MITIGATION OF 1998 EL NIÑO SEA CLIFF FAILURE, PACIFICA, CALIFORNIA

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MITIGATION OF 1998 EL NIÑO SEA CLIFF FAILURE, PACIFICA, CALIFORNIA

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ABSTRACT

Erosion of sea cliffs along the central California coastline has become a major concern for public and private improvements constructed in harm's way. Stability of the local sea cliff is adversely affected by young, poorly consolidated sediments comprising the bluff, vulnerability to wave attack under combined high wave and high tide conditions, and elevated groundwater levels resulting in seepage at the cliff face. Coastal geologic mapping and geomorphic analysis completed by the U. S. Geological Survey, after extensive local shoreline damage from the 1982-83 El Niño event, correctly identified critical erosion prone and unstable bluff areas. However, mitigation of identified hazards is often initiated only shortly after catastrophic failures occur, when public interest is sufficiently aroused to initiate Federal/State relief efforts.

Along Esplanade Avenue within the City of Pacifica, seven homes were lost or demolished during the 1998 El Niño event because of rapid sea cliff retreat. The adjacent public road, which provides access to 21 local residential properties, was recognized by City, State and Federal governments as being vulnerable to active coastal erosion processes. The recognition of local hazards (aided by daily television footage of homes perched at the edge of a retreating cliff) resulted in funding for the design and construction of a rock revetment (seawall) intended to guard against damage to or loss of the public roadway. This paper presents a summary of geologic, geotechnical, oceanographic, and practical factors taken into consideration during the design of the subject rock reventment.

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INTRODUCTION

In February 1998, several residences on the seaward side of Esplanade Avenue were in immediate danger of collapse due to failure of the steep sea cliff (Figure 1). Parts of some of the houses had already fallen over the bluff, others were overhanging the bluff, and a few were intact near the edge of the bluff. House-site stability at the top of the bluff is adversely affected by the weak nature of the oversteepened alluvial sediments comprising the lower to middle bluff, the steepness of the bluff, groundwater seeping from the bluff face, and a 10-foot thick layer of uncemented dune sand at the top of the bluff, upon which the houses were built. In addition, rapid bluff retreat represented a threat to Esplanade Avenue; consequently it was essential to implement appropriate coastal protection measures if loss of this public road and associated utilities was to be prevented.

The northern Pacifica coastline has been undergoing progressive sea cliff retreat with a long-term average erosion rate of approximately 2 feet per year (Lajoie and Mathieson, 1998). It is not unusual for several years to pass with little noticeable erosion, only to be followed by several feet of bluff retreat within a single day. The segment of bluff along Esplanade Avenue was placed in the most unstable category by the USGS in their Coastal Stability and Critical Erosion maps as early as 1985 (Griggs and Savory, 1985; Lajoie and Mathieson, 1998). When this area was subdivided in 1949, the length of bluff-top back yards (west of the house sites) was approximately 50 feet. Figure 2 illustrates local retreat of the sea cliff between 1956 and 1998 based on aerial photographs.

Increased rates of bluff erosion in early 1998 resulted from severe winter waves, high tides, and El Niño ocean water thermal expansion effects, coupled with a diminishing natural resupply of sand to the shoreline. Some bluff segments retreated over 30 feet during two weeks in February 1998. Erosion at the base of the bluffs, and landsliding of the undermined upper parts of the bluffs were the primary processes of sea cliff retreat. In addition, increased water seepage from the bluff face softened and loosened bluff sediments, contributing to instability and retreat.

Past efforts to stabilize the bluff were largely unsuccessful. In response to rapid local bluff retreat that occurred during the 1982

Figure 1. Residences along Esplanade Avenue, Pacifica, in danger of collapse in February, 1998.

El Niño event, a homeowners group in cooperation with the City of Pacifica funded construction of a rock revetment. The lack of adequate maintenance, coupled with adverse design aspects (rock size was relatively small and apparently not keyed into bedrock materials), resulted in a short life-span for this structure. Although relatively intact sections of this revetment were still visible in 1996, by March 1998 the earlier revetment had been reduced to an irregular scattering of boulders across the narrow sandy beach (Figure 3).

