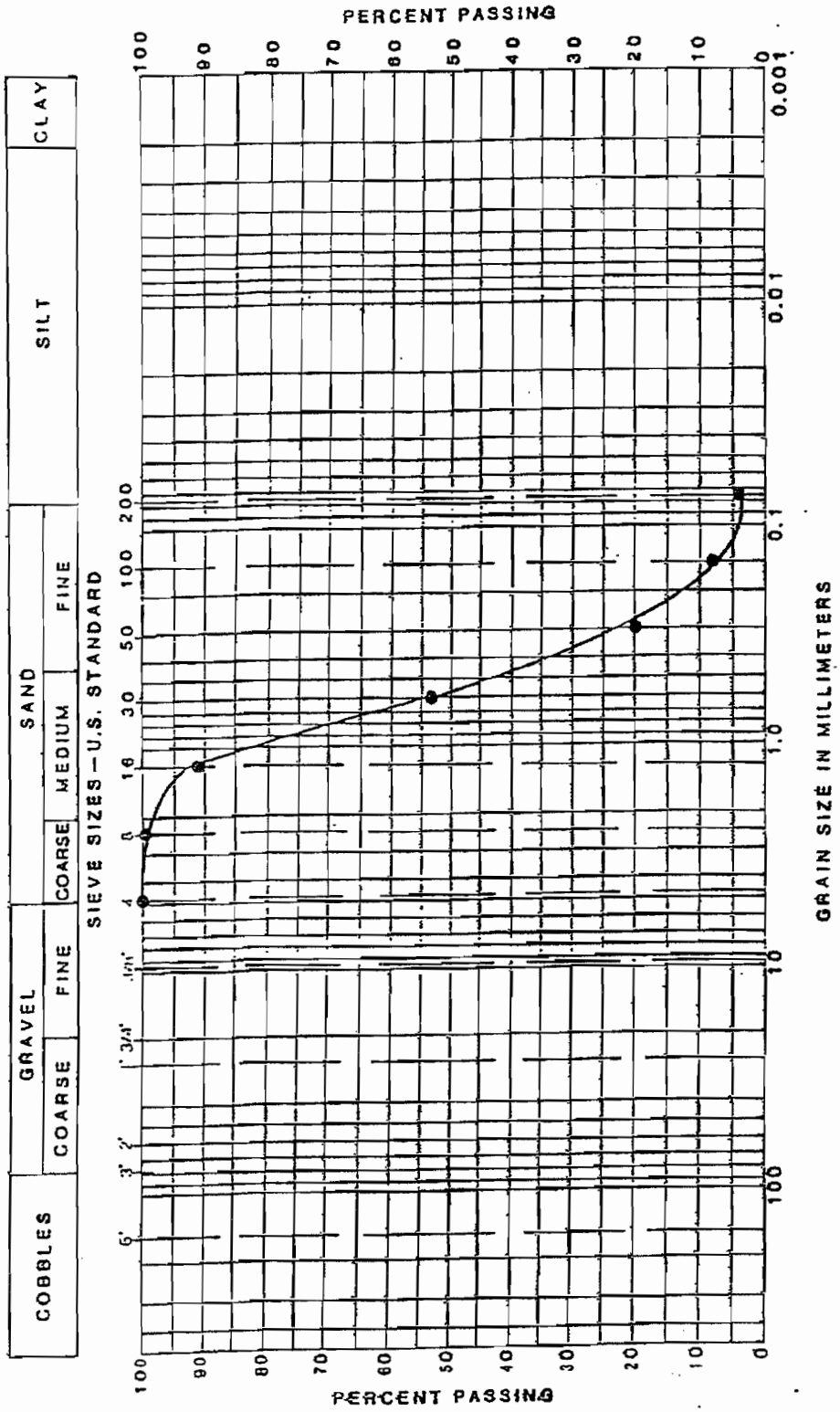
Moisture-Density Relationship: Laboratory tests were performed in accordance with ASTM D1557. A mechanically operated rammer was used during the compaction process. Test results are presented on Table C-2.

In-Situ Moisture/Density: The in-place moisture content and dry unit weight of selected samples were determined using relatively undisturbed samples from the liner rings of a 2.5 inch ID Modified California Sampler. The dry unit weight and moisture content are shown on the attached Boring Logs.



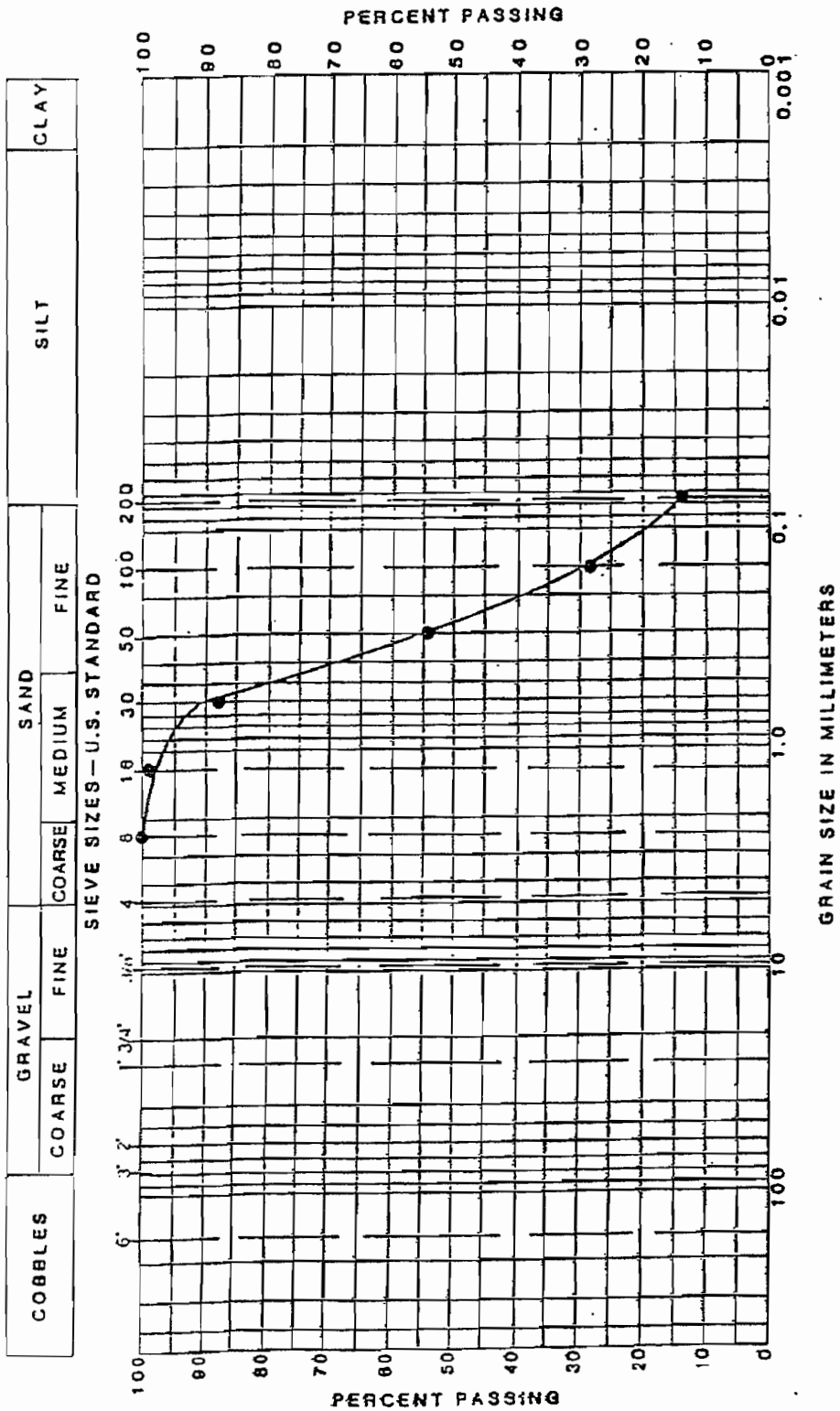
BORING NO.	DEPTH (FEET)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	CLASSIFICATION
B-1	5	●	-	NONPLASTIC	SM-(SILTY SAND)

JOB NO.:		FIGURE:
04-8529-001-00-00	PARTICLE SIZE ANALYSIS	C-1



BORING NO.	DEPTH (FEET)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	CLASSIFICATION
B-2	39	●		NONPLASTIC	SP-(POORLY GRADED SAND)

JOB NO.:		FIGURE:
04-6529-001-00-00	PARTICLE SIZE ANALYSIS	C-2



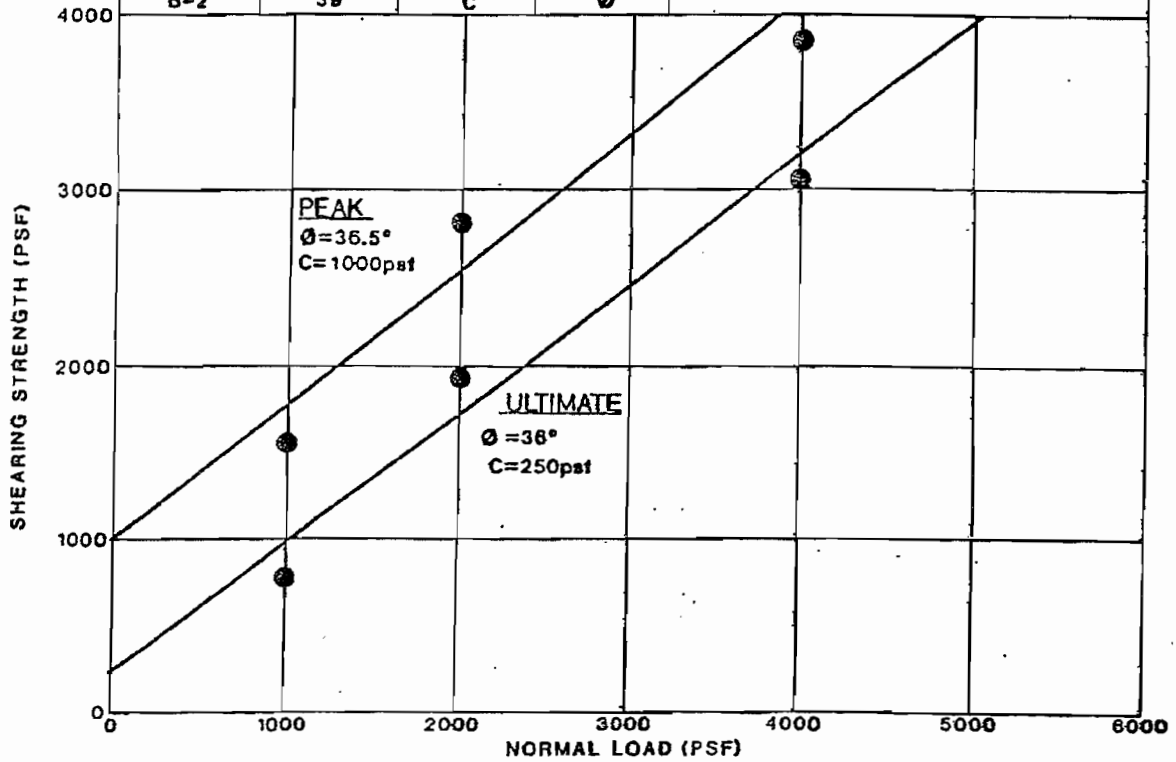
BORING NO.	DEPTH (FEET)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	CLASSIFICATION
B-4	33.5	●	28	5	SM-(FINE SILTY SAND)

JOB NO.: 04-8529-001-00-00

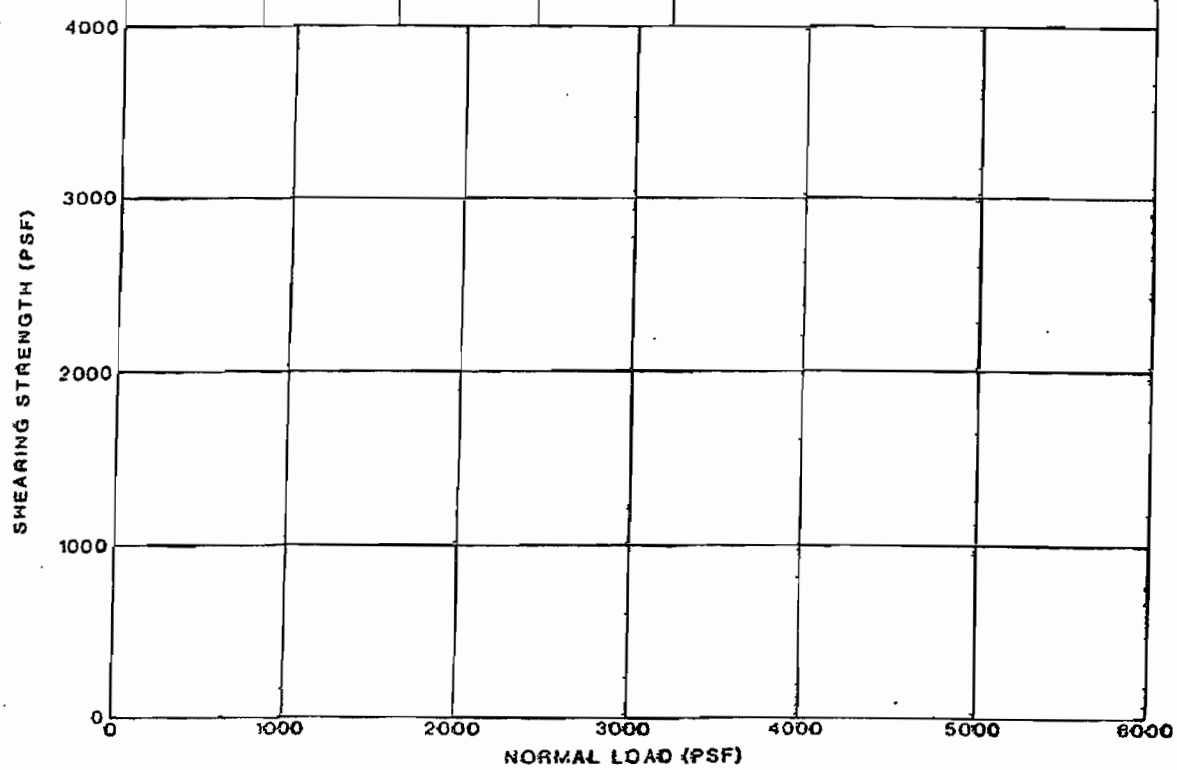
PARTICLE SIZE ANALYSIS

FIGURE: C-3

BORING NO.	DEPTH (FEET)	COHESION, (PSF)	ANGLE OF FRICTION, °	SAMPLE DESCRIPTION
B-2	39	C	Ø	BROWN MEDIUM TO FINE SAND (SP)



BORING NO.	DEPTH (FEET)	COHESION, (PSF)	ANGLE OF FRICTION, °	SAMPLE DESCRIPTION



JOB NO.: 04-8529-001-00-00	SHEARING STRENGTH TEST	FIGURE: C-4

APPENDIX D

Standard Guidelines for Grading Projects

STANDARD GUIDELINES FOR GRADING PROJECTS

1. GENERAL

1.1 Representatives of the Geotechnical Consultant should be present on-site during grading operations in order to make observations and perform tests so that professional opinions can be developed. The opinion will address whether grading has proceeded in accordance with the Geotechnical Consultant's recommendations and applicable project specifications; if the site soil and geologic conditions are as anticipated in the preliminary investigation; and if additional recommendations are warranted by any unexpected site conditions. Services do not include supervision or direction of the actual work of the contractor, his employees or agents.

1.2 The guidelines contained herein and the standard details attached hereto represent this firm's standard recommendations for grading and other associated operations on construction projects. These guidelines should be considered a portion of the report to which they are appended.

1.3 All plates attached hereto shall be considered as part of these guidelines.

1.4 The Contractor should not vary from these guidelines without prior recommendation by the Geotechnical Consultant and the approval of the Client or his authorized representative.

1.5 These Standard Grading Guidelines and Standard Details may be modified and/or superseded by recommendations contained in the text of the preliminary geotechnical report and/or subsequent reports.

1.6 If disputes arise out of the interpretation of these grading guidelines or standard details, the Geotechnical Consultant should determine the appropriate interpretation.

2. DEFINITIONS OF TERMS

2.1 ALLUVIUM -- Unconsolidated detrital deposits resulting from flow of water, including sediments deposited in river beds, canyons, flood plains, lakes, fans at the foot of slopes and estuaries.

2.2 AS-GRADED (AS-BUILT) -- The surface and subsurface conditions at completion of grading.

2.3 BACKCUT -- A temporary construction slope at the rear of earth retaining structures such as buttresses, shear keys, stabilization fills or retaining walls.

2.4 BACKDRAIN -- Generally a pipe and gravel or similar drainage system placed behind earth retaining structures such buttresses, stabilization fills, and retaining walls.

2.5 BEDROCK -- A more or less solid, relatively undisturbed rock in place either at the surface or beneath superficial deposits of soil.

2.6 BENCH -- A relatively level step and near vertical rise excavated into sloping ground on which fill is to be placed.

2.7 BORROW (Import) -- Any fill material hauled to the project site from off-site areas.

2.8 BUTTRESS FILL -- A fill mass, the configuration of which is designed by engineering calculations to retain slope conditions containing adverse geologic features. A buttress is generally specified by minimum key width and depth and by maximum backcut angle. A buttress normally contains a backdrainage system.

2.9 CIVIL ENGINEER -- The Registered Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topographic conditions.

2.10 COLLUVIUM -- Generally loose deposits usually found near the base of slopes and brought there chiefly by gravity through slope continuous downhill creep (also see Slope Wash).

2.11 COMPACTION -- Is the densification of a fill by mechanical means.

2.12 CONTRACTOR -- A person or company under contract or otherwise retained by the Client to perform demolation, grading and other site improvements.

2.13 DEBRIS -- All products of clearing, grubbing, demolition, contaminated soil material unsuitable for reuse as compacted fill and/or any other material so designated by the Geotechnical Consultant.

2.14 ENGINEERING GEOLOGIST -- A Geologist holding a valid certificate of registration in the specialty of Engineering Geology.

2.15 ENGINEERED FILL -- A fill of which the Geotechnical Consultant or his representative, during grading, has made sufficient tests to enable him to conclude that the fill has been placed in substantial compliance with the recommendations of the Geotechnical Consultant and the governing agency requirements.

2.16 EROSION -- The wearing away of the ground surface as a result of the movement of wind, water, and/or ice.

2.17 EXCAVATION -- The mechanical removal of earth materials.

2.18 EXISTING GRADE -- The ground surface configuration prior to grading.

2.19 FILL -- Any deposits of soil, rock, soil-rock blends or other similar materials placed by man.

2.20 FINISH GRADE -- The ground surface configuration at which time the surface elevations conform to the approved plan.

2.21 GEOFABRIC -- Any engineering textile utilized in geotechnical applications including subgrade stabilization and filtering.

2.22 GEOLOGIST -- A representative of the Geotechnical Consultant educated and trained in the field of geology.

2.23 GEOTECHNICAL CONSULTANT -- The Geotechnical Engineering and Engineering Geology consulting firm retained to provide technical services for the project. For the purpose of these guidelines, observations by the Geotechnical Consultant include observations by the Geotechnical Engineer, Engineering Geologist and those performed by persons employed by and responsible to the Geotechnical Consultants.

2.24 GEOTECHNICAL ENGINEER -- A licensed Civil Engineer who applies scientific methods, engineering principles and professional experience to the acquisition, interpretation and use of knowledge of materials of the earth's crust for the evaluation of engineering problems. Geotechnical Engineering encompasses many of the engineering aspects of soil mechanics, rock mechanics, geology, geophysics, hydrology and related sciences.

2.25 GRADING -- Any operation consisting of excavation, filling or combinations thereof and associated operations.

2.26 LANDSLIDE DEBRIS -- Material, generally porous and of low density, produced from instability of natural of man-made slopes.

2.27 MAXIMUM DENSITY -- Standard laboratory test for maximum dry unit weight. Unless otherwise specified, the maximum dry unit weight shall be determined in accordance with ASTM Method of Test D1557.

2.28 OPTIMUM MOISTURE -- Test moisture content at the maximum density.

2.29 RELATIVE COMPACTION -- The degree of compaction (expressed as a percentage) of dry unit weight of a material as compared to the maximum dry unit weight of the material.

2.30 ROUGH GRADE -- The ground surface configuration at which time the surface elevations approximately conform to the approved plan.

2.31 SITE -- The particular parcel of land where grading is being performed.

2.32 SHEAR KEY -- Similar to buttress, however, it is generally constructed by excavating a slot within a natural slope in order to stabilize the upper portion of the slope without grading encroaching into the lower portion of the slope.

2.33 SLOPE -- Is an inclined ground surface the steepness of which is generally specified as a ratio of horizontal:vertical (e.g., 2:1).

2.34 SLOPE WASH -- Soil and/or rock material that has been transported down a slope by mass wasting assisted by runoff water not confined by channels (also see Colluvium).

2.35 SOIL -- Naturally occurring deposits of sand, silt, clay, etc., or combinations thereof.

2.36 SOIL ENGINEER -- Licensed Civil Engineer experienced in soil mechanics (also see Geotechnical Engineer).

2.37 STABILIZATION FILL -- A fill mass, the configuration of which is typically related to slope height and is specified by the standards of practice for enhancing the stability of locally adverse conditions. A stabilization fill is normally specified by minimum key width and depth and by maximum backcut angle. A stabilization fill may or may not have a backdrainage system specified.

2.38 SUBDRAIN -- Generally a pipe and gravel or similar drainage system placed beneath a fill in the alignment of canyons or former drainage channels.

2.39 SLOUGH -- Loose, noncompacted fill material generated during grading operations.

2.40 TAILINGS -- Nonengineered fill which accumulates on or adjacent to equipment haul-roads.

2.41 TERRACE -- Relatively level step constructed in the face of graded slope surface for drainage control and maintenance purposes.

2.42 TOPSOIL -- The presumably fertile upper zone of soil which is usually darker in color and loose.

2.43 WINDROW -- A string of large rock buried within engineered fill in accordance with guidelines set forth by the Geotechnical Consultant.

3. SITE PREPARATION

3.1 Clearing and grubbing should consist of the removal of vegetation such as brush, grass, woods, stumps, trees, roots to trees and otherwise deleterious natural materials from the areas to be graded. Clearing and grubbing should extend to the outside of all proposed excavation and fill areas.

3.2 Demolition should include removal of buildings, structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, mining shafts, tunnels, etc.) and other man-made surface and subsurface improvements

from the areas to be graded. Demolition of utilities should include proper capping and/or re-routing pipelines at the project perimeter and cutoff and capping of wells in accordance with the requirements of the governing authorities and the recommendations of the Geotechnical Consultant at the time of demolition.

3.3 Debris generated during clearing, grubbing and/or demolition operations should be wasted from areas to be graded and disposed off-site. Clearing, grubbing and demolition operations should be performed under the observation of the Geotechnical Consultant.

4. SITE PROTECTION

4.1 The Contractor should be responsible for the stability of all temporary excavations. Recommendations by the Geotechnical Consultant pertaining to temporary excavations (e.g., backcuts) are made in consideration of stability of the completed project and, therefore, should not be considered to preclude the responsibilities of the Contractor. Recommendations by the Geotechnical Consultant should not be considered to preclude more restrictive requirements by the regulating agencies.

4.2 Precautions should be taken during the performance of site clearing, excavations and grading to protect the work site from flooding, ponding or inundation by poor or improper surface drainage. Temporary provisions should be made during the rainy season to adequately direct surface drainage away from and off the work site.

4.3 During periods of rainfall, the Geotechnical Consultant 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.).

4.4 Following periods of rainfall, the Contractor should contact the Geotechnical Consultant and arrange a review of the site in order to visually assess rain related damage. The Geotechnical Consultant may also recommend excavations and testing in order to aid in his assessments.

4.5 Rain related damage should be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress and other adverse conditions identified by the Geotechnical

Consultant. Soil adversely affected should be classified as Unsuitable Materials and should be subject to overexcavation and replacement with compacted fill or other remedial grading as recommended by the Geotechnical Consultant.

5. EXCAVATIONS

5.1 UNSUITABLE MATERIALS

5.1.1 Materials which are unsuitable should be excavated under observation and recommendations of the Geotechnical Consultant. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic compressible natural soils and fractured, weathered, soft bedrock and nonengineered or otherwise deleterious fill materials.

5.1.2 Material identified by the Geotechnical Consultant as unsatisfactory due to its moisture conditions should be overexcavated, watered or dried, as needed, and thoroughly blended to a uniform near optimum moisture condition (as per guidelines reference 7.2.1) prior to placement as compacted fill.

5.2 CUT SLOPES

5.2.1 Unless otherwise recommended by the Geotechnical Consultant and approved by the regulating agencies, permanent cut slopes should not be steeper than 2:1 (horizontal:vertical).

5.2.2 If excavations for cut slopes expose loose, cohesionless, significantly fractured or otherwise unsuitable material, overexcavation and replacement of the unsuitable materials with a compacted stabilization fill should be accomplished as recommended by the Geotechnical Consultant. Unless otherwise specified by the Geotechnical Consultant, stabilization fill construction should conform to the requirements of the Standard Details.

5.2.3 The Geotechnical Consultant should review cut slopes during excavation. The Geotechnical Consultant should be notified by the contractor prior to beginning slope excavations.

5.2.4 If, during the course of grading, adverse or potentially adverse geotechnical conditions are encountered which were not anticipated in the preliminary report, the Geotechnical Consultant should explore, analyze and make recommendations to treat these problems.

6. COMPACTED FILL

All fill materials should be compacted to at least 90 percent of maximum density (ASTM D1557) unless otherwise recommended by the Geotechnical Consultant.

6.1 PLACEMENT

6.1.1 Prior to placement of compacted fill, the Contractor should request a review by the Geotechnical Consultant of the exposed ground surface. Unless otherwise recommended, the exposed ground surface should then be scarified (6-inches minimum), watered or dried as needed, thoroughly blended to achieve near optimum moisture conditions; then thoroughly compacted to a minimum of 90 percent of the maximum density.

6.1.2 Compacted fill should be placed in thin horizontal lifts. Each lift should be watered or dried as needed, blended to achieve near optimum moisture conditions then compacted by mechanical methods to a minimum of 90 percent of laboratory maximum dry density. Each lift should be treated in a like manner until the desired finished grades are achieved.

6.1.3 When placing fill in horizontal lifts adjacent to areas sloping steeper than 5:1 (horizontal: vertical), horizontal keys and vertical benches should be excavated into the adjacent slope area. Keying and benching should be sufficient to provide at least 6-foot wide benches and a minimum of 4-feet of vertical bench height within the firm natural ground, firm bedrock or engineered compacted fill. No compacted fill should be placed in an area subsequent to keying and benching until the area has been reviewed by the Geotechnical Consultant. Material generated by the benching operation should be moved sufficiently away from the bench area to allow for the recommended review of the horizontal bench prior to placement

fill. Typical keying and benching details have been included within the accompanying Standard Details.

6.1.4 Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a false slope, benching should be conducted in the same manner as above described. At least a 3-foot vertical bench should be established within the firm core adjacent approved compacted fill prior to placement of additional fill. Benching should proceed in at least 3-foot vertical increments until the desired finished grades are achieved.

6.1.5 Fill should be tested for compliance with the recommended relative compaction and moisture conditions. Field density testing should conform to accepted test methods. Density testing frequency should be adequate for the geotechnical consultant to provide professional opinions regardings fill compaction and adherence to recommendations. Fill found not to be in conformance with the grading recommendation should be removed or otherwise handled as recommended by the Geotechnical Consultant.

6.1.6 The Contractor should assist the Geotechnical Consultant and/or his representative by digging test pits for removal determinations and/or for testing compacted fill.

6.1.7 As recommended by the Geotechnical Consultant, the Contractor may need to remove grading equipment from an area being tested if personnel safety is considered to be a problem.

6.2 MOISTURE

6.2.1 For field testing purposes "near optimum" moisture will vary with material type and other factors including compaction procedure. "Near optimum" may be specifically recommended in Preliminary Investigation Reports and/or may be evaluated during grading.

6.2.2 Prior to placement of additional compacted fill following an overnight or other grading delay, the exposed surface or previously compacted

fill should be processed by scarification, watered or dried as needed, thoroughly blended to near-optimum moisture conditions, then recompacted to a minimum of 90 percent of laboratory maximum dry density. Where wet, dry, or other unsuitable materials exist to depths of greater than one foot, the unsuitable materials should be overexcavated.

6.2.3 Following a period of flooding, rainfall or overwatering by other means, no additional fill should be placed until damage assessments have been made and remedial grading performed as described under Section 5.6 herein.

6.3 FILL MATERIAL

6.3.1 Excavated on-site materials which are considered suitable to the Geotechnical Consultant may be utilized as compacted fill, provided trash, vegetation and other deleterious materials are removed prior to placement.

6.3.2 Where import fill materials are required for use on-site, the Geotechnical Consultant should be notified in advance of importing, in order to sample and test materials from proposed borrow sites. No import fill materials should be delivered for use on-site without prior sampling and testing notification by Geotechnical Consultant.

6.3.3 Where oversized rock or similar irreducible material is generated during grading, it is recommended, where practical, to waste such material off-site or on-site in areas designated as "nonstructural rock disposal areas". Rock placed in disposal areas should be placed with sufficient fines to fill voids. The rock should be compacted in lifts to an unyielding condition. The disposal area should be covered with at least three feet of compacted fill which is free of oversized material. The upper three feet should be placed in accordance with the guidelines for compacted fill herein.

6.3.4 Rocks 12 inches in maximum dimension and smaller may be utilized within the compacted fill, provided they are placed in such a manner

that nesting of the rock is avoided. Fill should be placed and thoroughly compacted over and around all rock. The amount of rock should not exceed 40 percent by dry weight retained on the 3/4-inch sieve size. The 12-inch and 40 percent recommendations herein may vary as field conditions dictate.

6.3.5 Where rocks or similar irreducible materials of greater than 12 inches but less than four feet of maximum dimension are generated during grading, or otherwise desired to be placed within an engineered fill, special handling in accordance with the accompanying Standard Details is recommended. Rocks greater than four feet should be broken down or disposed off-site. Rocks up to four feet maximum dimension should be placed below the upper 10 feet of any fill and should not be closer than 20-feet to any slope face. These recommendations could vary as locations of improvements dictate. Where practical, oversized material should not be placed below areas where structures or deep utilities are proposed. Oversized material should be placed in windrows on a clean, overexcavated or unyielding compacted fill or firm natural ground surface. Select native or imported granular soil (S.E. 30 or higher) should be placed and thoroughly flooded over and around all windrowed rock, such that voids are filled. Windrows of oversized material should be staggered so that successive strata of oversized material are not in the same vertical plane.

6.3.6 It may be possible to dispose of individual larger rock as field conditions dictate and as recommended by the Geotechnical Consultant at the time of placement.

6.3.7 The construction of a "rock fill" consisting primarily of rock fragments up to two feet in maximum dimension with little soil material may be feasible. Such material is typically generated on sites where extensive blasting is required. Recommendations for construction of rock fills should be provided by the Geotechnical Consultant on a site-specific basis.

6.3.8 During grading operations, placing and mixing the materials from the cut and/or borrow areas may result in soil mixtures which possess unique physical properties. Testing may be required of samples obtained directly from the fill areas in order to determine conformance with the specifications. Processing of these additional samples may take two or more working days. The Contractor may elect to move the operation to other areas within the project, or may continue placing compacted fill pending laboratory and field test results. Should he elect the second alternative, fill placed is done so at the Contractor's risk.

6.3.9 Any fill placed in areas not previously reviewed and evaluated by the Geotechnical Consultant may require removal and recompaction. Determination of overexcavations should be made upon review of field conditions by the Geotechnical Consultant.

6.4 FILL SLOPES

6.4.1 Permanent fill slopes should not be constructed steeper than 2:1 (horizontal to vertical), unless otherwise recommended by the Geotechnical Consultant and approved by the regulating agencies.

6.4.2 Fill slopes should be compacted in accordance with these grading guidelines and specific report recommendations. Two methods of slope compaction are typically utilized in mass grading, lateral over-building and cutting back, and mechanical compaction to grade (i.e. sheepsfoot roller backrolling). Constraints such as height of slope, fill soil type, access, property lines, and available equipment will influence the method of slope construction and compaction. The geotechnical consultant should be notified by the contractor what method will be employed prior to slope construction.

Slopes utilizing over-building and cutting back should be constructed utilizing horizontal fill lifts (reference Section 6) with compaction equipment working as close to the edge as practical. The amount of lateral over-building will vary as field conditions dictiate. Compaction testing of slope faces will be required and

reconstruction of the slope may result if testing does not meet our recommendations.

Mechanical compaction of the slope to grade during construction should utilize two types of compactive effort. First, horizontal fill lifts should be compacted during fill placement. This equipment should provide compactive effort to the outer edge of the fill slope. Sloughing of fill soils should not be permitted to drift down the slope. Secondly, at intervals not exceeding four feet in vertical slope height or the capability of available equipment, whichever is less, fill slopes should be backrolled with a sheepsfoot-type roller. Moisture conditions of the slope fill soils should be maintained throughout the compaction process. Generally upon slope completion, the entire slope should be compacted utilizing typical methods, (i.e. sheepsfoot rolling, bulldozer tracking, or rolling with rubber-tired heavy equipment). Slope construction grade staking should be removed as soon as possible in the slope compaction process. Final slope compaction should be performed without grade sakes on the slope face.

In order to monitor slope construction procedures, moisture and density tests will be taken at regular intervals. Failure to achieve the desired results will likely result in a recommendation by the Geotechnical Consultant to overexcavate the slope surfaces followed by reconstruction of the slopes utilizing over-filling and cutting back procedures or further compactive effort with the conventional backrolling approach. Other recommendations may also be provided which would be commensurate with field conditions.

6.4.3 Where placement of fill above a natural slope or above a cut slope is proposed, the fill slope configuration as presented in the accompanying Standard Details should be adopted.

6.4.4 For pad areas above fill slopes, positive drainage should be established away from the top-of-slope, as designed by the project civil engineer.

6.5 OFF-SITE FILL

6.5.1 Off-site fill should be treated in the same manner as recommended in the specifications for site preparation, excavation, drains, compaction, etc.

6.5.2 Off-site canyon fill should be placed in preparation for future additional fill, as shown in the accompanying Standard Details.

6.5.3 Off-site fill subdrains temporarily terminated (up canyon) should be surveyed for future relocation and connection.

6.6 TRENCH BACKFILL

6.6.1 Utility trench backfill should, unless otherwise recommended, be compacted by mechanical means. Unless otherwise recommended, the degree of compaction should be a minimum of 90 percent of maximum density (ASTM D1557).

6.6.2 Backfill of exterior and interior trenches extending below a 1:1 projection from the outer edge of foundations should be mechanically compacted to a minimum of 90 percent of the laboratory maximum density.

6.6.3 Within slab areas, but outside the influence of foundations, trenches up to one foot wide and two feet deep may be backfilled with sand (S.E. > 30), and consolidated by jetting, flooding or by mechanical means. If on-site materials are utilized, they should be wheel-rolled, tamped or otherwise compacted to a firm condition. For minor interior trenches, density testing may be deleted or spot testing may be elected if deemed necessary, based on review of backfill operations during construction.

6.6.4 If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, the Contractor may elect the utilization of light weight mechanical compaction equipment and/or shading of the conduit with clean, granular material, (S.E. > 30) which should be thoroughly moistened in the trench, prior to

initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review of the Geotechnical Consultant at the time of construction.

6.6.5 In cases where clean granular materials are proposed for use in lieu of native materials or where flooding or jetting is proposed, the procedures should be considered subject to review by the Geotechnical Consultant.

6.6.6 Clean granular backfill and/or bedding are not recommended in slope areas unless provisions are made for a drainage system to mitigate the potential build-up of seepage forces and piping.

7. DRAINAGE

7.1 Canyon subdrain systems recommended by the Geotechnical Consultant should be installed in accordance with the Standard Details.

7.2 Typical subdrains for compacted fill buttresses, slope stabilizations or sidehill masses, should be installed in accordance with the specifications of the accompanying Standard Details.

7.3 Roof, pad and slope drainage should be directed away from slopes and areas of structures to disposal areas via suitable devices designed by the project civil engineer (i.e., gutters, downspouts, concrete swales, area drains, earth swales, etc.).

7.4 Drainage patterns established at the time of fine grading should be maintained throughout the life of the project. Property owners should be made aware that altering drainage patterns can be detrimental to slope stability and foundation performance.

8. SLOPE MAINTENANCE

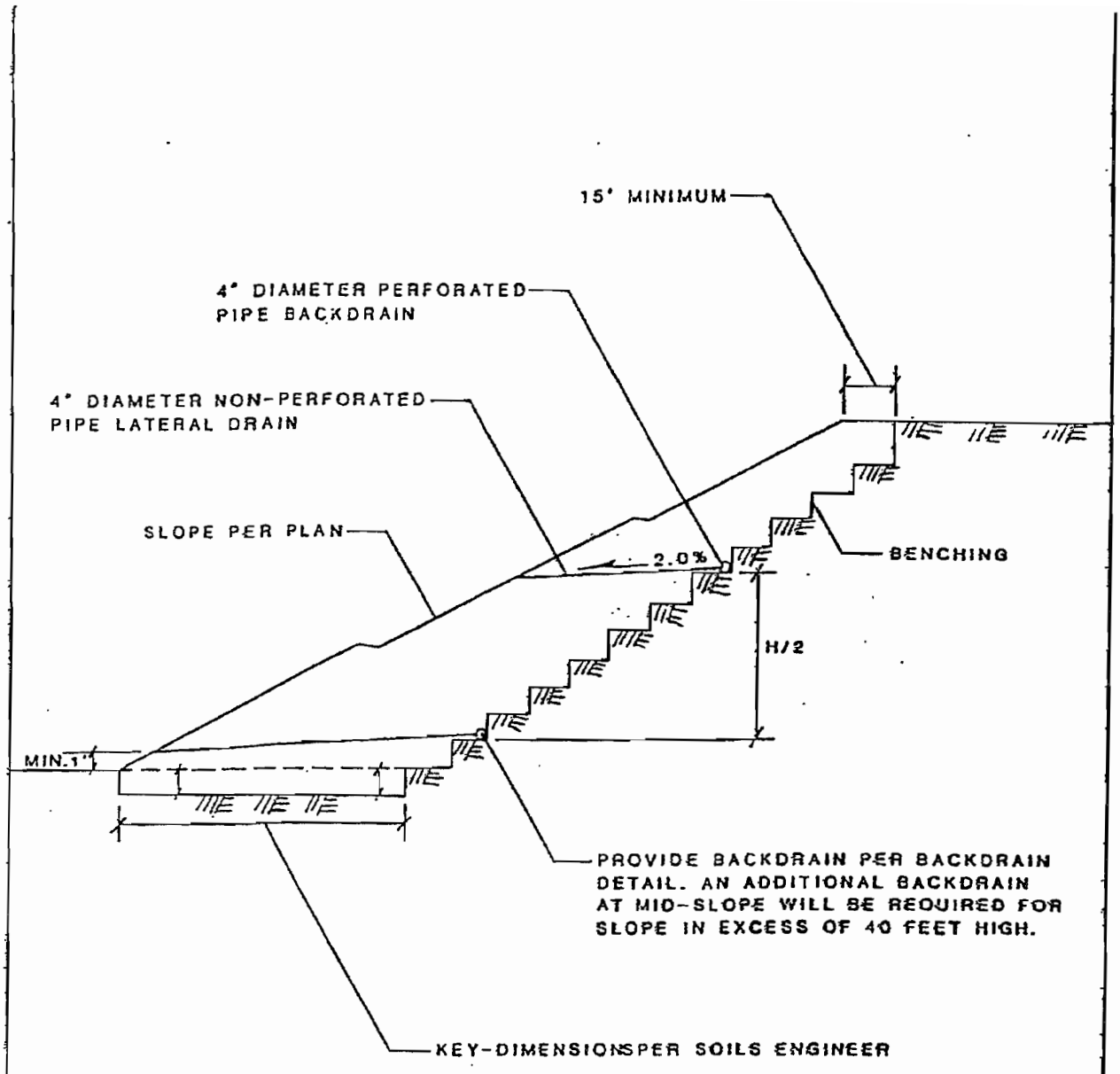
8.1 LANDSCAPE PLANTS

In order to decrease erosion surficial slope stability problems, slope planting should be accomplished at the completion of grading. Slope planting should consist of deep-rooting vegetation requiring little watering. A Landscape Architect would be the test party to consult regarding actual types of plants and planting configuration.

8.2 IRRIGATION

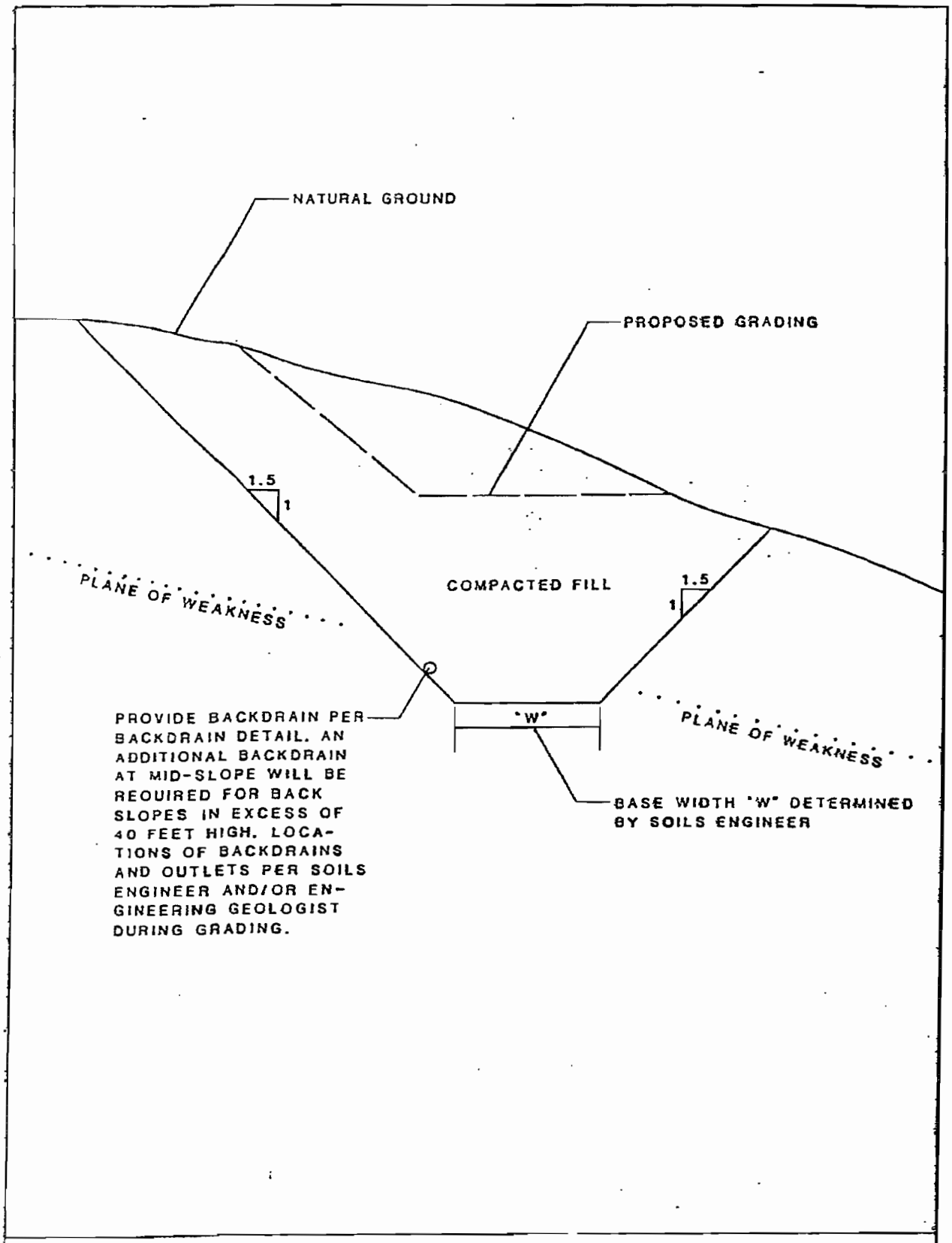
8.2.1 Slope irrigation should be minimized. If automatic timing devices are utilized on irrigation systems, provisions should be made for interrupting normal irrigation during periods of rainfall.

8.2.2 Property owners should be made aware that overwatering of slopes is detrimental to slope stability and may contribute to slope seepage, erosion and siltation problems in the subdivision.



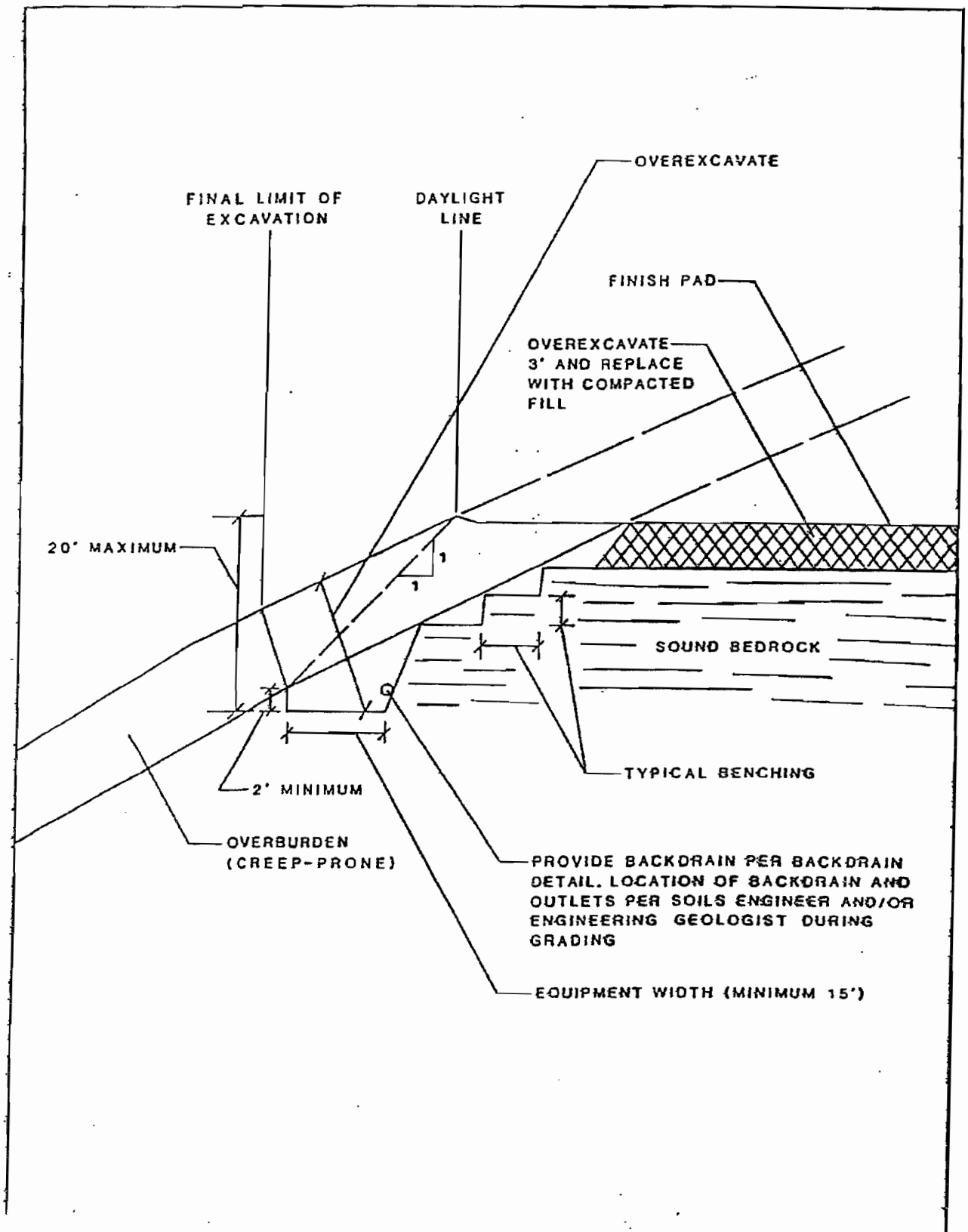
TYPICAL BUTTRESS OR STABILIZATION FILL DETAIL

JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 1



TYPICAL SHEAR KEY DETAIL

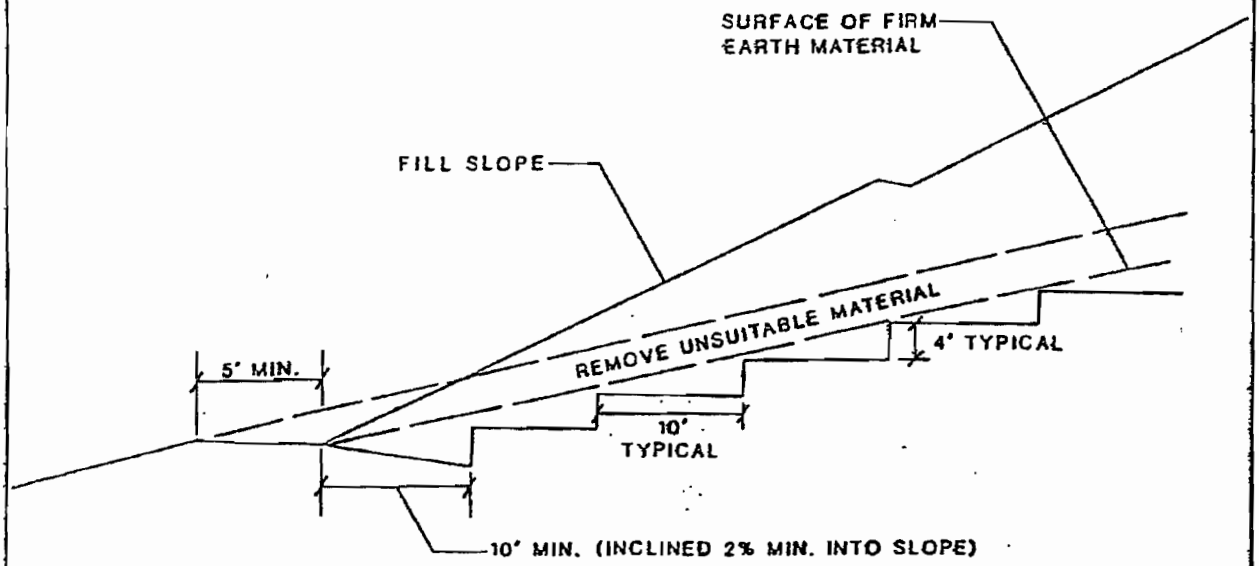
JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 2



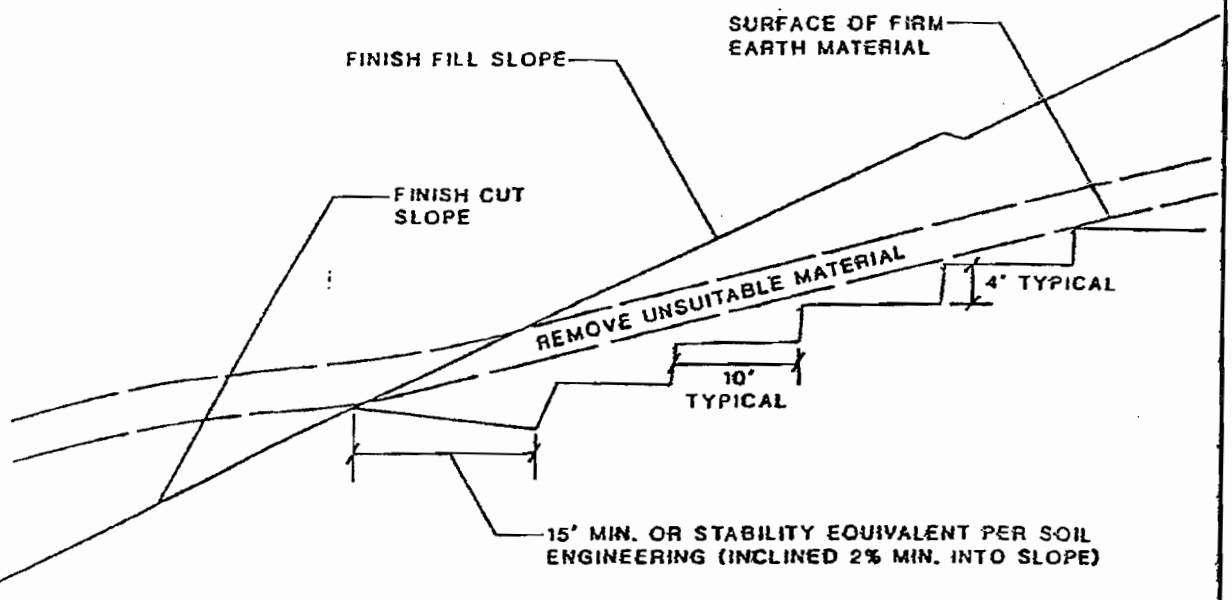
DAYLIGHT SHEAR KEY DETAIL

JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 3

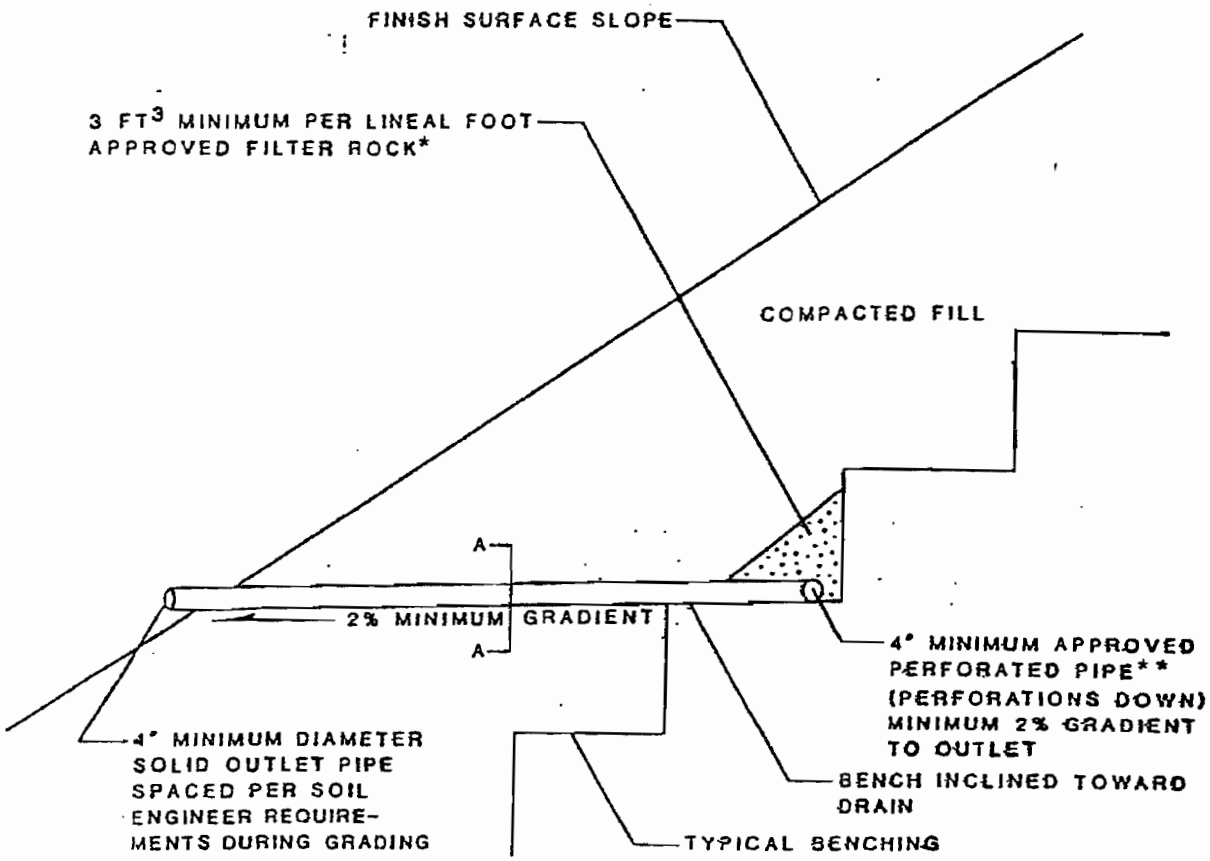
BENCHING FILL OVER NATURAL



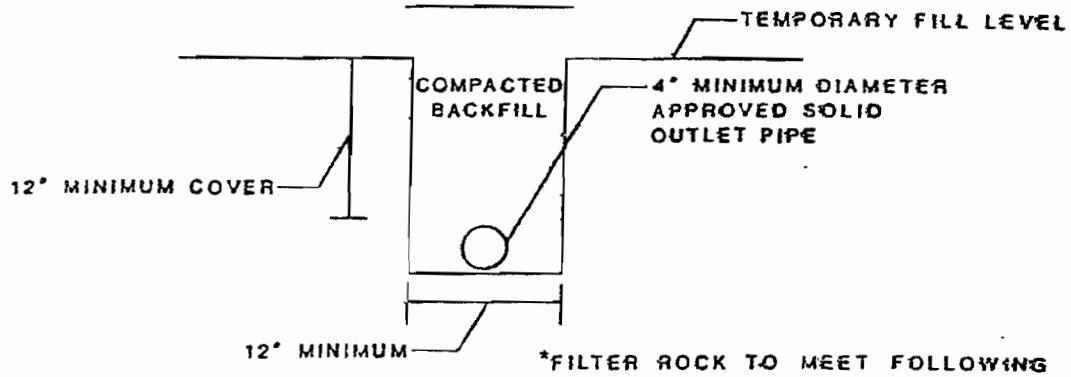
BENCHING FILL OVER CUT



BENCHING FOR COMPACTED FILL DETAIL		
JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 4



DETAIL A-A



*FILTER ROCK TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

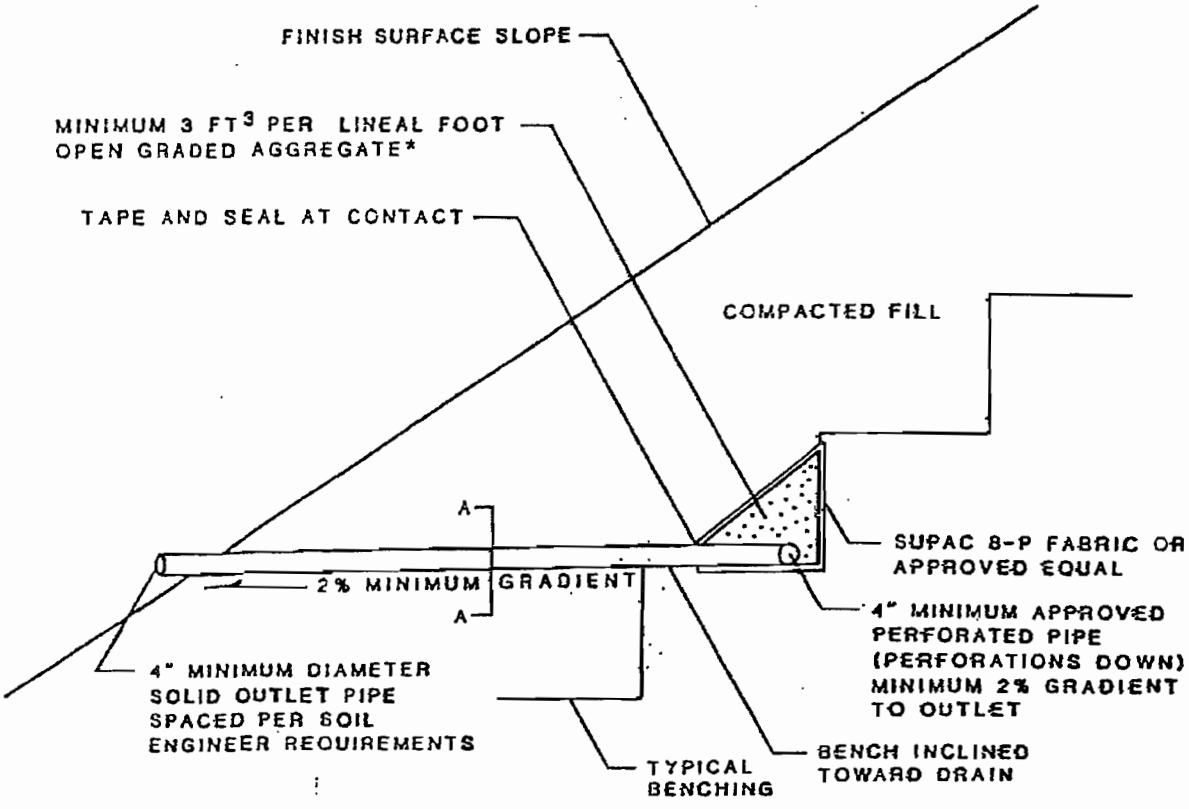
SIEVE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO.4	25-40
NO.30	5-15
NO.50	0-7
NO.200	0-3

**APPROVED PIPE TYPE:

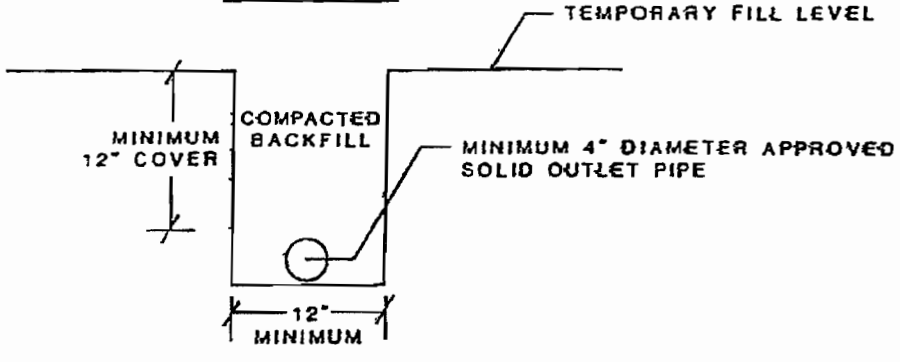
SCHEDULE 40 POLYVINYL CHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 PSI.

