Texas A&M Transportation Institute

# Riprap Filters and Stability of Riprap-Covered Slopes: Technical Report

Technical Report 0-7091-R1

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16. Abstract

This research report documents the outcome of a study performed on riprap filters (granular and geosynthetic) and the stability of riprap-covered slopes. Erosion of the underlying soil through the riprap is likely to happen if there is a sufficient gap in grain size between the soil and the riprap. This gap is often the case, which is why a filter is needed. By combining existing literature, survey results, and laboratory testing in the Erosion Function Apparatus, researchers observed that a filter increased the efficiency of the system significantly. The two main types of filters, granular and geosynthetic, were used and are discussed in this report. Each one has its advantages and disadvantages, but generally, geosynthetic filters are preferred in current practice. Regarding the design of filters, a step-by-step procedure is discussed in detail in this report. The installation and pricing of filters are also discussed: geosynthetic filters are typically less expensive and easier to install; however, the challenge with geosynthetic filters arises when installing them underwater.

Placing riprap on slopes can impact the stability of the slope. To study this impact, slope stability simulations were performed using the RocScience Slide 2-Dimensional (2D) software for geotechnical slope stability analysis and relying on the 2D limit equilibrium method (Bishop's simplified method). The results of both analyses led to the same conclusion: When a steep slope (i.e., steeper than 1.5:1) meets the 1.5 factor of safety requirement and riprap is installed on it, the riprap itself is very likely to fail because of the steepness of the slope, but this failure is unlikely to occur for flatter slopes (i.e., 2:1 or less). The stability of the riprap itself is uncertain when high-velocity water flows along a riprapped slope. In other words, when the slope angle increases, the critical erosion velocity of the riprap is likely to decrease. A total of 43 case histories were collected, with 31 directly related to riprap. The most common failures were found to be riprap washing away or tumbling down the slope and scour at the toe of the slope that undermined the stability of the riprap-covered slope. Many other causes are also discussed in detail in this report.

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# RIPRAP FILTERS AND STABILITY OF RIPRAP-COVERED SLOPES: TECHNICAL REPORT

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This report is not intended for construction, bidding, or permit purposes. The engineer (researcher) in charge of the project was Jean-Louis Briaud, P.E. #48690.

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# CHAPTER 1: LITERATURE REVIEW

## 1.1. RIPRAP FILTERS

### 1.1.1. Filter Importance

Scour at bridges is the leading cause of bridge failure (Briaud, 2008). One common countermeasure for bridge scour is riprap, which is rocks placed around piers or on abutment slopes to shield the underlying soil from the eroding effects of moving water. Among the objectives of this current research are to determine if a filter is necessary between the riprap layer and the underlying soil and to develop a set of guidelines for riprap filters (granular and geosynthetic), including design and installation. Figure 1 shows a sketch of what a typical riprap, filter, and underlying soil system look like. A filter's primary function is to protect the soil below the riprap rock from eroding through the riprap.

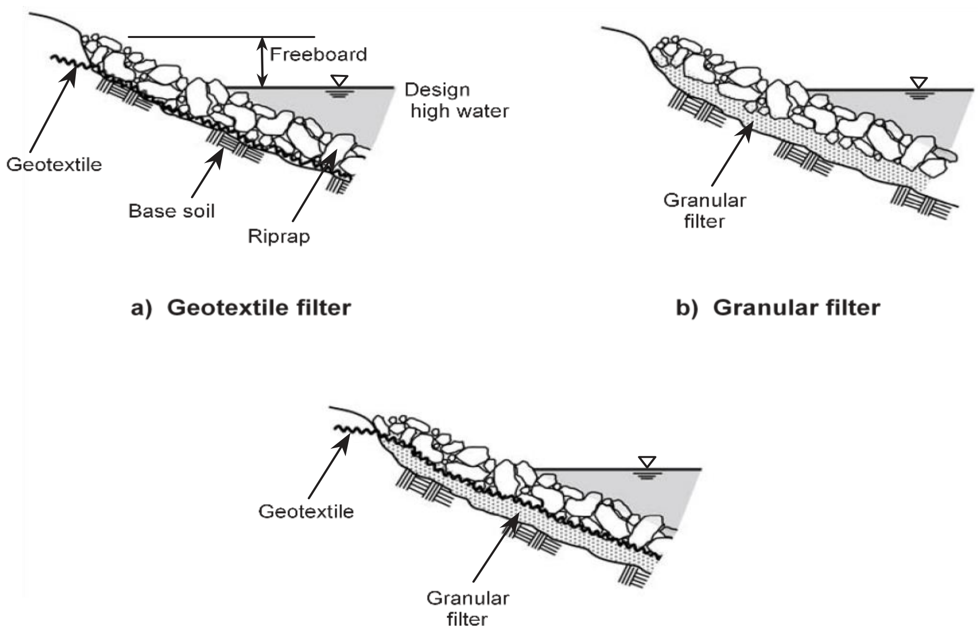


**Figure 1. Riprap, Filter, and Native Soil System (Lagasse et al., 2006).**

The filter should meet two criteria to successfully fulfill its role: First, it should efficiently prevent erosion of the soil beneath the riprap; second, it should allow efficient drainage.

The soil particles underneath the riprap may erode through the riprap because of the voids and gaps in particle sizes between the small soil particles and the large riprap rocks. According to Arneson et al. (2012), riprap should have a granular or geosynthetic filter between the rock and the subgrade to prevent loss of the finer subgrade material, whether on the bed or the bank. The filter should prevent internal erosion by forming an intermediate grain-size layer that still allows drainage. If drainage is not allowed, clogging of the flow is created by small soil particles, and water pressure may build up in the soil and may lead to failure. Both erosion through the riprap and pressure buildup in the underlying soil must be taken into consideration. These different cases are illustrated in Figure 2.

According to the Province of British Columbia Ministry of Environment, Lands and Parks (2000), when riprap is placed on banks composed of sand and fine gravel, a filter is necessary to avoid loss of bank material. Filters also assist in maintaining intimate contact between the revetment and the base soil by creating a stable interface. Depending upon the internal stability of the soil, several processes can occur over time at this interface. The filter pore size and the base soil stability influence these processes. Absence of filter is one of the main reasons behind riprap failures, according to case studies cited by the Federal Highway Administration (FHWA). In some scenarios, it is obvious that a filter is needed. For example, according to Brown and Clyde (1989), when riprap is placed on non-cohesive soil subject to subsurface drainage, a filter is required. Also, Lagasse et al. (2006) mentions that a filter layer is typically required at bridge piers. On the other hand, Soil Conservation Service (1989) states that, while taking into consideration seepage gradients, a filter is not necessary if no piping potential exists. Overall, filters are necessary if there is a large gap between the mean grain size of the underlying soil and the mean grain size of the riprap.

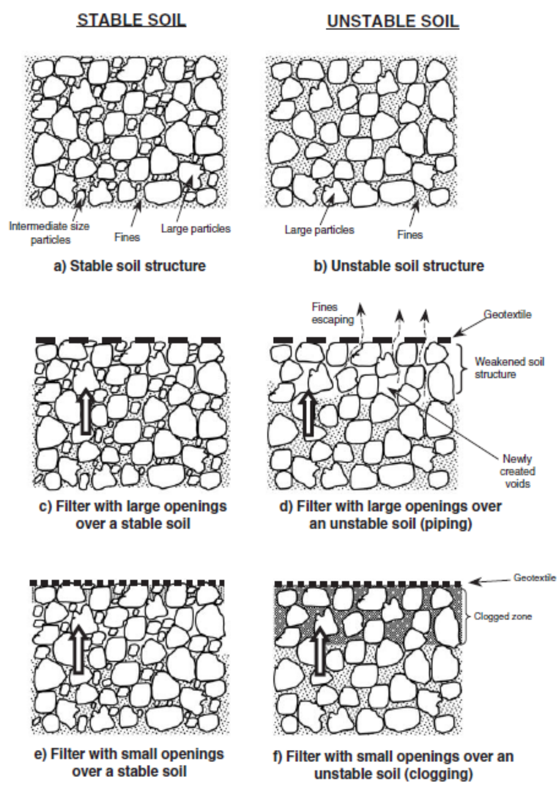


**Figure 2. Different Configurations of Soil and Filter Compatibility (Lagasse et al., 2006).**

### 1.1.2. Types of Filters

Two main types of filters are widely used across the industry: granular and geosynthetic. A geosynthetic filter is made of a permeable geotextile fabric. These fabrics, when placed on top of the soil, have the ability to separate, filter, reinforce, protect, and drain. Typically made from polypropylene or polyester, geotextile fabrics come in three basic forms: woven, needle punched, or heat bonded.

A granular filter is a set of particles with a specific grain size distribution—usually a combination of sand, gravel, and crushed rock particles—designed to form an intermediate grain size layer between the underlying soil grain size and the riprap rock grain size.

When it comes to deciding whether to use a granular filter or a geosynthetic filter, many factors must be considered, including site accessibility, construction logistics, contractor, price, and availability of material. However, there is no specific published guideline describing the performance of each type under specific conditions and highlighting which filter is better in terms of operating efficiency and cost. The guidance from the Province of British Columbia Ministry of Environment, Lands and Parks (2000) states that a geotextile filter is less costly and easier to install in certain circumstances. NCHRP Report 568 (Lagasse et al., 2006) strongly recommends the use of geosynthetic filters at bridge abutments in riverine systems where dune-type bed forms may be present during high flows and where the abutment and/or abutment riprap apron extend into the main channel.

Geotextiles are increasingly being used as the filter material of choice for riprap installations. It is typically assumed that if the geotextile survives the loads and stresses during initial construction, it will be fine for the remainder of its service life. Lagasse et al.'s (2006) study also distributed a questionnaire that compared the two filters; most responses (23 out of 33) indicated that geosynthetic filters were required with riprap installations. Comments generally indicated that geosynthetic are the preferred filter material. Brown and Clyde (1989) indicated that a geosynthetic filter is preferred over a granular filter. Thus, overall, the geotextile has become preferred over the granular filter for riprap applications.

### 1.1.3. Overview of Filter Design

NCHRP Report 568 (Lagasse et al., 2006) is the main reference used in filter design for this report. It lists the parameters involved in filter design, along with the detailed procedure to design a filter (geosynthetic and granular). Before designing the filter, the riprap size must be selected. Stone riprap resists erosion through a combination of stone size, weight, durability, and proper design of the filter (Province of British Columbia Ministry of Environment, Lands and Parks, 2000).

- For geosynthetic filter design, the steps include:
  - Step 1: Site Investigation.
  - Step 2: Particle Retention Criterion.
  - Step 3: Permeability of the Geotextile.
  - Step 4: Geotextile Mechanical Properties.
  - Step 5: Long-Term Clogging.
- For granular filter design, the steps include:
  - Step 1: Site Investigation.
  - Step 2: Underlying Soil Index Parameters.
  - Step 3: Granular Filter Index Parameters.
  - Step 4: Underlying Soil Filter Grain Size Matching.
  - Step 5: Filter Riprap Grain Size Matching.
  - Step 6: Thickness and Permeability.

The details of these steps are described in later sections.

#### 1.1.4. Filter Installation

A challenging part of the riprap protection layer is the proper installation of the filter because this installation is sometimes difficult, particularly underwater. No specific methodology exists for the installation of the filter. However, some organizations have their own steps and guidelines. Expenses associated with a quality placement include expensive equipment and sometimes divers. Ephemeral streams are the most suitable sites to install filters because the riprap and filter can be easily installed with the highest level of efficiency when there is minimal to no flow.

## 1.2. RIPRAP-COVERED SLOPES

It is common to find riprap being placed on slopes, especially when protecting abutments from potential scour, as shown in Figure 3. In this regard, this section will discuss the main parameters and aspects that govern the stability of riprap when placed on slopes—primarily hydraulic conditions and geometrical considerations (especially slope angle). Some countermeasures that improve the stability of riprap when placed on slopes will also be discussed.

**Figure 3. Riprap-Covered Slope (Ohio Department of Transportation [DOT], n.d.).**

### 1.2.1. Flow Conditions in Riprap

Whether it is placed for abutment or pier protection, riprap will be subjected to fluid flow. Most reported failures occur during floods and high-velocity events, which means that it is crucial to understand the behavior of the flow of fluids (mainly water) on riprap and how it can induce failure, especially when riprap is placed on a slope. Flow through riprap on a slope is a function of the gradation of the riprap, slope angle, and discharge velocities.

At slow velocities and low water level, the flow is mostly interstitial within the riprap itself. Failure of the riprap under these conditions is very unlikely, especially if the water level does not reach the top of the riprap layer. As the flow conditions become more severe and the velocity increases and the water level rises, turbulence of the flow increases, water reaches the top of the riprap, and a lifting force is created that may have enough intensity to move the riprap rocks until failure occurs through riprap movement. During this process, some displaced rocks might settle in other locations within the riprap cover.

Stone dimensions may be successfully designed to resist failure based on local experience, empirical guidelines, or hydraulic relationships that predict stable riprap sizes. Figure 4 and Figure 5 show the required $d_{50}$ size of the riprap as a function of the shear stress applied by the flow and the flow velocity, respectively.

Bank slope and stream characteristics must also be considered. Indeed, when the riprap is placed on a slope, it does not offer the same velocity resistance as when placed on a flat surface. Figure 6 shows a graph with a parameter combining the $d_{50}$ size of the riprap stones and the coefficient of uniformity $C_u$ (defined as the ratio of $d_{60}$ over $d_{10}$) on the vertical axis and the flowrate unit discharge $q$ on the horizontal axis for different values of the slope $S$ of the bank. The unit discharge is the flow within a unit width of the river cross section. The slope is expressed as the ratio of the vertical over the horizontal distance on the slope; a 2 to 1 slope corresponds to $S = 0.5$. This correlation is based on a riprap stone having a friction angle of $\Phi = 42$ degrees. Figure 6 can be used to properly select the riprap stone size when considering the bank slope angle, according to Frizell et al. (1997).

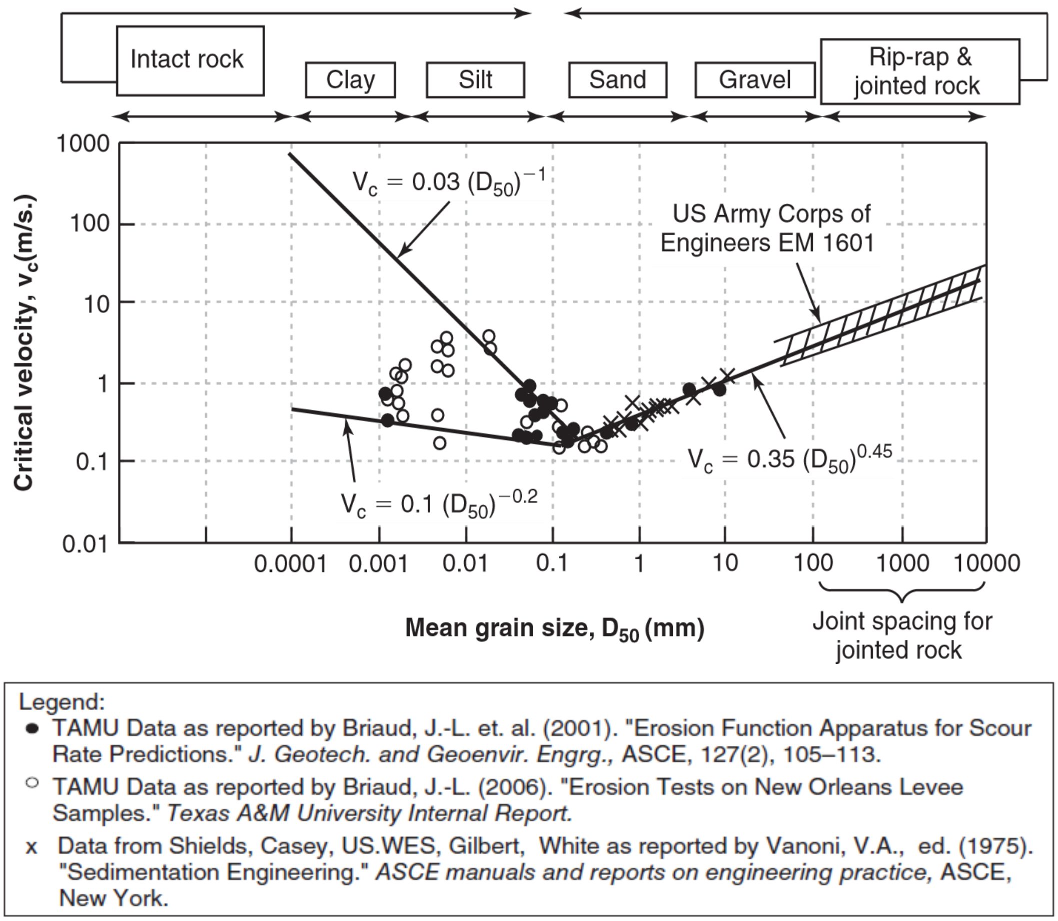


**Figure 4. Required $d_{50}$ in Function of the Critical Velocity (Briaud, 2008).**

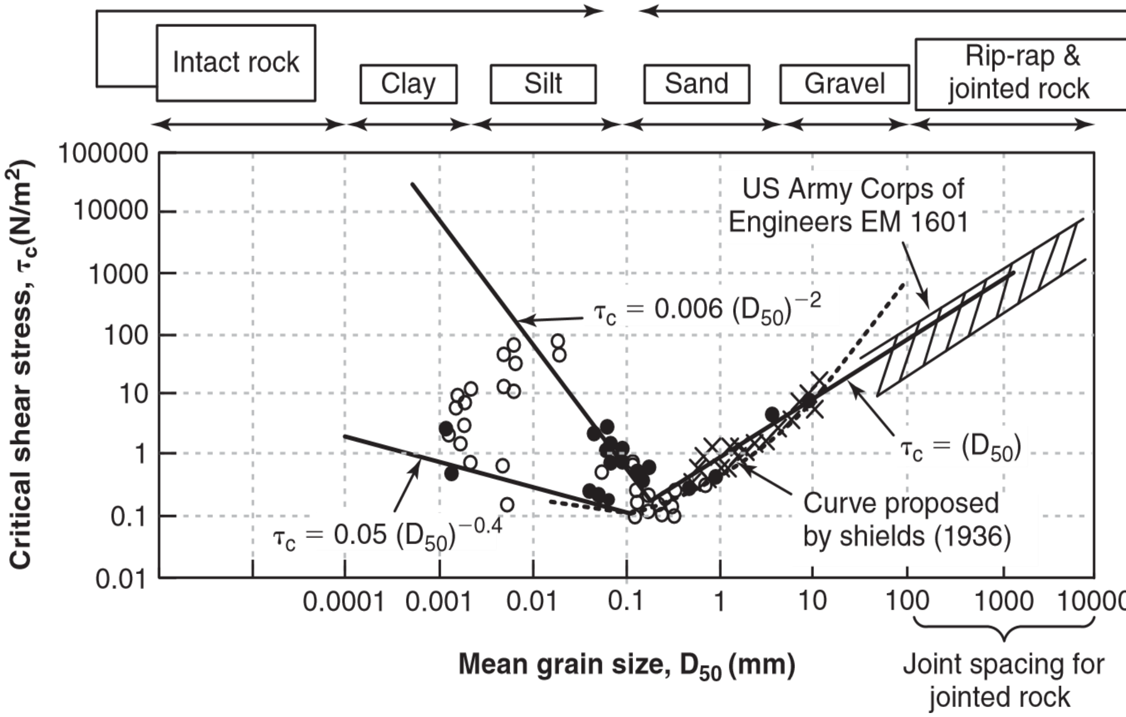


**Figure 5. Required $d_{50}$ in Function of the Critical Shear Stress (Briaud, 2008).**

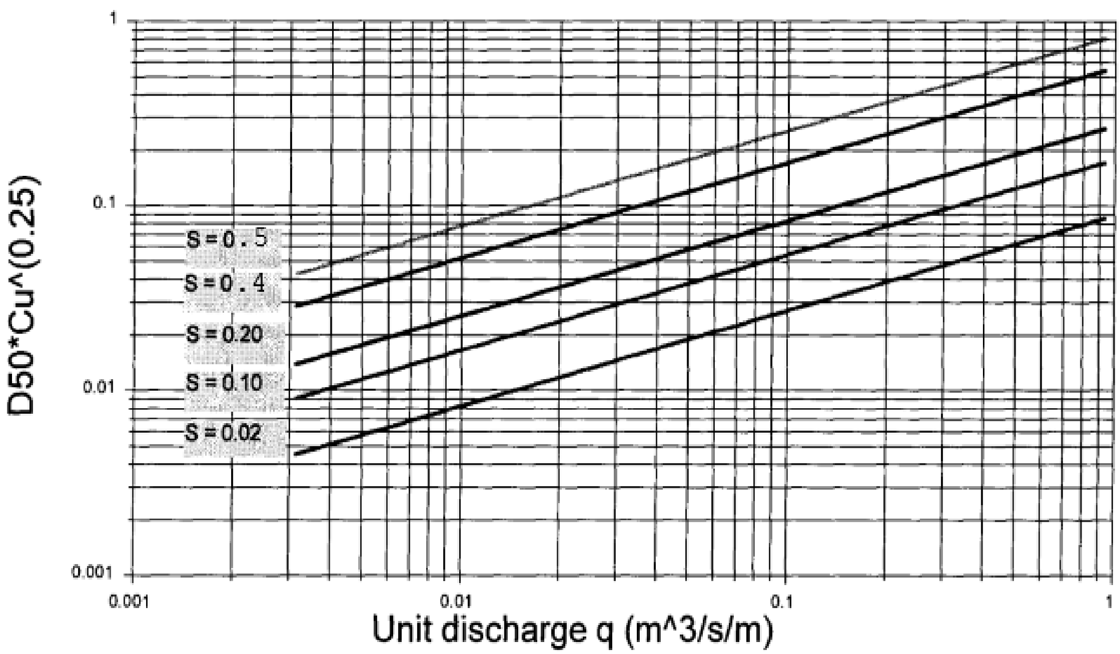


**Figure 6. Correlation between $d_{50}$ and Flowrate in Riprap (Frizell et al., 1997).**

### 1.2.2. Riprap Slope Geometry Restriction

In most of the failures of riprap-covered slopes, the main cause of failure is that the slope was too steep. Thus, the geometry of the slope plays a crucial role in the stability of the system. Moreover, the toe of the slope is identified as being the most sensitive part of the system.

Soil Conservation Service (1989) recommends that the slope on which the riprap is placed not be steeper than 2:1 under normal conditions. A flatter slope may be necessary, depending on the slope factor of safety, when considering groundwater seepage or the possible case of a rapid drawdown. In cases where slopes steeper than 2:1 cannot be avoided, increasing the median stone size and riprap thickness should be considered. According to Soil Conservation Service (1989), a 1:1 slope can be tolerated only if the thickness of the riprap is increased by 20 percent; for a 1.5 to 1 slope, the thickness needs to be increased by 10 percent. In addition, the minimum $d_{50}$ allowed when placing riprap on a slope steeper than 2:1 is 4 inches.

Another factor that must be considered is the angularity of the rock, the more angular the rock, the higher the angle of repose. If rounded stones are used, they should be placed on relatively flat slopes (not exceeding 2.5:1), and the $d_{50}$ size should be increased by 25 percent, as should the riprap thickness (Soil Conservation Service, 1989). Figure 7 gives recommendations for the steepest slope angle for riprap given the riprap angle of repose and the $d_{50}$ size of the riprap stones when taking into consideration the stone shape. Figure 8 illustrates the riprap critical velocity as a function of the $d_{50}$ size of the riprap stones, the riprap stone weight, and the slope angle. For example, a critical velocity of 10 ft/s corresponds to 1-ft-size riprap stones on a 2:1 slope. The stability of the slope loaded with the riprap layer is another design consideration that must be addressed separately by conventional slope stability analysis.

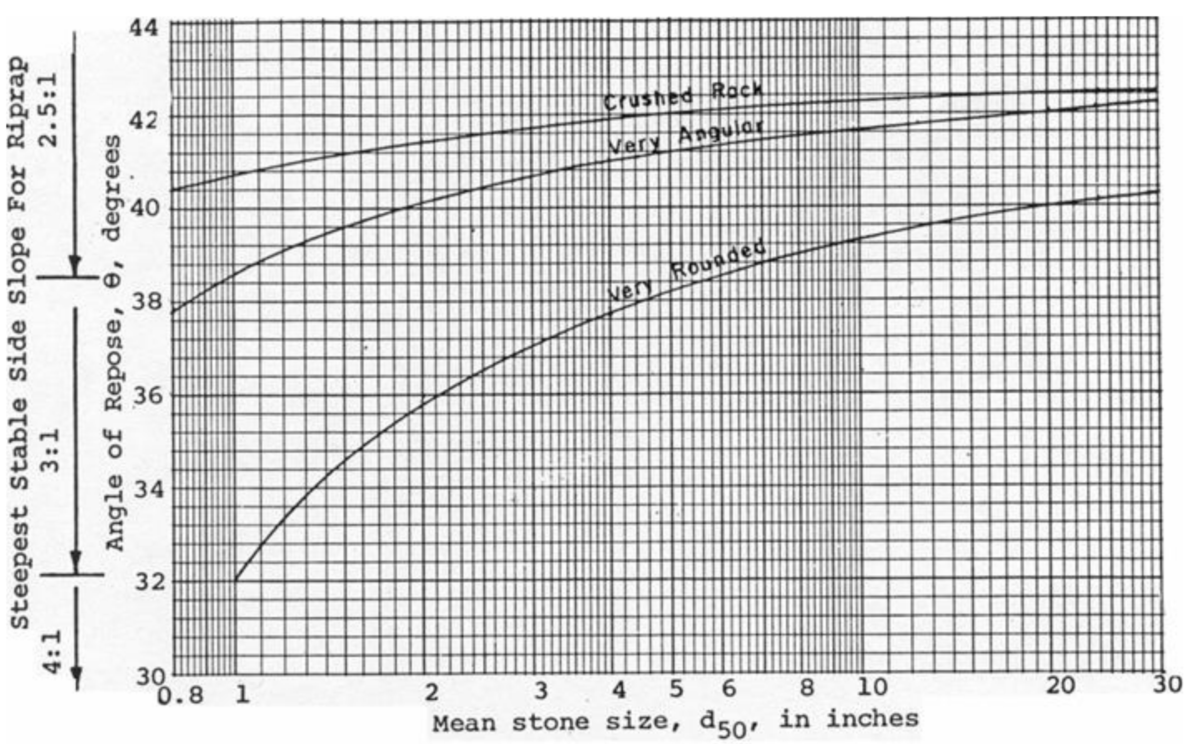


**Figure 7. Correlation between Angle of Repose, $d_{50}$, and Slope Angle (Soil Conservation Service, 1989).**

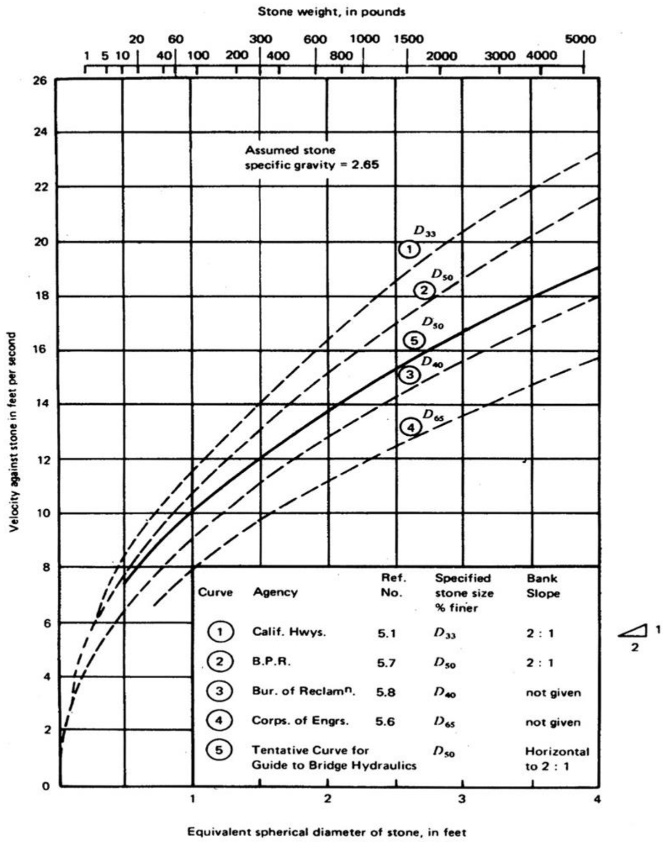


**Figure 8. Correlation between $d_{50}$ and the Critical Velocity, Riprap Stone Weight, and Slope Angle (Barkdoll et al., 2007).**

### 1.2.3. Countermeasures to Stabilize the Riprap-Covered Slope

Some countermeasures have been developed to improve the stability of riprap-covered slopes. The most sensitive region is the toe of the slope; therefore, most of the countermeasures that are described below will include stabilization methods at the toe.

