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Filter Design Criteria for Sediment Caps in Rivers and Harbors

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ABSTRACT

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Remediation of contaminated sediments in rivers and harbors by in-place capping is being increasingly considered at several sites along the United States and even worldwide. Currently, few design guidelines are available for use in designing such cap systems, especially the requirements for the filter layer. An in-depth review of the existing literature on filter design criteria is provided in this paper. In addition, details of the methodology and results of a physical model investigation undertaken for evaluating the various filter design options for in-place sediment caps in the Great Lakes are presented. Finally, design guidelines and nomograms are developed for application at contaminated river and harbor sites for filter design.

ADDITIONAL INDEX WORDS: *Capping, filter, design, contaminants, sediments, rivers, harbors.*

INTRODUCTION

Capping is a technology that is being increasingly considered for contaminated sediment (in-situ) remediation at various river and harbor sites across the United States and even worldwide. Capping involves isolation of the contaminated sediments using one or more layers of specially placed sediments or engineered systems. Typically, such a cap consists of a base isolation layer (usually sands with some organic fraction to provide chemical and biological isolation of the contaminants), a filter layer (usually gravels to provide hydraulic stability to the base layer) and an armor layer (usually rocks to provide protection from erosive forces). The need for and the engineering design criteria for the filter layer in marine cap systems is a subject that has been debated for long. Although various equations have been developed in the past for filter design criteria for hydraulic channels and dikes, there has been very few studies that looked at the case of in-situ sediment caps in rivers and harbors.

Filters have been used historically as part of engineered structures (such as dams, dikes, *etc.*) due to several reasons including: (i) to distribute the load evenly to poor foundation soils that are relatively unconsolidated, and (ii) to protect the base material from direct attack by wave forces and currents. The effectiveness of a protective layer is demonstrated by the stability of the material protected, when subjected to the forces against which protection has been designed. The movement of the bed material beneath a protective layer may be due either to flow-induced eddies which are transmitted to the bottom through the voids in the protective layer, or the seepage forces developed by percolating water, or a combination of both. If the material of the protective layer were so graded that the voids are small enough to prevent the transmission of the eddies, bed movement could not occur as a direct result of the disturbing forces. However, if the material were not of sufficient specific weight or of adequate size, it might move under the tractive force of the overflowing water. Also, if upward flow occurs through a sandy soil, and if the hydraulic gradient at the surface is larger than the ratio of the effective unit weight of the material to the unit weight of water, then a quick condition would result. MANAMPERI (1952) suggests that these effects can be prevented by loading the soil with a filter layer to satisfy the following conditions: (i) the filter layer should provide adequate weight (the ratio of the effective unit weight of the material to the unit weight of water must be greater than the pressure gradient through it), (ii) the material must have enough fines and must be tightly packed to prevent the passage of particles from below, through its pores, and (iii) the filter constituents must be coarse enough to dissipate the pressure head by the flow of water through its pores, and thus reduce the magnitude of the seepage forces developed within it.

In general, the effectiveness of a granular filter medium to protect a base material is dependent on three independent sets of factors: (i) the geometry of the void network of the filter material and, in particular, the sizes of the constrictions (openings connecting pores) within the void network; (ii) the sizes of the base particles and, if significantly small, the concentrations and the surface properties of the grains or grain aggregations being transported by seepage into the filter; and (iii) hydraulic conditions, such as velocity and seepage direction of flow. KENNEY *et al.* (1985) suggests that of the above mentioned factors, the constriction sizes in the void network and the hydrodynamic conditions within the filter are the

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controlling factors. In order for a filter to effectively protect the base material, the controlling constriction sizes (which in turn define the largest particle that can possibly be transported through the filter) should be as low as possible. The broader the gradation of the filter, and the greater the thickness of the filter layer; the smaller will be the controlling constriction size (and hence, the better will be the efficiency) of the filter.

However, for underwater applications, practical considerations (the accuracy of placing a thin filter layer in deep water) may make the filter quite expensive. For such applications, a filter could be integrated as part of an armor system by broadening the gradation of the armor system (for example, 20 percent gravel size particles and 80 percent cobble size particles). The stones could be well mixed during placement and any deficiencies in gradation will be minimized in due course of time by the self-segregation effects of filters (KENNEY *et al.,* 1985).

This paper provides an in-depth review of the various filter design criteria that is currently available in the literature. In addition, details of a physical model investigation undertaken for evaluating the various filter design options for in-place sediment caps in the Great Lakes are presented. Finally, filter design guidelines and nomograms are developed for application at contaminated river and harbor sites.