SITE GEOLOGY

The site is located along the California coastline (Figure 4), approximately 1.3 miles south of Mussel Rock where the San Andreas fault enters the Pacific Ocean. Steep bluffs fronted by narrow, sandy beaches characterize this section of the coastline. Bluff height increases from approximately 70 feet along Esplanade Avenue to greater than 300 feet north of Mussel Rock. A terrace surface to the northeast of the bluff is clearly warped as a result of geologically active uplift along the tectonic plate boundary.

Conspicuous within the City of Pacifica are several broad, relatively flat-floored valleys that open to the ocean, suggesting periods of channel incision into bedrock followed by fluvial deposition. During the last major ice age, sea level was more than 100 feet lower than it is at present. Upon final melting of the large continental glaciers (initiated approximately 15,000 years ago), sea level rose, changing the base levels of coastal drainage channels. Consequent reductions in stream gradients resulted in lower flow-velocities and burial of eroded channels with alluvial deposits. Near Esplanade Avenue, subsequent tectonic uplift and coastal erosion has notched sea cliffs into these alluvial deposits. This recent geologic history has resulted in the deposition and exposure of the relatively young, poorly consolidated sediments that form the local bluffs. Periods of rapid bluff retreat can be seen as one consequence of this dynamic geologic setting.

In the area under study, “greenstone” bedrock of the Franciscan Complex lies below the base of the bluff. This rock is a relatively firm to hard altered submarine basaltic assemblage composed of pillow lavas, flows and breccias. It is overlain partially by beach sand in the tidal zone, and by an approximately 50-foot thick section of poorly lithified Quaternary alluvial fan deposits (coarse basal breccia, intervals of poorly indurated silts, and fine to medium grained sands and gravels) exposed in the lower portion of the bluff. The alluvial deposits are overlain by a clayey, dark brown soil horizon, approximately 5 feet thick. Capping the soil horizon, and extending to the current ground surface, is a 10-foot thick layer of dune sand. Figure 5 illustrates the stratigraphy of the sea cliff and position of the rock revetment.

The uncemented dune sand is very weak when unconfined, especially when it loses moisture-related interstitial tensile forces (Figure 6). House loads provide some confinement pressure and help contain this sand when it is moist. When it becomes dry, it seeks its natural angle of repose at approximately 30 degrees (or approximately 1.7 horizontal to 1 vertical). In addition to the weak dune sand, the underlying soil and alluvial deposits, although stronger than the dune sand, are prone to sloughing, and larger-scale slumping.