The Province of British Columbia Ministry of Environment, Lands and Parks (2000) presents a list of the main countermeasures used. The most effective method is to extend the slope excavation to below the expected scour levels and backfill with riprap stones or unerodable material. Sometimes this process can be uneconomical and impractical if the scour depth is too deep. Sometimes this technique is applied to a depth above expected scour levels while placing the riprap on an unerodable layer in the soil if such material is detected. A flexible launching apron can be laid horizontally on the riverbed at the toe of the slope so that when scour occurs, the apron will settle and cover the scour toe and provide stability. A toe wall (concrete or sheet pile) can be installed from the toe of the slope down to an unerodable material or to the anticipated scour depth if no unerodable material is found. Toe trenching and backfilling with rocks is also a common countermeasure. For small and steep streams, the entire streambed can be paved with riprap or other material.

Table 1 and Table 2 show different causes and modes of failures with their effects on the overall system. Figure 9 (New York Standards and Specifications for Erosion and Sediment Control, 2005) shows a typical schematic of a riprap-covered slope.

**Table 1. Typical Failures and Protection Techniques (Lagasse et al., 2006).**

| Cause | Solution |
|---|---|
| Inadequate size of riprap | Larger riprap |
| Impingement of current directly upon riprap rather than having flow parallel to riprap | Heavier stones, flatten riprap slopes, redirect flow |
| Channel degradation | Provide a volume of reserve riprap at the revetment toe |
| Internal slope failure (slump) | Reduce the riprap slope angle |
| Riprap with high percentage of fines causes washing out of the fines | Follow gradation specifications |

**Table 2. Failure Modes, Detection, and Protection Methods (Lagasse et al., 2006).**

| Failure Modes | Effects on Other Components | Effects on Whole System | Detection Methods | Compensating Provisions |
|---|---|---|---|---|
| Translational slide or slump (slope failure) | Disruption of armor layer | Catastrophic failure | Mound of rock at bank toe; unprotected upper bank | Reduce bank slope; use more angular or smaller rock; use granular filter rather than geotextile fabric |
| Particle erosion (rock undersized) | Loss of armor layer, erosion of filter | Progressive failure | Rock moved downstream from original location, exposure of filter | Increase rock size; modify rock gradation |
| Piping or erosion beneath armor (improper filter) | Displacement of armor layer | Progressive failure | Scalloping of upper bank; bank cutting; voids beneath and between rocks | Use appropriate granular or geotextile filter |
| Loss of toe or key (under designed) | Displacement or disruption of armor layer | Catastrophic failure | Slumping of rock, unprotected upper bank | Increase size, thickness, depth, or extent of toe or key |

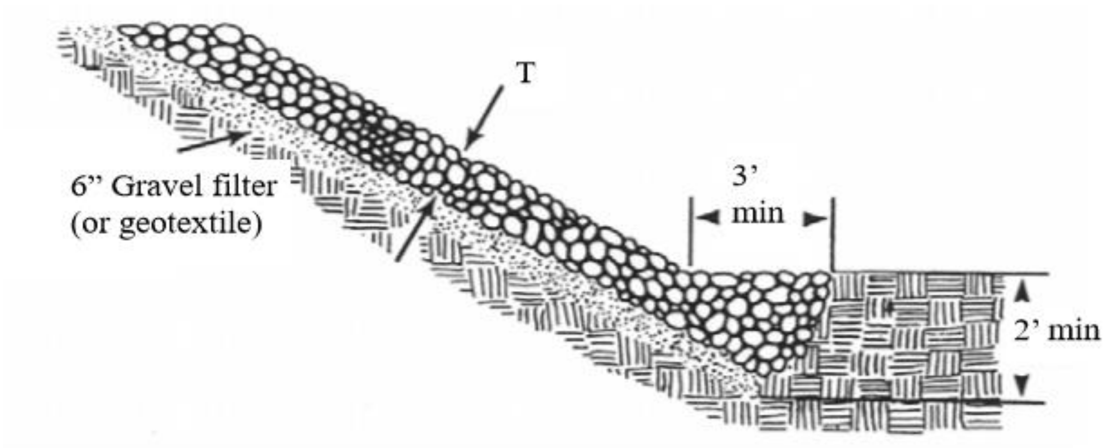


**Figure 9. Riprap-Covered Slope Protection (New York Standards and Specifications for Erosion and Sediment Control, 2005).**

## 1.3. FAILURE MODES

The four main types of riprap failures are particle erosion, translational slide, slump, and modified slump.

**Particle erosion** is considered the most common type of riprap-covered slope failure. Erosion (displacement) of the riprap stones at the toe of the slope will eventually lead to a failure of the riprap along the slope until the stream velocity is not high enough to move the remaining riprap stones lying near the channel (Figure 10). Figure 11 shows a riprap-covered slope failure due to toe erosion in Texas.

**Translational slide** is a failure consisting of downslope movement of riprap stones along a failure surface nearly parallel to the slope (Figure 12). If the moving mass is not significantly damaged and deformed, this mechanism can be referred to as a block slide. This phenomenon is governed by the variations of shear strength along the interface between the riprap and the underlying soil or filter. This mechanism is typically triggered when scour occurs at the channel bed and undermines the toe of the riprap; if erosion at the toe continues after failure occurs, the downslope movement of the riprap can continue indefinitely until the shear resistance along the interface is able to support the gravitational force driving the riprap stones down the slope. Another mechanism which can trigger a translational slide can be excess pore pressures in the underlying soil; these excess pore pressures reduce the frictional resistance at the interface. They can be due to high rainfall, floods, rapid fluctuation of water levels (rapid drawdown), or improper internal drainage. The geosynthetic filter fabric is a good candidate for a failure plane in such events, and this risk must be considered when conducting a slope stability analysis. Spacings in the upper part of the riprap cover can usually be seen in the early stages of translational slope failures.

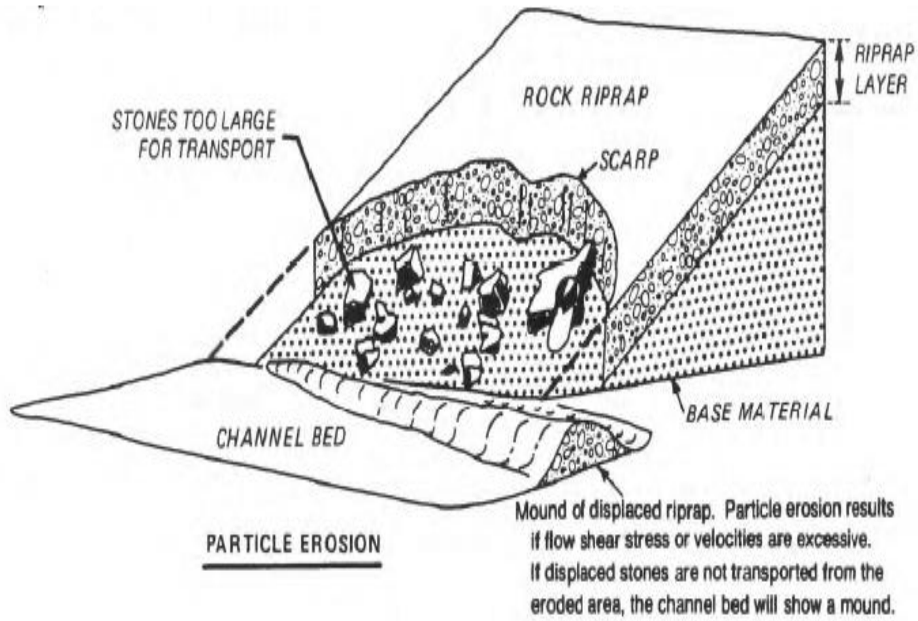


**Figure 10. Failure of Riprap Slope Due to Erosion (Blodget & McConaughy, 1986).**

**Figure 11. Toe Erosion Slope Failure (Delphia, 2018).**

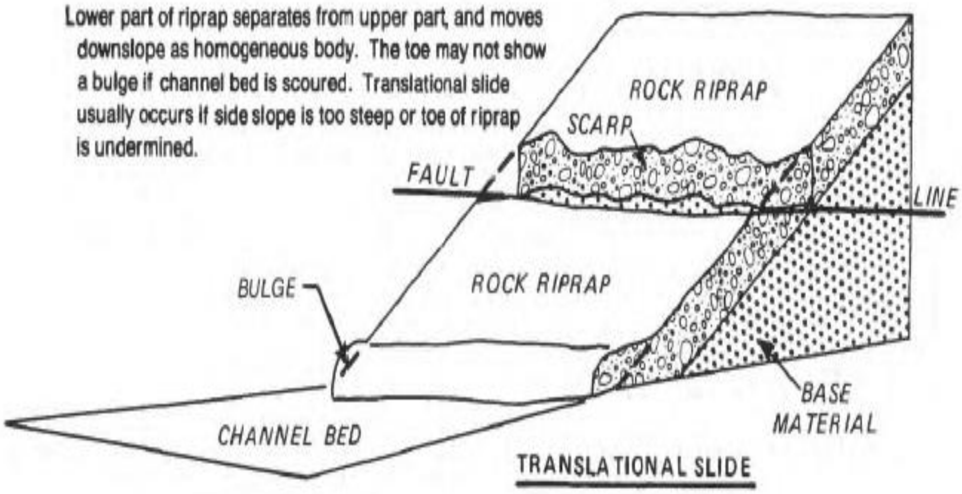


**Figure 12. Translational Slide Failure of Riprap-Covered Slope (Blodget & McConaughy, 1986).**

**Slump** is a rotational-gravitational movement of material along a concaved failure surface (Figure 13). This failure mode is related to the failure of the underlying soil, not the riprap itself. The displaced material (riprap and underlying soil) rotates while moving downslope. The main cause behind this mode of failure is the presence of heterogeneity in the underlying soil along which the applied shear stress becomes equal to the soil shear strength. The steeper the slope is, the more likely the failure. Excess loading on the slope caused by the riprap cover can trigger such a mechanism; this process should be considered in slope stability analysis by considering the riprap as a distributed load. The first feature observed in slump failure is movement of the underlying soil near the top of the slope.

**Modified slump** is similar to the slump failure mechanism described above. It is a rotational slide along a concaved failure surface, but the failure plane is located within the riprap and not in the underlying soil; no failure occurs in the latter (Figure 14). Because the failure is in the riprap, the circle associated with the failure surface is usually shallower than in the slump failure. A factor affecting this mechanism can be the median size of the riprap, $d_{50}$, which may have been properly selected, but movement of certain stones due to poor gradation can lead to a potential failure plane within the riprap. The slope being too steep is also a factor contributing to this failure mechanism.

Note that in all the types of failure of riprap-covered slopes, the slope angle is a major factor since it plays a role in every failure mechanism described above.

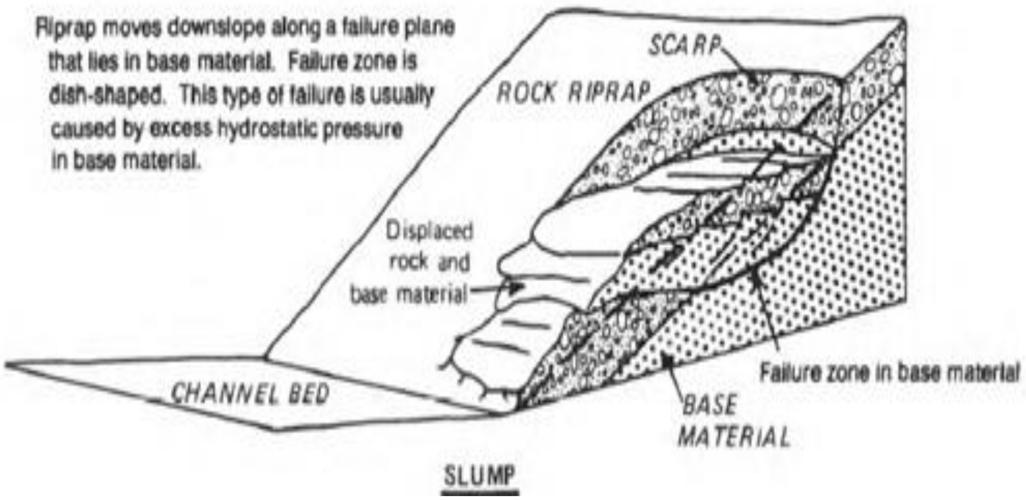


**Figure 13. Slump Failure of Riprap-Covered Slope (Blodget & McConaughy, 1986).**

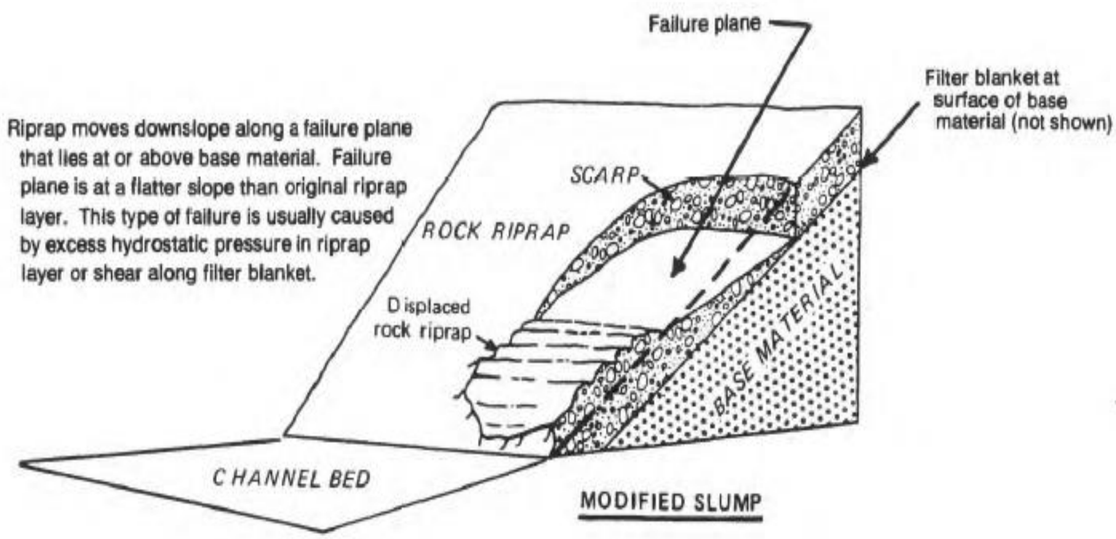


**Figure 14. Modified Slump Failure of Riprap-Covered Slope (Blodget & McConaughy, 1986).**

## 1.4. SURVEY

A survey was sent to all DOTs across the United States and to selected relevant committees around the globe. Thirty-six responses came in—31 from DOTs and five from the selected geotechnical engineering committees concerned with this topic. What follows is the list of questions as well as a summary of the answers.

### 1.4.1. Filter-Related Questions

**1. What standards, guidelines, and/or specifications do you follow for granular and geosynthetic filter design? Please give us the references.**

The most common standards used for filter design are the FHWA HEC 23 (Lagasse et al., 2009) and AASHTO M288 (Suits & Richardson, 1998). In addition, many DOTs and committees use their own local published guidelines, such as are published by Indiana, Louisiana, South Carolina, Oregon, Virginia, Georgia, California, Washington, New Jersey, North Dakota, Illinois, Mississippi, New Hampshire, South Dakota, Alaska, Sweden, and Holland. Other guidelines that were mentioned are listed below:

- FHWA HEC 15 (Kilgore & Cotton, 2005).
- FHWA-NHI-07-092 (Holtz et al., 2008).
- NCHRP 568 (Lagasse et al., 2006).
- Publication 15M, Design Manual 4, PP Section 7 (2019).
- Publication 218M, Bridge Design Standard, BD 667M (2014).
- Publication 408 Construction Specification, Section 735 Class 4 (2020).
- Caltrans Design Information Bulletin No. 87-01, Hybrid Streambank Revetments: Vegetated Rock Slope Protection (2014).
- U.S. Army Corps of Engineers Guidelines.
- WG 4 Guidelines for the design and construction of flexible revetments, Pianc (1987).
- Engineering Fabric.

**2. What percent of the time do you use a filter for stone riprap?**

Figure 15 summarizes the responses. It shows that 78 percent (28/36) of the respondents either use a filter all the time or at least 80 percent of the time. This answer illustrates the importance of the filter.

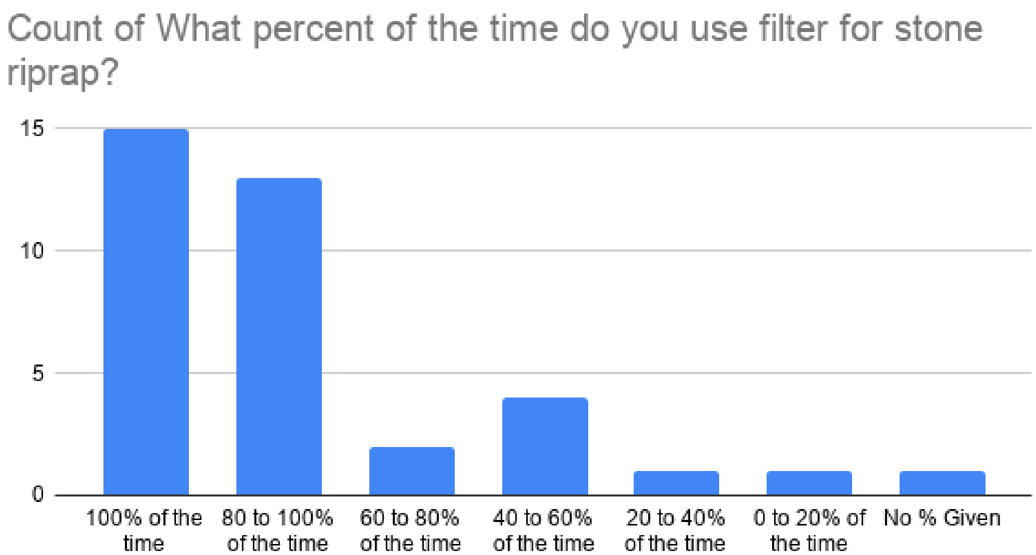


**Figure 15. Count of Percent of Time Filter Is Used with Stone Riprap.**

### 3. Do you prefer a granular filter or a geosynthetic filter? Why?

Regarding the option to choose between granular and geosynthetic filters, 80 percent of the respondents said that they prefer geosynthetic filters because it is more cost-efficient, easier to install, and provides satisfactory performance with more uniform properties. On the other hand, the main reason that some other entities prefer granular filters is because geosynthetics are hard to install underwater, and their strength properties can be undermined during installation. Some respondents did not have a rule of thumb answer and mentioned that it depends on the application, environment, and leading design parameters.

### 4. How is the filter (granular or geosynthetic) installed underwater?

Without question, this situation is the most challenging in which to install filters below riprap. The most common response was that the area should be dewatered or that the flow should be diverted. Cofferdams were mentioned as a technique for doing so. Waiting until the area has been naturally dried was also mentioned. When this is not possible, the geosynthetic filter can be weighed down before placement, and the operation should take place when the stream velocity is very low. Some respondents mentioned that it depends on the contractors. Other techniques mentioned are the use of fastener pins, divers, and excavators with divers to control. Geotextile containers consisting of sand enclosed and sealed inside a geosynthetic filter fabric and placed under the riprap were mentioned for deep water installations. One respondent from The Netherlands mentioned that a fascine mattress had been used recently on top of a geosynthetic filter placed by a crane with a special tool to unroll it on the bottom (0–3 m deep water). The guidelines mentioned for filter installation are NCHRP 24-42, FHWA guidelines, and Caltrans internal construction guidance.

5. **Can you share your experience and/or information comparing the behavior of granular and geosynthetic filters?**

Most respondents preferred one type of filter and had experience with only one type; therefore, they could not compare the performance. Some of them mentioned that geosynthetic filters do not wash away as quickly as granular filters during high flows and protect against scour longer. On the other hand, some dislike the idea of plastic washing down the stream and pointed out that geosynthetic filters tend to form a sliding plane when dealing with slopes. One respondent pointed out that granular filters are better because they are made of natural material and usually self-adapt to the environment.

6. **Can you share any experience and/or information comparing the behavior of riprap with and without filters?**

As mentioned in previous questions, a filter is used most of the time with riprap, so comparative data of the performance of riprap with and without filters are limited. When these data exist, all the respondents said the performance of riprap for scour applications is better when a filter is present. The absence of a filter can lead to the washing out of fine particles and induce failure. It was also mentioned that, in the event of a toe failure with the riprap slumping down, the filter fabric can temporarily aid in retention of granular soils by acting as a barrier from water turbulence. Some of the respondents gave different perspectives on this question, which included the following: the riprap rocks often slide off the fabric and cause slope failure (higher friction material can be used to prevent such a problem); and, if the soil conditions and hydraulic conditions are favorable, the riprap cover might perform well without a filter. However, the consensus was still that riprap performance is better with filters.

7. **Can you share with us some case histories on riprap failure?**

Many respondents stated that they have not experienced any riprap failure. Those respondents who had experienced failures attributed the failures to extreme high-velocity events. Undersizing stone riprap was also mentioned as a common cause for riprap failures. Driftless regions characterized by unglaciated steep terrain with flashy streams and incised channels were also mentioned as causes of riprap failures. Human-induced failures were also mentioned, such as poor installation and incorrect materials (rounded rocks instead of angular) and improper gradation.

Some of the specific shared case histories are listed below:

- A failure in a steep channel liner in Logan County, SH 74 Skeleton Creek, when a major flood occurred during construction.

- 2015, Tex Wash (I-10 in Riverside County); it appears that an impinging flow on the upstream embankment caused the failure. The design was from the late 1960s and included no filter. Abutments were spread footings founded on sand and backfill. The rainfall event was estimated to be in exceedance of a 1000-year flood; the actual discharge is unknown.
- I-55 over Hickahala: Armored (riprap) bank failure after the toe was undermined.
- SR 487 at Tuscolameta: Riprap was installed on a steep (1:1) bank with no filter. The rock held for a while but began to wash downstream.
- Detailed in the old *California Bank & Shore Protection Manual* by Jim Racin from the early 1990s. Toe failure has been the most common (i.e., no toe and the riprap failed).

### 1.4.2. Riprap-Covered-Slope-Related Questions

**1. What standards, guidelines, and/or specifications do you follow for riprap slope stability? Please give us the references.**

FHWA HEC 23 (P.F. Lagasse et al., 2009) is the most common guideline used for the design of riprap-covered slopes, along with the NCHRP 568 guidelines (Lagasse et al., 2006). Other DOTs and committees use locally published guidelines, such as are published by Arizona, Indiana, Louisiana, Iowa, South Carolina, Oregon, Virginia, Wisconsin, Washington, Illinois, New Hampshire, North Carolina, Sweden, and Holland. RocScience Slide software is also used as a tool for geotechnical slope stability analysis as well as numerical simulation.

Other guidelines used for riprap-covered slopes include:

- HEC 11 (Brown & Clyde, 1989).
- HEC 18 (Arneson et al., 2012).
- Tech Brief on Shallow Foundations.
- AASHTO.
- U.S. Corps of Engineers Guidelines.

**2. What are your typical stone riprap toe details?**

This question elicited the most diverse answers by far; the toe detail is mostly based on engineering judgment and is definitely site dependent. FHWA HEC 23 offers guidelines for toe details when installing riprap on slopes. Typically, the toe is trenched to a depth below the expected scour depth and backfilled with non-erodible material. Some entities use a standard predefined trenching depth of 3 or 4 ft below the channel bottom.

4 ft (or 3 ft) x 5 ft buckets are also used for toe details. Some DOTs use a larger thickness for the riprap cover at the toe—sometimes twice the thickness of the riprap blanket or one riprap layer embedded below the streambed. Aprons are sometimes used on abutments. Some

DOTs—Montana, Wisconsin, and Indiana—use their own standard guidelines expressed as drawings and schematics that can be found on their website.

**3. How often do you experience abutment or bank slope failure due to riprap loading (percent of riprap slope cases)?**

This question assesses the frequency of riprap-covered slope failures encountered by different entities. Some mentioned that such data are unknown; others pointed out that failure occurs with old designs, whereas failures have not occurred yet with new designs. The outcome of the responses received is plotted in Figure 16. It shows that the failure of riprap-covered slopes is rare, with 1 to 5 percent of the time being the prevailing percentages.

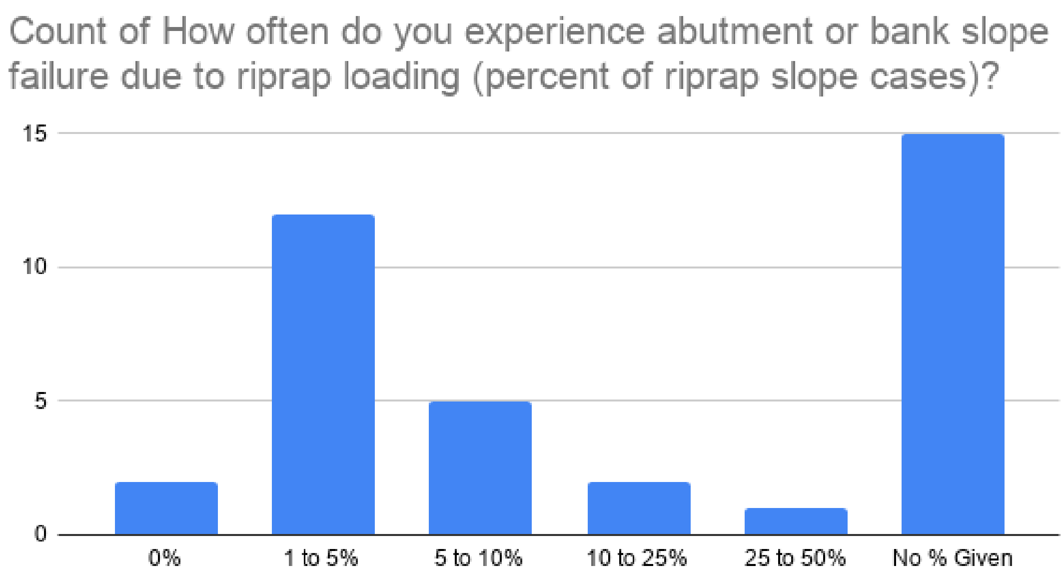


**Figure 16. Count of How Often Riprap-Covered Slope Failures Are Experienced.**

**4. What is the main cause of failure when it occurs?**

From the previous response, it can be seen that if proper design procedures are followed, failure is uncommon. Responses to this question show that failure occurs mostly in extreme events, such as floods and rain generating high-velocity flow. In some cold areas, river ice can contribute to failure. The slope being too steep is also a major issue of concern since it can induce riprap instability. Another common failure mechanism occurs when the toe is undermined due to scour and erosion of the riprap stones or of the soil beyond the riprap blanket; this failure can be remedied by maintenance procedures, and those procedures are crucial for riprap stability. Small and poorly graded riprap stones are also a cause of riprap failure due to erosion; poor placement alone can induce failure. Other failure causes mentioned are a high geometric contraction ratio, rapid drawdown, and flanking caused by quickly shifting meander bends. No respondents mentioned failure in the native soil itself, but failure of the riprap was the most common, which means that, according to the responses received, riprap loading does not typically induce slope failure or instability.