TYPICAL BACKDRAIN DETAIL

JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 5



DETAIL A-A



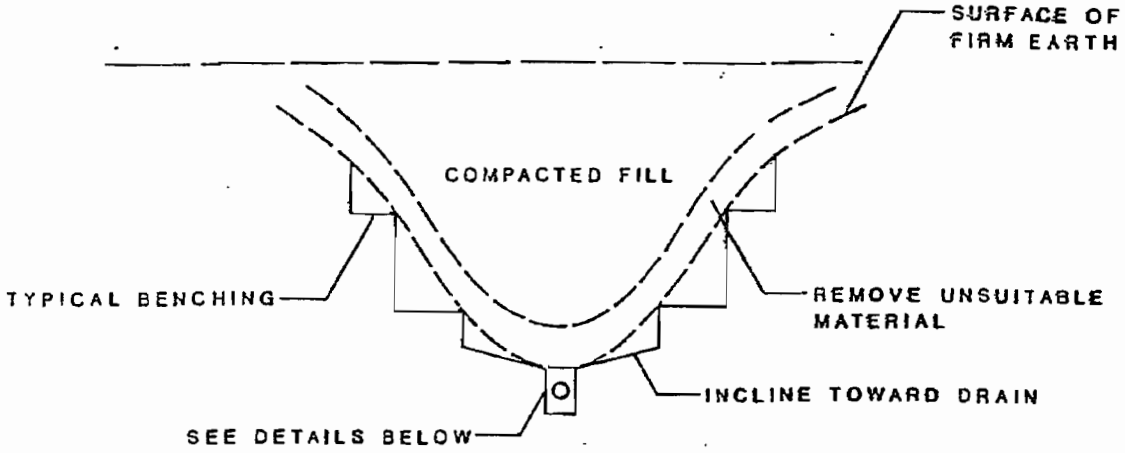
* NOTE: AGGREGATE TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1 1/2"	100
1"	5-40
3/4"	0-17
3/8"	0-7
NO. 200	0-3

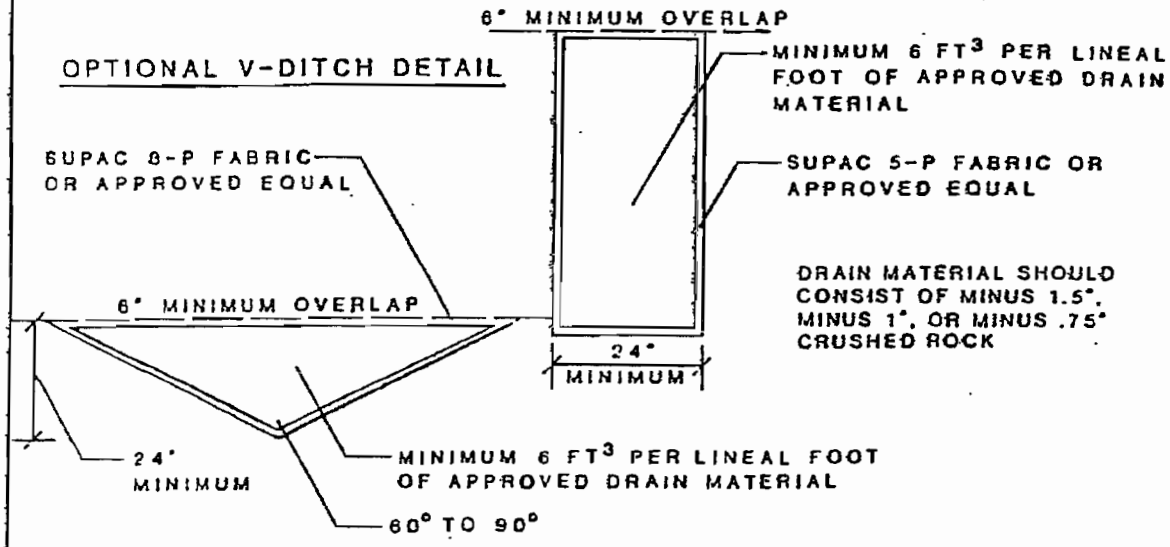
BACKDRAIN DETAIL (GEOFABRIC)

JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 6

CANYON SUBDRAIN DETAILS



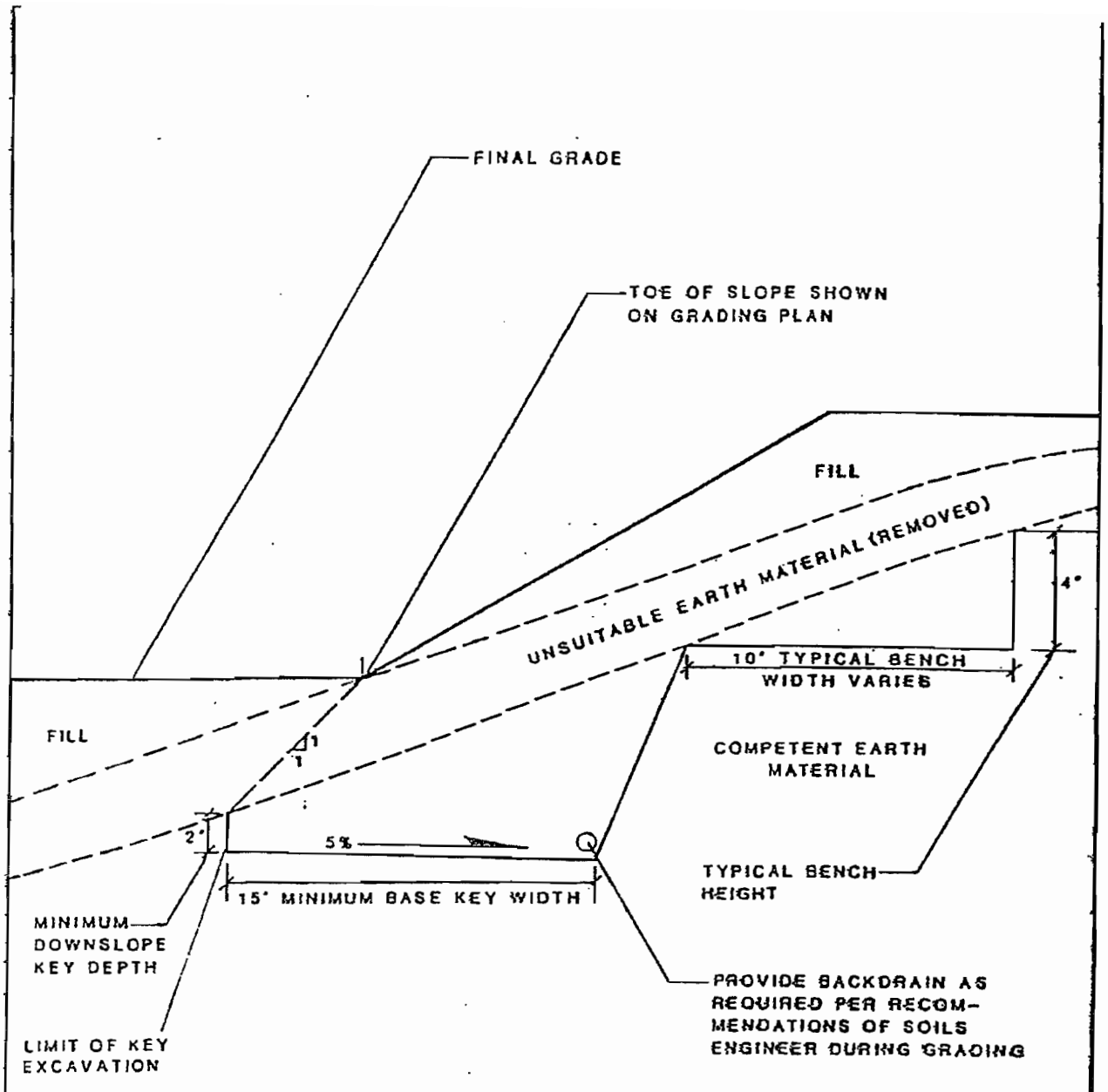
TRENCH DETAIL



ADD MINIMUM 4" DIAMETER APPROVED PERFORATED PIPE WHEN LARGE FLOWS ARE ANTICIPATED

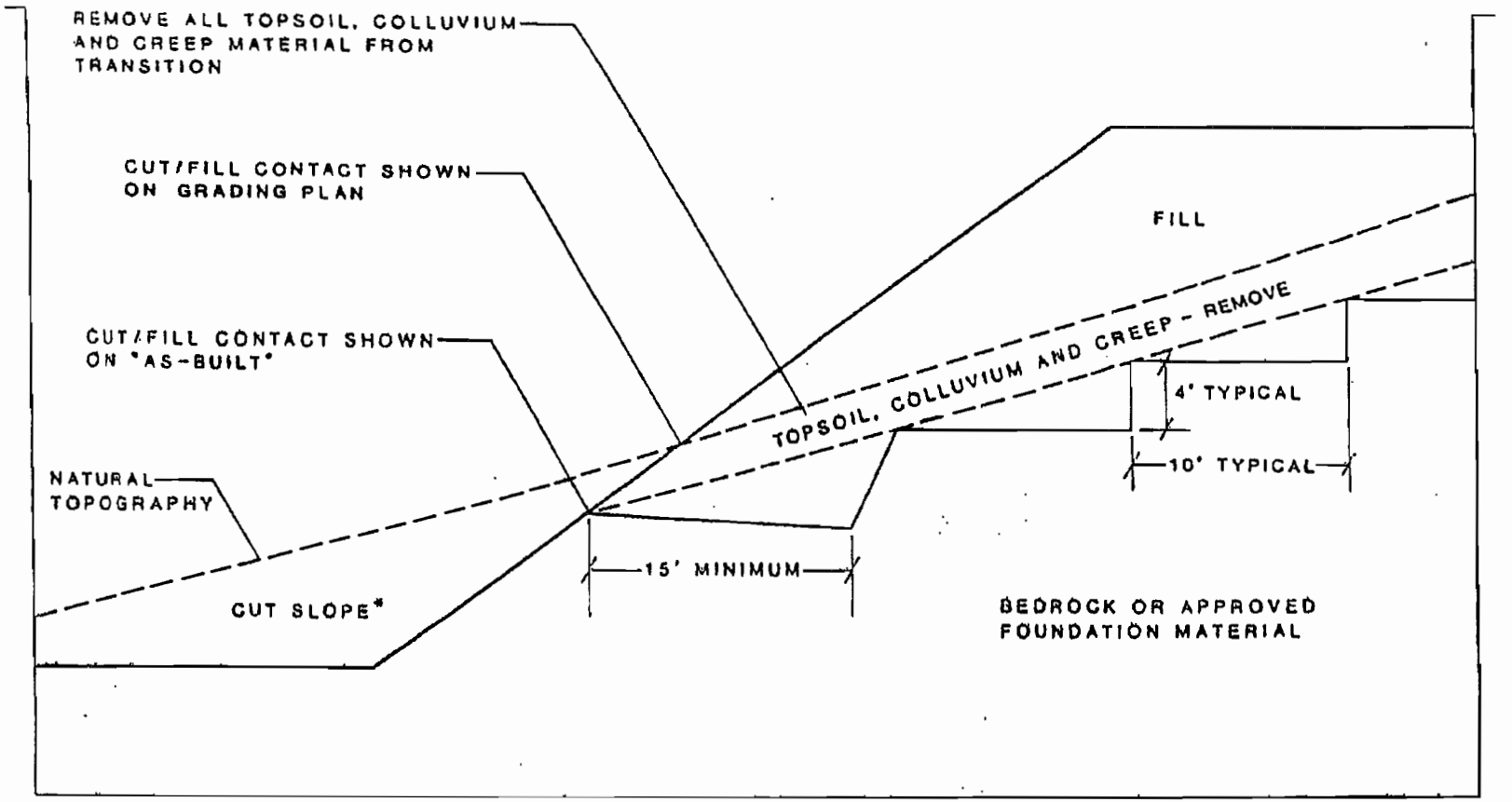
APPROVED PIPE TO BE SCHEDULE 40 POLY-VINYL-CHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 psi.

GEOFABRIC SUBDRAIN		
JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 7



WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY. HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.

FILL SLOPE ABOVE NATURAL GROUND DETAIL		
JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 8



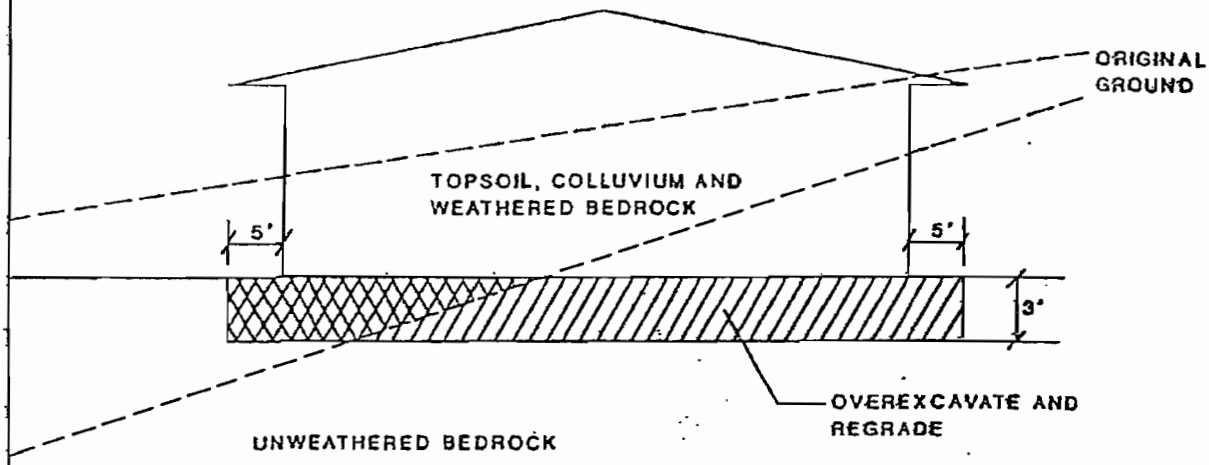
* NOTE: CUT SLOPE PORTION SHALL BE MADE PRIOR TO PLACEMENT OF FILL

FILL SLOPE ABOVE CUT SLOPE DETAIL

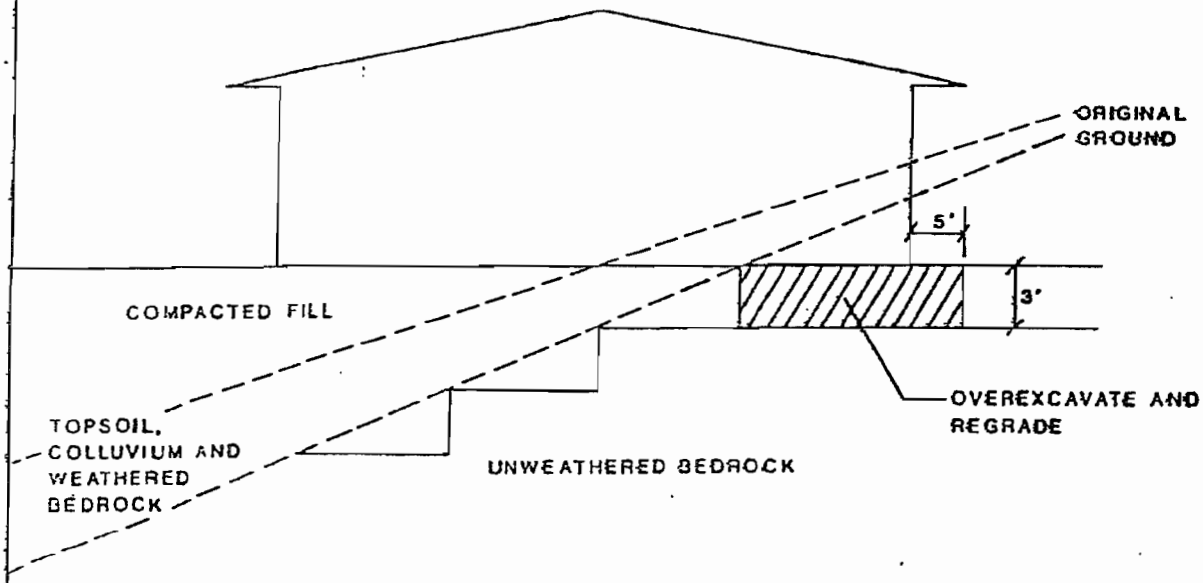
JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 9

GENERAL GRADING RECOMMENDATIONS

CUT LOT

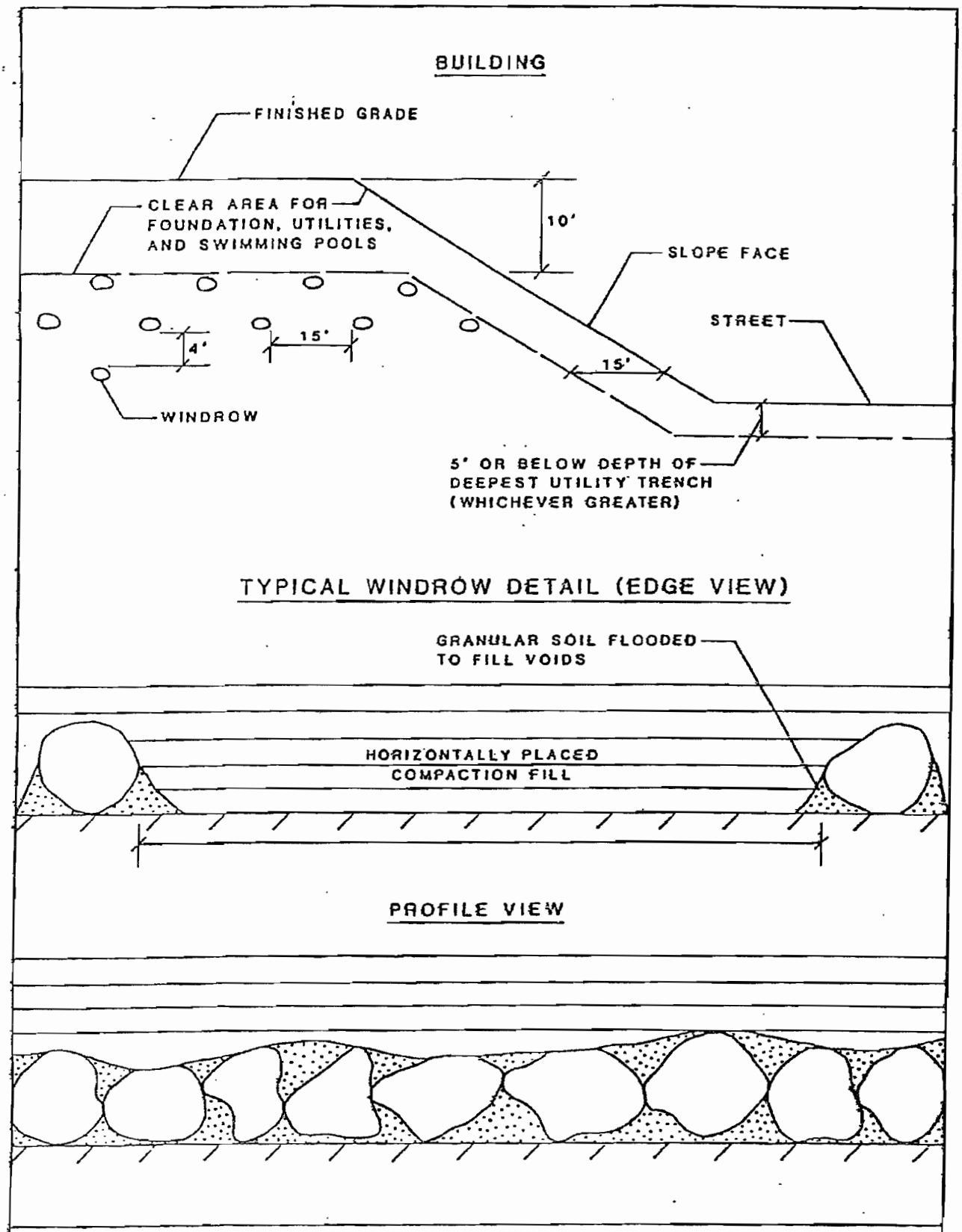


CUT/FILL LOT (TRANSITION)



TRANSITION LOT DETAIL

JOB NO.:	DATE:	FIGURE:
04-3529-001-00-00	MARCH 1991	10



ROCK DISPOSAL DETAIL

JOB NO.: 04-8529-001-00-00	DATE: MARCH 1991	FIGURE: 11

TABLE C-1

Results of Moisture Density Relationships ASTM D-1557

Sample Location	Sample Description	Maximum Density (lb/ft^3)	Optimum Moisture Content (%)
B4 @ 3'	Brown Silty Sand	132	8.7

TABLE C-2

Results of pH and Resistivity Tests

Sample Location	pH	Resistivity (ohm-cm)	Sulfate (ppm)
B4 @ 3'	8.22	1470	

EXHIBIT D

SGC *Southland Geotechnical Consultants*

GEOTECHNICAL EVALUATION OF COASTAL BLUFF PROPERTY VACANT PARCEL SOUTH OF 5305 CARLSBAD BOULEVARD CARLSBAD, CALIFORNIA

Project No. 126G21

March 10, 1997

PLAN CHECK NO. 1 006677

Prepared for:

L.D. RICHARDS CO., BUILDER
1830 Oxford Avenue
Cardiff, California 92007

BOB WOJACK
438 1161 x 4333

Pg 8978982 944 0765

• 1238 GREENFIELD DRIVE, SUITE A EL CAJON, CALIFORNIA 92021 •
(619)442-8022 • FAX (619)442-7859

SGC Southland Geotechnical Consultants

**GEOTECHNICAL EVALUATION
OF COASTAL BLUFF PROPERTY
VACANT PARCEL SOUTH OF
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SGC Southland Geotechnical Consultants

March 10, 1997 Project No. 126G21

To: L.D. Richards Co., Builder
1830 Oxford Avenue
Cardiff, California 92007

Attention: Mr. Larry Richards

Subject: Geotechnical Evaluation of Coastal Bluff Property, Vacant Parcel South of 5305 Carlsbad Boulevard, Carlsbad, California

Introduction

In accordance with your request, Southland Geotechnical Consultants has performed a geotechnical evaluation of the subject coastal bluff property. We understand that a single-family residence is proposed on the property. This report presents a summary of our field and research studies and our conclusions and recommendations relative to the proposed development. We understand that soils engineering services for foundation design for the proposed residence are being performed by others.

Purpose and Scope

This report presents the results of our geotechnical evaluation of the coastal bluff property located south of 5305 Carlsbad Boulevard in Carlsbad. The purpose of our study was to evaluate the geotechnical conditions at the coastal bluff property and provide recommendations relative to the proposed construction. The scope of our geotechnical evaluation included the following:

- Review of aerial photographs, geologic/topographic maps, and geologic literature pertaining to the site and vicinity. A list of the items reviewed is presented in Appendix A.
- Geologic reconnaissance to observe the existing site conditions including the coastal bluff and general vicinity.
- Preparation of a tape and compass profile of the bluff face.
- Geotechnical analysis of the data obtained including a computer-generated slope stability analysis of the coastal bluff.
- Preparation of this report summarizing the results of our geotechnical evaluation of the coastal bluff property.

• 1238 GREENFIELD DRIVE, SUITE A EL CAJON, CALIFORNIA 92021 •
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Site Description

The subject coastal bluff property is known as San Diego County Assessor's Parcel Number 210-120-31. The roughly rectangular property is located south of the existing residential development located at 5305 Carlsbad Boulevard in the City of Carlsbad (see Figure 1). The parcel to the south of the subject lot is currently undeveloped. The eastern property line at the site is located along the westerly side of Carlsbad Boulevard (formerly Highway 101) and is approximately 60 feet long. The relatively level bluff-top area of the property extends, from Carlsbad Boulevard to the top of the coastal bluff, approximately 230 feet along the northerly property line and approximately 200 feet along the southerly property line. An approximately 40-foot high coastal bluff with an overall gradient of approximately 45 degrees is located in the western portion of the property (see Photos 1 and 2). The approximate elevation of the bluff edge is about 40 feet above sea level based on the City of Carlsbad 1991 orthophoto map (Appendix A).

In general, the bluff-top area at the site slopes gently towards the west and is mostly vegetated with iceplant, weeds and scattered brush. On February 12, 1997, SGC representatives made approximate measurements of the western portion of the bluff-top area at the site. The resultant site plan (with the approximate location of the bluff edge) is included as Figure 2. Please note that the bluff edge on the site is somewhat obscured by vegetative growth and rodent burrowing activities (see Photos 3 and 4). We emphasize that our measurements are approximate. Figure 2 is not a surveyed map.

Bluff Description

During our site visit on February 12, 1997, a tape and compass profile of the coastal bluff on the property was prepared. The results of our approximate measurements are presented on Figure 3 (Coastal Bluff Profile). Following is a summary of the onsite bluff conditions.

The approximately 40-foot high coastal bluff slopes at an overall gradient of approximately 45 degrees (from the base of the seacliff to the upper bluff edge). There is an approximately 10-foot high, irregular but overall, near-vertical seacliff at the base of the coastal bluff (see Photos 1 and 2). This seacliff is not vegetated. Portions of the seacliff are slightly undercut apparently by wave action. Indications of minor sea cave development in the seacliff were observed at the site (see Photos 1, 2 and 5). A relatively level "bench" area, approximately 5 to 15 feet wide, has developed at the contact between the more resistant Santiago Formation sandstone that comprises the seacliff and the overlying, less resistant terrace deposits that comprise the upper portion of the bluff (see Photos 1 and 2). The upper approximately 30 vertical feet of the coastal bluff generally is sparsely vegetated with

iceplant and weeds. However, the uppermost portion of the bluff and the bluff edge are generally moderately to well vegetated with iceplant. Gunite covers the bluff face on the properties to the north and extends approximately 15 feet onto the subject coastal bluff (see Photos 1, 2 and 6).

Proposed Development

Project plans have not yet been finalized. However, based on our conversations with you, it is our understanding that a one- to two-story residence is proposed at the site. We understand that the residence will be set back a minimum of 25 feet from the bluff edge. We also understand that the residence will consist of typical, relatively light, residential construction.

Geologic Units

Based on our review of a geologic map (Appendix A, Reference 13) and our onsite observations, the property appears to be underlain by Eocene-aged Santiago Formation sandstone overlain by Quaternary-aged terrace deposits. Surficial deposits consisting of beach deposits along the toe of the seacliff were observed during our site visit and fill soils were observed on the bluff face and bluff-top area. The approximate limits of these units, as observed in our onsite studies, are shown on Figure 3 and are described below:

- Santiago Formation - The Eocene-aged Santiago Formation is exposed in the approximately 10-foot high, irregular but near-vertical seacliff located just easterly of the beach in the western portion of the site. The Santiago Formation sandstone generally consists of a light gray-brown, well-cemented, silty, fine-grained sandstone. The Santiago Formation is finer grained near its contact with the overlying terrace deposits. Localized concretions and cross-bedding were observed in the seacliff face.

- Terrace Deposits - Quaternary-aged terrace deposits unconformably overlie the Santiago Formation and comprise the majority of the bluff face. The terrace deposits consist of orange-brown to light brown, dense but friable, slightly silty fine- to medium-grained sand. Near the base of the terrace deposits, talus (or slopewash) derived from the terrace deposits has accumulated by the downslope actions of surface waters and gravity.

- Beach Deposits - A variable thickness of unconsolidated beach deposits occur on the beach at the base of the seacliff. During our site visits, the beach deposits consisted of sand. This material is subject to addition and removal in response to storm waves and currents.

- Fill Soils - Fill soils exist locally on the bluff face and bluff-top area (not shown on Figure 3). It appears that minor amounts of fill soils may have been used to infill two relatively small gullies at the site (see Photos 1 and 2). In addition, fill soils were apparently pushed over the bluff edge onto the bluff face. The fill soils generally consist of locally-derived terrace deposit materials and chunks of concrete and asphalt.

Geologic Structure

The Santiago Formation is well exposed in the seacliff at the property and to the south. The Santiago Formation at and near the subject property is nearly flat-lying. Bedding in the Quaternary terrace deposits can be observed as alternating more resistant and less resistant beds. Where observed on site and in the general site vicinity, the terrace deposits appear to be horizontally bedded with localized cross bedding.

Fractures were observed in the sea cliff and a dominant near-vertical joint orientation of N50W was measured. However, no major out-of-slope bedding components were observed that would be adverse to slope stability. Indications of deep-seated landslide features were not observed during our research studies or site visits.

Review of Geotechnical Hazards/Constraint Map

Our review of Geotechnical Hazards/Constraint Map page 14 of the City of Carlsbad Geotechnical Hazards Analysis and Mapping Study (Appendix A) indicates that the site is located in Hazard Categories 41 and 53. Category 41 pertains to "coastal stability: moderately stable: high steep bluffs, unfavorable structure, moderate to rapid erosion." Category 53 pertains to areas with "other terrain conditions: generally stable: relatively level mesa areas underlain by terrace deposits, sandstone or granitic/metavolcanic bedrock." Please note that the Geotechnical Hazards Analysis and Mapping Study is primarily intended for planning purposes and should not be construed as definitive data for a specific site.

Faulting

Our review of geologic literature (Appendix A) pertaining to the general site area indicates that there are no known "active" faults on or in the immediate vicinity of the site. An "active" fault is defined by the California Division of Mines and Geology as one which has "had surface displacement within Holocene time (about the last 11,000 years)" (Appendix A, Reference 4). Indications of active faulting were not observed in the subject coastal bluff. The nearest known active faults are the Rose

Canyon fault located offshore approximately 4 miles west of the site, the Coronado Bank fault located offshore approximately 19.5 miles west, and the Elsinore fault located approximately 25 miles northeast of the site. The San Andreas fault is located approximately 62 miles northeast of the site.

Our review of the City of Carlsbad Geotechnical Hazards Analysis and Mapping Study (Appendix A) indicates that a relatively short, northerly-trending fault trace has been mapped on and near the site. It appears to coincide with the gully located off site to the south of the subject property (see Figure 1). We did not observe direct, definitive evidence for the presence of this fault during our site visits, however, it may be obscured by fill soils present in the offsite gully. The City of Carlsbad study states that "all faults that have been mapped within the City limits based on current knowledge are all considered inactive". Based on its relatively short length and northerly trend, it is our opinion that this mapped fault does not present a seismic-source hazard to the subject project.

Tsunami and Storm Waves

Tsunami are sea waves generated by submarine earthquakes, landslides or volcanic action. Submarine earthquakes are common along the edge of the Pacific Ocean and coastal areas are subject to potential inundation by tsunami. Most of the 19 tsunami recorded on the San Diego tidal gauge (between 1854 to 1872 and 1906 to 1977) have only been a few tenths of a meter in height (Appendix A, Reference 1). The largest San Diego area tidal gauge excursion (1 meter) was associated with the tsunami of May 22, 1960 and was recorded at La Jolla (Scripps Pier) (Appendix A, Reference 12). The tsunami was generated by a Richter magnitude 8.5 earthquake in Chile. For comparison, the diurnal range of tides at San Diego Bay is 1.7 meters. The possibility of a destructive tsunami along the San Diego coastline is considered low (Appendix A, Reference 5). However, tsunami or storm waves (associated with winter storms), in conjunction with high tides, may erode the coastal bluff face (especially the friable terrace deposits) but generally are not anticipated to have the potential for inundation of the bluff-top building site.

Groundwater and Surface Water

During our site visits, groundwater seepage was observed in the coastal bluff near the southern edge of the gunite coating on the northern portion of the site (see Photo 6). Based on our experience and observations, our estimation to the depth of groundwater is that it is at or near sea level and may also be perched locally on the Santiago Formation at its contact with the overlying terrace deposits. Groundwater levels can be expected to fluctuate with the tides, seasonal precipitation and irrigation. Groundwater is not expected to be a constraint to construction of the

proposed residence. However, our experience indicates that near-surface groundwater conditions can develop in areas where no such groundwater conditions previously existed, especially in areas where a substantial increase in surface water infiltration results from landscape irrigation or unusually heavy precipitation.

The bluff-top surface waters appear to primarily drain towards the west as sheet flow. As they approach the bluff edge on the site, the surface waters are concentrated into generally shallow gullies that have been eroded into the bluff edge and discharge down the bluff face.

Historic Research Summary

We have reviewed the literature, maps and aerial photographs of the site and general vicinity listed in Appendix A. Following is a limited outline summary of our review observations:

- The oldest map we found on file at the County of San Diego is an 1898 survey for the Fifth Road District (Appendix A). The bluff along the coastline is sketched on the map and the railway line is shown.

- The oldest photograph we reviewed was one from the 1928-29 aerial photograph set on file at the County of San Diego. The railroad and coastal highway (approximately the current Carlsbad Boulevard) are shown on this photo. A dirt road west of the coastal highway, roughly parallel to the coastline, and along the bluff top in the general site vicinity is shown. A wider, cleared spot on this dirt road (a turn-out or viewpoint?) is located in the vicinity of the subject coastal bluff property. The offsite gully to the south is apparent.

- The subdivision roadway (Shore Drive) and some of the subdivision homes to the north of the subject property are shown on the 1953 aerial photographs. The single-family residence that currently exists adjacent to and north of the subject lot is not shown. A dirt road exists along the bluff edge in the site vicinity. The subject property appears it may have been cleared and used for agricultural purposes. A relatively small gully perpendicular to the bluff edge is apparent near the southern boundary of the subject property. The larger offsite gully (approximately 25 feet to the south of the site) is also apparent on the photos.

- The existing residence to the north of the subject property is shown on the April 9, 1964 photograph and subsequent photos. On the 1964 photo and photos from the late 1960's, 1970's and early 1980's and on the County's 1975 orthophoto topographic map, the subject coastal bluff face appears to be generally moderately vegetated. The bluff face appears generally smooth and

- uniform as it extends from the bluff edge to the westward edge of the near-vertical seacliff at its base. The currently existing "bench" on the resistant Santiago Formation is not apparent.

- Based on our review of a 1978 photo looking north towards the site (Appendix A, Reference 11), gunite appears to cover the lower part of the coastal bluff at the site and on the properties to the north and south of the site.

- Our review of the Army Corps of Engineers' report, "Coastal Cliff Sediments, San Diego Region" (Appendix A, Reference 11), indicates that an episode of erosion occurred between August 7 and 9, 1983, "as a result of waves from a southern hemisphere storm." In the 'cove' area, approximately 100 feet south of the site, the report states that "the cliff retreated 28 to 35 feet" and "this was more erosion at the site than had occurred during the previous 40-year period." The photos show that, at the subject property, a "bench" developed on the resistant Santiago Formation as a result of landward retreat (approximately 15 to 20 feet) of the toe of the terrace deposits at the site. However, even though the toe of the terrace deposits retreated, the upper portion of the bluff and bluff edge do not appear to have eroded, based on our review of the photos.

- The "bench" on the resistant Santiago Formation is shown on the 1984 and more recent photos and was observed during our site visits.

- On the 1991 orthophoto mapping sheet and 1992 photographs, the bluff-top area is vegetated generally similarly to what we observed during our recent site visits. However, the bluff edge and bluff face were less vegetated in the early 1990's than observed during our 1997 site visits.

Coastal Bluff Retreat

The Carlsbad coastline in the vicinity of the subject property consists of a slight headland to the north of the site and is indented (into a 'cove') to the south of the subject property (see Figure 1). The site is located on the northern edge of the indentation in the coast (concave to the ocean). Mechanisms for seacliff retreat at the site include slow abrasion and undercutting by marine erosion (wave action) of the hard, erosion-resistant Santiago Formation sandstone bedrock exposed in the near-vertical seacliff. Storm surf and higher high tides contribute to the natural process of marine erosion. Other factors affecting the rate of retreat of a near-vertical seacliff at the toe of a coastal bluff include degree of fracturing, jointing, consolidation of sediments, steepness of slope, groundwater and surface water conditions, vegetation or lack of, and intensity of pedestrian and animal traffic.

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In response to the landward retreat of the seacliff, the overlying coastal bluff becomes undermined and also retreats landward. During storm surf and higher tides, the base of the terrace deposits at the site are also subject to marine erosion. Other mechanisms contributing to bluff retreat include failure of overhanging bedrock projections, shallow failure of oversteepened portions of the bluff-face terrace deposits, and rilling and ravelling of the terrace deposits. Portions of coastal bluffs are also exposed to precipitation, wind, pedestrian/animal erosion (including foot traffic and burrowing rodents), variations in landscape, landscape maintenance, and other activities by humans.

During our studies, we did not observe indications of deep-seated instability, such as ancient or active landslides, on the site, and the Santiago Formation sandstone that comprises the seacliff at the site is not known to be prone to large, deep-seated failures. The terrace deposits are friable and commonly rill and ravel in oversteepened slopes, however, they are not known to be prone to large, deep-seated failures.

Coastal Bluff-Edge Retreat Rates

The rate and magnitude of coastal bluff retreat at a specific site are dependent on a variety of factors, both natural and manmade. Many of these factors are ongoing processes and historic documentation can be helpful in estimating general bluff-edge retreat rates. However, there are other factors affecting coastal bluff retreat that cannot be estimated from historic documentation. Such factors include future human activities or possible extreme variations in regional weather patterns.

Detrimental changes in factors affecting bluff-edge retreat, such as misdirected drainage, water line breaks, very heavy storm surf and/or precipitation, could increase the rate of future erosion. However, favorable changes in the factors affecting bluff-edge retreat could decrease the rate of future erosion. Some of these include eliminating detrimental human activities on the bluff, proper maintenance of a bluff-stabilizing vegetative cover, enhanced site drainage provisions and beach sand replenishment currently proposed by San Diego County coastal communities.

Research studies along the San Diego coast and historic photograph and map review are components in providing an estimation of the rate of bluff-edge retreat. We assume that the historic retreat rate may give an indication of the future retreat rate at a particular site. However, accurate and clear photographic and map documentation for measuring retreat is not always available or is of fairly short time intervals so changes may not be noticeable.

Lee and others (Appendix A, Reference 6) performed research studies of regional historic seacliff retreat and provide a maximum annual rate of seacliff recession of 0.7 inch. The research studies also estimated a maximum annual bluff-edge retreat rate

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of 0.22 to 0.33 feet per year. Over a 75-year period (assumed to be the economic lifetime of the new construction), this equates to a conservative estimate of bluff-edge retreat of a maximum of 16.5 to 24.8 feet. This maximum is based on research studies of regional historic bluff retreat that includes coastal bluffs with generally favorable conditions, as well as coastal bluffs that are affected by more adverse conditions (fracturing, sea caves, groundwater, human activities, etc.). The estimated values of maximum retreat are very conservative, and the actual rate of bluff retreat at the subject property is expected to be less considering the site conditions and historic bluff-edge retreat at the site.

Documentation of a two-day erosion period at and near the site in August 1983 (discussed above) is included in the U.S. Army Corps of Engineers' report, "Coastal Cliff Sediments, San Diego Region" (Appendix A, Reference 11). This report indicates that an episode of erosion occurred between August 7 and 9, 1983, "as a result of waves from a southern hemisphere storm." In the 'cove' area, approximately 100 feet south of the site, the report states that "the cliff retreated 28 to 35 feet" and "this was more erosion at the site than had occurred during the previous 40-year period." The photos show that at the subject property, a "bench" developed on the resistant Santiago Formation as a result of landward retreat (approximately 15 to 20 feet) of the toe of the terrace deposits at the site. However, even though the toe of the terrace deposits retreated landward, the upper portion of the bluff and bluff edge did not appear to have retreated landward at the site.

Our historic photograph review (Appendix A) indicates that the coastal bluff at the subject property is generally similar in configuration in the 1929, 1953, and 1964 through early 1983 photos. The location of the onsite bluff edge is generally similar on the vertical aerial photographs (1929 through 1989) listed in Appendix A and also on the 1975 and 1991 orthophoto topographic maps. However, in early August 1983, the base of the onsite terrace deposits eroded inland up to approximately 15 to 20 feet and resulted in the onsite "bench" on the resistant Santiago Formation sandstone.

If we assume that there were no definitive points of reference (such as the existing home to the north) to accurately measure prior to the 1964 aerial photographs (although the bluff appears to be similar in configuration), the resolution and scale of the photographs are such that if bluff-edge retreat were in excess of approximately 5 to 6 feet, it would likely be apparent. No apparent retreat of the bluff edge was seen on the photos, however, we could assume, as a "worst-case" scenario, that the bluff edge had retreated 6 feet over the 25-year time interval viewed (from 1964 to 1989), and a historic maximum bluff-edge retreat rate of approximately 0.24 feet per year may then be estimated from the photographs. Over a 75-year period, this equates to a "worst-case" or maximum of bluff-edge retreat of 18 feet.

From our site visits and photograph review, we observed, for the recent past, no indications that the current rate of retreat for the bluff edge is greater than this estimated "worst-case" historic rate of 0.24 feet per year and it apparently is significantly less. However, it is very difficult to predict the future and the magnitude of bluff-edge retreat that may occur in one year, during one storm event or over the 75-year assumed economic lifetime of the new construction. The rate of coastal bluff retreat over a particular interval of time (day, year, decade, etc.) may vary from very little to several tenths of a foot. However, severe erosion is generally episodic in nature and is dependent on the intensity of storms and combined high tides (or man's detrimental actions). It is probable that several feet of coastal bluff-edge retreat could occur at one time or over a short period of time. However, it is also likely that there will be periods in the future when erosion along the coast and bluff edge is rather insignificant and undetectable. Erosion is a naturally-occurring process that is affected by human actions. With time, the bluff edge will retreat landward.

If we assume a future retreat rate based on the "worst-case" site-specific historic aerial photograph evaluation, the maximum amount the bluff edge may retreat landward over the next 75 years would be approximately 18 feet. If a future retreat rate based on the regional study of bluff-top retreat rates (Appendix A, Reference 6) is assumed, the maximum amount the bluff edge may retreat over the next 75 years would range from 16.5 to 24.8 feet. However, as stated above, this maximum retreat rate is based on a research study of regional historic bluff-edge retreat which also included coastal bluffs that are more adversely affected by conditions that contribute to higher rates of erosion (fracturing, sea caves, groundwater, human activities, etc.) than those known at the subject property. We may speculate that the actual maximum value of future bluff-edge retreat at the site over the next 75 years will range somewhere from 18 (the site-specific, historic "worst-case") to 24.8 feet (the regional study "worst-case") and that the actual bluff-edge retreat is likely to be less.

It is our opinion that the residence, proposed to be set back a minimum of 25 feet from the bluff edge, will not be endangered by coastal bluff retreat over the next 75 years. However, improvements, such as patios, fences, etc., that are within this setback zone may become endangered (undermined) by bluff-edge retreat during the next 75 years and may need to be removed from the site.

- Sea Cave Influence - Sea cave formation and subsequent collapse are localized factors in the bluff retreat process. Indications of sea cave development in the seacliff were observed at the subject property during our site visits. Photos 1 and 2 are views of the subject coastal seacliff/bluff and Photo 5 is a closer view of a sea cave on site. Figure 1 shows the property, nearby vicinity and locations of the onsite and nearby known area of sea cave development. Our approximate measurements made on February 12, 1997, indicate that the onsite sea cave extends a maximum of approximately 5 feet obliquely into the

seacliff with a trend of roughly N75W. The sea cave is about 1 foot in height. The nearest offsite sea cave to the site is located approximately 3 feet southerly of the southern property boundary (see Photo 2). This sea cave extends approximately 2 feet into the seacliff with a maximum height of 1.5 feet. Fractures and bedding planes apparently control the development of these two caves. Due to their relatively small sizes, it is our opinion that if either (or both) of these sea cave areas enlarged and failed within the next 75 years, their collapse would not impact the proposed residence (which will be set back a minimum of 25 feet from the bluff edge).

- Seismic Shaking Influence - In general, the role seismic shaking plays in bluff retreat is dependent on bluff conditions at the moment of shaking. Failure of bedrock projections on seacliffs, failure of marginally stable blocks of bedrock, collapse of sea caves, and failures of undercut bluffs may be common occurrences along the San Diego coastline during strong ground shaking. The Shoreline Erosion Assessment and Atlas of the San Diego Region (Appendix A, Reference 3) states that the "potential for cliff failure due to earthquake activity in San Diego is probably small, but since San Diego has not experienced strong motion in such a long time, the cliffs here may be more susceptible to sliding than those in areas with shorter quake-return intervals."

Slope Stability Calculations

A computer-generated slope stability analysis was performed on the coastal bluff at the site. The slope stability was analyzed using 'Janbu's Simplified Method of Slices' with the PCSTABL 5M computer program. Groundwater was included in our slope stability analyses. The slope stability calculations are included in Appendix B. The soil strength parameters used in our analysis are presented below. These values are based on laboratory test results, back-calculation, our past experience in this area, and our professional judgement.

Soil Type	Unit Weight	Friction Angle	Cohesion
Terrace Deposits	120.0 pcf	40 degrees	500 psf
Santiago Fm Sandstone	120.0 pcf	35 degrees	200 psf

The results of the analyses (Appendix B) indicate that for the existing configuration, the calculated factor of safety against deep-seated failure is in excess of 1.5 (the generally accepted standard for the geotechnical industry). In addition, there are no indications of large ancient landslides in the site vicinity.

CONCLUSIONS AND RECOMMENDATIONS

Based on our geotechnical evaluation of the coastal bluff at the site, it is our opinion that the proposed residence (and the loading from this relatively light bluff-top construction) will not adversely impact the existing coastal bluff. In addition, if the residence is set back a minimum of 25 feet from the bluff edge as planned, it is our opinion that the proposed construction should not be affected by the maximum estimated coastal bluff retreat during its economic lifetime (assumed to be 75 years).

Slope Stability and Erosion

Our geotechnical evaluation of the present overall static stability on the subject property indicates that the bluff is grossly stable. In its present state, the slope is moderately to highly erodible and has a low to moderate potential for future surficial instability. We provide the following recommendations to help reduce erosion of the bluff and to reduce potential for future instability of the bluff face.

- Irrigation of the landscape areas on the property should be limited to the minimum amount required to establish vegetation and maintain plant vigor. The upper portion of the subject coastal bluff and the bluff edge are currently moderately to well vegetated with iceplant. At this time, it is our opinion that modifications to the vegetation in these areas should not be considered. However, if landscape planting and/or plant removal on the westerly bluff-top area is performed, it should be done without significantly disturbing the bluff-top soils. The surficial stability of those portions of the bluff that are not well vegetated may be increased by planting in accordance with the recommendations of a professional landscape company experienced with coastal bluffs. Terracing or excavation of the bluff-face soils should be avoided.

- Drainage at the site should be directed such that surface waters discharge into non-erosive drainage provisions. Runoff at the site should not be directed over the bluff edge. Eave gutters should be considered for the residence and should be properly maintained. Pedestrian and animal traffic (and burrowing, etc.) on the bluff face and bluff edge should be curtailed.

Bluff-Top Setback

As previously stated, we speculate that the maximum value of future bluff-edge retreat at the site over the next 75 years may range somewhere from 18 to 24.8 feet, and the actual value of bluff-edge retreat is likely to be less. The proposed residence will be set back a minimum of 25 feet from the bluff edge. It is our opinion that the

proposed setback will safeguard the proposed residence from bluff retreat during the economic lifetime of the residence.

Seismic Considerations

The principal considerations for most structures in southern California are surface rupturing of fault traces and damage caused by ground shaking or seismically-induced ground settlement or liquefaction. The possibility of damage due to ground rupture is considered minimal since no active faults are known to cross the site. It is our opinion that the potential for liquefaction or seismically-induced ground settlement at the site due to an earthquake is very low because of the dense nature of the underlying terrace deposits and Santiago Formation sandstone.

The seismic hazard most likely to impact the site is ground shaking resulting from an earthquake on one of the major active regional faults. The nearest known active fault is the Rose Canyon fault located offshore approximately 4 miles west of the site. It is estimated that a maximum earthquake on this portion of the Rose Canyon fault (magnitude 6.5) could produce moderate to severe ground shaking at the site.

As previously indicated, in general, the role seismic shaking plays in bluff retreat is dependent on bluff conditions at the moment of shaking. The seacliff at the subject property is near-vertical and locally portions are slightly overhanging. It is possible that some of the bedrock projections may fail as the result of severe ground shaking at the site. Oversteepened portions of the terrace deposits may undergo shallow failure and some ravelling of the poorly indurated bluff-face terrace deposits may also occur during ground shaking, especially on the unvegetated portions of the bluff face. However, it is our opinion that the potential for deep-seated or severe, catastrophic failure of the bluff due to expected seismic ground shaking is low at the site.

Other Considerations

The recommendations provided in this report are based on our understanding that a single-family residence (with its relatively light loading) is planned at the site and will be set back a minimum of 25 feet from the bluff edge. The site conditions and bluff edge indicated on Figures 2 and 3 have been compiled from approximate measurements made during our site visits. They should not be relied on for site development. If needed, we suggest that a licensed land surveyor be retained to prepare a site topographic plan that accurately delineates the property boundaries and bluff edge. In addition, a comprehensive site drainage study may be conducted to develop a site-specific drainage plan for the proposed development. Please note that the recommendations contained herein may be revised based on modified and/or additional information regarding the structure and improvements planned at the site.

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A qualified consultant should be retained to review site conditions and assess potential site impacts following significant erosion events in the future or if major changes in the bluff configuration are noticed.

Limitations and Uniformity of Conditions

This geotechnical evaluation report addresses the coastal bluff conditions at the property and is based on our understanding that the proposed development consists of design and construction of a new single-family residence set back a minimum of 25 feet from the bluff edge. We understand that soils engineering services for design and construction of this proposed residential development are being performed by others. In addition, we have not performed an evaluation of the presence of hazardous materials/contamination at the site.

This report is based on our document/photograph review, surficial observations of the geologic conditions exposed at the site and vicinity. This report assumes that the geologic/soils conditions do not deviate appreciably from those observed during our site visits. The recommendations of this report pertain only to the coastal bluff property evaluated.

The findings of this report are valid as of this date. Changes in conditions of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, from legislation or the broadening of knowledge in the fields of geotechnical engineering or geology. Hence, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two years without a review by us.

If there are questions regarding the information contained herein, we should be contacted. We will not be responsible for the interpretation by others of the information herein. Our services consist of professional consultation and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed by us.

If you have any questions regarding our report, please call. We appreciate this opportunity to be of service.

