TIDAL EROSIONAL EFFECTS ON A BULKHEAD SYSTEM

by

Ronald M. Noble[1], M. ASCE, Wolfgang H. Roth[2], M.ASCE
and Uday K. Patil[3], M.ASCE

ABSTRACT

In the design and construction of waterfront bulkhead systems, it is essential to consider the coastal effects of tides, waves, boat wakes, currents, bottom sediment movement and bottom scour. Many improperly designed bulkhead systems experience severe loss of backfill and toe materials with the bulkhead eventually failing if it is not corrected in time. Inadequate drainage, joint connections, and/or inadequate toe protection are typically the causes of failure.

This paper describes an investigation of a bulkhead system supporting a large waterfront development in southern California which was experiencing widespread sinkhole development in the bulkhead's backfill and was on the verge of losing toe material. The objective of this investigation was to determine the extent and cause of ongoing subsurface erosion, to evaluate its effect on the bulkhead stability, and to recommend and design mitigative measures. The cause of the erosion was determined to be piping of fine grained soils due to inadequate backfill drainage. A remedial drainage scheme was designed and field-tested, and several structural repair schemes were suggested for portions of the bulkhead where accumulated damage affected the integrity of the structure.

INTRODUCTION

The Channel Islands waterfront development in southern California consists of approximately 585 lots adjacent to artificial waterways connected to the Pacific Ocean. The lots are supported by concrete retaining walls or bulkheads completed in 1970. Two types of pile-supported bulkhead designs were used in this development: an L-shaped retaining wall without tie-back anchors and a precast panel and pilaster system with tie-back anchors. The latter was used for 231 lots and is the subject of this paper.

---

[1]President, R.M. Noble & Associates

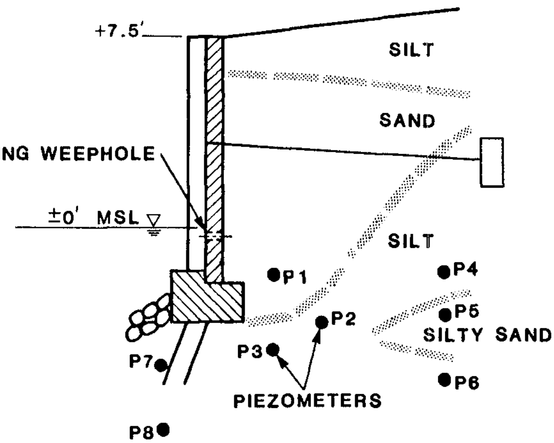
[2]Associate, Dames & Moore

[3]Project Engineer, Dames & Moore, Los Angeles, CA

Within a period of 2 to 4 years after the completion of construction, several sinkholes and areas of subsidence were observed in various lots. The purpose of this investigation was to inspect the bulkheads and to assess their safety and stability. Following the initial assessment, the physical causes of the observed defects were investigated and, where necessary, recommendations for remedial measures were made.

BULKHEAD STRUCTURE

A typical cross section of the bulkhead is shown on Figure 1. The bulkhead is 9.5 feet high and consists of precast concrete panels, which are supported by precast concrete columns, 11 feet on center. The columns are tied back by anchors to a continuous concrete deadman and rest on a continuous 4-foot-wide by 2.5-foot-high poured-in-place footing which is supported by one row of batter timber piles. Two weepholes per panel, with wire mesh screens, are provided for drainage of the sand backfill.



Note: MSL = Mean Sea Level

Figure 1. BULKHEAD CROSS SECTION

The waterway channel is dredged to a depth of approximately 6 feet below the bottom of the bulkhead footing. The underwater slope of 2(h):1(v) is protected by filter cloth and riprap. The elevation of the original ground surface was about 2 to 3 feet below the bulkhead top elevation. The construction of the waterfront development proceeded in three major steps: (1) excavation of channels with berms for bulkhead footing and concrete deadman; (2) construction of bulkhead; and (3) backfilling to existing grade, final grading and placing of slope protection.