### 5. What measures are taken to avoid riprap slope stability failures?

The most crucial steps to avoid riprap slope failure are taken in the design phase. Slope steepness, geometry, toe detail, riprap stone size selection, filter (drainage), and many other design issues are all good candidates to cause the failure of riprap-covered slopes. Riprap stones need to be large and angular enough for them to not erode easily. Trenching or similar measures at the toe should be selected to increase stability. The slope angle should not be too large because failure is more likely for steep slopes. Some entities do not use slopes steeper than 1.5:1 unless a geotechnical engineer provides efficient measures for a steeper slope, such as stacking rectangular stones. Rock wedges, geotextile fabric, concrete grouting, marine mattresses, articulated concrete blocks and AJacks are used as well. Sometimes riprap is placed all the way on the channel bottom, which reduces top loading. Further, a larger setback from the channel and bridges designed to have lower velocities at the abutment are other possible measures to reduce riprap-covered slope failures.

After the design, on-site execution is also key in order to complete a safe and efficient design. Riprap should be properly placed in the most stable configuration, which is usually not done. Most of the time, riprap stones are simply dumped.

Finally, after the installation takes place, proper inspection is key during the lifespan of the project. In many cases, failure could have been mitigated or even avoided if the early stages of the failure mechanism were detected via inspection. Even if inspection is taking place, sometimes inspectors are not properly trained to detect anomalies at early stages of a potential failure.

Design measures should be taken to avoid riprap slope stability failure, but they should be complemented by proper installation and efficient routine inspection.

### 6. What is the maximum slope angle you would allow for stone riprap placement?

The majority of respondents allowed a 2:1 slope (26.6 degrees), while the next-largest group adhered to a higher safety precaution with a 3:1 slope (18.4 degrees). Some allowed a 1:1 slope (45 degrees), and 1.5:1 slopes (33.7 degrees). The responses are shown in the bar chart below (Figure 17).

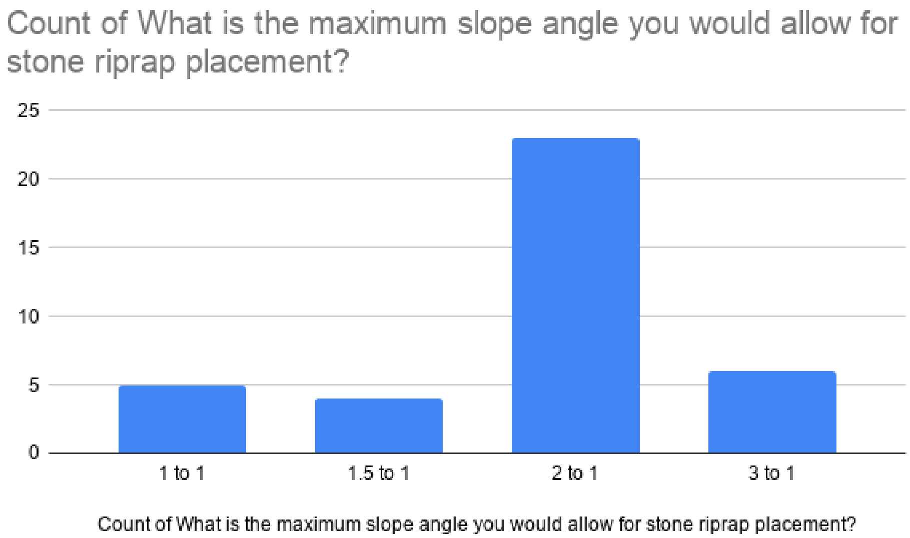


**Figure 17. Count of the Maximum Slope Angle Adopted to Install Riprap.**

By combining the answers to this question and Question 3 about the frequency of riprap-covered slope failures, Figure 18 was generated. This figure shows the failure frequency on the vertical axis and the slope angle on the horizontal axis. The trend is clear: the higher the maximum slope angle is, the higher the frequency of failures.

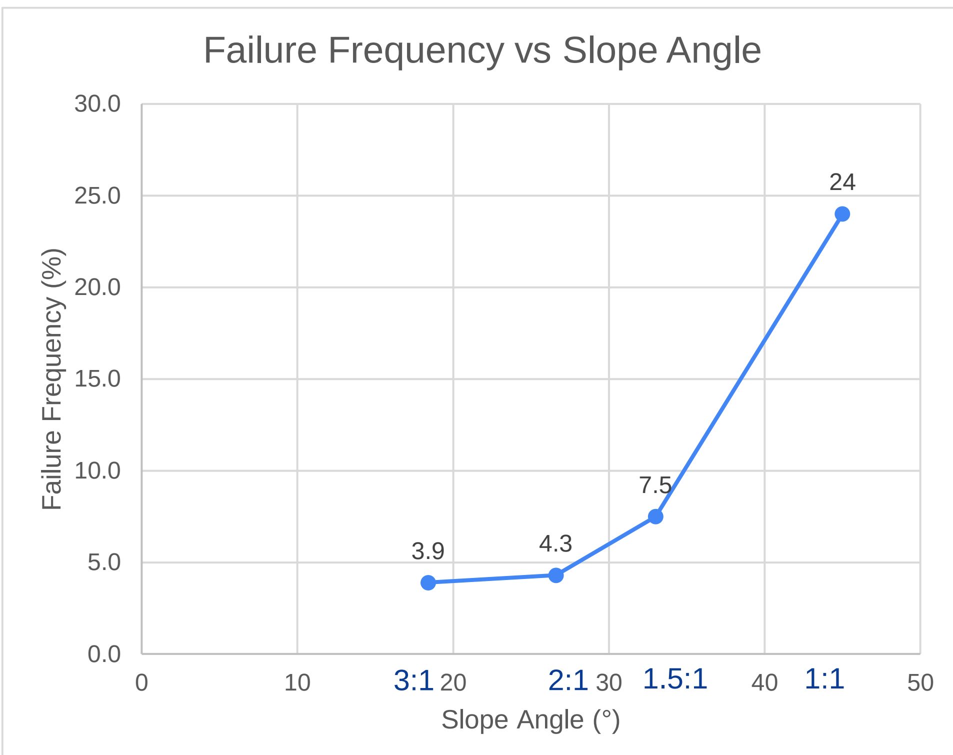


**Figure 18. Comparison between Steepest Slope Angle Adopted and Failure Frequency.**

**7. Can you share with us some case histories on slope failure due to riprap loading?**

Many of the respondents had not experienced any riprap failure; most of the issues that were reported were due to toe undermining, riprap stone size too small, steep slopes, improper placement, and inaccurate inspection. One case history was shared:

- I-55 at Hickahala: Undermining of the bank slope toe led to failure of the armored (riprap with geosynthetic filter fabric) bank.

## 1.5. CONCLUSIONS

Following is an overview of the existing knowledge on filters under riprap layers and on the stability of riprap-covered slopes.

- Riprap Filters:
    - Two types of riprap filters exist: granular and geosynthetic.
    - A filter enhances the efficiency of the riprap by not allowing erosion of the underlying soil through the riprap.
    - Many techniques for filter installation exist; they depend mainly on the type of filter, contractor, and environment. Placing riprap underwater is the biggest challenge.
    - In most cases, a geosynthetic filter is preferred over a granular filter.
- Riprap-Covered Slopes:
    - The flow of water in riprap must be carefully accounted for in order to properly select riprap size based on correlations and previous studies.
    - Slope angle where the riprap is being placed is the major parameter because it is the major cause of failure for riprap-covered slopes. A threshold value for the steepest allowable slope must be determined.
    - The four main types of riprap failure mechanisms are particle erosion, translational slide, slump, and modified slump.
    - Stabilization methods for riprap-covered slopes include (a) toe trenching, or excavation at the toe with backfilling; (b) a flexible launching apron; and (c) a toe wall.

Generally, in most common cases, a filter is needed to avoid erosion of the native soil, which will lead to the failure of the whole riprap layer. If it is shown that the native soil grain size distribution blends according to requirements into the grain size distribution of the riprap material, and if there are no gaps between particle sizes, the filter might not be necessary. If a filter is necessary, a soil investigation must be made in order to assess and properly design the filter’s mechanical and physical properties and also to decide on which type of filter to use.

The existing literature regarding riprap-covered slopes mainly deals with the stability of the riprap itself and its different failure modes. These failure modes are described in this report, but these modes do not involve the stability of the slope itself using typical drained and undrained analysis of the native soil. Some correlations have been developed in order to properly design and select the riprap stone sizes and weight, taking into consideration the riprap friction repose, the critical velocity, and the bank slope angle. These correlations appear to be mostly theoretical.

Following is a summary of the survey results:

- Filters:
  1. What is the most commonly used guideline for riprap filter design? HEC 23.
  2. What percent of the time do you use filters below riprap? Most of the time (>80 percent of the time).
  3. Do you prefer a granular filter or a geosynthetic filter? Geosynthetic.
  4. How is the filter installed underwater? This process is very difficult and often left to the contractor.
  5. What is your experience in the comparison of granular and geosynthetic filter? Practically no experience was shared.
  6. What is your experience in the comparison of riprap with and without filter? Riprap performance is best with a filter.
  7. Can you share case histories of riprap failure? Five case histories were mentioned.

- Riprap-covered slopes
  1. What is the most commonly used guideline for riprap-covered slope design? HEC 23.
  2. What are your typical stone riprap toe details? Trenching to anticipated scour depth and back filling with erosion-resistant material.
  3. How often do you experience riprap-covered slope failure? Rarely (1 to 5 percent of the time).
  4. What is the main cause of failure? Extreme event leading to undermining the toe of the slope and associated riprap stone movement.
  5. What measures are taken to avoid riprap-covered slope failures? Three components: proper design (riprap size and stone quality), proper construction/placement, regular monitoring.
  6. What is the maximum slope angle you would use for riprap? Most respondents would use 2:1 slopes.
  7. Can you share case histories of riprap failure? Only one case history was mentioned.

# CHAPTER 2: GRANULAR AND GEOSYNTHETIC FILTERS

## 2.1. FILTER IMPORTANCE

Determining the importance of a filter and whether it is necessary or not in riprap applications was one of the key objectives of this study. In the first chapter, the existing literature and the survey outcome were summarized. In this section, this subject was approached via laboratory testing using the Erosion Function Apparatus (EFA).

EFA testing was developed in the early 1990s to measure the erodibility of soils and soft rocks (Briaud, 2008). The principle is to go to the site where erosion is being investigated, collect samples within the depth of concern, bring them back to the laboratory, and test them in the EFA. The 75 mm outside diameter sampling tube is placed through the bottom of the conduit where water flows at a constant velocity (Figure 19).

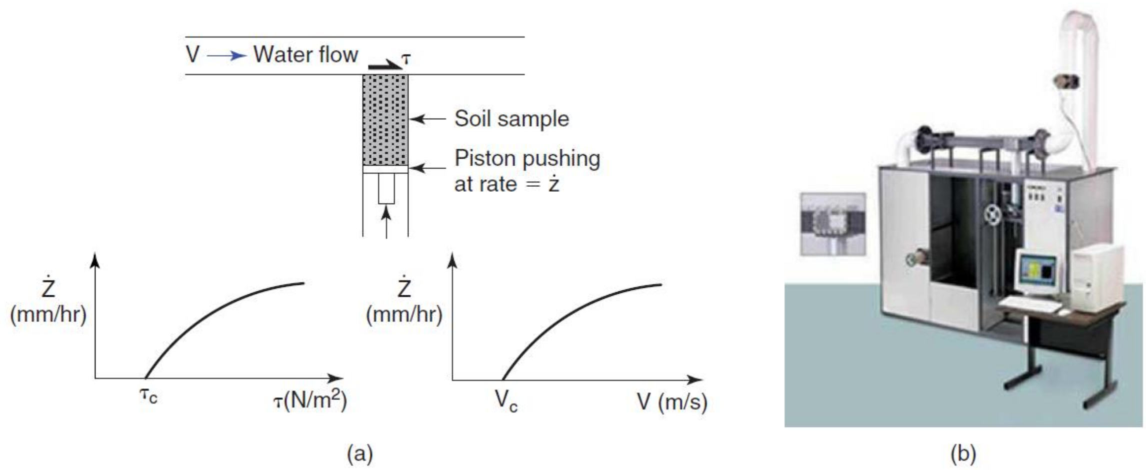


**Figure 19. Erosion Function Apparatus Test: (a) Principle and (b) Equipment (Briaud, 2013).**

The soil or rock is pushed by a piston out of the sampling tube only as fast as it is eroded by the water flowing over it. The test result consists of the erosion rate $\dot{z}$ versus the shear stress $\tau$ curve and the erosion rate $\dot{z}$ versus the mean flow velocity $V$ curve (Figure 19). For each flow velocity $V$, the erosion rate $\dot{z}$ (mm/hr) is simply obtained by dividing the length ($h$) of sample eroded by the time ($t$) required to do so: $\dot{z} = \frac{h}{t}$. The velocity $V$ is obtained by measuring the flow $Q$ and dividing by the flow area $A$. The shear stress $\tau$ is obtained by using the Moody Chart (Figure 20) for pipe flows: $\tau = \frac{1}{8} f \rho V^2$, where $\tau$ is the shear stress on the wall of the pipe, $f$ is the friction factor obtained from the Moody Chart, $\rho$ is the mass density of water (1000 kg/m$^3$), and $V$ is the mean flow velocity in the pipe. The friction factor $f$ is a function of the pipe's Reynolds Number $Re$ and the pipe roughness $\varepsilon/D$, where $\varepsilon$ is the depth of the asperities on the soil surface, and $D$ is the hydraulic diameter of the pipe. The

Reynolds Number is $VD/v$, where $v$ is the kinematic viscosity of water ($10^{-6}$ $m^2/s$ at 20°C) (Briaud, 2013).

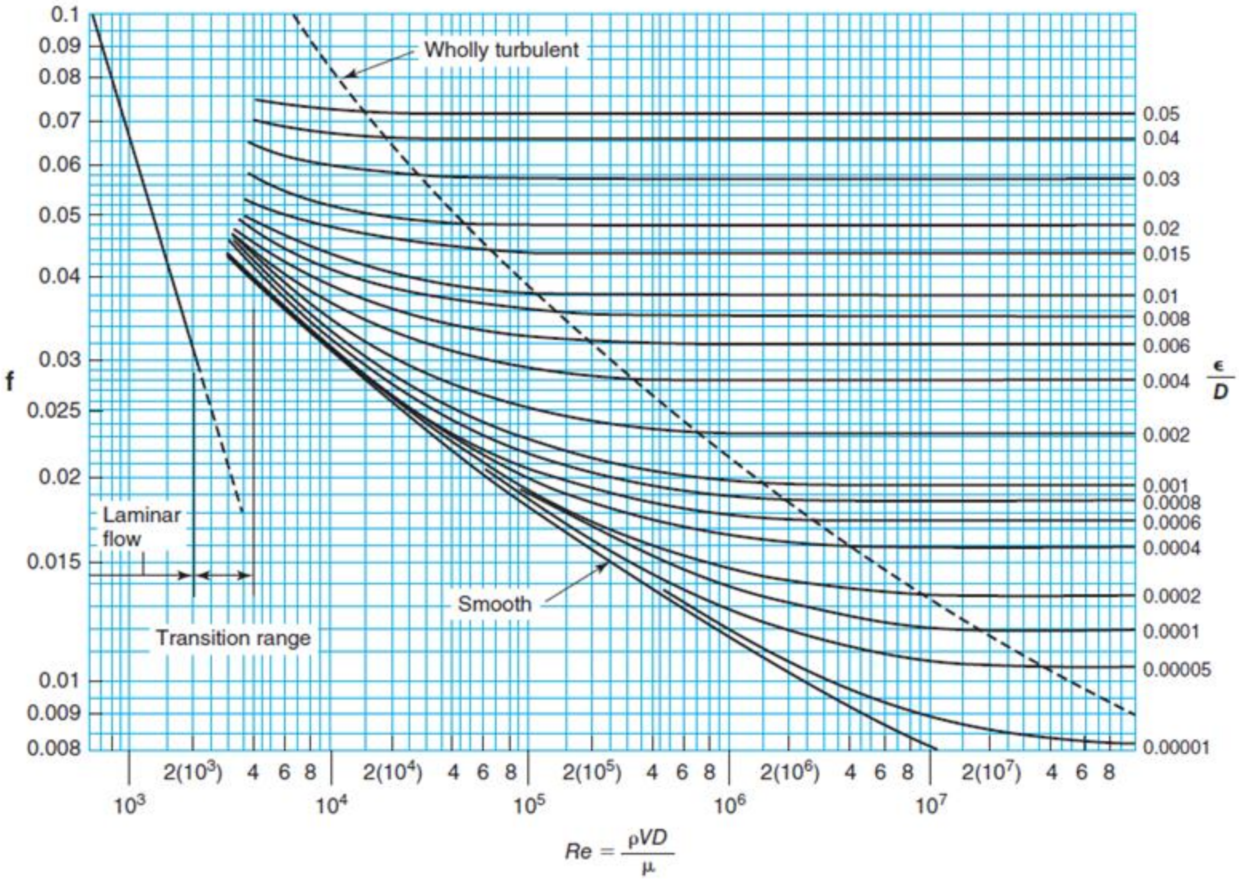


**Figure 20. Moody Chart (after Munson et al., 2012).**

In this project, the goal of the erosion testing in the EFA, although qualitative, was to physically model the behavior of the riprap and compare the underlying soil erodibility with and without a filter as well as with no riprap at all. A total of 12 tests (Table 3) were carried out, wherein two tests were used as reference tests for the soils (sand and clay) to only determine their erosion parameters for future comparison. Two reference tests were run on gravel (riprap), which were used to represent riprap stones in the EFA experiments.

**Table 3. Riprap EFA Testing Matrix.**

| No | Profile | Soil Type | | Geotextile Type | Gravel |
|---|---|---|---|---|---|
| | | Sand | Clay | | |
| 1 | Profile 1-S (sand) | + | | | |
| 2 | Profile 1-S-1R Test 1 | + | | | 1 layer + |
| 3 | Profile 1-S-1R Test 2 | + | | | 1 layer + |
| 4 | Profile 1-S-2R | + | | | 2 layers + |
| 5 | Profile 1-S-3R | + | | | 3 layers + |
| 6 | Profile 1-S-GTF1-1R Test 1 | + | | SoilTain PP 105/105DW | + |
| 7 | Profile 1-S-GTF1-1R Test 2 | + | | SoilTain PP 105/105DW | + |
| 8 | Profile 2-C (clay) | | + | | |
| 9 | Profile 2-C-1R | | + | | + |
| 10 | Profile 2-C-GTF1-1R | | + | SoilTain PP 105/105DW | + |
| 11 | Profile 1-R (riprap) Test 1 | | | | + |
| 12 | Profile 1-R (riprap) Test 2 | | | | + |

In order to investigate the effect of riprap, gravel was placed on sand and clay (Figure 21 and Figure 22). The test showed that without a filter the underlying soil erodes through the riprap. Then, a geosynthetic filter layer with known properties (Table 4) was introduced between the gravel and the soil in the sampling tube. Two tests were performed on one layer of gravel (riprap) placed on a geotextile underlined by sand, and one test was carried out on one layer of riprap placed on a geosynthetic filter underlined by clay. In both cases, the filter stopped the erosion of the underlying soil.

**Figure 21. Riprap with Geotextile in EFA.**

**Figure 22. Gravel (Riprap) in EFA.**

**Table 4. Geosynthetic Filter Properties.**

| Type of Geotextile | SoilTain PP 105/105DW |
|---|---|
| Material | Woven PP for dewatering applications, Polypropylene |
| Opening Size | 240 µm |
| Water Permeability Index Normal to the Plane | 20 x $10^{-3}$ m/s |
| Standard Dimensions: Width x Length | 5.20 x 200.0 m |
| Weight | ≈ 440 g/$m^2$ |
| Ultimate Tensile Strength: Longitudinal x Transversal | ≥ 105 kN/m |
| Application | Dewatering, Tailings Dam Embankment Construction. |

Table 5 shows a summary of the results obtained from all the EFA tests performed. These results generally follow the same trend: negligible to slight erosion in the beginning until the erosion rate increases suddenly when the shear stress reaches its critical value, at which point erosion progresses rapidly, and the sample finally fails. Only the sand sample being tested without riprap showed a different behavior wherein erosion started at the very first stage of the test (first velocity step).

**Table 5. Summary of EFA Testing Results.**

| Profile | Duration (h) | Critical Velocity $v_c$ (m/s) | | Critical Shear Stress $\tau_c$ (Pa) | |
|---|---|---|---|---|---|
| | | Initiation of Erosion | Riprap | Initiation of Erosion | Riprap or Geotextile Failure |
| Profile 1-S (sand) | 0.47 | 0.29 | - | 0.347 | - |
| Profile 1-S-1R Test 1 | 0.53 | 0.57 | 1.09 | 5.970 | 21.831 |
| Profile 1-S-1R Test 2 | 0.38 | 0.59 | 0.9 | 6.266 | 14.479 |
| Profile 1-S-2R | 0.88 | 0.57 | 0.9 | 5.848 | 14.479 |
| Profile 1-S-3R | 0.4 | 0.9 | 0.9 - 1.23 | 14.479 | 14.479 - 27.043 |
| Profile 1-S-GTF1-1R Test 1 | 1.47 | 1.19* | 4.88 | 25.313 | 425.682 |
| Profile 1-S-GTF1-1R Test 2 | 0.62 | 0.9* | 1.15 | 16.909 | 27.607 |
| Profile 2-C (clay) | 1.00 | 0.70 | - | 1.96 | - |
| Profile 2-C-1R | 0.44 | 0.9** | 1.78 | 16.909 | 66.140 |
| Profile 2-C-GTF1-1R | 1.43 | 0.9** | 1.19 | 16.909 | 29.561 |
| Profile 1-R (riprap) Test 1 | 0.44 | - | 0.94 | - | 15.794 |
| Profile 1-R (riprap) Test 2 | 0.41 | - | 0.74 | - | 9.788 |

Erosion improvement is observed when riprap is added on top of the sample, especially for sand. However, a significantly higher improvement in erosion resistance is observed when the geosynthetic filter layer is introduced underneath one layer of riprap. Figure 23 and

Figure 24 show the erosion functions generated in logarithmic scale fitted in a category chart. This chart qualitatively classifies the erodibility of a soil, such as high erodibility, low erodibility, and so on. It can be seen that when a geosynthetic filter is used, the soil being tested becomes classified as a lower erodibility material.

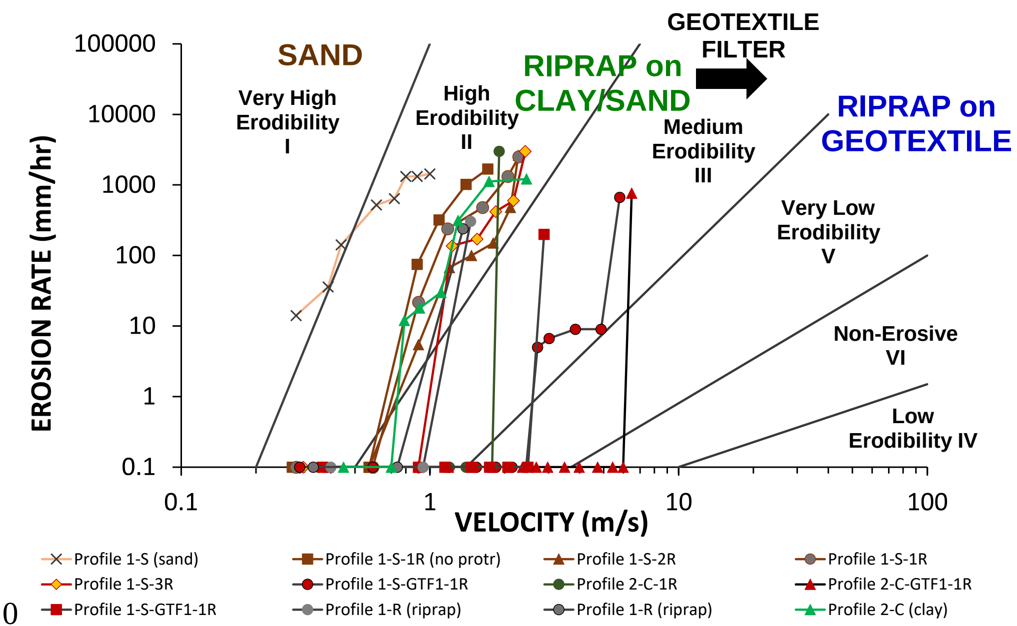


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Figure 23. Erosion Functions In Function of Velocity for All Tests.

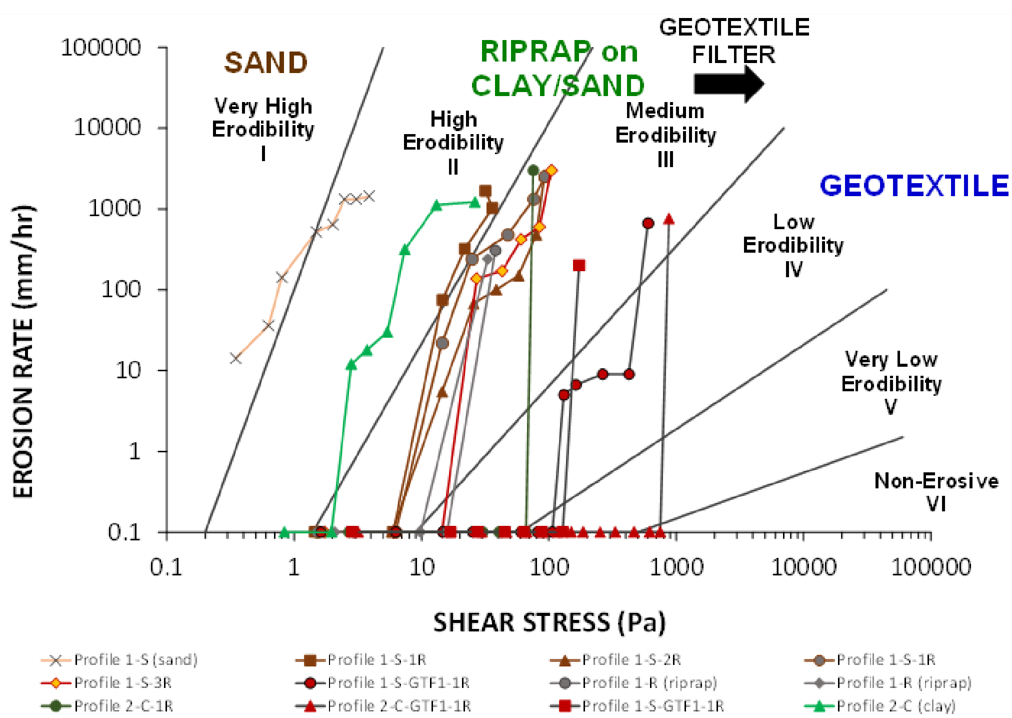



Figure 24. Erosion Functions In Function of Shear Stress for All Tests .