Sincerely,

SOUTHLAND GEOTECHNICAL CONSULTANTS

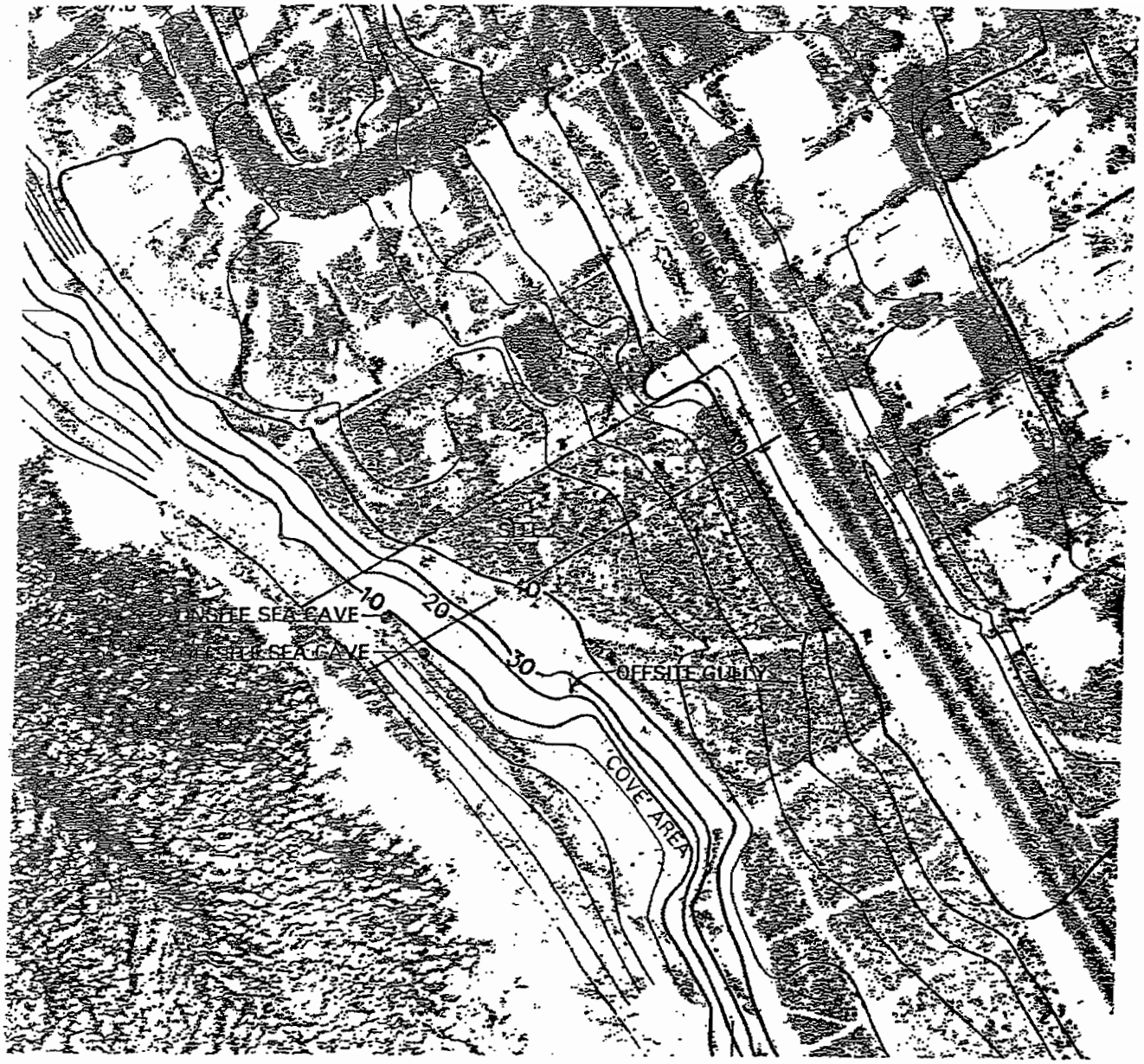
Susan E. Tanges

Susan E. Tanges, CEG 1386
Managing Principal/Engineering Geologist

Steven Norris, RCE 47672
Project Engineer

Attachments:
- Figure 1 - Site Location Map
- Figure 2 - Site Plan
- Figure 3 - Coastal Bluff Profile
- Photographs 1 through 6
- Appendix A - References
- Appendix B - Slope Stability Calculations

Distribution: (3) Addressee



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SITE LOCATION MAP

Project No. 126G21
Vacant Parcel South of 5305 Carlsbad Boulevard, Carlsbad
Scale (approximate): 1 inch = 100 feet

Base Map:
City of Carlsbad
Orthophoto Topographic Map 93
dated 1991

FIGURE 1

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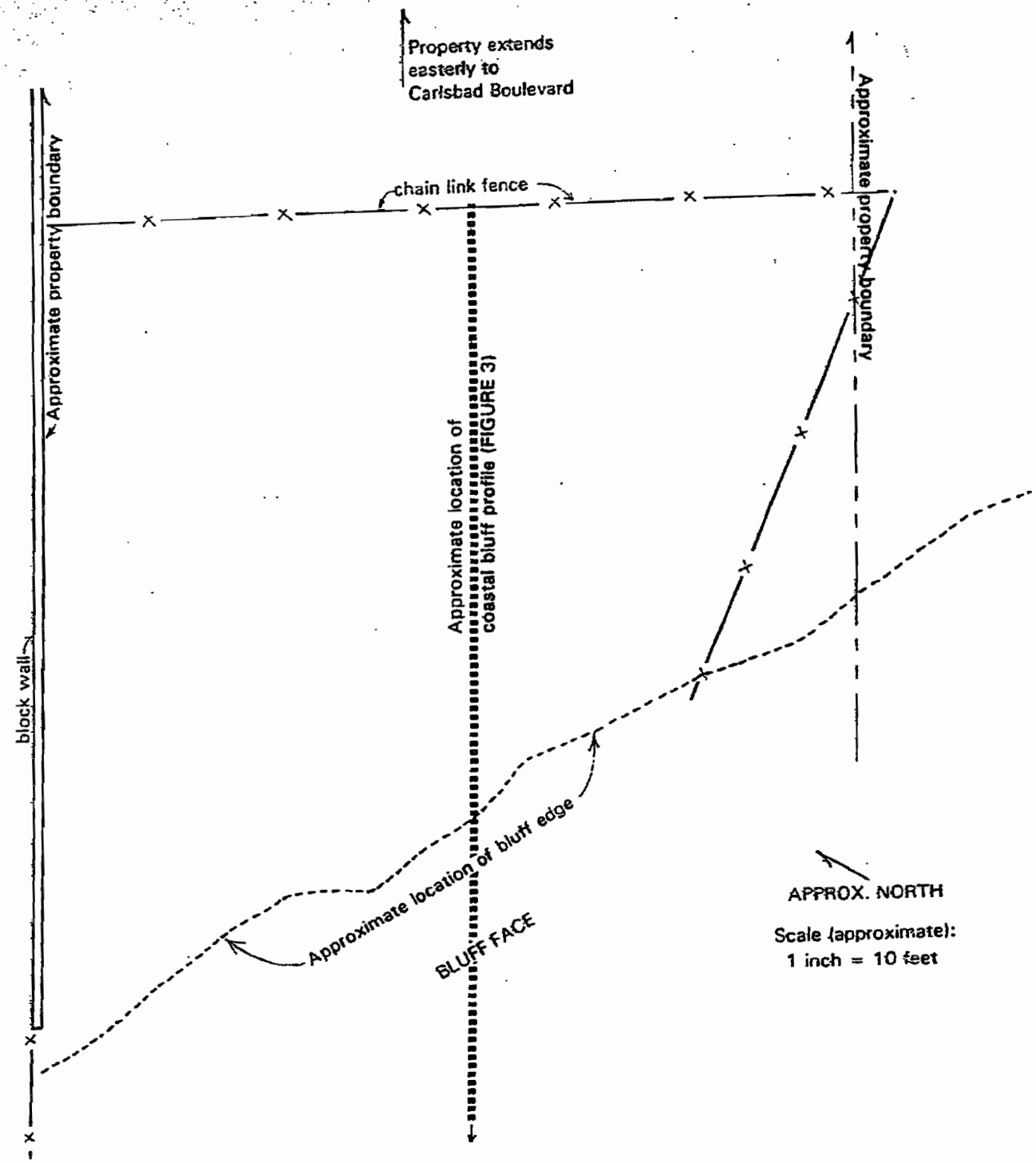


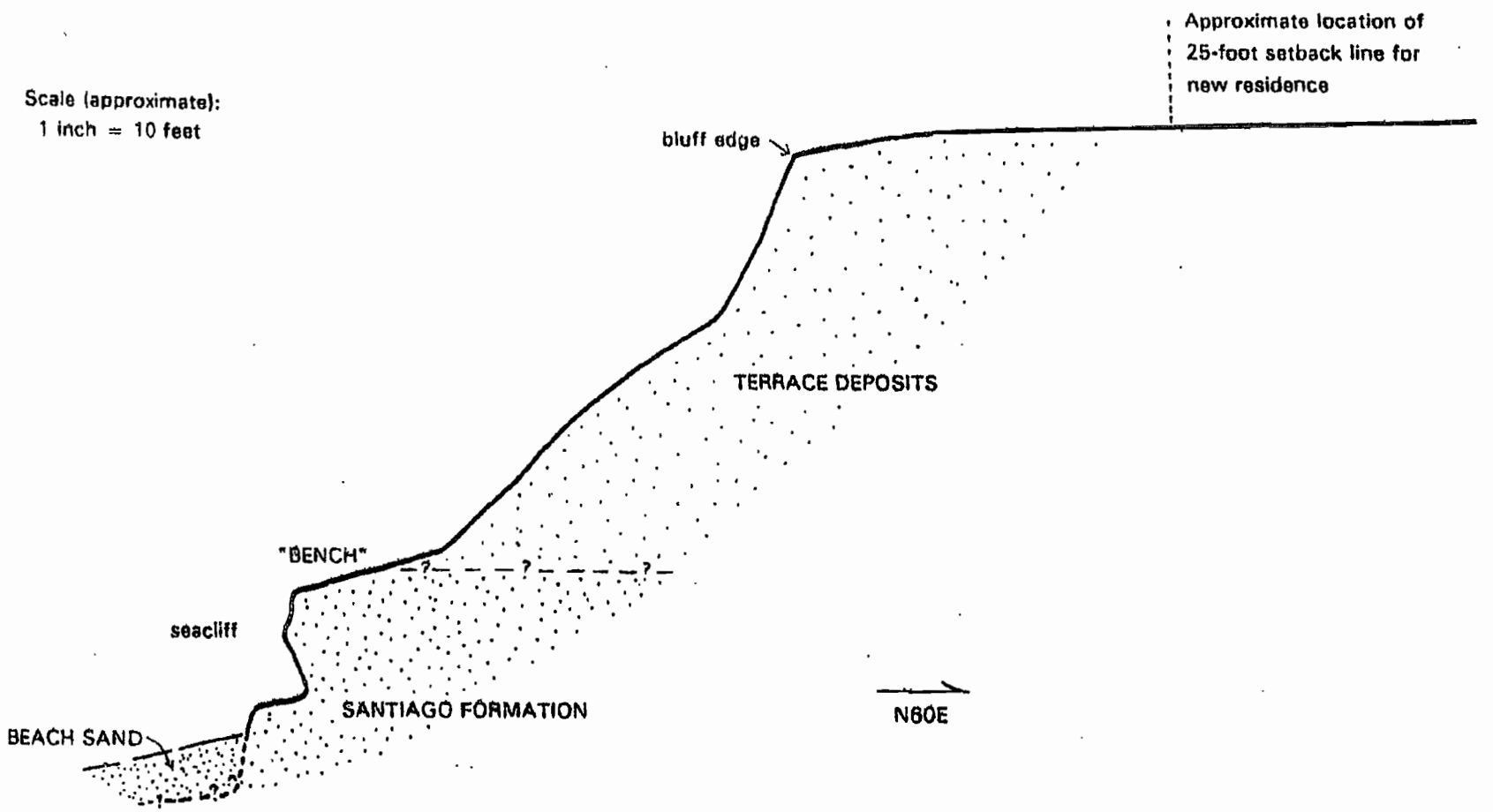
Site plan compiled from
approximate measurements
made by SGC representatives
on 02-12-97

THIS IS NOT A SURVEYED MAP

SITE PLAN
Vacant Parcel South of
5305 Carlsbad Blvd, Carlsbad

Project No. 126G21 FIGURE 2

SGC



Coastal bluff profile compiled
from approximate measurements
made by SGC representatives on 02-12-97
(see FIGURE 2 for profile location)

COASTAL BLUFF PROFILE

Vacant Parcel South of
5305 Carlsbad Blvd, Carlsbad

Project No. 126G21 FIGURE 3

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Project No. 126G21

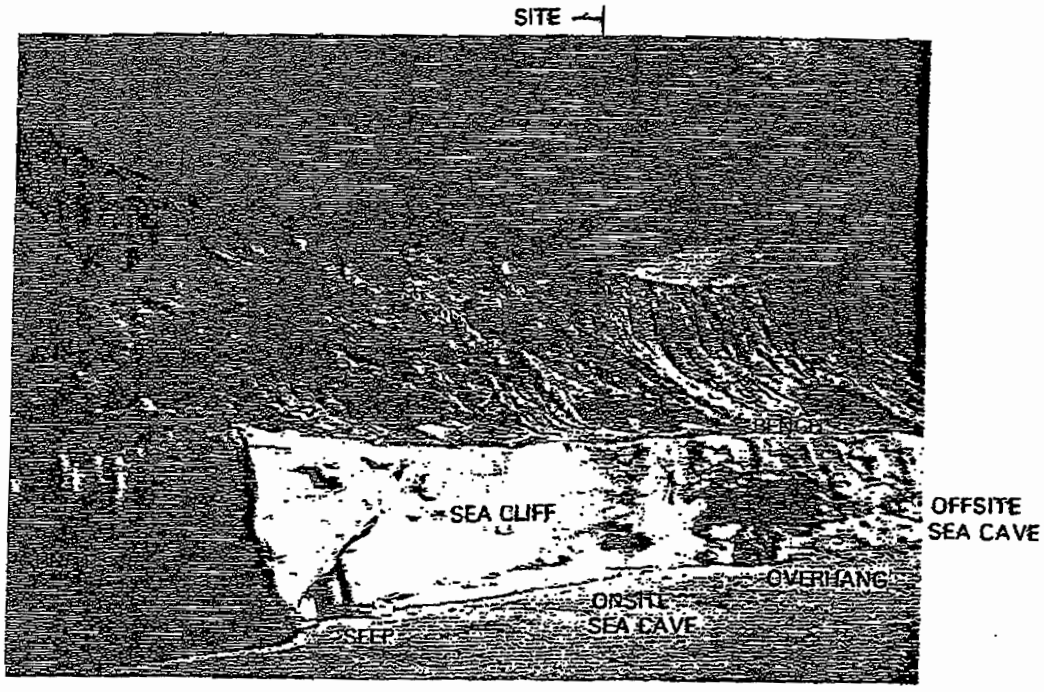

PHOTO 1 Coastal bluff, view southerly from near northern property boundary 1-31-97

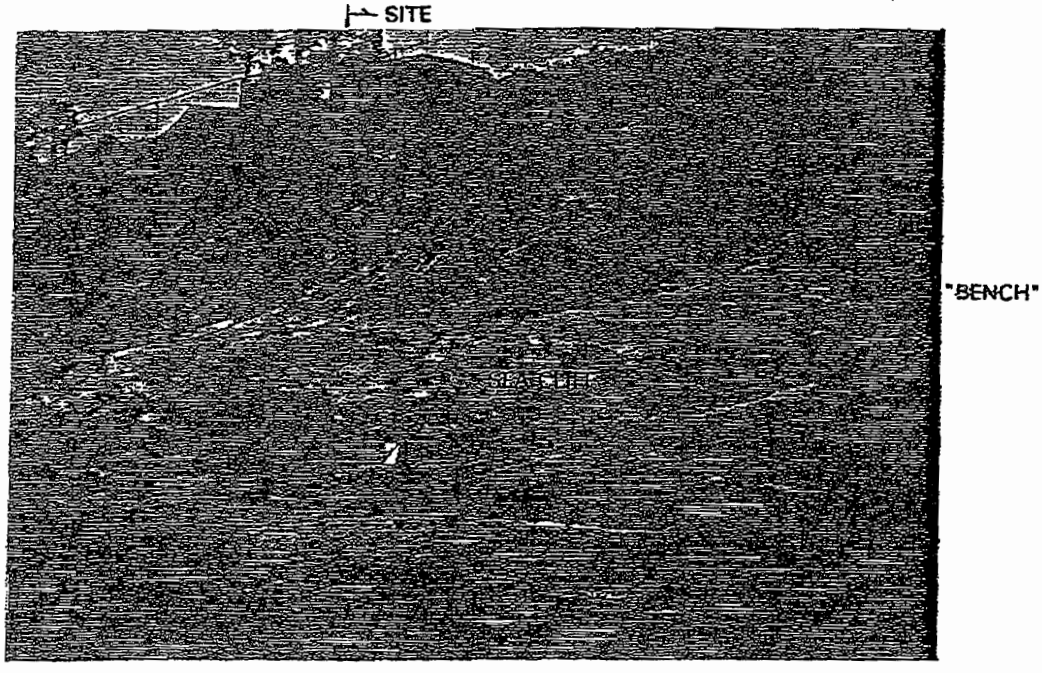

PHOTO 2 Coastal bluff, view northerly from near southern property boundary 2-12-97

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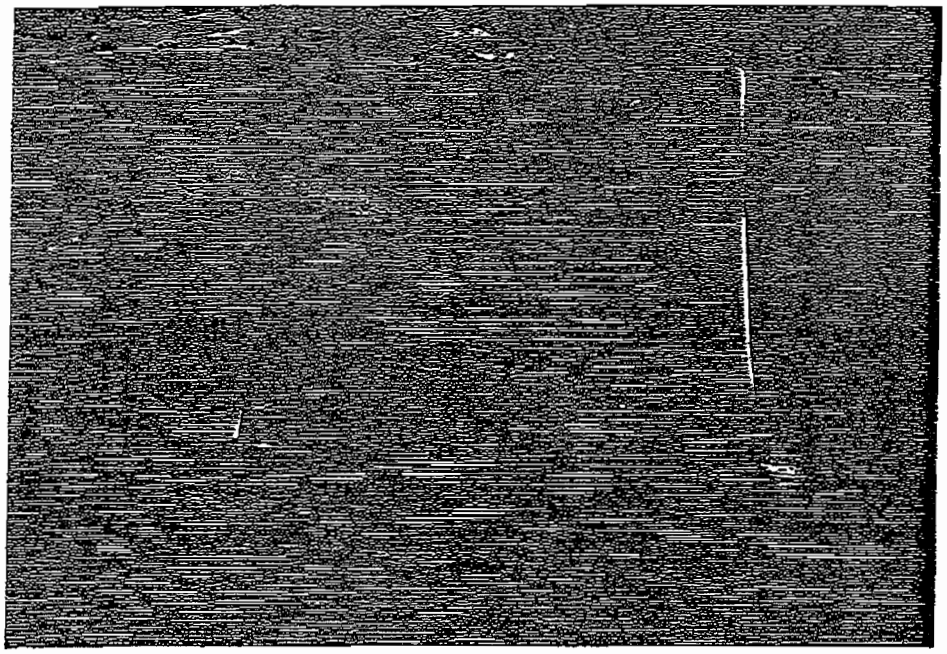

PHOTO 5 Onsite sea cave in sea cliff (see Figure 1 for location) 2-12-97

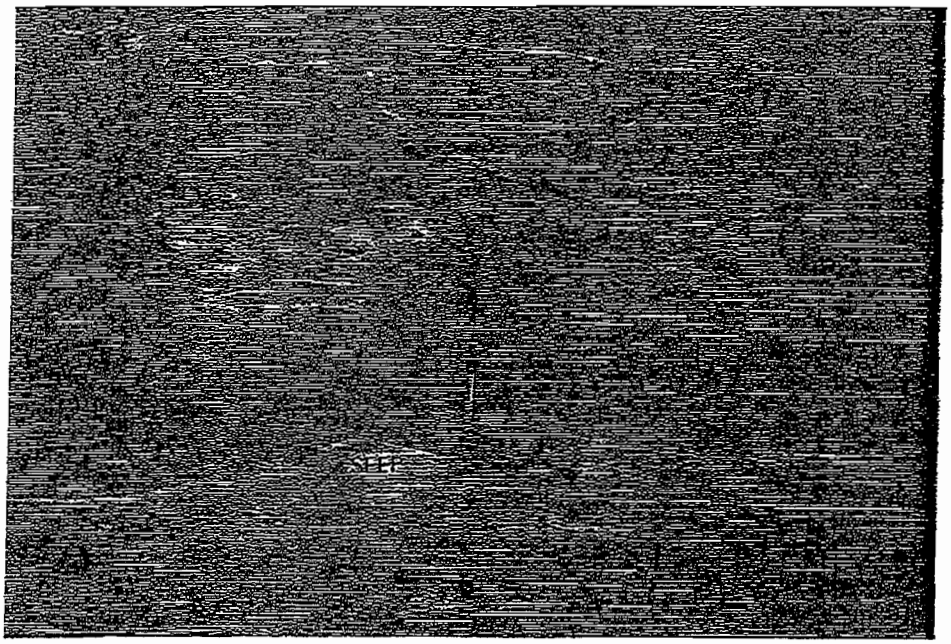

PHOTO 6 Onsite gunite and groundwater seep near northerly property boundary 2-12-97

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APPENDIX A

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APPENDIX A

REFERENCES
(continued)

12. Van Dorn, W.G., 1979, Theoretical aspects of tsunamis along the San Diego coastline, in Abbott, P.L., and Elliott, W.J., eds., Earthquakes and Other Perils: Geological Society of America field trip guidebook.

13. Weber, F.H., Jr., 1982, Recent slope failures, ancient landslides and related geology of the north-central coastal area, San Diego County, California: California Division of Mines and Geology, Open-File Report 82-12.

AERIAL PHOTOGRAPHS

Aerial Fotobank, Book 8 (vertical, not stereoscopic):
Frame 1528, dated March 29, 1965.
Frame 2252, dated August 12, 1965.
Frame 8-4531, dated November 21, 1966.

Aerial Fotobank, Book 8A (vertical, not stereoscopic):
Frame 10,602, dated November 8, 1968.
Frame 12,972, dated July 8, 1969.
Frame 15,073, dated November 2, 1969.

Aerial Fotobank, Book 8C (vertical, not stereoscopic):
Frame 51-356, dated September 27, 1971.

Aerial Fotobank, Book 8D (vertical, not stereoscopic):
Frame 25,039, dated December 16, 1980.
Frame 25,818, dated February 11, 1983.

Aerial Fotobank, Book 8E (vertical, not stereoscopic):
Frame 26,376, dated October 13, 1984.

Aerial Fotobank, 1990's and Carlsbad books:
Frame 0082 (3-7D), dated January 1, 1983.
Frame 1663 (4-8D), dated January 28, 1984.
Frame SD87-8D, dated January 21, 1987.
Frame CV90-5, dated January 18, 1990.

County of San Diego, 1929, Photo 30D1 (vertical, not stereoscopic).

APPENDIX A

AERIAL PHOTOGRAPHS
(continued)

County of San Diego, 1967, Series GS-VBTA, Flight Line 1, Photos 1-142 and 1-143, dated May 8 (vertical, stereoscopic).

County of San Diego, 1970, Series SDCO, Flight Line 2, Photos 2-1 (015) and 2-2 (014), dated October 9 (vertical, stereoscopic), scale 1:24,000.

County of San Diego, 1975, Flight SDPD, Flight Line 36, Photos 36-2 (129) and 36-3 (128), dated January 20 (vertical, stereoscopic), scale 1 inch = 1,000 feet.

County of San Diego, 1983, Flight C11109 83059, Photos 256 (023) and 257 (024), dated November 22 (vertical, stereoscopic), scale 1 inch = 2,000 feet.

County of San Diego, 1989, Series WAC-89A, Photo 3-7, dated April 7 (vertical, not stereoscopic).

Geo-Tech Imagery Intl., 1992, Carlsbad Frames 6 and 7, dated November 1 (infrared, oblique, stereoscopic).

U.S. Department of Agriculture, 1953, Series AXN, Flight Line 14M, Photos 17 and 18, dated May 2 (vertical, stereoscopic), scale 1:20,000.

U.S. Department of Agriculture, 1964, Series AXN, Flight Line 4DD, Photo 97, dated April 9 (vertical, not stereoscopic).

MAPS

City of Carlsbad, 1992, Geotechnical Hazards Analysis and Mapping Study, dated November.

City of Carlsbad, 1991, Orthophoto Mapping, Sheet 93, scale 1" = 100' (aerial photography dated September-October 1988).

County of San Diego, 1975, Orthophoto Topographic Map 350-1665, dated September 17, scale 1" = 200'.

County of San Diego, Assessor's Map Book, page 210-12.

County of San Diego, 1898, Survey Number 148, Fifth Road District, Oceanside to Encinitas, two parts, dated October.

APPENDIX A

MAPS
(continued)

County of San Diego, 1915, Map of Partition of a Portion of the Land Owned by the Undersigned in the Rancho Agua Hedionda, Map No. 823, dated May 1.

County of San Diego, 1958, Poinsettia Beach Unit No.1, Map No. 3897, dated May 21.

County of San Diego, 1984, Record of Survey, Map No. 9595, dated February 29.

County of San Diego, 1987, Record of Survey, Map No. 11280, dated October 22.

APPENDIX B

SGC

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	March 4, 1996
Time of Run:	5:00 pm
Input Data Filename:	Carlsbad.in
Output Filename:	Carlsbad.out
Plotted Output Filename:	Carlsbad.plt

PROBLEM DESCRIPTION STABILITY ANALYSYS - Carlsbad bluff

BOUNDARY COORDINATES

10 Top Boundaries
11 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	20.00	20.00	33.00	25.00	1
2	33.00	25.00	34.00	30.50	1
3	34.00	30.50	37.70	31.00	1
4	37.70	31.00	39.00	39.00	1
5	39.00	39.00	46.70	41.00	2
6	46.70	41.00	65.00	56.80	2
7	65.00	56.80	70.00	67.30	2
8	70.00	67.30	79.50	69.00	2
9	79.50	69.00	114.80	70.00	2
10	114.80	70.00	220.00	70.00	2
11	39.00	39.00	220.00	39.00	1

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	135.0	500.0	40.0	.00	.0	0
2	120.0	135.0	200.0	35.0	.00	.0	0

1 PIEZOMETRIC SURFACE HAS BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	39.00	39.00
2	220.00	40.00

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (lb/sqft)	Deflection (deg)
1	95.00	96.00	1000.0	.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Irregular Surfaces, Has Been Specified.

Janbus Empirical Coef. is being used for the case of c & phi both > 0

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced Along The Ground Surface Between X = 28.00 ft. and X = 60.00 ft.

Each Surface Terminates Between X = 70.00 ft. and X = 170.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

10.00 ft. Line Segments Define Each Trial Failure Surface.

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	28.00	23.08
2	35.39	16.34
3	43.45	10.42
4	52.02	5.27
5	61.99	6.12
6	71.97	6.62
7	79.76	12.90
8	81.00	22.82
9	81.08	32.82
10	81.51	42.81
11	82.27	52.78
12	82.54	62.78
13	82.69	69.09

Factor Of Safety For The Preceding Specified Surface = 5.619

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	28.00	23.08
2	36.39	17.64
3	45.56	13.65
4	55.38	15.54
5	62.54	22.52
6	69.71	29.50
7	70.62	39.46
8	70.98	49.45
9	71.25	59.45
10	71.31	67.53

Factor Of Safety For The Preceding Specified Surface = 3.284

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	35.11	30.65
2	42.48	23.89
3	52.42	22.79
4	62.00	25.66
5	71.41	29.06
6	79.02	35.55
7	79.87	45.51
8	79.96	55.51
9	79.97	65.51
10	79.99	69.01

Factor Of Safety For The Preceding Specified Surface = 3.715

Factor Of Safety Calculation Has Gone Through Ten Iterations

The Trial Failure Surface In Question Is Defined
By The Following 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	52.89	46.34
2	60.06	39.37
3	67.18	32.35
4	75.06	26.19
5	83.94	21.59
6	92.11	15.83
7	100.39	10.22
8	108.62	4.54
9	118.12	1.42
10	128.07	2.41
11	135.69	8.88
12	136.54	18.85
13	136.97	28.84
14	137.00	38.84
15	137.82	48.81
16	139.57	58.65
17	142.13	68.32
18	142.14	70.00

Factor Of Safety For The Preceding Specified Surface = 83.303

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	52.89	46.34
2	62.50	49.12
3	71.06	54.28
4	79.20	60.09
5	84.98	68.26
6	85.34	69.17

*** 1.648 ***

Individual data on the 8 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	9.6	3178.3	.0	.0	.0	.0	.0	.0	.0
2	2.5	1755.6	.0	.0	.0	.0	.0	.0	.0
3	5.0	5947.5	.0	.0	.0	.0	.0	.0	.0
4	1.1	1711.0	.0	.0	.0	.0	.0	.0	.0
5	8.1	10770.4	.0	.0	.0	.0	.0	.0	.0
6	.3	312.8	.0	.0	.0	.0	.0	.0	.0
7	5.5	3082.1	.0	.0	.0	.0	.0	.0	.0
8	.4	19.4	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	45.78	40.76
2	55.78	40.75
3	65.14	44.27
4	71.69	51.82
5	73.64	61.63
6	74.11	68.04

*** 1.735 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	45.78	40.76
2	55.38	37.96
3	64.82	41.24
4	73.73	45.79
5	79.59	53.90
6	87.03	60.57
7	94.65	67.05
8	97.67	69.51

*** 1.766 ***

Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.67	36.95
2	48.58	38.29
3	56.43	44.49
4	65.18	49.33
5	71.14	57.36
6	75.42	66.39
7	75.94	68.36

*** 1.807 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	28.00	23.08
2	35.31	16.25
3	45.31	16.17
4	53.93	21.23
5	62.10	27.00
6	68.33	34.83
7	73.13	43.60
8	78.92	51.75
9	83.46	60.66
10	88.95	69.02
11	88.98	69.27

*** 1.821 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	31.56	24.44
2	40.27	19.53
3	49.97	21.96
4	56.30	29.70
5	62.86	37.24
6	67.84	45.91
7	71.81	55.09
8	75.64	64.33
9	76.18	68.41

*** 1.875 ***

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	56.44	49.41
2	66.39	48.41
3	75.08	53.36
4	78.63	62.71
5	80.92	69.04

*** 1.895 ***

Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.67	36.95
2	48.66	36.55
3	58.63	37.26
4	67.09	42.60
5	72.78	50.82
6	75.99	60.29
7	77.39	68.62

*** 1.897 ***

Failure Surface Specified By 6 Coordinate Points

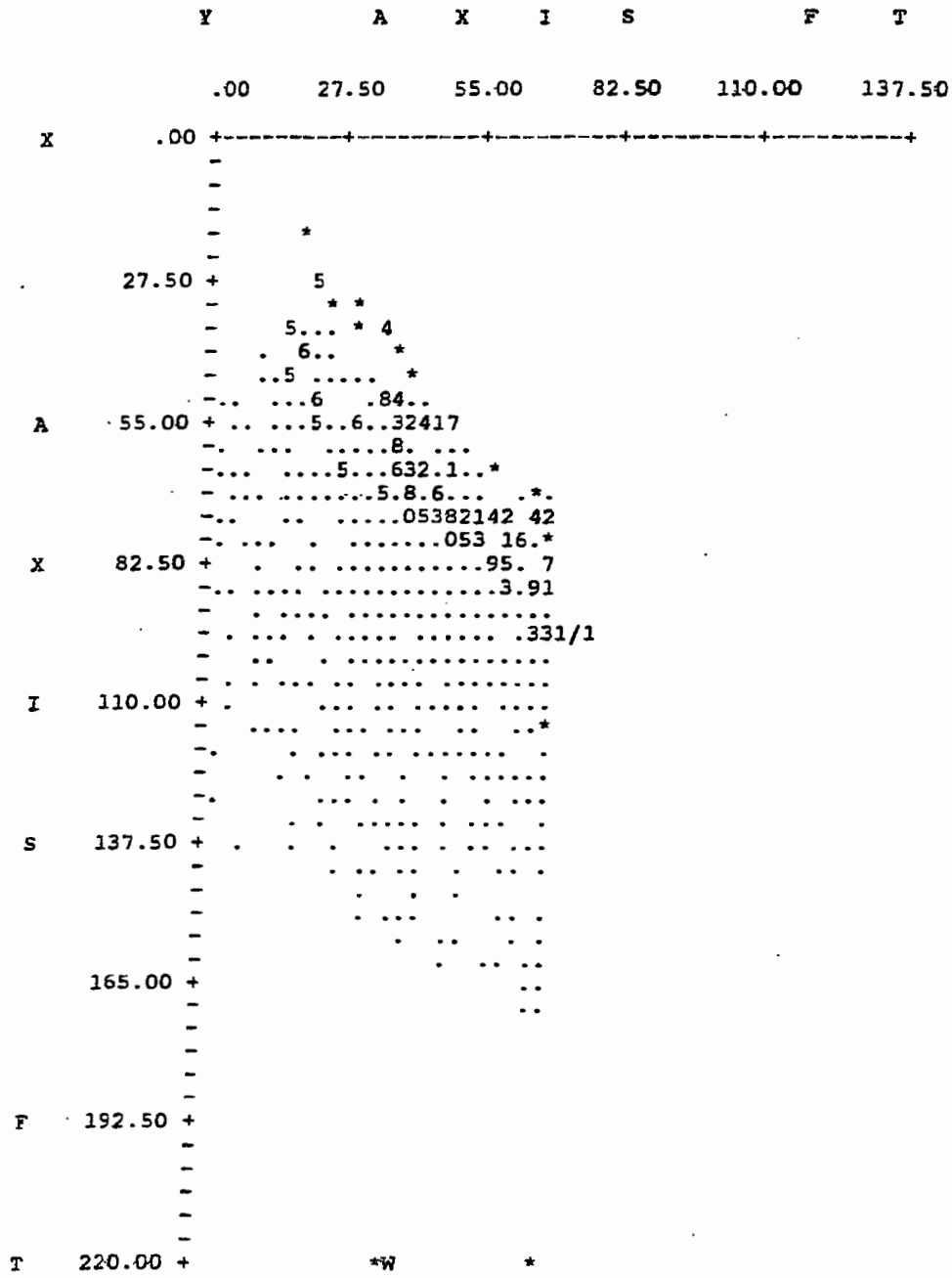
Point No.	X-Surf (ft)	Y-Surf (ft)
1	52.89	46.34
2	62.16	50.08
3	71.29	54.18
4	80.24	58.63
5	86.78	66.19
6	87.67	69.23

*** 1.907 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	42.22	39.84
2	52.15	38.60
3	62.13	39.08
4	71.65	42.16
5	77.50	50.27
6	81.46	59.45
7	84.37	69.02
8	84.40	69.14

*** 1.916 ***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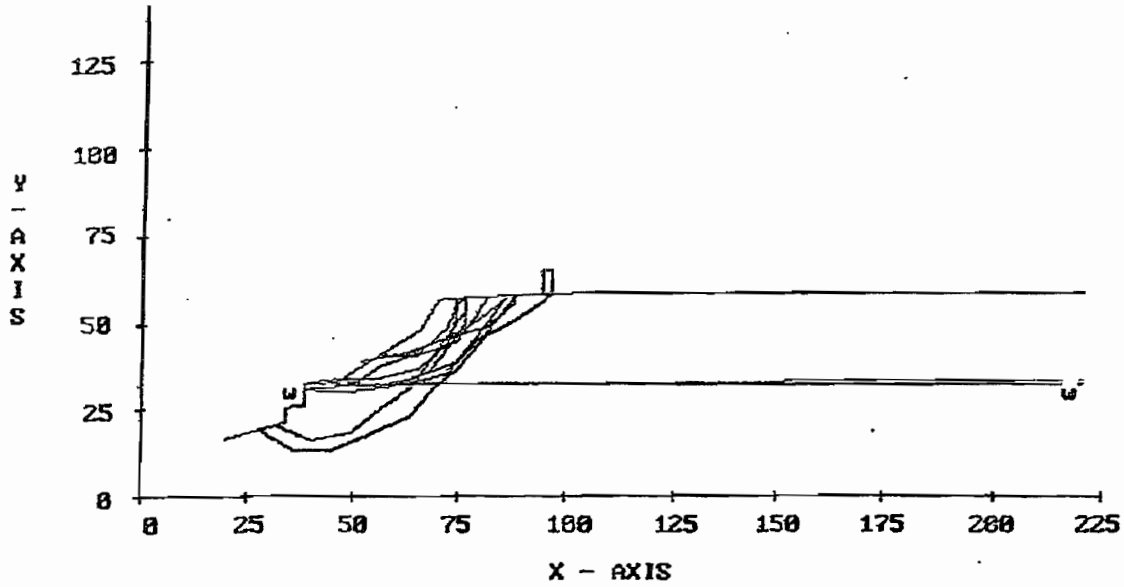
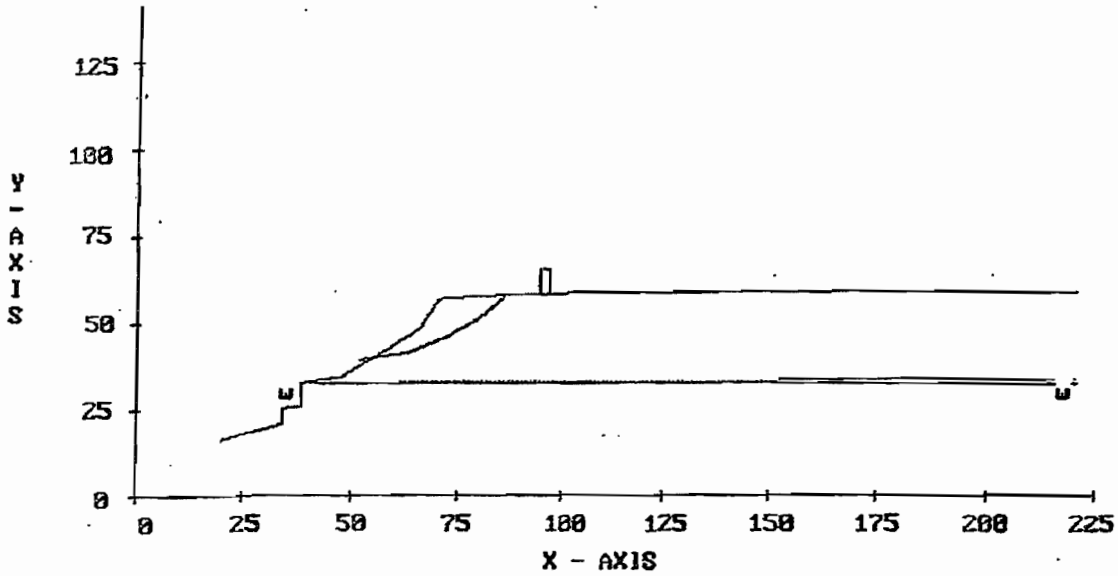
T 220.00



Project No. 126521



Project No. 126G21

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced Along The Ground Surface Between X = 20.00 ft.
and X = 60.00 ft.

Each Surface Terminates Between X = 70.00 ft.
and X = 170.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

10.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	51.11	44.81
2	60.52	48.20
3	68.57	54.13
4	74.59	62.11
5	76.99	68.55

*** 1.518 ***

Individual data on the 6 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	9.4	2674.6	.0	.0	.0	.0	.0	.0	.0
2	4.5	2698.8	.0	.0	.0	.0	.0	.0	.0
3	3.6	3316.5	.0	.0	.0	.0	.0	.0	.0
4	1.4	1838.8	.0	.0	.0	.0	.0	.0	.0
5	4.6	4764.5	.0	.0	.0	.0	.0	.0	.0
6	2.4	865.3	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	37.78	31.48
2	47.44	34.05
3	56.66	37.92
4	65.27	43.01
5	73.11	49.23
6	80.02	56.45
7	85.88	64.56
8	88.38	69.25

*** 1.584 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.00	20.00
2	29.92	21.27
3	39.57	23.89
4	48.77	27.81
5	57.34	32.97
6	65.11	39.26
7	71.95	46.55
8	77.72	54.72
9	82.30	63.61
10	84.24	69.13

*** 1.598 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	42.22	39.84
2	52.22	39.80
3	61.49	43.55
4	68.66	50.52
5	72.65	59.69
6	72.84	67.81

*** 1.646 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.00	20.00
2	30.00	20.01
3	39.82	21.90
4	49.11	25.61
5	57.54	30.99
6	64.80	37.86
7	70.65	45.98
8	74.86	55.05
9	77.30	64.74
10	77.52	68.65

*** 1.650 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	51.11	44.81
2	61.04	46.02
3	70.35	49.66
4	78.48	55.49
5	84.90	63.16
6	87.81	69.24

*** 1.651 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	28.89	23.42
2	38.88	23.00
3	48.77	24.47
4	58.21	27.77
5	66.86	32.79
6	74.42	39.34
7	80.61	47.20
8	85.21	56.07
9	88.06	65.66
10	88.43	69.25

*** 1.677 ***

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	60.00	52.48
2	69.29	56.18
3	76.70	62.89
4	79.87	69.01

*** 1.707 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	28.89	23.42
2	38.88	23.87
3	48.73	25.60
4	58.27	28.60
5	67.34	32.80
6	75.80	38.14
7	83.49	44.53
8	90.29	51.87
9	96.08	60.02
10	100.77	68.85
11	101.06	69.61

*** 1.760 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	51.11	44.81
2	60.35	48.64
3	69.22	53.25
4	77.67	58.60
5	85.63	64.66
6	90.78	69.32

*** 1.779 ***

EXHIBIT E

C. J. Randle, P.E., Civil Engineer
5858 Mt. Alifan Drive, Suite 235
San Diego, CA 92111
Telephone (619) 571-6271
Fax (619) 571-3943

March 16, 1998

Mr. Jon Jensen
Jon A. Jensen and Associates
451 South Escondido Blvd.
Escondido, CA 92025

Subject: Assumption of Geotechnical Responsibility
Updated Geotechnical Report
Assessor Parcel Number 210-120-30
Carlsbad, California

References:

1. Geotechnical Update, Carlsbad Beach Lot, APN 210-120-30, Carlsbad Drive, Carlsbad, California; 1996, by Geotechnics, Inc., Project No. 0319-001-00, dated September 3, 1996.

2. Geotechnical Investigation and Bluff Retreat Study, Parcel No. 210-120-30, Carlsbad, California; 1991, by ICG, Inc., Job No. 04-8529-001-00-00, dated March 28, 1991.

3. Geotechnical Investigation and Bluff Retreat Study, Parcel No. 210-120-31, Carlsbad, California; 1989, by ICG, Inc., Job No. 05-8109-001-00-00, dated November 6, 1989.

4. California's Battered Coast; 1985, California Coastal Commission Meeting, Professional Papers and Publication, San Diego, California, September, 1985 - San Diego Association of Geologists.

5. Supplemental Geotechnical Investigation, Proposed Coastal Protection Measures for the 2+/- acre coastal site, Carlsbad, California; 1984, by Converse Consultants, Project No. 83-2299-02, dated September 20, 1984.

6. Geotechnical Investigation, Proposed Coastal Protection Measures for the Ecke Site, Carlsbad, California; 1984, by Converse Consultants, Project No. 83-02299-01, dated February 1, 1984.

INTRODUCTION

In accordance with your request, we have completed an updated geotechnical study of the subject site, APN 210-120-30, in the City of Carlsbad, California. The purpose of this study was to review geologic and soils engineering data as they relate to future site development, evaluate sea bluff rate of retreat, and establish building and construction set-backs relative to the top of the sea bluff.

SCOPE OF WORK

The scope of our work has included the following tasks:

Review available geological reports and data pertinent to the subject site. The list of referenced and/or review data is included on page 1 of this report.

Field review of the site and adjacent areas including an assessment of the nearby geological units and conditions, to include existing sea bluff conditions.

Preparation and processing of this report.

In addition, our study is supplemented by our engineering services, subsurface work and observation services during construction of a stairway access to the beach along the south property boundary of the subject site placed into the bedrock and constructed on caissons.

PROPOSED DEVELOPMENT

The actual plans for future development at the property are not yet complete, but include provisions that will preclude any excess drainage from within the site to be channeled over the sea bluff face. This is proposed to be accomplished through landscaping controls, graded earthen swales and other devices to collect and divert water away from the sea bluff. Additionally, the surface drainage of the property is enhanced by the surface drainage provisions, which have been developed in and around the concrete beach access stairway along the south boundary of the property.

SUMMARY OF REVIEW

Based on our study, references 1, 2, 5 and 6 adequately describe the property soil and geological conditions and the report conclusions and recommendations are still appropriate and applicable to development of the property except for those items described within the discussion section of this report.

DISCUSSION

Previous reports based sea cliff retreat and construction setbacks upon an "empirical" figure of 25 feet extrapolated over 62 years (ref. 2). However, the same report indicates "relatively little changes" to the bluff top. Reference 3 indicates "There has been little retreat of the bluff top in the vicinity. . . " of the site based on a study interval from 1929 through 1984.

The reports also indicate a concern relative to the cove area of the site. The cove area is reported to have undergone a short term interval of episodic and relative rapid erosion. Cove areas are not uncommon along the coastal area of San Diego County, and even if underlain by more erodible or "softer" material once the weaker or more erodible material forms the cove feature, that natural set back is sufficient to create a buffer from the brunt of direct wave attack. Thus, once formed, the top of bluff retreat rate for the cove area becomes similar to that of adjacent tops of bluff to both sides of the cove.

Reference 4 (a compilation of professional studies), the 1985 California Coastal Commission Meeting Publication at San Diego provides documented studies for sea bluff retreat and erosion of the San Diego County Coastal area. These documents include both short term and long term studies, as well as work comparison of similar rates for the base and tops of bluffs.

These studies confirmed an overall rate of retreat for both the upper and lower parts of the bluffs at 1 to 3 inches per year. The studies also confirmed that rates tended to decrease slightly in areas after episodic rapid erosion and/or retreat had occurred in any given area. These studies were based on the time interval of about 50 to 75 years.

For the property, we can assume for the 75 year design period that a rate of retreat of about 6 to 19 feet might occur. Because there was a period of relatively rapid episodic erosion for the site area in about 1977 through 1983, we can assume that the longer term overall rate will be less and more likely on the order of about 6 to 12 feet, or less. These rates will be significantly affected and reduced in the event that longer term sea cliff erosion control devices are constructed at the site.

0.3 → 0.16

CONCLUSIONS AND RECOMMENDATIONS

Based on our study, the previous soil and geologic findings for the site are still appropriate and applicable for the site, except for those items outlined in the Discussion section of this report and updated in the following paragraphs. We assume soil and geologic responsibility for the previous reports, findings, conclusions and recommendations of the previous reports.

Based on the site conditions and our study, the rate of retreat for the top of bluff at the project site is established to be on the order of 1 to 2 inches per year for the 75 year planned life of the proposed development of the property.

0.08 → 0.16

The bluff retreat may be addressed with the following dimensions and criteria:

1. A 25 foot set back from the face of the bluff with all foundations to be placed on deep caissons founded in firm dense to very dense, formational sandstone (bedrock). The bedrock appears to be at maximum depths of elevations of -4 to -6 feet (msl) at elevation +8 and increases easterly.

2. Bluff setbacks will be established at 30 feet from the face of the bluff for all foundation systems which conform to the current Uniform Building Code criteria. However, conventional foundation systems will require development of a deepened grade beam foundation system which will be a minimum of 36 inches below lowest adjacent grade.

3. Typical slab on grade construction with minimal footing depths may be constructed with a set back of 40 feet established from the face of the bluff.

Based on our elevation of the recommended setbacks (with relation to the proposed 3 lot Parcel Map) each lot will easily provide for the necessary building area, without encroaching into the proposed setback. Similarly, the remaining land area will easily accommodate this proposed development.

The option of set backs is prudent and will easily address coastal retreat, which essentially will be mitigated by the restrictions placed on the site drainage controls. The 30 and 40 foot options are for all intents and purposes a conservative response to the bluff top setbacks. When compared to the proposery 25 foot setback for the northerly and adjacent vacant parcel.

EXHIBIT F

SCANNED

OWEN ENGINEERING GROUP

Civil, Geotechnical and Structural Engineering

October 10, 2001

CDP01-13

RECEIVED
MAR 0 8 2002
ENGINEERING DEPARTMENT

OK
1st check
DAVID RICK
3/26/02

Mr. Dean Goetz
603 North Highway 101
Solana Beach, CA 92075

Subject: Geotechnical Review and Update of Applicable Geology and Geotechnical Reports for 5323 Carlsbad Blvd. Carlsbad, CA

Re: Assumption of Geotechnical Responsibility, Updated Geotechnical Report, Assessor Parcel Number 210-120-30, Carlsbad, CA, by C. J. Randle, P.E., Dated March 16, 1998.

Dear Mr. Goetz:

This geotechnical letter is being written to updates the referenced report and previously referenced reports, relative to the development of your site.

Prior to this update, the referenced report addressed bluff stability and recommended setbacks relative to the construction of the proposed dwellings at this site. In addition, said report is applicable to all three parcels which are part of Parcel Map 18236 previously identified as APN 210-120-30.

Applicable criteria for this development are summarized as followed:

Where the design parameters differ from the referenced reports, these values are updated herein and therefore apply to your proposed residence.

As noted in previous reports by this office, the recommended setback from the bluff is 40 feet from the top of the existing bluff.

Design criteria is as follows:

Item 1. The lateral pressure evaluation should be based on 32-degrees angle of internal friction and 100 PSF of cohesion.

Item 2. The unit soil bearing for continuous foundations located a minimum of 18 inches below lowest adjacent grade shall not exceed 2500 PSF. This is a conservative value based on the Terzaghi Unit Bearing Value method for

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this type soil. The unit soil bearing value may be increased 500 PSF for each additional 12 inches of embedment of the footing relative to the lowest adjacent grade to a maximum of 4000 PSF.

In the event of a basement-type structure, relate to the measurement to the inside or lowest adjacent grade. This is not a reference to the finished exterior pad grade.

Item 3. The soil, as tested, is non-expansive and no special provisions for slab-on-grade or similar considerations are required.

Item 4. The foundation system along the westerly perimeter of the dwelling may include end-bearing caissons placed on formation material, located approximately 5 to 8 feet below grade. Unit soil bearing on formation material shall not exceed 6000 psf.

Caisson distribution and spacing shall be a part of the structural evaluation and the dwelling and its foundation system. Note: this is only a recommendation and is not a requirement for the dwelling "bluff Setback" of 40 feet.

The caisson system is an option. The proposed perimeter grade beam/foundation will adequately support the proposed structure.

Item 5. All slabs on grade systems will require a sub-base material to consist of compact dense free draining soil and/or gravel. Sub-slab materials, within the living areas, will require the placement of at least 6-mil visqueen and 2 inches of sand as a leveling course. Concrete slab-on-grades are recommended to be a minimum of 4 inches of thickness and contain a minimum 6 gauge welded wire mat or as directed by the structural engineer.

The above items are specifically directed to the criteria necessary to the design of the building. The construction procedures shall be observed during embankment construction, or prior to foundation concrete placement. This observation will be to verify that the foundation width and depths are per this report and the associated restrictions imposed by these referenced reports, which are considered part of this document.

Site drainage must conform to the criteria previously established addressing bluff protection. All drainage shall be directed away from the dwelling foundations. All drainage to the public right-of-way (roadway) shall be in conformance with applicable standards (City of Carlsbad, California).

This report is an update to the attached report dated March 16, 1998.

Limitations

Professional judgments presented herein are based partly on our evaluation of the available technical information that was reviewed, partly on our understanding of the proposed construction, and partly on our general experience in the civil and geotechnical field.

The recommendations and civil engineering basis of design were based on the assumption that the soil conditions present at the site do not deviate considerably from those presented in the referenced reports and investigations performed by others (see references). If variations or undesirable geotechnical or soil conditions are encountered during construction, the engineering geologist and civil engineer should be notified immediately and consulted for further recommendations. We do not guarantee the performance of the project in any respect.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractors operations, and cannot be responsible for the safety of other than our own personnel on the subject site; therefore, 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 safe.

Sincerely,

Owen Engineering Group

REGISTERED PROFESSIONAL ENGINEER
CHARLES JOHN RANDLE
No. C22096
Exp. Date 9/30/05
CIVIL
STATE OF CALIFORNIA

C. J. Randle, P.E.
Principal Engineer
RCE 22096 CA

References:

1. Geotechnical Update, Carlsbad Beach Lot, APN 210-120-30, Carlsbad Drive, Carlsbad, California; 1996, by Geotechincs, Inc., Project No. 0319-001-00, dated September 3, 1996.

2. Geotechnical Investigation and Bluff Retreat Study, Parcel No. 210-120-30, Carlsbad, California; 1991, by ICG, Inc., Job No. 04-8529-001-00-00, Dated March 28, 1991. 0.40

0.33 3. Geotechnical Investigation and Bluff Retreat Study, Parcel No. 210-120-31, Carlsbad, California; 1989, by ICG, Inc., Job No. 05-8109-001-00-00, Dated November 6, 1989.

4. California's Battered Coast; 1985 California Coastal Commission Meeting, Professional Papers and Publications, San Diego, California, September, 1985 San Diego Association of Geologists.

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6. Geotechnical Investigation, Proposed Coastal Protection Measures for Ecke Site, Carlsbad, California; 1984, by Converse Consultants, Project No. 83-02299-01, dated February 1, 1984.

Attachment:

1. Assumption of Geotechnical Responsibility, Updated Geotechnical Report, Assessor Parcel Number 210-120-30, Carlsbad, CA, by C. J. Randle, P.E., Dated March 16, 1998.

Excerpts from:

1. From Reference # 2 above, Pages 14 through 19.

2. From Reference # 3 above, Pages 10, 12, & 13 and Figures 2 & 3.

3. Jon Jensen, Tentative Parcel Map

ATTACHMENT 1

C. J. Randle, P.E., Civil Engineer
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Telephone (619) 571-6271
Fax (619) 571-3943

March 16, 1998

Mr. Jon Jensen
Jon A. Jensen and Associates
451 South Escondido Blvd.
Escondido, CA 92025

Subject: Assumption of Geotechnical Responsibility
Updated Geotechnical Report
Assessor Parcel Number 210-120-30
Carlsbad, California

References:

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Review available geological reports and data pertinent to the subject site. The list of referenced and/or review data is included on page 1 of this report.

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Preparation and processing of this report.

In addition, our study is supplemented by our engineering services, subsurface work and observation services during construction of a stairway access to the beach along the south property boundary of the subject site placed into the bedrock and constructed on caissons.

PROPOSED DEVELOPMENT

The actual plans for future development at the property are not yet complete, but include provisions that will preclude any excess drainage from within the site to be channeled over the sea bluff face. This is proposed to be accomplished through landscaping controls, graded earthen swales and other devices to collect and divert water away from the sea bluff. Additionally, the surface drainage of the property is enhanced by the surface drainage provisions, which have been developed in and around the concrete beach access stairway along the south boundary of the property.

SUMMARY OF REVIEW

Based on our study, references 1, 2, 5 and 6 adequately describe the property soil and geological conditions and the report conclusions and recommendations are still appropriate and applicable to development of the property except for those items described within the discussion section of this report.

DISCUSSION

Previous reports based sea cliff retreat and construction setbacks upon an "empirical" figure of 25 feet extrapolated over 62 years (ref. 2). However, the same report indicates "relatively little changes" to the bluff top. Reference 3 indicates "There has been little retreat of the bluff top in the vicinity. . . " of the site based on a study interval from 1929 through 1984.

The reports also indicate a concern relative to the cove area of the site. The cove area is reported to have undergone a short term interval of episodic and relative rapid erosion. Cove areas are not uncommon along the coastal area of San Diego County, and even if underlain by more erodible or "softer" material once the weaker or more erodible material forms the cove feature, that natural set back is sufficient to create a buffer from the brunt of direct wave attack. Thus, once formed, the top of bluff retreat rate for the cove area becomes similar to that of adjacent tops of bluff to both sides of the cove.

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For the property, we can assume for the 75 year design period that a rate of retreat of about 6 to 19 feet might occur. Because there was a period of relatively rapid episodic erosion for the site area in about 1977 through 1983, we can assume that the longer term overall rate will be less and more likely on the order of about 6 to 12 feet, or less. These rates will be significantly affected and reduced in the event that longer term sea cliff erosion control devices are constructed at the site.

0.8 → 0.16

CONCLUSIONS AND RECOMMENDATIONS

Based on our study, the previous soil and geologic findings for the site are still appropriate and applicable for the site, except for those items outlined in the Discussion section of this report and updated in the following paragraphs. We assume soil and geologic responsibility for the previous reports, findings, conclusions and recommendations of the previous reports.

Based on the site conditions and our study, the rate of retreat for the top of bluff at the project site is established to be on the order of 1 to 2 inches per year for the 75 year planned life of the proposed development of the property.

0.08 → 0.16

The bluff retreat may be addressed with the following dimensions and criteria:

1. A 25 foot set back from the face of the bluff with all foundations to be placed on deep caissons founded in firm dense to very dense, formational sandstone (bedrock). The bedrock appears to be at maximum depths of elevations of -4 to -6 feet (msl) at elevation +8 and increases easterly.

2. Bluff setbacks will be established at 30 feet from the face of the bluff for all foundation systems which conform to the current Uniform Building Code criteria. However, conventional foundation systems will require development of a deepened grade beam foundation system which will be a minimum of 36 inches below lowest adjacent grade.

3. Typical slab on grade construction with minimal footing depths may be constructed with a set back of 40 feet established from the face of the bluff.

Based on our elevation of the recommended setbacks (with relation to the proposed 3 lot Parcel Map) each lot will easily provide for the necessary building area, without encroaching into the proposed setback. Similarly, the remaining land area will easily accommodate this proposed development.

The option of set backs is prudent and will easily address coastal retreat, which essentially will be mitigated by the restrictions placed on the site drainage controls. The 30 and 40 foot options are for all intents and purposes a conservative response to the bluff top setbacks. When compared to the proposery 25 foot setback for the northerly and adjacent vacant parcel.

Actual foundation recommendations for future proposed site development and construction will require a specific analysis study which should address the type of foundations outlined above and other factors unique to each structure.

LIMITATIONS

Soil and bedrock conditions may vary in character and soil moisture content from those disclosed during the previous site subsurface studies. We assume no responsibility or liability for work, testing, or recommendations performed or provided by others.

The conclusions and recommedations contained herein are professional opinions. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied.

Should you have any questions, or require additional service please do not hesitate to contract us.

Very truly yours,

Ernest R. Artim
CEG 1084

Charles J. Randle
RCE 22096

Distribution: (3) client

EXCERPT 1

8.0 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

8.1 General Discussion

No geotechnical conditions were apparent during our investigation which would preclude the site development as planned. The site condition which should most severely impact the development is the potential for future erosion and slope stability of the bluff area. The on-site soils are considered non-expansive and should be suitable to support the proposed structure once the existing fills and loose soils have been removed or recompacted.

The following recommendations are intended to provide a suitable bearing pad for foundations. We recommend that all loose soils or undocumented fill soils within the building area be removed and replaced as compacted fill. Once these soils have been prepared, the structure can be supported on a conventional shallow foundation system bearing entirely on bedrock or compacted fill.

The remainder of Section 8.0 presents our recommendations for foundation design and construction in detail. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations appear not to cover any specific feature of the project, please contact our office for additions or revisions to our recommendations.

