15 Design of Revetments, Seawalls and Bulkheads: Sloped Revetments

Ref: Shore Protection Manual, USACE, 1984
EM 1110-2-1614, Design of Revetments, Seawalls and Bulkheads, USACE, 1995
Breakwaters, Jetties, Bulkheads and Seawalls, Pile Buck, 1992
Coastal, Estuarian and Harbour Engineers’ Reference Book, M.B. Abbot and W.A. Price, 1994, (Chapter 27)

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Design of Sloped Bank Retention Structures

Types of Sloped Bank Structures

Sloped bank protection and earth retention structures come in a great variety of types and forms. On open water such as facing oceans or bays, they are usually called as revetment. Along river banks they are usually known as levees. When used for retention or impounding purposes, they are generally referred to as dike structures. Based on the construction material, they can be roughly divided into the following groups:

1. Stone rip-raps of loose construction.
2. Grouted or cemented slopes of stones, gravels or other aggregates.
3. Manufactured blocks, usually made of concrete.
4. Impervious layers such as asphalt and bituminous paved banks.
5. Gabion structures constructed with metal wires. (Gabions are rectangular baskets or mattresses made of galvanized, and sometimes also PVC-coated, steel wire in a hexagonal mesh. The individual baskets are wired together and filled with 4- to 8-in.-diam stone.)
6. Vegetation on top soil such as hard clay and/or retention structures.
7. Flexible structures such as sand bags, longard tubes. (Longard tubes are patented, woven, polyethylene tubes that are hydraulically filled with sand and available in 40- and 69-in. diameters and lengths up to 328 ft.)

Because of the great variety of types, shapes and construction methods, it is difficult to address the design details. Only general design principles are discussed here.
Design considerations

There are a great variety of sloped bank protection structures. There appears to be an infinite number of possibilities. To select a specific type for local application a number of criteria should be considered.

1. **Strength and vulnerability**: Structure strength is the still the primary concern in design. The vulnerability is measured in terms of the steepness of the damage curve; the steeper the damage curve the more vulnerable the structure. Therefore, for structures with comparable costs, the selection should favor those with milder damage curves.

2. **Flexibility**: This is associated with vulnerability. For earth retention structures, a certain degree of settlement is anticipated. A compliant structure that adjusts to the change is usually less vulnerable to catastrophic failures.

3. **Material availability**: The availability of material becomes a major factor for large size projects. This may dictate the type of structure selected.

4. **Construction**: Easy and fast construction usually translates into lower costs. Also, when construction time is a major factor, construction may dictate the selection of structural types.

5. **Maintenance**: The cost of maintenance should be factored in the at the design stage.

6. **Durability**: The durability of material concerning physical, biological, sea water effects, sunlight effects, etc. should be considered.

7. **Others**: Such as environment impact, recreation value, aesthetic, other utilities, etc.

**Design Procedure** (paragraph references are to EM 1110-2-1614)

1. Determine the water level range for the site (paragraph 2-5).
2. Determine the wave heights (paragraphs 2-6 to 2-11).
3. Select suitable armor alternatives to resist the design wave (Appendix B).
4. Select armor unit size (paragraphs 2-15 to 2-18).
5. Determine potential runup to set the crest elevation.
6. Determine amount of overtopping expected for low structures (paragraph 2-14).
7. Design under-drainage features if they are required.
8. Provide for local surface runoff and overtopping runoff, and make any required provisions for other drainage facilities such as culverts and ditches.
9. Consider end conditions to avoid failure due to flanking (paragraph 2-21).
10. Design toe protection (paragraph 2-19).
11. Design filter and underlayers (paragraph 2-20).
12. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications. Also, due allowance for compaction must be made in the cost estimate.
13. Develop cost estimate for each alternative.

**Stone Revetment and Riprap**

(1) The design practice for stone revetments is basically the same as for rubble mound breakwaters.
(2) Since the primary function is to protect bank and preventing loss of upland material, more care should be exercised in filter design.
(3) Application of geotextile filter is common.
(4) Close attention should be paid to the hydraulic properties of the structure to prevent toe scouring, piping, bank instability and other hydraulically related failure modes.
(5) Pressure build up in the soil behind the structure can result in leaching and loss of soil. Therefore, grading of the stone must be more tightly controlled than for breakwater design.