The effect of the geosynthetic filter placed on the top of the soil sample (sand or clay) and covered with one layer of the riprap is investigated in the tests Profile 1-S-GTF1-1R and Profile 1-C-GTF1-1R. The plots indeed prove that using the geotextile reduces soil erodibility and can successfully minimize soil erosion. Use of the geotextile allows the erodibility category to move from high erodibility (II) to medium erodibility (III) and less.

However, to check whether the results of the EFA testing are applicable to the field scale, full scale tests or case histories are needed; the section on case histories addresses this issue. The riprap without the filter does not work even with several layers of riprap. In addition, increasing the number of riprap layers cannot replace the role of the filter because when the geotextile was installed under one layer of riprap, the results obtained were the best. However, the EFA testing observations and results suggest that the risk of uplift failure of the filter becomes significant when the gravel/riprap is detached, and the average velocity of flow exceeds about 5–6 m/s. In one case, the geosynthetic filter failed due to uplift at 2.88 m/s after the gravel/riprap got entrained by the flow.

It can be concluded from this analysis that a filter mitigates erosion on soils significantly when introduced as a transition layer between the riprap and the underlying soil.

The outcomes from the three sections discussed above—literature review, survey, and EFA testing—all bolster the conclusion that riprap performs better when a filter is used (i.e., the scour countermeasure is more efficient when a filter layer is used between the underlying soil and the riprap).

## 2.2. FILTER DESIGN

### 2.2.1. Granular Filter Design

First, the different parameters involved in the design will be defined, then a step-by-step procedure for granular filter design will be described.

The main properties of a granular filter are particle size distribution permeability, porosity, thickness, and durability. Particle size distribution is the variation of the size of the particles within a sample. Permeability of a granular filter is usually obtained by laboratory experiments (most likely the Falling Head Test or Constant Head Test) or by using existing correlations. Porosity is the parameter that quantifies the volume of voids in the filter compared to the bulk volume. Thickness of the granular filter depends on the number of layers and the grain size. Durability and quality of the filter is related to the aggregates used in the filters; these aggregates should be dense, hard, and durable.

For granular filter design, the steps include:

- Step 1: Site investigation.
- Step 2: Underlying soil index parameters.
- Step 3: Granular filter index parameters.
- Step 4: Underlying soil-filter grain size matching.
- Step 5: Filter-riprap grain size matching.
- Step 6: Thickness and permeability.

These steps are described below in detail. Note that the design proceeds by trial and error whereby a granular filter is selected a priori, and the steps below are used to see if that pre-selected geotextile will satisfy the design or not.

#### *2.2.1.1. Design Procedure*

**Step 1: Site investigation**—Underlying soil information must be gathered, consisting mainly of grain size distribution, permeability, and plasticity parameters, if more than 20 percent of the base soil is clay.

**Step 2: Underlying soil index properties**—The underlying soil index parameters, such as $d_{50}$ and $C_u$, which is the ratio of $d_{60}$ over $d_{10}$, called the coefficient of uniformity, must be determined. $d_{50}$ is the grain size corresponding to 50 percent passing by weight. The same type of definition applies for $d_{60}$ and $d_{10}$.

**Step 3: Granular filter index properties**—A candidate granular filter must be selected, and the index parameters for the granular filter, such as $d_{50}$ and $C_u$, must be determined.

**Step 4: Underlying Soil-Filter Grain Size Matching**—This step consists of determining the grain size requirements for the filter to perform well on top of the underlying soil. This step can be achieved by using the Terzaghi criterion for granular filters or the Cisten-Ziems graphical method. According to NCHRP 568 (Lagasse et al., 2006), using the Cisten-Ziems method is more robust for riprap application. Cisten-Ziems developed a graph, shown in Figure 25, that can be used to obtain the proper grain size distribution.

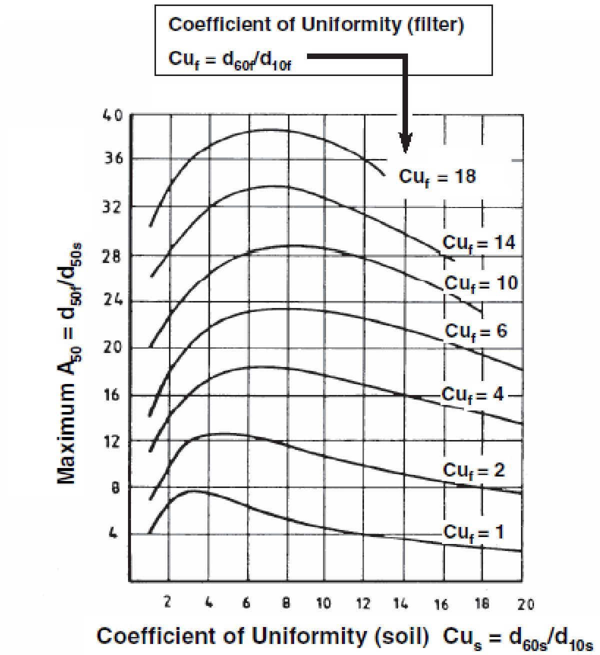



**Figure 25. Cisten-Ziems Graph (Lagasse et al., 2006).**

*Cisten-Ziems method*: To use the chart (Figure 25), first select the base soil $C_u$ value, then select the corresponding curve based on the candidate filter $C_u$. $A_{50}$ can then be obtained on the *y*-axis, which is the ratio of $d_{50f}$ to $d_{50s}$. Then, the maximum possible value for the granular filter $d_{50f(max)}$ can be obtained by:

$$d_{50f(max)} = A_{50\ max} \times d_{50s}$$

where $d_{50s}$ is the base soil $d_{50}$ parameter. After following this procedure, check to see if the $d_{50}$ of the selected granular filter material meets the required $d_{50}$ obtained from the chart to meet retention criterion.

*Terzaghi Criterion method*: This criterion is expressed by the following inequality:

$$d_{15c} / d_{85f} < 5 < d_{15c} / d_{15f} < 40$$

where $d_{15}$ and $d_{85}$ are the grain sizes corresponding to 15 percent and 85 percent passing sizes, and the subscripts "*c*" and "*f*" refer to the coarse and finer layers, respectively.

For example, if the compatibility of the grain size of the filter and of the riprap are to be checked, the filter is the finer layer, and the riprap is the coarser layer. If, on the other hand, the compatibility of the grain size of the underlying soil and of the filter is to be checked, then the underlying soil is the finer layer, and the filter is the coarser layer. The inequality above must be satisfied for both boundaries—for the lower boundary in order to prevent piping and for the upper boundary to provide a uniformity criterion; if that is not possible, more than one granular filter or a combination of a granular filter and a geosynthetic filter will be needed.

**Step 5: Filter-Riprap Grain Size Matching**—The analysis performed in Step 4 should be repeated, but for this step it is to consider the riprap as the "filter" and the filter as the "base soil" in order to check if the grain size selected for the filter is compatible with the riprap gradation. If this condition fails, an additional filter layer with a different gradation is needed.

**Step 6: Thickness and Permeability**—The thickness of the filter layer must be selected. For ease of placement, a minimum thickness of 15 cm (6 inches) is usually adopted, but it may go up to 38 cm (15 inches) if large riprap stones are used. The minimum thickness of the riprap layer shall be 1.5 times the maximum aggregate diameter if $d_{50}$ < 15 inches and 1.2 times the maximum aggregate size if $d_{50}$ > 15 inches (Riprap Stabilized Outlet, 2012).

The permeability of the granular filter must be checked; it is obtained using correlations or laboratory testing. The permeability of the filter should be 10 times greater than that of the base soil: $k_f = 10k_s$.

#### *2.2.1.2. Design Example*

This design example was taken from NCHRP 568 (Lagasse et al., 2006), with some modifications made by the authors of this report. In the example, revetment riprap using gradation Class II is to be placed on a channel bank. The native soil on the channel banks is a silty sand. A locally produced sand is proposed as a granular filter material for the riprap. The grain size distribution of the native soil and candidate filter material are shown in Figure 26. Other characteristics of the design are listed in Table 6.

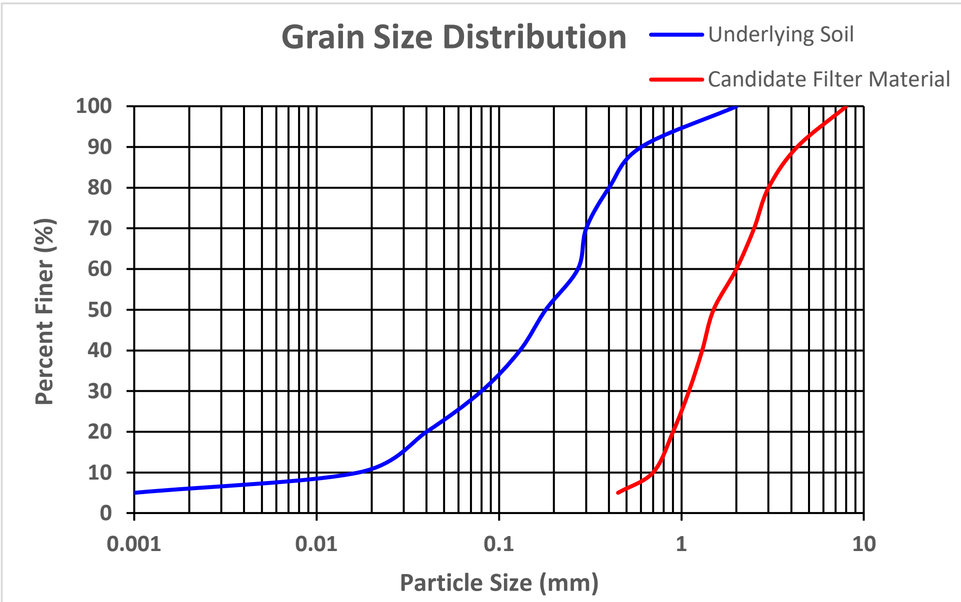



**Figure 26. Filter and Underlying Soil Grain Size Distribution.**

**Table 6. Design Example Soil, Filter, and Riprap Parameters (Lagasse et al., 2006).**

| Soil Property | Native Soil | Filter | Riprap Class II |
|---|---|---|---|
| Hydraulic conductivity K, cm/s | $4.2 \times 10^{-4}$ | $2.3 \times 10^{-2}$ | n/a |
| Coefficient of uniformity $C_u = d_{60}/d_{10}$ | .25/.015 = 16.6 | 1.9/.66 = 2.9 | 2.1 |
| Median diameter $d_{50}$, mm | 0.17 | 1.5 | 230 (9 in) |
| Plasticity index | 3.3 | (np) | (np) |

**Step 1**, **Step 2**, and **Step 3** are given in Figure 26 and Table 6. Those three steps, as previously discussed, consist of gathering required native soil information and filter parameters, such as grain size distribution; computation of $d_{50}$, $d_{60}$, and $C_u$; and permeability.

**Step 4: Cisten-Ziems**—To assess the suitability of the candidate filter material for compatibility with the native soil, enter the Cisten-Ziems chart (Figure 25) with $C_u$ = 16.6 of the native soil on the *x*-axis. Go vertically up to a location corresponding to $C_u$ = 2.9 for the candidate filter material. Read the maximum allowable value $A_{50}$ of approximately 12 on the *y*-axis. Compute the maximum allowable $d_{50}$ of the filter material:

Max. allowable $d_{50f} = A_{50}(d_{50s}) = 12(0.17) = 2.0$ mm

Because the actual $d_{50f}$ of the candidate material is 1.5 mm, this material is suitable as a filter for the native soil based on its particle retention function.

The Terzaghi criterion is expressed by the following inequality:

$d_{15c} / d_{85f} < 5 < d_{15c} / d_{15f} < 40$

For the candidate filter, $d_{15c}$ = 0.8 mm, whereas for the native soil $d_{15f}$ = 0.03 mm, $d_{85f}$ = 0.45 mm: $\frac{0.8}{0.45} = 1.78 < 5 < \frac{0.8}{0.03} = 26.6 < 40$, which means the candidate filter index properties satisfy the required equation and this candidate filter is compatible with the soil based on the Terzaghi filter criterion as well.

**Step 5**: **Assess the suitability of the riprap for compatibility with the candidate filter material**—Enter the Cisten-Ziems chart (Figure 25) with $C_u$ = 2.9 for the filter material on the *x*-axis. Go vertically up to the location corresponding to $C_u$ = 2.1 for the riprap. Read a maximum allowable value $A_{50}$ of approximately 13 on the *y*-axis. Compute the maximum allowable $d_{50}$ of the riprap:

Max. allowable $d_{50r} = A_{50}(d_{50f}) = 13(1.5) = 19.5$ mm

Because the actual $d_{50r}$ of the riprap is 230 mm, the filter particles will leach through the voids of the Class II riprap. Therefore, a second (coarser) filter layer will need to be designed to retain the first filter layer while simultaneously being retained by the Class II riprap. A coarse, gravelly material must be found and analyzed as a candidate material for the second filter layer using the same procedure. Moreover, because the above example resulted in a two-layer granular filter system, a geotextile option can be explored.

This can also be checked using Terzaghi's criterion given that the required index properties ($d_{15}$ and $D_{85}$) of the riprap material are available.

**Step 6: Check the permeability ratio**—$K_f/K_s$ = (2.3 x $10^{-2}$) / (4.2 x $10^{-4}$) = 55. Because this ratio is greater than 10, the filter is convenient from a permeability standpoint. Thickness will be determined after obtaining the second layer characteristics.

### 2.2.2. Geosynthetic Filter Design

Briaud (2013) gave a good background on geosynthetic fabrics, including parameter definitions and typical ranges of values. Based on that material, the different parameters involved in the design will next be defined, then a step-by-step procedure for geosynthetic filter design will be described.

The main parameters for geosynthetic filters are permeability, transmissivity, porosity, apparent opening size, percent open area, thickness, grab strength, tear strength, and puncture strength. *Permeability* of the geotextile quantifies the ease of flow of fluids through the porous medium and is calculated as the flow through the geotextile divided by the geotextile area through which it flows and by the hydraulic gradient across the geotextile. It is expressed in units of distance per unit of time ($LT^{-1}$). It is common to use the term *permittivity*, denoted by $\psi$, defined as the permeability divided by the geotextile thickness; therefore, permittivity has units of ($T^{-1}$). *Transmissivity* of a geotextile is also commonly used; it is defined as the permeability multiplied by the geotextile thickness. It is expressed as $L^2/T^{-1}$. *Porosity* is the ratio of voids to the total bulk volume; it must be chosen to avoid long-term clogging. *Apparent opening size* (AOS), denoted by $O^{95}$, is the value (in mm) of the opening corresponding to 95 percent of all the openings in the geotextile being smaller than $O^{95}$. *Percent open area* (POA) is the total open area divided by the total area of the geotextile expressed in percent and is typically measured by shining a light through the geotextile. It is chosen to minimize long-term clogging potential. The thickness of the geotextile is obtained from manufacturer specifications.

A geotextile is subject to high tensile loads, especially during the installation phase of the project. Therefore, various strengths must be checked. The *grab strength* is the force per unit length of geotextile required to tear the fabric in tension. The grab strength and the associated elongation are important design parameters. The *tear strength* is the force per unit length of geotextile required to propagate a tear in the fabric, and the *puncture strength* is the force required to puncture the fabric material using a standard penetration apparatus.

The following design steps are taken for geotextile filter design:

- Step 1: Site investigation.
- Step 2: Particle retention criterion.
- Step 3: Permeability of the geotextile.
- Step 4: Geotextile mechanical properties.
- Step 5: Long-term clogging.

These steps are described below in detail. Note that the design proceeds by trial and error whereby a geotextile fabric is selected a priori, and the steps below are used to see if that pre-selected geotextile will satisfy the design or not.

#### *2.2.2.1. Design Procedure*

**Step 1: Site investigation.** A site investigation must be done in order to obtain the base soil information, which includes grain size distribution, permeability, and plasticity (if more than 20 percent of the base soil is clay). Important parameters are $d_{50}$, which is the particle size where more than 50 percent of the particles are finer, and the coefficient of uniformity $C_u$, which is the ratio of $d_{60}$ to $d_{10}$.

**Step 2: Particle retention criterion.** As a first approach, the engineer can consider the Terzaghi particle retention criterion used for granular filters. However, according to Giroud (2010), the Terzaghi particle retention criterion for granular filters is incomplete when applied to geosynthetic filters.

Figure 27 shows a decision tree used to determine the AOS and whether a transitional granular filter layer is needed along with the geosynthetic filter. This decision tree is based on the underlying soil gradation and the classification of the soil (fine grained, coarse grained, plasticity, etc.).

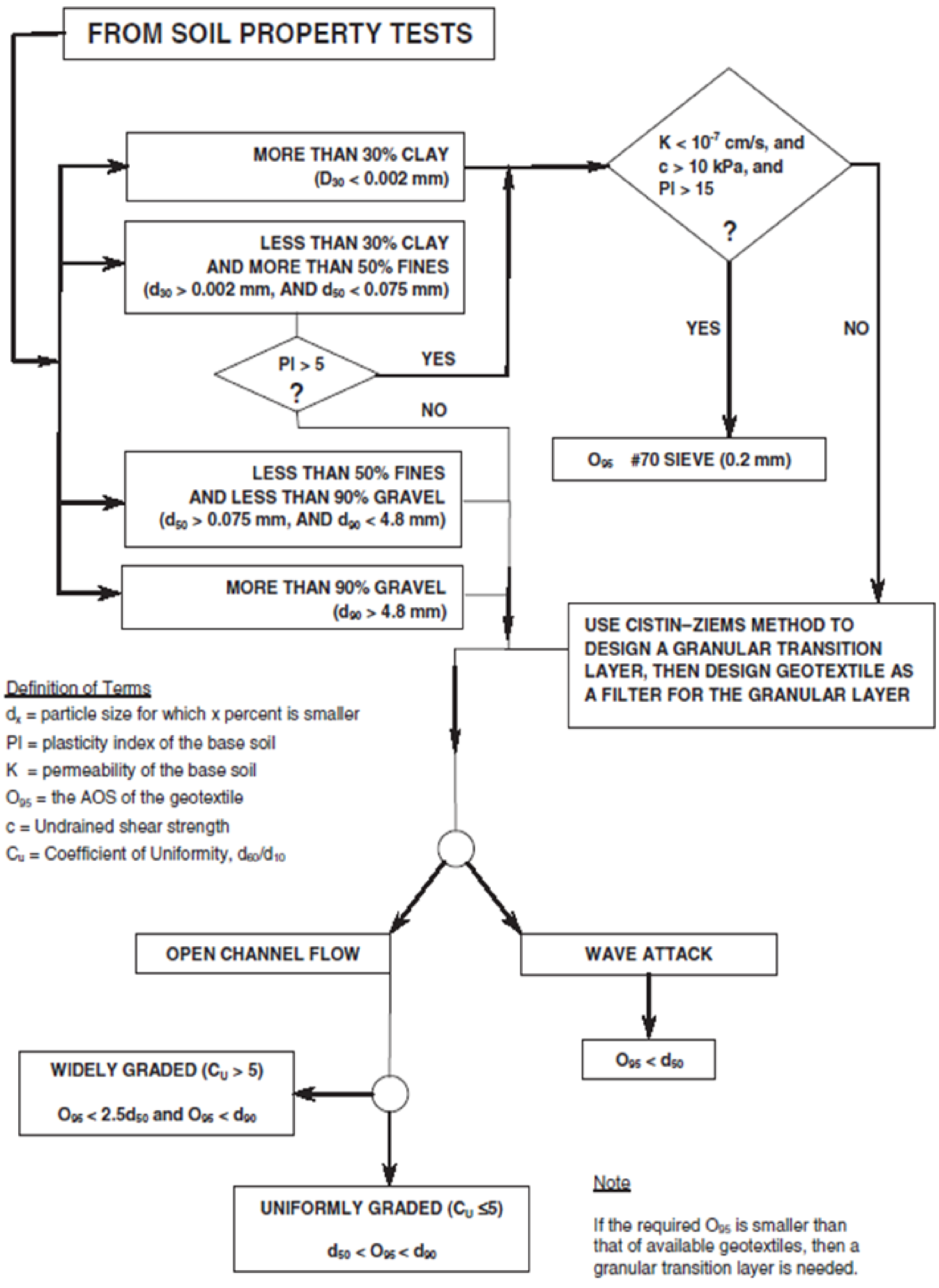



**Figure 27. Decision Tree for Geotextile Design (Lagasse et al., 2006).**

The decision tree is entered based on the grain size distribution of the underlying soil (e.g., percentage of fines, gravel). Then, given each type of soil, a decision is made on whether a transition layer is needed or not.

Next, the opening size of the geotextile is chosen depending on the soil grain size distribution and the type of flow—open channel flow or wave attack flow. Open channel flow is the type of flow where a free surface exists, such as in rivers. Wave attack flow is a more severe environment in terms of flow velocities and turbulence and is usually found in coastal areas.

For example, a composite filter is needed when more than 30 percent of the natural soil is clay with low cohesion. In such a case, a granular filter layer should be designed based on the procedure in the previous section; then, the opening size of the geotextile will be based on the grain size of the granular filter adopted.

In addition, in the case of open channel flow, inequalities are set between $O_{95}$ and $d_{50}$ as well as $d_{90}$ (Figure 27); the designer must check both and adopt the one that yields a lower $O_{95}$.

**Step 3: Permeability of the Geotextile**. The permeability of the geotextile must be at least four times greater than that of the native soil (Koerner, 1998) and up to 10 times greater for severe cases (for example, areas subject to high-velocity flow and overtopping). Permeability parameters of the soil can be obtained by laboratory testing (Constant Head Test or Falling Head Test). If those tests results are not available, the permeability of the soil can be obtained using correlations based on grain size distribution data, which are already generated from the first step. The permeability of the geotextile is obtained by multiplying the thickness (cm) of the geotextile by its permittivity ($s^{-1}$); these parameters are obtained from manufacturer specifications. These requirements for the geotextile permeability are aimed to allow for drainage of water and avoid hydrostatic pressure buildup that could lead to failure. Table 7 illustrates typical values for porosity and permeability for different kinds of soils.

**Step 4: Geotextile Mechanical Properties.** The geotextile must be selected to meet the strength criteria, primarily to avoid mechanical failure during placement and handling. Strength parameters such as grab, tear, and puncture strength are obtained from manufacturer specifications. The environment of the project and the contractor installation technique for the filter determine the severity of the forces applied to the geotextile during installation.

According to the Harris County Flood Control District (2001), recommended values for geotextiles mechanical properties are 90 lb for grab strength (ASTM D 4632), 15 percent for elongation (ASTM D 4632), 40 lb for puncture strength (ASTM D 4833), and 30 lb for trapezoidal tear (ASTM D 4533).

**Table 7. Permeability and Porosity Values for Different Type of Soils (Lagasse et al., 2006).**

| Type of Material | Porosity (vol/vol) | Permeability (cm/s) |
|---|---|---|
| Gravel, coarse | 0.28 | $4 \times 10^{-1}$ |
| Gravel, fine | 0.34 | |
| Sand, coarse | 0.39 | $5 \times 10^{-2}$ |
| Sand, fine | 0.43 | $3 \times 10^{-3}$ |
| Silt | 0.46 | $3 \times 10^{-5}$ |
| Clay | 0.42 | $9 \times 10^{-8}$ |

**Step 5**: **Long-Term Clogging**. Long-term clogging must also be considered since it is a common issue with geotextiles. The *POA*, *AOS*, and porosity are the main parameters involved in designing to prevent clogging. If a woven geotextile is used, the *POA* should be greater than 4 percent. If a nonwoven geotextile is used, porosity should be greater than 30 percent by volume. The *POA* has to meet two requirements, one for the retention criterion and one for the clogging criterion; the geotextile having the largest of the two required *POA* values is selected.

### *2.2.2.2. Design Example*

This design example was taken from NCHRP 568 (Lagasse et al., 2006), with some modifications made by the authors of this report. In the example, revetment riprap using gradation Class II is to be placed on a channel bank. The native soil on the channel bank is a silty sand. A locally produced sand is proposed as a granular filter material for the riprap. The grain size distribution of the native soil and candidate filter material are shown in Figure 28. Other characteristics of the design are listed in Table 8.

The granular filter design was covered in the previous section. Based on the outcome, the design guideline requires a second layer of granular filter; therefore, a single geosynthetic filter layer will be explored in this example using the same riprap and underlying soil properties. The design of the geosynthetic filter is covered below.

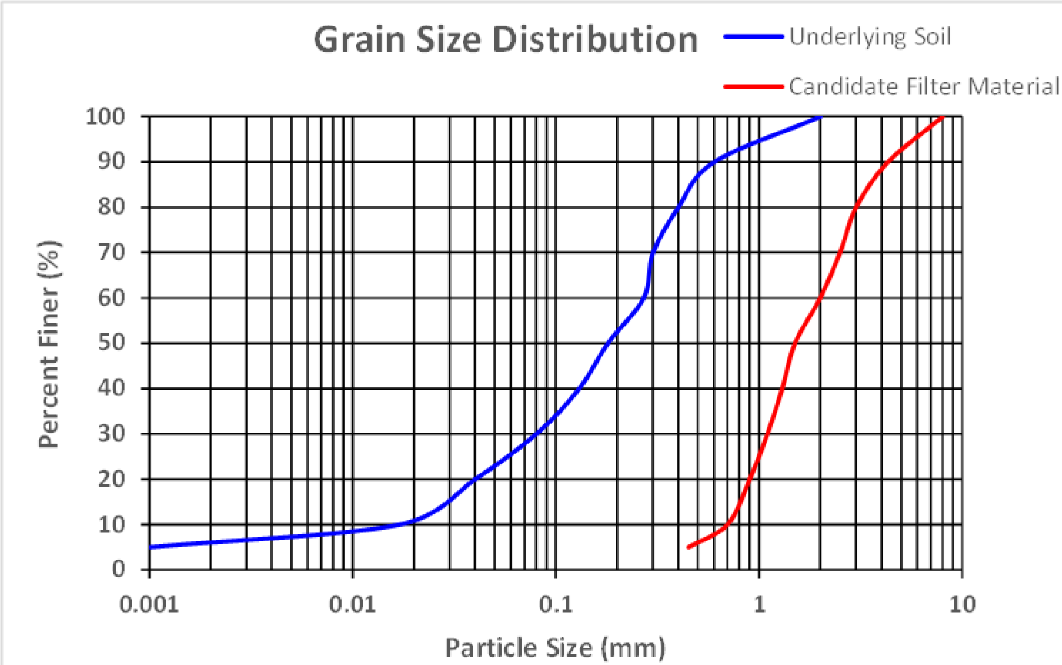


**Figure 28. Filter and Underlying Soil Grain Size Distribution.**

**Table 8. Design Example Soil, Granular Filter, and Riprap Parameters (Lagasse et al., 2006).**

| Soil Property | Native Soil | Riprap Class II |
|---|---|---|
| Hydraulic Conductivity K, cm/s | $4.2 \times 10^{-4}$ | - |
| Coefficient of Uniformity $C_u$ = d60/d10 | 0.25/0.015=16.6 | 2.1 |
| Median Diameter $d_{50}$, mm | 0.17 | 230 (9 in) |
| Plasticity Index | 3.3 | - |

**Step 1**: The properties of the native soil, the first granular filter, and the riprap are given in Figure 28 and Table 8.

**Step 2**: Since the base soil characteristics are known, enter the decision tree in Figure 27 with a soil that is "less than 50% fines and less than 90% gravel." Follow the decision tree down to the "open channel flow" box and select the "widely graded" branch because the native soil has a $C_u$ of 16.6 (Table 5), which is greater than 5. Determine the allowable limits on the $O_{95}$ opening of the geotextile. $O_{95}$ is the *AOS* (see Section 2.3). Next, the following inequalities are used to obtain the $O_{95}$ opening:

$O_{95} < 2.5(d_{50})$, so $O_{95} < 2.5(0.17$ mm), or 0.425 mm

$O_{95} < d_{90}$, so $O_{95} < 0.6$ mm

The first inequality is more restrictive than the second one, so the geotextile must have an AOS that is less than 0.425 mm, which is approximately equivalent to a #40 U.S. standard sieve size.