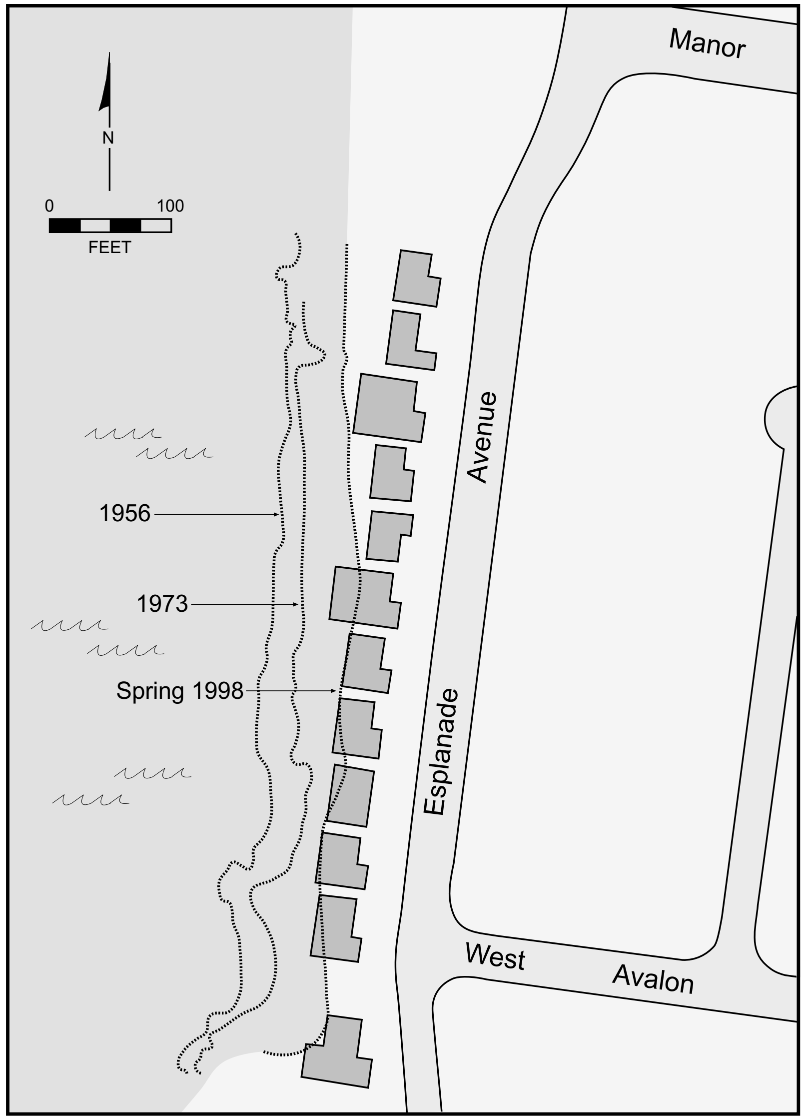

Figure 2. Top of bluff retreat from 1956 to Spring 1998 (modified from Lajoie and Mathieson, 1998)

REVETMENT DESIGN

The following section summarizes the methodology utilized for the design of a quarry stone revetment (seawall) intended to reduce the potential for future, rapid sea cliff erosion and provide protection for Esplanade Avenue and associated utilities.

Oceanographic design considerations

The recommended coastal structure design criteria reflect consideration of nearshore bathymetry, water level, wave height, maximum scour elevation, beach slope, and bedrock material properties. The design methods used in our analysis were taken from Chapter 7 of the U.S. Army Corps of Engineers Shore Protection Manual (U.S. Army, 1984). With this method, design criteria are developed for a set of recurrence interval oceanographic conditions. Both 50-year and 100-year recurrence interval oceanographic conditions were evaluated in our analysis.

The offshore bathymetry is characterized by ridges and valleys aligned perpendicular to the shoreline. An approximate nearshore slope of 0.2% was assumed. The beach slope varies across the proposed revetment site from as steep as 30% to less than 18%.

The "*design water level*" is the maximum possible still-water elevation. During storm conditions, the sea surface rises along the shoreline (super-elevation) and allows waves to break just before, or at, the revetment structure. In this study, super-elevation of the sea surface was accounted for by wave set-up (1.0 to 2.5 feet), wind set-up and inverse barometer (0.5 to 1.5 feet), wave group effects (1.0 to 2.5 feet), and El Niño thermal water expansion effects (0.5 to 1.0 feet). The 50-year recurrence interval maximum high tide elevation is +5.4 feet MSL (Mean Sea Level) which, when combined with the effects of super-elevation, yields a 50-year recurrence interval water level of +7.0 feet MSL. The 100-year recurrence interval maximum tide elevation is +5.9 feet, which could result in a maximum water level of +7.5 feet MSL.

The "*maximum scour depth*" is determined by the rate at which the bedrock, that the revetment structure is founded upon, wears down. The lower formational material at this site is greenstone bedrock of the Franciscan Complex, a firm, but erodible material. The down-wearing rate was estimated to be approximately 1 inch per year for the purposes of project design (the actual rate of abrasion would be expected to diminish at depths significantly below MSL). The elevation of the existing grade at the toe of the

Figure 3. Remnants of previous rock revetment scattered across the beach after the 1998 El Niño event.

lowest segment of the revetment is approximately 3 feet below mean sea level. Accounting for bedrock down-wearing, and using maximum still-water levels, the design water depth (i.e., elevation difference between the revetment toe and maximum still-water level) at the revetment for the 50-year recurrence interval conditions is approximately 12 feet. The design water depth for the 100-year recurrence interval is approximately 15 feet. These static submergence depths are utilized in the following section for the calculation of wave run-up.