REVIEW OF PREVIOUS RESEARCH ON FILTER DESIGN

TERZAGHI (1949) established the following criteria for the design of filters:

$$d_{15(F)} < 4d_{85(B)} \quad (1)$$

$$d_{15(F)} > 4d_{15(B)} \quad (2)$$

where $d_{15(F)}$ is the diameter through which 15 percent of the filter passes, $d_{15(B)}$ is the diameter through which 15 percent of the base material passes, and $d_{85(B)}$ is the diameter through which 85 percent of the base material passes. The first criteria (Equation 1) prevents the largest base material grains from being washed into the pores of the filter material. Washout of the smaller grains will then be prevented by means of internal formation of filters in the base material (self-filtration). The second criteria (Equation 2) improves the drainage characteristics of the filter.

Experiments on riprap protection for the upstream slopes of earth embankments have been made by the U.S. Army Corps of Engineers at Vicksburg (USACE, 1949), which resulted in some modifications of the Terzaghi criteria (given below) and is commonly referred to as the Terzaghi-Vicksburg criteria.

$$d_{15(F)} < 5d_{85(B)} \quad (3)$$

$$d_{15(F)} > 5d_{15(B)} \quad (4)$$

$$d_{15(F)} < 20d_{15(B)} \quad (5)$$

$$d_{50(F)} < 25d_{50(B)} \quad (6)$$

where $d_{50(F)}$ is the diameter through which 50 percent of the filter passes, and $d_{50(B)}$ is the diameter through which 50 percent of the base material passes.

JETTER (1951) conducted experimental studies on the feasibility of using sand dikes along the Mississippi River and concluded that the desired filter should have coarse sand in contact with the dike surfaces, varying gradually through materials of increasing size to stone of sufficient dimensions at the top to withstand erosion. However, it should be noted that the dikes are subjected to considerable forces from seepage flow and overflow resulting from river level fluctuations and hence this design may be conservative for an underwater cap.

DE ABREU-LIMA and MORGAN (1951) investigated protection against overflow of earth embankments with riprap and observed that the thickness of uniformly graded riprap layers increased in more than direct proportion to the velocities against which protection was afforded. Based on De Abreu-Lima and Morgan's results, MANAMPERI (1952) derived the following relationship between thickness of protective layer (t in inches), and failure velocity (V_f in fps):

For 0.75 to 1.00-inch (1.91 to 2.54 cm) thick armor layer:

$$V_f = 1.40 + 0.45t \quad (7)$$

For 1.00 to 1.50-inch (2.54 to 3.81 cm) thick armor layer:

$$V_f = 1.40 + 0.23t \quad (8)$$

In the above formulation, V_f is the mean flow velocity, while the actual near bed velocity may be the more relevant parameter as Manamperi reports that higher mean velocities are required to initiate scour when a thicker boundary layer exists upstream due to the placement of a rough surface prior to the test section. However, most formulations of this sort are based on the mean flow velocity instead of near bed velocity or shear stress due to the ease of computation of the mean velocity in a unidirectional flow. MANAMPERI (1952) conducted extensive studies on the use of graded riprap for protection of erodible material, and suggested the following: (i) the thickness of uniformly graded stone layers when used to protect an erodible bed, increased in more than direct proportion to the velocities at which movement of the sand bed commenced, (ii) roughening of the boundaries of the filter layer necessitated higher velocities for disturbing the surface of the protected layer, and (iii) a three inch protective layer of crushed stone graded in accordance with the Terzaghi-Vicksburg criteria for an effective filter blanket against upward flow would provide sufficient protection of the base layer.

For uniform filters on a base of medium to coarse uniform sand, KARPOFF (1955) suggest the following relationship:

$$5 < (d_{50(F)}/d_{50(B)}) < 10 \quad (9)$$

Data by ZWECK and DAVIDENKOFF (1957) suggest that for a base of fine uniform sand, the upper bound of Equation 9 could be up to 15. Further, SHERMAN'S (1953) data suggest that the upper bound of Equation 9 could be up to 20 as long as the following relationship is valid:

$$6 < (d_{15(F)}/d_{15(B)}) < 20 \quad (10)$$

ABERG (1993) suggests that there are two potential reasons for grading instability: (i) scantiness of intermediate grain sizes, which interrupts an internal filter formation process; and (ii) loose grains, which move through the void space between fixed grains. KENNEY and LAU (1985) investigated internal instability of a granular soil caused by the inability of the soil to act as a filter to prevent the loss of its own small particles. They found that if the pore spaces of the uniform material (armor stone layer) were packed with small particles (smaller size stones) in a dense arrangement, the density would be maximum and the mixture would be hydraulically stable because of the absence of loose particles. They also state that for any particular gradation, hydraulic stability increases as density increases. However, they caution that density, by itself, is not a valid criterion by which to judge the hydraulic stability of a granular layer. They note that from an engineering viewpoint, it may be concluded that with regard to grading stability of a soil medium, the important concern is not as much the existence of loose movable particles but, rather, the potential travel distances of these particles relative to the dimensions of the soil layer. If the thickness of the transition zone at the influx boundary is small and grading of the central soil zone remains unchanged, the soil system can be considered stable. Finally, KENNEY and LAU (1985) state that the most reliable method for determining whether or not a soil system is potentially stable is to perform a hydraulic stability test simulating the external disturbing forces.

The Canadian Foundation Engineering Manual (CGS, 1992) states that filter design is based on the phenomenon that if perfect spheres have diameters greater than six and one-half times the diameter of a smaller sphere, the smaller sphere can move between the larger spheres. They state that the criteria most frequently used is the one suggested by the U.S. Bureau of Reclamation (USBR, 1974) given below:

$$d_{15(F)} < 5d_{85(B)} \tag{11}$$

$$d_{50(F)} < 25d_{50(B)} \tag{12}$$

WORMAN (1989) investigated the need for a filter layer for scour protection around bridge piers and concluded that the conventional multi-layered rip-rap and filter protections could be substituted by one thick single layer of uniform riprap without any adverse effects. For a state of flow at which the sand grains are transported through the riprap layer, the vertical components of the pore water velocities should exceed the fall velocities of the sand grains, assuming that the relative pore channel width is large enough not to significantly affect the exchange of momentum between the flow and the sand grains. Further, for the transport of a sand grain to occur, the vertical component of the drag force on the sand grain generated by the flow should exceed the submerged weight of the sand grain. Worman's experimental results clearly indicated that a riprap protection can be constructed efficiently as a single, homogeneous layer without filter layers. For round quartz sands used as base material and round uniform riprap material, Worman defined stability of the protective layer in a riverine environment by the following relationship:

$$(V^2/gS) = 6(d_{85(F)}/d_{15(A)}) \tag{13}$$

where V is the mean flow velocity above the revetment, g is the acceleration due to gravity, S is the riprap layer thickness and $d_{15(A)}$ is the 15 percent passing size of the armor material (riprap). Although the form of this equation may hold good for caps in harbors, the empirical coefficient is overly conservative due to the harsh environment near the bridge piers of the Worman study. In particular, although Equation 13 is formulated in terms of the mean flow velocity approaching a bridge pier, the analysis behind the formulation assumed that the velocity in the horseshoe vortex formed at the pier (presumed to be responsibility for any armor instability) would be twice the local velocity. Worman notes that for $d_{85(F)}/d_{15(A)}$ greater than 0.12, a geometrical filter criterion is satisfied and the transport of the base material through the riprap layer becomes impossible, except for an initial insignificant amount of grains.

MAYNORD (1995) modified the coefficient in Equation 13 using MANAMPERI'S (1952) data to yield a more realistic design criteria for filters in harbors as follows:

For uniform riprap ($d_{85(A)}/d_{15(A)} = 1.3$):

$$(V^2/gS) = 24(d_{85(F)}/d_{15(A)}) \tag{14}$$

For graded riprap ($d_{85(A)}/d_{15(A)} = 6.7$):

$$(V^2/gS) = 10(d_{85(F)}/d_{15(A)}) \tag{15}$$

The velocity in Equations (14) and (15) was intended to be the near bottom velocity predicted by a suitable wave theory. The above formulation by MAYNORD (1995) is probably one of the more applicable design criteria for filters in the harbor environment, while the formulation by Worman (Equation 13) holds good for the riverine environment. A major difference between the two applications is in the difference between the velocity used in the formulation and the local velocity responsible for creating the scour condition.

In reality, construction of a thin layer of filter layer in deep water conditions in rivers and harbors is extremely difficult to implement and unattractive due to cost considerations. Therefore, the authors proposed that it is possible to integrate the filter layer into the armor layer by widening the gradation of the armor material, while providing sufficient hydraulic stability to the cap system. If this option is technically feasible, it would result in considerable savings during cap construction due to the ease of construction. In order to determine the effects of the various filter design options on the hydraulic stability of the cap system, a laboratory model investigation was conducted at the University of Michigan.