**Step 3**: Specify the geotextile, considering that its hydraulic conductivity should be at least 10 times greater than that of the native soil: $K_f = K_s \times 10 = 10 \times (4.2 \times 10^{-4}) = 4.2 \times 10^{-3}$ cm/s.

**Step 4**: The geotextile hydraulic criteria have now been established. Next, the geotextile must satisfy the strength criteria to avoid mechanical failure. Strength parameters such as grab, tear, and puncture strength are obtained from the geotextile manufacturer. These parameters will depend on the environment in which the riprap will be placed. They can be determined by using engineering judgment assisted by the contractor's method of installation if it is known. Based on the recommended values mentioned earlier in the design procedure, the following values can be used: 90 lb for grab strength (ASTM D 4632), 15 percent for elongation (ASTM D 4632), 40 lb for puncture strength (ASTM D 4833), and 30 lb for trapezoidal tear (ASTM D 4533).

**Step 5**: The long-term clogging of the geotextile must be considered next. For simplicity, after meeting the retention criterion, the geotextile having the largest average opening size is selected; this approach usually meets the POA criteria for clogging. This element is also given by the manufacturer for the available geotextile.

## 2.3. FILTER INSTALLATION AND PRICING

### 2.3.1. Filter Installation

A challenging aspect of using filters is proper installation because it can be complicated, particularly underwater. No specific methodology exists for the installation of the filter. However, some organizations have their own steps and guidelines. Expenses associated with a quality placement include expensive equipment and sometimes divers. Ephemeral streams are the most suitable sites to operate because the riprap and filter can be easily installed with the highest level of efficiency when there is no or minimal flow (Figure 29).

**Figure 29. Riprap Installation in Water (Dodd Construction, n.d.).**

**Granular Filter:** Ideally, particles should be installed evenly and carefully; they should not be simply dumped in place. It is extremely difficult to install riprap and filters in a riverine environment with high-flowing velocities because of the inability to control where the particles end up being placed. When installing in shallow water environments, it is preferred that personnel physically assist in the placement of the granular filter. When deeper water environments are encountered, divers are also preferred.

**Geosynthetic Filter:** According to NHRP 568 (Lagasse et al., 2006), most geotextiles that are used as filters beneath riprap are made of polyethylene or polypropylene. These materials have specific gravities around 0.90, which means they will float unless anchored down (Koerner, 1998). Flow velocities greater than about 1.0 ft/s (0.3 m/s) create large forces on the geotextile during installation. These forces cause the geotextile to act like a sail, often resulting in wavelike undulations of the fabric that are extremely difficult to control. In mild currents, geotextiles (precut to length) have been placed using a roller assembly, with sandbags attached to hold the fabric temporarily. To overcome these problems, engineers in Germany have developed sandbags that consist of two nonwoven geotextiles (or a woven and a nonwoven) with sand in between (Figure 30). This blanket-like product, known as SandMat, has layers that are stitch-bonded or sewn together to form a heavy, filtering recomposite. The composite blanket exhibits an overall specific gravity ranging from approximately 1.5 to 2.0, so it sinks readily. The survivability of the geotextile must also be considered; survivability refers to the ability of the geotextile to avoid excessive damage when placed against a rough or angular subgrade or against riprap materials, or upon impact with riprap during placement if drop heights are not minimized or carefully controlled (Dewey, 2014). Moreover, Brown and Clyde (1989) stated that one good installation practice is to ensure the geotextile does not extend into the channel and that it is wrapped around the toe.

**Figure 30. SandMat (Lagasse et al., 2009).**

A brief questionnaire was distributed to two contractors involved in riprap applications in Texas in order to assess what kind of filter is preferred from an installation point of view. When asked which type of filter is easier to install, one contractor said geosynthetic, while the other specified that on land a geosynthetic filter is easier to install, but in water, a granular filter is easier. These answers match what was mentioned in the literature; that is to say, a geosynthetic filter is easier to install, but when it comes to installing underwater, installation of geotextiles becomes problematic.

When asked about how filters and riprap are installed underwater, one of the contractors said that on rare occasions riprap is placed underwater. Dewatering or soil stabilization must be performed before riprap placement. The few times they placed concrete underwater, they placed filter fabric on the stream bed by securing 12- to 18-inch rocks on the corners, spreading it on the stream bed like a blanket, and then immediately placing 18- to 24-inch rock riprap with an excavator over the complete filter fabric area. A thin layer of 1.5- to 3-inch rock must be sprinkled over the large rock to lock the large rock in place. The other contractor said that if working from barges, the barge gets positioned, then rock is dragged or pushed off the barge. Underwater surveys determine whether placement is adequate; however, rock riprap has not been installed in a river on a Texas DOT (TxDOT) project in 23 years. Based on the contractors' experience in Texas, installing riprap and filters underwater is not very common, which is a helpful factor.

Existing literature, survey results (current practice), and contractor interviews (installation point of view) all reveal two major points: geosynthetic filters are easier to install; however, it becomes problematic when installing them underwater. Underwater, the most common approach is to dewater the area, which is why ephemeral environments are the most convenient since they are naturally dewatered during a given time of the year and the operations can take place. Because the contractors revealed that installing riprap underwater is not very common in Texas, the decision of whether to use a geosynthetic filter or a granular filter is much easier to make.

### 2.3.2. Filter Pricing

In terms of pricing, quantitative data are hard to describe because of the volatility of prices, which vary by time, region, contractor, and project conditions. However, geosynthetic filters are cheaper than granular filters, and according to both contractors who filled the questionnaire, geosynthetic filters are also cheaper to install than granular filters.

The two contractors each gave a total estimate of how much it costs to install riprap and a filter: one of them quoted $250 per cubic yard, and the other quoted between $150 and $200 per cubic yard.

## 2.4. CONCLUSIONS

The following list summarizes the findings on granular and geosynthetic filters presented in this chapter:

- There are two types of riprap filters: granular and geosynthetic.
- A filter enhances the efficiency of the riprap by not allowing erosion of the underlying soil through the riprap.
- Using a filter between the riprap and underlying soil improves the erosion resistance of the soil and of the riprap.
- Using a filter is better than using multiple layers of riprap without a filter.
- The main parameters in granular filter design are particle size distribution, permeability, porosity, thickness, and durability.
- The main parameters in geosynthetic filter design are opening size, permeability, porosity, thickness, and durability.
- The design procedures commonly used were described in step-by-step procedures and an example was given.
- Sometimes the design of a filter may require a combination of a granular and a geosynthetic filter or two layers of granular filters.
- Many techniques for filter installation exist that depend mainly on the type of filter, the contractor, and the environment. Placing riprap and filters underwater is the biggest challenge.
- Geosynthetic filters are easier to install than granular filters but are harder to install underwater.
- Granular filters are more expensive than geosynthetic filters.

# CHAPTER 3: STABILITY OF RIPRAP-COVERED SLOPES

## 3.1. INTRODUCTION

It is common to find riprap being placed on slopes, especially when protecting abutments from potential scour. Indeed, abutment scour can lead to disastrous failures. Riprap is a very widely used countermeasure for scour because of its availability and simplicity. When riprap is placed on slopes, a stability analysis is important. Many factors influence the stability analysis results, including different approaches and different soil and riprap parameters. The simulations for this report were performed using the RocScience Slide 2D software for 2D limit equilibrium method (Bishop's simplified method). Another factor related to riprap on slopes is the interaction between the riprap being placed on slopes and the flowing water. Many cases were reported in which riprap failure was caused by flow entrainment of the riprap particles. Sections below will describe in detail the approach and outcome of slope stability analysis for riprap-covered slopes.

## 3.2. ANALYSIS APPROACH

In this report, the stability of riprap-covered slopes was assessed using two different approaches. First, an effective stress analysis considering rapid drawdown conditions was simulated. Second, a total stress undrained analysis was performed. These analyses helped develop proposals for design guidelines for the stability of riprap-covered slopes. According to the TxDOT online *Geotechnical Manual* (2020), a factor of safety equal to 1.5 is required when designing a slope that will support structures such as bridge abutments.

To start with, a sensitivity analysis was performed in order to assess which parameters affect the analysis most. Indeed, there are a variety of parameters that come into play, such as various geomaterial types, including the native soil, the embankment fill, and the riprap, as well as different water conditions and slope geometries. The goal of the sensitivity analysis was to determine, given a riprap-covered slope angle, what soil parameters lead to a factor of safety of 1.5 for the following:

- Different soil conditions (fill, native soil, riprap)?
- Different water regimes (high water level, low water level, rapid drawdown)?
- Different types of analyses (undrained approach, effective stress approach)?

Then, the worst-case scenario was assumed to determine threshold values for shear strength parameters that should be adopted in design. Finally, a riprap layer was added on the slope in order to observe the effect of riprap loading on the slope factor of safety.

### 3.2.1. Parameters

The slope angles adopted for these analyses were 1:1 (45 degrees), 1.5:1 (33.7 degrees), 2:1 (26.6 degrees), 2.5:1 (21.8 degrees), and 3:1 (18.4). Based on the case histories that will be presented later, a slope height of 10 m was selected to be conservatively reasonable.

Three different materials were taken into consideration for the analysis: riprap stones, embankment fill, and native soil. Each of these materials has their own varying properties, such as unit weight, friction angle, and cohesion, for the sensitivity analysis (Table 9).

**Table 9. Riprap, Embankment Fill, and Native Soil Properties.**

| Natural Soil | | | Embankment Fill | | | Riprap | | | |
|---|---|---|---|---|---|---|---|---|---|
| Unit weight (kN/m3) | Cohesion (kPa) | Friction Angle (°) | Unit weight (kN/m3) | Cohesion (kPa) | Friction Angle (°) | Unit weight (kN/m3) | Friction Angle (°) | D50 (m) | Thickness (1.5D50) |
| 19 | 10 | 25 | 21 | 0 | 32 | 22 | 40 | 0.4 | 0.6 |
| 17 | 5 | 20 | 9 | 5 | 30 | 21 | 35 | 0.3 | 0.45 |
| 21 | 20 | 30 | 23 | 10 | 34 | 23 | 45 | 0.5 | 0.75 |

### *3.2.2. Riprap Parameters*

The material properties of the riprap stones were the same in effective stress and total stress analysis. The thickness and unit weight of the riprap layer was not found to have any major effect on the factor of safety, which is why one layer of riprap is always used. A cohesion value of zero is used for the riprap stones, as well as a friction angle of 40 degrees. One exception is the use of a friction angle of 45 degrees when the slope is very steep.

### *3.2.3. Embankment Fill Parameters*

For effective stress analysis with rapid drawdown, the embankment fill material was given a standard friction angle value of 34 degrees. This value is considered to be a reasonable assumption for fill material. Then, the cohesion of the embankment fills necessary to obtain a factor safety of 1.5 as required was back-calculated. For the total stress undrained analysis, the natural soil and the embankment fill were merged into one layer that behaves in an undrained fashion.

### *3.2.4. Natural Soil Parameters*

For the effective stress analysis with rapid drawdown, several runs were performed using varying parameters for the native soil and for the embankment fill. The failure surface was found to always be very shallow and either passing only through the riprap or through the embankment fill but not through the natural soil. As a result, constant $c$-$\phi$ parameters were used from that point on for the natural soil: $c'$ = 15 kPa and $\phi'$ = 25 degrees. However, for the total stress analysis, a single soil layer was assumed (natural soil), where the undrained shear strength $S_u$ is back-calculated in order to obtain a factor of safety of 1.5.

### *3.2.5. Effective Stress Analysis with Rapid Drawdown Parameters*

In the rapid drawdown case, the water level drops rapidly, and the pore water pressure in the soil is maintained because the drainage is not as rapid as the drawdown. For the slope stability, it means that the water support on the slope face decreases significantly (water lever down) and the shear strength in the soil mass has not changed (pore pressures and total stresses have not changed). For the effective stress analyses in this project, the slope was 10 m high, and the water level dropped from 9 m to 5 m above the bottom of the slope, which means a 4 m drop in the water level (Figure 31). In Figure 31 and the remaining figures in the report, the yellow layer represents backfill, the brown represents native soil, and the green represents riprap.

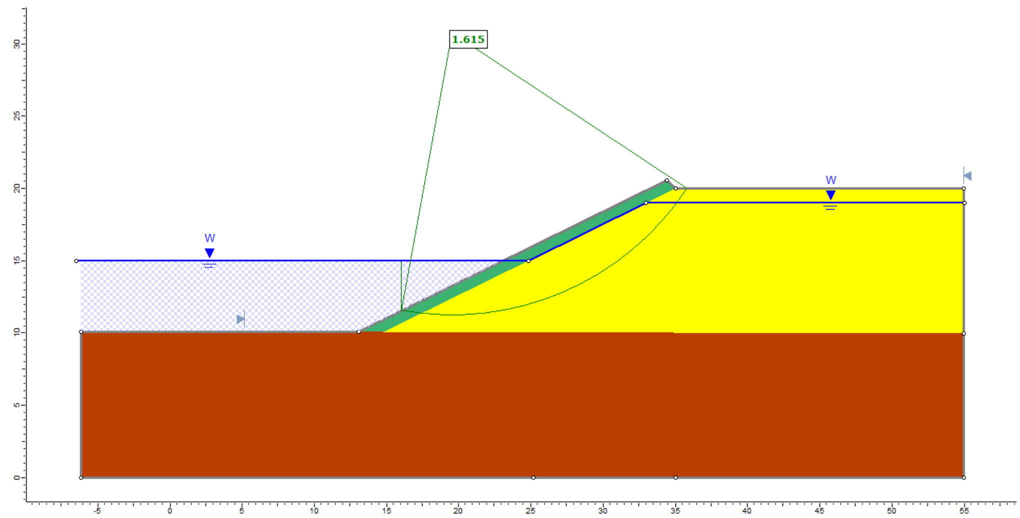



**Figure 31. Slope Stability Analysis Schematic.**

### *3.2.6. Total Stress Undrained Analysis Parameters*

In the total stress undrained analyses, the water level was assumed to be at mid-height of the slope, which is 5 m above the bottom of the slope based on a slope height of 10 m, as previously mentioned.

## 3.3. SLOPE STABILITY SIMULATIONS

This section conveys the results of the effective stress analysis with rapid drawdown and the results of the total stress undrained analysis for different slope angles.

### 3.3.1. Effective Stress Analysis with Rapid Drawdown

TxDOT guidelines require a 1.5 factor of safety for the slope before installing the riprap. As previously mentioned, the slope was 10 m high and for the rapid drawdown analysis, and the water level dropped 4 m (from 9 m to 5 m height of the slope). The riprap friction angle was 40 degrees, the native soil strength parameters were $c'$ = 10 kPa and $\phi'$ = 25 degrees, and the embankment fill had a friction angle of $\phi'$ = 34 degrees. The purpose of the analysis was to obtain the required cohesion in the fill material to satisfy the 1.5 factor of safety requirement for a given slope angle before installing the riprap cover and to analyze the effect of adding riprap on the 1.5 factor of safety.

**1:1 slope (45 degrees)**: The findings indicated that in this case the slope was too steep, which is one of the main causes of failure in the case histories. This slope requires relatively high strength properties for the soil, such as having a 30 kPa cohesion in the fill material, which is usually compacted sand (Figure 32). This cohesion is considered unusually high for fill material. After installing the riprap, the failure surface shown in Figure 33 is shallow and only passes through the riprap. Therefore, in this case, the factor of safety for the riprap-covered slope is directly related to the friction angle of the riprap; clearly, a slope angle of 45 degrees with a friction angle for the riprap of 40 degrees will lead to failure of the slope in the riprap. If a friction angle of 45 degrees is used for the riprap, the safety factor will be 1. A higher friction angle for the riprap is unrealistic, which means that a safety factor of 1.5 cannot be reached if the riprap is to be placed on a 1:1 slope. In this specific case, the slope geometry is the most influential factor, not the native soil or the embankment fill strength properties. As such, a 1:1 slope is not recommended for riprap applications because failure is highly likely.

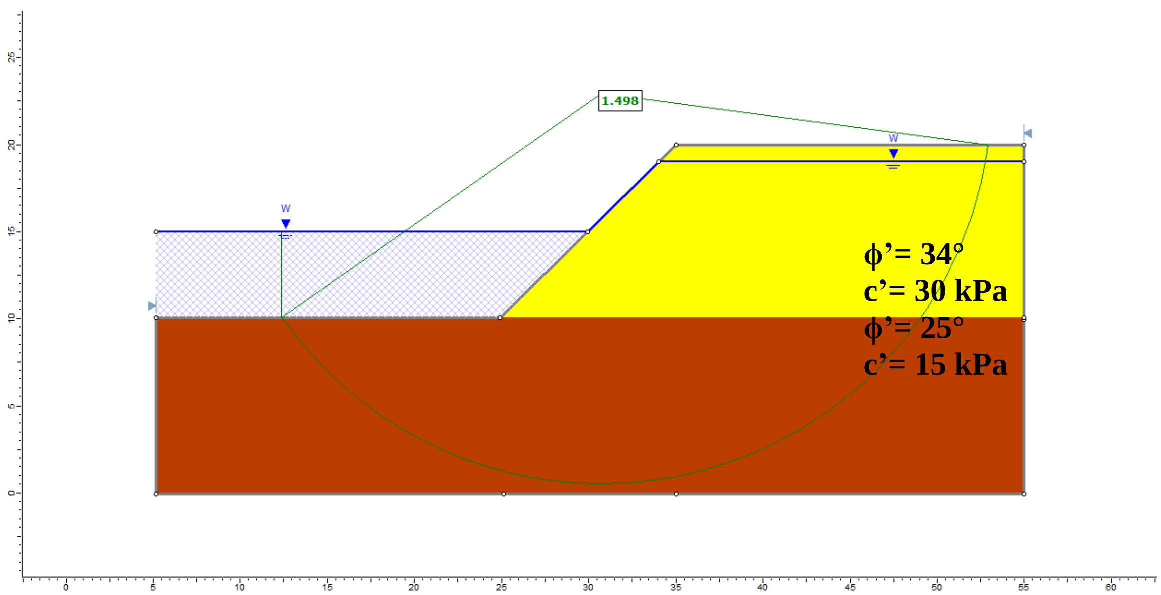



**Figure 32. 1:1 Slope Rapid Drawdown Effective Stress Analysis without Riprap.**

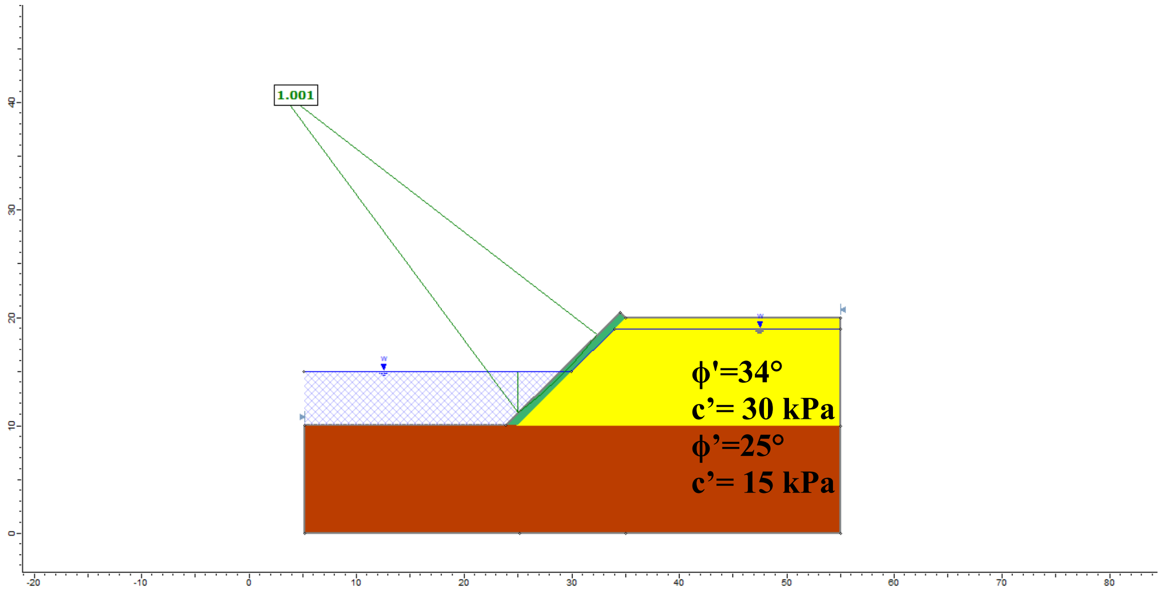



**Figure 33. 1:1 Slope Rapid Drawdown Effective Stress Analysis with Riprap ($\phi'_{riprap}$ = 45 degrees).**

**1.5:1 slope (33.7 degrees)**: To achieve a 1.5 safety factor for the slope without riprap, a well-compacted embankment fill material is needed with a 14 kPa cohesion, along with a medium-stiff native soil. For the parameters used in Figure 34, the slope without riprap has a factor of safety of 1.5. The factor of safety drops to 1.25 after installing the riprap (Figure 35) for a riprap friction angle of 40 degrees. However, since the failure surface passes only through the riprap layer, if the riprap friction angle is increased to 45 degrees, the factor of safety becomes 1.5. A 1.5:1 slope is not recommended because the requirements on the soil and riprap strength are quite high and seem to be at the upper end of the possible scale.

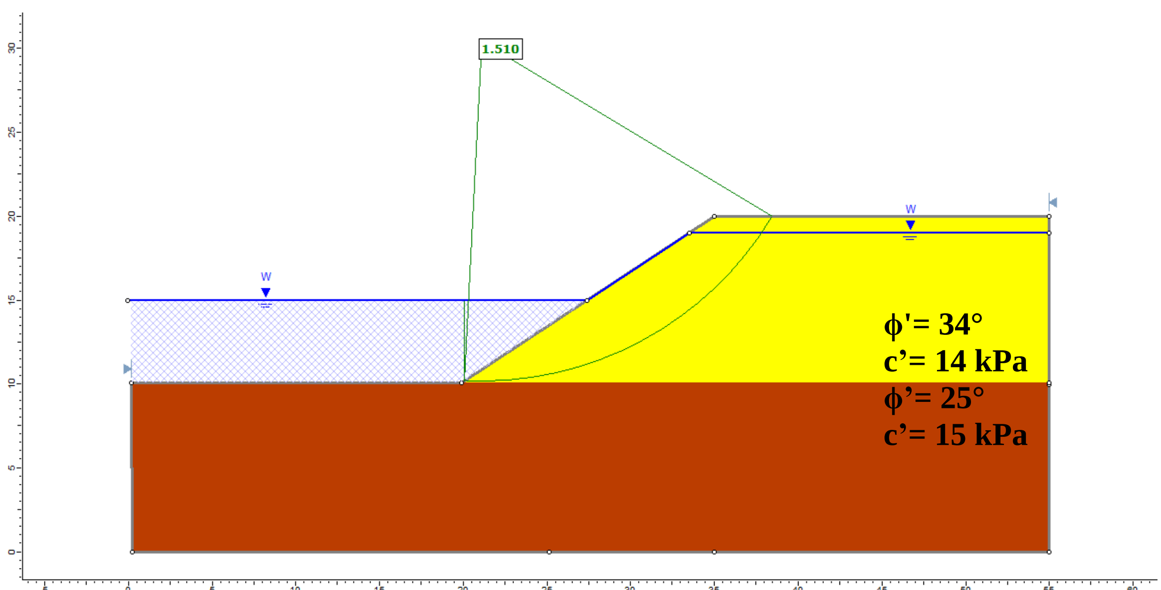



**Figure 34. 1.5:1 Slope Rapid Drawdown Effective Stress Analysis without Riprap.**

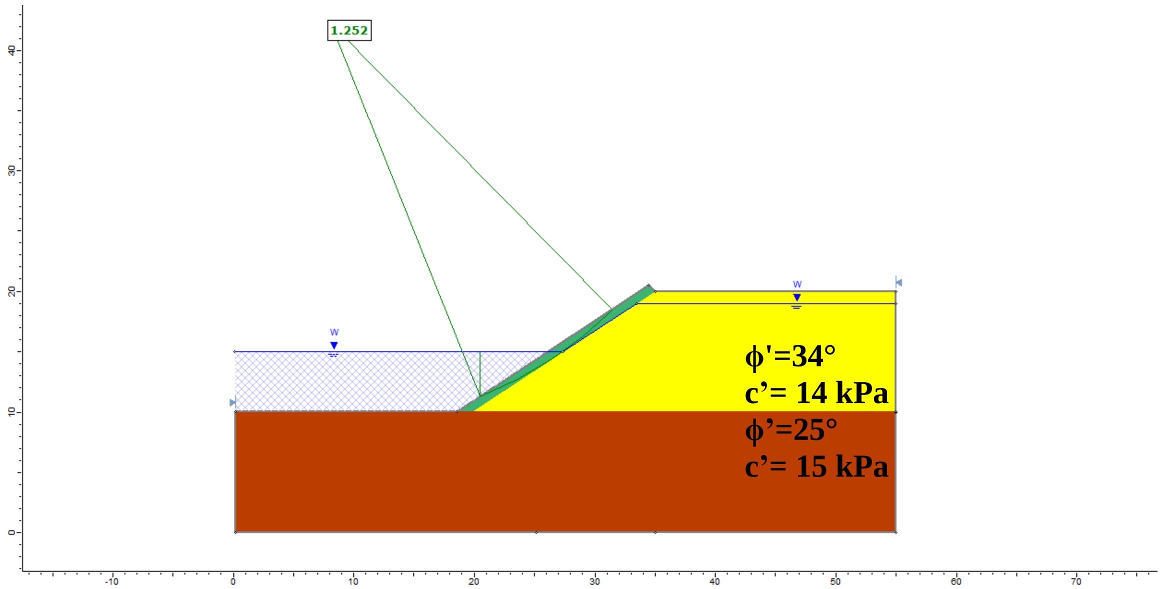



**Figure 35. 1.5:1 Slope Rapid Drawdown Effective Stress Analysis with Riprap ($\phi'_{riprap}$ = 40 degrees).**

**2:1 slope (26.6 degrees):** This is the most widely adopted geometry, and this slope rarely fails because of the reasonable factor of safety associated with it. A cohesion of 8.5 kPa was needed in a well-compacted embankment fill to meet the 1.5 factor of safety requirement without riprap, as shown in Figure 36. When the riprap was installed (Figure 37), the factor of safety increased by 6.7 percent, from 1.5 to 1.6. This result indicates that in this case it is safe to install riprap on a 2:1 slope that already has a factor of safety equal to 1.5, assuming rapid drawdown conditions. This design is the recommended configuration for riprap practice.