In general, high waves in combination with high water levels locally result in erosion of beaches and wave attack at the base of the coastal bluffs (Figure 7). At this site, offshore wave heights exceeding 20 feet are not uncommon during winter storms. However, the design wave condition for a shoreline structure is generally not the largest wave, because the largest waves break offshore in water depths approximately equal to the wave's height. The largest "*design wave force*" will occur when a wave breaks directly on the shoreline structure. The largest wave that can break on the revetment is determined by the depth of water at the toe of the structure. Using the water depths defined earlier, the resulting design wave heights are 10.0 feet for the 50-year recurrence interval and 12.0 feet for the 100-year recurrence interval. Incoming wave periods vary from 9 to 20 seconds. A design period of 20 seconds was selected because this period wave would produce the highest run-up.

Wave run-up

As waves encounter a revetment, they break and the water rushes up the face of the structure. Often, wave run-up and overtopping strongly influence the design and cost of coastal projects (Weggel, 1976). "*Wave run-up*" is defined as the vertical height above the still water level to which a wave will rise on a structure of infinite height. "*Overtopping*" is the flow rate of water over the top of a revetment as a result of wave run-up. The run-up analysis is performed to determine the design height of the revetment so that no overtopping can occur. Overtopping of the structure would exacerbate erosion of the alluvial deposits comprising the middle of the bluff.

Wave run-up and overtopping for the revetment was calculated using the U.S. Army Corps of Engineers Automated Coastal Engineering System, (ACES). ACES is an interactive, computer-based design and analysis system commonly used in the field of coastal engineering. The methods to calculate run-up and overtopping implemented within this ACES application are discussed in greater detail in Chapter 7 of the *Shore Protection Manual* (U.S. Army, 1984). The run-up estimates calculated herein are corrected for the effect of onshore winds (i.e., wind direction from sea to land).