LABORATORY MODEL INVESTIGATION

Prototype Design Conditions

A laboratory model study was conducted to assist in the design of a specific armor layer. The application involved a cap over contaminated sediments in a harbor in Lake Michigan. The harbor is located in a river mouth but also subjected to wind waves generated on Lake Michigan. Flow velocities were estimated for conditions associated with a 100 year return period flood in the river as well as an even more extreme design wave condition; a comparison of the estimat-

Table 1. *Details of Tested Cap Configurations*

Item	Test No. 1	Test No. 2	Test No. 3	Test No. 4	Test No. 5
Sand Layer					
Median Size (cm):	0.1	0.1	0.1	0.1	0.1
Thickness (cm):	9	9	9	9	9
Filter Layer					
Median Size (cm):	5–10	1.3–2.5	10–20 & 1.3–2.5*	10–20	n/a**
Thickness (cm):	10	10	10 (i.e., 5+5)*	31	n/a**
Armor Layer					
Median Size (cm):	10–20	10–20	10–20	1.3–2.5	1.3–2.5 & 10–20***
Thickness (cm):	20	20	20	10	25

* Represents two filters.
** n/a = not applicable (test had no separate filter layer).
*** Test used a mixture of the two stone types as a graded armor layer.

ed velocities for the two conditions indicates that wave induced velocities will control the armor layer design. Based on the assumption of 100 year return periods simultaneously on water level (which are control by variations in annual precipitation and other climate influences) and wave height, the design condition was established to be a water depth of approximately 6.7 m, a wave height of 5.0 m and a period of 12.4 s. This water depth was based on a armor layer thickness of not more than 30 cm, a thickness that would not be possible using conventional filter design criteria.

The laboratory study was conducted to determine the feasibility of developing a protective armor layer with small thickness. Since it was not possible to reproduce the prototype wave conditions at full scale in the laboratory, two choice were possible, either to reproduce the wave climate at a reduced scale along with a similar reduction in the armor materials or to utilize prototype size armor materials in conjunction with a velocity representative of the maximum expected under the prototype wave. This latter approach was deemed more appropriate since a reduction in the physical scale of the armor units would introduce an uncertain distortion in viscous effects in the flow within the pore structure of the armor layer. An analysis of maximum wave velocities under the depth and wave conditions listed above was carried out using second order cnoidal wave theory and indicated a maximum bottom velocity of approximately 1.8 m/s. Velocities in this range were established in a laboratory flume utilizing a uni-directional free surface flow. Consideration was made of the fact that the boundary layers in uniform free surface flow and under oscillatory waves are not the same and the laboratory setup precluded significant boundary layer development.

Based on the Isbash equation (USACE, 1970) an armor stone with an equivalent diameter of approximately 6.6 cm would be stable under a bottom velocity of 1.8 m/s. A conservative design approach was taken, however, to avoid the possibility of armor instability and the mean size of the primary armor stone was more than doubled above the threshold limit. Therefore, it was expected that the primary armor layer would be unconditionally stable and that the major concern related to the possible movement of the materials in the sediment cap below the armor layer due to a significant component of flow within the large pore spaces within the armor stone.

Model Setup and Instrumentation

The model setup is illustrated in Figure 1 and consisted of a 9 m (~30 ft) long and 0.6 m (~2 ft) wide plexiglass flume with a constant head supply. In order to evaluate the effect of cap configuration on the resulting hydraulic stability, several configurations of the sand isolation layer (sediment cap), filter layer, and armor layer materials were set up in the flume. The material properties and dimensions of the tested materials are summarized in Table 1. Note that all tested rock products were angular due to the production of the material (by crushing larger stones). The plexiglass walls of the flume permitted visual (and photographic) observation of the performance of the cap and armor system over time.

Tests were set-up using a 3 m (~10 ft) section of the flume as shown in Figure 1. An overflow weir was installed at the downstream end of the flume, which was used to control the flow depth in the test section and also to trap any sand that was eroded from the test section. The flow velocity over the cap was controlled by adjusting the flow rate into the channel and the height of the weir. A sand isolation layer thickness of approximately 9 cm (~3.5 inches) was reproduced in the test section since erosion of sand in this regime could be used as a good indicator of the erodibility of any thickness of the sand layer. The sand layer was confined at either end of the test section using plexiglass plates, thereby forcing erosion from the top of the sand layer as opposed to scouring from the ends of the test section.