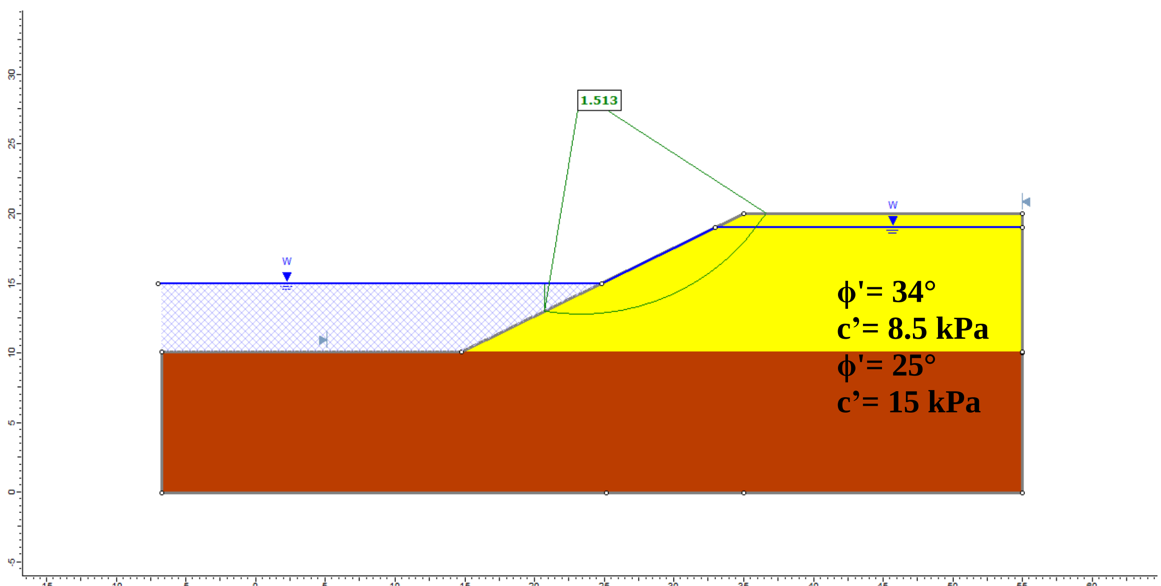



**Figure 36. 2:1 Slope Effective Stress Analysis without Riprap.**

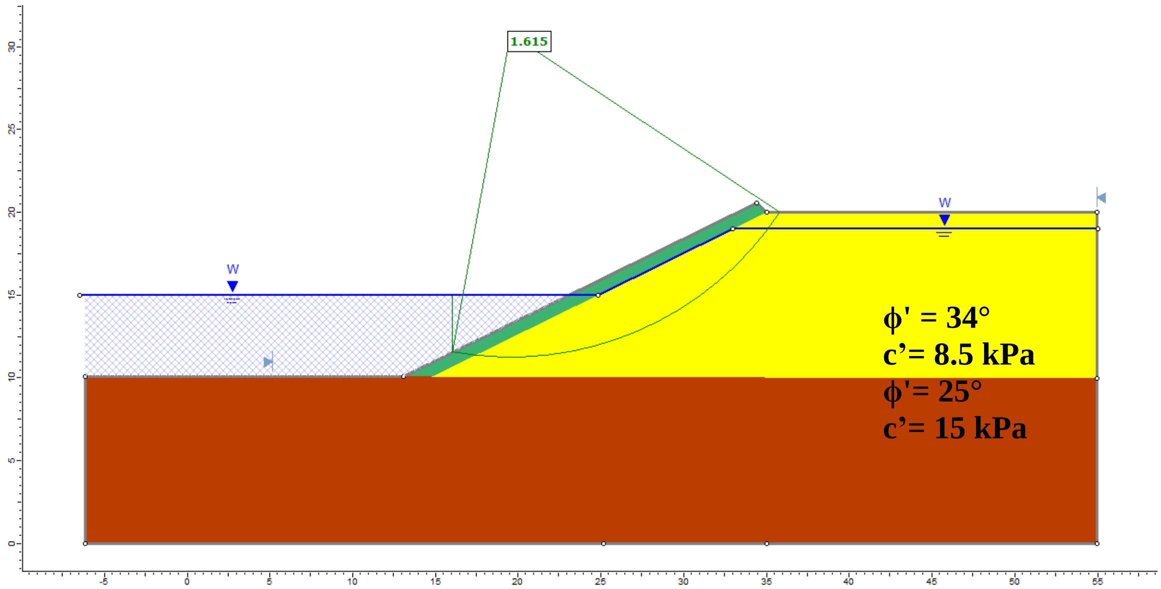



**Figure 37. 2:1 Slope Rapid Drawdown Effective Stress Analysis with Riprap ($\phi'_{riprap}$ = 40 degrees).**

**2.5:1 slope (21.8 degrees):** This scenario is a flatter slope, and the soil strength parameters do not have to be as high as in previous cases to satisfy the 1.5 factor of safety criterion per TxDOT guidelines. A well-compacted fill with a 6 kPa cohesion is sufficient to secure the 1.5 factor of safety for slope stability requirement. After adding the riprap on the slope, the factor of safety did not drop but instead increased by 10 percent, from 1.54 to 1.7 (Figure 38 and Figure 39), which means that when a 2.5:1 slope is properly designed per TxDOT guidelines, adding the riprap layer on the slope will not decrease the factor of safety of the riprap-covered slope.

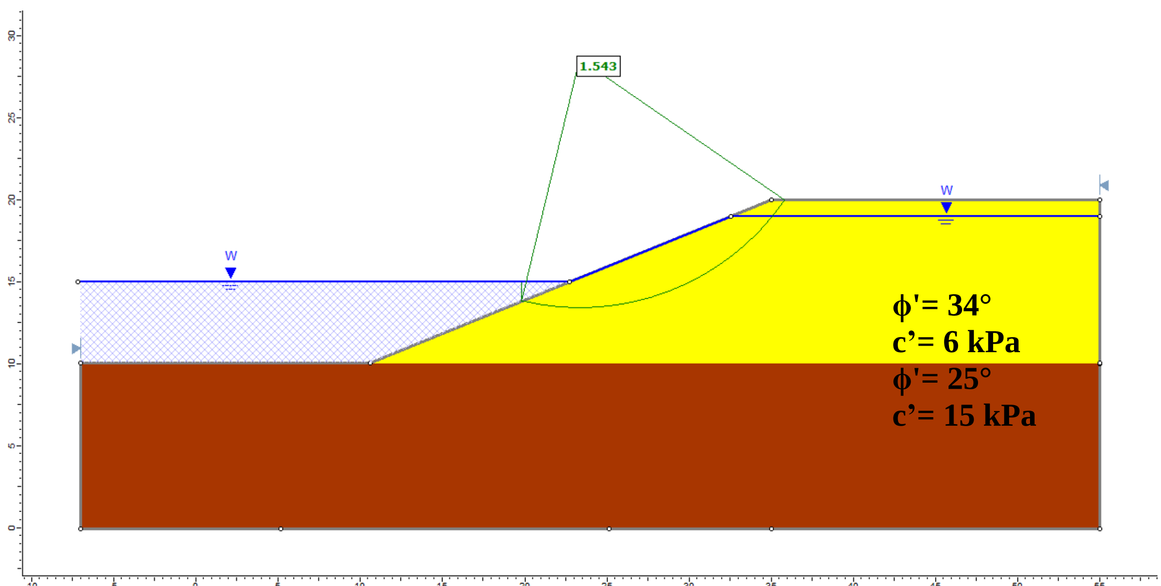



**Figure 38. 2.5:1 Slope Rapid Drawdown Effective Stress Analysis without Riprap.**

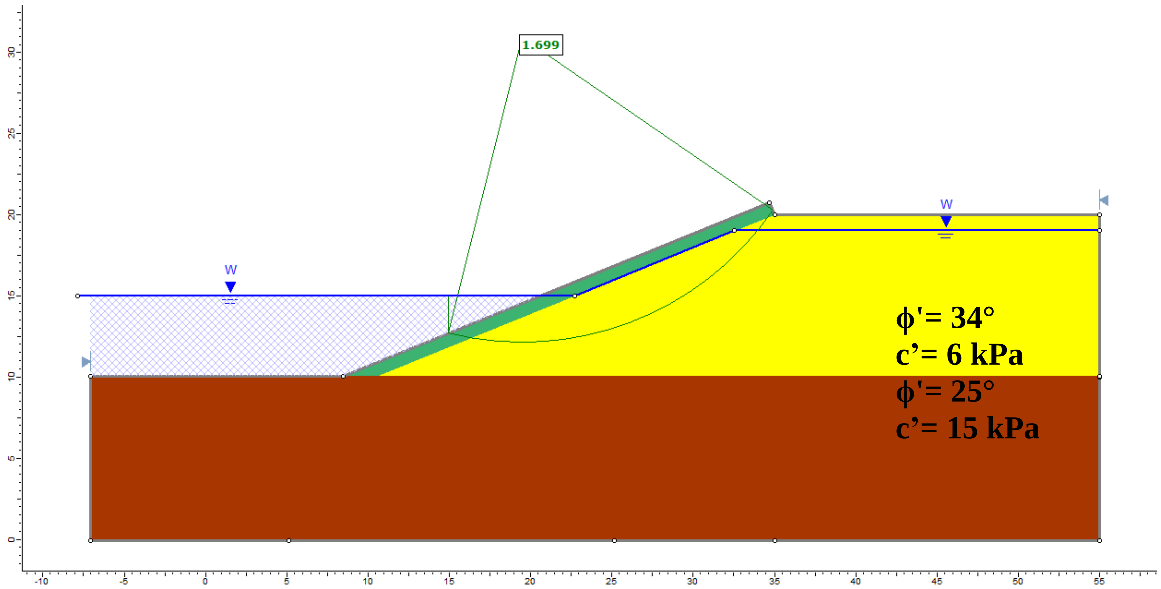



**Figure 39. 2.5:1 Slope Rapid Drawdown Effective Stress Analysis with Riprap ($\phi'_{riprap}$ = 40 degrees).**

**3:1 slope (18.4 degrees):** In this case, a cohesion of 3.5 kPa was necessary to obtain a 1.5 factor of safety. Installing the riprap on the slope having a factor of safety of 1.5 leads to a 14 percent increase in the factor of safety—to 1.79, which means that riprap loading will not affect the slope stability negatively. Figure 40 and Figure 41 show the slope stability analysis for this case.

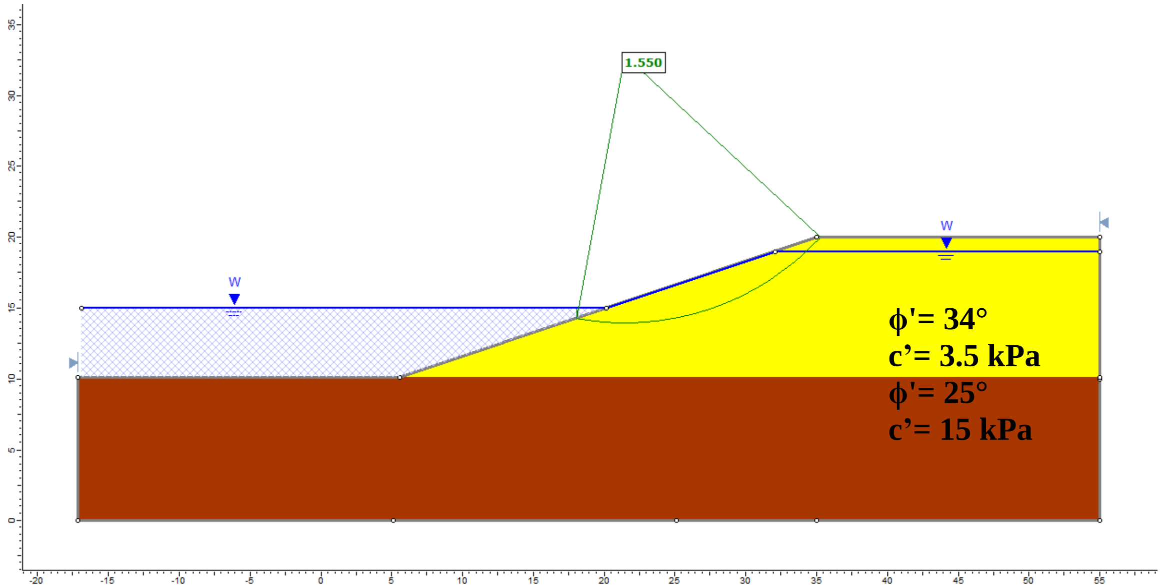



**Figure 40. 3:1 Slope Rapid Drawdown Effective Stress Analysis without Riprap.**

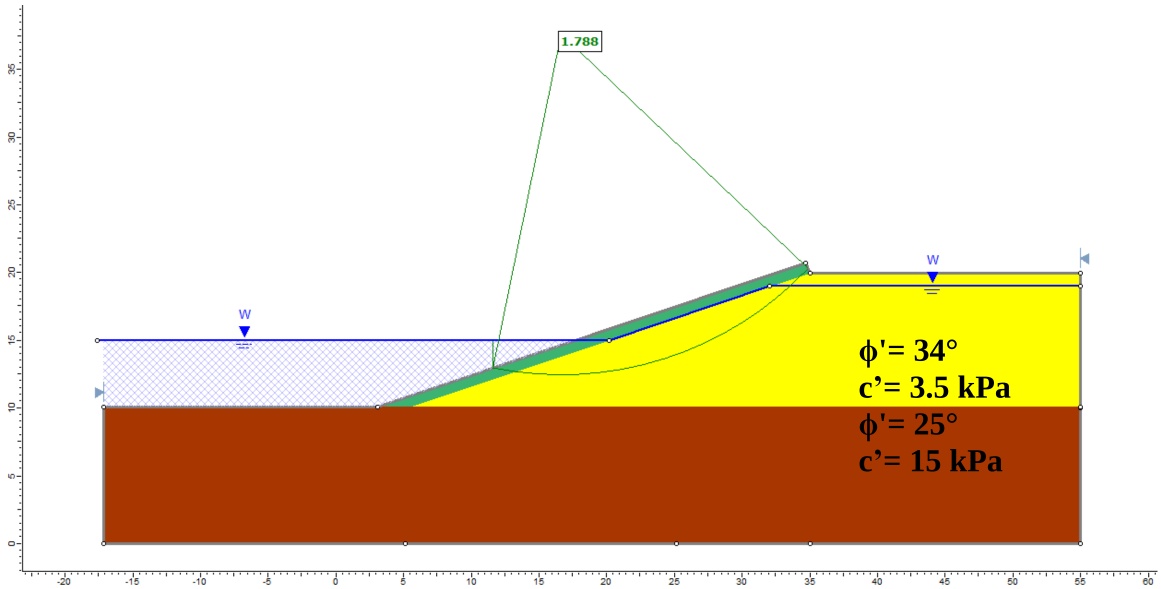



**Figure 41. 3:1 Slope Rapid Drawdown Effective Stress Analysis with Riprap ($\phi'_{riprap}$ = 40 degrees).**

### 3.3.2. Total Stress Undrained Analysis

As previously mentioned, for the total stress analysis, the water level was at mid-slope (5 m height of the slope), and the embankment fill and the natural soil were considered as being one soil mass. This scenario might represent the case of a bank slope at the juncture of the main channel and the flood plain. The purpose was to obtain the required undrained shear strength of the soil to satisfy the 1.5 factor of safety for a given slope angle without riprap. Next, the analysis was repeated after adding the riprap, and the impact on the 1.5 factor of safety was quantified. Threshold $S_u$ values were used to classify the native soil as soft, stiff, or medium strength. Each slope angle case was analyzed and will be explained separately. The outcome of this approach is similar to the one obtained in the effective stress analysis in terms of maximum allowable slope angle for required stability.

**1:1 slope (45 degrees):** An undrained shear strength $S_u$ for the native soil of 44 kPa is needed for the slope to meet the 1.5 factor of safety requirement per TxDOT guidelines. This value of 44 kPa is not uncommon in the field. However, when the riprap is added, the factor of safety drops drastically to 1, as shown Figure 42 and Figure 43. It can be seen that this is a shallow failure within the riprap layer itself and that a riprap friction angle of 45 degrees is necessary to get a factor of safety equal to 1. As determined previously, it was found that a 1:1 slope is not recommended because it is too steep for the riprap to be stable.

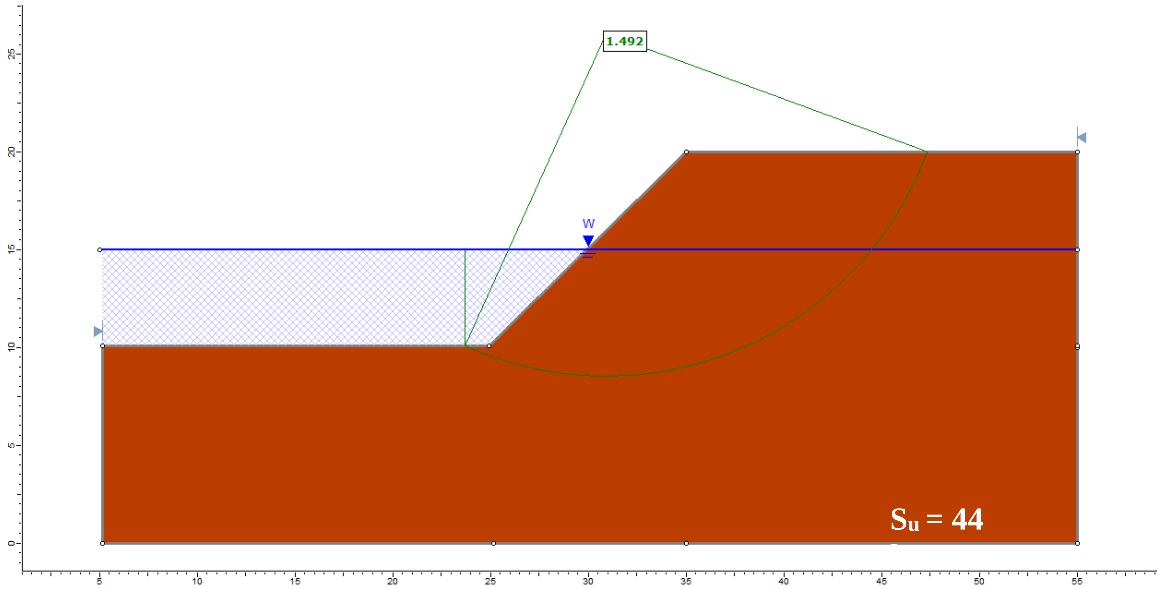



**Figure 42. 1:1 Slope Total Stress Analysis without Riprap.**

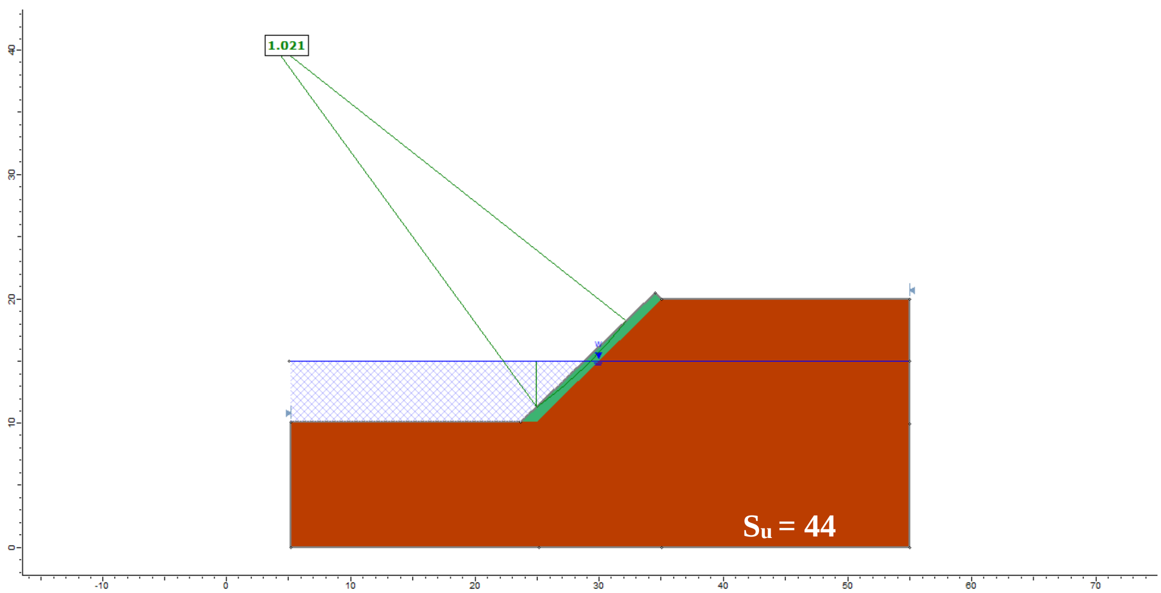



**Figure 43. 1:1 Slope Total Stress Analysis with Riprap ($\phi_{riprap}$ = 45 degrees).**

**1.5:1 slope (33.7 degrees):** This slope angle is also problematic because the results obtained from the total stress analysis are similar to those obtained from the effective stress analysis. An undrained shear strength of 43 kPa is needed to obtain a 1.5 factor of safety, and then when the riprap is installed, the factor of safety drops to 1.25 for a riprap friction angle of 40 degrees, and the failure occurs within the riprap itself (Figure 44 and Figure 45). Since the failure surface passes only through the riprap if the friction angle of the riprap stones is increased to 45 degrees, a factor of safety of 1.5 is obtained, but this friction angle is quite high, and a 1.5:1 slope is not recommended for riprap application.

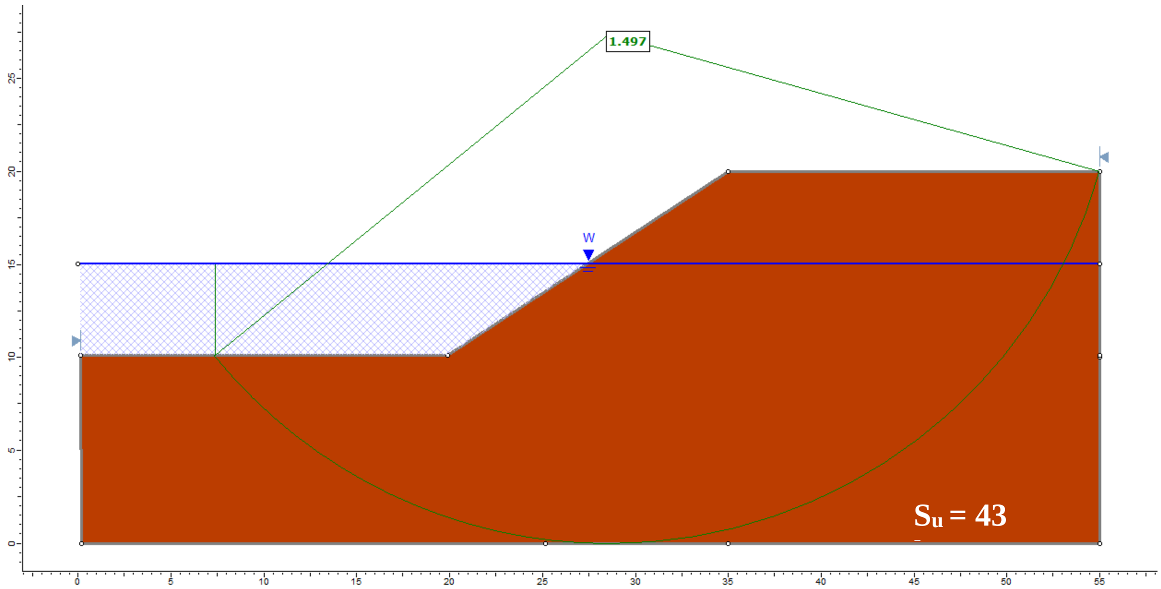



**Figure 44. 1.5:1 Slope Total Stress Analysis without Riprap.**

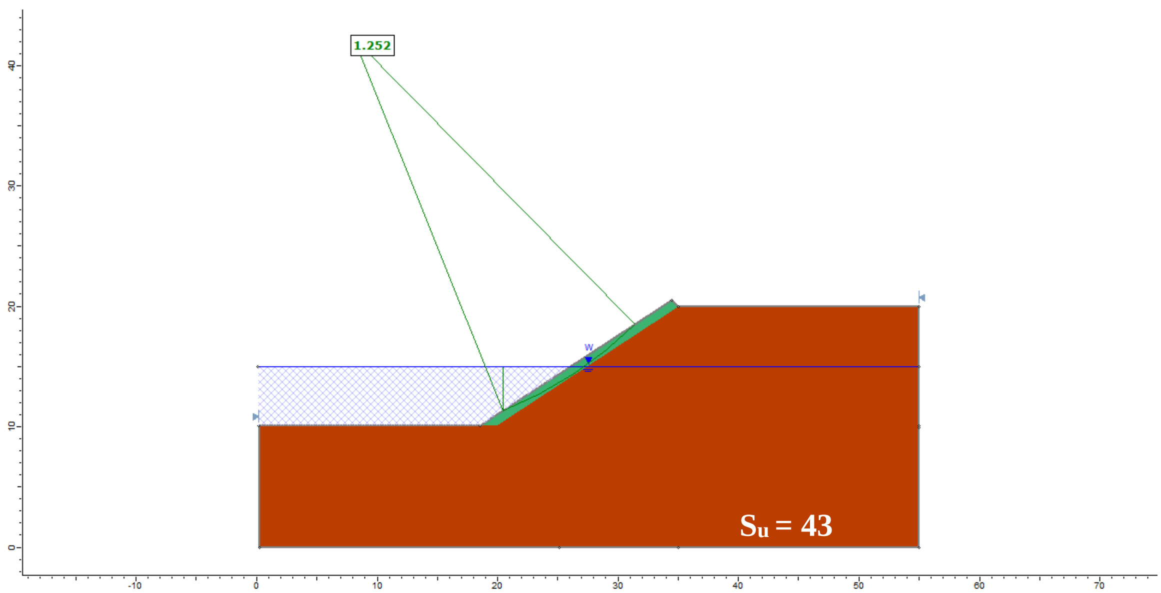



**Figure 45. 1.5:1 Slope Total Stress Analysis with Riprap ($\phi_{riprap}$ = 40 degrees).**

**2:1 slope (26.6 degrees):** In this case, an undrained shear strength $S_u$ = 39 kPa was needed to achieve the 1.5 factor of safety before installing the riprap. When the riprap layer is added, the factor of safety remains almost the same, as shown in Figure 46 and Figure 47, which means that, for the parameters used, when a 2:1 slope is properly designed, loading it with a riprap layer will not decrease the factor of safety. This result matches what was observed in the effective stress analysis with rapid drawdown.