8.2 Structural Setback and Slope Stability

A structural setback from the bluff rim is recommended to protect dwellings from collapse within the economic life of the residence. The setback depicted on the Geotechnical Maps (Plates 1 and 3) is based on slope stability analysis along two cross sections through the site (Plate 2). Engineering practice uses a factor of safety of 1.5 as the minimum criteria for a stable slope (a factor of safety of less than 1.0 indicates that a slope should fail). Slope stability calculations indicate that the trial failure surfaces behind the setback line exhibit a factor of safety greater than 1.5. Trial

failure surfaces west of the setback line had factors of safety less than 1.5. Our analysis was conducted using the STABL5 computer program. Failure surfaces were generated for circular type failures. Circular failures were used in the analysis because of the absence of continuous places of weakness which are used in block glide type failures. Soil strengths parameters used in our calculations were based on the residual strength of the soil as calculated from direct shear tests in our laboratory. It is our understanding that no new slopes are currently planned for the site. If future plans include slopes or the placement of fill beyond the setback line, ICG Inc. should be contacted to analyze the slope stability.

The major factor affecting the future stability of the existing bluff is the erosion potential of the bluff soils. As discussed in Section 6.4, the bluff retreat rate is estimated to be 0.4 feet per year. A factor of safety of 1.5 is applied, and the result is multiplied by 75 years, ~~to arrive at an estimated 45 feet of~~ bluff retreat over a 75 year economic life for the proposed residence. The loss of 45 feet of bluff, back from the existing rim, would still fall well short of the structural setback establish through slope stability calculations and the 1.5 factor of safety. Appurtenant structures, temporary structures, decks and landscaping constructed ~~beyond the setback will incur a high risk of potential loss over time.~~ The risk for damage increases as the bluff top is approached. It is recommended that measures be taken to prevent the erosion of the bluff face. Such measures could include the channeling of surface drainage away from the bluff face, construction of a sea wall and/or covering the bluff face with an erosion resistant material, ie. rip rap, or gunite.

8.3 Grading and Earthwork

8.3.1 Geotechnical Observation

During grading, ICG Inc. should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by the preliminary investigations, to adjust designs to actual field conditions, and to determine that the grading

is in general accordance with the recommendations of this report. Our personnel should perform sufficient testing of any fill placed to support our opinion as to whether compaction recommendations have been complied with.

8.3.2 Site Preparation

Site preparation should begin with the removal of all vegetation and deleterious materials within the area to be graded. In addition all loose soils and undocumented fill should be completely removed down to firm bedrock materials. The exposed bedrock should be scarified to a depth of 8 to 12 inches and recompacted to 90 percent of maximum density, ASTM D-1557. Native on-site soils can be used as fill material, provided all deleterious materials have been completely removed.

If during grading a cut/fill transition is created, and deep foundations are not used in the fill portion, the following recommendation should be implemented. The cut portion of the pad should be overexcavated to a depth of 5 feet below finish grade and replaced as a compacted fill. The lateral extent of the excavation should be 5 feet out side of the building perimeter. All fill soils should be compacted as recommended in Section 8.3.3 Fill Compaction.

8.3.3 Fill Compaction

All fill and backfill to be placed in association with site development should be accomplished at slightly over optimum moisture conditions and using equipment that is capable of producing a uniformly compacted product. The minimum relative compaction recommended for fill is 90 percent of maximum density based on ASTM D1557 (modified Proctor). Sufficient observation and testing should be performed by the geotechnical consultant so that an opinion can be rendered as to the degree of compaction achieved.

Representative samples of imported materials and on site soils should be tested by the geotechnical consultant in order to evaluate the maximum density, optimum moisture content, and where appropriate, shear strength, consolidation, and expansion characteristics of the soil.

During grading operations, soil types other than those analyzed in the geotechnical reports may be encountered by the contractor. The geotechnical consultant should be notified to evaluate the suitability of these soils for use as fill and as finish grade soils.

8.3.4 Trench Backfill

All trench backfill should be compacted by mechanical means in uniform lifts of 8 to 12 inches. The backfill should be uniformly compacted to at least 90 percent of ASTM D1557.

8.4 Site Drainage

Foundation and slab performance depends greatly on how well the runoff waters drain from the site. This is true both during construction and over the entire life of the structure. The ground surface around structures should be graded so that water flows rapidly away from the structures without ponding. The surface gradient needed to achieve this depends on the prevailing landscape. In general, we recommend that pavement and lawn areas within five feet of buildings slope away at gradients of at least two percent. Densely vegetated areas should have minimum gradients of at least five percent away from buildings in the first five feet. Densely vegetated areas are considered those in which the planting type and spacing is such that the flow of water is impeded.

Planters should be built so that water from them will not seep into the foundation, slab, or pavement areas. Site irrigation should be limited to the minimum necessary to sustain landscaping plants. Should excessive irrigation, water line breaks, or

unusually high rainfall occur, saturated zones or "perched" groundwater may develop in fill soils.

Because of the highly erosion prone nature of the on-site soils additional care should be taken to direct runoff water away from the top of the bluff. Drainage channels and swales should be constructed to help channel water away from erosion prone areas. We recommend that drainage water be carried in a pipe or conduit to the base of the bedrock bench in the lower portion of the bluff, rather than be allowed to flow freely over the bluff face.

8.5 Foundation Recommendations

8.5.1 General

Special design considerations for soil expansion are not considered necessary. All foundations should bear entirely on either uniformly compacted fill or on bedrock. The following paragraphs in Section 8.6 will present our recommendations for foundation design. Our recommendations are considered to be generally consistent with the standards of practice. They are based on both analytical methods and empirical methods derived from experience with similar geotechnical conditions. Reinforcement recommendations are considered the minimum necessary for the likely soil conditions and are not intended to supersede the design of the Structural Engineer or criteria of governing agencies.

8.5.2 Foundations on Bedrock

If the building area is prepared such that all footings will bear entirely on bedrock the following foundation design parameters should be applicable.

Allowable Soil Bearing:	3000 psf (allow a one-third increase for short-term wind or seismic loads)
Minimum Footing Width:	12 inches
Minimum Footing Depth:	18 inches
Minimum Reinforcement:	1 no. 4 bar at both top and bottom in continuous footings, or design as simply supported beam capable of supporting the applied loads over a span of 4 feet, whichever is greater. Equal resistance to positive and negative moments should be provided.

8.5.3 Foundations on Fill

If all footings will bear entirely on compacted fill soils, the following foundation design parameters should be applicable.

Allowable Soil Bearing:	2000 psf (allow a one-third increase for short-term wind or seismic loads)
Minimum Footing Width:	12 inches
Minimum Footing Depth:	18 inches
Minimum Reinforcement:	1 no. 5 bar at both top and bottom in continuous footings, or design as simply supported beam capable of supporting the applied loads over a span of 6 feet, whichever is greater. Equal resistance to positive and negative moments should be provided.

8.5.4 Settlement

The anticipated total and differential settlement for the proposed structure should be within tolerable limits provided that the recommendations of this report are followed. In general, total settlements are estimated to be less than 1 inch, and differential settlement is expected to be less than ½ inch. It is recommended that we review the actual foundation plans to evaluate the footing configurations and loading conditions.

EXCERPT 2

during extremely high tides and high surf. Additional erosion occurs as a result of ground water seepage at the contact between the terrace deposit and Santiago Formation bedrock.

The other major factor in bluff stability is the effect of surface water erosion. The mesa area above the bluff generally drains to the west. As the sheet flow nears the bluff top it is concentrated into shallow gullies. As the gullies coalesce, they become deeper. Where the gullies dump over the bluff top, the gullies are over 5 feet deep. Over time the gullies migrate up-gradient (eastward) and laterally, effectively moving the top of the bluff eastward.

6.3 Bluff Retreat Rates

Bluff collapse or block topples are episodic and catastrophic. There are long periods when there are no failures followed by one or two events where tens of feet of bluff may retreat. Aerial photographs taken in 1929 and 1953 (figures 2 and 3) were compared with photographs taken by Gerald Kuhn in 1978 and 1983 and maps prepared in 1984. The base line for measurements were the railroad tracks east of the site (Location Map, Figure 1). The photographs indicate that there has been little retreat of the bluff top in the vicinity with the exception of the "cove" area described by Converse Consultants (1984).

The anomalous retreat in the "cove" area is a result of a lack of a bedrock bench to protect the erodible terrace deposit in that portion of the bluffs. Additionally,

there is an offshore channel that refracts unbroken waves into this section of beach. Without the benefit of an offshore reef to dissipate wave energy, the "cove" is subject to more violent wave attack.

The bluff top within the subject property does not appear to have retreated in the past 61 years. The storm in August, 1983, did remove roughly 20 feet from the base of the upper bluff without collapse of the bluff top. The result was to steepen the bluff above the bedrock bench.

Future retreat of the bluff at the subject site will be controlled by several factors:

a. A large reef is located just offshore from the property. Waves break on the reef rather than on the beach or against the bluffs thereby reducing their energy significantly.

b. The bedrock bench has not been undermined and does not contain open joints or other structural weaknesses. The bench is 20 feet wide and 5 feet above the present beach. The bench should protect the more erodible terrace deposit and fill by acting like a natural sea wall. Direct wave attack on the base of the upper bluff would only occur when the tide and storm surge is in excess of +13 feet of present mean sea level.

c. The fill present in the old erosional gully and exposed in the upper bluff face within the property contains chunks of concrete and asphalt. The fill

appears to be draped over the erodible portion of the terrace deposit. The fill will act, to a certain extent, as rip-rap. While the fill is erodible, it appears much less erodible than the friable sand in the terrace deposit.

d. The subject site is roughly 100 feet north of the "cove" area. Further erosion in the "cove" will not likely affect the proposed development for well beyond the economic life of the residence.

A structural setback has been established for the subject property based on the historical bluff retreat in the area and a 75 year economic life of the residence. Although the bluff top does not appear to have retreated, the 20 feet of erosion at the base of the upper bluff has been extrapolated over the 61 years of documentation to estimate a average retreat rate of 0.33 feet per year. A factor of safety of 1.5 is applied and the result is multiplied by 75 years to arrive at the 40 foot setback from the bluff top shown on Plate I.

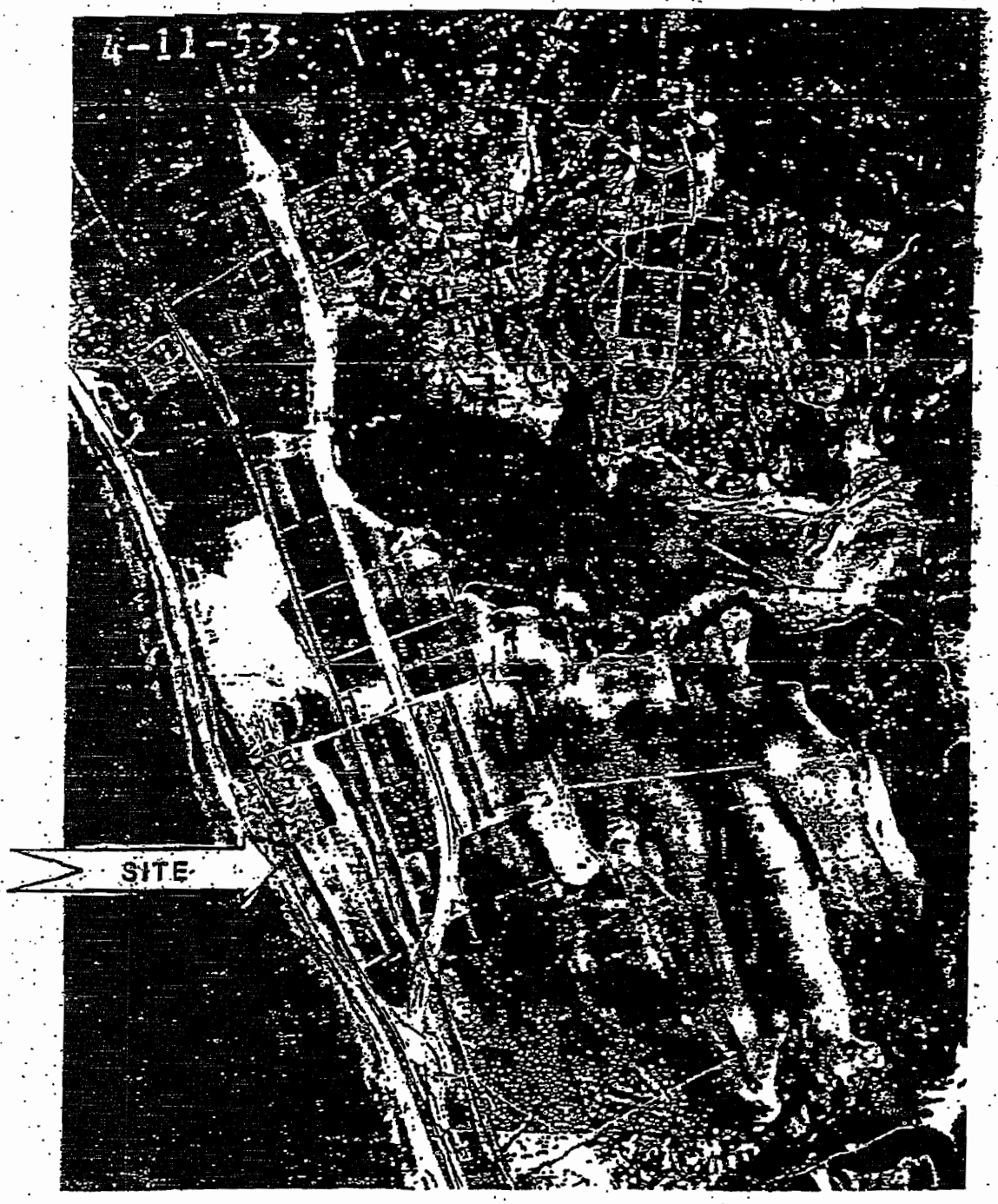
7.0 SEISMICITY

7.1 General

The site is considered to be a seismically active area, as can all of southern California. There are, however, no known active faults either on or adjacent to the project site. Figure 4 shows the known active faults and major earthquake epicenters in the region and their geographic relationship to the site. Because these active faults are at a substantial distance, the seismic

1929 PHOTO

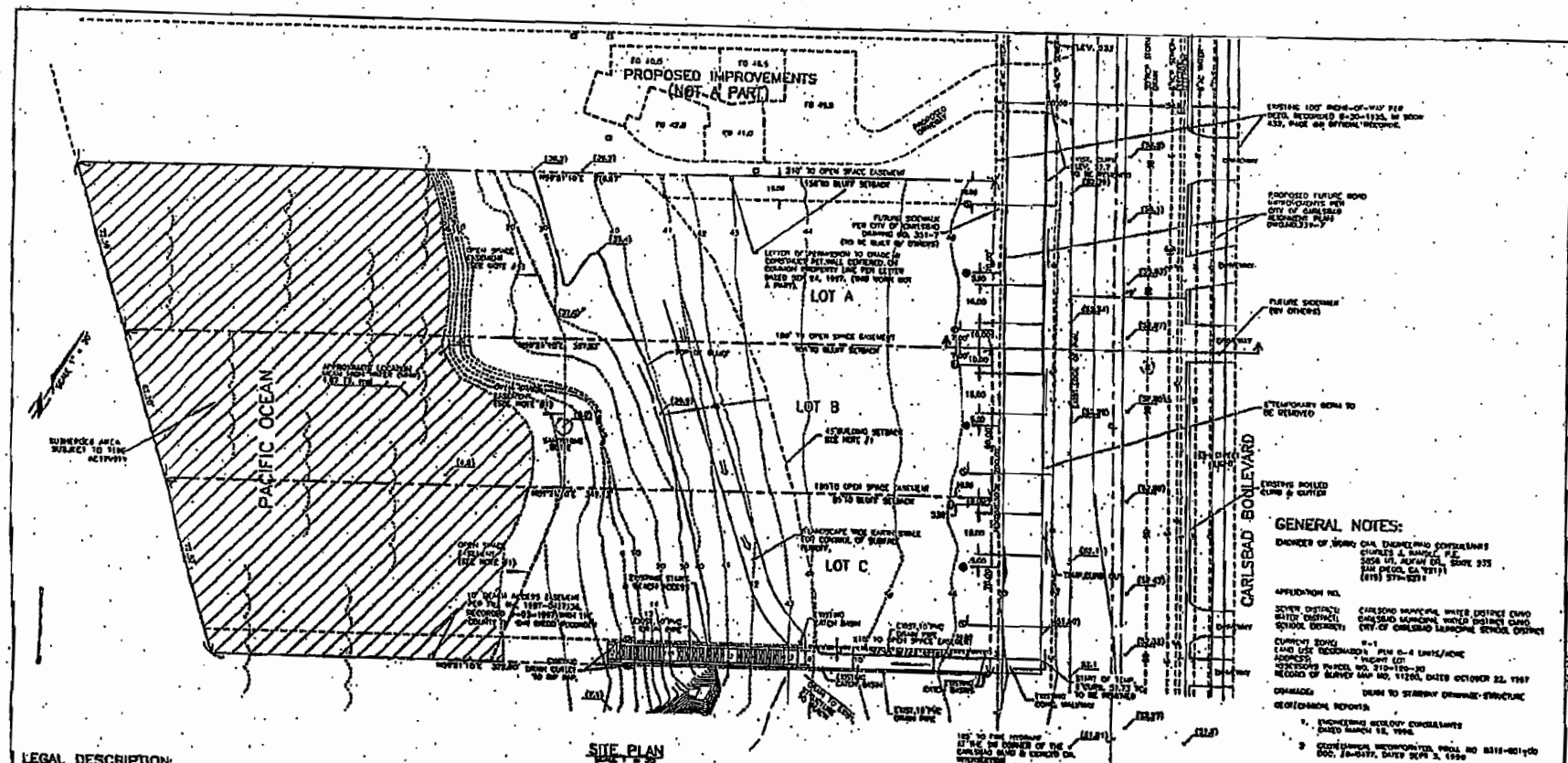
JOB NO.: 04-8109-001-00-00	DATE: November 1989	FIGURE: 2



1953 PHOTO

JOB NO.: 04-8109-001-00-00	DATE: November 1989	FIGURE: 3

EXCERPT 3



LEGAL DESCRIPTION:

THIS PORTION:

OF PENINSULA BEACH UNIT NO. 1, IN THE CITY OF CARLSBAD, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO MAP THEREOF NO. 1906, FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, MAY 26, 1954, TOGETHER WITH THE SOUTHERLY HALF OF THE VACATED STREET ADJOINING, AS SHOWN ON RESOLUTION NO. 5180 OF THE BOARD OF SUPERVISORS, RECORDED IN BOOK 5180, PAGE 169 OF OFFICIAL RECORDS, IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, NOVEMBER 16, 1964, MORE PARTICULARLY DESCRIBED AS FOLLOWS:

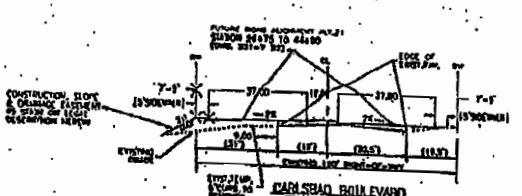
BEGINNING AT A POINT ON THE SOUTHWESTERLY RIGHT-OF-WAY LINE OF CARLSBAD BOULEVARD BEARING SOUTH 30°00'00" EAST, 30.0 FEET FROM THE NORTHEASTERLY CORNER OF LOT 1 OF TOWNSITE UNIT NO. 1, ACCORDING TO MAP THEREOF NO. 3494, FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, THE TRUE POINT OF BEGINNING; THENCE SOUTH 30°00'00" EAST 200.00 FEET; TO THE NORTHERLY CORNER OF LOT 2 OF SAID UNIT NO. 1; THENCE SOUTH 60°00'00" WEST ALONG THE NORTHWESTERLY LINE OF THE PACIFIC OCEAN; THENCE NORTHWESTERLY ALONG SAID MEAN HIGH TIDE LINE TO A LINE BEARING NORTH 60°00'00" WEST FROM THE TRUE POINT OF BEGINNING; THENCE NORTH 60°00'00" EAST TO THE TRUE POINT OF BEGINNING.

EXISTING EASEMENTS:

THE PROPERTY HAS THE RIGHT TO CONSTRUCT, MAINTAIN, REPAIR AND REPLACE A DRAINAGE SLOPE BEYOND THE LINES OF THE RIGHT OF WAY GRANTED THEREIN TO THE CITY OF CARLSBAD WHERE REQUIRED FOR THE CONSTRUCTION AND MAINTENANCE OF THE RIGHT OF WAY AS CONTAINED IN THE DEED RECORDED AUGUST 31, 1959, IN THE BOOK 7317, PAGE 90, OFFICIAL RECORDS, EXCEPTING THEREFROM:

AN EASEMENT FOR THE SOLE PURPOSE OF PEDESTRIAN BEACH ACCESS, TO THE PUBLIC FOR, IN AND ALONG DEVICES ADOPTION, RECORDED DECEMBER 5, 1990, AS FILE NO. 90-[illegible], OFFICIAL RECORDS. (EASEMENT NOT PLOTTABLE)

(A) EASEMENT FOR THE PURPOSE OF PEDESTRIAN BEACH ACCESS TO THE PUBLIC FOR THE CITY OF CARLSBAD, BETWEEN CARLSBAD BOULEVARD AND THE PACIFIC OCEAN, RECORDED SEPTEMBER 3, 1997, AS FILE NO. 1997-[illegible], OF OFFICIAL RECORDS.



LEGEND:

- EXISTING SPOT ELEVATION
- PROPOSED SPOT ELEVATION
- PROPERTY LINE
- EDGE OF PAVEMENT
- CENTERLINE
- TOP OF EXISTING BLUFF
- BLUFF SETBACK
- DRAINAGE TO EXISTING DRAINS
- TEMPORARY 6" CURB
- TEMPORARY 6" WIDE BY 12" DEEP EARTH SWALE OR SAND BAG BERM
- WATER LATERAL PROPOSED LOCATION
- SEWER LATERAL PROPOSED LOCATION
- UTILITIES LATERAL PROPOSED LOCATION
- EXISTING CATCH BASINS
- PROPOSED CATCH BASIN

NOTES:

1. AN OPEN SPACE AREA SHALL BE A CONDITION OF MAP DEDICATION OF OPEN SPACE SHALL BE THAT PART OR PORTION OF EACH RESIDENTIAL LOT BEGINNING ON THE WESTERN BOUNDARY OF THE PROPERTY AND PROCEEDING EASTERLY FOR A DISTANCE NOT EXCEEDING 4 LOT NOT EXCEEDING A DEPTH OF 50 LENGTH TO WIDTH PURSUANT TO THE CARLSBAD MUNICIPAL CODE 20.16.20 (3).
2. ALL SURFACE DRAINAGE AND DRAINAGE SYSTEMS ON THESE PROPERTIES SHALL BE PLANTED WITH SPECIFIC GROUND COVERING AS REQUIRED TO CONTROL EROSION AND POLLUTION, IMMEDIATELY FOLLOWING TENTATIVE MAP APPROVAL.
3. ALL SURFACE DRAINAGE AND FUTURE WATER SERVICE ROUTES, DECKS, PATIOS AND DRIVEWAYS SHALL BE GRADED AWAY FROM THE TOP OF THE BLUFF AND TO THE EXISTING SWALES, OR STORMS LOCATED ON THE NORTH SIDE OF PARCEL 2.
4. THE EXISTING CURB LOCATED EAST OF THE PROPERTY HAS CONSTRUCTED AS PART OF A PREVIOUSLY APPROVED COASTAL DEVELOPMENT PERMIT, HOWEVER, SUCH CURB IS TEMPORARY AND WILL BE REMOVED IMMEDIATELY FOLLOWING TENTATIVE MAP APPROVAL.
5. NO GRADING IS BEING REQUESTED WITH THIS SUBMITTAL AND A GRADING PLAN IS NOT INCLUDED HERE.

BENCH MARK

WALL MONUMENT, CORNER ROAD ELEVATION [illegible] (CLAIM) PER CITY OF CARLSBAD DEVELOPMENT PLAN OF CITY OF CARLSBAD DEVELOPMENT PLANS, DRAWING [illegible]

GENERAL NOTES:

ENGINEER OF WORK: CIVIL ENGINEERING CONSULTANTS
CHARLES A. RANDLE, P.E.
5858 MT. ALIFAN DR., SUITE 238
SAN DIEGO, CA 92111
(619) 571-6271

APPLICATION NO.

SEWER DISTRICT: CARLSBAD MUNICIPAL WATER DISTRICT (SMD)
WATER DISTRICT: CARLSBAD MUNICIPAL WATER DISTRICT (SMD)
SCHOOL DISTRICT: CITY OF CARLSBAD MUNICIPAL SCHOOL DISTRICT

CURRENT ZONING: R-1
LAND USE DESIGNATION: PLN 0-4 UNITS/ACRE
ADDRESS: VACANT LOT
ASSESSORS PARCEL NO. 210-120-30
RECORD OF SURVEY MAP NO. 11260, DATED OCTOBER 22, 1987

DRAINAGE: DRAIN TO STORMDRAIN DRAINAGE STRUCTURE

GEOTECHNICAL REPORTS:

1. ENGINEERING GEOLOGY CONSULTANTS DATED MARCH 18, 1996.
2. GEOTECHNICAL INCORPORATED, PROJ. NO. 0318-001-00 DOC. 7/0-0177, DATED SEPT. 3, 1990
3. KCI INCORPORATED, JOB NO. 03-0109-001-00-00 LOG NO. 1-[illegible], DATED NOVEMBER 8, 1995 AND JOB NO. 01-0370-001-00-00, LOG NO. 1-[illegible] DATED MARCH 28, 1996
4. CONTROL CONSULTANTS, PROJECT NO. 03-[illegible]-02 & 01 DATED SEPTEMBER 20, 1988 AND FEBRUARY 1, 1991

CIVIL ENGINEERING CONSULTANTS
5858 MT. ALIFAN, SUITE 238
SAN DIEGO, CA 92111
(619) 571-6271

JON JENSEN
TENTATIVE
PARCEL MAP

JON JENSEN
CARLSBAD, CA

DATE	[illegible]
DRAWN	[illegible]
SCALE	1" = [illegible]
REVISION	[illegible]
REVISION	[illegible]
REVISION	[illegible]
DESIGN	[illegible]
DRAWN	[illegible]
CHECKED	[illegible]
DATE	[illegible]

1 of 1

00221

EXHIBIT G

PLANNING COMMISSION RESOLUTION NO. 5151

A RESOLUTION OF THE PLANNING COMMISSION OF THE CITY OF CARLSBAD, CALIFORNIA, APPROVING COASTAL DEVELOPMENT PERMIT CDP 01-13 ON PROPERTY GENERALLY LOCATED ALONG THE WEST SIDE OF CARLSBAD BOULEVARD, BETWEEN CEREZO DRIVE AND SHORE DRIVE IN LOCAL FACILITIES MANAGEMENT ZONE 3.

CASE NAME: GOETZ RESIDENCE
CASE NO.: CDP 01-13

WHEREAS, **Roy Asaro**, "Developer," has filed a verified application with the City of Carlsbad regarding property owned by **Dean Goetz Trust**, "Owner," described as

> **Parcel 2 of Parcel Map 18236, City of Carlsbad, County of San Diego, State of California, filed in the Office of the County Recorder, County of San Diego, on April 13, 1999**

("the Property"); and

WHEREAS, said verified application constitutes a request for a Coastal Development Permit as shown on Exhibits "A" – "M" dated **February 20, 2002**, on file in the Planning Department, **GOETZ RESIDENCE – CDP 01-13** as provided by Chapter 21.201.040 of the Carlsbad Municipal Code; and

WHEREAS, the Planning Commission did, on the **20th** day of **February 2002**, hold a duly noticed public hearing as prescribed by law to consider said request; and

WHEREAS, at said public hearing, upon hearing and considering all testimony and arguments, if any, of all persons desiring to be heard, said Commission considered all factors relating to the CDP.

NOW, THEREFORE, BE IT HEREBY RESOLVED by the Planning Commission of the City of Carlsbad as follows:

A) That the foregoing recitations are true and correct.

B) That based on the evidence presented at the public hearing, the Commission APPROVED **GOETZ RESIDENCE – CDP 01-13** based on the following findings and subject to the following conditions:

Findings:

1. That the proposed development is in conformance with the Mello II segment of the Certified Local Coastal Program and all applicable policies in that the site is designated for single-family residential developments and the development is a single-family house on a previously subdivided lot; the project conforms to the already established blufftop setback; no agricultural activities, sensitive resources, geological instability, flood hazard or coastal access opportunities exist on site; that buildings, structures and landscaping have been so located as to preserve to the extent feasible any ocean views from the nearest public street. The project has been designed with a one-story element on Carlsbad Boulevard and the structure complies with applicable side yard requirements, which will, to the extent feasible, preserve existing ocean views from the street. Moreover, unobstructed views from Carlsbad Boulevard of the ocean exist less than 100 feet south of the project and extend for at least several thousand feet to the south.

2. The project is consistent with the provisions of the Coastal Resource Protection Overlay Zone (Chapter 21.03 of the Zoning Ordinance) in that the project will adhere to the City's Master Drainage and Storm Water Quality Management Plan and Grading Ordinance to avoid increased runoff and soil erosion. The development is not located in an area prone to landslides or susceptible to accelerated erosion, floods, or liquefaction. The existing 25+% slopes do not support any endangered plant/animal species and/or coastal sage scrub and chaparral plant communities and therefore are not considered dual criteria slopes. Appropriate findings, included in the staff report, can be made to allow the development of the 25+% slopes.

3. The proposal is in conformity with the public access and recreation policies of Chapter 3 of the Coastal Act in that a beach access stairway was previously installed (CDP 98-05) within 75 feet of the property and the subject property was not identified as a potential lot for future additional public access to the shoreline in the Local Coastal Program.

4. The project is consistent with the provisions of the Coastal Shoreline Development Overlay Zone (Chapter 21.204 of the Zoning Ordinance) in that vertical access exists within 75' of the project. Historically, the site has not been used for vertical access. No vertical access is warranted for this development based upon the ordinance criteria. A geotechnical analysis of the project site was prepared. The analysis concluded that the proposed development will have a useful life of at least 75 years and that the development will have no adverse effects on the stability of the coastal slope. The proposed residential structure has been designed to conform to the already established blufftop setback and will be compatible with the surrounding development and natural environment. Grading has been kept to a minimum, although a grading permit will be required. The project adheres to all coastal "stringline" setback requirements for the placement of structures, decks/balconies, fences and pool/spas.

5. That the Planning Director has determined that the project belongs to a class of projects that the State Secretary for Resources has found do not have a significant impact on the environment, and it is therefore categorically exempt from the requirement for the

preparation of environmental documents pursuant to Section **15303 (single-family residence in an urbanized area)** of the state CEQA Guidelines. In making this determination, the Planning Director has found that the exceptions listed in Section 15300.2 of the state CEQA Guidelines do not apply to this project.

6. The project is consistent with the City-Wide Facilities and Improvements Plan, the Local Facilities Management Plan for Zone 3 and all City public facility policies and ordinances. The project includes elements or has been conditioned to construct or provide funding to ensure that all facilities and improvements regarding: sewer collection and treatment; water; drainage; circulation; fire; schools; parks and other recreational facilities; libraries; government administrative facilities; and open space, related to the project will be installed to serve new development prior to or concurrent with need. Specifically,

   A. The project has been conditioned to provide proof from the **Carlsbad Unified School District** that the project has satisfied its obligation for school facilities.

   B. The Public Facility fee is required to be paid by Council Policy No. 17 and will be collected prior to the issuance of building permit.

7. The project is not located in the Coastal Agriculture Overlay Zone, according to Map X of the Land Use Plan, certified September 1990 and, therefore, is not subject to the provisions of the Coastal Agriculture Overlay Zone (Chapter 21.202 of the Zoning Ordinance).

8. The Planning Commission has reviewed each of the exactions imposed on the Developer contained in this resolution, and hereby finds, in this case, that the exactions are imposed to mitigate impacts caused by or reasonably related to the project, and the extent and the degree of the exaction is in rough proportionality to the impact caused by the project.

Conditions:

Note: Unless otherwise specified herein, all conditions shall be satisfied prior to **building permit or grading permit, whichever occurs first.**

1. If any of the following conditions fail to occur; or if they are, by their terms, to be implemented and maintained over time, if any of such conditions fail to be so implemented and maintained according to their terms, the City shall have the right to revoke or modify all approvals herein granted; deny or further condition issuance of all future building permits; deny, revoke or further condition all certificates of occupancy issued under the authority of approvals herein granted; institute and prosecute litigation to compel their compliance with said conditions or seek damages for their violation. No vested rights are gained by Developer or a successor in interest by the City's approval of this **Coastal Development Permit.**

2. Staff is authorized and directed to make, or require the Developer to make, all corrections and modifications to the **Coastal Development Permit** documents, as necessary to make them internally consistent and in conformity with the final action on the project.

Development shall occur substantially as shown on the approved Exhibits. Any proposed development different from this approval, shall require an amendment to this approval.

3. The Developer shall comply with all applicable provisions of federal, state, and local laws and regulations in effect at the time of building permit issuance.

4. If any condition for construction of any public improvements or facilities, or the payment of any fees in-lieu thereof, imposed by this approval or imposed by law on this Project are challenged, this approval shall be suspended as provided in Government Code Section 66020. If any such condition is determined to be invalid this approval shall be invalid unless the City Council determines that the project without the condition complies with all requirements of law.

5. The Developer/Operator shall and does hereby agree to indemnify, protect, defend and hold harmless the City of Carlsbad, its Council members, officers, employees, agents, and representatives, from and against any and all liabilities, losses, damages, demands, claims and costs, including court costs and attorney's fees incurred by the City arising, directly ~~or indirectly, from (a) City's approval and issuance of this Coastal Development Permit,~~ (b) City's approval or issuance of any permit or action, whether discretionary or non-discretionary, in connection with the use contemplated herein, and (c) Developer/Operator's installation and operation of the facility permitted hereby, including without limitation, any and all liabilities arising from the emission by the facility of electromagnetic fields or other energy waves or emissions. This obligation survives until all legal proceedings have been concluded and continues even if the City's approval is not validated.

6. Prior to the issuance of a building permit, the Developer shall provide proof to the Director from the **Carlsbad Unified School District** that this project has satisfied its obligation to provide school facilities.

7. This project shall comply with all conditions and mitigation measures which are required as part of the Zone 3 Local Facilities Management Plan and any amendments made to that Plan prior to the issuance of building permits.

8. Building permits will not be issued for this project unless the local agency providing water and sewer services to the project provides written certification to the City that adequate water service and sewer facilities, respectively, are available to the project at the time of the application for the building permit, and that water and sewer capacity and facilities will continue to be available until the time of occupancy.

9. The applicant shall apply for and be issued building permits for this project within **two (2) years** of approval or this coastal development permit will expire unless extended per Section 21.201.210 of the Zoning Ordinance.

10. The developer shall submit to the City a Notice of Restriction to be filed in the office of the County Recorder, subject to the satisfaction of the Planning Director, notifying all interested parties and successors in interest that the City of Carlsbad has issued a **Coastal Development Permit** by Resolution No. 5151 on the property owned by the Developer. Said Notice of Restriction shall note the property description, location of the file

containing complete project details and all conditions of approval as well as any conditions of restrictions specified for inclusion in the Notice Restriction. The Planning Director has the authority to execute and record an amendment to the notice which modifies or terminates said notice upon a showing of good cause by the Developer or successor in interest.

11. The Developer shall pay to the City an inclusionary housing in-lieu fee as an individual fee on a per market rate dwelling unit basis in the amount in effect at the time, as established by City Council Resolution from time to time.

12. **Prior to the issuance of building permits, the Developer shall prepare and record a Notice that this property may be subject to noise impacts from the proposed or existing Transportation Corridor, in a form meeting the approval of the Planning Director and City Attorney.**

13. Prior to the issuance of building permits, the applicant shall apply for and obtain a grading permit issued by the City Engineer.

14. If a grading permit is required, all grading activities shall be planned in units that can be completed by October 1st. Grading activities shall be limited to the "dry season", April 1st to October 1st of each year. Grading activities may be extended to November 15th upon written approval of the City Engineer and only if all erosion control measures are in place by October 1st.

Engineering

NOTE: Unless specifically stated in the condition, all of the following conditions, upon the approval of this proposed Coastal Development Permit, must be met prior to approval of a building or grading permit, whichever occurs first.

General

15. Prior to hauling dirt or construction materials to or from any proposed construction site within this project, Developer shall apply for and obtain approval from, the City Engineer for the proposed haul route.

16. Prior to issuance of any building permit, Developer shall comply with the requirements of the City's anti-graffiti program for wall treatments if and when such a program is formally established by the City.

Fees/Agreements

17. Prior to the issuance of a building permit for any structures, the developer shall cause property owner to enter into an encroachment agreement with the City for the existing non-standardized concrete curb located within the public right-of-way.

Grading

18. Based upon a review of the proposed grading and the grading quantities shown on the site plan, a grading permit for this project is required. Developer shall apply for and obtain a

grading permit from the City Engineer prior to issuance of a building permit for the project.

Coastal

19. If a Grading Permit is required, all grading activities shall be planned in units that can be completed by October 1st. Grading activities shall be limited to the "dry season", April 1st to October 1st of each year. Grading activities may be extended to November 15th upon written approval of the City Engineer, obtained in advance, and only if all erosion control measures are in place by October 1st.

Carlsbad Municipal Water District

20. Prior to issuance of building permits, Developer shall pay all fees, deposits, and charges for connection to public facilities. Developer shall pay the San Diego County Water Authority capacity charge(s) prior to issuance of Building Permits.

21. The Developer shall install the potable water service and meter at a location approved by the District Engineer. The locations of said services shall be reflected on public improvement plans.

22. The Developer shall install a sewer lateral and clean-out at a location approved by the District Engineer. The locations of the sewer lateral shall be reflected on the public improvement plans.

Code Reminders

23. Developer shall pay the License Tax on new construction imposed by Carlsbad Municipal Code Section 5.09.030, and CFD #1 special tax (if applicable), and the Citywide Public Facilities Fee imposed by City Council Policy #17, subject to any credits authorized by Carlsbad Municipal Code Section 5.09.040. Developer shall also pay any applicable Local Facilities Management Plan fee for Zone 3, pursuant to Chapter 21.90. All such taxes/fees shall be paid at issuance of building permit. If the taxes/fees and not paid, this approval will not be consistent with the General Plan and shall become void.

24. Approval of this request shall not excuse compliance with all applicable sections of the Zoning Ordinance and all other applicable City ordinances in effect at time of building permit issuance, except as otherwise specifically provided herein.

25. Addresses, approved by the Building Official, shall be placed on all new and existing buildings so as to be plainly visible from the street or access road; color of identification and/or addresses shall contrast to their background color, as required by Carlsbad Municipal Code Section 18.04.320.

...

...

...

PASSED, APPROVED AND ADOPTED at a regular meeting of the Planning Commission of the City of Carlsbad, California, held on the 20th day of February 2002, by the following vote, to wit:

AYES: Chairperson Trigas, Commissioners Heineman, Segall, White, and Whitton

NOES:

ABSENT: Commissioner Dominguez

ABSTAIN: None

Seena Trigas

SEENA TRIGAS, Chairperson
CARLSBAD PLANNING COMMISSION

ATTEST:

Michael J. Holzmiller

MICHAEL J. HOLZMILLER
Planning Director

EXHIBIT H

PLANNING COMMISSION RESOLUTION NO. 5218

A RESOLUTION OF THE PLANNING COMMISSION OF THE CITY OF CARLSBAD, CALIFORNIA, APPROVING COASTAL DEVELOPMENT PERMIT CDP 01-11 ON PROPERTY GENERALLY LOCATED ALONG THE WEST SIDE OF CARLSBAD BOULEVARD, BETWEEN CEREZO DRIVE AND SHORE DRIVE IN LOCAL FACILITIES MANAGEMENT ZONE 3.
CASE NAME: FRYE RESIDENCE
CASE NO.: CDP 01-11

WHEREAS, **Steven T. Ash**, "Developer," has filed a verified application with the City of Carlsbad regarding property owned by **Christopher J. Frye**, "Owner," described as

> **Parcel 3 of Parcel Map No. 18236, City of Carlsbad Minor Subdivision 98-01, in the City of Carlsbad, County of San Diego, State of California, filed in the Office of the County Recorder of San Diego County on April 13, 1999**

("the Property"); and

WHEREAS, said verified application constitutes a request for a Coastal Development Permit as shown on Exhibits **"A" – "H"** dated **July 17, 2002**, on file in the Planning Department, **FRYE RESIDENCE – CDP 01-11** as provided by Chapter 21.201.040 of the Carlsbad Municipal Code; and

WHEREAS, the Planning Commission did, on the **19th** day of **June 2002** and on the **17th** day of **July 2002** hold a duly noticed public hearing as prescribed by law to consider said request; and

WHEREAS, at said public hearing, upon hearing and considering all testimony and arguments, if any, of all persons desiring to be heard, said Commission considered all factors relating to the CDP.

NOW, THEREFORE, BE IT HEREBY RESOLVED by the Planning Commission of the City of Carlsbad as follows:

A) That the foregoing recitations are true and correct.

B) That based on the evidence presented at the public hearing, the Commission APPROVES FRYE RESIDENCE – CDP 01-11 based on the following findings and subject to the following conditions:

Findings:

1. That the proposed development is in conformance with the Mello II segment of the Certified Local Coastal Program and all applicable policies in that the site is designated for single-family residential developments and the development is a single-family house on a previously subdivided lot; the project conforms to the already established blufftop setback; no agricultural activities, sensitive resources, geological instability, or flood hazard exist on site; and coastal access has already been provided with blufftop stairs that buildings, structures and landscaping have been so located as to preserve to the extent feasible any ocean views from the nearest public street. The project has been designed with a single-story building edge running the length of the building and the structure complies with applicable side yard and building height requirements, which will, to the extent feasible, preserve existing ocean views from the street. Moreover, unobstructed views from Carlsbad Boulevard of the ocean exist immediately to the south of the project and extend for at least several thousand feet.

2. The project is consistent with the provisions of the Coastal Resource Protection Overlay Zone (Chapter 21.03 of the Zoning Ordinance) in that the project will adhere to the City's Master Drainage Plan, Grading Ordinance, Storm Water Ordinance, Standard Urban Storm Water Mitigation Plan (SUSMP), and Jurisdictional Urban Runoff Management Program (JURMP) to avoid increased urban runoff, pollutants and soil erosion. The development is not located in an area prone to landslides or susceptible to accelerated erosion, floods, or liquefaction. The existing 25+% slopes do not support any endangered plant/animal species and/or coastal sage scrub and chaparral plant communities and therefore are not considered dual criteria slopes. Appropriate findings, included in the staff report, can be made to allow the development of the 25+% slopes.

3. The proposal is in conformity with the public access and recreation policies of Chapter 3 of the Coastal Act in that a beach access stairway was previously installed (CDP 98-05) on the subject property as it was identified for public access to the shoreline in the Local Coastal Program.

4. The project is consistent with the provisions of the Coastal Shoreline Development Overlay Zone (Chapter 21.204 of the Zoning Ordinance) in that vertical access was previously obtained with the underlying parcel map (MS 98-01) and exists on the site for public access. A geotechnical analysis of the project site was prepared. The analysis concluded that the proposed development will have a useful life of at least 75 years and that the development will have no adverse effects on the stability of the coastal slope. The proposed residential structure has been designed to conform to the already established blufftop setback and will be compatible with the surrounding development and natural environment. Grading has been kept to a minimum, although a grading permit will be required. The project adheres to all coastal "stringline" setback requirements for the placement of structures, decks/balconies, fences and pool/spas.

5. That the Planning Director has determined that the project belongs to a class of projects that the State Secretary for Resources has found do not have a significant impact on the environment, and it is therefore categorically exempt from the requirement for the preparation of environmental documents pursuant to Section **15303 (single-family residence in an urbanized area)** of the state CEQA Guidelines. In making this determination, the Planning Director has found that the exceptions listed in Section 15300.2 of the state CEQA Guidelines do not apply to this project.

6. The project is consistent with the City-Wide Facilities and Improvements Plan, the Local Facilities Management Plan for Zone 3 and all City public facility policies and ordinances. The project includes elements or has been conditioned to construct or provide funding to ensure that all facilities and improvements regarding: sewer collection and treatment; water; drainage; circulation; fire; schools; parks and other recreational facilities; libraries; government administrative facilities; and open space, related to the project will be installed to serve new development prior to or concurrent with need. Specifically,

   A. The project has been conditioned to provide proof from the **Carlsbad Unified School District** that the project has satisfied its obligation for school facilities.

   B. The Public Facility fee is required to be paid by Council Policy No. 17 and will be collected prior to the issuance of building permit.

7. The project is not located in the Coastal Agriculture Overlay Zone, according to Map X of the Land Use Plan, certified September 1990 and, therefore, is not subject to the provisions of the Coastal Agriculture Overlay Zone (Chapter 21.202 of the Zoning Ordinance).

8. The Planning Commission has reviewed each of the exactions imposed on the Developer contained in this resolution, and hereby finds, in this case, that the exactions are imposed to mitigate impacts caused by or reasonably related to the project, and the extent and the degree of the exaction is in rough proportionality to the impact caused by the project.

Conditions:

Note: Unless otherwise specified herein, all conditions shall be satisfied prior to **building permit or grading permit, whichever occurs first.**

1. If any of the following conditions fail to occur; or if they are, by their terms, to be implemented and maintained over time, if any of such conditions fail to be so implemented and maintained according to their terms, the City shall have the right to revoke or modify all approvals herein granted; deny or further condition issuance of all future building permits; deny, revoke or further condition all certificates of occupancy issued under the authority of approvals herein granted; institute and prosecute litigation to compel their compliance with said conditions or seek damages for their violation. No vested rights are gained by Developer or a successor in interest by the City's approval of this **Coastal Development Permit.**

2. Staff is authorized and directed to make, or require the Developer to make, all corrections and modifications to the **Coastal Development Permit** documents, as necessary to make them internally consistent and in conformity with the final action on the project. Development shall occur substantially as shown on the approved Exhibits. Any proposed development different from this approval, shall require an amendment to this approval.

3. The Developer shall comply with all applicable provisions of federal, state, and local laws and regulations in effect at the time of building permit issuance.

4. If any condition for construction of any public improvements or facilities, or the payment of any fees in-lieu thereof, imposed by this approval or imposed by law on this Project are challenged, this approval shall be suspended as provided in Government Code Section 66020. If any such condition is determined to be invalid this approval shall be invalid unless the City Council determines that the project without the condition complies with all requirements of law.

5. The Developer/Operator shall and does hereby agree to indemnify, protect, defend and hold harmless the City of Carlsbad, its Council members, officers, employees, agents, and representatives, from and against any and all liabilities, losses, damages, demands, claims and costs, including court costs and attorney's fees incurred by the City arising, directly or indirectly, from (a) City's approval and issuance of this **Coastal Development Permit**, (b) City's approval or issuance of any permit or action, whether discretionary or non-discretionary, in connection with the use contemplated herein, and (c) Developer/Operator's installation and operation of the facility permitted hereby, including without limitation, any and all liabilities arising from the emission by the facility of electromagnetic fields or other energy waves or emissions. This obligation survives until all legal proceedings have been concluded and continues even if the City's approval is not validated.

6. Prior to the issuance of a building permit, the Developer shall provide proof to the Director from the **Carlsbad Unified School District** that this project has satisfied its obligation to provide school facilities.

7. This project shall comply with all conditions and mitigation measures which are required as part of the Zone 3 Local Facilities Management Plan and any amendments made to that Plan prior to the issuance of building permits.

8. Building permits will not be issued for this project unless the local agency providing water and sewer services to the project provides written certification to the City that adequate water service and sewer facilities, respectively, are available to the project at the time of the application for the building permit, and that water and sewer capacity and facilities will continue to be available until the time of occupancy.

9. The applicant shall apply for and be issued building permits for this project within 24 months of approval or this coastal development permit will expire unless extended per Section 21.201.210 of the Zoning Ordinance.

10. The developer shall submit to the City a Notice of Restriction to be filed in the office of the County Recorder, subject to the satisfaction of the Planning Director, notifying all

interested parties and successors in interest that the City of Carlsbad has issued a Coastal Development Permit by Resolution No. 5218 on the property owned by the Developer. Said Notice of Restriction shall note the property description, location of the file containing complete project details and all conditions of approval as well as any conditions of restrictions specified for inclusion in the Notice Restriction. The Planning Director has the authority to execute and record an amendment to the notice which modifies or terminates said notice upon a showing of good cause by the Developer or successor in interest.

11. The Developer shall pay to the City an inclusionary housing in-lieu fee as an individual fee on a per market rate dwelling unit basis in the amount in effect at the time, as established by City Council Resolution from time to time.

12. **Prior to the issuance of building permits, the Developer shall prepare and record a Notice that this property may be subject to noise impacts from the proposed or existing Transportation Corridor, in a form meeting the approval of the Planning Director and City Attorney.**

13. Prior to the issuance of building permits, the applicant shall apply for and obtain a grading permit issued by the City Engineer.

14. Prior to any landscape work within the 10 foot beach access easement, the applicant shall apply for and obtain Planning Director approval of a Landscape and Irrigation Plan consistent with the City's Landscape Manual and receive a right-of-way permit from the City Engineer.

Engineering Conditions

General

15. Prior to hauling dirt or construction materials to or from any proposed construction site within this project, Developer shall apply for and obtain approval from, the City Engineer for the proposed haul route.

16. Prior to issuance of any building permit, Developer shall comply with the requirements of the City's anti-graffiti program for wall treatments if and when such a program is formally established by the City.

17. Prior to occupancy, Developer shall install rain gutters to convey roof drainage to an approved drainage course or street to the satisfaction of the City Engineer.

Grading

18. Based upon a review of the proposed grading and the grading quantities shown on the site plan, a grading permit for this project is required. Developer shall apply for and obtain a grading permit from the City Engineer prior to issuance of a building permit for the project.

PASSED, APPROVED AND ADOPTED at a regular meeting of the Planning Commission of the City of Carlsbad, California, held on the 17th day of July 2002, by the following vote, to wit:

AYES:	Chairperson Trigas, Commissioners Baker, Dominguez, Heineman, White, and Whitton
NOES:	None
ABSENT:	Commissioner Segall
ABSTAIN:	None

SEENA TRIGAS, Chairperson
CARLSBAD PLANNING COMMISSION

ATTEST:

MICHAEL J. HOLZMILLER
Planning Director

EXHIBIT I

GeoSoils Inc.

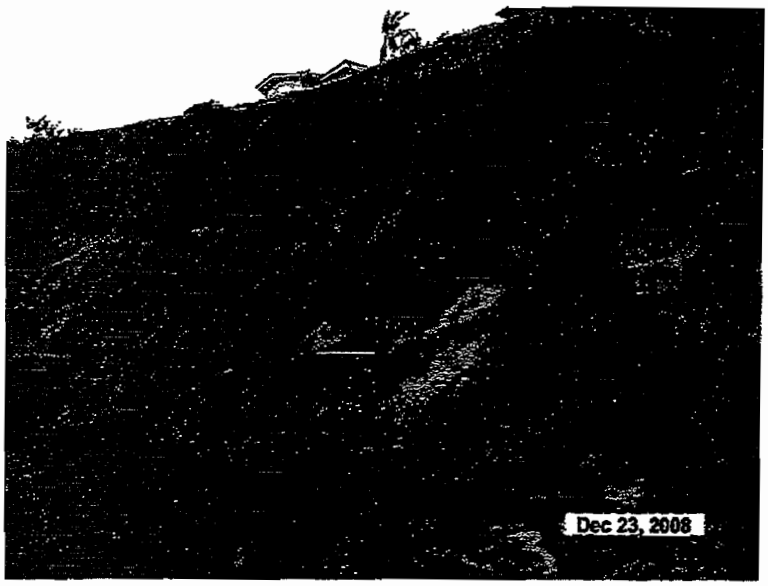
December 24, 2008

Mr. Dean Goetz & Marshall Sylver
5323 Carlsbad Blvd
Carlsbad, CA 92108

SUBJECT: Bluff Collapse Inspection 5323 & 5327 Carlsbad Blvd, Carlsbad

Dear Mr. Goetz & Sylver:

At your request, GeoSoils, Inc., (GSI) is pleased to provide this summary of our observations of the bluff collapse fronting the subject properties in Carlsbad, California. Our scope of work includes a site inspection, a brief review of recent site photographs available on the Coastal Records website and preparation of this letter. On or about December 19, 2008 a large portion of the bluff fronting 5323 & 5327 Carlsbad Boulevard failed onto the beach below. The failure was inspected by the undersigned on December 23, 2008. The photograph below shows the failure. The area of failure is approximately 50 feet along the shoreline by about 32 feet high. The bluff top receded as much as 5 feet. The total volume of the failure is approximately 150 cubic yards of bluff material.



5741 Palmer Way, Suite D, Carlsbad, CA 92010 *W.O. S5828* *760-438-3155*

GeoSoils Inc.

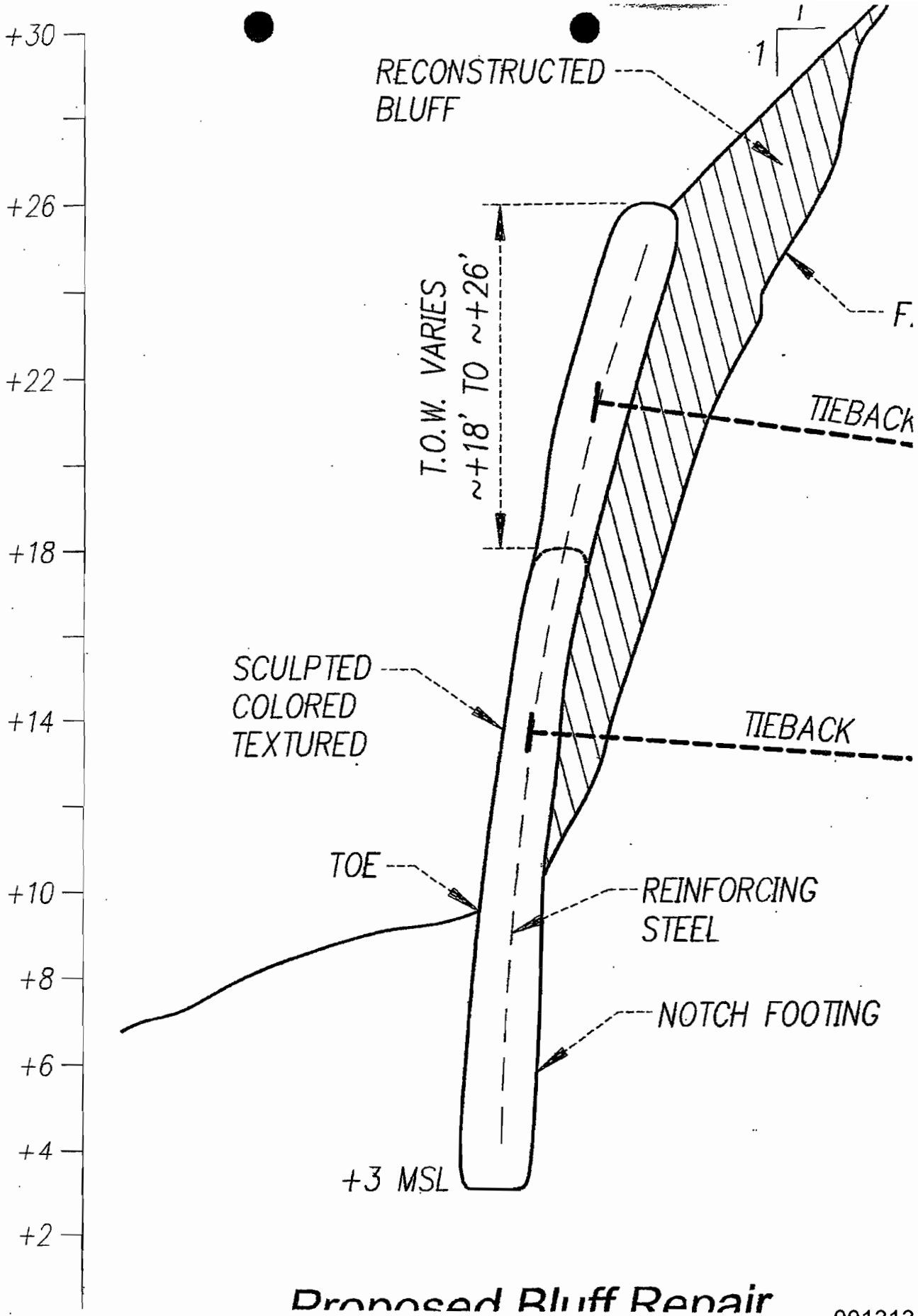
The bluff is currently failing and can be expected to continue to fail. The bluff condition in this small cove area pose a significant threat to any persons on the beach below the bluff. We strongly recommend that you inform the City of Carlsbad of these dangerous conditions and seek direction from the City as to permissible methods to mitigate this hazard to the public. Bluff stabilization and reconstruction will be necessary to protect the public from this hazard.

We appreciate this opportunity to be of service. Should you have any questions, please do not hesitate to contact the undersigned at (760) 438-3155.