**Effect of Current on Riprap**

If the revetment is for channel protection, the armor stone along the bank should be able to withstand current induced force without being dislocated. The basic approach is to compute the near bottom flow induced shear stress. The stone stability is then determined by comparing the frictional resistance of embedded stone with the flow induced shear stress.

The following equation can be used to estimate the required stone size:

\[
d \geq 0.7 \frac{V^2}{2g\Delta} \left(1 - \frac{\sin^2 \theta}{\sin^2 \phi}\right)^{1/2}
\]

Eq. 1

where \( V \) is the current velocity (see below),
\( \Delta \) is the specific weight of the submerged stone, \( \Delta = (SG - 1) = (\gamma_s / \gamma_w - 1) \)
\( \theta \) is bank slope.
\( \phi \) is the internal friction angle taken to be about 40° for stone riprap.

The above equation is a modified version of Izbash equation originally meant for horizontal bed. The coefficient 0.7 is the suggested Izbash’s constant for embedded stone. In the original Izbash equation the velocity is not clearly defined. It was suggested to use two-thirds of the mid channel velocity to account for the reduction in velocity near the bank.

An alternative equation is to utilize Shields diagram. A shields parameter is defined as:

\[
\psi = \frac{\tau_o}{(\rho_s - \rho_w)gd} = \frac{u^2}{\Delta gd}
\]

Eq. 2

For a flat bed, to ensure that the armor stone will not move, the shields parameter must be less than the critical shields parameter \( (\psi_c) \). The plot of \( \psi_c \) vs Reynolds number is the well known Shields diagram and can be found in any standard hydraulic text book. For turbulent flow the value of \( \psi_c \) is nearly constant in the range of 0.06. The shear velocity can also related to the mean stream velocity through a variety of empirical equations such as Chezy or Manning equations. The Chezy equation, for instance, gives:
\[ u^* = V \frac{\sqrt{g}}{C} \]  
\text{Eq. 3}

where \( C \) is the Chezy coefficient. Substituting Eq. (3) into Eq. (2) yields,

\[ d = \frac{V^2}{\Delta \psi C^2} \]  
\text{Eq. 4}

This equation can be modified for a sloped bank by utilizing Lane's relationship (see \textit{Erosion and Sedimentation} by P.Y. Julien, p. 126-127):

\[ \frac{\tau_0}{\tau} = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \]  
\text{Eq. 5}

Applying this relationship gives:

\[ d \geq \frac{V^2}{\Delta \psi C^2} \left( 1 - \frac{\sin^2 \theta}{\sin^2 \phi} \right)^{1/2} \]  
\text{Eq. 6}

The similarity between Eq. (1) and Eq. (6) is clear.

\textbf{Riprap Gradation}

\textbf{Gradation Limits Within Layers}

For graded stone revetment, the \( d \) value as determined from stability analysis is generally taken as the required minimum of \( d_{50} \). The gradation should be confined to the limits discussed previously:

1. The lower limit of \( W_{50} \) stone (\( W_{50\text{min}} \)) should be selected based on stability requirements using Hudson's equation and the above stability analysis.
2. The upper limit of the \( W_{100} \) stone (\( W_{100\text{max}} \)) should equal the maximum size that can be economically obtained from the quarry but not exceed 4 times \( W_{50\text{min}} \).
3. The lower limit of the \( W_{100} \) stone (\( W_{100\text{min}} \)) should not be less than twice \( W_{50\text{min}} \).
4. The upper limit of the \( W_{50} \) stone (\( W_{50\text{max}} \)) should be about 1.5 times \( W_{50\text{min}} \).
5. The lower limit of the \( W_{15} \) stone (\( W_{15\text{min}} \)) should be about 0.4 times \( W_{50\text{min}} \).
6. The upper limit of the \( W_{15} \) stone (\( W_{15\text{max}} \)) should be selected based on filter requirements specified in EM1110-2-1901. It should slightly exceed \( W_{50\text{min}} \).