INVESTIGATION

At the time of this investigation, approximately 50 percent of the waterfront lots were unimproved and accessible, while the rest were built on, and thus were not inspected. The investigation consisted of a visual inspection of all accessible lots, an exploratory drilling program on a few selected lots, a diving survey of selected underwater slopes, and a field testing program involving the installation of pore water pressure gauges on two selected lots for the purpose of measuring hydraulic gradients due to tidal movements within the backfill. Finally, one of the instrumented lots was selected to field-test remedial drainage measures.

Visual Inspection

During the visual inspection, numerous sinkholes measuring approximately 0.5 to 2 feet in diameter and 1 to 3 feet in depth were encountered immediately behind the wall. Such sinkholes were found mainly at the joints between the columns and wall panels and in a few cases at the locations of the weepholes in the center sections of the panels. In addition to the sinkholes, several areas (approximately 10 by 10 feet) with a subsidence on the order of 2 to 4 inches were encountered, usually at convex bulkhead corners.

Subsurface Investigation

Subsurface conditions were explored by drilling at least two borings each on seven selected lots, ranging in depth from 9 to 17 feet below the ground surface. Undisturbed soil samples were obtained at intervals of aproximately 2 feet. The emphasis of the laboratory testing program focused on grain-size analysis and determination of dry density, both considered to render data for evaluating the soil's potential for subsurface erosion (piping).

A typical subsurface profile in the immediate vicinity of the bulkhead is shown on Figure 1. A backfilled wedge of loose fine sand extends down to the footing base, which is underlain by soft natural sandy to clayey silts. The loose sand backfill is generally covered by a stiff silt layer of 3 to 4 feet in thickness. The soils behind the backfill wedge range from loose silty fine sands to dense sandy silts.

Underwater Slopes

The upper, riprap-covered portion of the underwater slope extends from an elevation of 1 foot below the top of the footing to 4 feet below the top, and slopes 2(h):1(v). After having reviewed preliminary data from the Phase I investigation and results of the bathymetric survey, several locations were explored qualitatively by divers. Generally, sandy silt deposits up to 8 inches in thickness were encountered on top of the footings, as well as on the riprap-covered slope extending downwards from the toe of the wall.

Judging qualitatively, the underwater slopes, seemed to be intact with the exception of the slopes at the convex corners of two lots. At one of these lots, the top of the slope had settled extensively, and a gap had developed between the footing base and soil. The gap was up to 17 inches high, and up to 4 feet deep. At the other lot, the top of the slope seemed to have settled also, however, without exposing a gap.

Pore Pressure Measurements

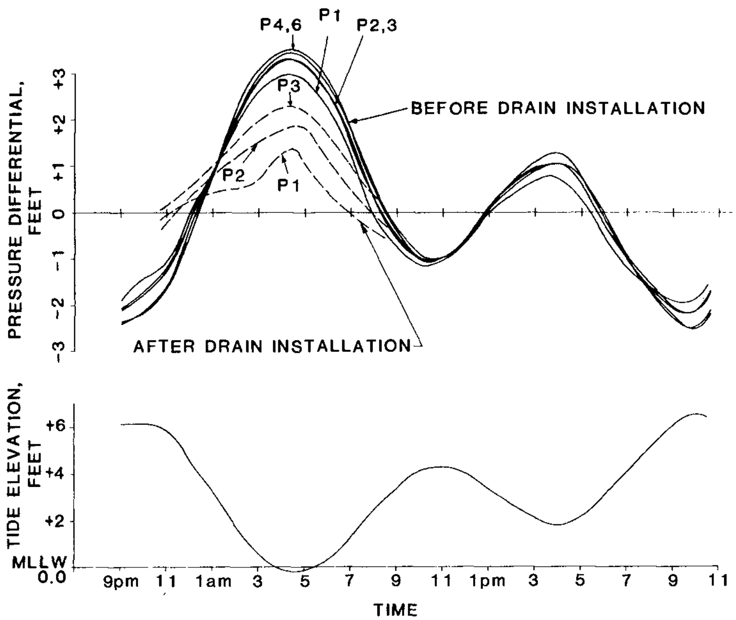
Pore pressures were measured in order to evaluate pressure gradients resulting from seepage toward the channel at low tide. Figure 1 shows the layout of the installed piezometers which consisted of instant-responding pneumatic pore pressure probes. After allowing a minimum of 7 days for stabilization of seepage conditions after installation, the piezometers were read hourly during one 24-hour cycle of extreme tidal movements. Relating the measured pore pressures to the free water level in the channel, it was possible to construct flow nets indicating areas of high hydraulic gradients in the backfill and bulkhead foundation which are especially endangered by piping.