Respectively submitted,

REGISTERED PROFESSIONAL ENGINEER
DAVID W. SKELLY
No. C 47857
Exp. 12/31/09
CIVIL
STATE OF CALIFORNIA

GeoSoils, Inc.
David W. Skelly
Civil Engineer, RCE #47857



Proposed Bluff Repair

EXHIBIT J

REC'D 05/21/09

FILE COPY

City of Carlsbad
Planning Department

May 19, 2009

Mr. Dean Goetz
5323 Carlsbad Boulevard
Carlsbad, CA. 92008

SUBJECT: CDP 09-07 – 5323 and 5327 CARLSBAD BOULEVARD BLUFF REPAIR – EMERGENCY PERMIT

Dear Mr. Goetz:

On April 16, 2009, the City of Carlsbad issued an Emergency Coastal Development Permit (CDP 09-07) for repairs to the coastal bluff located at 5323 and 5327 Carlsbad Blvd. The permit was approved subject to conditions that needed to be fulfilled to exercise the permit. Specifically the project was subject to completion of the emergency work and submittal of a formal Coastal Development permit and Special Use Permit within 30 days of the permit approval.

Because the conditions of the approval have not been met, The City of Carlsbad Planning Department hereby determines that CDP 09-07 is null and void.

At this time, no work can be done on the site without securing new permits or approvals.

Should you have any questions, please contact Van Lynch at (760) 602-4613 or Van.Lynch@carlsbadca.gov.

Sincerely,

DON NEU
Planning Director

c: Marshall Sylver, 5327 Carlsbad Boulevard, Carlsbad CA 92008
Chris DeCerbo, Principal Planner
Steve Bobbett, Project Engineer
File Copy
Data Entry

EXHIBIT K

GeoSoils Inc.

March 30, 2010

Mr. Dean Goetz
Mr. Marshall Sylver
5323 Carlsbad Blvd
Carlsbad, CA 92108

SUBJECT: Sand Mitigation Fee Calculation for Goetz Emergency Sea Wall, City of Carlsbad CDP 09-11

Dear Mr. Goetz and Mr. Sylver:

At your request, GeoSoils Inc (GSI) is pleased to provide the foregoing sand mitigation fee ("SMF") calculation for seawall installed in front of 5323 and 5327 Carlsbad Boulevard, Carlsbad, CA. This calculation uses the guideline formula that has been employed by the California Coastal Commission for recent seawall projects in Solana Beach and Encinitas, California. Unlike those projects, because your seawall is located entirely on private property and significantly landward of both the mean high tide line and the high water mark, there is some question about the applicability of this formula in this case. That being said, my assignment is merely to describe the formula, place a value on its many variables, and to undertake the math required to derive a dollar amount. We have done that here.

I. The SMF Formula

The California Coastal Commission employs the following formula as its guideline for determination of the SMF for seawall projects in Southern California:

Sand Mitigation Fee = Sand Cost x V_t

Sand Cost is the cost, per cubic yard of sand, of purchasing and transporting beach quality material to the project vicinity ($ per cubic yard). Derived from the average of three written estimates from sand supply companies within the project vicinity that would be capable of transporting beach quality material to the subject beach, and placing it on the beach or in the near shore area.

V_t is the sum of V_b, V_w, and V_e as further described below.

$$V_t = V_b + V_w + V_e$$

V_b is the amount of beach material that would have been supplied to the beach if natural erosion continued, or the long-term reduction in the supply of bluff material to the beach, over the life of the structure; based on the long-term average retreat rate, design life of the structure, percent of beach quality material in the bluff, and bluff geometry (cubic yards).

$$V_b = (S \times W \times L) \times [(R \times h_s) + (1/2h_u \times (R + (R_{cu} - R_{cs})))]/27.$$

V_w is the volume of sand to rebuild the area of beach lost due to long-term erosion (V_w) of the beach and near-shore, resulting in stabilization of the bluff face and prevention of landward migration of the beach profile; based on the long-term regional bluff retreat rate, and beach and near shore profiles (cubic yards).

$$V_w = A_w \times v$$

V_e is the volume of sand to rebuild the area of beach lost due to encroachment by the seawall; based on the seawall design and beach and near shore profiles (cubic yards).

$$V_e = A_e \times v$$

LEGEND

S	Fraction of beach quality material in the bluff material, based on analysis of bluff material to be provided by the applicant.
W	Width of the property to be armored (ft.).
L	The length of time the back beach or bluff will be fixed or the design life of the armoring without maintenance (yr.). For repair and maintenance projects, the design life should be an estimate of the additional length of time the proposed maintenance will allow the seawall to remain without further repair or replacement.
R	The retreat rate which must be based on historic erosion, erosion trends, aerial photographs, land surveys, or other acceptable techniques and documented by the applicant. The retreat rate should be the same as the predicted retreat rate used to estimate the need for shoreline armoring.
h_s	Height of seawall from base of bluff to the top, in feet.
h_u	Height of unprotected upper bluff, from the top of the Bluff Retention Device to the crest of the bluff, in feet.

R_{cu} Predicted rate of retreat of the crest of the bluff, during the period that the seawall would be in place, in feet per year, assuming no seawall were installed. This value can be assumed to be the same as R unless the applicant provides site-specific geotechnical information supporting a different value.

R_{cs} Predicted rate of retreat of the crest of the bluff, in feet per year, during the period that the seawall would be in place, assuming the seawall has been installed. This value will be assumed to be zero unless the applicant provides site-specific geotechnical information supporting a different value.

v Volume of material required, per unit width of beach, to replace or reestablish one foot of beach seaward of the Bluff Retention Device; based on the vertical distance from the top of the beach berm to the seaward limit of reversible sediment movement (cubic yards/ft. of width and feet of retreat). The value of v is often taken to be 1 cubic yard per square foot of beach. If a vertical distance of 40 feet is used for the range of reversible sediment movement, v would have a value of 1.5 cubic years/square foot (40 feet x 1 foot x 1 foot/27 cubic feet per cubic yard). If the vertical distance for a reversible sand movement is less than 40 feet, the value of v would be less than 1.5 cubic yards per square foot. The value of v will vary from one coastal region to another. A value of 0.9 cubic yards per square foot has been suggested for the Oceanside Littoral Cell (Oceanside Littoral Cell Preliminary Sediment Budget Report, December 1997, prepared as part of the Coast of California Storm and Tide Wave Study).

A_w The area of beach lost due to long-term erosion is equal to the long-term average annual erosion rate (R) times the number of years that the bluff will be fixed (L) times the width of the property that will be protected (W), in feet per year, and determined by the following formula: $A_w = R \times L \times W$

A_e The encroachment area of the Bluff Retention Device which is equal to the width of the Bluff Retention Device (W) times its seaward encroachment (E) and is determined by the following formula: $A_e = W \times E$.

E Encroachment by seawall, measured from the toe of the bluff or back beach to the seaward limit of the protection (ft.)

II. Application of SMF Formula to the Goetz Seawall Project

The values for each of the SMF formula variables are set forth below, along with an explanation.

Variable	Value	Discussion
S	65/100	The bluff is composed of two distinct soil materials. The lower few feet of the wall is Santiago Formation and the upper ~30 feet is Quaternary Terrace deposits. Samples of each of these materials were taken and analyzed for the percent sand. The Santiago Formation is primarily clay with about 30% sand. The Quaternary Terrace deposits are primarily sands with about 75% sand content. Using a weighted average across the bluff, the area of the relevant area of the bluff is approximately has a percent sand of 65%.
W	85 feet	The width of the property (bluff) armored is 85 feet, based upon survey data. The actual length of the seawall is longer because it is not straight but aesthetically curved to follow the natural bluff line.
L	75 years	From an engineering perspective, and based on the training and experience of the undersigned, the design life without maintenance for this seawall is estimated to be 25 years. With reasonable maintenance, however, the seawall can last up to 75 years.
R	0.16 ft/yr	The long-term bluff retreat rate is determined by looking at the changes in the bluff at the two subject properties, retreat rates on nearby properties, and the referenced 2006 United States Geological Survey (USGS) report. Retreat rates reported in site-specific geotechnical investigations (circa 1991) on nearby properties to the north range from 0.22 ft/yr to 0.4 ft/yr. These retreat rates were used to establish the bluff top setback requirement for new development at those properties. By contrast, however, the USGS report is a recent comprehensive report that covers the coast of California and specifically this location using survey data from as early as the 1890s through the present time. The USGS report shows that this particular section of bluff has not retreated over the last 115 years. This stability relative to other seacliffs in

		North San Diego County is likely due to the fact that this section is at the back of cove and significantly landward of both the mean high tide line and the high water mark. Figure 1 shows the USGS summary data plots and the location of the subject seawall. That being said, the bluff did fail in December 2008 prompting the emergency repairs. Using the USGS bluff retreat rate of 0 feet over the period from 1890 to 2006 along with the approximate 6 feet of retreat from the well documented failure in 2008, the actual retreat rate is calculated to be 6 ft over 120 years. This translates to an average annual retreat rate of 0.05 feet. That being said, the Coastal Commission has used 0.27 feet per year for other recent projects. Although dissimilar to these projects, to be conservative, we have chosen the midway point between the actual rate (0.05) and the Commission's number (0.27) to come up with 0.16 as the annual retreat rate for this site.
h_s	22 feet	The average height is based on direct measurements.
h_u	11.5 feet	This is taken from a direct measurement.
R_{cu}	0.16 ft/yr	This value is always assumed to be the same as R unless the applicant can demonstrate otherwise.
R_{cs}	0	This value is always assumed to be zero unless the applicant can demonstrate otherwise.
v	0	This variable is not applicable in this case because the seawall is entirely on private property, and will likely remain so for the duration of its 75-year design life.
A_e	0	This variable is not applicable in this case because the seawall is entirely on private property. That is to say there is no encroachment.
E	0	This variable is not applicable in this case because the seawall is entirely on private property. That is to say there is no encroachment.
Sand Cost	$\$3/yd^3$	Sand replenishment projects occur only a large-scale (i.e., in excess of 100,000 cubic yards), regional basis where sand is obtained through offshore dredging methods and deposited

		on the beach. To be effective, sand replenishment projects must be done a large scale as small deposits are more easily subject to erosion from wave action, wind, and rain. SANDAG's 2001 sand replenishment project utilized this methodology and put a massive volume of sand on the beach for approximately $2.10 per cubic yard. If performed today, this cost would likely be $3.00 per cubic yard.

III. Calculation

$V_b = (S \times W \times L) \times [(R \times h_s) + (1/2\ h_u \times (R + (R_{cu} - R_{cs})))]/27 = 823\ yds^3$

$V_e = A_e \times v = W \times E \times v = 0\ yds^3$

$V_w = A_w \times v = R \times L \times W \times v = 0\ yds^3$

$V_t = V_b + V_e + V_w = 823\ yds^3$

Sand Mitigation Fee = V_t x Sand Cost, or 823 x $3 = $2,469.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact our office.

Respectfully submitted,

GeoSoils, Inc.
David W. Skelly MS, PE
RCE#47857

Reference

USGS 2006 "National Assessment of Shoreline Change Part 3: Historical Shoreline Change and Associated Coastal Land Loss Along Sandy Shorelines of the California Coast", Open File Report 2006-1219

EXHIBIT L

RECEIVED
JUL 18 2012
CALIFORNIA
COASTAL COMMISSION
SAN DIEGO COAST DISTRICT

GEOTECHNICAL EVALUATION OF COASTAL BLUFF STABILITY
5323 AND 5327 CARLSBAD BOULEVARD
CARLSBAD, SAN DIEGO, CALIFORNIA

FOR

AXELSON-CORN LAW FIRM
1220 NORTH COAST HIGHWAY 101
ENCINITAS, CALIFORNIA 92024

W.O. 6364-A-SC JULY 12, 2012

GeoSoils Inc.

Geotechnical • Geologic • Coastal • Environmental

5741 Palmer Way • Carlsbad, California 92010 • (760) 438-3155 • FAX (760) 931-0915 • www.geosoilsinc.com

July 12, 2012

W.O. 6364-A-SC

Axelson-Corn Law Firm
1220 North Coast Highway 101
Encinitas, California 92024

Attention: Mr. Jon Corn

Subject: Geotechnical Evaluation of Coastal Bluff Stability, 5323 and 5327 Carlsbad Boulevard, Carlsbad, San Diego, California

Dear Mr. Corn:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) has performed a geologic and geotechnical evaluation of the stability of the coastal bluff at the subject site. The accompanying report presents the findings of our study and provides recommendations regarding the recently constructed seawall that fronts the subject properties.

Based on our discussions with you, it is our understanding that the California Coastal Commission (CCC) has raised concerns regarding the necessity for the seawall and requests justification for the wall to remain. This report will explain why it is our opinion that the subject seawall, that was legally constructed through the City of Carlsbad emergency permitting process, is necessary to protect the subject properties, the life and safety of the beach-going public, and existing public beach access. In short, the seawall is necessary to protect against the danger of incipient bluff failure that would damage both public and private property and protect the public welfare and safety.

BACKGROUND

As described in our 2009 application to the City of Carlsbad for an Emergency Coastal Development Permit (GSI, 2009e [see Appendix A]), large portions of the coastal bluff in the rear-yards of the subject lots failed in response to late December 2008 storms. These large-scale failures prompted the homeowners of the subject properties to apply for an emergency permit to construct a seawall to protect their homes as well as the public who use this very popular beach area, some of which is located on their properties. As allowed by the City of Carlsbad's Municipal Code, an emergency permit was granted in 2009 to construct a seawall for the purpose of protecting the public from geologic hazards (serving coastal dependent uses and protecting the beach from erosion), as well as the adjacent public beach access stairway, and the subject properties. In June 2009, construction

commenced, with GSI providing as-needed geotechnical special inspection and testing services. Following completion of the seawall construction in September 2009, the property owners applied for a regular grading permit. Based on compliance with the Carlsbad Municipal Code, the seawall project received final approval from the City of Carlsbad City Council in May 2010.

Currently, opponents represented by the Coast Law Group have filed an appeal with the CCC to have the seawall removed. Their argument is that the seawall was constructed with out proper California Environmental Quality Act (CEQA) studies.

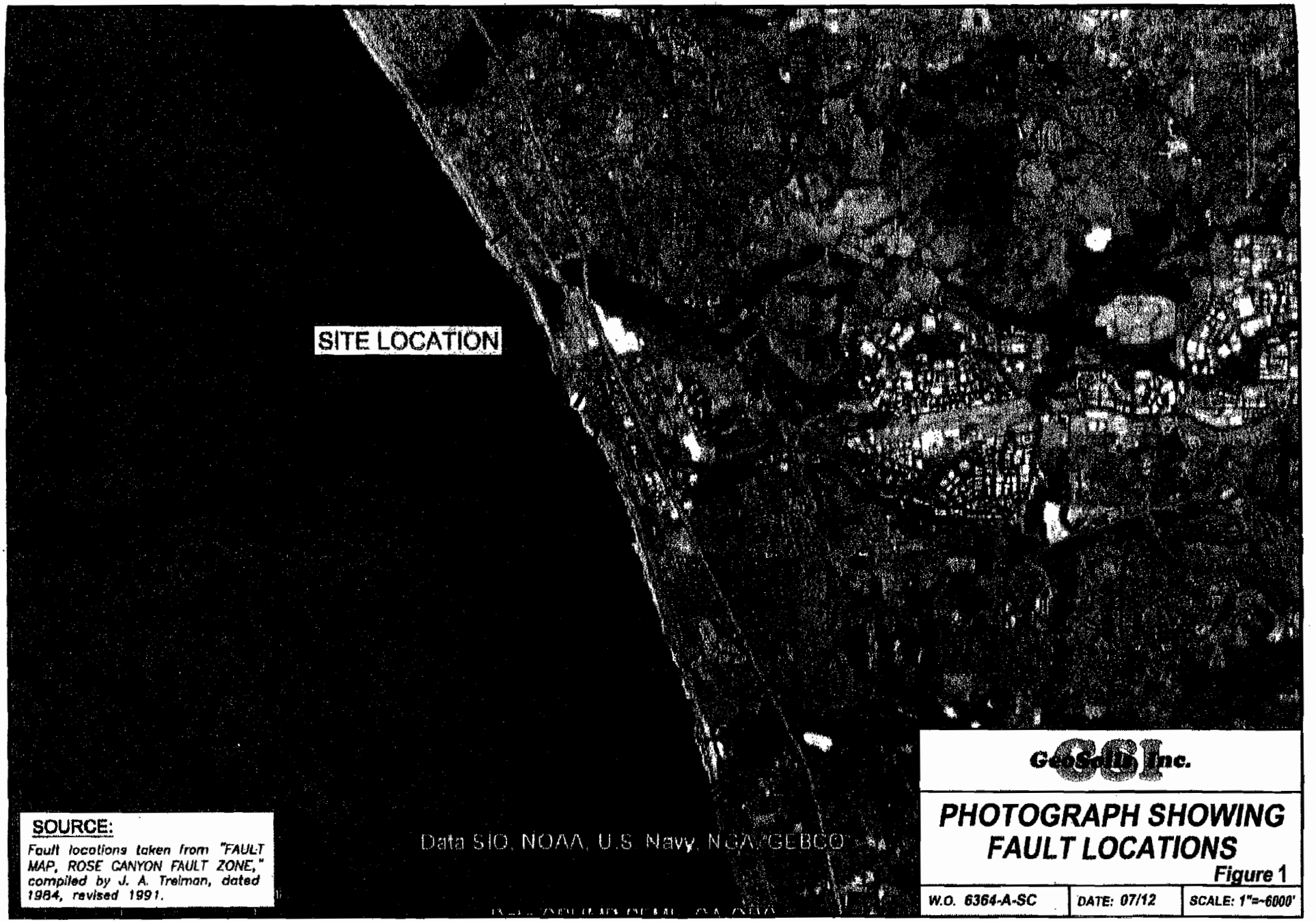
GEOLOGIC CONDITIONS OF THE COASTAL BLUFF

The subject coastal bluff is located within a shallow cove at the southerly end of Terramar Beach and the northerly end of South Carlsbad State Beach in Carlsbad, California. The subject bluff is on the order of [redacted] to the Eocene-age Santiago Formation. The site geologic conditions are presented on Plates 1 through 3. Our understanding of the site geology is based on logging bluff exposures, and our review of published State maps, reports of regional geologic studies, and the data reported by Converse Consultants, Inc. ([CCI] 1984b) and ICG, Inc. ([ICG] 1991) who also investigated the site and near vicinity. Logs of selected borings from ICG (1991) are presented in Appendix B.

Unique to this site, as compared to other areas along this reach of the coastline, is the formation [redacted] Within this cove, [redacted] From our review of site geologic conditions, available published and unpublished documents, including state geologic maps, [redacted]

Directly offshore from the site is a complex of fault splays associated with both the Rose Canyon and Newport Inglewood fault zones (State of California Department of Conservation-Division of Mines and Geology, 1987). According to Treiman (1991), several short northeasterly trending faults, associated with these fault zones, extend onshore along the Carlsbad coast. One of these faults crosses the northerly end of the subject site (Figures 1 through 3). This fault is now obscured by talus and fill.

Ryan et al. (2009) point out that the transition between the Rose Canyon fault zone (RCFZ) and the Newport-Inglewood fault zone (NIFZ) is located directly offshore of the subject site. Due to the more northerly trend of the RCFZ (N30°W) as compared to the trend of the NIFZ (N45°W), a left bend is required between the faults. This bend is accommodated by



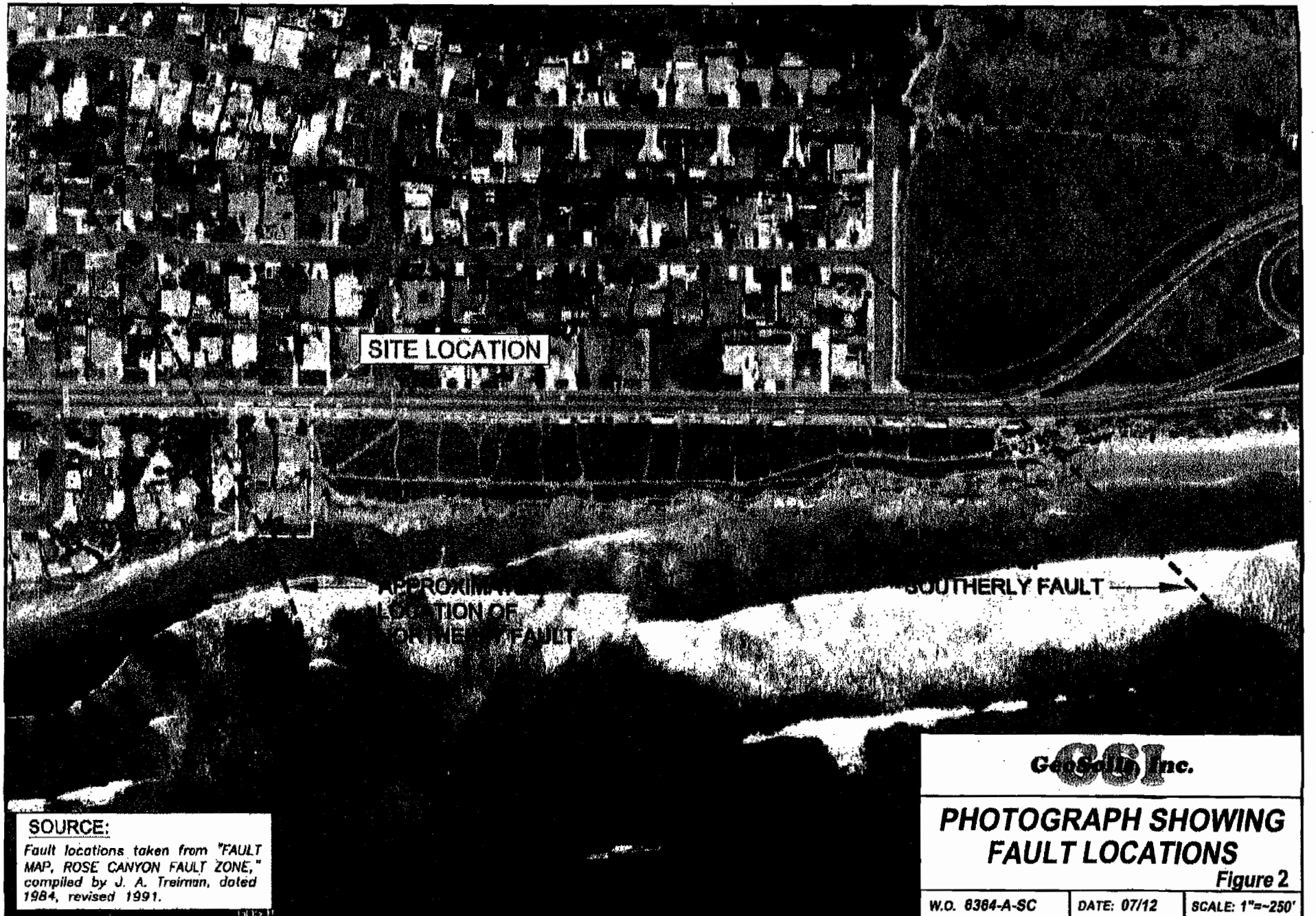
SOURCE:
Fault locations taken from "FAULT MAP, ROSE CANYON FAULT ZONE," compiled by J. A. Treiman, dated 1984, revised 1991.

GeoSoils Inc.

PHOTOGRAPH SHOWING FAULT LOCATIONS

Figure 1

W.O. 6364-A-SC	DATE: 07/12	SCALE: 1"=~6000'



SOURCE:
Fault locations taken from "FAULT MAP, ROSE CANYON FAULT ZONE," compiled by J. A. Treiman, dated 1984, revised 1991.

GeoSoils Inc.

PHOTOGRAPH SHOWING FAULT LOCATIONS

Figure 2

W.O. 6364-A-SC	DATE: 07/12	SCALE: 1"=~250'

SOURCE:
Fault location taken from "FAULT MAP, ROSE CANYON FAULT ZONE," compiled by J. A. Treiman, dated 1984, revised 1991.

GeoSolutions, Inc.

PHOTOGRAPH SHOWING FAULT LOCATIONS

Figure 3

W.O. 6364-A-SC	DATE: 07/12	SCALE: 1"=~50'

transtensional faulting and fracturing, and is evidenced by the abrupt change in strike of the coastline that occurs at the subject site.

The short northeasterly fault that trends onshore at the site is a subsidiary fault to the offshore principal displacement zone. Our review of fault kinematics in strike-slip regimes suggests that this fault is an extensional fault, given the angle between this fault and the principal traces of the Rose Canyon and Newport - Inglewood fault zones (Sylvester, 1988; Sylvester and Smith, 1976).

In summary, the data show that complex shear deformation has resulted in the extensive fracturing of the Santiago Formation and terrace deposits, and the related loss of cementation by subaerial meteroic infiltration and chemical dissolution. These factors have resulted in a less indurated, less resistant Santiago bedrock in the cove area than that exposed in the seacliffs to the north and south. In addition, down-faulting has resulted in a lower elevation of the geologic contact between the terrace deposits and Santiago Formation in the cove than in the bluff exposures to the north and south. The lower elevation of this geologic contact in the cove area places friable terrace deposit sands, which are inherently less cemented that bedrock, in the direct path of storm wave attack, which results in rapid undercutting of the bluff. Further, the presence of uncemented dune sands within the terrace deposits, and faulting and fracturing within the Santiago Formation and terrace deposits, makes these rocks more susceptible to erosion by wave-induced percussion hammering and spring sapping. To that end, regional geologic structure is the primary contributor in forming this recessed portion of the Carlsbad coastline.

GROUNDWATER CONDITIONS

Our review of the referenced studies indicates that in 1984, test borings advanced by Converse Consultants, Inc. (1984a and 1984b) encountered groundwater at approximately 1½ and 3 feet above mean sea level (MSL). In February 1991, ICG, Inc. (1991) encountered perched groundwater at elevations of approximately 3 to 6 feet above mean sea level. During installation of the lower tiebacks at an approximate elevation of 10 feet MSL, groundwater was encountered that resulted in caving conditions. The high groundwater conditions, combined with the low cohesive nature of the terrace deposits near the base of the bluff, required the construction of an additional row of tiebacks to provide the necessary restraint. The observed increase in groundwater elevation is likely due to infiltrated irrigation water form increased urbanization since 1984.

The more deeply incised, "trough-like" Santiago Formation and terrace deposits in the cove area are more favorable for the accumulation and transport of groundwater from upland areas to and through the site vicinity. It is our opinion that this elevated groundwater condition will continue, or may even increase as urbanization of the upland areas continues or rainfall increases.

The elevated groundwater also tends to decrease slope stability by creating hydrostatic forces that contribute to the instability of the bluff and, more importantly, make the weaker portions of the sea cliff behind the cove more susceptible to marine erosion.

HISTORICAL CONDITIONS AND COASTAL EROSION

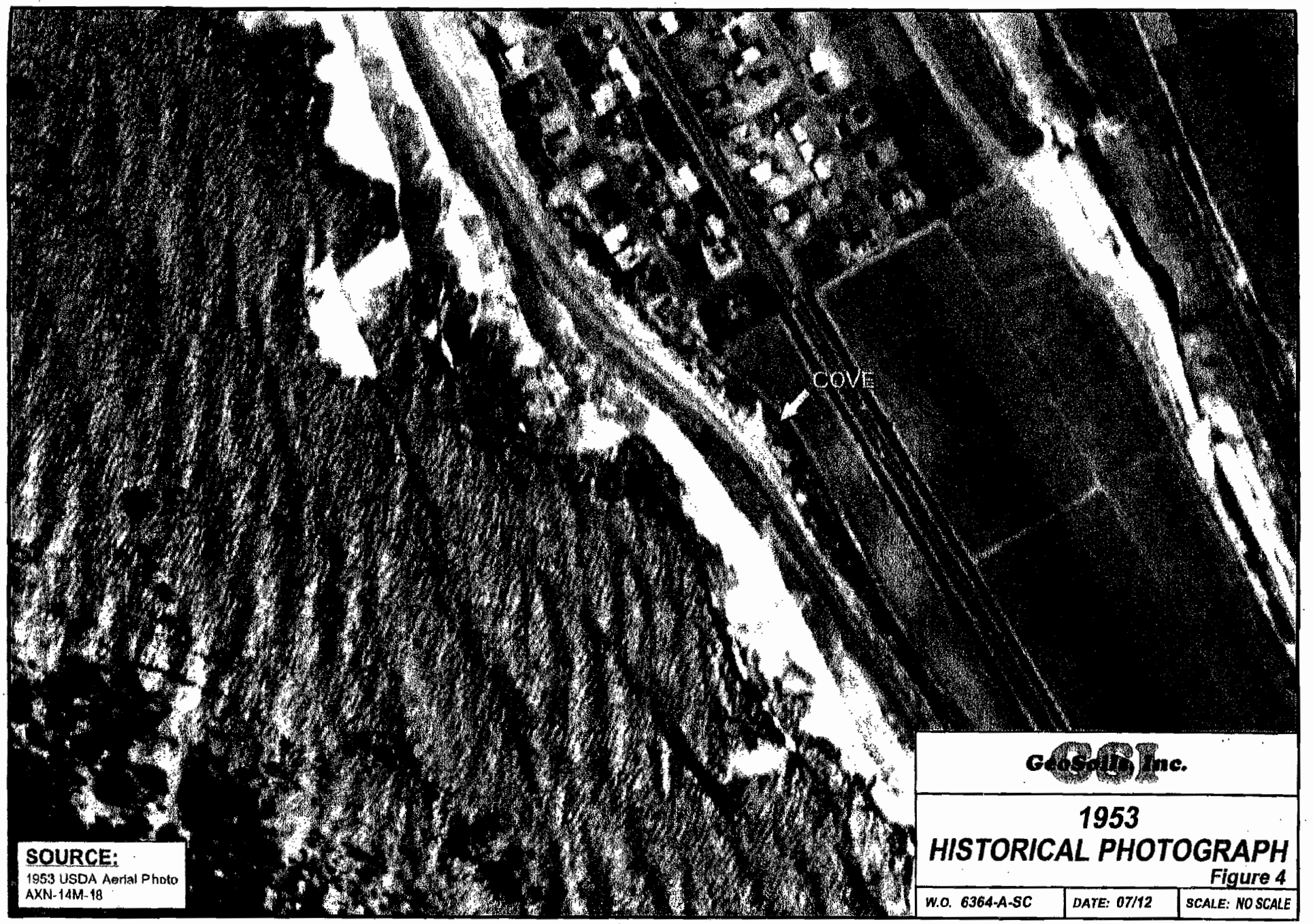
Historically, in 1800s and early 1900s, the beaches along this reach of the coastline were as much as 300-feet wide (Kuhn and Shepard, 1984). However, in the early 1900s, the construction of Lake Henshaw Reservoir and other flood control projects on the local creeks and drainages reduced the flood peaks and hence sediment loads within the San Luis Rey River, thereby diminishing the natural supply of beach material to the Oceanside Littoral Cell. By 1925, beaches narrowed and the beach width could be correlated with the reduced stream runoff from the San Luis Rey River.

During World War II, a small boat harbor facility was constructed at the newly created Camp Pendleton, immediately adjacent to the northern limits of Oceanside. Even though a jetty was constructed to protect the Harbor entrance, the sediment, which would have normally nourished the beaches to the south, began to deposit in the harbor, requiring continual dredging and pumping of the spoils to the south. Despite the problems that the Marine Corps small boat basin was experiencing and evidence that the jetties were causing problems along the beaches to the south, a small boat harbor was constructed to serve the Oceanside area south of the harbor at Camp Pendleton. During construction of the Oceanside Harbor (and Camp Pendleton Harbor), dredge spoils were deposited on the downcoast beaches. It is reported that this dredge spoil material contained many cobbles derived from the floods of the San Luis Rey River. Despite the large amount of material placed on the beaches, erosion began almost immediately, with much of the sand being removed by storm events, leaving behind the cobble beaches and berms that are seen along the North County beaches. Kuhn and Shepard report that these cobbles may act as projectiles and actually increase erosion and damage during more severe storm events.

Historical Aerial Photo Review

As part of our work, we reviewed historical aerial photographs from the USDA 1953 collection, as well as photographs found on the California Coastal Records Project web site (www.californiacoastline.org).

A review of the 1953 USDA and 1954 Navy oblique aerial photographs (Figures 4 and 5) indicates that construction of homes along the bluffs immediately north of the site was well underway by 1953. Additionally, construction of the Encina Power Plant to the north of the site was also underway. The 1953 and 1954 aerial photographs also indicate that bluff erosion was rather substantial, especially within the area of the cove.



SOURCE:
1953 USDA Aerial Photo
AXN-14M-18

GeoSolis Inc.

1953
HISTORICAL PHOTOGRAPH
Figure 4

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

SOURCE:
1954 U. S. NAVY Oblique
Photo No. 0161

GeoSoils Inc.

1954
HISTORICAL PHOTOGRAPH
Figure 5

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

As part of our study, we also reviewed selected photographs from the California Coastal Records Project web site. These oblique aerial photographs provide a very good record of changes along the coastline beginning in 1972. A review of the 1972 oblique aerial photograph of the area (Figure 6) indicates that the subject cove and lower half of the coastal bluff was protected with gunite (Converse 1984b). This pre-Coastal Act bluff protection indicates the possibility of a vested right to continued protection, now in the form of a modern seawall. The 1972 photograph also indicates that a relatively wide protective sand beach also existed along this reach of the coastline and that the bluff profile suggested that bluff erosion was minimal.

nothing protecting

A review of the 1979 photograph (Figure 7) indicates that portions of the protective gunite wall were damaged or lost, and that most of the protective sand beach was gone, leaving behind a cobble beach. Additionally, a substantial amount of erosion was occurring along the bluffs, primarily within the cove area and in the bluffs to the south, as evidenced by the loss of the protective vegetation on the bluff face. Additionally, the cove area had begun to expand landward, primarily by marine erosion. This erosion is discussed later in this report.

A review of the 1989 aerial photograph (Figure 8) indicates that a substantial amount of marine erosion was occurring along the bluff, primarily because of the loss of the protective beach sand and gunite slope protection. The 1989 photograph also reveals the development of small sea caves and undercutting within the bedrock. Remnants of the gunite can also be seen in this photograph at the base of the bluff in the cove. A comparison between the 1972 and 1989 photographs suggests that approximately 10+ feet of bluff erosion had occurred within the cove area between 1972 and 1989. Bluff profiles also indicate that the bluff retreat was substantially caused by marine erosion. Kuhn (1984) reports that up to 30 feet of bluff was lost during this period.

is that gunite or is it cobble stone?

The 1993 oblique aerial photograph (Figure 9) shows the cove with a cobble and sand beach. Notable in the photograph is the clear lack of the typical more erosion-resistant Santiago Formation bedrock. Also notable, the notching by marine erosion within the Santiago Formation on either side of the cove.

The 2002 oblique aerial photograph (Figure 10) shows construction of the two subject residences had begun, and that the public access stairway had been constructed. The beach deposits shown in the photograph primarily consist of sand.

A review of the 2004 aerial photograph (Figure 11) shows the elevation of the terrace - bedrock contact. This photograph also shows two small bluff failures that occurred on either side of the cove. A trail system has been constructed on the bluff on the property to the north. Approximately 5 to 10 feet of bluff erosion has occurred since 2002. Beach deposits in the cove area primarily consist of sand and cobble.

A review of the 2006 aerial photograph (Figure 12) indicates that approximately 5 feet of beach sand was lost during the period between 2004 and 2006. Marine erosion was again

SOURCE:
California Coastal Records Project, Photo dated 1972, Image No. 7240103.

GeoSoils Inc.

1972
HISTORICAL PHOTOGRAPH
Figure 6

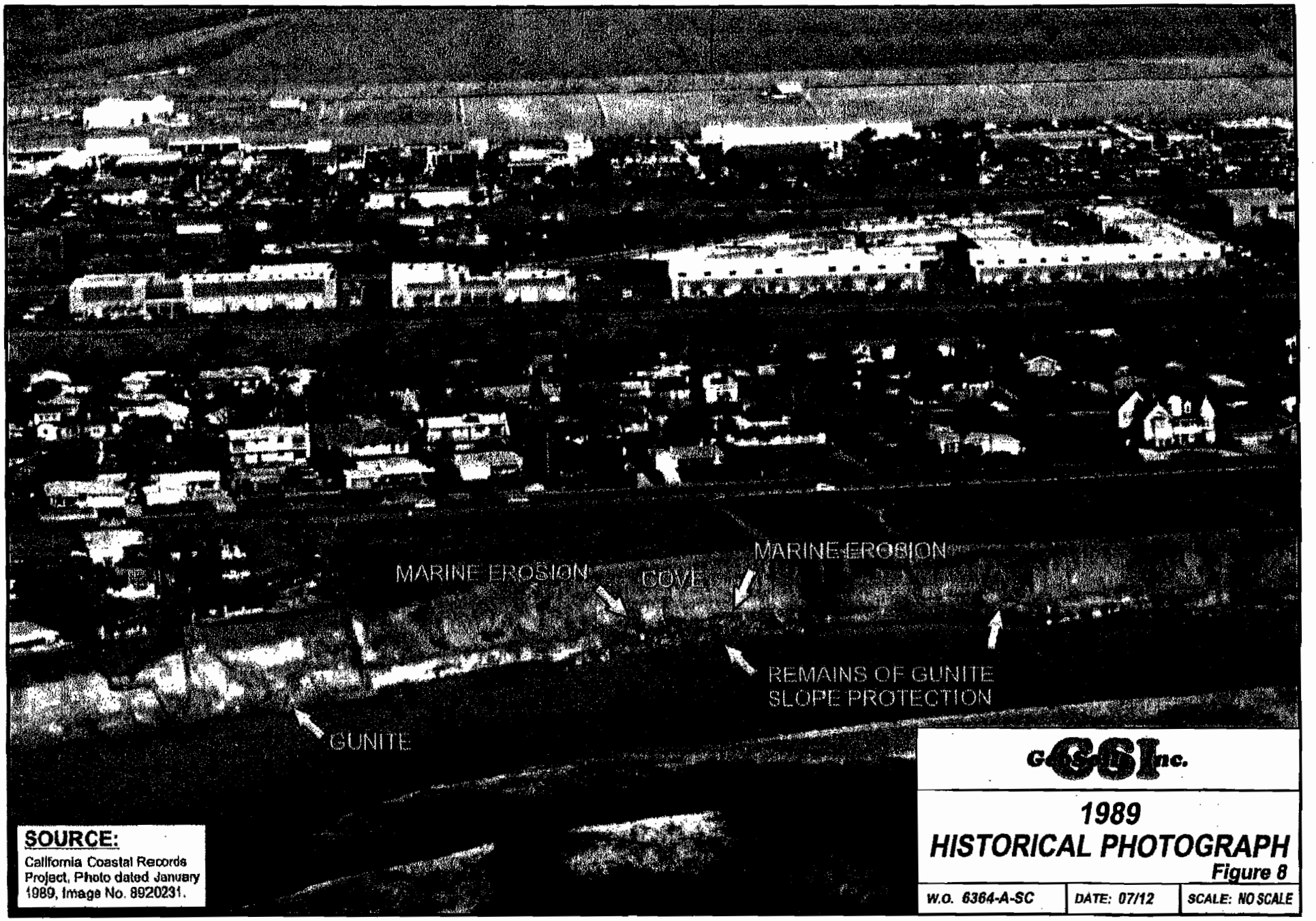
W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

SOURCE:
California Coastal Records Project, Photo dated May 3, 1979, Image No. 7954108

GeoSoil, Inc.

1979
HISTORICAL PHOTOGRAPH
Figure 7

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

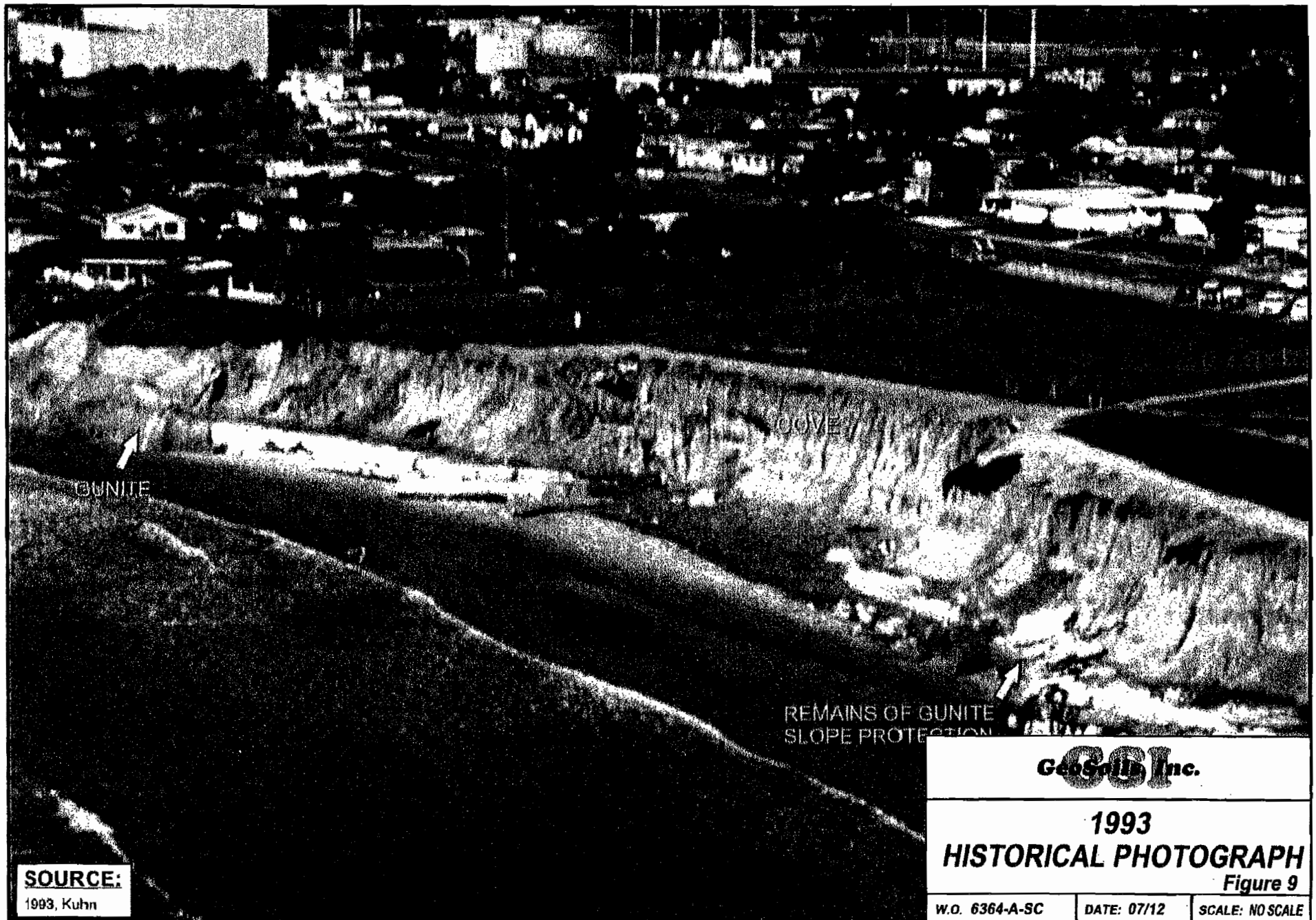


SOURCE:
California Coastal Records Project, Photo dated January 1989, Image No. 8920231.

GSI Inc.

1989
HISTORICAL PHOTOGRAPH
Figure 8

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

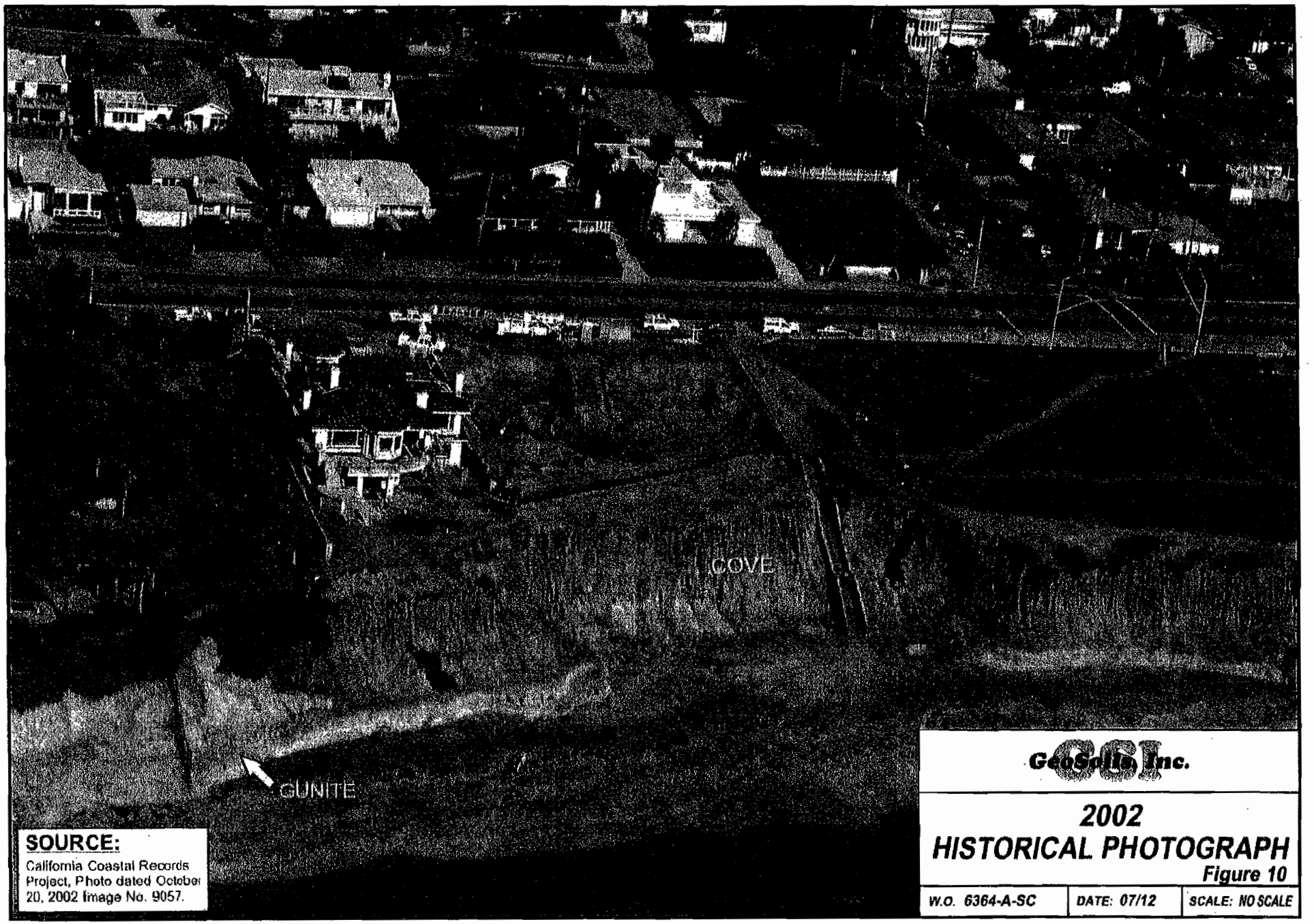


SOURCE:
1993, Kuhn

GeoSoils Inc.

1993
HISTORICAL PHOTOGRAPH
Figure 9

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

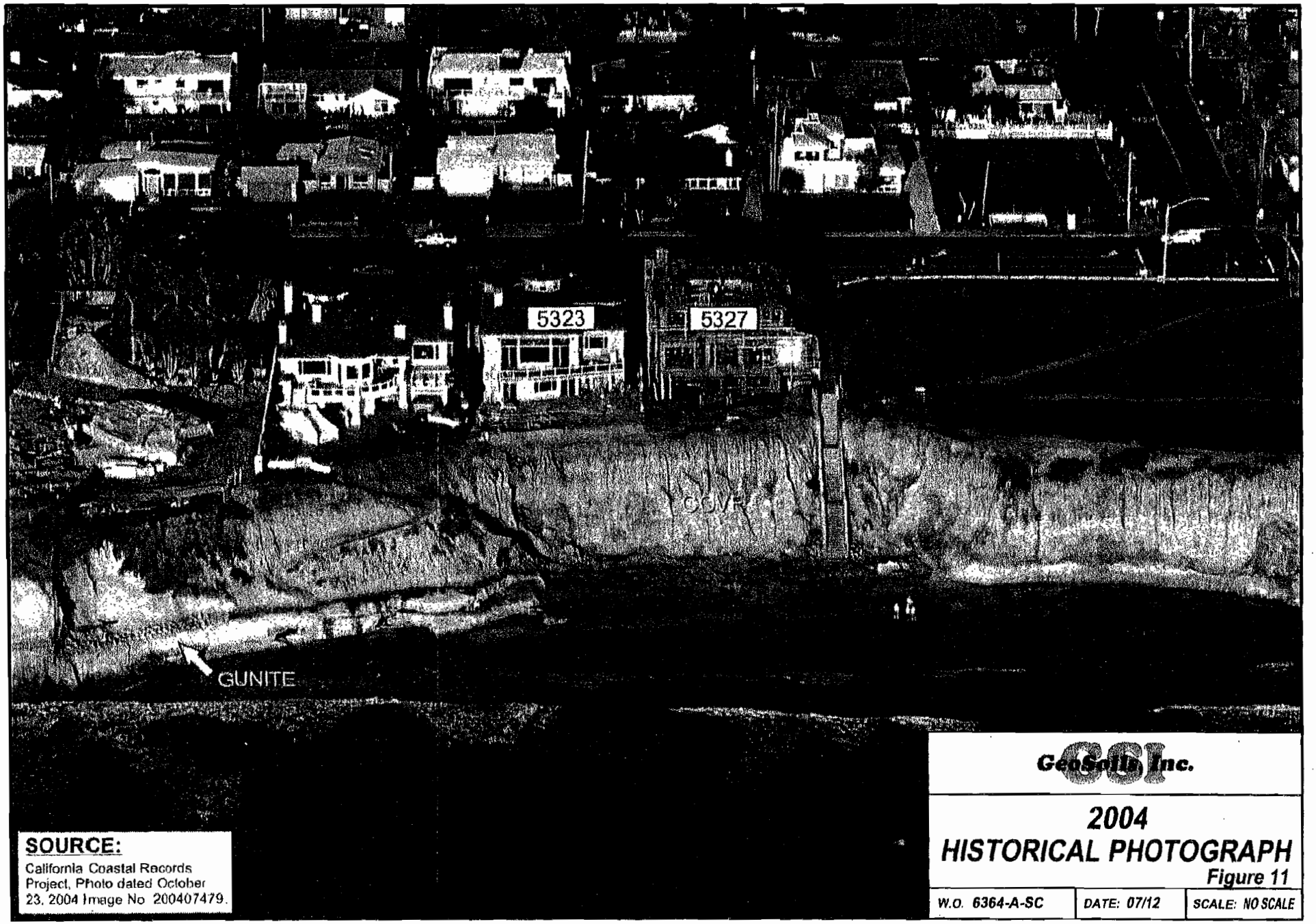


SOURCE:
California Coastal Records Project, Photo dated October 20, 2002 Image No. 9057.

GeoSoils, Inc.

2002
HISTORICAL PHOTOGRAPH
Figure 10

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

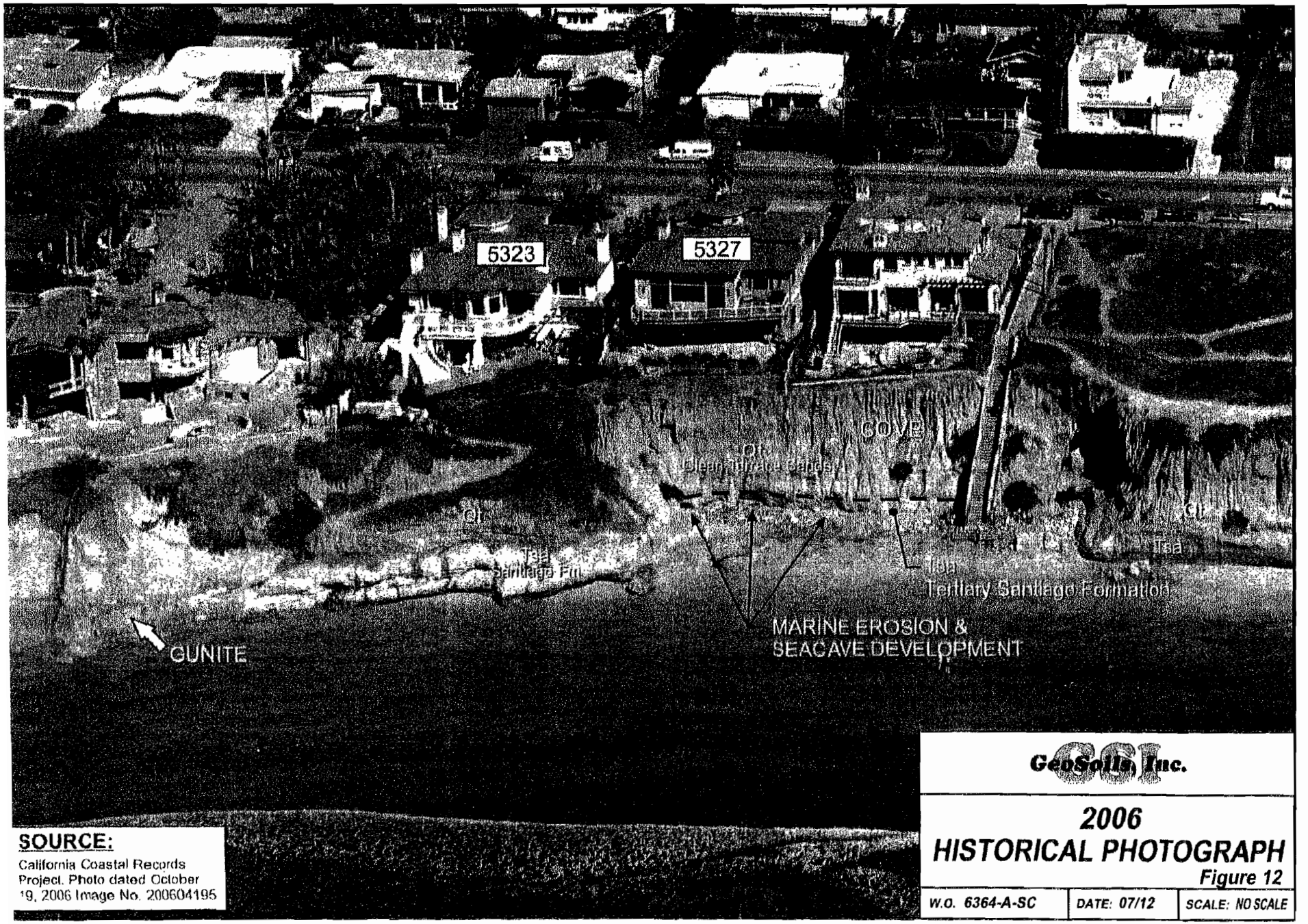


SOURCE:
California Coastal Records Project, Photo dated October 23, 2004 Image No 200407479.

GeoSoils Inc.

2004
HISTORICAL PHOTOGRAPH
Figure 11

W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE



SOURCE:
California Coastal Records Project. Photo dated October 19, 2006 Image No. 200604195

GeoSolis, Inc.

2006 HISTORICAL PHOTOGRAPH

Figure 12

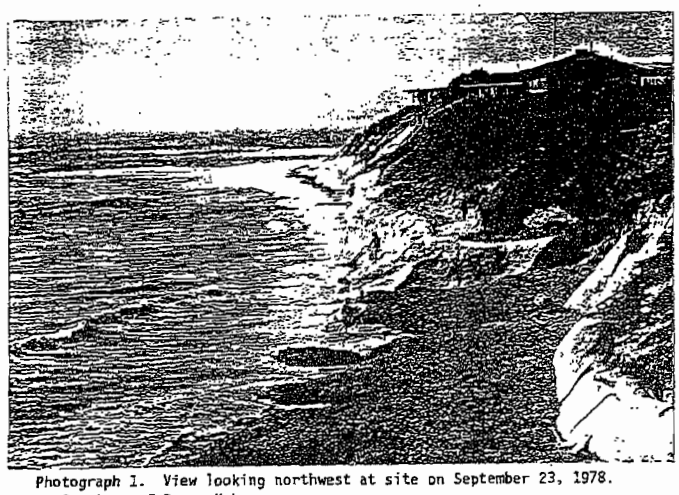
W.O. 6364-A-SC	DATE: 07/12	SCALE: NO SCALE

the beach sand by cobbles (see Photograph 1). In early 1978, sections of the sea cliff immediately south of the site experienced rockfalls, topples, and block-glide failures. At the site, the gunite covering portions of the cliff face was destroyed by beach cobble ("artillery") attack during storm periods. A second application of gunite was destroyed when surface water runoff gullied the bluff face behind the gunite covering.