\textbf{Size Relationship Between Layers}

The size relationship between layers should follow the same practice as discussed in Breakwater Design. The hydraulic behavior of porous flows through the layers should be carefully considered as material used in protective layers as well the based material are usually finer than the breakwater case.
Manufactured Blocks

Manufactured blocks are popular revetment material. There are numerous shapes and types. In general, they are all aesthetically pleasing and can be arranged imaginatively. The blocks can be interlocking pieces or held together by grouting, cables, clips, pins or other devices. A typical layer arrangement is shown in Fig. 1. Of course, any of the layers can be omitted or expanded depending upon actual soil condition.

![Figure 1, Typical layer arrangement of block revetment](image1)

Loose Block

Articulated Mat

Figure 2, Types of block arrangements

Analysis of block stability is a difficult task due to the complexity of breaking waves impinging upon an inclined surface that is supported by flexible porous material. Some simple design considerations are given here.

The force analysis for a breaking wave hitting a sloped bank can be roughly divided into the uprushing phase and the downwash phase. The force components during these two phases are described in Fig. 3.

Opinions differ on which force components are most destructive because the material of the blocks and the underlayers play an important role. For instance, (1) if the material is rigid and brittle and the ground is stiff, wave impact loading will likely be the primary cause of structural damage whereas (2) if the blocks are laid tightly together on filter layer with large permeability the uplift forces induced by wave setup and the dynamic uplift is critical to dislodging the blocks.
1. Wave impact force  1. Uplift force due to wave setup
2. Wave dynamic force  2. Uplift force due to wave front
3. Uprush force   3. Downwash force

**Figure 3. Force components due to breaking waves**

### Impact Induced Stress

The impact induced stress in the blocks can be analyzed utilizing the basic approach of fractural mechanics by treating the impact as a line force on a slab supported by a linear damped elastic soil system. In this case, the bending moment \( M \) is

\[
M = \frac{P}{4K^{1/4}} \quad \text{Eq. 7}
\]

with:

\[
K = \frac{Sd^3}{12(1 - \nu^2)} \quad \text{Eq. 8}
\]

in which \( d \) = block thickness

- \( S \) = stiffness modulus of block -16-in\(^2\) = \( EI = \text{lb} \cdot \text{in}^4 \)
- \( \nu \) = Poission ratio = 1/3
- \( c \) = spring constant of subsoil
- \( P \) = wave induced impact loading (see equation 11)

The bending stress for the block is given by

\[
\sigma_b = \frac{M}{Z} = \frac{6M}{d^2} \quad \text{Eq. 9}
\]

where \( Z \) is the section modulus and is equal to \( d^2/6 \). Substituting Eqs. (7) and (8) into (9) and solving for \( d \) gives:

\[
d = \frac{27P^4}{16 \sigma_b^4 c(1 - \nu^2)^{1/3}} \quad \text{Eq. 10}
\]
The wave impact loading is one of the more difficult forces to determine. The following empirical equation is suggested:

\[ P = p_b = 12 \rho g H_s \tan \theta \left( 0.4 H_s \right) \]

Eq. 11

The last term on the right hand side is the assumed width upon which the pressure force acts. The above analysis yields an estimate of the required block thickness under a single impact. In reality, the fracture failure is likely to occur due to repeated impacts. Therefore, the fatigue property of the blocks must be considered.

Uplift Pressure

One of the most common modes of failure is the dislodging of blocks due to uplift pressure. Individual blocks can be lifted out once the uplift force overcomes the block weight and the resistance offered by the friction and interlocking forces. Also, a cluster of blocks can bulk out due to excess water pressure. Again, precise determination of block stability is difficult. A simple case is examined here to serve as a design guideline.

Figure 4, Critical uplift force during maximum downrush

A critical situation of uplift pressure is at maximum downrush as shown in Fig. 4. The pressure is low above the block due to downrush flow while the pressure under the block is high due to the upward velocity and the positive pressure created by the wave setup. This combination creates a relatively high uplift force on the block. The magnitude of this uplift-force is a function of permeability of the top layer and filter layer as well as their layer thickness. A simple analysis will show that the pressure difference in the normal direction over the block is proportional to the so-called leakage length, \( \Lambda \), defined as

\[ \Lambda = \left( \frac{K_F b d}{K_r} \right) \sin \theta \]

where \( K_F \) and \( K_r \) are permeability coefficients of filter and top layer, respectively; \( b \) and \( d \) are the thickness of the filter and the top layers, respectively; and \( \theta \) is slope angle of the revetment.