Several factors were evaluated to minimize the effects of the finite ends of the test section with regards to water flow within the armor stone layer. It was determined that extending the plexiglass plates to the top of the armor layer would be undesirable since it would exclude the flow from the armor layer, thereby leading to an underestimation of the scour potential of the sand isolation layer. By placing no obstruction on either end, the scour potential would be overestimated since the approach flow and that leaving the test section can extend deeper into the armor layer than would actually be the case in a riverine or harbor environment (which has a

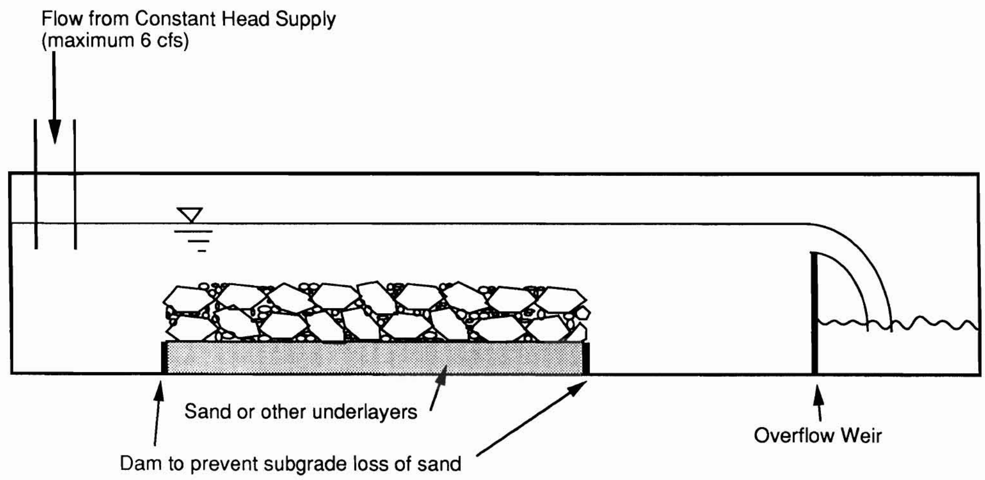

Figure 1. Schematic of Experimental Configuration.

larger surface area). However, it was determined that a conservative approach is better suited to test the graded filter concept and therefore no obstruction was placed on either end of the test section.

Flow velocities in the test section were measured using a min-propeller meter which provided velocities by counting rotations over 10 s averaging periods. Several meter readouts were averaged to obtain the average velocity at any one measurement location. It was found that the velocity was basically uniform in the flow area above the tops of the uppermost armor stones. Below this, the indicated velocities were erratic due to the fact that measurements were being made in the wakes of individual stones. Therefore the velocity approximately 5 cm above the top of the stone surface was recorded as the representative velocity for the test condition investigated. The flow meter was typically placed in the middle of the last 0.8 m (~2.5 ft) of the test section, since the highest flow velocities generally occurred near the downstream end of the test section. Erosion of the sand isolation layer during the experiment was monitored by video and photographs. Data on sand layer erosion was also noted in the experimental log from visual observations throughout the experiment.

Experimental Variables and Test Procedure

Five different configurations of the cap (sand, filter and armor stone) system were tested for hydraulic stability. The major difference between the configurations was the presence and type of the filter layer in the cap system as described in Table 1. Note that tests 1 and 2 had two different sizes of filter layer (1.3–2.5 cm and 5–10 cm, respectively), while test 3 used two intermediate filter layers (one each for the 1.3–2.5 cm and 10–20 cm sizes, respectively) with an intention to produce a mixed armor layer (also called the "graded armor layer"). Test 4, on the other hand, consisted of the armor layer immediately above the sand layer, and the filter layer was placed on top of the armor layer. This setup was studied to investigate the effectiveness of a proposed underwater construction concept, where the stone layer is placed first and the filter layer is then rained on top of the armor layer to fill in its large voids. The concept of the graded armor layer was also evaluated using test 5, which comprised of a mixture of 1.3–2.5 cm stones (filter) and 5–10 cm stones (armor) placed over the sand cap layer. The rationale for this test was to keep the total cap thickness small. This was based on the assumption that the smaller stone would be transported back and forth under wave action until it found a stable position within the large pore spaces between the primary armor stone. Thus while the smaller stone would be unstable under the velocities induced by the design wave, once it fell into and filled the pore spaces of the primary armor, the sheltering effect of the larger stone would prevent further displacement.