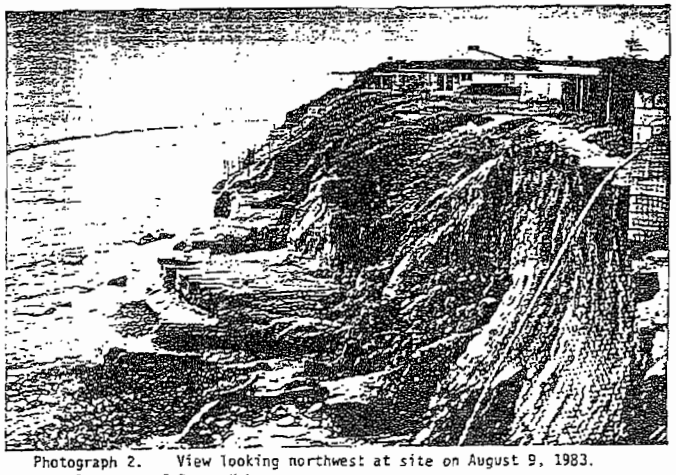
artificially introduced cobble

Between 7 and 9 August 1983, storm waves generated from the southern hemisphere combined with the perigean tide of August 1983 caused extremely rapid erosion at the site. Waves to the south of Palomar Airport Road and from Tamarack Avenue to the north (see Figure 1) were reported between 3 and 4 feet in height. Directly offshore from the site, wave heights ranged from 8 to 10 feet because of wave convergence along the axis of Carlsbad Submarine Canyon (see Figure 2, below). After the gunite was broken up and scattered by these waves on 7 August 1983, the upper portion of the sea cliff was eroded approximately 23-27 (horizontal) feet during the following 2-day period (see Photograph 2).

3



Photograph 1. View looking northwest at site on September 23, 1978.
Courtesy of Gerry Kuhn.



Photograph 2. View looking northwest at site on August 9, 1983.
Courtesy of Gerry Kuhn.

The above excerpt clearly illustrates the effect that the existing geologic and geomorphic conditions have contributed to the erosion along this segment of the coastline.

The site geologic conditions, combined with the offshore marine geomorphic conditions, also contribute to the erosion in this area. Besides the loss of sand from upstream drainages due to the construction of the harbors and jetties, a significant portion of the sand that travels downcoast is also periodically lost to the Carlsbad Submarine Canyon, never reaching the beaches of South Carlsbad, Encinitas, and Solana Beach.

STABILITY OF THE COASTAL BLUFF WITHOUT THE SEAWALL

As part of our engineering evaluation, GSI has also performed slope stability analyses of the coastal bluff without the seawall as required by the CCC. These analyses were performed along Geologic Cross Sections C-C' and D-D' (see Plates 1 and 2). These sections were selected because they represent the closest point of the residences to the top of the bluff and are therefore considered the most critical slope. For our analyses, GSI used the two-dimensional slope stability computer program "GSTABL7 v.2" (Gregory, 2003).

GSI modeled the bluff slope without the seawall by drawing cross sections through the bluff shown on the topographic survey by Melchoir Land Surveying ([MLS] 2009). This survey was performed shortly following seawall foundation construction and prior to the construction of the wall superstructure, and the geogrid reinforced upper bluff area. The MLS (2009) survey represents a reasonably accurate configuration of the post-failure bluff slope. However, it should be noted that as with all bluff restoration projects, bluff failure can continue during the course of construction. As such, GSI used the MLS (2009) survey data and then re-created the bluff slope about 2½ feet landward to more accurately model the post-construction bluff face (see Plate 2). To account for bluff disturbance in the event that the seawall and upper bluff stabilization are removed, GSI reasonably re-created the bluff face an additional 2½ feet landward (see Plate 2). The configuration of the bluff face (post-seawall removal) was then analyzed for slope stability.

The Simplified Janbu (non-circular block) Method was used to calculate the factor-of-safety (FOS) for specified surfaces or searches for the block, or irregular slip surfaces along the cross sections having the minimum FOS. The calculations incorporated soil shear strengths, ascertained through direct shear testing of relatively undisturbed samples of onsite and near-site materials performed by GSI.

Based on our slope stability analyses, the slope without the seawall possesses static factors of safety of 1.03 to 1.19 and a seismic factor of safety of less than 0.73 (see Figures 16 through 19). In addition, our analyses indicate that portions of both residences would have a static factor-of-safety less than 1.2 if the seawall were to be removed. Slopes possessing a static factor-of-safety less than 1.2 are approaching their stability equilibrium

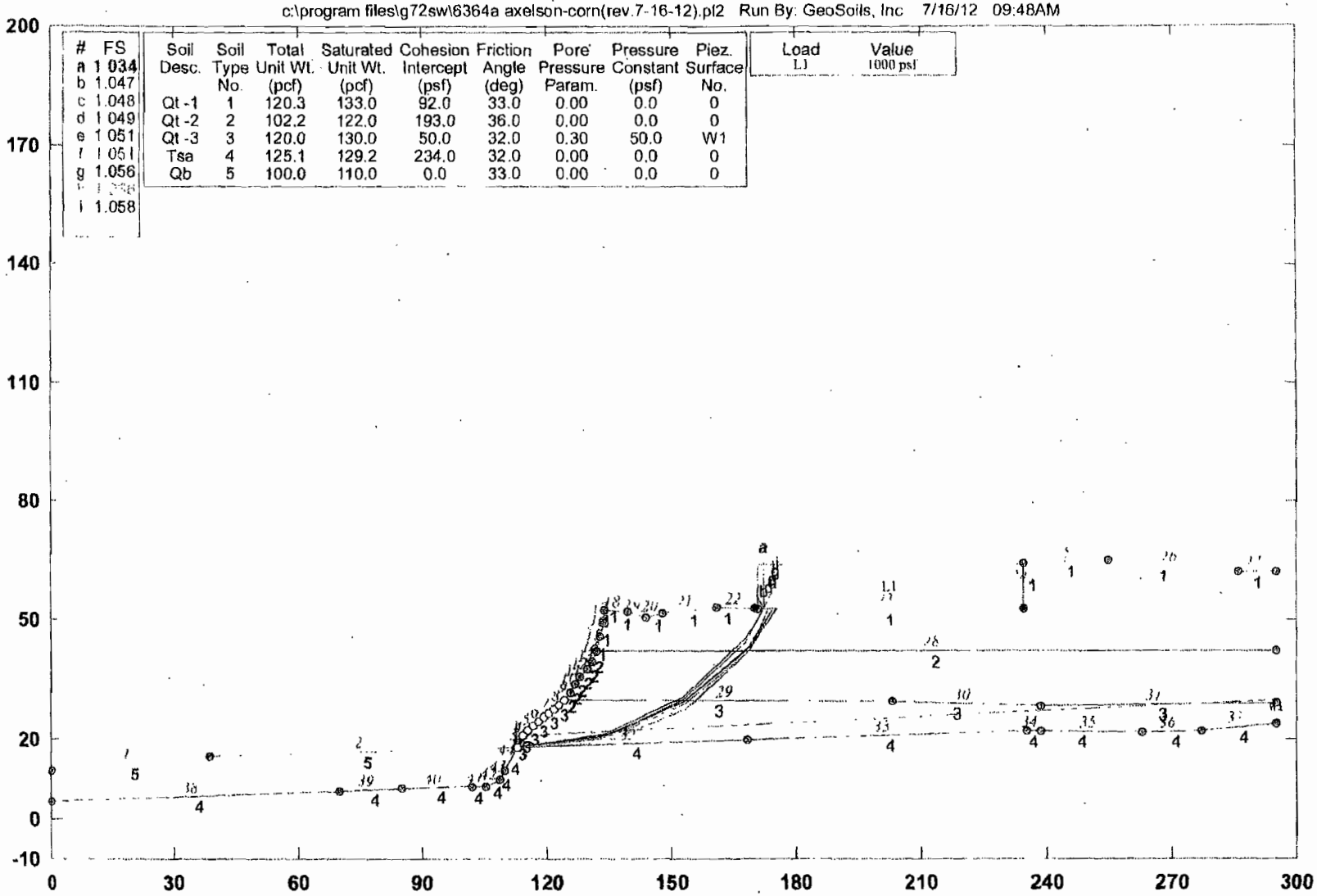
6364-A-SC - 5323 Carlsbad Blvd. Section C-C' - Static

c:\program files\g72sw\6364a axelson-corn(rev.7-16-12).pl2 Run By: GeoSoils, Inc 7/16/12 09:48AM

#	FS
a	1.034
b	1.047
c	1.048
d	1.049
e	1.051
f	1.051
g	1.056
h	1.058
i	1.058

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qt -1	1	120.3	133.0	92.0	33.0	0.00	0.0	0
Qt -2	2	102.2	122.0	193.0	36.0	0.00	0.0	0
Qt -3	3	120.0	130.0	50.0	32.0	0.30	50.0	W1
Tsa	4	125.1	129.2	234.0	32.0	0.00	0.0	0
Qb	5	100.0	110.0	0.0	33.0	0.00	0.0	0

Load	Value
L1	1000 psf



GSTABL7 v.2 FSmin=1.034

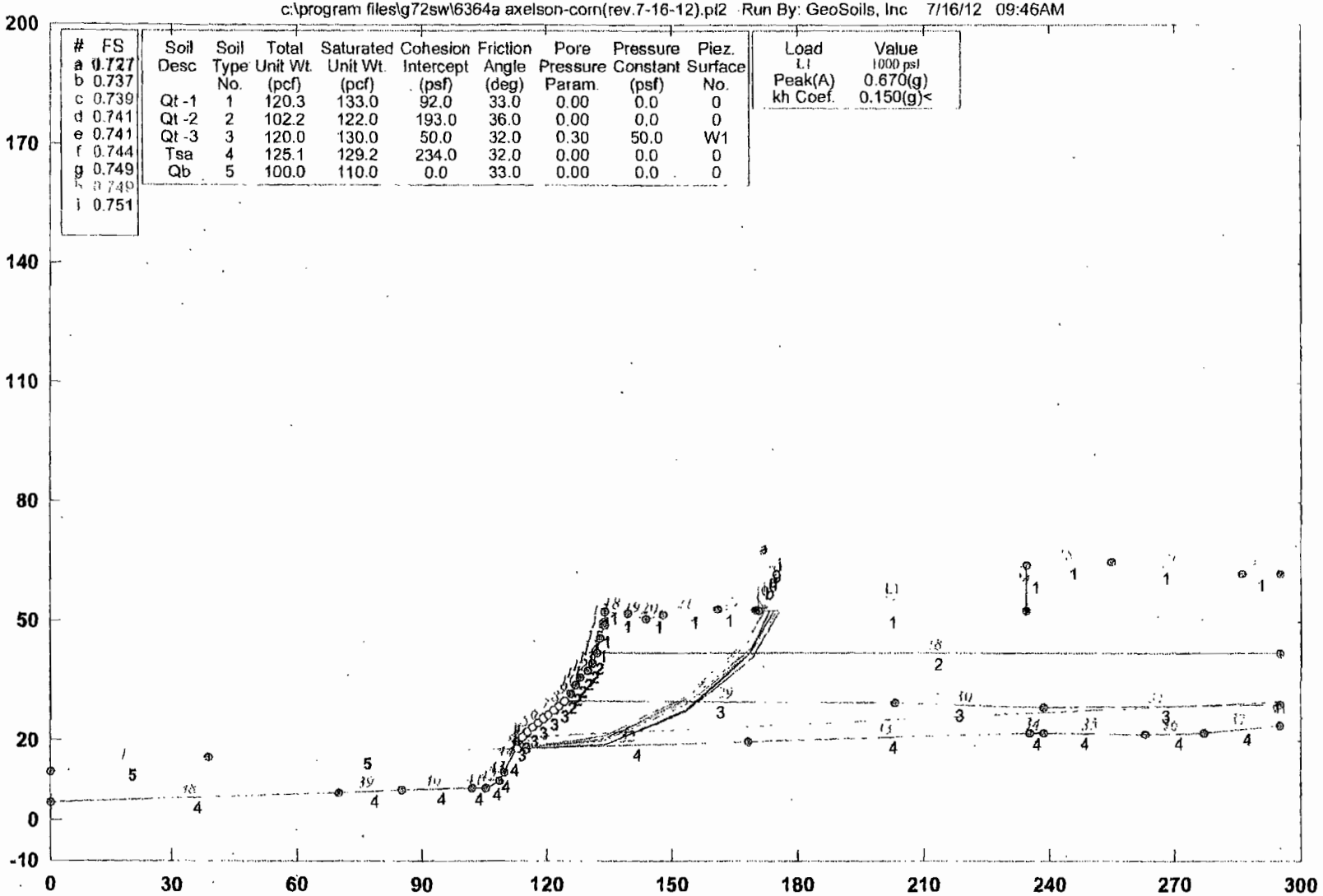
Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

FIGURE 16

6364-A-SC - 5323 Carlsbad Blvd. Section C-C' - Seismic

c:\program files\g72sw\6364a axelson-corn(rev.7-16-12).pl2 Run By: GeoSoils, Inc 7/16/12 09:46AM



#	FS
a	0.727
b	0.737
c	0.739
d	0.741
e	0.741
f	0.744
g	0.749
h	0.749
i	0.751

Soil Desc	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qt -1	1	120.3	133.0	92.0	33.0	0.00	0.0	0
Qt -2	2	102.2	122.0	193.0	36.0	0.00	0.0	0
Qt -3	3	120.0	130.0	50.0	32.0	0.30	50.0	W1
Tsa	4	125.1	129.2	234.0	32.0	0.00	0.0	0
Qb	5	100.0	110.0	0.0	33.0	0.00	0.0	0

Load	Value
LI	1000 psf
Peak(A)	0.670(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=0.727

Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

FIGURE 17

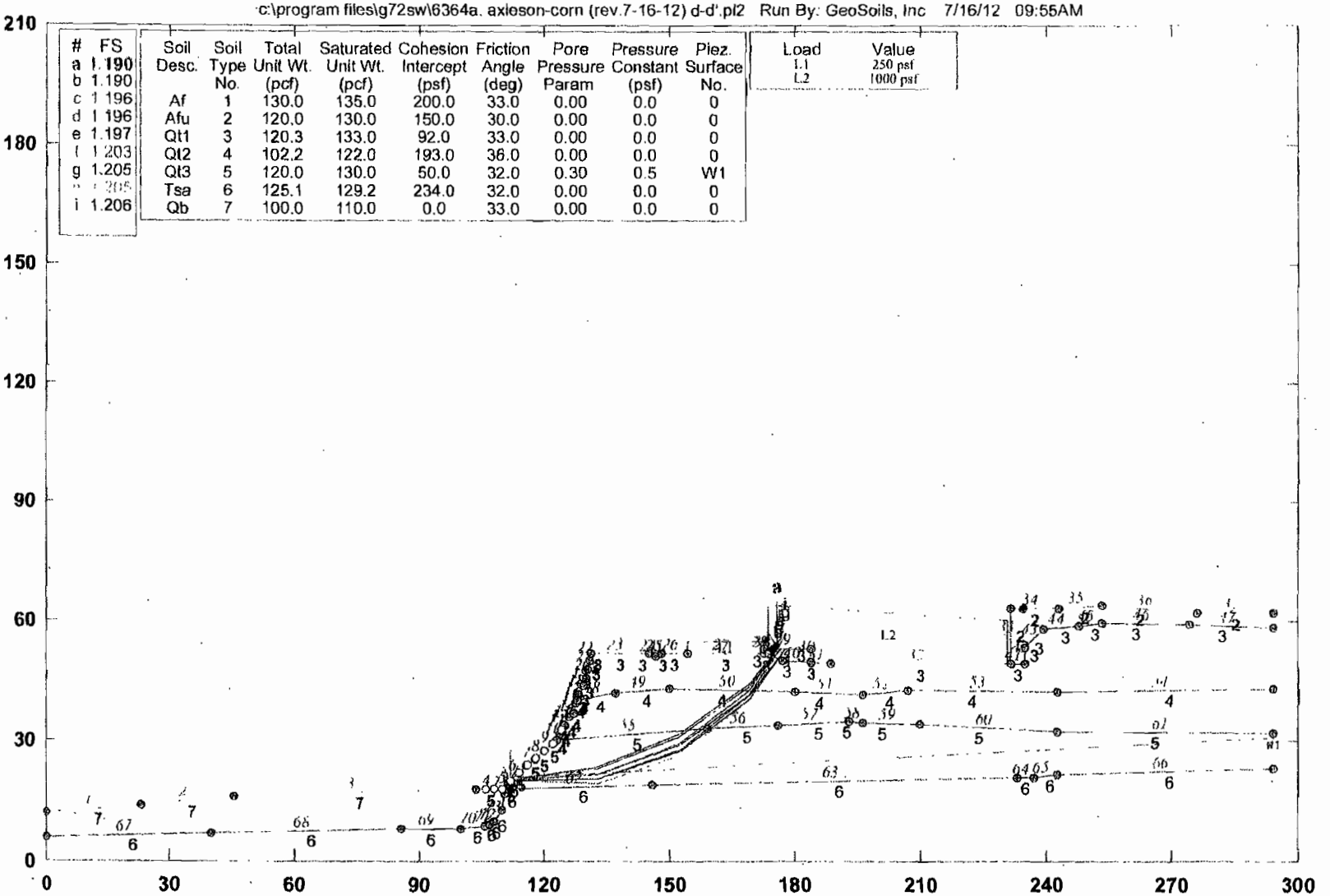
6364-A-SC 5327 Carlsbad Blvd. Section D-D' - Static

c:\program files\g72sw\6364a. axleson-corn (rev.7-16-12) d-d'.pl2 Run By: GeoSoils, Inc 7/16/12 09:55AM

#	FS
a	1.190
b	1.190
c	1.196
d	1.196
e	1.197
f	1.203
g	1.205
h	1.205
i	1.206

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param	Pressure Constant (psf)	Piez. Surface No.
Af	1	130.0	135.0	200.0	33.0	0.00	0.0	0
Afu	2	120.0	130.0	150.0	30.0	0.00	0.0	0
Qt1	3	120.3	133.0	92.0	33.0	0.00	0.0	0
Qt2	4	102.2	122.0	193.0	36.0	0.00	0.0	0
Qt3	5	120.0	130.0	50.0	32.0	0.30	0.5	W1
Tsa	6	125.1	129.2	234.0	32.0	0.00	0.0	0
Qb	7	100.0	110.0	0.0	33.0	0.00	0.0	0

Load	Value
L1	250 psf
L2	1000 psf



GSTABL7 v.2 FSmin=1.190

Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

FIGURE 18

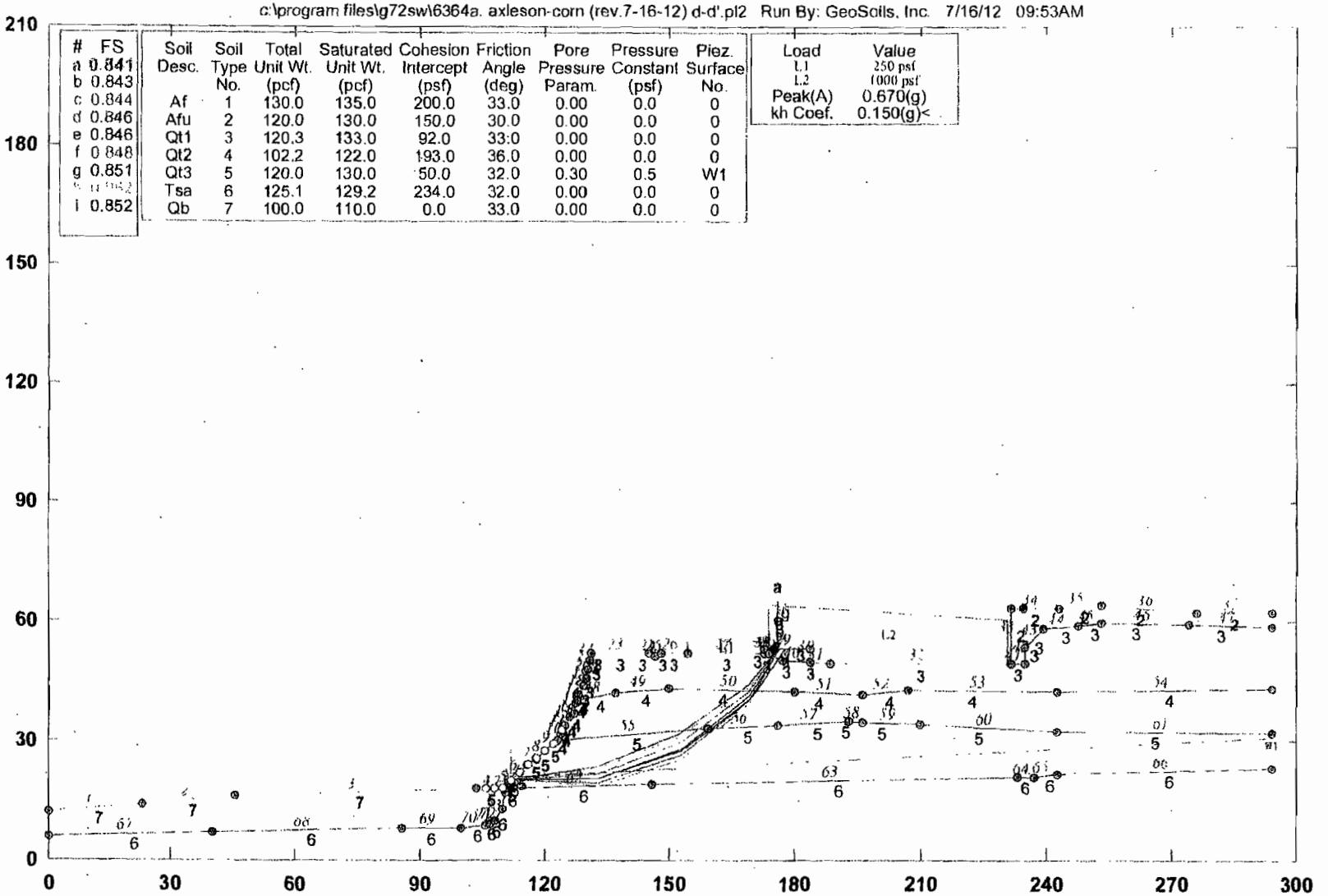
6364-A-SC 5327 Carlsbad Blvd. Section D-D' - Seismic

c:\program files\g72sw\6364a. axleson-corn (rev.7-16-12) d-d'.pl2 Run By: GeoSoils, Inc. 7/16/12 09:53AM

#	FS
a	0.841
b	0.843
c	0.844
d	0.846
e	0.846
f	0.848
g	0.851
h	0.852
i	0.852

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Af	1	130.0	135.0	200.0	33.0	0.00	0.0	0
Afu	2	120.0	130.0	150.0	30.0	0.00	0.0	0
Qt1	3	120.3	133.0	92.0	33.0	0.00	0.0	0
Qt2	4	102.2	122.0	193.0	36.0	0.00	0.0	0
Qt3	5	120.0	130.0	50.0	32.0	0.30	0.5	W1
Tsa	6	125.1	129.2	234.0	32.0	0.00	0.0	0
Qb	7	100.0	110.0	0.0	33.0	0.00	0.0	0

Load	Value
L1	250 psf
L2	1000 psf
Peak(A)	0.670(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=0.841

Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

FIGURE 19

and generally considered in need of emergency stabilization by the geotechnical community.

FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

Based on our review of available data and reports, and on our stability analysis of the coastal bluff, it is our opinion that the seawall provides protection to the public accessing and using Terramar Beach, protects the public beach access stairway, and provides protection to the subject properties. It is our further opinion, that if the seawall is removed, portions of the both residential properties would be in imminent danger of collapse, if not immediately upon removal of the seawall, or shortly thereafter. The unique geologic and geomorphic factors (i.e., regional and onsite faulting, groundwater, bluff instability, bluff geometry, susceptibility to wave attack and marine erosion, etc.) have significantly contributed to the increased erosion on this section of the coastline, and this erosion would only continue, and most likely accelerate, upon removal of the seawall.

As outlined in this report, the cove on the subject properties is more susceptible to rapid marine erosion and bluff instability than other areas of the North San Diego County coastline. Our review of available documents and historical photographs indicates that historically, this section of coastline was relatively well protected by a wider sand beach until the construction of the Camp Pendleton and Oceanside harbors and jetties and, to a lesser extent, the Encina Power Plant jetty immediately to the north, and later by a gunite seawall. Following the late 1977 storm season, the protective sandy beach was lost and beach sand was replaced by cobbles. During subsequent storm periods, these cobbles battered and destroyed the gunite protection, leaving it damaged and weakened. The 1983 storm period ultimately rendered the gunite protection useless, and the bluff eroded an estimated approximately 23 to 27 feet during a two-day storm period (Kuhn and Shepard, 1984). Absent the protective sand beach and gunite seawall, the geomorphic environment has significantly changed, leaving the lower bluff susceptible to rapid marine erosion. Removal of the seawall would re-subject the cove area to rapid marine erosion, which in turn, would instantaneously put the homes, the beach-going public and the vertical access stairway in jeopardy.

didn't address impacts associated w/ physical removal of seawall.

Our engineering analysis indicates that absent the wall, the cove area will have a significantly reduced factor of safety against failure and will be highly susceptible to marine erosion, placing the public access stairway and two subject residences in imminent danger.

The emergency permit application was reviewed by the City and the project approved and built in compliance with the current approved Local Coastal Plan. Given the pre-existing conditions, the project was necessary and absent the seawall, a catastrophic failure could occur that would injure or possibly kill a member of the public using this very popular public beach area and result in the loss of valuable public access and the subject residences.

Being one of the few locations allowing safe access to this reach of coastline, we feel that it is necessary for the seawall to remain and that removal of the seawall will result in accelerated marine erosion and loss of both private and public property and potentially place the public in danger of a geologic hazard that is currently mitigated. To that end, GSI recommends that the seawall remain in place.

This opportunity to be of service is sincerely appreciated. Should you have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,

GeoSoils, Inc.

Ryan Boehmer
Staff Geologist

David W. Skelly
Civil Engineer, RCE 47857

John P. Franklin
Engineering Geologist, CEG 1340

RB/JPF/DWS/jh

Attachments:
- Appendix A - References
- Appendix B - Selected Borings from ICG (1991)
- Plate 1 - Geotechnical Map
- Plate 2 - Geologic Cross Sections C-C' and D-D'
- Plate 3 - Geologic Cross Section E-E'

Distribution: (4) Addressee (via email and US mail)

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

Bryant, W.A., and Hart, E.W., 2007, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps; California Geological Survey, Special Publication 42, interim revision.

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APPENDIX B

SELECTED BORINGS FROM ICG (1991)

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 43' LOCATION: See Map

LOG OF BORING NO. 1

Sheet 1 of 1

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		56					TERRACE DEPOSITS (Qt): Reddish brown silty fine to medium SAND, dry to damp, very dense, some organics in upper foot	PARTICLE SIZE ANALYSIS
5		76						
10		28					Becoming lighter color, friable, damp, dense	
15		26						
20		22					Light to dark gray salt and pepper fine SAND, damp, medium dense, very friable	
							@21', Refusal on cobbles	
25							Total Depth 21' No Groundwater Refusal on Cobbles Backfilled 2/15/91	
30								
35								

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-2

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 47' LOCATION: See Map

LOG OF BORING NO. 2

Sheet 1 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		8					FILL/TOPSOIL: Light reddish brown silty fine SAND, damp, loose	
5		21					TERRACE DEPOSITES (Qt): Dark reddish brown silty fine to medium SAND, damp, medium dense, moderately friable	
10		17						
15		27					Yellowish gray fine to medium SAND, damp, medium dense, friable	
20		23					Light to dark gray fine SAND, damp, medium dense, slightly silty, micaceous, friable	
25		30						
30		26					Light to dark gray fine SAND, slightly moist, dense, micaceous, very friable, laminated/ cross bedded	
35		16					SANTIAGO FORMATION (Tsa): Olive gray with orange mottling silty claystone, slightly moist, very stiff	DIRECT SHEAR PARTICLE SIZE ANALYSIS
		42						

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-3

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 47' LOCATION: See Map

LOG OF BORING NO. 2

Sheet 2 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
40							Light orange brown fine to medium SANDSTONE, moist, dense, trace gravel	
		12					▽ Hole sloughing	
45								
		31						
50							Greenish gray silty fine SANDSTONE, wet, dense	
							Total Depth 50.5' Groundwater at 44' Backfilled 2/15/91	
55								
60								
65								
70								
75								

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-4

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 44' LOCATION: See Map

LOG OF BORING NO. 3

Sheet 1 of 2

DEPTH (FEET)	CLASSIFICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
0		10					FILL/TOPSOIL: Light reddish brown silty fine SAND, dry to damp, loose to medium dense	
5		40					TERRACE DEPOSITES (Qt): Reddish brown silty fine to medium SAND, damp, dense, moderately cemented	
10		34						
15		16					@15', Becoming light gray fine SAND, slightly silty, damp, medium dense, friable	
20							@19.5', some cobbles, no sampling	
25		25					Light gray fine to medium SAND, trace gravel, damp, medium dense, friable	
30		38						
35		26					@32', Cobble bed	
		25					Yellow-orange fine to medium SAND, slightly silty, moist, medium dense, water added during drilling	

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-4

DATE OBSERVED: 2-15-91 METHOD OF DRILLING: 8" Hollow Stem Auger

LOGGED BY: MD GROUND ELEVATION: 44' LOCATION: See Map

LOG OF BORING NO. 3

Sheet 2 of 2

DEPTH (FEET)	CLASSIF-ICATION	BLOWS/FOOT	UNDISTURBED SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (PCF)	DESCRIPTION	SOIL TEST
40								
							Total Depth 41' Backfilled 2/15/91 Groundwater at 38'	
45								
50								
55								
60								
65								
70								
75								

JOB NO.: 04-8529-001-00-00

ICG Incorporated

FIGURE: B-5

EXHIBIT M

STATE OF CALIFORNIA – THE NATURAL RESOURCES AGENCY EDMUND G. BROWN, JR., *Governor*

CALIFORNIA COASTAL COMMISSION
SAN DIEGO AREA
7575 METROPOLITAN DRIVE, SUITE 103
SAN DIEGO, CA 92108-4421
(619) 767-2370

January 8, 2013

Jon Corn
The Axelson Corn Law Firm
1220 N. Coast Highway 101, Suite 120
Encinitas, CA 92024

Re: Coastal Development Permit Appeal #A-6-CII-10-043:

Dear Mr. Corn:

To provide a brief project history, in April 2010 the Commission appealed a City issued follow-up coastal development permit (CDP #09-07) authorizing the construction of a 97-foot long by 17 to 24-foot high bluff-colored and textured seawall anchored in place with tiebacks, between the top of the seawall and the bluff top is a 1:1.5 landscaped fill slope located at 5323/5327 Carlsbad Boulevard. The seawall was originally approved and constructed through the issuance of a City issued emergency coastal development permit approved in June of 2009. The Commission appealed by project based on a number of issues including that: 1) the City's approval concluded that the seawall was considered necessary to protect a public beach from bluff failure, and not to protect an existing structure; 2) the City conditioned the approval using an inappropriate sand calculation for a total mitigation amount of $2,469.00; 3) no analysis or mitigation for impacts to loss of public recreation opportunities were identified, and; 4) the project's inconsistency with the City's certified LCP policy requiring a 25' lateral beach access dedication associated with the construction of any seawall or shoreline protective device, as no such access was required through the City's approval.

In October, 2011 Commission staff finalized a staff report and prepared a recommendation for the Commission requesting that the Commission find a Substantial Issue for the City approved CDP and subsequent denial of the proposed structure. The item was then scheduled for the Commission's November, 2011 hearing. At that time, the applicant requested the item be postponed in order to allow for the applicant to provide additional information. Among the additional information proposed was an updated geotechnical report indicating why the seawall was not capable of being removed.

On July, 12, 2012 the applicant then submitted an updated geotechnical evaluation completed by Geosoils, Inc. (GSI), titled "*Geotechnical Evaluation of Coastal Bluff Stability 5323 and 5327 Carlsbad Boulevard.*" Since that time, staff and other interested parties have had the opportunity to review the submitted document. While staff appreciates the additional material, we disagree that the report provided is sufficient to determine that the seawall cannot be removed. In addition, in order to determine whether the seawall can be permitted, it is necessary to evaluate the risk to the principal structures

at the site in the absence of the seawall (i.e., in the pre-construction condition). This is not the same as an analysis of whether the principal structures would be safe if the seawall was removed. As such, the information provided in the July, 12 2012 GSI report is insufficient to make that analysis. Therefore, a list of additional and necessary findings has been included below:

1) Please repeat the slope stability analysis with the bluff position as it was before the seawall was constructed, as surveyed by Melchior Land Surveying, inc.

2) Although the slope stability analyses provided show very low factors of safety at the location of the principal structures, they do not demonstrate that the failure surfaces corresponding to those low factors of safety are the most likely failure surfaces. The analysis did not seek out the unconstrained minimum factor of safety, and it is possible that the most likely failure surface will not intersect the principal structures. Please provide such an analysis.

3) The soil strength parameters adopted are unsupported by direct shear or other test data, and they do not appear to be from relatively undisturbed samples taken from the subject site. The cohesion values, in particular, are very low in comparison with similar materials with which the Commission's staff geologist is familiar. Please provide justification for your choice of soil strength parameters, or obtain new data from relatively undisturbed samples taken from the subject site.

4) Please repeat the analysis with an analytic method that is less sensitive to the ratio of normal to shear forces between slices, such as Spencer's, Morgenstern-Price, GLE, or Bishop's method. Reproduced below are excerpts from Table 1 from Johnsson (2005), for guidance in performing slope stability analyses for Coastal Commission review:

4) All slope analyses should ideally be performed using shear strength parameters (friction angle and cohesion), and unit weights determined from relatively undisturbed samples collected at the site. The choice of shear strength parameters should be supported by direct shear tests, triaxial shear test, or literature references, and should be in conformance with the guidelines published by the Geotechnical Group, American Society of Civil Engineers, Los Angeles Section (2002).

...

7) When planes of weakness are oriented normal to the slope or dip into the slope, or when the strength of materials is considered homogenous, circular failure surfaces should be sought through a search routine to analyze the factor of safety along postulated critical failure surfaces. In general, methods that satisfy both force and moment equilibrium, such as Spencer's (Spencer 1967; 1973), Morgenstern-Price (Morgenstern and Price 1965), and General Limit Equilibrium (Fredlund et al. 1981; Chugh 1986) are preferred. Methods based on moment equilibrium alone, such as Simplified Bishop's Method (Bishop 1955)

also are acceptable. In general, methods that solve only for force equilibrium, such as Janbu's method (Janbu 1973) are discouraged due to their sensitivity to the ratio of normal to shear forces between slices (Abramson et al. 1995).

8) If anisotropic conditions are assumed for units containing critical failure surfaces determined above, and when planes of weakness are inclined at angles ranging from nearly parallel to the slope to dipping out of slope, factors of safety for translational failure surfaces should also be calculated. The use of a block failure model should be supported by geologic evidence for anisotropy in rock or soil strength. Shear strength parameters for such weak surfaces should be supported through direct shear tests, triaxial shear test, or literature references.

From: Johnsson, M.J., 2005, Establishing development setbacks from coastal bluffs, *in* Magoon, O.T., Converse, H., Baird, B., Jines, B., and Miller-Henson, M., eds., California and the World Ocean '02: Revisiting and revising California's Ocean Agenda: Reston, Virginia, American Society of Civil Engineers, p. 396-416.

In additional to the above listed items, an email from Todd Cardiff representing Surfrider (an identified appellant) has been included and also lists additional questions regarding the updated geotechnical report. Please also provide a response to all of the attached questions to the best of your ability. If you have any questions please do not hesitate to contact me directly.

Sincerely,

Toni Ross
Coastal Planner

cc: Dean Goetz
Marshall Sylver
Dave Skelly
Todd Cardiff
Mark Johnsson

(Document2)

EXHIBIT N

6 OETZ Seawall

GEOTECHNICAL RESPONSE TO CALIFORNIA COASTAL COMMISSION REVIEW COMMENTS, SEAWALL LOCATED AT 5323 AND 5327 CARLSBAD BOULEVARD CARLSBAD, SAN DIEGO COUNTY, CALIFORNIA COASTAL DEVELOPMENT PERMIT APPEAL # A-6-CII-10-043

FOR

AXELSON & CORN, P.C.
160 CHESTERFIELD DRIVE, SUITE 201
CARDIFF BY THE SEA, CALIFORNIA 92007

W.O. 6364-A1-SC JANUARY 24, 2014

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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COASTAL COMMISSION
SAN DIEGO COAST DISTRICT

GEOTECHNICAL RESPONSE TO CALIFORNIA COASTAL COMMISSION REVIEW COMMENTS, SEAWALL LOCATED AT 5323 AND 5327 CARLSBAD BOULEVARD CARLSBAD, SAN DIEGO COUNTY, CALIFORNIA COASTAL DEVELOPMENT PERMIT APPEAL # A-6-CII-10-043

GeoSoils, Inc.

FOR

AXELSON & CORN, P.C.
160 CHESTERFIELD DRIVE, SUITE 201
CARDIFF BY THE SEA, CALIFORNIA 92007

W.O. 6364-A1-SC JANUARY 24, 2014

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January 24, 2014

W.O. 6364-A1-SC

Axelson & Corn, P.C.
160 Chesterfield Drive, Suite 201
Cardiff by the Sea, California 92007

Attention: Mr. Jon Corn

Subject: Geotechnical Response to California Coastal Commission Review Comments, Seawall Located at 5323 and 5327 Carlsbad Boulevard, Carlsbad, San Diego County, California, Coastal Development Permit Appeal # A-6-CII-10-043

Dear Mr. Corn:

In accordance with your request and Mr. Dean Goetz's authorization, GeoSoils, Inc. (GSI) is providing this geotechnical response to review comments prepared by the California Coastal Commission (2013 [see Appendix A]) as well as inquiries from appellant Todd Cardiff, Esq. who has been in contact with Coastal Commission staff regarding this appeal. The services GSI performed for this response included a review of the referenced documents listed in Appendix A and contained in Appendix B, sampling of coastal bluff materials, additional laboratory testing (Appendix C), supplemental slope stability analyses, and the preparation of this response. Unless specifically superseded herein, the conclusions and recommendations contained in GSI (2012) are still considered valid and applicable.

GEOTECHNICAL RESPONSE TO CALIFORNIA COASTAL COMMISSION (2013) REVIEW COMMENTS

For ease of review, the California Coastal Commission ([CCC], 2013) review comments are repeated in **bold** font and followed by GSI's response.

CCC Comment No. 1

Complete a new slope stability analysis assuming the bluff position as it was before the seawall was constructed, as surveyed by Melchoir Land Surveying, inc.

GSI Response to CCC Comment No. 1

GSI has re-analyzed slope stability of the coastal bluff along Geologic Cross Sections C-C' and D-D' (see Plates 1 and 2), assuming the bluff position as surveyed by Melchoir Land Surveying, Inc. (MLS). Our previous analyses, performed in preparation of GSI (2012), were conducted to evaluate the factor-of-safety (FOS) for the principal structures, if the existing seawall was removed. As demonstrated in GSI (2012), if the seawall was removed, the FOS for the residential structures at the subject properties would be unacceptable and would provide justification for emergency mitigation.

Our current analyses incorporate the bluff position and profile prior to seawall construction, as surveyed by MLS (2009). Slope stability analyses were evaluated with the aid of the computer program GSTABL7 (Gregory, 2003).

Shortly following our assignment to respond to CCC (2013), GSI contacted Dr. Garry Gregory, P.E. in an attempt to verify our approach to the slope stability analyses, using the GSTABL7 program, and to review our results. Dr. Gregory is an Adjunct Professor of Civil Engineering at Oklahoma State University, resides on the Board of Governors of the Geo-Institute, is the Principal Engineer at Gregory Geotechnical, Inc. and is the author/creator of the GSTABL7 program. The GSTABL7 program is one of the industry standards for slope stability analyses throughout the United States. Dr. Gregory agreed to offer his expertise, and suggested that GSI wait until the end of summer 2013 when his newest version of the GSTABL slope stability program would be available, so that our analyses incorporated the latest technology. In the fall of 2013, we were informed that the new program would not be released until sometime in 2014, so we decided to move forward with the current version (GSTABL7, Version 2.004). In November 2013, Dr. Gregory provided guidance and comments on how the site-specific geologic conditions should be modeled and analyzed. Dr. Gregory's comments included the following:

1. GSI could consider the use of a progressive failure approach. This involves determining the point on the bluff top where the FOS equals 1.0 and then modeling the associated failure surface as the failed bluff profile. Following this evaluation, the FOS for the principal structures could be evaluated.

2. GSI could also consider the addition of pore pressure into each sub-unit of the Quaternary terrace deposits (Qt) owing to a likely wetted front from irrigation and rainfall in the uppermost sub-units (Qt_1 and Qt_2), and saturated conditions below the phreatic surface for the lowest sub-unit (Qt_3).

3. Softening of the exposed soils at the face of the bluff from weathering should also be considered in the analyses. This weathering typically occurs when there is a 20 percent or greater fines fraction, as is the case at this site. For these exposed layers, the phi angle increases and the cohesion is essentially reduced to zero (pounds per square foot [psf]) the closer the soil is to the face of the bluff. This

condition would necessitate a sub-area at the face of the bluff within each sub-unit of the terrace deposits.

All of these suggestions, if incorporated in the analysis, would result in lower FOS values at the seaward edge of the principal structures. After analyses of multiple iterations of soil profiles using various soil properties, as suggested by Dr. Gregory, and in consideration of the standard of practice currently used in bluff stability in San Diego North County, GSI proposed a very conservative approach to the analysis, as demonstrated herein.

Our current analyses consider soil strengths determined from relatively undisturbed samples collected directly from the project site as well as the Qt_1 materials from a neighboring site located at the northeast corner of the Cerezo Drive and Carlsbad Boulevard intersection (approximately 270 feet southeast of the subject site). Our current analyses also incorporate a pore pressure parameter of 0.15 within the Qt_3 materials to account for the groundwater conditions encountered by ICG (1991) as well as a transient, elevated phreatic surface which could be expected during sustained or short interval, high intensity rainfall, similar to conditions witnessed during the 1982-83, 1997-98, and 2008-2009 El Niño events. In addition, GSI used a pore pressure parameter of 0.10 within the Qt_1 and and Qt_2 materials to model a wetted front produced from infiltrated onsite and near-site irrigation, and precipitation. According to Fredlund and Barbor (1986), pore pressure parameter design values, used in slope stability analyses, typically vary between 0.30 and 0.45; the higher the pressure, the greater the instability. Thus, the pore pressure parameters used in current analyses are very conservative, and the use of higher (and perhaps more exact) values would result in an even lower FOS.

The current analyses demonstrate unacceptable FOS values for both residential structures at the time of the MLS (2009) survey (see attached Figures 1 through 4). These analyses demonstrate that at the time of the MLS (2009) survey (i.e., post-failure and pre-seawall), both residences needed bluff protection against seismically induced bluff failure (see attached Figures 2 and 4), and are within one (1) to two (2) storm-driven failure events that could quickly push the FOS to even lower levels, placing the homes in further jeopardy.

GSI also points out that at the time of the MLS (2009) survey, some talus from the December 2008 bluff failure, with thicknesses on the order of approximately 1 foot on the mid-face to several feet at the toe, was present (see attached Figure 7), and included in the surveyed geometry. That is, the MLS (2009) survey incorporated the failed earth materials and therefore, does <u>not</u> accurately represent the actual bluff face position and profile as some of the intact bluff slope was concealed beneath these loose materials. With the exception of the bluff top and failure scarp, the actual position of the lower two-thirds of the bluff slope was approximately 2 to 4 feet landward of the survey position. Based upon our observations and as evidenced by the adjacent natural bluff to the south of the stairway, the profile of the lower two-thirds of the unseen bluff slope at the site was steeper than shown in the MLS (2009) survey. To that end, GSI concludes that the actual FOS for the residential structures are most likely less than indicated in our current analyses and are

likely consistent with the FOS calculated in GSI (2012), since the former analyses modeled a bluff slope positioned 2½ feet more landward than shown in the MLS (2009) survey. For these reasons, it is GSI's opinion that our previous analyses (GSI, 2012) remain accurate and valid.

CCC Comment No. 2

Although the slope stability analyses provided show very low factors of safety at the location of the principal structures, they do not demonstrate that the failure surfaces corresponding to those low factors of safety are the most likely failure surfaces. The analysis did not seek out the unconstrained minimum factor of safety, and it is possible that the most likely failure surface will not intersect the principal structures. Please provide such an analysis.

GSI Response to CCC Comment No. 2

Attached Figures 5 and 6 provide the unconstrained minimum factors of safety for Geologic Cross Sections C-C' and D-D'.

CCC Comment No. 3

The soil strength parameters adopted are unsupported by direct shear or other test data, and they do not appear to be from relatively undisturbed samples taken from the subject site. The cohesion values, in particular, are very low in comparison with similar materials with which the Commission's staff geologist is familiar. Please provide justification for your choice of soil strength parameters, or obtain new data from relatively undisturbed samples taken from the subject site.

GSI Response to CCC Comment No. 3

The soil strength parameters assigned to the Tertiary Santiago Formation (i.e., Tsa) and the middle sub-unit of the Quaternary terrace deposits (i.e., Qt_2) in the GSI (2012) analyses were based on the results of direct shear testing performed on relatively undisturbed samples collected from the subject coastal bluff, prior to seawall construction. Soil strength parameters assigned to the uppermost sub-unit of the terrace deposits (i.e., Qt_1) in the GSI (2012) analyses were based on direct shear testing results for a relatively undisturbed sample collected from the property located at the northeast corner of the Cerezo Drive and Carlsbad Boulevard intersection (i.e., about 270 feet away from the subject site). In our professional judgement, this sample is representative of Qt_1. However, GSI points out that we increased the cohesion of the Qt_1 materials above the laboratory values to account for the light iron-oxide cementation observed in this sub-unit at the site.

Owing to the relatively cohesionless and highly friable nature of the lowest sub-unit of the terrace deposits (i.e., Qt_3), GSI could not readily acquire a relatively undisturbed ring

sample of this material. Thus, for our 2012 study, GSI assigned reasonably conservative strength values to this sub-unit, based on our experience with other coastal bluff properties within the region.

In order to obtain representative strength values of the Qt_3 sub-unit for use in this current study, GSI collected a chunk sample of the Qt_3 materials from bluff exposures at the south end of Terramar Cove (adjacent to the site). GSI then evaluated the in-place (field) dry density and moisture content of the chunk sample in general accordance with guidelines presented in United States Army Corps of Engineers Engineering Manual EM 1110-2-1906. Subsequently, GSI performed direct shear testing on ring samples remolded to the field dry density and field moisture content obtained from the chunk sample in general accordance with ASTM D 3080. Shear testing results of the coastal bluff materials are presented in Appendix B.

GSI points out that this particular sub-unit of the terrace deposits is strongly anisotropic. That is to say that this layer is much stronger under vertical stress than under horizontal stress. The strength parameters obtained from the remolded sample of Qt_3 is for a homogeneous (isotropic) soil mass and not an anisotropic soil mass. The calculated strength properties represent the average strength of the soil. Thus, the stability of this portion of the slope is overestimated, and the actual FOS is likely lower than the FOS found in the analyses.

GSI also points out that the cohesion values assigned to the terrace deposits in these as well as the GSI (2012) analyses are actually <u>higher</u> (i.e., more conservative) than values recommended by the CCC's staff geologist for analyses of nearby coastal bluffs in Encinitas, and Solana Beach (CCC, 2002[see Appendix B]) in that we used a cohesion of 100 psf, and the CCC geologist recommends using a cohesion of 0 psf. The use of a lower cohesion value would only <u>decrease</u> the calculated FOS.

<u>CCC Comment No. 4</u>

Please repeat the analysis with an analytic method that is less sensitive to the ratio of normal to shear forces between slices, such as Spencer's, Morgenstern - Price, GLE, or Bishop's method. Reproduced below are excerpts from Table 1 from Johnsson (2005), for guidance in performing slope stability analyses for Coastal Commission review:

<u>GSI Response to CCC Comment No. 4</u>

GSI has re-analyzed Geologic Cross Sections C-C' and D-D' using the suggested Modified Bishop's method. Results of the analysis are presented on the attached Figures 1 through 6. GSI points out that the GSTABL7 program does not freely model progressive failures without user input and each successive bluff failure at the site would lower the FOS of the residential structures. Considering that Kuhn and Shepard (1984) chronicled that

up to 27 feet of material was removed from the coastal bluff at the actual site during a single storm event in August 1983, it is reasonable to assume that without the seawall, the FOS for the residential structures could dramatically decrease and place both residential structures in immediate peril in one to perhaps two future failure events. GSI also points out that adherence to the guidelines in CCC (2003 [see Appendix B]) for combining slope stability calculations with long-term erosion rates when establishing safe setbacks from coastal bluffs demonstrates that the existing residential structures are not safe from bluff failure and retreat without the seawall.

CONCLUSIONS

The site has been documented to have unique geologic conditions which explain the relatively large indentation of the shoreline (cove), immediately adjacent to the two subject residences. These unique conditions were described in GSI (2012) and TerraCosta Consulting Group, Inc. (2012). During the construction of the wall, when drilling through the weak, lowermost terrace deposit sub-unit, borehole collapse and excessive groundwater were encountered. The inability to properly drill, place and test tie-backs into this very weak soil layer resulted in the addition of another row of tie-backs during construction. The GSI (2012) stability analysis showed that if the seawall were removed, both of the residences would have an inadequate FOS for both static and seismic conditions (see attached Figures 1 through 4). The current study shows that an inadequate FOS existed immediately after the failure, and prior to the installation of the seawall. In fact, our current analyses indicate that at the time of the MLS (2009) survey, both residential structures required protection from seismically induced bluff failure.

As stated previously, the analyses provided herein are based upon a historical survey performed in 2009, not of the actual bluff face, but rather included some of the failed bluff material that lay on the unseen 2009 bluff face. Under this unrealistic scenario, the static and seismic FOS values for both residential structures are inadequate and demonstrate that the existing seawall is required to protect the structures. While it is impossible to know exactly where the bluff face was in 2009 based upon available information, the fact that the landslide debris was present during the survey suggests that portions of the mid-bluff slope and toe lied further landward than analyzed herein, and the actual FOS values are likely less than that demonstrated herein. Finally, our 2012 analyses modeled conditions as if the existing seawall was removed. These analyses show that a replacement seawall would immediately be needed to resume protection of the residential structures.

GEOTECHNICAL RESPONSE TO INQUIRIES MR. TODD CARDIFF, ESQ.

For ease of review, the inquiries by Todd Cardiff, that Coastal staff requested GSI to respond to are repeated in **bold** font and followed by GSI's response. Inquiry No. 5 appears to be directed to the CCC and no response is provided for that particular question.

Inquiry No. 1

Does the seawall extend underneath the stairway? On the most recent coastal photograph on California Coastal Records Project, the seawall appears to end right next to the stairs. If the seawall ends next to the stairs, wouldn't end effects from the seawall potentially increase the erosion on the unprotected bluff?

GSI Response to Inquiry No. 1

No. The seawall does not extend underneath the stairway. The northern face of the stairway structure is the southerly terminus of the seawall. There are no structural connections between the seawall and the stairway. Rather, there is elastomeric joint material between the two structures to allow them to perform independently of each other.

Based on our observations, post-seawall construction, there is no readily visible evidence of exacerbated marine erosion on the bluffs directly north or south of the seawall, resulting from so-called end effects. The only notable area of accelerated bluff erosion is occurring along the upper portion of the bluff to the south of the staircase. This erosion is the result of an overwhelmed, existing storm drain catch basin, near the top of the staircase, which allows overflow waters to cascade down the bluff face. It is not the result of exacerbated marine erosion from seawall end effects but from improper surface drainage along the bluff top, and is evidenced by the rills and large gullies exposed along this portion of the upper coastal bluff. The aforementioned storm drain catch basin was not a component of the seawall construction. Rather, it existed prior to seawall construction, and is part of the public infrastructure that was installed long ago by the City of Carlsbad or some other public agency (e.g., State Parks).

During a December 6, 2013 site reconnaissance, GSI also observed an excavation beneath the access stairway. This excavation appears to have been created by beach users. In addition, there is also some minor erosion to the mid-bluff slope south of the access stairway. This minor erosion has been exacerbated by foot traffic along this portion of the bluff face, and speaks to the high volume of beach goers in this particular area.

Inquiry No. 2

We see a factor of safety analysis for Sections C-C' and Sections D-D'. Are there other sections that were analyzed? (i.e., Sections A-A', B-B'?)

GSI Response to Inquiry No. 2

No other sections were analyzed. The purpose of labeling the Geologic Cross Sections C-C' and D-D' rather than A-A' and B-B' on Plates 1 and 2 of GSI (2012) was to avoid confusion between the A-A and B-B section locations the project civil engineer included on the base map GSI used for our Plate 1.

Inquiry No. 3

The previous factor of safety analysis prepared by GSI for the seawall on 1.21.2012 [2009] included the back-up data. Could we please have all the back-up data used to analyze the factor of safety? One of our members who is a trained geologist has been unable to replicate or properly evaluate the models without the back-up data.

GSI Response to Inquiry No. 3

GSI did not prepare January 21, 2009 or January 21, 2012 reports for this project. GSI issued an application for an emergency coastal development permit for this project on January 20, 2009. However, there were no slope stability calculations included in that report. All the information necessary to replicate the slope stability analyses in our July 12, 2012 report (GSI, 2012) is on the slope stability output figures attached to that report.

Inquiry No. 4

GSI claims that the erosion continued during construction of the seawall, and therefore GSI assumed that the bluff edge was 2.5 feet landward of the MLS survey. That does not correspond with my memory of the construction. In fact, it appeared that the applicant built the seawall vertically (or almost vertically) from the base of the bluff and backfilled the space between the bluff edge and the seawall, further extending his back yard. Is an assumption that 2.5 feet of bluff edge eroded due to seawall construction reasonable? Is it also reasonable to assume that another 2.5 feet of bluff would collapse with removal of the seawall? What is this assumption based upon?

GSI Response to Inquiry No. 4

In GSI (2012), we stated that "as with all bluff restoration projects, bluff failure can continue during the course of construction." Given the friable nature of terrace deposits, some raveling of the bluff face materials occurred during construction as a result of physical weathering and construction activity, and vibrations, as with all similarly sloped sites. GSI also points out that during this particular project, talus material from the bluff failure was present on the bluff face which required its removal for proper support of the wall backfill. It was also necessary to anchor the landward edge of the geogrid reinforcement for the reconstructed bluff slope into slurry. This required some benching into the bluff slope. Combined, these factors reasonably model the post-seawall construction bluff edge ~2½ feet more landward than that shown on the MLS (2009) survey in the GSI (2012) analysis. This relatively small distance of failure and bluff retreat is reasonable to account for the observed sloughing, removal of talus, and benching into the bluff that occurred during construction.

Contrary to the assertion that the homeowners extended their rear yards, the City of Carlsbad Planning Department mandated, as a condition of permit, that the reconstructed bluff edge could not extend seaward of the pre-existing bluff edge shown on the approved plans dated April 14, 2009 (Second Revision) by Charles J. Randle. Thus, there were no extensions of usable space in the rear yards of the subject properties.

Inquiry No.5

This inquiry was directed to the Coastal Commission.

Inquiry No. 6

Were the logs from ICG - 1991 used in determining soil cohesion? How was soil cohesion calculated? Why does the soil cohesion change from the geology report prepared on 1.21.2009 and the most recent geology report? Are there other unexplained inconsistencies between the reports? Have any recent borings been analyzed? Does the CCC have the boring map?

GSI Response to Inquiry No. 6

The soil cohesion values used in the GSI (2012) analyses were generally acquired through direct shear testing of relatively undisturbed samples of onsite and near-site earth materials, with the exception of the lowest sub-unit of the Quaternary terrace deposits (Qt_3). The cohesion for this sub-unit was reasonably assumed, based our regional experience with coastal bluff projects, as discussed previously. As also previously stated, GSI has no records of a report dated January 21, 2009 for this project. GSI is not aware of any borings advanced at the subject site that are more recent than those included in the ICG (1991) study. The locations of the ICG (1991) borings advanced at the subject site were shown on Plate 1 of GSI (2012), which is part of the record.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed slope repair. Site conditions may vary due to seasonal changes or other factors.

The conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an

agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project.

If you have any questions or comments regarding this letter, please do not hesitate to contact the undersigned.

Respectfully submitted,

GeoSoils, Inc.