The larger the leakage length the larger the pressure difference. Thus, leakage length can be used as a measure of block stability. In theory a permeable top layer and a
An empirical relationship on placed block stability based on limited experimental results has been suggested as follows:

\[ \frac{H_s}{\Delta d} = K_c \left( \frac{d}{\Lambda \xi} \right)^{0.67} \]  

Eq. 12

The constant of proportionality \( (K_c) \) is dependant upon the types of construction and has to be determined experimentally. Since \( d \) appears on both sides of the equation and in the leakage length, an explicit solution on \( d \), though theoretically possible, is not recommended.

With all other parameters held constant, it appears that \( d \propto H^{1/2} \). For a revetment of loose stone, the stability formula gives \( d \propto H \), with \( d \) being the stone size. Therefore, the placed block is likely to be more efficient than loose stones. Figure 5 shows the results of the stability analysis for placed blocks compared with the loose stones. Here, one can see that placed blocks form a superior revetment than loose stones.

**Figure 5, Comparisons of stability between placed blocks and loose stones**

**Impervious Revetments**

The common damage modes of impervious revetments, such as asphalt, are

- buckling, sliding and floating under hydrostatic pressure,
- fracture under impact loading,
- piping of underlying material and subsequent collapsing.

For sliding and floating stability under hydrostatic pressure, the following criteria could be applied:
\[ d \geq \frac{fP}{\gamma_s (f \cos \alpha - \sin \alpha)} \]

against sliding which is much the same as rubble structure, and

\[ d \geq \frac{P}{\gamma_s \cos \alpha} \]

against floating.

Analysis of buckling stability is more complicated. The top layer can be treated as a thin shell. Both local and general buckling should be considered. Since the material for the impervious top layer such as asphalt is inhomogeneous and non-elastic a rational analysis is not an easy task. Design is still largely based on experience.

**Geotextiles**

Geotextiles are seeing more and more applications in coastal structures mainly as filter material. There are a great varieties of fabrications. Selecting the proper type for application requires special knowledge and is best to consult with the manufacturers. Some basic considerations are given here.

**Filtering Properties**

The basic design concept of utilizing geotextiles as revetment filter material is the same as granular material discussed earlier in that they should be sufficiently sand tight to retain base material and porous enough to prevent pore pressure buildup. To fulfill sand tightness criterion one may prescribe positive retention of the smallest or a certain threshold sand size or attribute the soil with a self-filtering capacity. To characterize the pore size of the filter material, \( O_{90} \) and \( O_{98} \) are customarily used where the subscript refers to percentage of passing. \( O_{98} \), therefore, almost corresponds to the largest pore size. A number of empirical filtration criteria have been proposed.

1. Heerten (1982) for cohesive soil:
   \( (O_{90} \rightarrow \text{pore size of filter material, } d_{50} \rightarrow \text{sand size}) \)
   \[ O_{90} < 10 d_{50} \]
   \[ O_{90} \leq d_{90} \]
   \[ O_{90} \leq 100 \mu m \]

2. Ingold et. al. (1984):

   A. For \( 1 \leq U \leq 50 \) (\( U \rightarrow \text{coefficient of soil uniformity} \))
   \[ \frac{O_{90}}{d_{50}} = 2U^{(1 - \frac{d_{50}}{U})} \]
   but, \( \frac{O_{90}}{d_{50}} \leq 45 \)
B. For $U < 5$

$$\frac{O_{90}}{d_{90}} \leq 1$$

C. For $U > 5$

$$\frac{O_{90}}{d_{90}} = 2U^{(0.2 - \frac{5}{U})}$$

but,

$$\frac{O_{90}}{d_{90}} \leq 2$$

where $U$ is coefficient of soil uniformity defined as $d_{60} / d_{10}$.