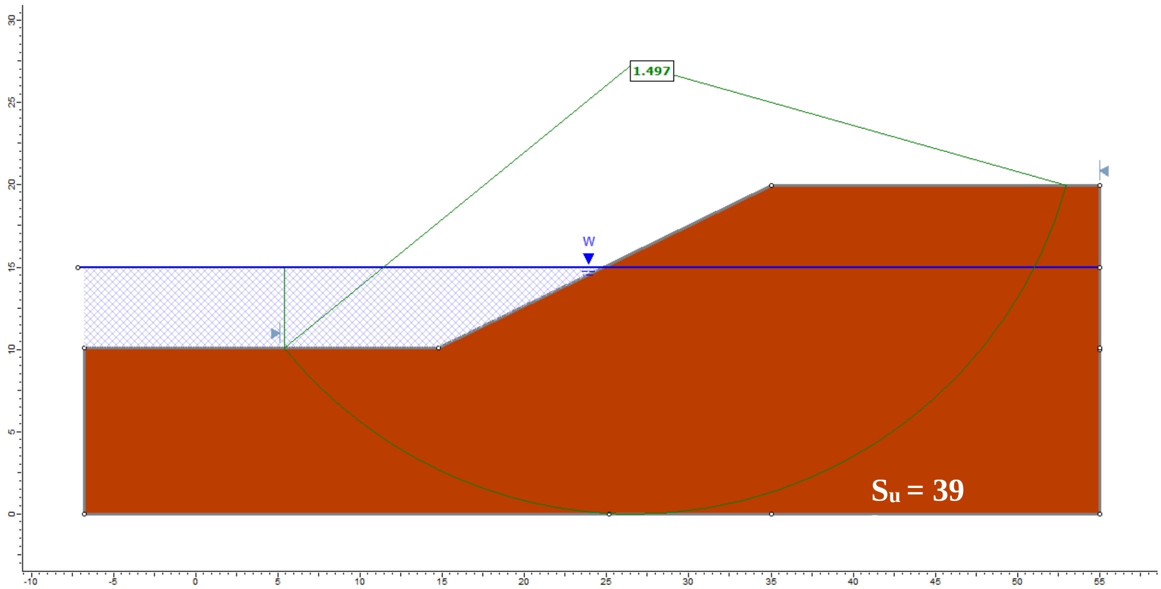



**Figure 46. 2:1 Slope Total Stress Analysis without Riprap.**

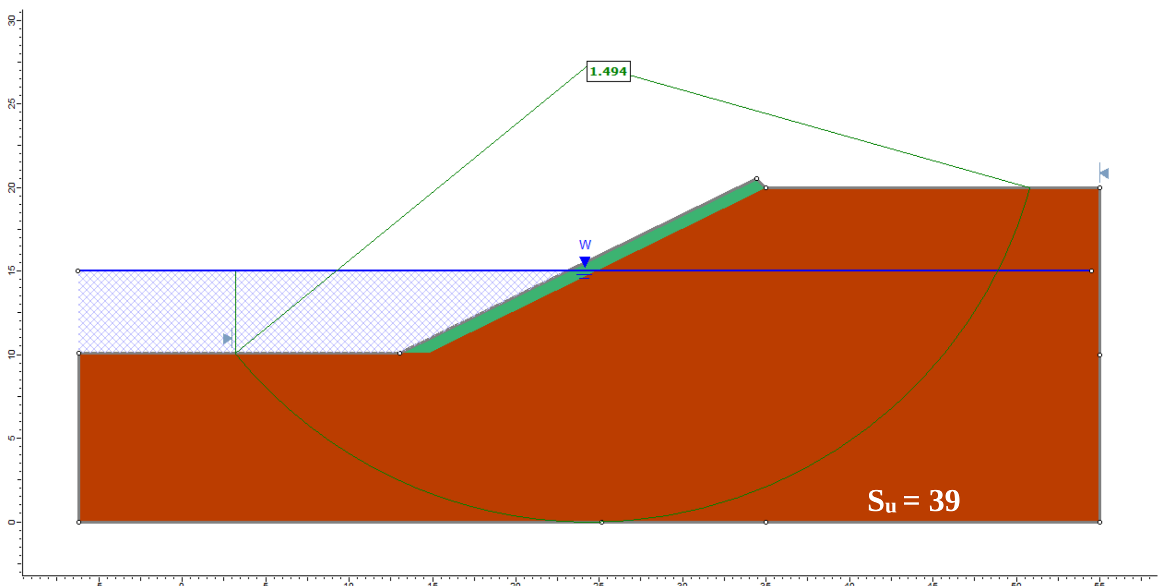



**Figure 47. 2:1 Slope Total Stress Analysis with Riprap ($\phi_{riprap}$ = 40 degrees).**

**2.5:1 slope (21.8 degrees)**: In this case, an undrained shear strength of 37 kPa is needed to satisfy the 1.5 factor of safety requirement. When the riprap layer is applied to the system, the factor of safety remains almost the same. Therefore, for the parameters used in this analysis, it is safe to install a riprap layer on a 21.8 degrees slope that has a factor of safety of 1.5. Figure 48 and Figure 49 show the results for this geometry.

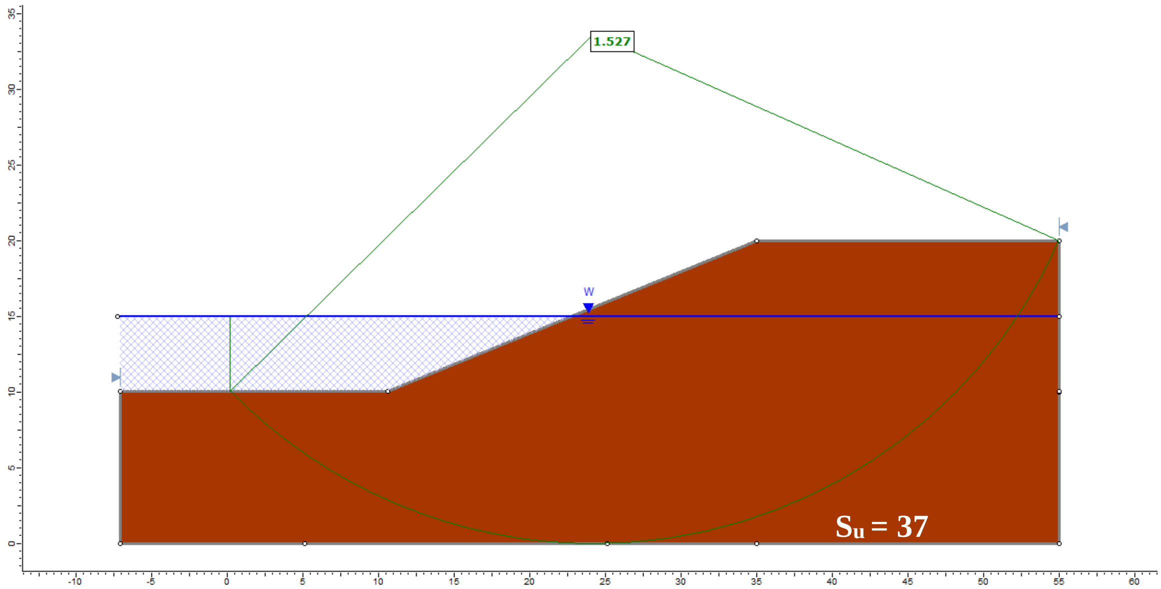



**Figure 48. 2.5:1 Slope Total Stress Analysis without Riprap.**

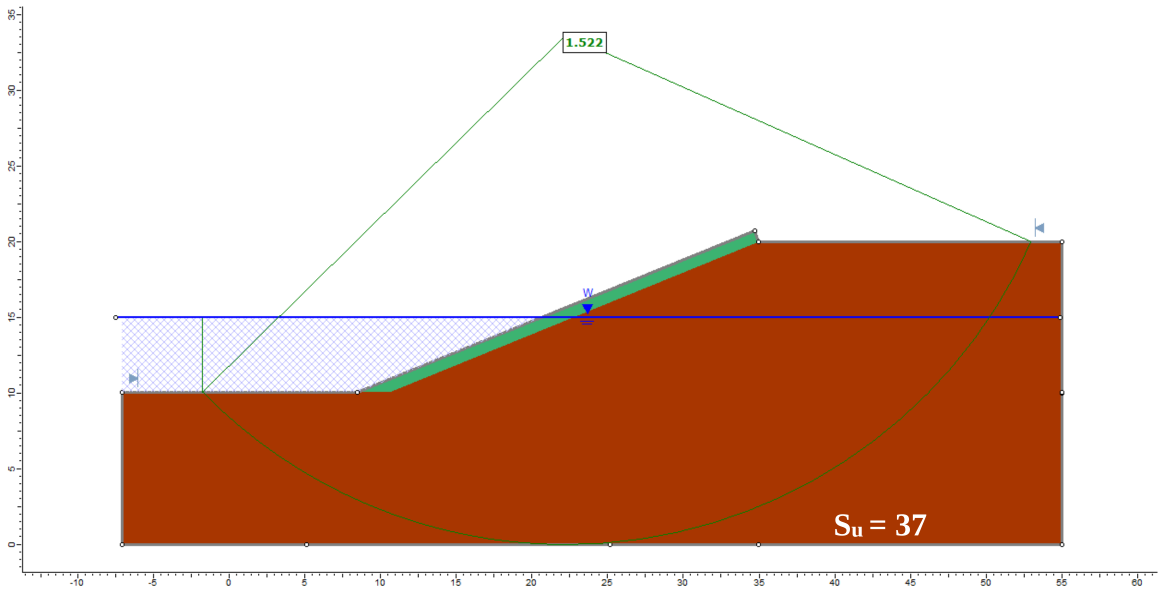



**Figure 49. 2.5:1 Slope Total Stress Analysis with Riprap ($\phi_{riprap}$ = 40 degrees).**

**3:1 slope (18.4 degrees)**: In this case and for the parameters used, an undrained shear strength of 36 kPa is required to obtain a 1.5 factor of safety. When the riprap layer is added on top of the slope, the factor of safety remains almost the same, as shown in Figure 50 and Figure 51.

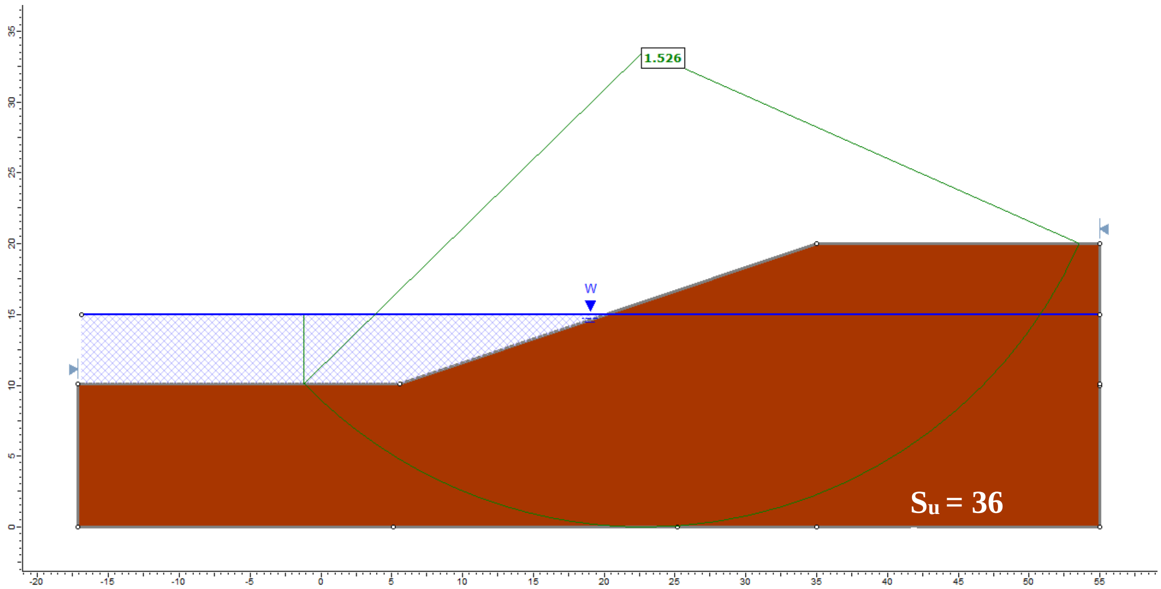



**Figure 50. 3:1 Slope Total Stress Analysis without Riprap.**

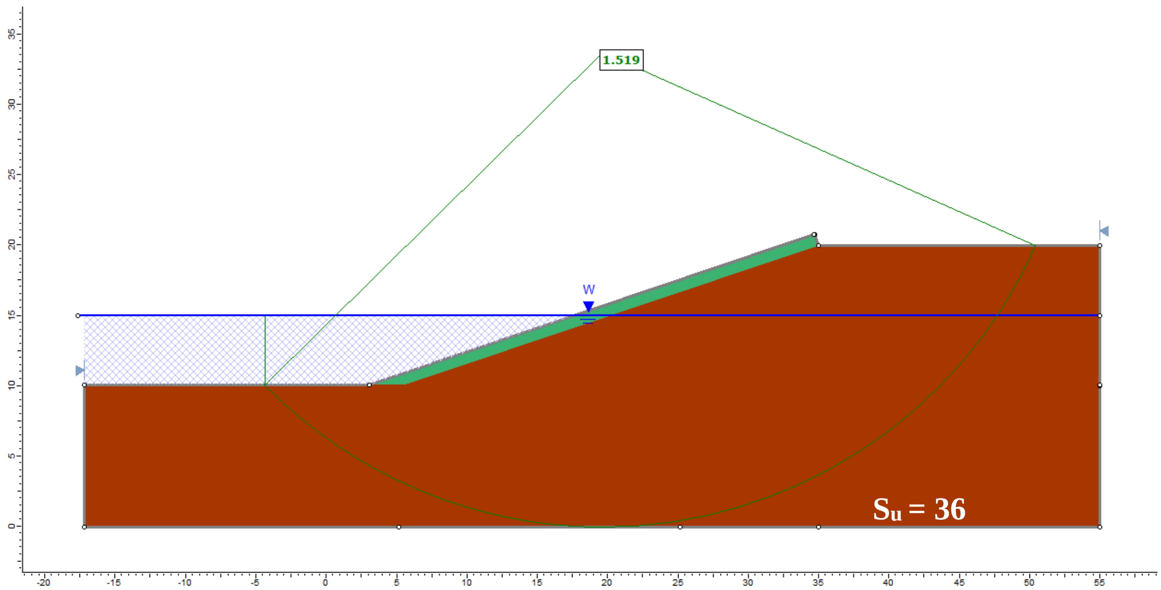



**Figure 51. 3:1 Slope Total Stress Analysis with Riprap ($\phi_{riprap}$ = 40 degrees).**

### 3.3.3. Summary

Table 10 is based on the results of the simulations described above and gives the strength threshold values needed for a riprap layer to be safely installed on a given slope.

**Table 10. Design Guideline Summary.**

| Required Parameters for Factor of Safety = 1.5 | | | | | | | |
|---|---|---|---|---|---|---|---|
| Slope Angle(°) | Slope (H:V) | Undrained Analysis * | | Effective Stress Analysis ** | | Factor of Safety | |
| | | Required Undrained Shear Strength (kPa) for FS = 1.5 | Classification | Given This Friction Angle (°) | Here is the Required Cohesion (kPa) for FS = 1.5 | Without Riprap | With Riprap |
| 45 | 1:1 | 44 | Medium-Firm | 34 | 30 | 1.5 | Riprap Friction Angle = 45°. Failure in the Riprap FS = 1 |
| 33.7 | 1.5:1 | 43 | Medium-Firm | 34 | 14 | 1.5 | Riprap Friction Angle = 40° Failure in the Riprap FS = 1.25 Riprap Friction Angle = 45° Failure in the Riprap FS=1.49 |
| 26.6 | 2:1 | 39 | Medium-Firm | 34 | 8.5 | 1.5 | Riprap Friction Angle = 40°. Failure in the Soil FS = 1.6 |
| 21.8 | 2.5:1 | 37 | Medium-Firm | 34 | 6 | 1.5 | Riprap Friction Angle = 40°. Failure in the Soil FS = 1.7 |
| 18.4 | 3:1 | 36 | Medium-Firm | 34 | 3.5 | 1.5 | Riprap Friction Angle = 40°. Failure in the Soil FS = 1.78 |

* For the Undrained Analysis, the parameters belong to one soil layer (Natural Soil and Embankment fill).
** For Effective Stress analysis, constant parameters are assumed for the natural soil c=15 kPa and φ=25°, and the parameters in the table belong to the embankment fill material.

The results of the effective stress analysis with rapid drawdown and the results of the total stress undrained analysis are consistent and show the steepest slope on which a layer of riprap can be safely placed. As mentioned previously, for the effective stress analysis, the approach was to determine the cohesion value necessary in the embankment fill to obtain a 1.5 factor of safety for the slope assuming a 34-degree friction angle for the embankment fill, given that $c'$ = 15 kPa and $\phi'$ = 25 degrees for the natural soil below. For the undrained total stress analysis, a single soil layer was considered, and the approach was to determine the undrained shear strength of the slope material necessary to obtain a 1.5 factor of safety. All the undrained shear strength values required for the total stress analysis fell in the same range of medium-firm clay, which is relatively common in the field. However, in the effective stress rapid drawdown analysis, a cohesion of 30 kPa was required for the 1:1 slope; such an effective stress cohesion value is unusually high. In addition, when the riprap layer was added, the factor of safety will drop to 1. Similar results were obtained for the 1.5:1 slope. For the 2:1 slope and for flatter slopes, the cohesion values required are less than 10 kPa, which is much more likely to exist. For steep slopes, the problem is not in the native soil but in the riprap layer itself. Assuming a high friction angle of 45 degrees for riprap will yield a maximum factor of safety of 1 when placed on a 1:1 slope, which is why it is not recommended to use slopes steeper than 2:1 for riprap applications.

In order to assess the effect of the friction angle on the required cohesion to achieve a factor of safety of 1.5, the following sensitivity analysis was performed using Taylor's chart (Briaud, 2013) for a dry soil. Table 11 shows the required cohesion value to maintain a factor of safety of 1.5 on a given slope angle, given a certain friction angle. Friction angle values of 28, 30, 32 and 34 are considered for each slope angle. A standard unit weight value of 19 kN/$m^3$ was assumed for the homogenous soil layer, with a slope height of 10 meters.

**Table 11 Effect of Friction Angle Variation on Required Cohesion Value Using Taylor Chart.**

| Taylor Chart | | | | |
|---|---|---|---|---|
| | Slope | Slope Angle (°) | ϕ' (°) | c' (kPa) |
| FACTOR OF SAFETY = 1.5 | 1:1 | 45 | 34 | 14.3 |
| | | | 32 | 15.7 |
| | | | 30 | 18.5 |
| | | | 28 | 20 |
| | 1.5:1 | 33.7 | 34 | 5.7 |
| | | | 32 | 7.1 |
| | | | 30 | 10 |
| | | | 28 | 11.4 |
| | 2:1 | 26.6 | 34 | 1.4 |
| | | | 32 | 2.9 |
| | | | 30 | 4.3 |
| | | | 28 | 5.7 |
| | 2.5:1 | 21.8 | 34 | 0 |
| | | | 32 | 0 |
| | | | 30 | 1.4 |
| | | | 28 | 2.9 |
| | 3:1 | 18.4 | 34 | 0 |
| | | | 32 | 0 |
| | | | 30 | 0 |
| | | | 28 | 0 |

### 3.4. FLOW THROUGH RIPRAP NUMERICAL SIMULATION

In the design of the riprap layer, the bank slope angle is important. It is important not only for the mechanical stability of the riprap (slope stability) but also for the hydraulic stability of the riprap (erosion displacement through entrainment). Indeed, the steeper the slope is, the less likely the riprap is to resist the water velocity. It seems reasonable to assume that the critical velocity of the riprap (water velocity at which the riprap stones start moving) is less for a steep slope than it is for a flatter slope, and that assumption is investigated numerically in this section. Four different models were constructed by using the computational fluid dynamics–discrete element method (CFD-DEM) and the STAR-CCM+ software. The flow is simulated with the $k - \varepsilon$ turbulence model, and the discrete element model is utilized to model the riprap particles. The efficiency and accuracy of the CFD-DEM model was validated by comparing the simulation predictions with the results of several erosion tests previously performed on sand and gravel in the EFA. Figure 52 shows two tests comparing the results obtained in the EFA (experimental) and in the STAR-CCM+ software (numerical); the good match gave credibility to the simulations.

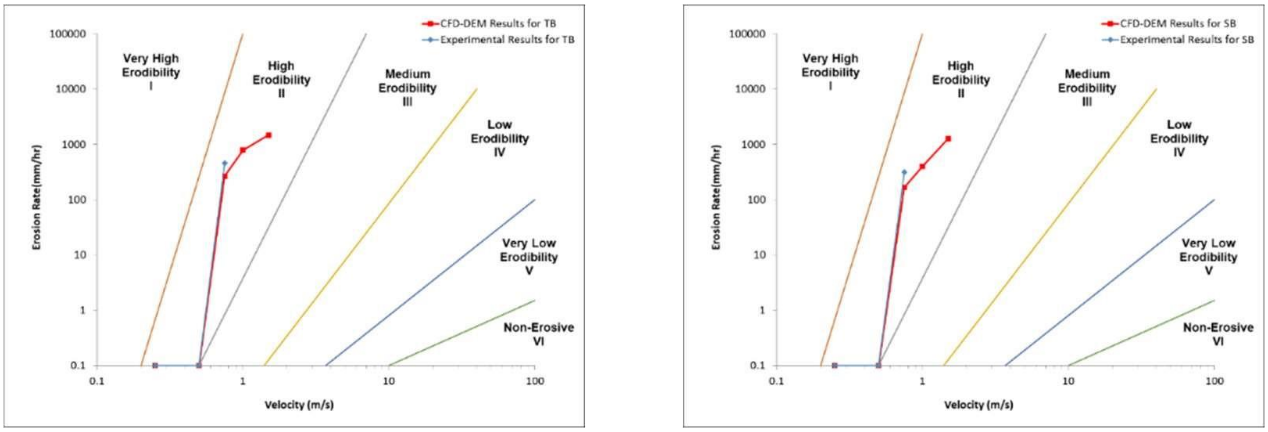



**Figure 52. Comparing Numerical and Experimental Erosion Testing.**

Subsequently, four different slopes were modeled to study the effect of the slope angle on the critical velocity of the riprap. The particles used in the DEM to simulate the riprap were spheres with a 15 cm dimeter. The dimensions of the flat model utilized for the simulation are shown in Figure 53. The inclined models had the same overall dimensions but different lateral bed slopes: 3:0.5 (9.5 degrees), 3:1 (18.4 degrees), and 3:2 (33.7 degrees). Table 12 summarizes the physical properties used in this simulation. The flow comes into the channel from the inlet on the left with the intended velocity and exits from the outlet on the right. For each velocity, 10 seconds of flow time is simulated. The mass of particles that exit from the outlet during these 10 seconds is considered to be a measure of the magnitude of erosion.

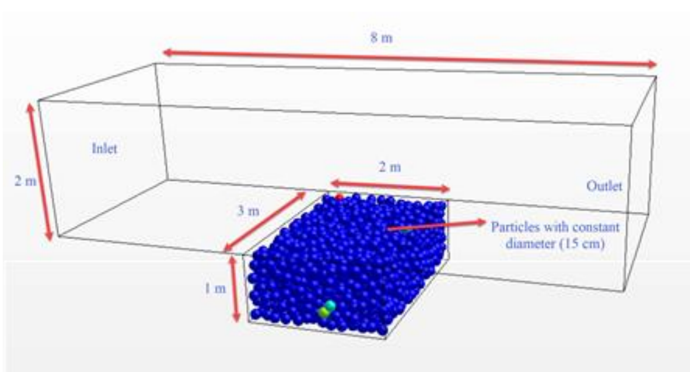


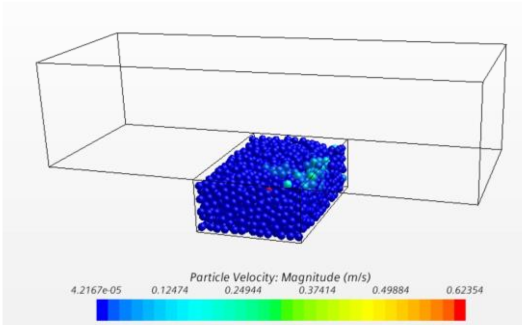

**Figure 53. Geometry Adopted for Simulation.**

**Table 12. Physical Properties of Particles and Fluid for the Simulation.**

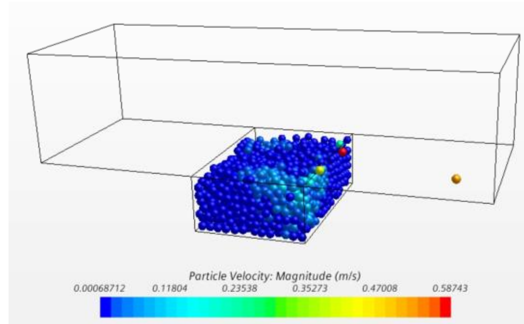
| Parameter | G2 |
|---|---|
| Density ($kg/m^3$) | 2650 |
| Modulus of Elasticity (Pa) | 1×10 |
| Poisson Ratio | 0.3 |
| Friction Coefficient | 0.4 |
| Normal Restitution Coefficient | 0.5 |
| Tangential Restitution Coefficient | 0.5 |
| Coefficient of Rolling Resistance | 0.2 |
| Computational parameters | |
| DEM time step | 4.35 × 10 |
| CFD time step | 1 × 10 |
| CFD mesh size (mm) | 60 × 60 × 60 |
| CFD turbulence model | k−ε |
| Drag Force Model | Schiller-Naumann |

Figure 54 shows the results after 10-second simulations for velocities equal to 1 m/s, 2 m/s, 3 m/s, and 4 m/s. As expected, the magnitude of erosion increases with an increase in velocity in the case of the flat surface.

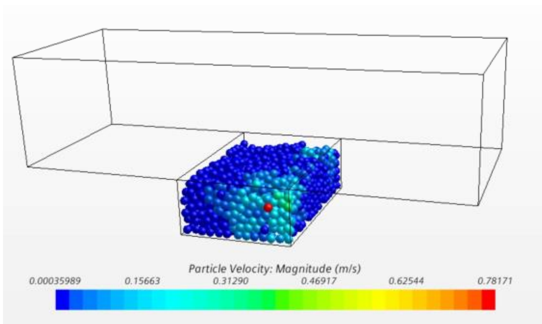
Figure 55, Figure 56, and Figure 57 show the results of the simulations for different slope angles and different velocities. The final values of erosion after 10 seconds of flow are plotted in Figure 58. Figure 58 shows both an increase in erosion mass and a decrease in critical velocity as the slope angle increases, which can be attributed to the fact that the initial shear stress mobilized to keep the riprap particles stable on the slope increases as the slope angle increases; it leads to an associated initial instability of the particles on the 3 to 0.5 slope (especially due to their spherical shape).