PROFESSIONAL GEOLOGIST
JOHN P. FRANKLIN
No. 1340
Certified Engineering Geologist
STATE OF CALIFORNIA

John P. Franklin
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David W. Skelly
Civil Engineer, RCE 47857

Ryan B. Boehmer
Project Geologist

RB/JPF/DWS/jh

Attachments:
- Figures 1 through 6 - GSTABL7 Output Plots
- Figure 7 - Site Photograph
- Appendix A - References
- Appendix B - CCC (2003)
- Appendix C - Shear Test Results
- Plate 1 - Cross Section Location Map
- Plate 2 - Geologic Cross Sections C-C' and D-D'

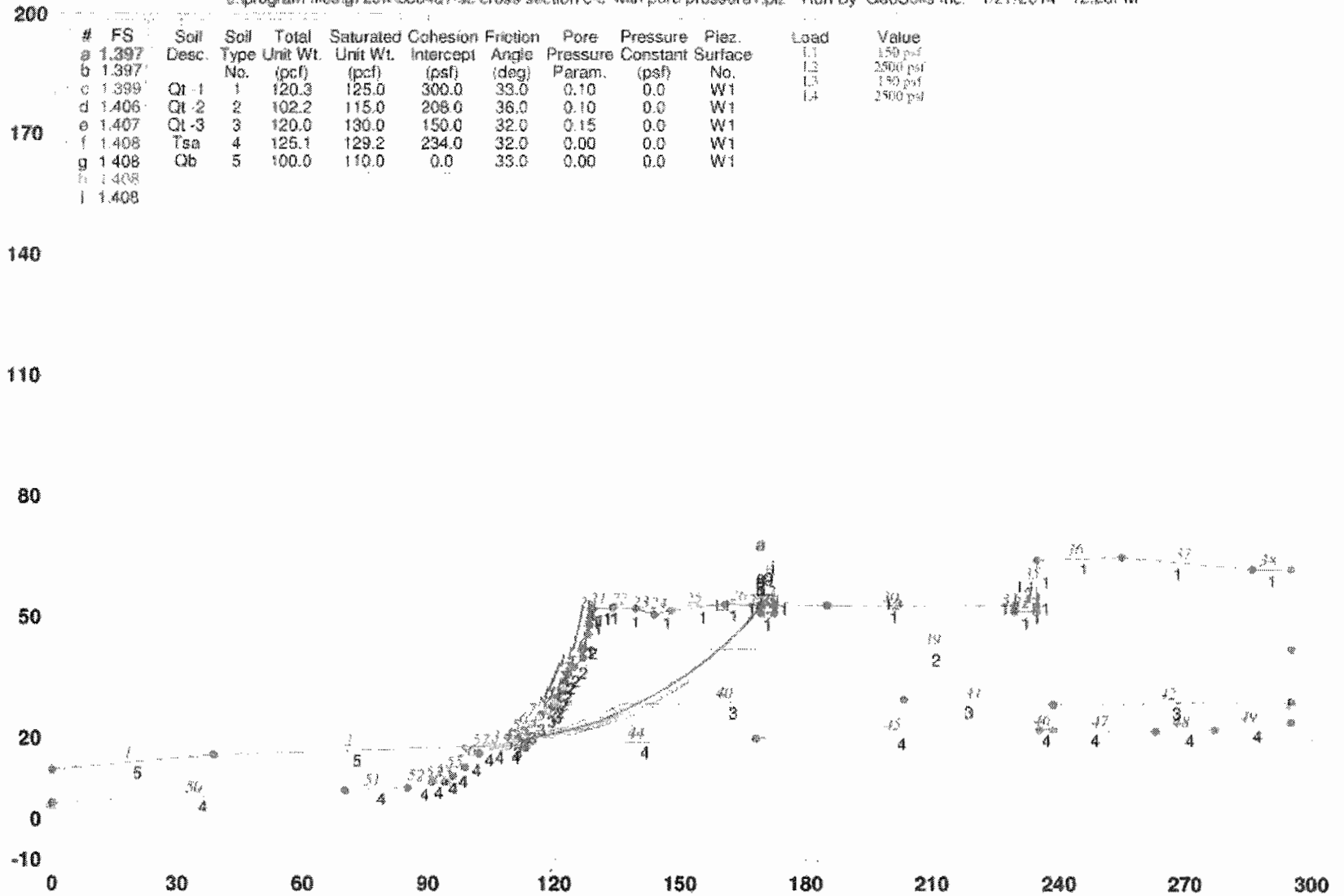
Distribution:
- (1) Addressee (via US mail and email)
- (1) TerraCosta Consulting Group, Inc. Attention: Mr. Walter Crampton
- (3) California Coastal Commission, Attention: Ms. Toni Ross

6364-A1-SC 5323 Carlsbad Blvd. C-C' Post-failure Pre-seawall Static

c:\program files\g72sw\6364a1-sc cross section c-c' with pore pressure1.pl2 Run By GeoSoils Inc. 1/27/2014 12:26PM

#	FS	Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
a	1.397									
b	1.397									
c	1.399	Qt -1	1	120.3	125.0	300.0	33.0	0.10	0.0	W1
d	1.406	Qt -2	2	102.2	115.0	208.0	36.0	0.10	0.0	W1
e	1.407	Qt -3	3	120.0	130.0	150.0	32.0	0.15	0.0	W1
f	1.408	Tsa	4	125.1	129.2	234.0	32.0	0.00	0.0	W1
g	1.408	Qb	5	100.0	110.0	0.0	33.0	0.00	0.0	W1
h	1.408									
i	1.408									

Load	Value
L1	150 psf
L2	2500 psf
L3	150 psf
L4	2500 psf



GSTABL7 v.2 FSmin=1.397

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Static FOS at Seaward Foundation

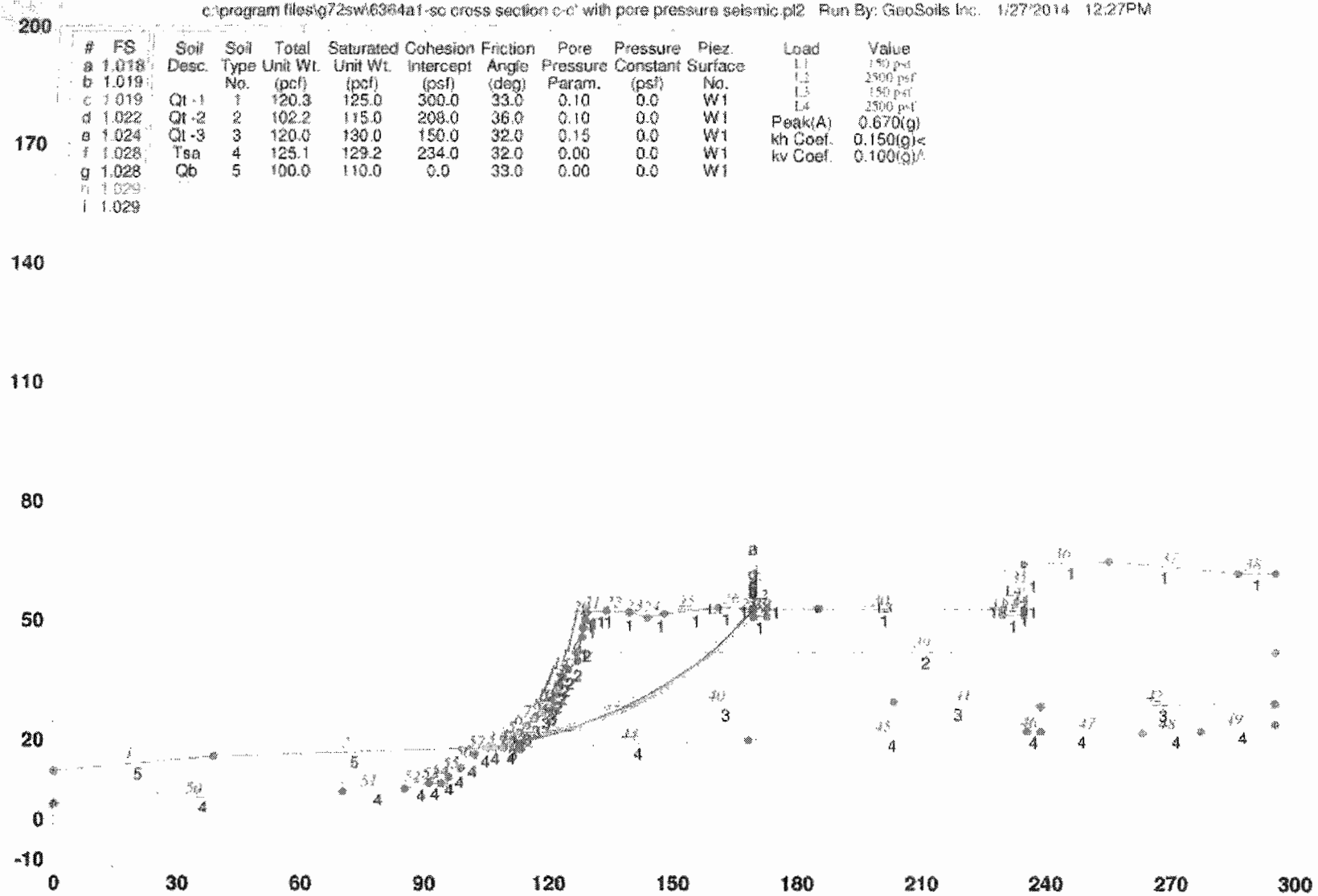
Figure 1

6364-A1-SC 5323 Carlsbad Blvd. C-C' Post-failure Pre-seawall Seismic

c:\program files\g72sw\6364a1-sc cross section c-c' with pore pressure seismic.pl2 Run By: GeoSoils Inc. 1/27/2014 12:27PM

#	FS	Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
a	1.018									
b	1.019									
c	1.019	Qt -1	1	120.3	125.0	300.0	33.0	0.10	0.0	W1
d	1.022	Qt -2	2	102.2	115.0	208.0	36.0	0.10	0.0	W1
e	1.024	Qt -3	3	120.0	130.0	150.0	32.0	0.15	0.0	W1
f	1.028	Tsa	4	125.1	129.2	234.0	32.0	0.00	0.0	W1
g	1.028	Qb	5	100.0	110.0	0.0	33.0	0.00	0.0	W1
h	1.029									
i	1.029									

Load	Value
L1	150 psf
L2	2500 psf
L3	150 psf
L4	2500 psf
Peak(A)	0.670(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)/



GSTABL7 v.2 FSmin=1.018

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Seismic FOS at Seaward Foundation

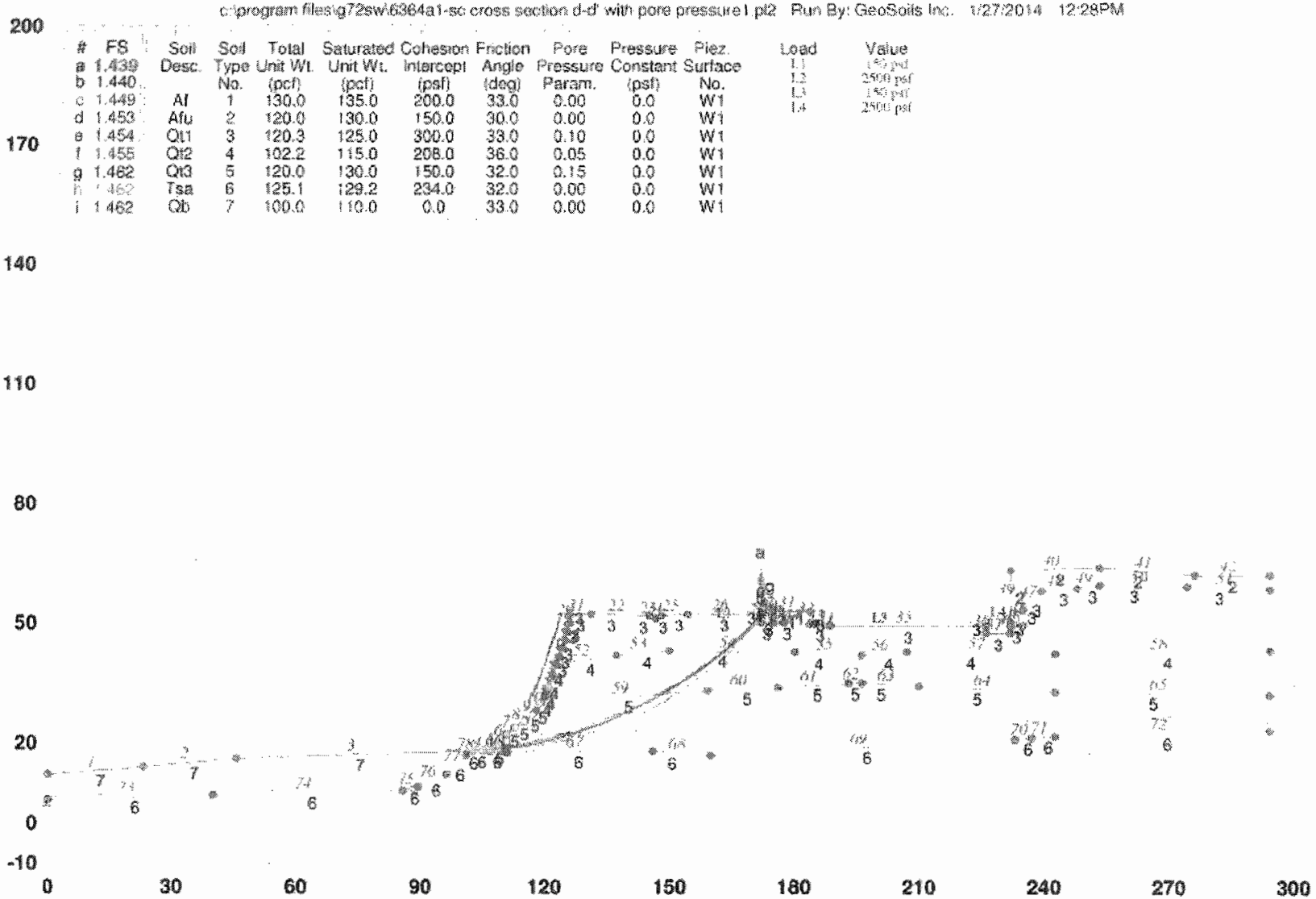
Figure 2

6364-A1-SC 5327 Carlsbad Blvd. D-D' Post-failure Pre-seawall Static

c:\program files\g72sw\6364a1-sc cross section d-d' with pore pressure1.pl2 Run By: GeoSoils Inc. 1/27/2014 12:28PM

#	FS	Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
a	1.439									
b	1.440									
c	1.449	Af	1	130.0	135.0	200.0	33.0	0.00	0.0	W1
d	1.453	Afu	2	120.0	130.0	150.0	30.0	0.00	0.0	W1
e	1.454	Qt1	3	120.3	125.0	300.0	33.0	0.10	0.0	W1
f	1.455	Qt2	4	102.2	115.0	208.0	36.0	0.05	0.0	W1
g	1.462	Qt3	5	120.0	130.0	150.0	32.0	0.15	0.0	W1
h	1.462	Tsa	6	125.1	129.2	234.0	32.0	0.00	0.0	W1
i	1.462	Qb	7	100.0	110.0	0.0	33.0	0.00	0.0	W1

Load	Value
L1	150 psf
L2	2500 psf
L3	150 psf
L4	2500 psf



GSTABL7 v.2 FSmin=1.439

Safety Factors Are Calculated By The Modified Bishop Method

Static FOS at Seaward Foundation

GSTABL7

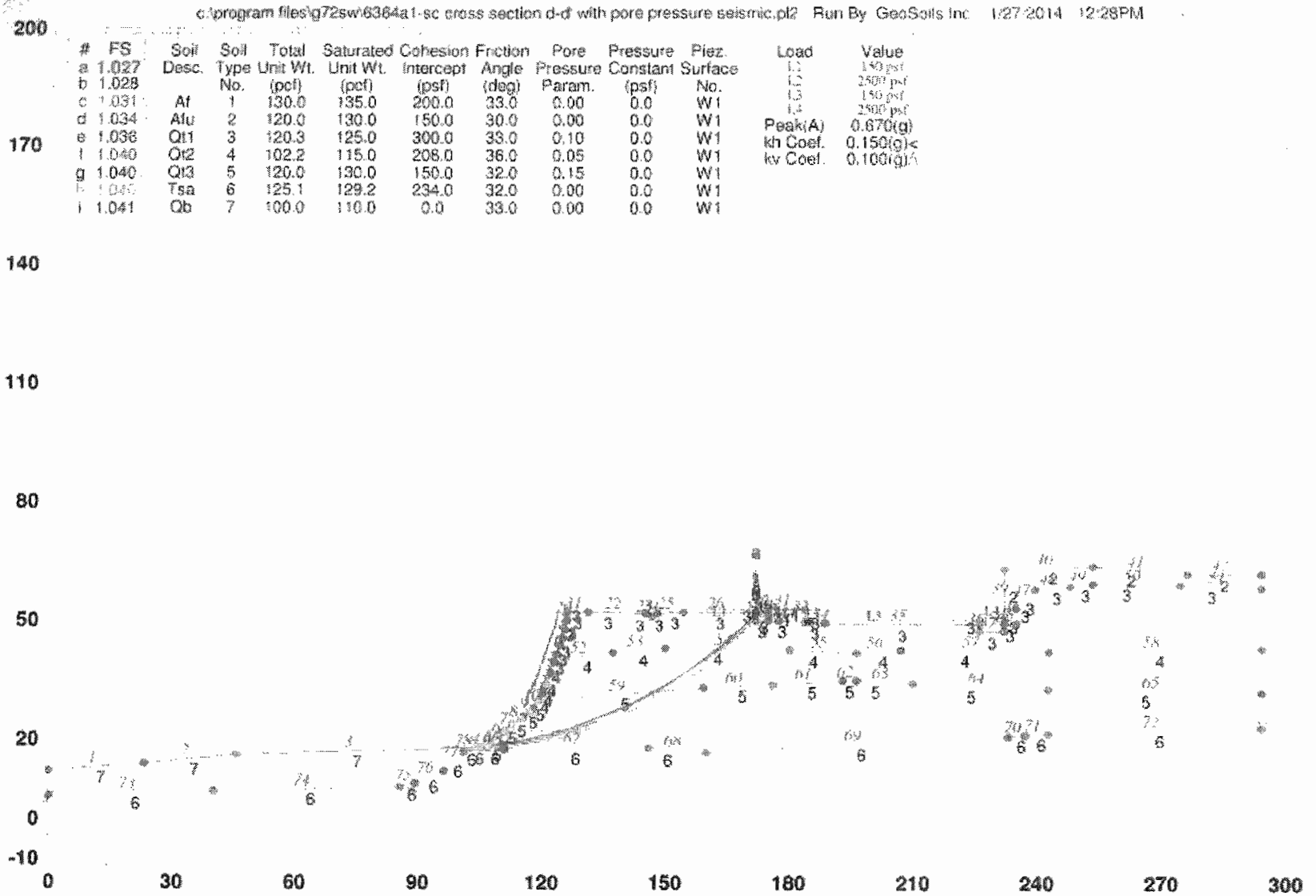
Figure 3

6364-A1-SC 5327 Carlsbad Blvd. D-D' Post-failure Pre-seawall Seismic

c:\program files\g72sw\6364a1-sc cross section d-d' with pore pressure seismic.pl2 Run By GeoSoils Inc 1/27/2014 12:28PM

#	FS	Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
a	1.027									
b	1.028									
c	1.031	Af	1	130.0	135.0	200.0	33.0	0.00	0.0	W1
d	1.034	Afu	2	120.0	130.0	150.0	30.0	0.00	0.0	W1
e	1.036	Qt1	3	120.3	125.0	300.0	33.0	0.10	0.0	W1
f	1.040	Qt2	4	102.2	115.0	208.0	36.0	0.05	0.0	W1
g	1.040	Qt3	5	120.0	130.0	150.0	32.0	0.15	0.0	W1
h	1.040	Tsa	6	125.1	129.2	234.0	32.0	0.00	0.0	W1
i	1.041	Qb	7	100.0	110.0	0.0	33.0	0.00	0.0	W1

Load	Value
L1	150 psf
L2	2500 psf
L3	150 psf
L4	2500 psf
Peak(A)	0.670(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)<



GSTABL7 v.2 FSmin=1.027

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Seismic FOS at Seaward Foundation

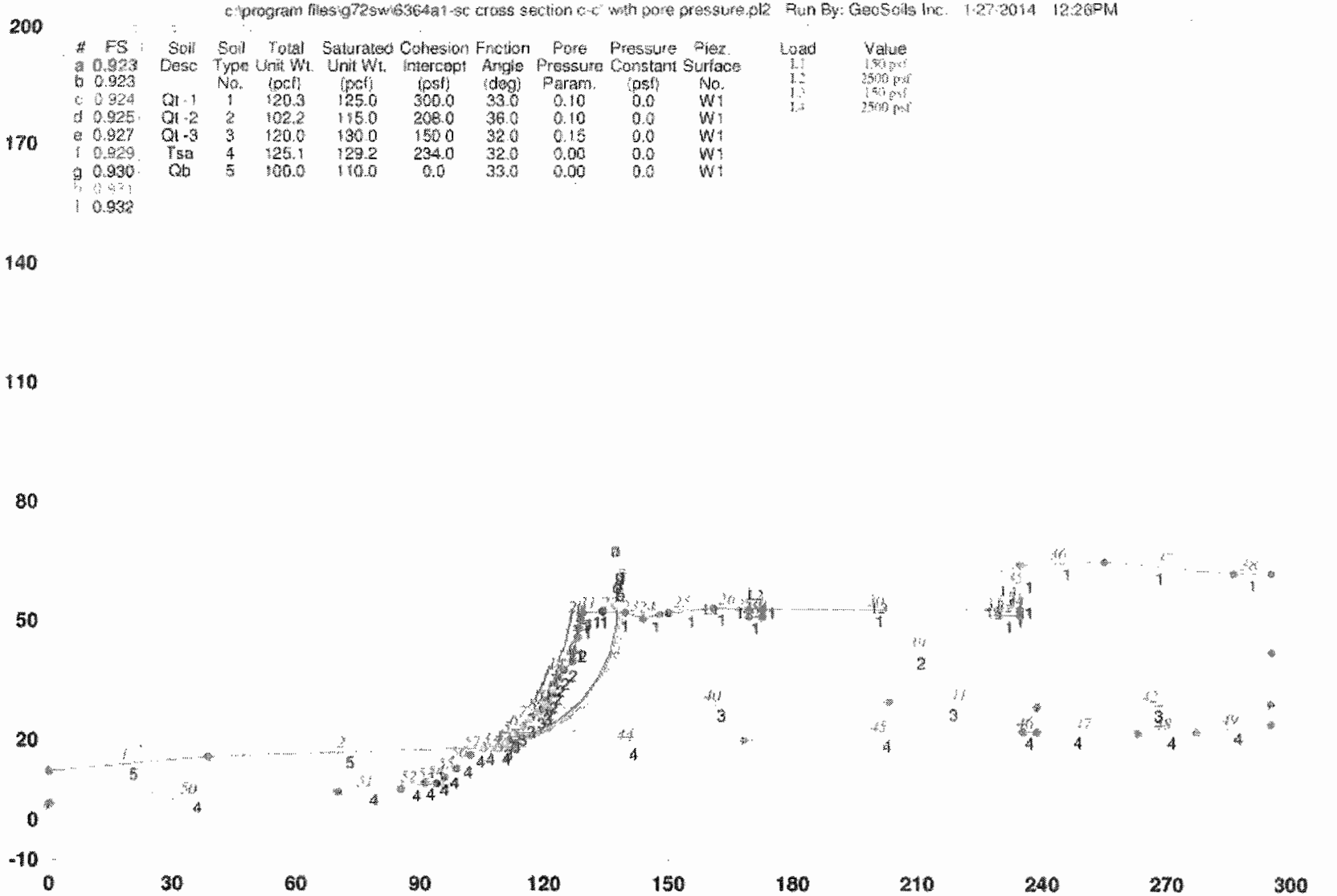
Figure 4

6364-A1-SC 5323 Carlsbad Blvd. C-C' Post-failure Pre-seawall Static

c:\program files\g72sw\6364a1-sc cross section c-c' with pore pressure.pl2 Run By: GeoSoils Inc. 1-27-2014 12:26PM

#	FS	Soil Desc	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
a	0.923									
b	0.923									
c	0.924	Qt -1	1	120.3	125.0	300.0	33.0	0.10	0.0	W1
d	0.925	Qt -2	2	102.2	115.0	208.0	36.0	0.10	0.0	W1
e	0.927	Qt -3	3	120.0	130.0	150.0	32.0	0.15	0.0	W1
f	0.929	Tsa	4	125.1	129.2	234.0	32.0	0.00	0.0	W1
g	0.930	Qb	5	100.0	110.0	0.0	33.0	0.00	0.0	W1
h	0.931									
i	0.932									

Load	Value
L1	150 psf
L2	2500 psf
L3	150 psf
L4	2500 psf



GSTABL7 v.2 FSmin=0.923

Safety Factors Are Calculated By The Modified Bishop Method

Unconstrained Minimum FOS

GSTABL7

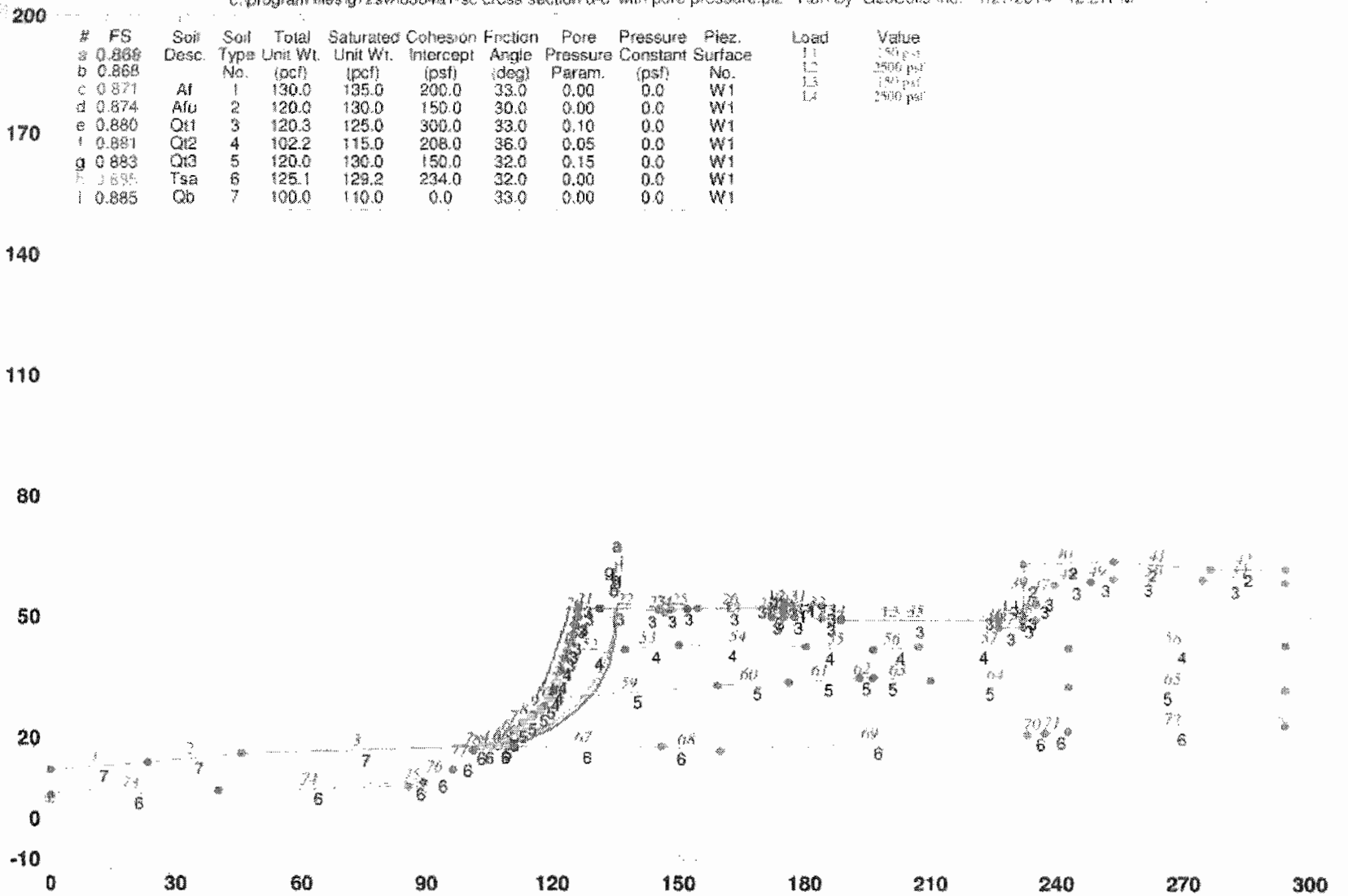
Figure 5

6364-A1-SC 5327 Carlsbad Blvd. D-D' Post-failure Pre-seawall Static

c:\program files\g72sw\6364a1-sc cross section d-d' with pore pressure.pl2 Run By GeoSoils Inc. 1/27/2014 12:27PM

#	FS	Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
a	0.868									
b	0.868									
c	0.871	Af	1	130.0	135.0	200.0	33.0	0.00	0.0	W1
d	0.874	Afu	2	120.0	130.0	150.0	30.0	0.00	0.0	W1
e	0.880	Qt1	3	120.3	125.0	300.0	33.0	0.10	0.0	W1
f	0.881	Qt2	4	102.2	115.0	208.0	36.0	0.05	0.0	W1
g	0.883	Qt3	5	120.0	130.0	150.0	32.0	0.15	0.0	W1
h	0.885	Tsa	6	125.1	129.2	234.0	32.0	0.00	0.0	W1
i	0.885	Qb	7	100.0	110.0	0.0	33.0	0.00	0.0	W1

Load	Value
L1	250 psf
L2	2500 psf
L3	180 psf
L4	2500 psf



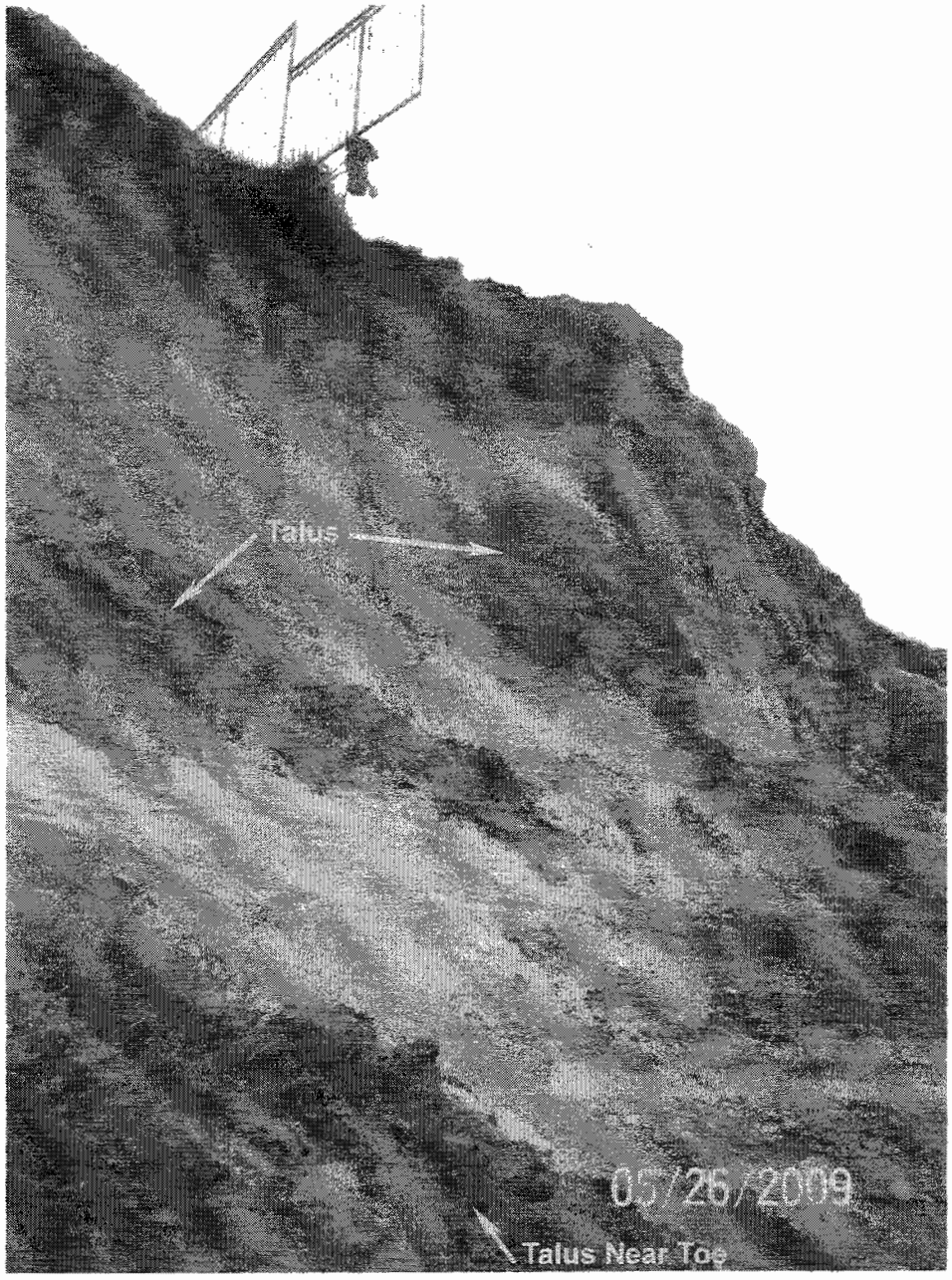
GSTABL7 v.2 FSmin=0.868

Safety Factors Are Calculated By The Modified Bishop Method

Unconstrained Minimum FOS

GSTABL7

Figure 6



GeoSoils, Inc.

SITE PHOTOGRAPH

Figure 7

DATE 01/2014	W.O. NO. 6364-A1-SC

Geotechnical · Geologic · Coastal · Environmental

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

CCC (2003)

STATE OF CALIFORNIA – THE RESOURCES AGENCY GRAY DAVIS, GOVERNOR

CALIFORNIA COASTAL COMMISSION
45 FREMONT, SUITE 2000
SAN FRANCISCO, CA 94105-2219
VOICE AND TDD (415) 904-5200
FAX (415) 904-5400

W11.5

MEMORANDUM

Date: 16 January 2003

To: Commissioners and Interested Parties

From: Mark Johnsson, Staff Geologist

Subject: **Establishing development setbacks from coastal bluffs**

STAFF NOTE

Consistency with section 30253 of the Coastal Act requires that:

> New development shall:
>
> (1) Minimize risks to life and property in areas of high geologic, flood, and fire hazard.
>
> (2) Assure stability and structural integrity, and neither create nor contribute significantly to erosion, geologic instability, or destruction of the site or surrounding area or in any way require the construction of protective devices that would substantially alter natural landforms along bluffs and cliffs.
>
> ...

This section requires that new development be located such that it will not be subject to erosion or stability hazard over the course of its design life. Further, the last clause requires the finding that no seawall, revetment, jetty, groin, retaining wall, or other shoreline protective structure, inasmuch as such a structure would substantially alter natural landforms along bluffs and cliffs, will be needed to protect the development over the course of its design life. The Commission has found on many occasions that siting new development away from eroding bluffs is the preferred means of assuring consistency with this section, and the establishment of bluff-top setbacks for new development is an integral part of most local coastal programs. Further, the State's draft Policy on Coastal Erosion Planning and Response states that avoidance of geologic hazards, such as eroding coastal bluffs, should be the primary means of safeguarding new development.

Accordingly, the determination of what constitutes an adequate setback is a critical component of the analysis of proposals for new development.

Because coastal bluffs are dynamic, evolving landforms, establishing appropriate development setbacks from coastal bluffs is far more challenging than it is for manufactured or natural slopes not subject to erosion at the base of the slope. The mechanisms of coastal bluff retreat are complex, but can be grouped into two broad categories. Bluff retreat may occur suddenly and catastrophically through slope failure involving the entire bluff, or more gradually through grain-by-grain erosion by marine, subaerial, and ground water processes. For both processes, the setback must be adequate to assure safety over the design life of the development.

In an effort to clarify the analytical procedures undertaken by Coastal Commission staff in evaluating proposed development setbacks, the Commission's staff geologist made two presentations at the *California and the World Ocean '02* conference held in Santa Barbara in October 2002. These presentations were combined into a single manuscript to be published in the proceedings volume for that Conference, which is attached to this staff report.

In order to bring these procedures before the Commission, and to further the exposure of them to the public, the staff geologist will brief the Commission on this methodology at the February 2003 hearing. This methodology does not represent a formal policy or position of the Coastal Commission. In fact, there may be other appropriate methodologies to establish development setbacks, and the Commission has the discretion to base a decision on any method that it finds technically and legally valid. Further, as new techniques and information become available, these methodologies may change. Nevertheless, the type of analysis outlined here represents the current analytical process carried out by Coastal Commission staff in evaluating proposals for new development on the California coast, and in recommending action upon those proposals to the Commission. The Commission then makes its decisions on a case-by-case basis, based upon the site-specific evidence related to the particular development proposal.

Attachment: Preprint of manuscript entitled "Establishing development setbacks from coastal bluffs," by Mark J. Johnsson, to appear in *Proceedings, California and the World Ocean, '02*, Orville Magoon, ed., 21 p.

Johnsson, Mark J., *in press*, Establishing Development Setbacks from Coastal Bluffs, *in* Magoon, Orville (ed.) *Proceedings, California and the World Ocean '02.*

Establishing Development Setbacks from Coastal Bluffs

Mark J. Johnsson[1]

Abstract

Responsible development, and California law, requires that coastal development be sited a sufficient distance landward of coastal bluffs that it will neither be endangered by erosion nor lead to the construction of protective coastal armoring. In order to assure that this is the case, a development setback line must be established that places the proposed structures a sufficient distance from unstable or marginally stable bluffs to assure their safety, and that takes into account bluff retreat over the life of the structures, thus assuring the stability of the structures over their design life. The goal is to assure that by the time the bluff retreats sufficiently to threaten the development, the structures themselves are obsolete. Replacement development can then be appropriately sited behind a new setback line. Uncertainty in the analysis should be considered, as should potential changes in the rate of bluff retreat and in slope stability. The deterministic approach presented here is based on established geologic and engineering principals, and similar approaches have been used to establish development setbacks from slope edges throughout the world for some time. Alternative approaches based on probabilistic methods may allow, however, for better quantification of uncertainties in the analysis. Although probabilistic coastal hazard assessment is in its infancy and data needs are large, the approach shows great promise. Developing probabilistic methods for establishing development setbacks should be a goal for future coastal zone management in California.

Introduction

In an era of sea-level rise such as has persisted on Earth for the past ~20,000 years (Curray 1965; Emery and Garrison 1967; Milliman and Emery 1968), the landward recession of coastal bluffs is an inevitable natural process wherever tectonic or isostatic uplift rates are lower than the rate of sea-level rise. New structures should be sited a sufficient distance landward of coastal bluffs that they will neither be endangered by erosion nor require the construction of coastal armoring to protect them from erosion over their design life. Because coastal bluffs are dynamic, evolving landforms, establishing responsible development setbacks from coastal bluffs is far more challenging than it is for manufactured or natural slopes not subject to erosion at the base of slope. Although internationally agreed-upon methods for establishing setbacks from static slopes have been developed, and codified in the International Building Code, no such consensus has emerged with respect to setbacks from dynamic slopes such as coastal bluffs. This paper presents a methodology for establishing such setbacks given the types of data generally available through relatively inexpensive geologic studies.

Relatively little work has been undertaken towards developing rational methodologies for establishing development setbacks from bluffs and cliffs. Coastal development setbacks have generally focused primarily on beach erosion, rather than on coastal bluff recession (*e.g.*, Healy 2002). Generally, the approach has been to simply

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extrapolate historic long-term erosion rates into the future, and establish setbacks at a particular predicted future shoreline position. This approach does not work well for shorelines with coastal bluffs, where the setback also must consider the possibility of bluff collapse (see Priest 1999 for a discussion of these issues). Komar and others (2002) presented a methodology for establishing setbacks for use on coasts where the principal hazards are wave runup and storm surge. They showed how their method could be extended to use on coasts with sea cliffs by determining the average number of hours that a sea cliff would be subject to wave attack. Their method does not, however, include a quantitative assessment of bluff stability. Given the significance of the coastal erosion threat in California, where public safety, financial investments, and environmental resources are at stake, and given the call for action urged by such recent national studies as the Heinz Center's FEMA-sponsored studies (The Heinz Center 2000a; 2000b), it is critical that a rational method be established for establishing development setbacks on coastal bluff tops.

The California Coastal Act (California Public Resource Code Sections 30000 *et seq.*) regulates coastal development in California. Section 30253 states, in part, that:

> New development shall:
>
> (1) Minimize risks to life and property in areas of high geologic, flood, and fire hazard.
>
> (2) Assure stability and structural integrity, and neither create nor contribute significantly to erosion, geologic instability, or destruction of the site or surrounding area or in any way require the construction of protective devices that would substantially alter natural landforms along bluffs and cliffs.
>
> ...

This law requires that new development be sited in such a way that it will not be subject to erosion or stability hazard over the course of its design life. Further, the last clause requires the finding that no seawall, revetment, jetty, groin, retaining wall, or other shoreline protective structure will be needed to protect the development over the course of its design life.

The principal challenge in meeting these requirements is predicting the amount and timing of coastal erosion to be expected at a particular site. The landward retreat of coastal bluffs is far from uniform in space or time (Komar 2000). Marine erosion tends to be concentrated at points and headlands due to wave refraction, occurs more quickly in weak rocks, and may vary along a coastline as these and other factors vary (Honeycutt et al. 2002). Further, coastal bluff retreat tends to be temporally episodic due to a variety of external and internal factors.

The mechanisms of coastal bluff retreat are complex (Emery and Kuhn 1982; Sunamura 1983; Vallejo 2002), but can be grouped into two broad categories. Bluff retreat may occur suddenly and catastrophically through slope failure involving the entire bluff, or more gradually through grain-by-grain erosion by marine, subaerial, and ground water processes. The distinction between the two categories may be blurred in

some cases—"grains" may consist of relatively large blocks of rock or shallow slumps, for example. Nevertheless, in establishing structural setbacks it is important to evaluate the susceptibility of the bluff to both catastrophic collapse and to more gradual erosion and retreat.

For both slope stability and long-term bluff retreat by "grain-by-grain" erosion, the setback must be adequate to assure safety over the design life of the development. For this reason, it is necessary to specify the design life of the structure. Many Local Coastal Programs (the implementation of the California Coastal Act at the local government level) specify a particular value, although the Coastal Act itself does not. The most commonly assumed design lives for new development range from 50 to 100 years; the most common value is 75 years. The reasoning behind establishing a setback based on the design life is that by the time the bluff retreats sufficiently to threaten the structure, the structure is obsolete and is ready to be demolished for reasons other than encroaching erosion. Replacement development can then be appropriately sited at a new setback, appropriate for conditions at the time of its construction. This process may be thwarted by limitations imposed by parcel size, and Constitutional takings issues may complicate land use decisions. Nevertheless, the only alternative to an armored coast—with all of its attendant impacts—is to continually site, and reposition, development in harmony with coastal erosion as it inevitably moves the shoreline landward.

What follows is the methodology employed by the staff of the California Coastal Commission in evaluating setbacks for bluff top development. I would suggest that this methodology is useful on other coasts with coastal bluffs, as well. This methodology does not represent a formal policy or position of the Coastal Commission. In fact, there may be other appropriate methodologies to establish development setbacks, and the Commission has the discretion to base a decision on any method that it finds technically and legally valid. Any such alternative methods should, however, be at least as protective of coastal zone resources as those outlined here. Further, as new techniques and information become available, these methodologies may change. Nevertheless, the type of analysis outlined here represents the current analytical process carried out by Coastal Commission staff in evaluating proposals for new development on the California coast, and in recommending action upon those proposals to the Commission. The Commission then makes its decisions on a case-by-case basis, based upon the site-specific evidence related to the particular development proposal.

Definition of "Bluff Edge"

Development setbacks normally are measured from the upper edge of the bluff top. Accordingly, a great deal of effort often is focused on defining that "bluff edge." The bluff edge is simply the line of intersection between the steeply sloping bluff face and the flat or more gently sloping bluff top. Defining this line can be complicated, however, by the presence of irregularities in the bluff edge, a rounded or

stepped bluff edge, a sloping bluff top, or previous grading or development near the bluff edge. Accordingly, a set of standards for defining the bluff edge is necessary.

Under the California Coastal Act, the bluff edge is defined as:

> ... the upper termination of a bluff, cliff, or seacliff. In cases where 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 bluff line or edge shall be defined as that point nearest the cliff beyond which the downward gradient of the 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..." (California Code of Regulations, Title 14, §13577 (h) (2).

This definition is largely qualitative, and the interpretation of the topographic profile to yield a bluff edge determination at any given coastal bluff may be subject to various interpretations. Accordingly, it may be useful to use more quantitative means to define "bluff edge." One approach, adopted, for example, by the City of Laguna Beach, is to define the bluff edge as that point at which the coastal bluff attains a certain specified steepness. This steepness is equivalent to the first derivative of the topographic profile. Such a definition may, however, be inconsistent with the legal definition above. Further, ambiguous results may be obtained when the upper portion of the bluff fluctuates around the specified steepness value. Better results may be obtained by finding the point at which the second derivative, the rate of change in steepness, of the topographic profile increases sharply. This approach may be amenable to computer analysis, although such analysis is rarely employed.

The position of the bluff edge may be changed by a variety of processes, natural and anthropogenic. Most obvious is the landward retreat of the bluff edge through coastal erosion. A bluff edge also may move seaward, through tectonic processes, but such movement is rare and usually small on human time scales. More significant is the anthropogenic modification of the bluff edge by grading or the construction of structures. A landward shift of the bluff edge commonly occurs through cutting into and removing natural materials during grading operations or the construction of seawalls. Conversely, placing artificial fill on or near the bluff edge generally does not alter the position of the natural bluff edge; the natural bluff edge still exists, buried beneath fill, and the natural bluff edge is used for purposes of defining development setbacks.

Slope Stability

Once the bluff edge is located, the first aspect to consider in establishing development setbacks from the bluff edge is to determine whether the existing coastal bluff meets minimum requirements for slope stability. If the answer to this question is "yes," then no setback is necessary for slope stability considerations. If the answer is "no," then the distance from the bluff edge to a position where sufficient stability exists to assure safety must be found. In other words, we must determine how far back from the unstable or marginally slope must development be sited to assure its safety.

We are guided in this analysis by the industry-accepted standards for artificial slopes (codified in many local grading ordinances), which require that a particular minimum "factor of safety" against landsliding be attained. A more difficult situation is the case of overhanging or notched coastal bluffs, or bluffs undermined by sea caves.

Landslides. Assessing the stability of slopes against landsliding is undertaken through a quantitative slope stability analysis. In such an analysis, the forces resisting a potential landslide are first determined. These are essentially the strength of the rocks or soils making up the bluff. Next, the forces driving a potential landslide are determined. These forces are the weight of the rocks as projected along a potential slide surface. The resisting forces are divided by the driving forces to determine the "factor of safety." A value below 1.0 is theoretically impossible, as the slope would have failed already. A value of 1.0 indicates that failure is imminent. Factors of safety at increasing values above 1.0 lend increasing confidence in the stability of the slope. The industry-standard for new development is a factor of safety of 1.5, and many b-cal grading ordinances in California and elsewhere (including the County of Los Angeles, and the Cities of Irvine, Malibu, and Saratoga, among others) require that artificial slopes meet this factor of safety.

A slope stability analysis is performed by testing hundreds of potential sliding surfaces. The surface with the minimum factor of safety will be the one on which failure is most likely to occur. Generally, as one moves back from the top edge of a slope, the factor of safety against landsliding increases. Therefore, to establish a safe setback for slope stability from the edge of a coastal bluff, one needs to find the distance from the bluff edge at which the factor of safety is equal to 1.5.

Inherent in the calculation of a slope stability analysis is the shape (topographic profile) and geologic makeup of the coastal bluff. There are many ways to calculate the forces involved in slope stability analyses. All methods must consider such factors as rock or soil strength, variations in rock and soil strength values due to different types of materials making up the slope, anisotropy in these values, and any weak planes or surfaces that may exist in the slope (Abramson et al. 1995). More subtly, other factors that must be considered include: pore water pressure, which produces a buoyant force that reduces the resisting forces, the particular failure mechanism that is most likely (*e.g.*, a block slide mechanism vs a circular failure mechanism), and seismic forces. Seismic forces normally are considered through a separate analysis, in which a force equal to 15% of the force of gravity is added to the driving forces. Because seismic driving forces are of short duration, a factor of safety of 1.1 generally is considered adequate to assure stability during an earthquake. This type of analysis is fairly crude, and other methods for evaluating slope stability based on maximum permanent displacement experienced during earthquakes do exist, but the pseudo-static method represents the current standard of practice for most development in California (Geotechnical Group of the Los Angeles Section of the American Society of Civil Engineers 2002). Guidelines for conducting slope stability analyses for review by the California Coastal Commission are presented in Table 1.

Table 1. Guidelines for performing quantitative slope stability analyses

1) The analyses should demonstrate a factor of safety greater than or equal to 1.5 for the static condition and greater than or equal to 1.1 for the seismic condition. Seismic analyses may be performed by the pseudostatic method or by displacement methods, but in any case should demonstrate a permanent displacement of less than 50 mm.

2) Slope stability analyses should be undertaken through cross-sections modeling worst case geologic and slope gradient conditions. Analyses should include postulated failure surfaces such that both the overall stability of the slope and the stability of the surficial units is examined.

3) The effects of earthquakes on slope stability (seismic stability) may be addressed through pseudostatic slope analyses assuming a horizontal seismic coefficient of 0.15g. Alternative (displacement) methods may be useful, but should be in conformance with the guidelines published by the Geotechnical Group, American Society of Civil Engineers, Los Angeles Section (2002).

4) All slope analyses should ideally be performed using shear strength parameters (friction angle and cohesion), and unit weights determined from relatively undisturbed samples collected at the site. The choice of shear strength parameters should be supported by direct shear tests, triaxial shear test, or literature references, and should be in conformance with the guidelines published by the Geotechnical Group, American Society of Civil Engineers, Los Angeles Section (2002).

5) All slope stability analyses should be undertaken with water table or potentiometric surfaces for the highest potential ground water conditions.

6) If anisotropic conditions are assumed for any geologic unit, strike and dip of weakness planes should be provided, and shear strength parameters for each orientation should be supported by reference to pertinent direct sheer tests, triaxial shear test, or literature references.

7) When planes of weakness are oriented normal to the slope or dip into the slope, or when the strength of materials is considered homogenous, circular failure surfaces should be sought through a search routine to analyze the factor of safety along postulated critical failure surfaces. In general, methods that satisfy both force and moment equilibrium, such as Spencer's (Spencer 1967; 1973), Morgenstern-Price (Morgenstern and Price 1965), and General Limit Equilibrium (Fredlund et al. 1981; Chugh 1986) are preferred. Methods based on moment equilibrium alone, such as Simplified Bishop's Method (Bishop 1955) also are acceptable. In general, methods that solve only for force equilibrium, such as Janbu's method (Janbu 1973) are discouraged due to their sensitivity to the ratio of normal to shear forces between slices (Abramson et al. 1995).

8) If anisotropic conditions are assumed for units containing critical failure surfaces determined above, and when planes of weakness are inclined at angles ranging from nearly parallel to the slope to dipping out of slope, factors of safety for translational failure surfaces should also be calculated. The use of a block failure model should be supported by geologic evidence for anisotropy in rock or soil strength. Shear strength parameters for such weak surfaces should be supported through direct shear tests, triaxial shear test, or literature references.

Establishing a safe setback line. Once the stability of the coastal bluff has been assessed, the development setback line to assure safety from marginally stable slopes is simply the line corresponding to a factor of safety of 1.5 (static) or 1.1 (pseudostatic), whichever is further landward. In establishing this line one can either use a single cross section and specify a single distance from the bluff edge at which the factor of safety rises to 1.5 (or 1.1 for the pseudostatic case), or use several cross sections and contour the factors of safety on the bluff top. Then, by choosing the 1.5 contour (or 1.1 for the pseudostatic case, if it lies further landward), a setback line is established. The latter method generally is necessary for large or complicated sites.

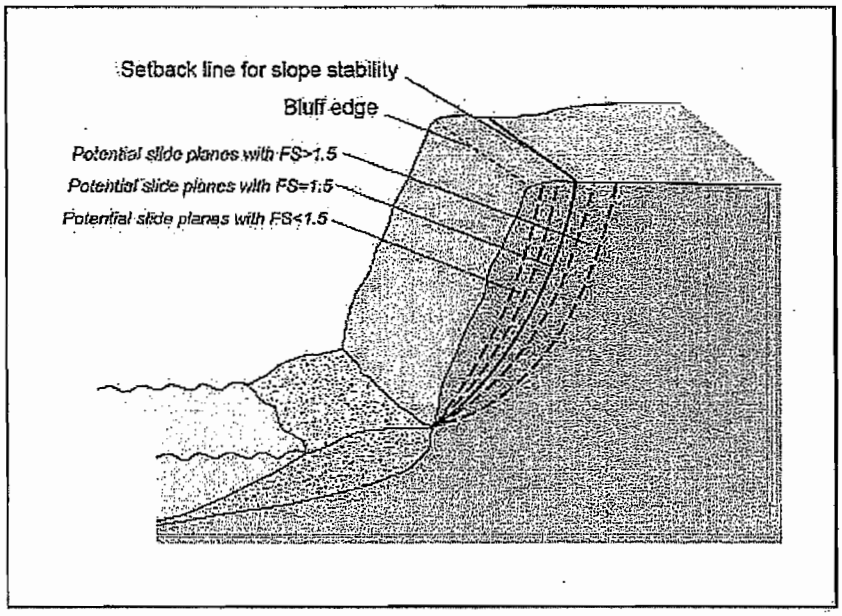

Figure 1. Establishing a development setback for slope stability. The potential slide plane possessing a defined minimum standard of stability is identified, and its intersection with the bluff edge is taken as a minimum development setback. The minimum standard for stability is usually defined as a factor of safety (FS) against sliding of 1.5 for the static case, or 1.1 for a pseudostatic (seismic) case, whichever is further landward.

Block failure of overhanging bluffs and sea caves. Assessing the factor of safety against block failure for overhanging or notched coastal bluffs, or bluffs undermined by sea caves, is far more difficult than conducting a slope stability analysis against landsliding. This is due to several factors, the most important of which are: 1) uncertainty as to the presence of local heterogeneities or planes of weakness, hidden in the bluff, that commonly control block failures, 2) difficulty in assigning shear strength values to such heterogeneities even if they can be identified, and 3) greater complexity in modeling the stress field within a bluff in terms of heterogeneities or planes of weakness as compared to a modeling a homogenous slope. The current state of the science does not allow for the calculation of a factor of safety against block failure

for such overhanging or notched coastal bluffs, or bluffs undermined by sea caves, and even makes any form of quantitative assessment of the risk of failure extremely difficult. Promise is shown in mathematical models such as that of Belov and others (1999), but translating such process-oriented models into setback methodologies has not yet been attempted.

Accordingly, establishing appropriate setbacks from overhanging or undermined coastal bluffs is problematic at best. An appropriate conservative approach is to project a vertical plane upward from the rear wall of the overhang, notch, or sea cave, and establish this as the minimum setback line. This approach has been adopted by the City of San Diego, and codified in the City's Local Coastal Program. Although it is certainly possible that failure could occur along a line inclined either seaward or landward from the rear wall of the overhang, notch, or sea cave, a vertical plane would seem to be a good default configuration to assume in the absence of more compelling evidence for another configuration. Further, vertical, bluff-parallel fractures—perhaps related to stress-relief at the free face represented by the bluff face—are a common feature of otherwise homogenous coastal bluffs. In many cases, such a plane will intersect the sloping bluff face seaward of the bluff edge, and no setback from the bluff edge would be necessary to assure stability from block collapse. In cases where the plane intersects the bluff top seaward of a setback line established for landsliding, as discussed above, no additional setback would be necessary to assure stability from block collapse. In the rather rare case, however, in which the plane intersects the bluff top landward of both the bluff edge and any setback line for landsliding, the line of intersection of the plane and the bluff top would be an appropriate setback line for slope stability considerations.

Long Term Bluff Retreat

The second aspect to be considered in the establishment of a development setback line from the edge of a coastal bluff is the issue of more gradual, or "grain by grain" erosion. In order to develop appropriate setbacks for bluff top development, we need to predict the position of the bluff edge into the future. In other words, at what distance from the bluff edge will bluff top development be safe from long-term coastal erosion?

The long-term bluff retreat rate can be defined as the average value of bluff retreat as measured over a sufficient time interval that increasing the time interval has negligible effect on the average value (a statistical basis could be applied to the term "negligible," but this is rarely done). This definition implies that the long-term bluff retreat rate is linear, an assumption that certainly is not valid over time scales of more than a few centuries, or in periods of rapid sea-level change such as the late Pleistocene/early Holocene (Curray 1965; Emery and Garrison 1967; Milliman and Emery 1968). There is some overlap between slope stability issues and long-term bluff retreat issues, in that the "grains" may be fairly large rocks, and in that shallow slump-

ing is a common mechanism for gradual bluff retreat. In addition even gradual bluff retreat tends to be highly episodic due to a host of internal and external factors.

The rate at which gradual bluff retreat occurs generally is measured by examining historic data. This is somewhat problematic in that the historic bluff retreat rate may not accurately predict the future bluff retreat rate (Watson 2002). This is a particularly issue in light of the likelihood of an acceleration in the rate of sea level rise as a result of global warming (Intergovernmental Panel on Climate Change 2001) and the resulting likely increase in bluff retreat rate (Bray and Hooke 1997; Watson 2002).

Nevertheless, historic data currently are our best indicators of future erosion at any given site. Such data may include surveys that identify the bluff edge, in which case the criteria used to identify the bluff edge must be the same in the surveys that are compared. Sufficiently detailed surveys are rare, however, and vertical aerial photography is more commonly used to assess changes in bluff position through time. The best data are those compiled photogrammetrically, whereby distortions inherent to aerial photography (due, for example, to tilting of the camera, variations in the distance from the camera to various parts of the photograph, and differences in elevation across the photograph) are corrected (see, for example, Moore 2000). Sometimes such data have been gathered as parts of specific studies of coastal bluff retreat, but more commonly they are collected as part of other work, and must be sought out for coastal erosion studies.

Coastal bluff retreat tends to be temporally episodic due to a variety of external and internal factors. External factors include tides, episodic wave events (spurred by either local or distant storms), episodic rainfall events (Kuhn 2000), El Niño-Southern Oscillation events (Griggs and Johnson 1983; Griggs 1998; Griggs and Brown 1998; Lajoie and Mathieson 1998; Storlazzi and Griggs 2000), major earthquakes (Plant and Griggs 1990; Griggs and Scholar 1997) and long-term climate change on a multidecadal to century scale (Inman and Jenkins 1999). Internal factors include the autocyclicity inherent to many bluff failure mechanisms (Leighton and Associates Inc. 1979; Hampton and Dingler 1998) and bluff response to continued toe erosion (Sunamura 1992).