The test procedure consisted of the following steps. Sand was placed in the flume to a thickness of 9 cm (~3.5 inches) and leveled to the top of the plexiglass plates. The filter and armor layers were then added to the top of the sand layer with the exact procedure dependent on the type of stone to be placed. The flow probe was then placed 5 to 8 cm above the top of an armor stone unit which had a relatively flat upper surface so that velocity measurements in zones of flow separation were avoided. Water was then added to the flume to raise the level to the downstream weir crest. This prevented the initial flow of water from rushing through the air-filled voids of the armor stones and potentially scouring the sand at low flow rates. The downstream water level and discharge were then adjusted to obtain the desired flow velocity for the experiments. In general, a flow depth of approximately 15 cm (~6 inches) was maintained above the top of the armor layer.

The experiments were conducted for a particular sand layer and armor cross-section by first starting the flow at a relatively low velocity (0.65 m/s or ~2 ft/s) and gradually increasing the velocity until either significant erosion of the

Figure 2. Preliminary Stage of Test Section Construction (Placement of 3.5 inches of the Sand Isolation Layer).

sand layer material occurred or until the maximum velocity of the experimental setup was exceeded (*i.e.,* velocity greater than 2.0 m/s or ~6 ft/s or somewhat greater than the bottom velocity representative of the design wave condition). Throughout the experiments, observations were made regarding the flow velocity, potential transport of sand through erosion of the sand isolation layer, and whether any sand motion (or other instabilities) was observed through the plexiglass walls near the flume sides. Instability of the sand layer was defined as a continuous erosion process with no indication of stabilization. The entire test was also monitored through video and camera photography. Figures 2–4 illustrate the progression of the test setup for test no. 1.

Experimental Results

During the course of the testing, it was observed that there was a slight decline in flow depth across the test section (see Figure 5). This was attributed to head losses in the flow through the armor stone and over the irregular surface. Associated with the decreasing depth was a corresponding increase in the measured velocity. The largest velocities were always observed near the downstream end of the test section and no scour was ever observed at the upstream end. Therefore, the entrance condition was judged not to be important as the flow had nearly 3.3 m (~10 ft) of flow across and through the armor layer to develop a flow profile within the armor stone.

During the startup of individual tests, small amounts of sand were noticed to be eroded from the test section as the flow was initiated with velocities on the order of above 0.6 m/s, with no subsequent erosion once the flow stabilized. This occurrence was assumed to be associated with the migration of a few unstable sand grains at the top of the sand layer near the downstream end and not indicative of the long-term stability of the sand layer itself. Observations also indicated that while individual sand grains moved to more stable locations near the stability limit, none were transported downstream outside the test section. If the flow velocity was increased by less than 0.15 m/s at this stage, these sands were clearly transported downstream. Therefore, the identification of the stability limit was generally well defined.

The following effects were observed during experimental runs for the various test conditions described in the previous section.

- When the flow velocity reached 1.3 m/s for test 1, the sand layer started to move. Sand transport initiated when the velocity reached 1.5 m/s and considerable amount of sand was lost when the flow velocity reached 1.6 m/s.
- For test 2, the sand layer was stable for flow velocities exceeding 2.0 m/s.
- When the flow velocity reached 1.3 m/s, local rearrangement of the 1.3–2.5 cm stone was observed for test 3. Once the isolated stones stabilized, there was no further movement in the armor layer.
- For test 4, slight movement of the sand layer was observed at a flow velocity of 1.3 m/s. However, no transport of sand was observed at velocities exceeding 2.0 m/s. It was observed that the smaller stone at the armor layer surface rearranged to fill spaced between the larger stones. At flow velocities exceeding 2.0 m/s, some of the smaller stones at the surface were carried out of the test section. However, there was no significant loss of armor or sand material.

Figure 3. Intermediate Stage of Test Section Construction (Placement of 4 inches of 2–4 inch Armor over Sand Isolation Layer).

- For test 5, some local rearrangement of the smaller stones occurred at flow velocities exceeding 1.3 m/s. There was no significant movement of the sand layer except some minor rearrangement in the test section at velocities exceeding 2.0 m/s.

It is not possible to compare these results to previous measurements on bottom scour since the erosion of the sand will be directly dependent on the velocities within the pore spaces of the various armor layer configurations. Our test layers are sufficiently unique that there is no comparable set of experimental conditions to compare to.

The graded armor layer (test 5) was therefore found to provide sufficient hydraulic stability to the sand layer in all cases. It was observed during the experiments that the smaller (filter) stones rearranged themselves to fill most of the voids in the larger (armor) stones, which resulted in the overall stability of the graded armor system. The ratio of the weight of the smaller stone fraction was found to be about 20 percent of the weight of the larger stone fraction to yield this hydraulically stable graded armor configuration.