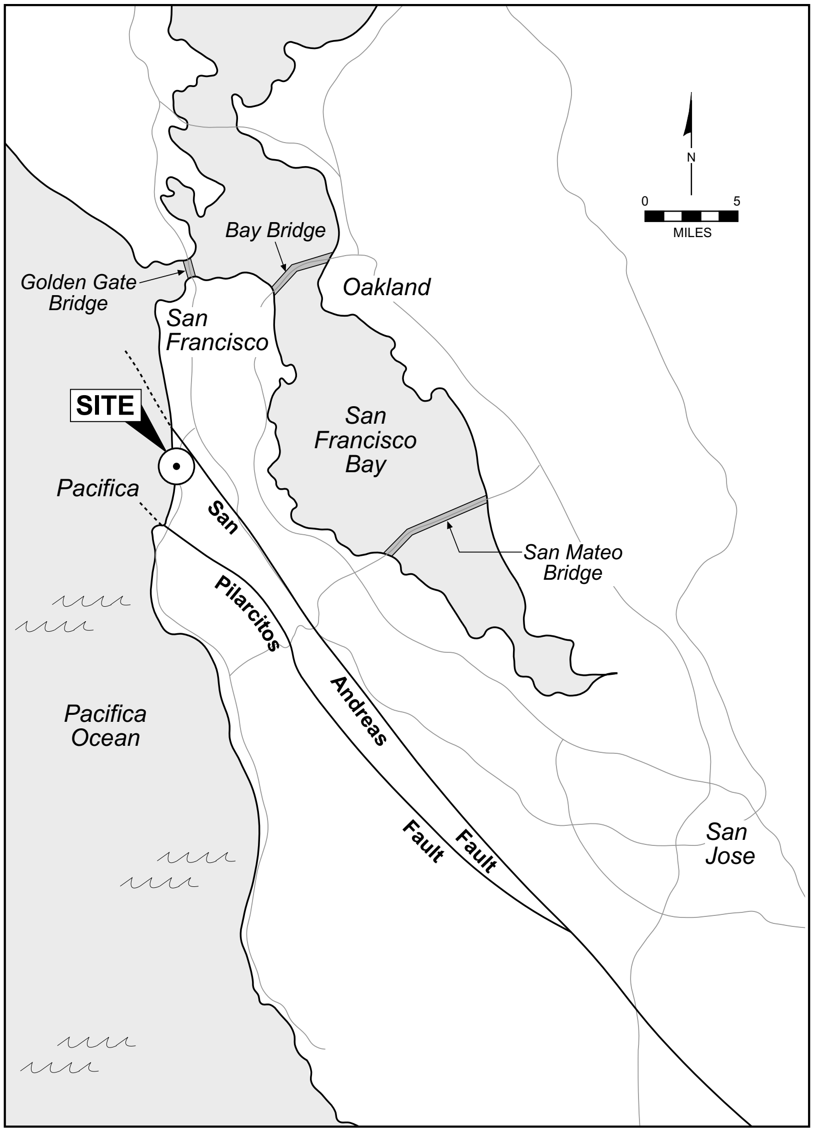

Figure 4. Location of the Esplanade Avenue site within the City of Pacifica.

The empirical expression for the monochromatic-wave overtopping rate is:

$Q = C_W (g\, Q_0{}^*\, H_0{}^3)^{1/2} [(R+F)/(R-F)]^{-0.1085/\propto}$, where:

Q = overtopping rate per unit length of structure (ft^3/sec.ft)

C_W = wind correction factor,

g = gravitational acceleration (ft/sec^2),

$Q_0{}^*$, $\propto$ = empirical coefficients (see SPM Figure 7-27),

H_0 = unrefracted deepwater wave height (ft),

R = run-up (ft),

$F = h_S - d_S$ = freeboard (ft),

h_S = height of structure (ft), and

d_S = water depth at structure (ft).

The correction for onshore winds is:

$C_w = 1 + W_f (F/R + 0.1) \sin(\theta)$, where:

$W_f = U^2/1800$

U = onshore wind speed (mph)

$F = h_S - d_S$ = freeboard (ft)

R = run-up (ft)

θ = angle of the ocean-facing revetment slope, measured from horizontal in degrees.