D. For non-cohesive soils containing more than 50% silt by weight,

$$O_{90} \leq 200\mu m$$

(3). The German’s Standard:

A. For soil with $d_{50} \geq 0.06$ mm

“Static loading conditions and non-turbulent flow”: $D_w < 2.5d_{50}$ and $d_w < d_{90}$

in case of high uniformity number $U > 5$ and $d_{50} < D_w < d_{90}$.

“Dynamic load conditions”: $D_w < d_{50}$

B. For soils with $d_{50} < 0.06$ mm and all load conditions:

$$D_w < 10d_{50} \text{ and } D_w < d_{90} \text{ and } D_w < 0.1 \text{ mm}$$

where $D_w$ is the effective opening size of the geotextile filter equivalent to $O_{90}$ except that wet sieving method is used with natural sand of wide grain size range.

(4). The Dutch’s Standard:

<table>
<thead>
<tr>
<th>Flow condition</th>
<th>Base material</th>
<th>Filter rule</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stationary</td>
<td>---------</td>
<td>$O_{90} &lt; 2d_{90}$</td>
</tr>
<tr>
<td>Non-stationary</td>
<td>internal stable ($U &lt; 10$)</td>
<td>$O_{98} &lt; 2d_{85}$</td>
</tr>
<tr>
<td>Non-stationary</td>
<td>internal not stable</td>
<td>$O_{98} &lt; d_{15}$</td>
</tr>
</tbody>
</table>

**Design considerations**

Once the geotextile pore size requirement is determined, a number of potential problem areas should be examined that include the potential of blocking, clogging, pumping and piping. Unfortunately, none of the above can be predicted quantitatively at present.

**Blocking** is a phenomenon where large particles seal the openings in the textile. In this case, the permeability of the filter can decrease dramatically causing pressure build-up and eventual separation of the textile layer from the base material. Geotextile is found to be
susceptible to blocking when the base material is rather uniform and the ratio of d to \( \Omega_{90} \) in the order of unity.

**Clogging** is the trapping of very fine particles in the openings of the textile, also leading to a decrease of permeability and an increase in the pressure gradient. Clogging may result when water is contaminated with chemicals such as iron or detergent or other agents that tend to bind fine particles to the cloth.

**Pumping** occurs when erodible soil in contact with free water is subject to cyclic loading, such as breaking waves. The cyclic loading acts like a pump that forces the erodible material through the filter and results in the progressive loss of bank material. The potential for pumping cannot be established through experiments. A remedial for reducing the pumping hazard is to slightly increase the textile pore size. If pumping persists, geotextile may not be suitable for the specific application and granular type of filters should be considered.

**Piping** is the loss of bank material from underneath the toe of the filter. The potential for piping can be analyzed by such techniques as flow nets or an equivalent that produces the equal potential lines in the soil. End structures are usually installed to increase soil resistance and prevent piping. The types of end structures will be shown later.

**Material Properties**

Geotextiles are manufactured from a variety of synthetic polymers including polyamide, polyester, polyethylene, polypropylene or others. The typical products may be woven, non-woven or knitted. The former two types are used for geotextiles.

A woven fabric is a flat sheet of at least two sets of threads running across each other. The threads can be mono-film fabrics, multi-filament fabrics or tape fabrics made from flat strips. A non-woven geotextile is manufactured by bonding together strands, either mono-filaments or multi-filaments arranged at random, using mechanical, chemical or thermal means. As a single sheet, the woven type has more uniform pore size whereas the non-woven type could contain a range of pore sizes depending upon designs. Both types can then be stretched to form grids or overlaid to form mats. Three dimensional mats can also be produced directly from mono-filaments by profile rollers.

By weight the non-woven type is generally lighter than the woven type. Unit weight ranges from 100-1,000 g/m\(^2\) for the non-woven type and up to 2,000 g/m\(^2\) for the woven type. The lighter fabrics are usually sought for filter applications while the heavier ones for reinforcement and fluid transmission.