DISCUSSION OF RESULTS

Internal stability of a granular material matrix could result from its ability to prevent loss of its own small particles due to disturbing forces such as seepage and wave effects. All soils possess a primary fabric of particles which supports loads and transfers stresses. Within pores in the load-bearing fabric of the cohesionless, granular material, there can exist loose particles, and whether or not those particles can be removed by external forces depends upon: (i) particle size distribution curve of the material, (ii) placement density of the material, and (iii) severity of the disturbing external forces. In concept, if the constrictions in the pore network of the primary fabric are larger than some of the loose particles, water flowing through the pore network would tend to move the loose particles in the direction of the resultant of the external forces. However, if the transported particles encounter smaller constrictions, their travel will be halted and they will act as part of the filter fabric. Therefore, if the pore spaces of the armor stone layer were packed with smaller size stones in a dense arrangement, the density would be maximum and the mixture would be hydraulically stable because of the absence of loose particular spaces. Further, studies by SHERARD *et al.* (1984b) indicate that filters of angular particles of crushed stone are as satisfactory as those of rounded alluvial particles. It is therefore necessary for the filter particle size distribution to have a general shape similar to that of the base particle size distribution.

SHERARD *et al.* (1984a) suggest that for sandy clays and silts, the filter criterion $d_{15(F)}/d_{85(B)} < 5$ is quite conservative. Further, LAFLEUR *et al.* (1989) states that the filtration mechanism for broadly graded soils is different from that of uniform soils and therefore warrants special consideration. For broadly graded soils, the filter criteria must take into account the self-filtration process, whereby the fine grained fraction of the filter moves down closer to the base soil. Within the base layer adjacent to the filter, the retained coarser particles filtrate finer particles, which in turn filter even smaller ones. This process takes place until no more particles can migrate. Thus, the self-filtration process of broadly graded filters will eventually lead to a stable base-filter system. It is this property of broadly graded filters that supports the

Figure 4. Completed Test Section (With Final Layer of 4–8 inch Armor Layer).

use of an armor stone layer consisting of a mixture of smaller and larger stones for underwater caps.

The results of our experiments suggest that a graded armor layer comprised of approximately 20 percent by weight of smaller filter-sized stones mixed with 80 percent by weight of larger armor-sized stones would yield sufficient hydraulic stability to the underlying sand layer. Design nomograms for filter, base material and armor particle size gradations and

Figure 5. Test Section During the Experiment (Showing Water Surface Level Decline in the Direction of Flow).

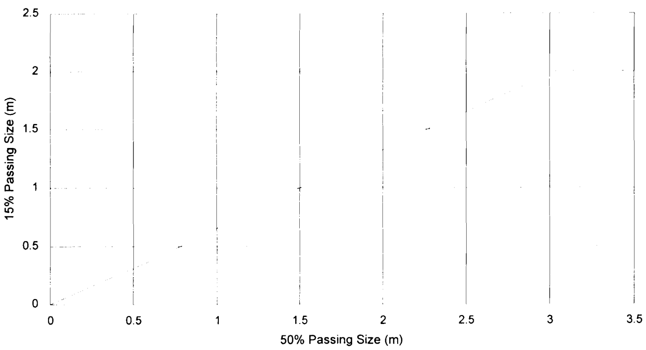

Figure 6. Relationship between 15% Passing Size and 50% Passing Size for Filters.

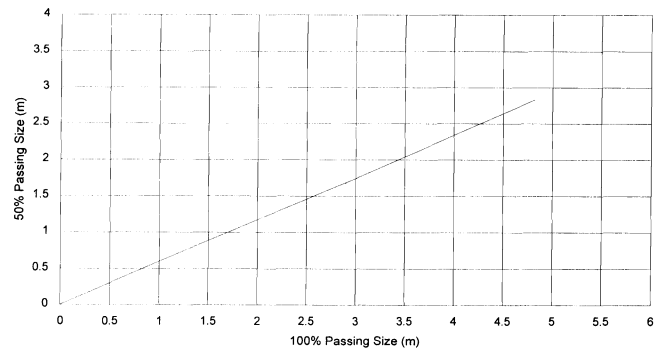

Figure 8. Relationship between 50% Passing Size and 100% Passing Size for Filters.

maximum flow velocity for use in practice are provided in Figures 6–10. In general, the smaller stones initially tend to fill the void spaces between the larger stones and reduces the velocity at the top of the sand layer. Observations of the graded armor tests revealed that the smaller stones in the armor layer could be moved at velocities exceeding about 1.3 m/s. These stones would thus be rearranged under moderate flow velocities to fill any remaining voids in the armor stone layer, thereby increasing its hydraulic stability. Thus, the graded armor system consisting of the mixture of 20 percent (by weight) smaller filter-sized stones and 80 percent (by weight) larger armor-sized stones, yielded sufficient hydraulic stability to the sand layer throughout the tests. No indication of any movement of the sand layer was observed with this armor layer configuration even at the maximum test velocity of 2.0 m/s. Thus, the graded armor system provides sufficient hydraulic stability to the underwater cap system in an environmentally sound and cost-effective manner. Such a cap system is much more easier to construct in deeper water environments, thereby yielding considerable savings without compromising the structural integrity of the system.