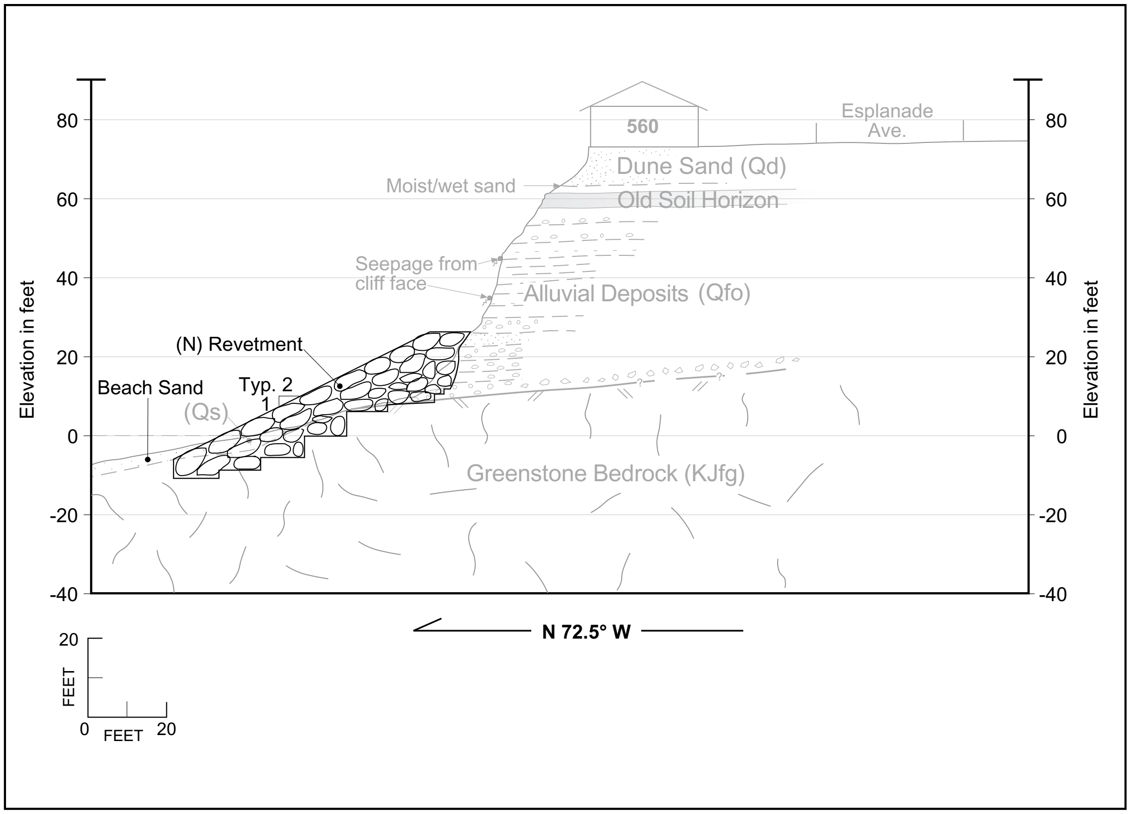

Figure 5. Stratigraphy of the sea cliff and rock revetment position near the southern terminus of Esplanade Avenue.

The severity of storm impacts to the local coast are partially dependent on the direction of wave approach and the local shoreline orientation (Fulton-Bennet and Griggs, 1987). The ACES analysis was performed on two sets of local oceanographic conditions that represent typical 50- and 100-year storms. The onshore wind speed was chosen to be 60 knots (69 mph) for each case.

The output from the ACES analysis indicates that the maximum wave run-up for the 50-year recurrence interval oceanographic conditions is approximately +17 feet MSL, and for the 100-year recurrence conditions, approximately +19 feet MSL. Based upon this analysis, the height of the revetment for a "no overtopping" condition should be a minimum of +20 feet MSL. A top of revetment elevation of +26 feet MSL was selected to buttress a lower sand lens on the bluff face, to allow for future settling of the revetment and to increase the factor-of-safety in project design. Site observations during storm conditions and engineering judgement also influenced the selection of final revetment height.

Revetment geometry and stone weight considerations

For initial design purposes, an armor stone unit weight of 165 pcf and a 50% slope for the face of the structure were selected. The primary factor in determining the design stone weight is the incident wave energy, which is proportional to the wave height. The output of the ACES analysis includes the stone weight and revetment crest width. The weight of the stone required to withstand the design wave conditions is given by the following formula:

$W = W_r H^3/[(K_D(S_r-1)^3\cot(\theta)]$, where:

W = Weight of the individual armor stone in the primary cover unit (lbs),

W_r = Unit weight of the armor stone (pcf),

H = Design wave height (ft),

S_r = Specific gravity of the armor stone relative to water ($S_r = W_r/W_W$),

W_W = The unit weight of water (pcf),

θ = Angle of the structure slope measured from horizontal in degrees, and

K_D = Stability coefficient which depends on the shape of the armor stone.

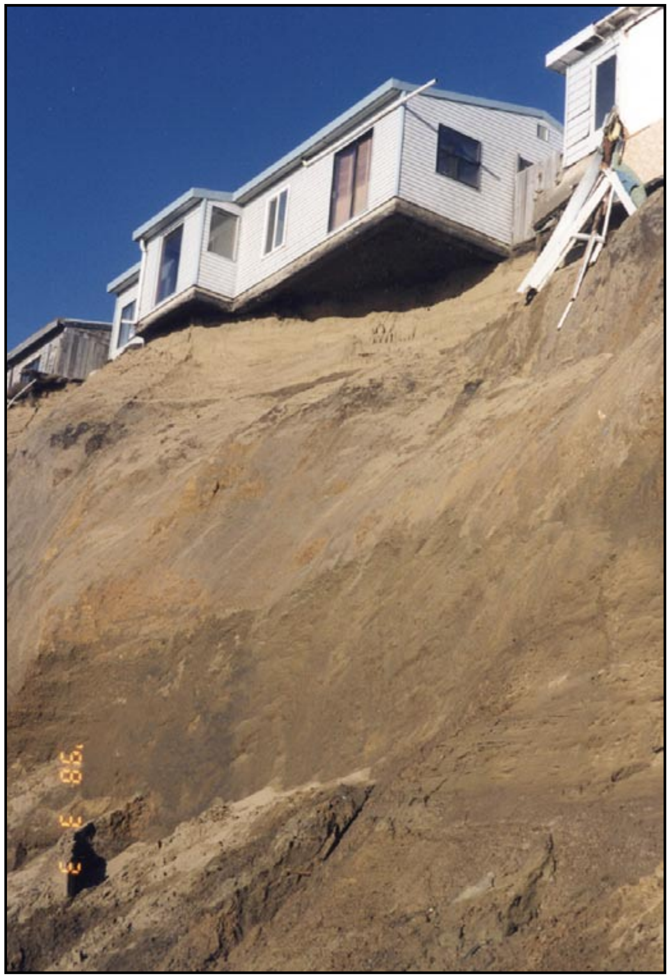

Figure 6. Loose dune sand exposed in the upper cliff face, immediately beneath the residence.