Despite the episodic nature of coastal bluff retreat, it is necessary to identify the future long-term bluff retreat rate in order to establish appropriate development setbacks. The episodic nature of bluff retreat makes any calculated rate highly dependent on sampling interval. To illustrate the dependence of calculated long-term bluff retreat rates on sampling interval, it is useful to perform a sensitivity analysis from real data. Unfortunately, there are insufficient data to perform a meaningful analysis for any one site in California. Accordingly, a synthetic data set was created as part of this study.

A Synthetic Data Set. Creating and examining a synthetic data set allows for testing the effects of sampling on the determination of long-term bluff retreat rates. The long-term retreat rate is, by definition, known for the synthetic data set. Further, a

synthetic data set can be created that is both longer and more complete than any such data set available from nature. The data set considered here (available upon request from the author) was created for a hypothetical 200-year period, assigned the dates 1800-2000. Figure 2 is a graphical representation of the data set, and charts the progressive retreat of the hypothetical bluff edge through that time period. Although the data are fictitious, they roughly correlate with well-known periods of episodic erosion in coastal California, at least for the second half of the data set.

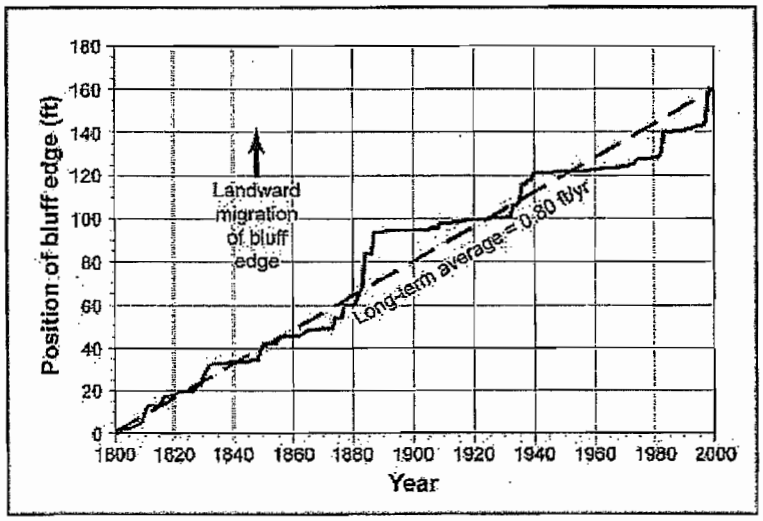

Figure 2. Plot of the position of the top edge of a hypothetical coastal bluff over time. These data represent a synthetic data set that is meant to roughly mimic typical episodic bluff retreat. Although fictitious, the data correlate well with what is know of temporal variations in erosion rate for a typical California bluff experiencing moderate erosion. The data set is far more complete than actual data available at any given site; however, making possible a sensitivity analysis of sampling interval on the calculation of the long-term bluff retreat rate.

Moving averages. A standard statistical method to smooth spikes in data is to average the data over a window of some width, while moving that window through the data set. Figure 3 shows the effect of applying this technique to the synthetic data set, using averaging windows of various widths. The first derivative of the curve representing bluff edge position through time (Figure 2) is the "instantaneous" bluff-retreat rate, and varies from 0 to 15 ft/yr for the synthetic data set (Figure 3). As the averaging window increases in width, the maximum retreat rate values decrease and the minimum values increase, effectively smoothing and broadening the "peaks" representing episodic erosion events. Depending on how the window is centered on the point representing the window average, peaks may be offset in time as well. With the widest sampling windows, peaks are essentially eliminated, and the retreat rate calculated approaches the average long-term retreat rate for the entire data set (0.80

ft/yr). Note that it is only when the window width approaches (and exceeds) 50 years in width that the calculated bluff retreat rate approaches the long-term average rate.

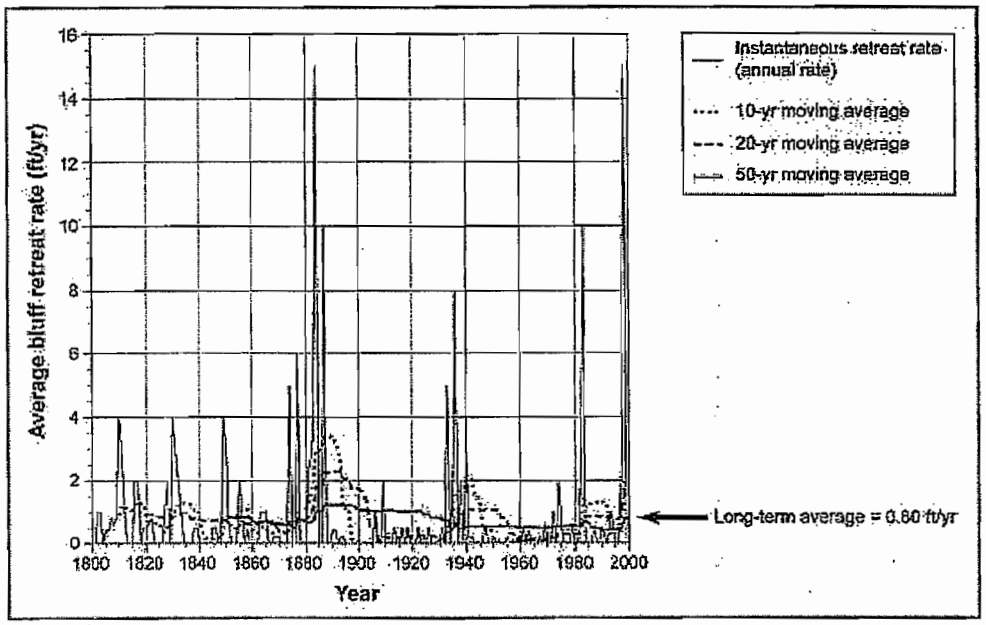

Figure 3. Average annual bluff retreat rate calculated from the synthetic data set using moving averages with various averaging window sizes. Only when data are averaged over ~50 years or more does the calculated annual bluff retreat rate approach the known long-term average for the data set.

Data gathered at intervals. Data regarding bluff edge position are almost always gathered at widely spaced intervals, corresponding to the dates of surveys or photographs. This precludes the use of a moving average technique, which depends on continuous data. Figure 4 shows the calculated bluff retreat rates at regularly spaced intervals of 10, 20, and 50 years. A wide range of values for the bluff retreat rate are obtained at the shorter sampling intervals. Although short sampling intervals give the most information on the variability of bluff retreat, the best estimate of the long-term bluff retreat rate is provided by sampling at long time intervals. Even at these long time intervals, if a statistically greater- or lesser-than-average number of "episodic events" are included in the sample, then the bluff retreat rate calculated for that interval will seriously over- or underestimate actual the long-term average bluff retreat rate.

Principal observations from the synthetic data set. A few simple generalities can be made from this limited analysis. First, instantaneous bluff retreat rates can exceed the long term average rate by a factor of many times. This is also true for data collected at short (= ~10 years for the synthetic data set) time intervals. Second, data collected at relatively short time intervals give useful information on the episodic nature of bluff retreat, but do not provide accurate estimates of long-term average

bluff retreat rates. Third, the best estimate of long-term average bluff retreat rate is obtained by sampling over long (= ~50 years for the synthetic data set) time intervals. Finally, in order to accurately estimate the long-term bluff retreat rate, a stochastically appropriate number of episodic events must be included in the sampling interval. These observations, as well as similar observations from real data, lead to the general guidelines for estimating the long-term average bluff retreat rate at a site that are presented in Table 2.

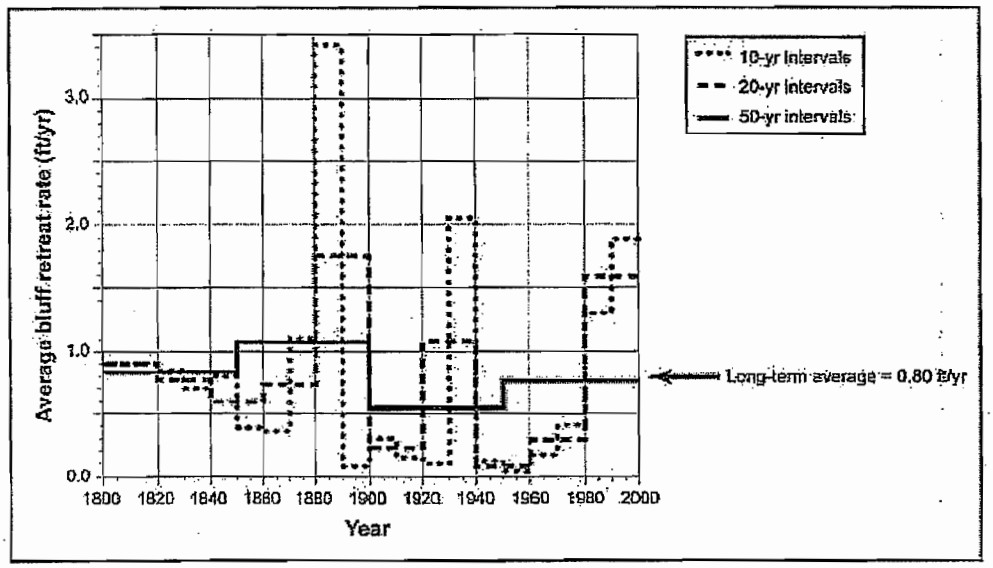

Figure 4. Average annual bluff retreat rate calculated from the synthetic data set using discrete sampling intervals of various sizes. Only when data are sampled at intervals of ~50 years or more does the calculated annual bluff retreat rate approach the known long-term average for the data set.

Establishing setbacks for long-term bluff retreat. Once an historic long-term bluff retreat rate has been estimated, establishing a setback for long-term bluff retreat rate is a simple matter of multiplying that rate, B, by the design life of the development, t. This is equivalent to predicting the position of the coastal bluff edge at the end of the design life of the structure (Figure 5).

Although this is the usual method of establishing setbacks for long-term bluff retreat in California, inherent assumptions and difficulties must be born in mind. Foremost among these is the necessity of defining the design life of the development. Because the landward retreat of an unarmored shoreline is inevitable and ongoing during a period of relative sea level rise, it is impossible to assure the safety of development from coastal erosion unless a time frame is assigned at the onset. But assigning a design life is difficult, and there is nothing in land use law that requires the abandonment of development at the end of its assigned design life.

Other problems associated with this type of analysis revolve around its inherently historic approach. There is no *a priori* reason to believe that bluff retreat rates are, or will continue to be, linear. This is especially relevant in light of expected acceleration of the historic rate of sea level rise as a result of global warming (Intergovernmental Panel on Climate Change 2001). Further, there is good evidence that erosion rates can be highly variable through time (Jones and Rogers 2002). For all of these reasons it is important to adopt a conservative approach to estimating long-term bluff retreat rates.

Table 2. Guidelines for establishing long-term bluff retreat rates

1) Determine bluff edge positions at as many times as possible, but covering a minimum of about 50 years and extending to the present. Common data sets include vertical aerial photographs, surveys that identify the bluff edge, and detailed topographic maps. These sources must be of sufficient scale or precision to locate accurately the position of the bluff edge to within a few feet.

2) If aerial photographs are used, the best results are obtained through photogrammetric methods, whereby distortions inherent to aerial photography are corrected (orthorectified). Even if photogrammetric methods are not used, the scale of the photographs must be carefully determined by comparison of the image size of known features to their actual size.

3) When comparing bluff edge positions on aerial photographs or unanchored surveys, a "shoreline reference feature" must be identified that has been static through time and is identifiable in each data set. Bluff positions throughout the area of reference can be measured relative to this feature. Common shoreline reference features are road centerlines, structures, large rock outcrops, or trees.

4) When comparing bluff edge positions on surveys, it is critical that the same criteria for the identification of the bluff edge was used in each survey. The Coastal Act definition of a bluff edge can be found in California Code of Regulations, Title 14, § 13577 (h) (2).

5) Although the short-term erosion rate for each time interval between data points provides valuable information regarding the nature of bluff retreat at the site, the long-term erosion rate should be determined from the extreme end-points of the time series examined. This time series should exceed 50 years in length, and should include both relatively quiet periods, such as the 1950's-1960's; and the more erosive subsequent time periods (especially the 1982-1983 and 1997-1998 El Niño winters).

6) In larger study areas, the bluff retreat rate should be determined at intervals along the bluff edge, paying special attention to potential differences in retreat rate between headlands and coves, and amongst areas underlain by differing geologic materials.

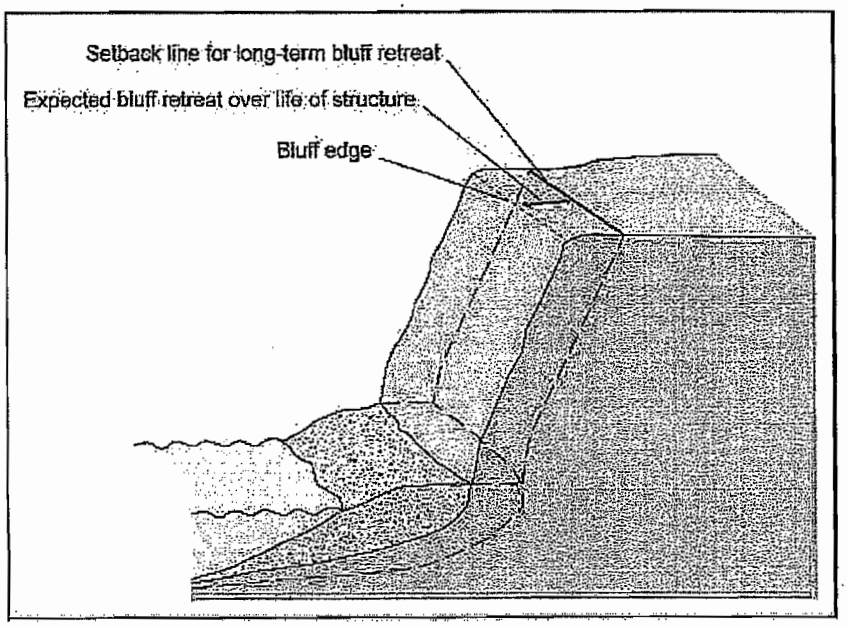

Figure 5. Establishing a development setback for long term bluff retreat. The expected bluff position at the end of the development's useful life is found by multiplying the average annual bluff retreat rate by the design life of the development; this line is taken to represent the minimum setback for long-term bluff retreat.

Uncertainty

There is a great deal of uncertainty in many parts of the analysis discussed above. The deterministic approach outlined here does not deal well with such uncertainty. Various methods have been used to build in some margin for error in establishing safe building setbacks. One approach, commonly used by geologists working in northern California, is to multiply the long-term bluff retreat rate by a factor of safety (used in a different sense than for slope stability), generally ranging from 1.5 to 4.0. More commonly, a simple "buffer" is added to the setback generated by multiplying the long-term bluff retreat rate by the design life of the structure. This buffer, generally on the order of ten feet, serves several functions: 1) it allows for uncertainty in all aspects of the analysis; 2) it allows for any future increase in bluff retreat rate due, for example, to an increase in the rate of sea level rise (Bray and Hooke 1997; Watson 2002); 3) it assures that at the end of the design life of the structure the foundations are not actually being undermined (if that were to be the case the structure would actually be imperiled well before the end of its design life); and 4) it allows access so that remedial measures, such as relocation of the structure, can be taken as erosion approaches the foundations. If a slope stability setback is required (*i.e.*, if the bluff does not meet minimum slope stability standards), that setback can do double duty as this buffer.

Summary: Defining the Total Setbacks for Bluff-Top Development

To define the total development setback, one must combine the two aspects of the setback considered above: the setback to assure safety from landsliding or block failure, and the setback for long-term bluff retreat. The resulting setback assures that minimal slope stability standards are maintained for the design life of the structure.

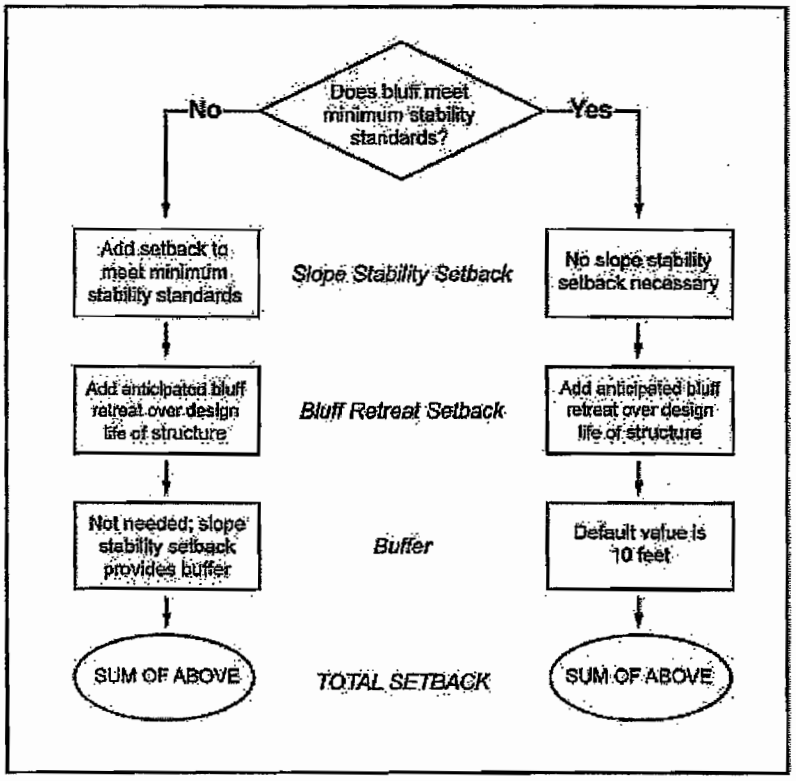

Figure 6: Flowchart for establishing bluff edge setback for development, taking into account stability of the bluff, long-term bluff retreat, and uncertainty in the analysis.

A methodology for combining these setbacks is outlined in Figure 6. First, it must be determined whether the coastal bluff meets minimum slope stability standards. Normally, this will be a factor of safety of 1.5 (static) or 1.1 (pseudostatic). If the answer to this question is "yes," then no setback is necessary to assure slope stability. If the answer is "no," then it is necessary to determine the position on the bluff top where the minimum slope stability standards are attained. This position, as measured relative to the bluff edge, is the setback necessary for slope stability determined as described above. In the case of block failure of an overhanging bluff or collapse of a sea cave, the setback necessary to assure stability from this type of collapse is equivalent to the slope stability setback. Although the current state of the science makes it impossible to quantitatively assess stability relative to this type of failure, a conservative, yet realistic, setback line is the projection of a vertical plane from the rear wall

of the overhang or sea cave on the bluff top. If the plane does not intersect the bluff top (*i.e.*, intersects the inclined bluff face seaward of the bluff edge), then no setback for this type of collapse is necessary.

The next step is to determine the expected bluff retreat over the design life of the structure, as described above. This setback is added to the slope stability setback, if any.

Finally, a buffer, generally a minimum of 10 feet, should be added to address uncertainty in the analysis, to allow for any future increase in the long-term bluff retreat rate, to assures that the foundation elements aren't actually undermined at the end of the design life of the development, and to allow access for remedial measures. A buffer is not necessary if the slope stability setback equals or exceeds about ten feet, as it can do "double duty" as both a setback to assure slope stability and a buffer for the purposes listed above.

The total setback is meant to assure that minimum slope stability standards are maintained for the design life of the development. Inherent in this analysis is the assumption that factors affecting slope stability (steepness and shape of the slope, ground water conditions, geometry of rock types exposed in the bluff) will remain constant through the design life of the development, that the future bluff-retreat rate will be linear and of comparable magnitude to the historic rate, and that the nature of erosion processes at the site will remain unchanged. All of these assumptions are potentially flawed, but in the absence of convincing evidence to the contrary, are a means of establishing reasonable development setbacks.

Towards Probabilistic Coastal Erosion Hazard Assessment

The deterministic approach presented above is based on established geologic and engineering principals, and similar approaches have been used to establish development setbacks from slope edges throughout the world for some time. However, the approach suffers from its limited ability to consider uncertainties in the analysis. Probabilistic approaches, on the other hand, inherently consider analytical uncertainties, and allow for a better definition of risk. This type of risk assessment has been routine for decades in the field of hydrology, where design basis and land use priorities are based on the magnitude of the "100-year flood," for example. Probabilistic coastal hazard assessment similarly can be used to quantify the likelihood that the bluff edge will erode to any particular point on a bluff top in a given time. Then, by establishing an acceptable level of risk (for example, a probability of <5% that the bluff edge will reach a certain point over the design life of the development) a setback line can be established that inherently includes uncertainties in the analysis. Just as the seismological community has moved away from deterministic methods towards probabilistic ones, such an approach allows for better consideration of the uncertainties in estimating future coastal erosion.

Probabilistic coastal hazard assessment is in its infancy, and no standardized methods have won acceptance—or even much discussion. The failure of coastal bluffs along Lake Michigan through landsliding has been assessed probabilistically by Chapman and others (2002), through the use of probabilistic slope stability analyses. Lee and others (2001) applied a variety of probabilistic methods to questions of coastal bluff retreat in England. Methods that they evaluated include the simulation of recession of episodically eroding cliffs through Monte Carlo techniques, the use of historical records and statistical experiments to model the behavior of cliffs affected by episodic landslide events, event-tree approaches, and the evaluation of the likelihood of the reactivation of ancient landslides. All of these techniques show promise, but the authors restricted themselves to specific cases. What is needed is the development of probabilistic methods that will work in more general cases, and combine both slope stability and long-term bluff retreat considerations. One way to approach this problem is to consider separately the two aspects of defining a development setback as outlined above.

Probabilistic slope stability analyses already are routine (Mostyn and Li 1993; Yang et al. 1993). In addition to quantifying the probability of slope failure (something not done in a deterministic slope stability analysis, which only establishes whether or not failure will occur), probabilistic slope stability analysis allows for consideration of variability or uncertainty in soil or rock strength parameters (Lumb 1970). Uncertainties in these input parameters are quantified by the standard deviation of each parameter. Then, using Monte Carlo techniques, a probability distribution for the factor of safety associated with any given failure plane is produced. From this, the probability of failure along the chosen potential failure plane can be calculated. The probability of failure is the probability that the factor of safety will be less than 1.0, and can be calculated for any given potential failure surface. By performing such analyses on a variety of potential failure surfaces intersecting different portions of the bluff top, a probability could be assigned to any position on the bluff top quantifying the likelihood that a failure will occur landward of that point.

Although not routine, several possibilities present themselves for developing probabilistic models for gradual, episodic, bluff retreat. Perhaps the simplest method of quantifying uncertainty is the application of a confidence interval to the estimate of the long-term average bluff retreat rate. Each time interval examined in estimating this rate is one sample of the mean value. For normally distributed data (or data that can be transformed to a normal distribution by, for example, a log transform), the sample standard deviation is a traditional estimate of uncertainty. There is a ~68.26% probability that the true mean value will lie within ±1 standard deviation of the sample mean. Different probabilities apply to different multiples of the standard deviation. Thus, uncertainties in the product ($B \times t$), above, can be quantified and contoured on the bluff top. For populations that cannot be shown to be normally distributed (likely the case with the small sample sizes available for bluff retreat rates), a better estimate of uncertainty may be a confidence interval based on Student's t distribution, or on nonparametric statistics.

A second approach to probabilistic assessment of coastal bluff recession is to treat annual bluff retreat in a manner analogous to river floods. Thus, the recurrence interval of a particular amount of annual bluff retreat can be calculated by the formula

$$R = \frac{N+1}{M}$$

where R is the recurrence interval, N is the number of years of record, and M is the rank of the annual bluff retreat in the total data set. For the synthetic data set considered above, there are many duplicate values due to the limited precision with which bluff retreat data are generally reported. Eliminating duplicates, and ranking the annual bluff retreat rates, recurrence intervals can be calculated. These data can be graphed in order to arrive at the expected amount of bluff retreat for any particular recurrence interval (Figure 7). The inverse of the recurrence interval is the annual probability that a given amount of bluff retreat will be exceeded. Such data may be especially valuable in assessing the risk of occurrence of an episodic event sufficient to threaten an existing structure.

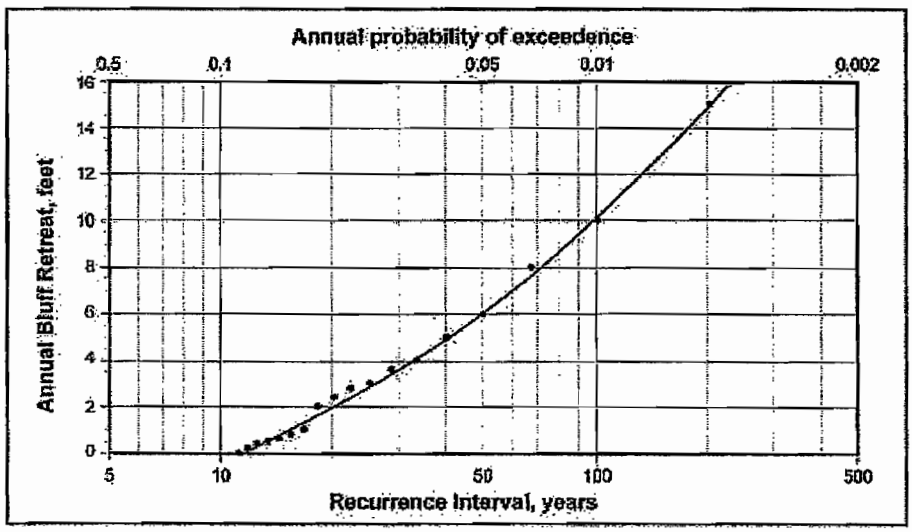

Figure 7. Recurrence interval for annual bluff retreat, calculated for the synthetic data set. The recurrence interval, calculated in a manner analogous to flood recurrence interval, gives the average time between years with a given amount of bluff retreat. The inverse of the recurrence interval is the statistical probability that a given amount of bluff retreat will occur (or be exceeded) in any given year.

The total risk to bluff-top development, which includes both long-term bluff retreat and slope failure, can be calculated by multiplying the probability of slope failure at a given position by the probability that bluff retreat will reach that point by a given time. The geotechnical and planning communities will need to establish what is an acceptable probability, or risk, that the bluff will reach a given point in order to de-

velop setback criteria. Once that probability is established, the setback line can be defined as the locus of points on the bluff top at that probability.

A prime difficulty in applying probabilistic methods to assessing coastal erosion risk will be the difficulty in acquiring sufficiently rich data sets with which to work. More effort is needed at acquiring long, precise data sets on coastal erosion in a variety of geologic conditions throughout the state.

Acknowledgements

This paper grew out of the need to clarify and make public the analytic methods of Coastal Commission staff in evaluating proposals for bluff-top development. As such, the ideas presented here grew out of numerous discussions with various members of the Commission staff, especially Lesley Ewing and others on the Shoreline Erosion Task Force, and with members of the geotechnical and coastal geology community at large. The State of California provided support to develop this paper, and the manuscript benefited from critical reviews by Ralph Faust, Sandy Goldberg, and Amy Roach.

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APPENDIX C

SHEAR TEST RESULTS

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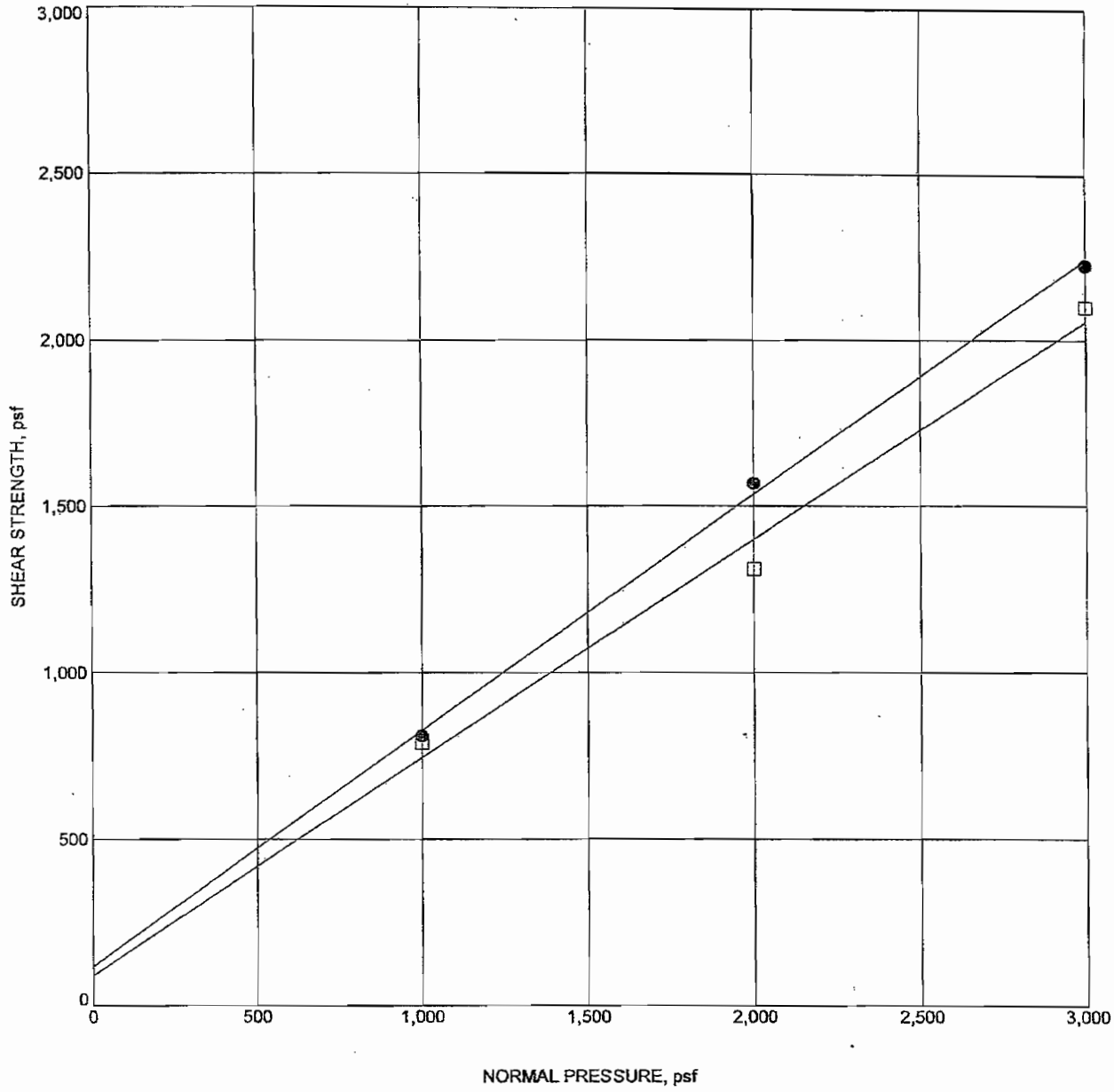


	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	TP-1	5.0		Silty Sand	Primary Shear	Undisturbed	111.8	7.6	120	35
□	TP-1	5.0			Residual Shear	Undisturbed	111.8	7.6	92	33
					Reshear Shear	Undisturbed				

Note: Sample Innundated Prior To Test

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

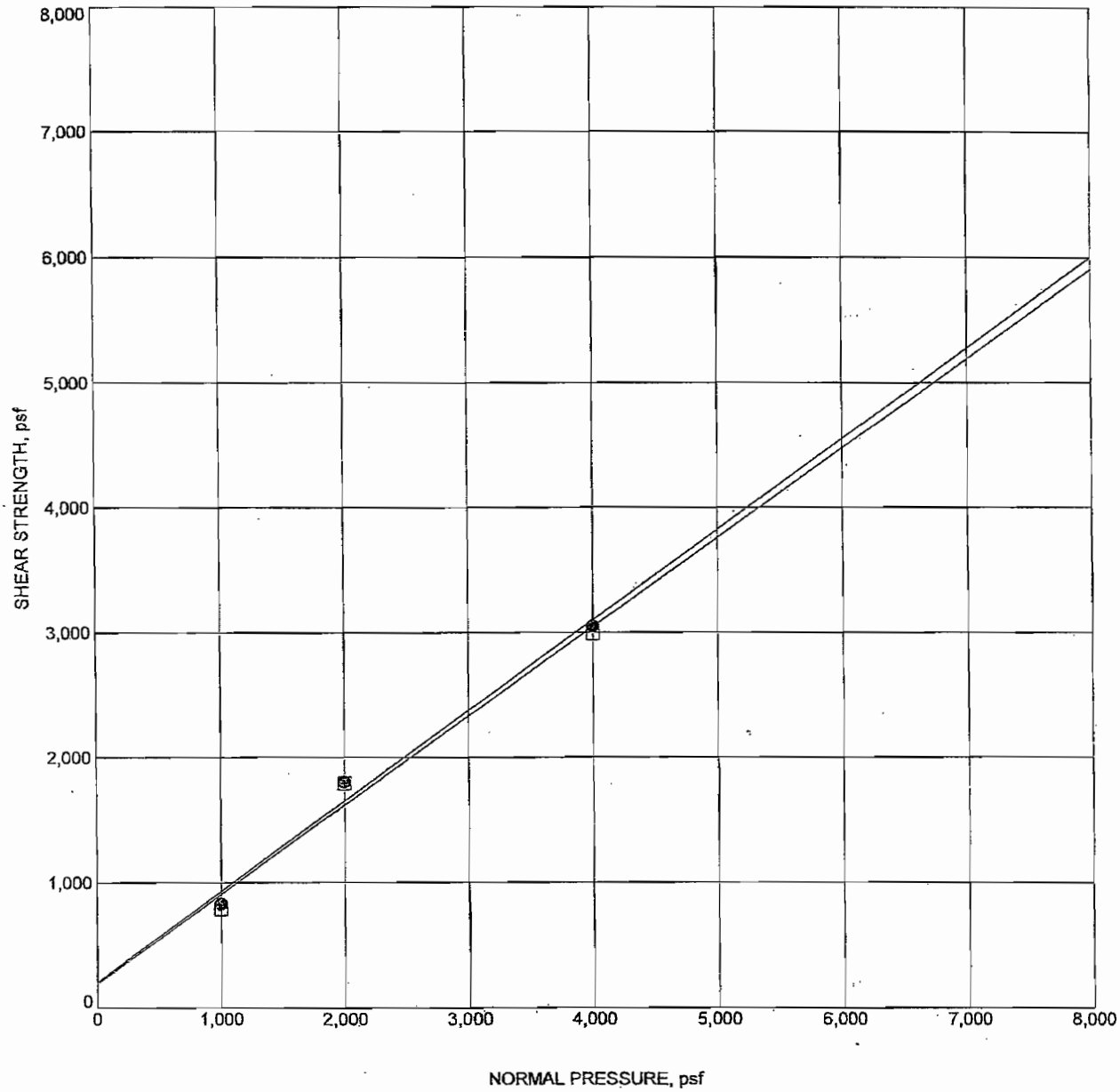
Project: KOKO PELLI BUILDERS, INC.

Number: 4646-B-SC

Date: August 2013 Plate: - 1

US_DIRECT_SHEAR_4646.GPJ US_LAB.GDT 8/1/13

Qt_2



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●	Qt	24.0		Sand w/ Silt	Primary Shear	Undisturbed	95.3	7.2	208	36
□	Qt	24.0			Residual Shear	Undisturbed	95.3	7.2	193	36
					Reshear Shear	Undisturbed				

Note: Sample Innundated Prior To Test

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

Project: GOETZ & SYLVER

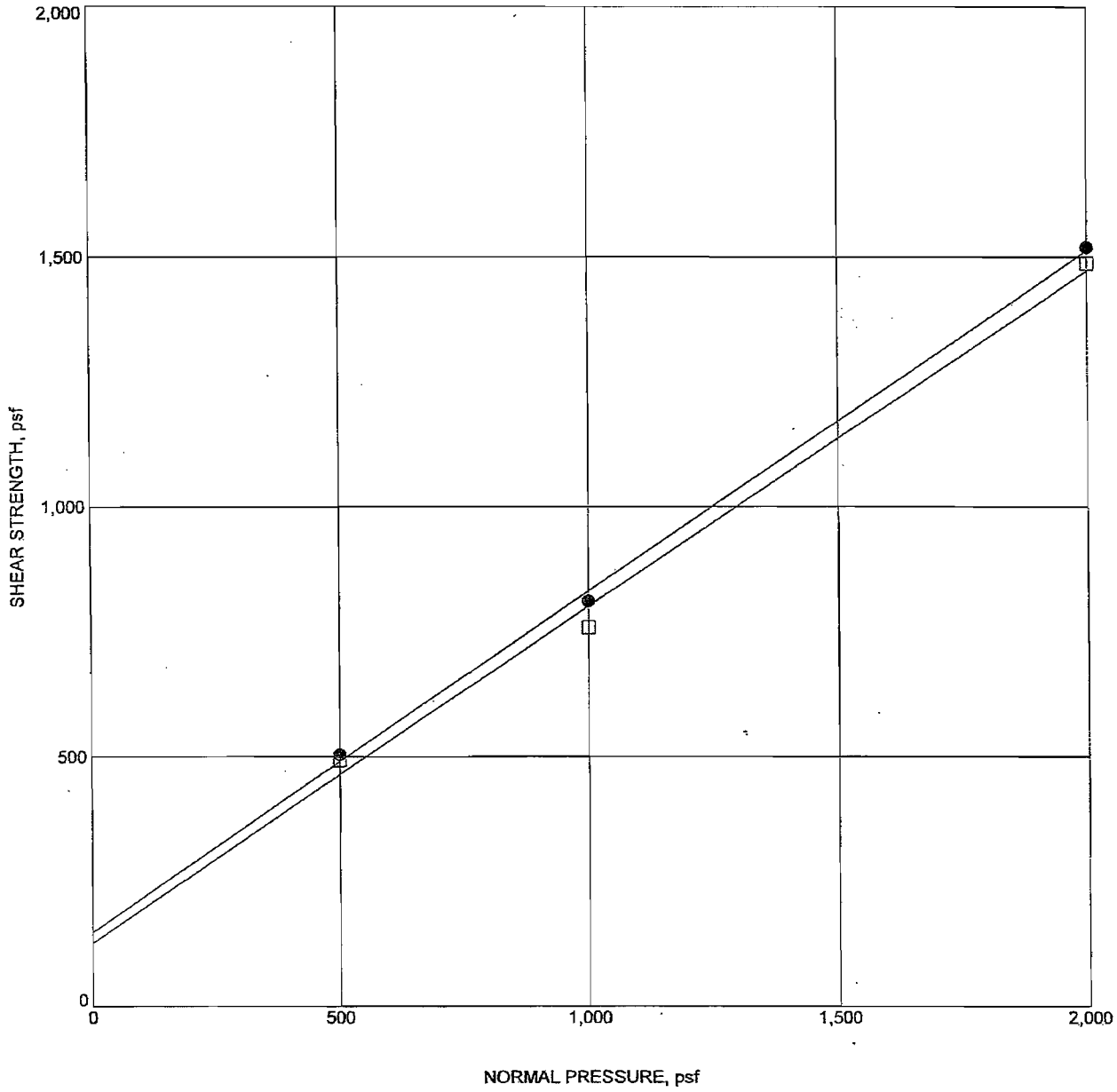
Number: S5828-B-SC

Date: August 2013

Plate: D - 1

US_DIRECT_SHEAR_5828.GPJ US_LAB.GDT 8/1/13

Qt_3



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●	Qt	0.0		Sand w/ Silt	Primary Shear	Remolded	100.0	0.3	150	34
□	Qt	0.0			Residual Shear	Remolded	100.0	0.3	128	34
					Reshear Shear	Remolded				

Note: Sample Innundated Prior To Test

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

Project: AXELSON-CORN

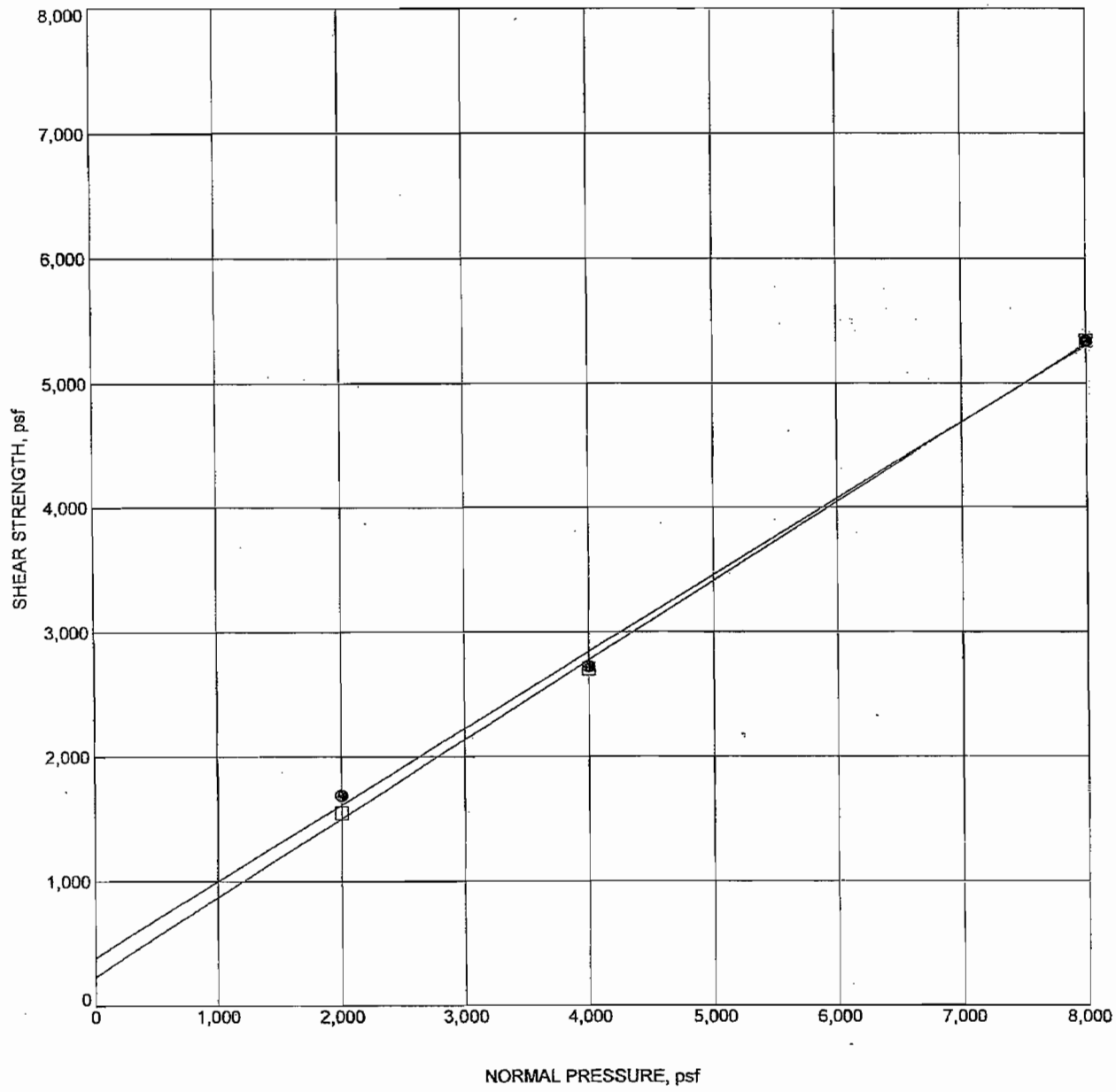
Number: 6364-A1-SC

Date: August 2013

Plate: A - 1

US_DIRECT_SHEAR_6364.GPJ US_LAB.GDT 8/1/13

Tsa



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	Tsa	10.0		Silty Sand	Primary Shear	Undisturbed	106.1	17.9	386	32
□	Tsa	10.0			Residual Shear	Undisturbed	106.1	17.9	234	32
					Reshear Shear	Undisturbed				

Note: Sample Innundated Prior To Test

GeoSoils, Inc.
5741 Palmer Way
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DIRECT SHEAR TEST

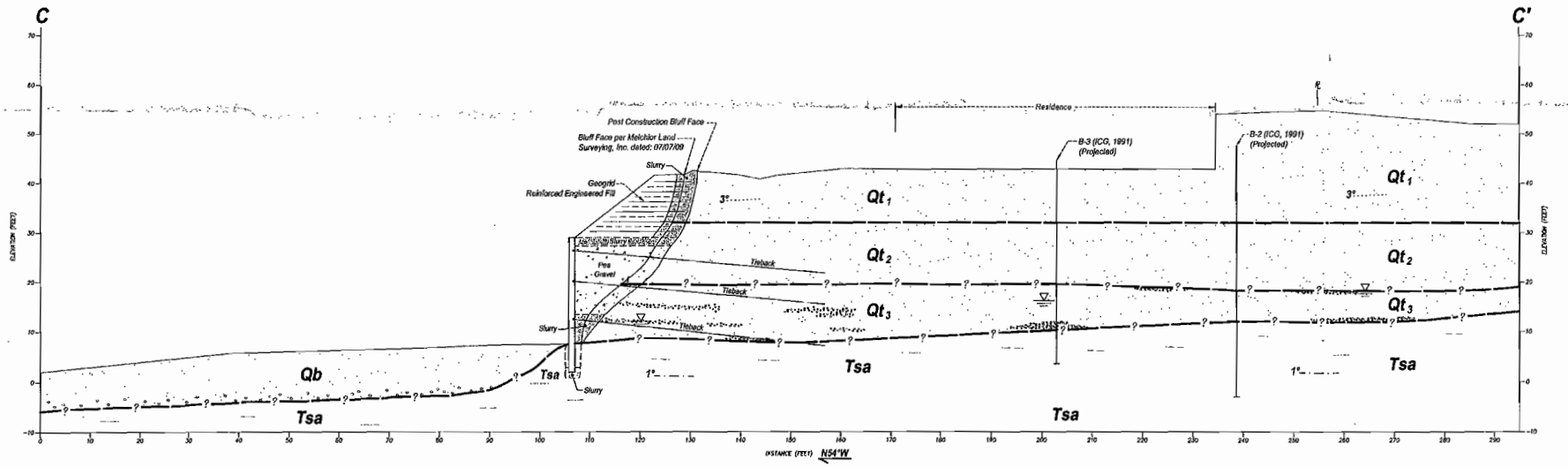
Project: GOETZ & SYLVER

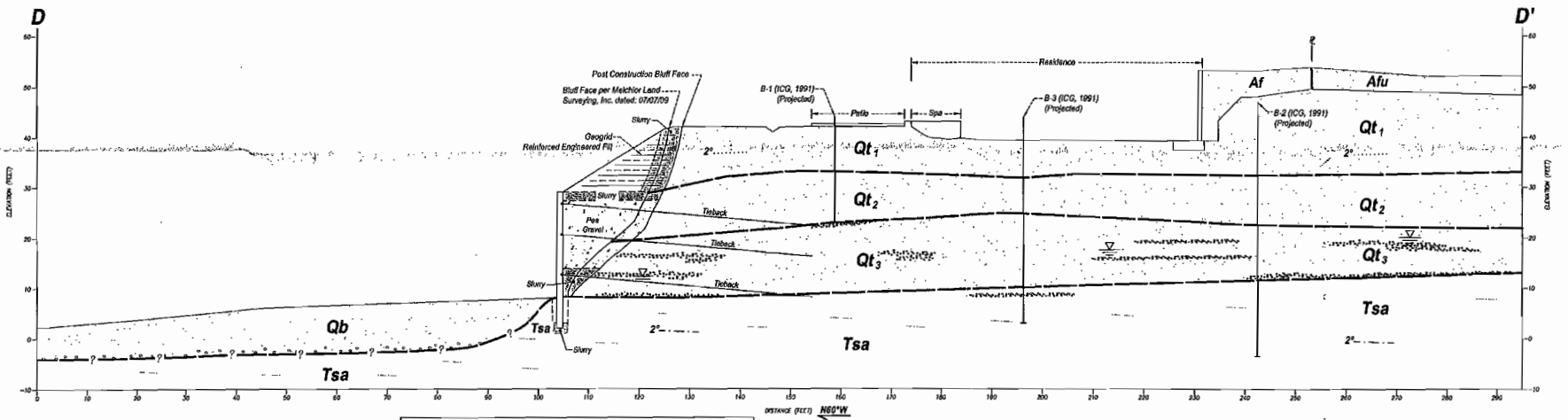
Number: S5828-B-SC

Date: August 2013

Plate: D - 2

US_DIRECT_SHEAR_5828.GPJ US_LAB.GDT 8/1/13





GSI LEGEND

Af	—	ENGINEERED FILL
Afu	—	ARTIFICIAL FILL – UNDOCUMENTED
Qb	—	QUATERNARY BEACH DEPOSITS
Qt_1	—	QUATERNARY TERRACE DEPOSITS, SUB UNIT 1
Qt_2	—	QUATERNARY TERRACE DEPOSITS, SUB UNIT 2
Qt_3	—	QUATERNARY TERRACE DEPOSITS, SUB UNIT 3
Tsa	—	TERTIARY SANTIAGO FORMATION
?—	—	APPROXIMATE LOCATION OF GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)
1°, 3°	—	APPARENT DIP OF BEDDING (CGI, 1984b)
▽	—	PERCHED GROUNDWATER

GRAPHIC SCALE

10 0 5 10 20 40

1" = 10'

ALL LOCATIONS ARE APPROXIMATE

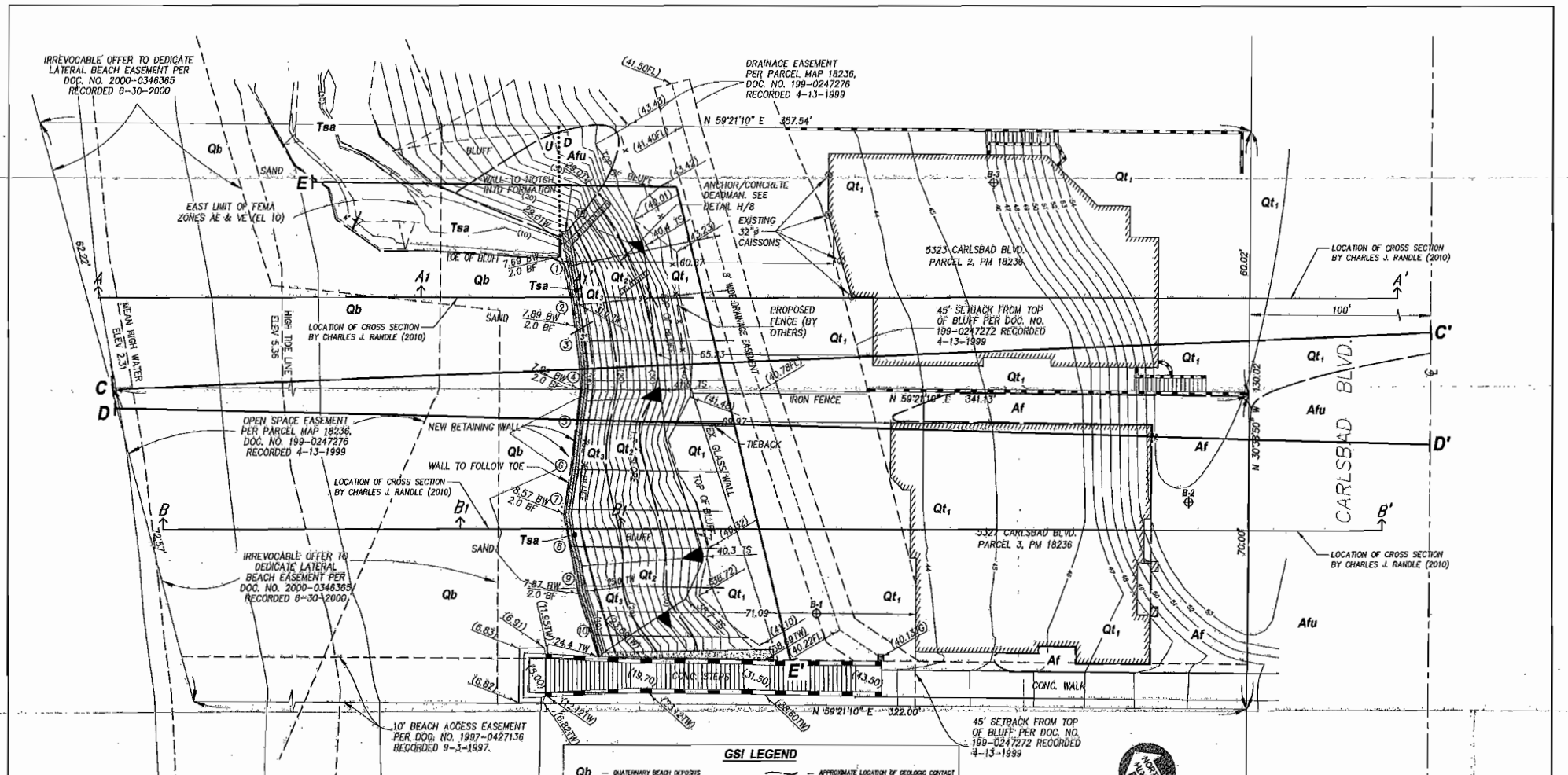
This document or site is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

GeoSoils, Inc.

GEOLOGIC CROSS SECTIONS C-C', D-D'

Plate 2

W.O. 6364-A1-SC	DATE: 01/14	SCALE: 1" = 10'



GSI LEGEND

Qb	QUATERNARY BEACH DEPOSITS
Af	ENGINEERED FILL
Afu	UNDOCUMENTED ARTIFICIAL FILL
Qt_1	QUATERNARY TERRACE DEPOSITS, SUB UNIT 1
Qt_2	QUATERNARY TERRACE DEPOSITS, SUB UNIT 2
Qt_3	QUATERNARY TERRACE DEPOSITS, SUB UNIT 3
Tsa	TERTIARY SANTIAGO FORMATION
	APPROXIMATE LOCATION OF GEOLOGIC CONTACT
U D	APPROXIMATE LOCATION OF FAULT, DOTTED WHERE CONCEALED (U, UPTHROWN SIDE; D, DOWNTHROWN SIDE) PER TREMAIN (1991)
4°	ATTITUDE OF BEDDING (CGI, 1984b)
E—E'	LOCATION OF GEOLOGIC CROSS SECTION
B-3	APPROXIMATE LOCATION OF EXPLORATORY BORING (TCG, 1991)

LEGEND:

E	EAST		BUILDING LINE
W	WEST	CB	CAISSON, 32"Ø
N	NORTH		CENTERLINE
S	SOUTH		PROPERTY LINE
(25.3)	EXISTING ELEVATION		EASEMENT LINE
FL	FLOWLINE	36.69	SPOT ELEVATIONS
TG	TOP OF GRATE	(36.69)	EXISTING SPOT ELEVATIONS
TW	TOP OF WALL		
TS	TOP OF SLOPE		EXISTING WALL
BW	BOTTOM OF WALL		PROPOSED SHOTCRETE WALL
BF	BOTTOM OF FOOTING		PROPOSED SHOTCRETE SLOPE
MHW	MEAN HIGH WATER (2.31)		
MHTW	MEAN HIGH TIDE LEVEL (5.36)		

SURVEY REFERENCES:

THESE PLANS ARE BASED ON TOPOGRAPHICAL SURVEY DRAWINGS PREPARED BY:

WILCHER LAND SURVEYING INC.
5731 PALMER WAY, SUITE C
CARLSBAD, CA. 92010
DATED: JULY 7, 2009

AND

CRENELE SURVEYING
164 S. ESCONDIDO BLVD.
ESCONDIDO, CA 92025

BASIS OF BEARINGS:

THE BASIS OF BEARING FOR THIS SURVEY IS TE LINE BETWEEN POINT 137 AND POINT 138 PER RECORD OF SURVEY NO. 17271 (E N 74°27'43" W

BENCHMARK:

POINT 137 PER RECORD OF SURVEY NO. 17271, FOUND BRASS CAP STAMPED CLS9-137 LS 8215 IN CENTER OF TURN AROUND CIRCLE, SOUTH BOUND CARLSBAD BLVD. , IN THE SOUTHWEST QUADRANT OF THE JUNCTION WITH PALOMAR AIRPORT ROAD, EL= 89.700 NGVD 29

PLANS PREPARED BY

CHARLES J. RANDLE
234 WEST JUNIPER STREET, SUITE 101
SAN DIEGO, CA 92101
(619) 246-7497

1/15/2010

DATE | INITIAL | ENGINEER OF WORK

GeoSoil, Inc.

CROSS SECTION LOCATION MAP

Plate 1

W.O. 6364-A1-SC | DATE: 01/14 | SCALE: 1" = 10'

"AS BUILT"

RCE 22098 EXP. 9-30-10 DATE

REVIEWED BY:

INSPECTOR DATE

SHEET 2 | CITY OF CARLSBAD ENGINEERING DEPARTMENT | SHEETS 6

GRADING PLANS FOR:

GOETZ SEAWALL

SITE PLAN

APPROVED: GLEN K VAN PESKI

SENIOR CIVIL ENGINEER RE 41204 EXPIRES 3/31/11 DATE

DWN BY: | CHKD BY: | RVWD BY:

PROJECT NO. CDP 09-13/SUP09-05 | DRAWING NO. 463-2A