The general trend of seepage gradients versus time in the backfill is shown graphically on Figure 2. The upper portion of the figure shows the pore pressure differentials, with reference to the free water level in the channel, at the locations of the installed piezometers. For instance, a pore pressure differential of +2 feet at a particular location and time would mean that the ground water in a hypothetical stand pipe installed at this location would rise 2 feet above the water table in the channel. The ground water seepage would thus be directed towards the water channel. From the standpoint of piping, the most severe conditions exist at the peak of positive pore pressure differentials. The lower portion of Figure 2 is a plot of the corresponding tide elevations versus time.

Flow nets were constructed at several critical times, making simplified assumptions, such as homogeneous soil conditions and two-dimensional flow conditions. The flow nets shown on Figure 3 represent two "snapshots" of a constantly changing flow pattern in the backfill. The indicated times (1 a.m. and 3 a.m.) for which the flow patterns are depicted correspond to the time scale of the graph on Figure 2.

The flow net at 1:00 a.m. indicates a zone of flow reversal moving away from the wall as the free water level in the channel moves down. The fact that this flow reversal condition, involving very small relative pressure differentials, could consistently be derived from actual pore pressure measurements increased confidence in the piezometer data.

The most critical flow condition in terms of underground erosion (piping) is demonstrated with the flow net at 3:00 a.m. The flow lines in this and subsequent "snapshot" flow nets (not shown here) suggest that the bulk of the seepage water escapes through the vertical panel/column joints above the footing.

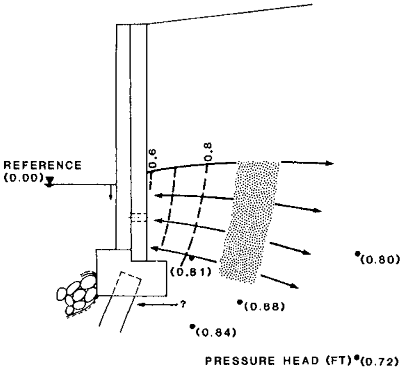


Note: MLLW = Mean Lower Low Water = -2.8' MSL

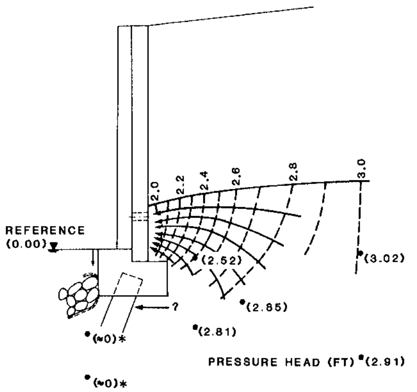
Figure 2. PORE PRESSURE MEASUREMENTS

Although the test results did not indicate significant seepage underneath the footing, a second test was performed with two additional piezometers (P-7 and P-8 in Figures 1 and 3) inserted on the waterside beneath the footing, with the objective to study the seepage in the foundation soils. It was observed that these two additional piezometers essentially fluctuate in phase with the free water level

a) 1:00 A.M.



b) 3:00 A.M.



* PIEZOMETERS ADDED FOR 2nd TEST

Figure 3. FLOWNETS

in the channel. This indicates minimal seepage in the lower portions of the subsoil beneath the footing. However, it does not exclude the possibility of concentrated seepage through a gap or thin soil layer just beneath the footing base.

CONCLUSIONS

The primary cause for the development of sinkholes and subsided areas behind the bulkhead is loss of sand backfill through the panel joints caused by seepage forces (piping). Sinkholes and subsidence are initially of only cosmetic consequences. However, if such conditions were neglected for long periods, progressive failure involving the bulkhead footings and/or deterioration of the underwater slope could develop. Such conditions were actually encountered at some locations as described above.