The calculated individual armor stone weight for the 50-year oceanographic conditions is approximately 5 tons. The crest width is approximately 12 feet, with approximately 80 armor stones per 1,000 square-feet for 50-year recurrence conditions. The individual armor stone weight for the 100-year wave conditions is approximately 10 tons, with a crest width of approximately 14 feet. This results in approximately 50 armor stones per 1,000 square feet. Ultimately, an armor stone size of 8 to 10 tons was selected for project design considering several years of performance observarions by the Pacifica Public Works Department, available funding, and the restricted availability of rock in the 10-ton range.

Quarry stone selection and placement

Two weight ranges of stone are generally selected for revetment construction: armor stone and core stone. The armor stone weight ranged from 8 to 10 tons, whereas the core stone weight ranged from 100 pounds to 5 tons. The smaller size fraction of the core stone was placed deepest within the revetment. All the stone material was examined to verify that the rock was free of undesirable qualities that might contribute to crumbling or breaking during handling. The armor stone consisted of select quarry rock free of open fissures and apparent planes of weakness. Ideally, armor stone should be rough and angular in shape, with the shortest principal dimension not less than one-third the longest dimension to improve interlocking qualities.

The qualitative evaluation of rock durability relates to the geological origin of the rock, as well as specific tests to evaluate rock properties important to longevity as a revetment component. Rock evaluation should include consideration of the following:

1. Examination of the petrographic make-up of the rock should be completed.

2. Evaluation should be made of the performance of the candidate rock type in other marine structures.

3. Testing to address the Standard Practice for Evaluation of Rock to be Used for Erosion Control (ASTM D 4992-94)should be conducted. Testing may include:

   a. C 88 - Test method for soundness of aggregate by use of sodium sulfate or magnesium sulfate;

Figure 7. Offshore wave height may exeed 20 feet and wave surge from shore break attacks the base of coastal bluffs during high tides.

b. C 127 - Test method for specific gravity and absorption of coarse aggregate;

c. C 294 - Descriptive nomenclature of constituents of natural mineral aggregate;

d. C 295 - Practice for petrographic examination of aggregate for concrete;

e. C 535 - Test method for resistance to degradation of large-size coarse aggregate by abrasion and impact in the Los Angeles machine; and

f. D 5313 - Test method for evaluation of durability of rock for erosion control under wetting and drying conditions.

CONSTRUCTION CHALLENGES

Suitable, large armor stones in the 8- to 10-ton size range were difficult to obtain because significant demands for large rock had been placed on local quarries during El Niño conditions. Rock samples from four quarries, with haul distances of approximately 40 to 100 miles to the site, were delivered for detailed examination. Samples included limestone, graywacke sandstone, welded volcanic tuff and a metaconglomerate. Submitted samples, other than the limestone, generally had favorable density and durability properties. Due to the scarcity of large rock, favorable rock types available from three quarries were utilized for project construction.

Trucks delivering armor stones to the site would typically accommodate only two large stones per load, with possibly room for a few smaller core stones. Because the keyway for the revetment was to extend into Franciscan bedrock located below mean sea level, stone placement was possible only during periods of low tide (Figure 9). Sufficient stone for the construction of a 100-foot segment of the revetment was delivered to a staging area high on the beach, and placement of rock into the keyway (excavated the previous day) occurred during low tide.

Rock was placed in conformance with the following guidelines, with the guiding principle that good craftmanship during stone placement is essential to structural integrity: 1) rock-to-rock contact was maximized (at least three points of contact per stone) and the voids were minimized; 2) stones that were flat in one dimension were preferred and round stones were avoided; 3) stones that had one particularly long dimension were placed with the longer dimension perpendicular to the shoreline to prevent rolling down slope; and 4) “chink” armor stones (smaller than 3 feet in their longest dimension) were not usedvthe larger armor stones.