However, note that for the tested configuration, even with consideration of the filtering mechanism, the sand would not be locked into place with the smaller stone above due to the large difference in particle sizes. Then, a major operative mechanism would be that the velocity required to move the sand should not propagate down through the graded armor stone system. Therefore, a system which involved seepage with a high enough gradient to induce sand motion might not be stable in the tested configuration.

CONCLUSIONS

An investigation was undertaken to investigate alternate design and construction approaches for underwater capping projects. The results revealed that underwater caps could be constructed in either of two ways: (i) the traditional base layer, filter, armor structure designed to satisfy the well known Terzaghi-Vicksburg criteria, and (ii) a graded armor layer (with filter-sized stones and armor-sized stones mixed in a 20:80 percent by weight ratio) over the base layer. While option (i) is the more common method for traditional shallow marine construction projects, it could be very costly to construct with sufficient accuracy in the deep water environment. Option (ii) is preferable for such applications and can be constructed much more cost-effectively without compromising the structural integrity of the system. Finally, as with any marine civil engineering project, the success and perfor-

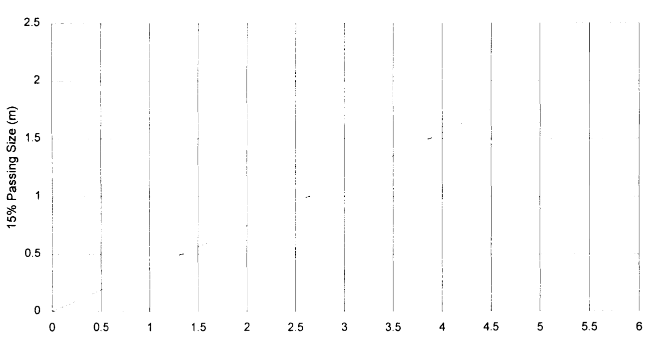

Figure 7. Relationship between 15% Passing Size and 100% Passing Size for Filters.

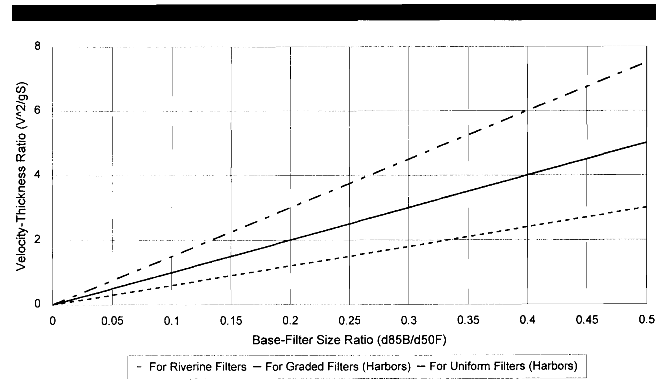

Figure 9. Relationship between Velocity-Thickness Ratio and Base-Filter Size Ratio.

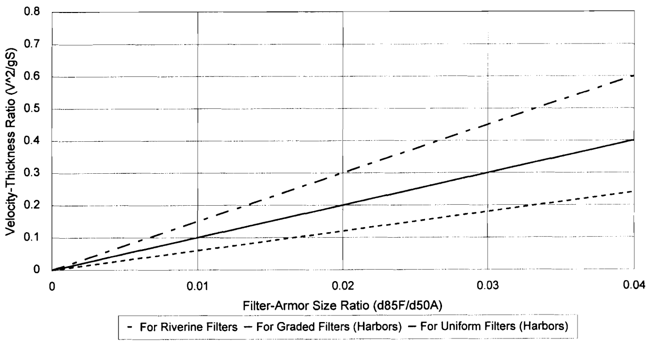

Figure 10. Relationship between Velocity-Thickness Ratio and Filter-Armor Size Ratio.

mance of the cap system in the long-term will depend upon the use of proper construction techniques and quality control/ quality assurance methods during construction.

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