Piping underneath the footing base does not appear to be a primary cause for loss of material. However, once a gap has developed between footing base and subsoil, migration of the sandy backfill progressively accelerates underneath the footing. Because the bulkhead rests on piles, the soil tends to settle away from the footing due to minor creep movements of the underwater slope. such creep movements are believed to have caused the slumping of the slope at some locations observed by the underwater survey.

Summarizing, there is evidence that cyclic seepage forces (piping) mainly due to tidal action, are responsible for existing subsidence, sinkholes and gaps. Therefore, the main objective of remedial measures discussed in the following sections is the reduction of these seepage forces. In addition, at selected locations where progressive undermining of the footings is already taking place, repair schemes for foundation and/or underwater slope will be discussed.

REMEDIAL MEASURES

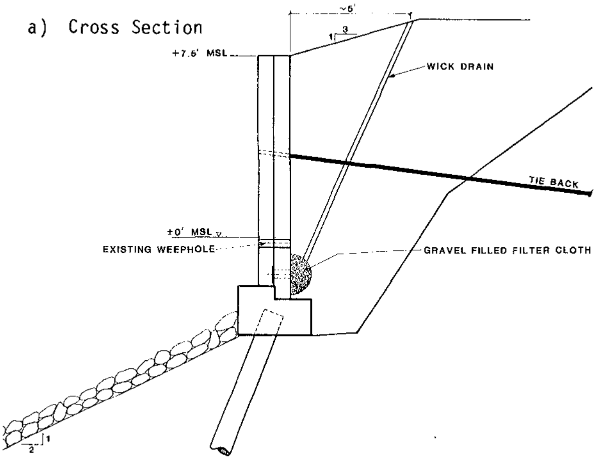
Backfill Drainage

Consideration was given to various kinds of drainage systems, including vertical sand (or wick) drains, gravel drainage trenches, inclined wick drains, and horizontal well-point drains. A desk study narrowed the alternatives down to two: the inclined wick-drain system and the horizontal well-point system.

The schematic of the inclined wick-drain system is shown on Figure 4. A typical wick drain is 3 to 4 inches wide and consists of a corrugated plastic core wrapped in filter fabric. These drains are installed with a mandrill pushed into the ground with the wick drain attached to its point. Upon withdrawal of the mandrill, the drain stays in the ground acting as an effective drainage channel. The drains would be installed from the land side, aiming at a center location just above the footing. After installation of the drains, a gravel-filled filter-cloth pouch would be inserted, from the water side, through a hole drilled in the concrete panel.

Because access from the land side was judged to be quite difficult for the majority of the waterfront properties, the drainage method involving horizontal well-points (installed from the water side) was finally selected for a field test. No. 7 well points (0.007-inch-wide slots) 3/4-inch in diameter were used for this test.

a) Cross Section



b) Plan View

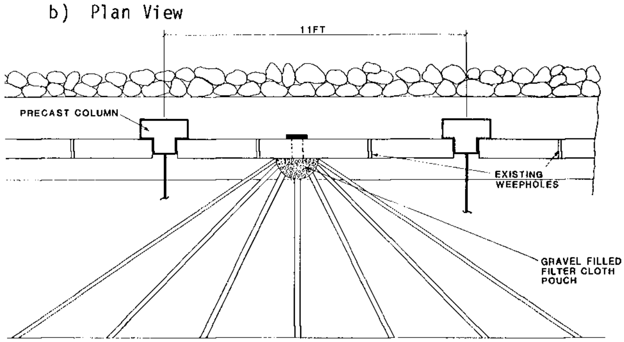


Figure 4. INCLINED WICK-DRAIN SYSTEM

The grain size distribution of the backfill material indicated that approximately 10 to 15 percent would pass the 0.007-inch sieve. While initially small amounts of the fine soils might pass through the well screens, rearrangement of the grains around the well screen will eventually develop a natural filter preventing further material loss. The field test was conducted at the location which was previously instrumented with piezometers, in order to compare hydraulic gradients before and after installation of the drainage system. Figure 5 shows the layout of the field test.