MAINTENANCE

Any large engineered structure placed along the base of a sea cliff will interact with dynamic shoreline erosional processes. Consequently, such structures require periodic inspection and maintenance. Inspections should be performed by an engineer with experience in coastal structures. In addition, coastal structures should be inspected by the property owners after any major storm for damage caused by wave attack. When damage is observed, an engineer should be consulted to determine the nature and extent of necessary maintenance. Maintenance on a quarry stone revetment would include re-shaping the revetment to the design profile through addition or repositioning of stones. Maintenance of the revetment should be undertaken in a manner that will improve the quality of the profile, as well as the contact and orientation of the individual stones. The rehabilitation of

Figure 8. Filter fabric is being placed in the keyway and along the base of the revetment prior to stone placement.

a revetment should be supervised by a coastal engineer. The City of Pacifica has reportedly taken steps toward establishing a maintenance assessment district to ensure that funding is available for periodic upkeep of the revetment.

SUMMARY/COMMENTARY

Key aspects of revetment design included selection of adequate armor stones, keying of imported stone below beach and alluvial deposits well into firm bedrock, and selection of an appropriate revetment face-slope and height. Design parameters were based on oceanographic analysis including consideration of maximum possible still-water levels, wave run-up, the design wave force, and anticipated scour depth. Final quarry stone selection included consideration of constituent mineralogy, rock density and anticipated durability in a dynamic marine environment.

Even though the design intent of the revetment is to help protect the nearby public roadway from coastal erosion, there may be pressures to redevelop what remains of the top lots on the bluff. Landsliding along the precipitous bluffs is a significant potential hazard to adjacent residential development. One limitation for the placement of a revetment at the base of the bluff is that it will not significantly improve stability of the slope above the revetment crest (elevation of 26 feet MSL). Although dewatering measures may improve slope stability by reducing adverse groundwater seepage from the face of the bluff, the viability of the bluff lots for redevelopment will depend on the outcome of detailed geotechnical studies. In addition, revetments have design-life limitations and maintenance requirements that must be considered during redevelopment evaluations.

Engineering efforts to arrest coastal erosion processes should be viewed as temporary solutions that are often not free of collateral impacts (Griggs, Pepper and Jordan, 1992). In the case of engineered revetments or seawalls, these structures are typically constructed at locations that already have inadequate protective beach or dune buffer zones. The sand-deficient beaches may become narrower and steeper with time after the protective structure is installed. These changes may result from increased rebound energy of waves reflected off relatively hard, fixed engineered structures, and the reduction of cliff detritus

Figure 9. Keyway excavation below mean sea level required strategic construction timing with respect to tidal conditions.

descending to the beach. Consequent alteration to the beach and near-shore profiles can ultimately undermine foundation support of the protective structure. It is also possible that erosion of adjacent vulnerable coastal bluffs may result in gradual outflanking of the protective structure. With the best revetment or seawall designs, protective success over the time period of a human life span can occasionally be achieved. From a long-term geologic perspective, however, protective revetments placed within wave impact zones will eventually face inevitable consequences. Utilization of protective design alternatives in dynamic coastal zones should follow full consideration of a cost-and-benefit analysis, impacts to beaches and adjacent properties, and alternative hazard avoidance/ relocation options.

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AUTHOR PROFILE

Cotton, Shires & Associates, Inc. is a geotechnical consulting firm established in 1974 providing services from offices in Los Gatos and Carlsbad, California. We offer expertise to engineers, designers and local governments with regard to development, hazard analysis and mitigation, commercial and municipal construction and failure analysis. We also provide expert witness testimony for litigation and arbitration related to geotechnical engineering and engineering geology. Mr. Ted Sayre is a Certified Engineering Geologist with 17 years of local experience. Mr. Patrick O. Shires is the firm's Principal Geotechnical Engineer with 28 years of experience addressing complex geotechnical problems in the western United States. Mr. David W. Skelly is a Professional Engineer with expertise in Coastal Engineering who works with Cotton, Shires and Associates on special projects.

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