The mechanical and chemical properties are clearly important in the selection of geotextile material. Table 1 provides a general comparisons among the most common polymer materials in geotextile applications. Manufacturers should be consulted for applications in unique environment.
<table>
<thead>
<tr>
<th>COMPARATIVE PROPERTIES</th>
<th>TYPES OF POLYMER</th>
<th>Polyester</th>
<th>Polyamide</th>
<th>Polypropylene</th>
<th>Polyethylene</th>
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<tr>
<td>Strength</td>
<td></td>
<td>H</td>
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<td>Elastic modulus</td>
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<td>M</td>
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<td>RESISTANCE TO:</td>
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<td>M</td>
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<td>Detergents</td>
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<td>H</td>
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</tbody>
</table>

H = High, M = Medium, L = Low

**Installation**

Standard widths of geotextile material range from 3, 4, 5 to 5.5 m. Typical roll contains 50-200 m fabrics. The length can be customized to fit the transport facilities and/or the laying methods. To achieve greater dimensions, jointings are made by overlapping, thermo-bounding (heat welding) for non-woven types, threading for woven types, or pinning. Figure 6 shows field sewing and pinning operations.

The correct method for laying geotextile depends upon the specific application. In general, the section should be parallel to the dominant environmental force. For instance, if it used for river bank protection where current is the main force, the sheet should be laid lengthwise along the bank. On the other hand, if it used on the open coast to protect wave-
induced erosion, the sheet should be oriented normal to the bank. The typical arrangements for these two cases are shown in Fig. 7.

(A) Fabric sections being sewn together

(B) Fabric being pinned in place

(c) In situ heat welding operation

Figure 6, Examples of geotextile jointings
Toe protections and transitions

Toe and transition are two of most critical areas in damage initiation. Therefore, special attention to the detail is warranted. A structural toe should be designed to prevent layer sliding, toe scouring and base material piping much the same as rubble mound breakwater design. The design principles are also the same as addressed in the rubble mound toe structure design. Geotextiles are more often used in revetment toe structures. Figure 8 shows a few examples of revetment toes utilizing composite constructions of stones and geotextiles.

Transitions are also common in revetments, mostly for cost savings but also for accommodating different bank slopes. Discontinuities of pressure and permeability are likely to occur at the joints which make them the weak links. Material discontinuity also could lead to the development of seams and eventual separation and loss of material. The above concerns should be incorporated into the design. Overlapping layers are often used to soften the impact.
Other types of structures

As mentioned earlier, there are a great variety of revetment types. New kinds are continually being developed and invented. A few examples are given here.

(A) Gabions

Gabions are simply open containers made of steel wires or flexible polymer grids. They are tied together by wires and filled with quarry stones or other fillers. The wires are either galvanized or PVC-coated. Gabions are flexible structure but are primarily only suitable for mild environment such as banks along streams or inside the bays. Figure 9 illustrates two types of gabions, a box type of steel wire and a tubular type of polymer material.

(B) Armorflex revetment

Armorflex is a prefabricated flexible armored revetment manufactured by Thomas Telford Ltd. London. It is constructed of interlocking concrete blocks into an articulated mat, linked together by wires through precast holes in the blocks. Therefore, it can be laid on the bank as if it were a sheet. The armorflex block and mat is shown in Fig. 10.

(C) Terrafix revetment system

The terrafix revetment system is another type articulated concrete mat manufactured in Germany. It consists of two components: a heavy needle punched nonwoven geotextiles serving as filter layer and the interlocking concrete block mat as the armor layer. Figure 11 shows the block construction and the installation operation.
Figure 9, Examples of gabion structures

(A) Steel wire box gabions

(B) Polymer tubular gabions
Figure 10, Armoflex blocks and mat construction

(A) Terrafix interlocking blocks
(B) Installation of terrafix revetment

Figure 11, Block construction and the installation operation
Environmental-friendly design

(A) Interlocking concrete grids serve as base for plants  
(B) Salt water resistance grass planted on top

**Figure 12, A form of environmentally friendly grass on block design**

Environmental-friendly design has become the catch word in recent years. Substantial research effort has been spent mainly in the direction of utilizing vegetation as bank protection, or mixing vegetation with compliant structures. At present, erosion prevention by plants and grasses is only suitable for mild environments. In a hostile environment such as along an open coast, vegetation can only serve as a secondary defense and as a sand trapping device. This technique has found more success along river banks.