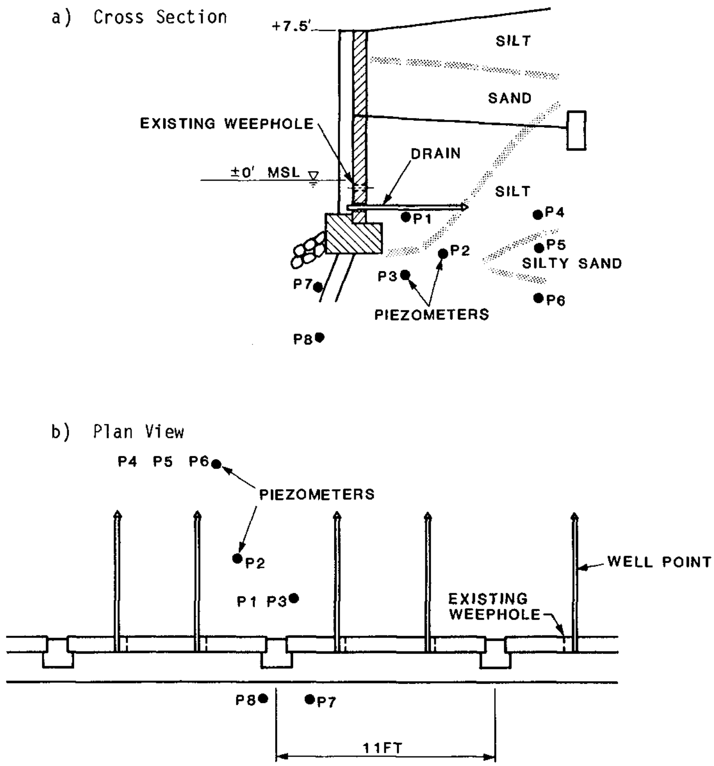


Figure 5. HORIZONTAL WELL-POINT SYSTEM

Based on experience with dewatering in similar soils, two drains per panel were estimated to be sufficient for significant drawdown of the water table in the backfill between the drains. A total of six horizontal drains, two per panel, were installed for the field test. Two well points are of plastic (PVC) and four are of stainless steel. While the material is insignificant for the functioning of the drains, it was found that the stainless steel well points were installed more conveniently than the plastic ones. The latter had to be driven with a mandrill acting on the pointed tip, and care had to be taken not to separate the tips during the driving procedure.

The evaluation of the drains' effectiveness relied on pore pressure measurements in the backfill before and after the installation. Figure 2 shows the pore pressure differential towards the free water level, measured during a critical tidal cycle before the drain installation, and during a similar cycle after the installation. Plotted pressure potentials (after installation) are those of locations P-1 through P-3, represented by the piezometers closest to the wall. Any potential piping which could lead to development of a gap would occur through seepage from the loose backfill at the footing/subsoil interface rather than through deeper zones of the underlying silty soils. The most important piezometer location is P-1 since it represents the conditions in the loose backfill just above the footing base. P-2 and P-3, on the other hand, were expected to record higher pore pressure differentials than P-1, because of the time lag caused by slower drainage of the natural silts in which they are embedded.

Piezometer P-1 shows a rather large reduction, due to the drains, of the maximum pressure potential towards the free water level (from 3 to 1.4 feet). The well points are approximately 1 foot higher than the minimum water level of the tidal cycle under consideration. Therefore, at the time of minimum water level, the ground water behind the wall forms a pool whose water table remains roughly at the well point outlet. Thus, the recorded pressure differential can not be smaller than 1 foot, even under perfect drainage conditions. Hence, the field test with horizontal well points was considered successful and this drainage system was recommended for remedial measure of the entire bulkhead system.

For the locations where apparent gaps beneath the footing base have already developed, repair measures for the footing and/or underwater slope protection were designed as described in the subsequent section.

Foundation Repair and Slope Protection

For lots where apparent gaps beneath the footing base had already developed, it was recommended that the gaps should be closed by grouting with provisions for proper formwork to retain the grout. The slope protection on these corner lots was to be upgraded to resist future erosion. Two schemes were developed for the foundation repair work. One scheme was to allow visual pile inspection at the pile-footing connection in case of marine borer attack on the piling. The potential for marine borers reaching exposed piling during extreme low tides was considerable in areas with large gaps beneath the bulkhead footing. The second scheme only differed from the first in that it did not easily allow for visual pile inspection.

The two schemes are shown in Figures 6a and 6b. These schemes called for driving sheet piling several feet into the bottom in front of the bulkhead footing. The sheet piling would be secured to the footing and act as a protective curtain wall from bottom scour (erosion) exposing the pile-footing connection. The space between the footing and sheet pile wall would then be filled with concrete, and a grout pipe would be inserted behind the bulkhead to the base of footing to grout all void areas beneath the footing.

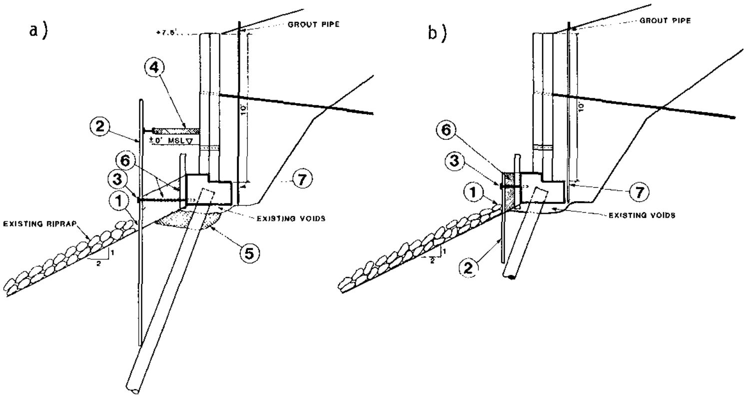


Figure 6. FOUNDATION REPAIR SCHEMES

(1) Remove rock slope protection.

(2) Drive sheet piles.

(3) Secure sheet piles to concrete footing.

*(4) Install support members and dewater.

*(5) Excavate and inspect pile.

(6) Fill with concrete and insert 4" pipes 3-5 feet on center, extending into the cavity (5).

(7) Pressure grout until grout extrudes from PVC pipe (6) (alternatively, pressure grout through PVC pipe).

(8) Cut off sheet pile at top elevation on concrete.

Note: Applies for Figure 6(a) only.

It was recommended that the slope protection for the corner lots be upgraded by use of either rock riprap and filter cloth or a concrete mattress and filter cloth as shown in Figures 7a and 7b. In addition, concentrated seepage of surface water into the backfill immediately behind the bulkhead was to be prevented by appropriate surface drainage.

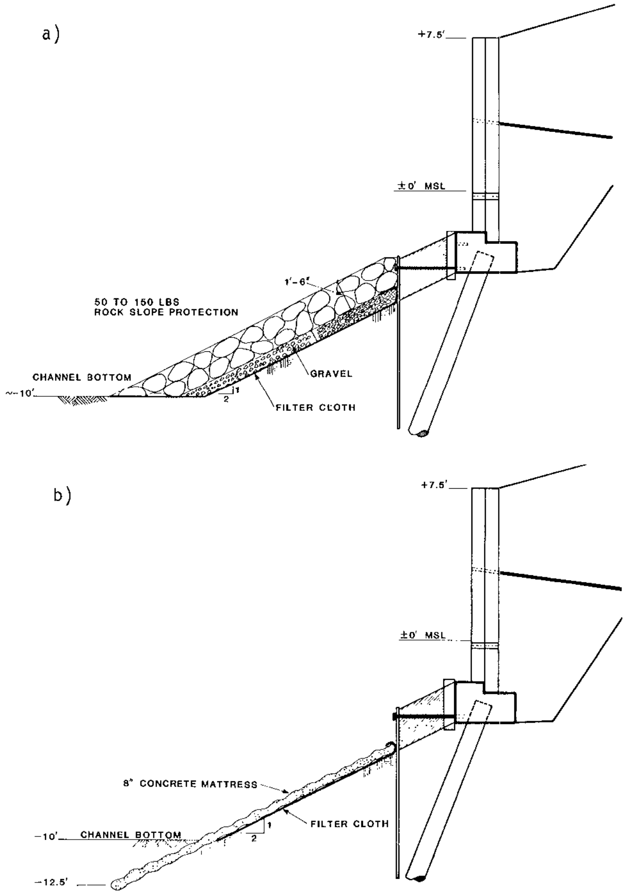


Figure 7. SLOPE PROTECTION