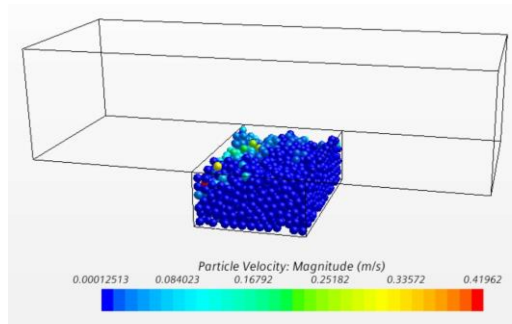

a) V = 1 m/s




b) V = 2 m/s

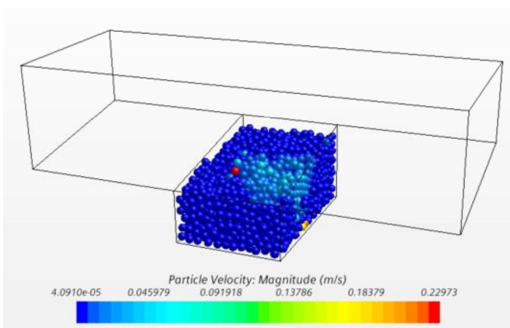



c) V = 3 m/s

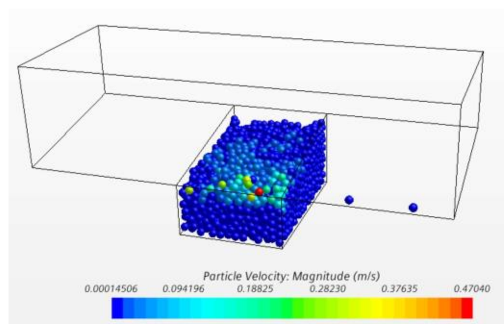



d) V = 4 m/s

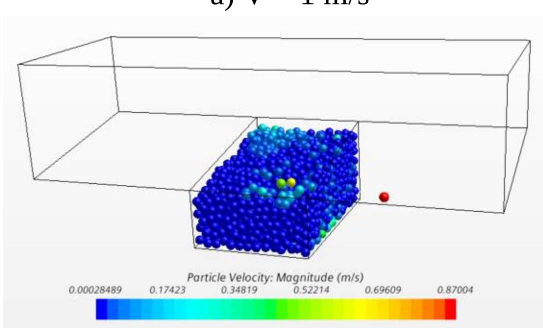
**Figure 54. CFD-DEM Simulation of Flat Slope for Different Flow Velocities.**



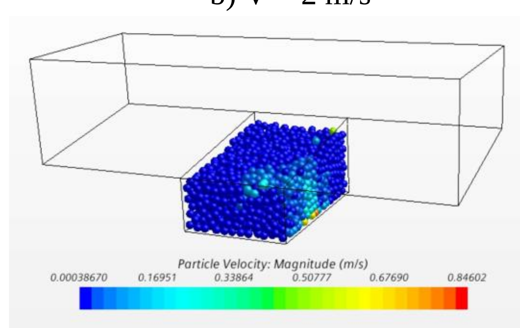

a) V = 1 m/s




b) V = 2 m/s

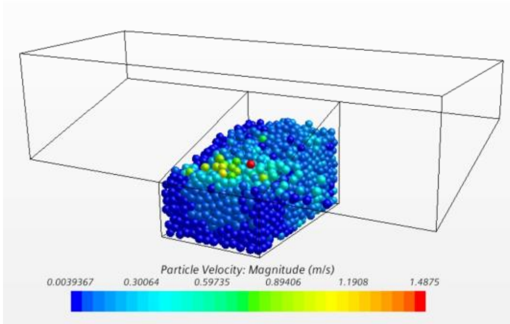



c) V = 3 m/s

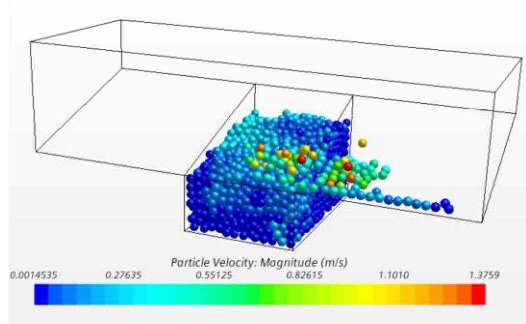



d) V = 4 m/s

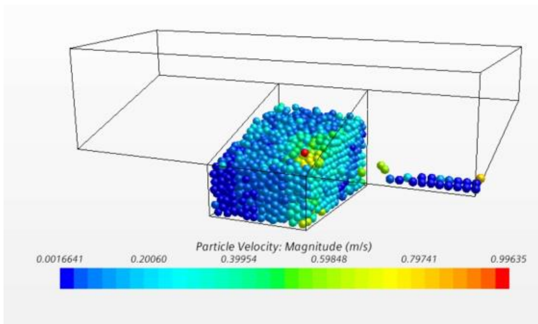
**Figure 55. CFD-DEM Simulation of the Sloped Case (3:0.5) for Different Flow Velocities.**



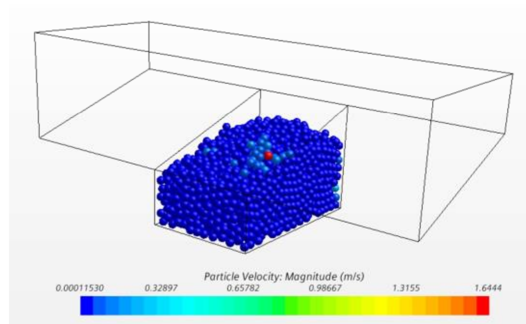

a) V = 1 m/s




b) V = 2 m/s

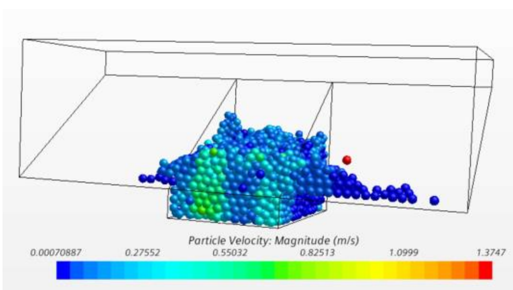



c) V = 3 m/s

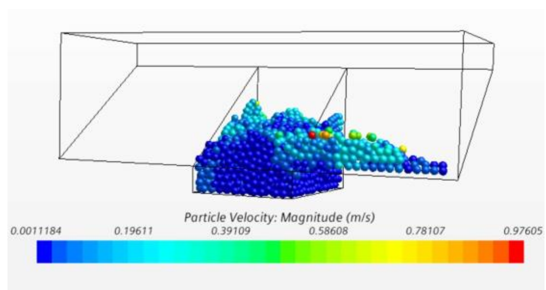



d) V = 4 m/s

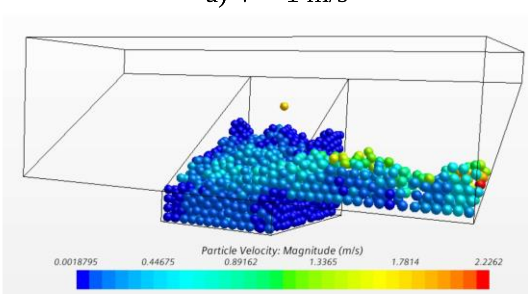
**Figure 56. CFD-DEM Simulation of the Sloped Case (3:1) for Different Flow Velocities.**



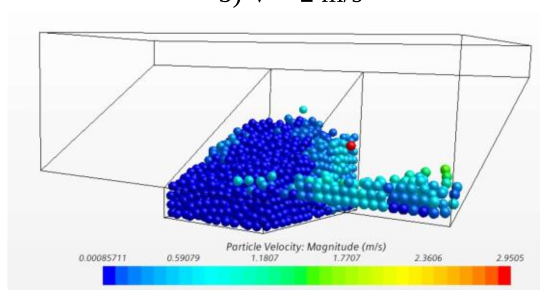

a) V = 1 m/s




b) V = 2 m/s




c) V = 3 m/s




d) V = 4 m/s

**Figure 57. CFD-DEM Simulation of the Flat Case Sloped Case (3:2) for Different Flow Velocities.**

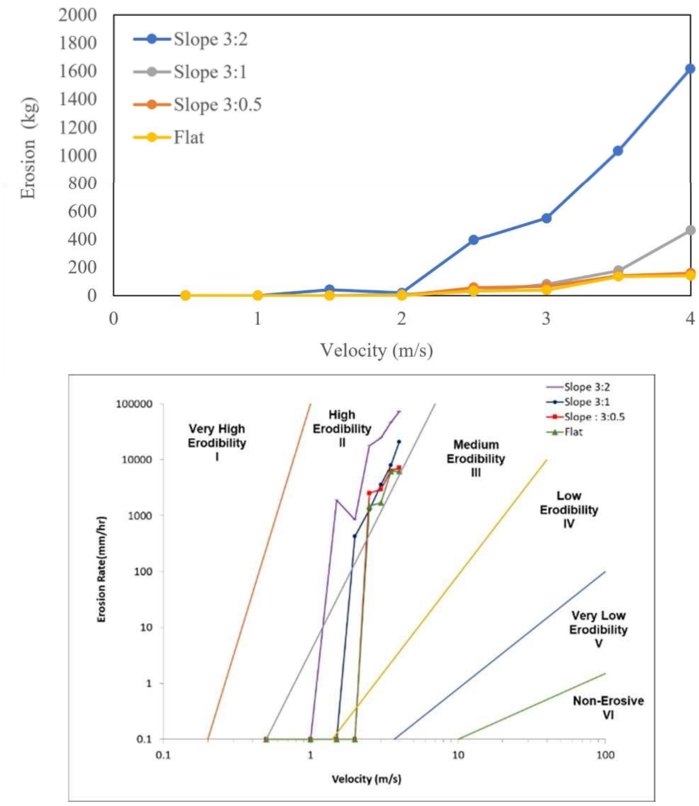



**Figure 58. Erosion Mass (kg) and Erosion Rate (mm/hr) versus Velocity for the Flat Case and Cases with Different Slopes.**

The stability of the particles can be increased by creating more friction between the particles. To study this effect, another shape of particles was created by combining two spherical particles (particle clumps). Figure 59 shows the non-spherical particles and the container full of simulated non-spherical particles. Figure 60 presents the simulation for a 3:2 slope with the non-spherical particles, and the final values of erosion and critical velocity are shown in Figure 61 and Figure 62. As can be seen, the non-spherical particles lead to less erosion and higher critical velocities than the spherical ones. Therefore, particle shape does impact erosion parameters.

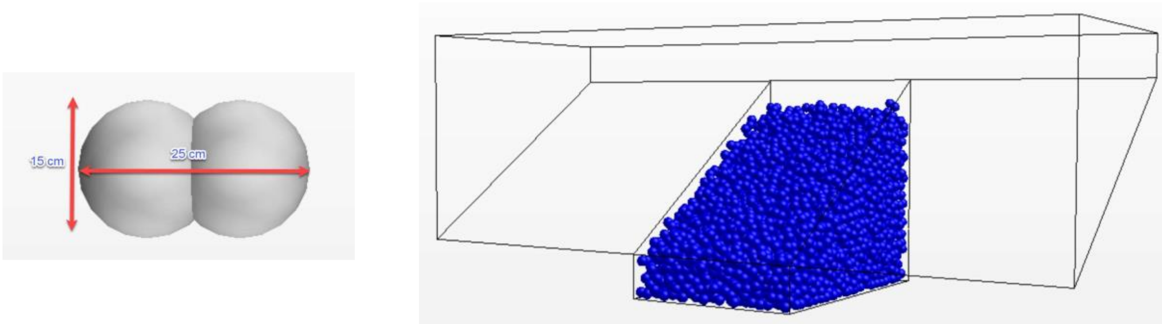
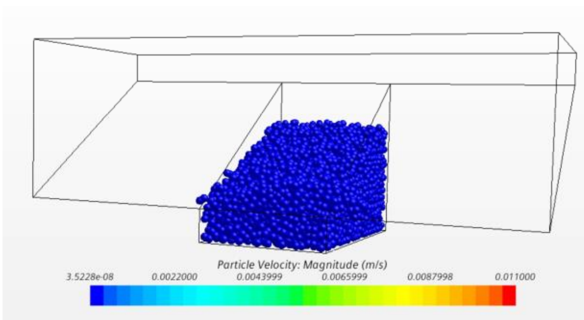
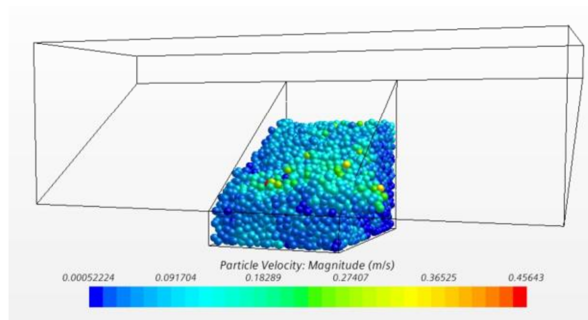


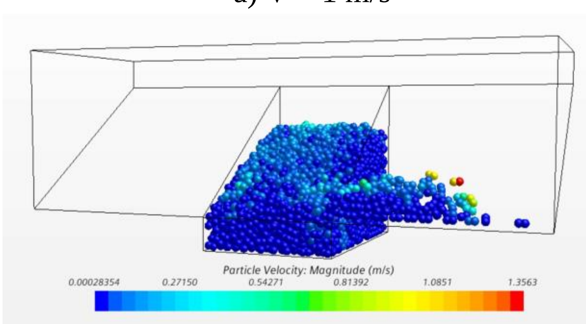

**Figure 59. Non-spherical Particle Shape and the Simulated Model before Application of Flow.**



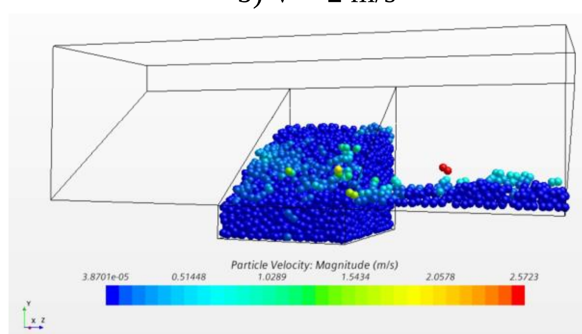

a) V = 1 m/s




b) V = 2 m/s




c) V = 3 m/s




d) V = 4 m/s

**Figure 60. CFD-DEM Simulation of the Flat Case Sloped Case (3:2) for Different Flow Velocities and Non-spherical Particles.**

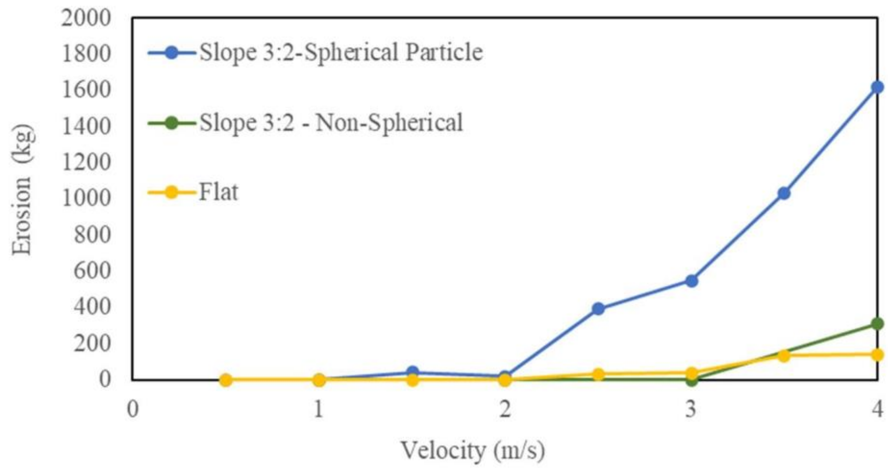



**Figure 61. Effect of Particle Shape on Critical Velocity.**

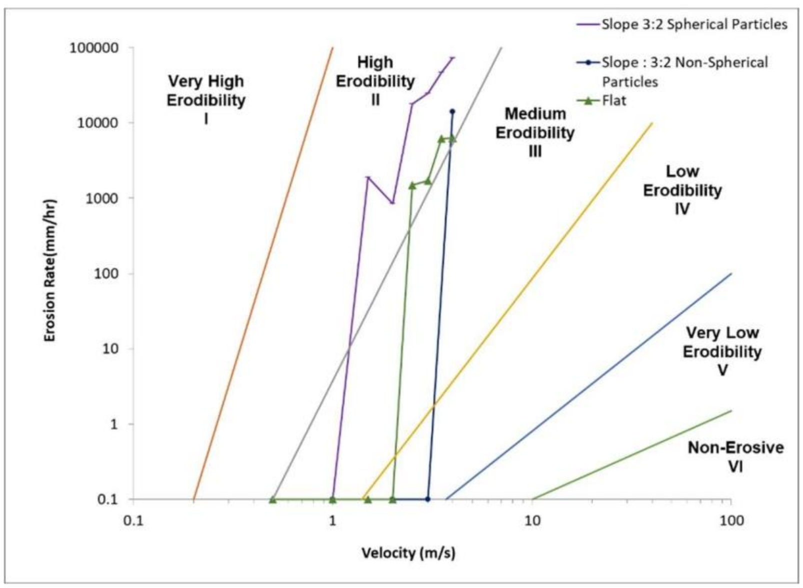



**Figure 62. Erosion Mass (kg) and Erosion Rate (mm/hr) versus Velocity for the Flat Case and a Sloped Case with Two Different Shapes of Particles.**

More experimental and numerical studies are needed to assess this phenomenon and reach firm conclusions about the effect of the slope angle on the critical velocity of the riprap. Although it is beyond the scope of this project, it is a very important issue since a lot of riprap failures are reported as riprap particle movement, riprap particle erosion, or riprap washing away. A chart showing the critical velocity versus riprap particle size for various slope angles would be very valuable. Figure 63 shows a graph that gives the required $d_{50}$ size of a given material to be able to resist erosion at a given velocity for flat surfaces. The simulations above show that this critical velocity might actually decrease with an increasing slope angle.

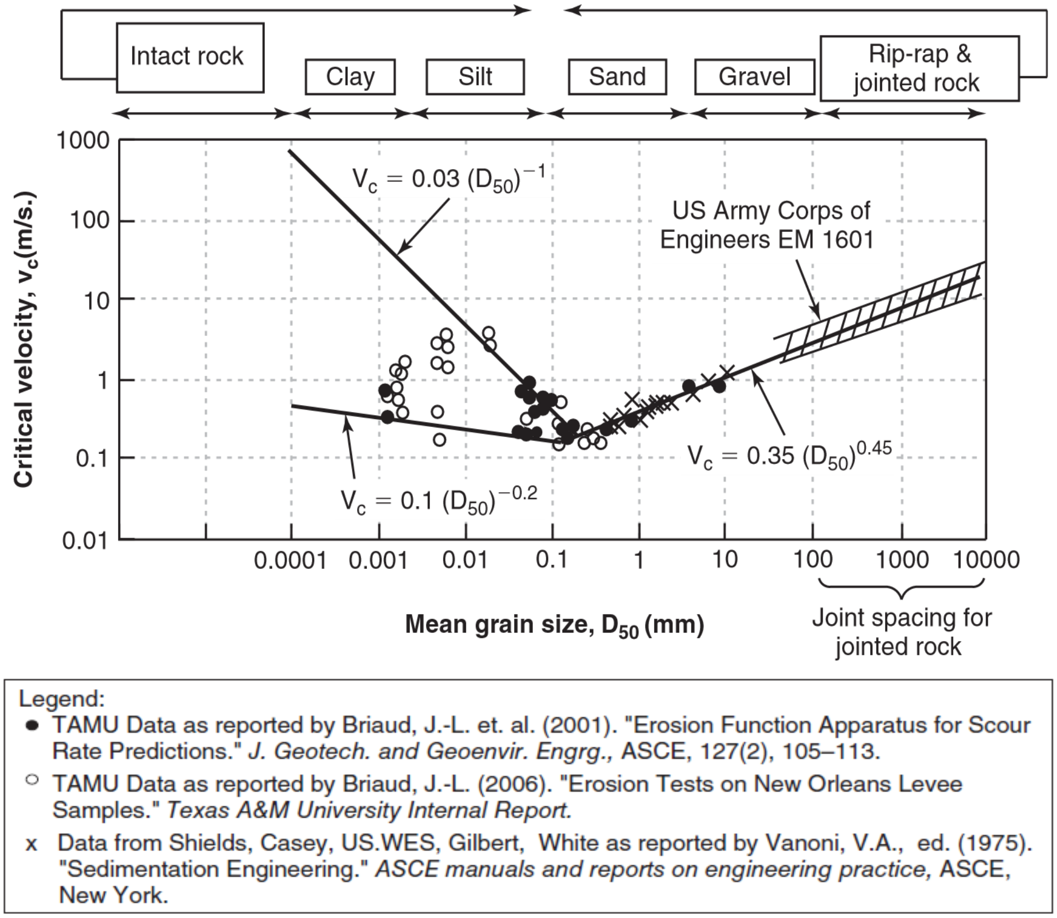



**Figure 63. Required $d_{50}$ in Function of the Critical Velocity (Briaud, 2008).**

## 3.5. CONCLUSIONS

Two different approaches were taken for the analysis—effective stress analysis with rapid drawdown and total stress undrained analysis; both used Bishop's method.

Based on the analyses performed, the following conclusions were reached:

- For 1:1 and for 1.5:1 slopes, the riprap is very likely to fail.
- The steepest slope where riprap can be safely installed while meeting the TxDOT design requirements is a 2:1 slope (26.7 degrees).

Riprap on slopes have less erosion resistance than riprap on flat ground. The steeper the slope is, the more likely the riprap is to move during high flow. Further research is needed to quantify this phenomenon.

# CHAPTER 4: RIPRAP FAILURE CASE HISTORIES

## 4.1. INTRODUCTION

Riprap failures are relatively common, but when the structure (i.e., bridge) has not failed, the riprap is simply repaired. A lot of the case histories that were identified are more than 30 years old. Figure 64 shows a picture of riprap installed on a slope. The main sources of case histories described in this report are the *USGS Rock Riprap Design for Protection of Stream Channels Near Highway Structures* (Blodget & McConaughy, 1986); NCHRP Report 568: *Riprap Design Criteria* (Lagasse et al., 2006); *California Bank and Shore Rock Slope Protection Design Practitioner's Guide and Field Evaluations of Riprap Methods* (Racin et al., 2000), and a survey that was sent by the receiving agency through the TxDOT mailing list of all the DOTs across the United States.

**Figure 64. Riprap on Slope (TranBC, 2011).**

## 4.2. RIPRAP FAILURE CASE HISTORIES

### 4.2.1. SH 80 over San Antonio River, Texas

#### *4.2.1.1. Introduction*

The site visit with Jean-Louis Briaud, Jerome Sfeir, and Anna Shidlovskaya took place on Sunday February 21, 2021. The information obtained regarding this case history before the visit was from a previous report, *Realtime Monitoring of Bridge Scour Using Remote Monitoring Technology* (Briaud et al., 2010).

#### *4.2.1.2. Site Details*

The site investigated was a riprap-covered slope next to one of the piers at a bridge near Karnes City, Texas, on SH 80 crossing the San Antonio River. A riprap failure was observed in 2010; the failure occurred around a pier located on the slope between the main channel and the flood plain. The geographic coordinates are 28° 56'16.47" N 97° 50'03.40" W. Figure 65 shows the site location on Google Earth.


**Figure 65. Site Location on Google Earth (Briaud et al., 2010).**

#### *4.2.1.3. Site Visit Objective*

The purpose of the site visit was to observe the site conditions, take samples for laboratory testing (EFA, grain size distribution, and direct shear test) and run in-situ tests (shear vane test, pocket penetrometer test [PPT], pocket erodometer test [PET]). The size of the riprap and the river flow velocity were also measured during the visit.

#### *4.2.1.4. Previous Information about the Site*

The riprap failure occurred in 2010. Now, the northern and southern banks of the main channel downstream of the bridge are experiencing problems due to meandering of the river. The pier in the center of the river shows large accumulations of debris lodged at its upstream nose. Figure 66 and Figure 67 show the riprap failure that was investigated.


**Figure 66. Site Picture (Briaud et al., 2010).**


**Figure 67. Riprap-Covered Slope Failure (Briaud et al., 2010).**

#### *4.2.1.5. Site Observations*

Figure 68 and Figure 69 show a global picture of the site taken on the day of the site visit. Two testing locations were chosen near the top of the slope and slightly downstream of the pier. It was also noticed that a lot of debris had accumulated around the center pier, thus accelerating the flow next to the slope of the main channel. The slope where the riprap was placed had a nonuniform cross section that was steeper at the bottom of the slope and flatter at the top, which seemed to indicate scour at the bottom of the slope. The slope studied was the steeper slope, which had an angle of 34 degrees from horizontal.


**Figure 68. Riprap-Covered Slope with Two Angles.**

**Figure 69. Riprap-Covered Slope between Main Channel and Flood Plain.**

#### *4.2.1.6. In-Situ Tests*

##### 4.2.1.6.1. Shear Vane Test

Two locations were chosen to run a vane shear test; all three vanes were used: mini-vane, medium vane, and large vane. Then another vane test was performed 1 m away from the first one using only the medium vane. Figure 70 shows one of the trials.

Table 133 shows the data obtained from the tests. Averaging all the $S_u$ data obtained, it can be concluded that the average undrained shear strength of the soil measured by the shear vane test was 77 kPa.

**Figure 70. Shear Vane Test.**

**Table 13. Shear Vane Test Result.**

| Shear Vane Test | | | | | |
|---|---|---|---|---|---|
| Su (kPa) | BH | Trial 1 | Trial 2 | Trial 3 | Average |
| | BH1 | 76 | 65 | 74 | 71.67 |
| | BH2 | 84 | 81 | | 82.5 |
| | | | | | **77.1** |

### 4.2.1.6.2. Pocket Penetrometer Test

The PPT is another way to obtain the undrained shear strength in-situ value. It was performed three times (Figure 71). Table 14 shows the data obtained during the test. The average value of these trials was 55 kPa, calculated as 0.3 times the pocket penetrometer reading; this is 28 percent lower than the value obtained by the shear vane test.

**Figure 71. Pocket Penetrometer Test.**

**Table 14. Pocket Penetrometer Test Results.**

| Pocket Penetrometer Test (PPT) | | | | |
|---|---|---|---|---|
| Trial # | Trial 1 | Trial 2 | Trial 3 | Average |
| Su (kPa) | 64.6 | 43.1 | 57.5 | 55.1 |

### 4.2.1.6.3. Pocket Erodometer Test

The PET is a quick in-situ test that helps to quantify the erodibility of the soil being tested by repeated jetting and measurement of the depth of the hole generated (Briaud et al., 2012). Two tests were performed with the PET (Table 15). Figure 72 shows the penetration hole after the PET. A penetration depth of 24.5 mm indicated that the soil is highly erodible.

**Table 15. Pocket Erodometer Test Results.**

| Pocket Erodometer Test (PET) | | | |
|---|---|---|---|
| Trial # | Trial 1 | Trial 2 | Average |
| Penetration (mm) | 25 | 24 | 24.5 |

**Figure 72. Pocket Erodometer Test.**

### 4.2.1.6.4. Other Site Properties

Table 16 shows some properties also estimated at the site, including the riprap $d_{50}$ (Figure 73).

**Table 16. Site Properties.**

| | |
|---|---|
| Flow Velocity (m/s) | 2.67 |
| Slope Angle (°) | 35 |
| Slope | 1.37:1 |
| Riprap Size $D_{50}$ (m) | 0.45 |

**Figure 73. Riprap Size Measurement.**

Figure 74 can be used to investigate what velocity is required to start eroding a geomaterial given its $d_{50}$—in other words, finding the critical velocity given the $d_{50}$. Using Table 16 and Figure 74, it can be seen that the 0.45 m $d_{50}$ leads to a critical velocity of 5 m/s. This value is very high for river flows, which means that the riprap washing away is not the issue in this case, unless the critical velocity tends to decrease when the riprap is placed on a slope.

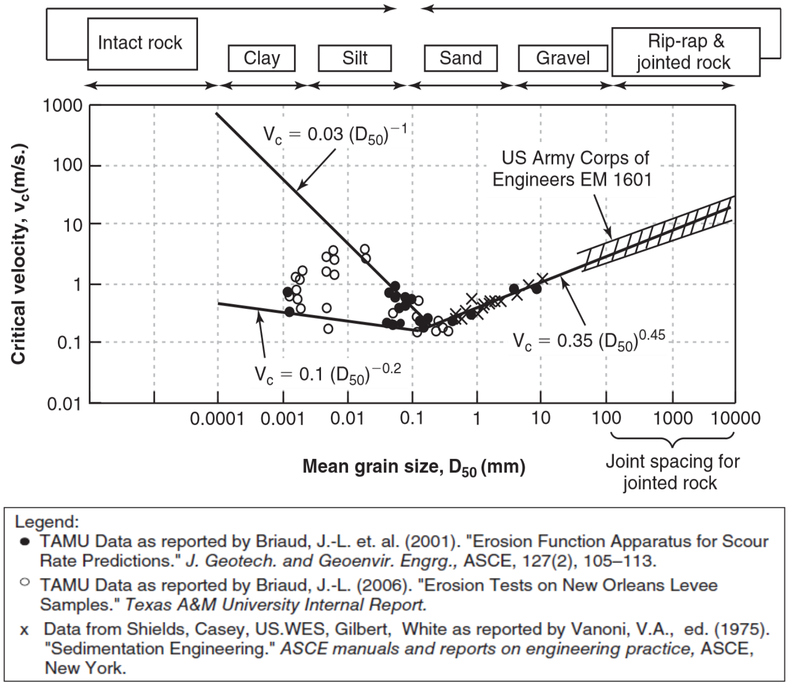



**Figure 74. Required $d_{50}$ in Function of the Critical Velocity (Briaud, 2008).**

### *4.2.1.7. Laboratory Test*

Four Shelby tubes taken from the site were transferred to the Soil Erosion Laboratory at Texas A&M University in order to run laboratory testing, including moisture content, grain size distribution, direct shear tests, and EFA tests, on the samples. All tests were performed following ASTM standards.

#### 4.2.1.7.1. Moisture Content and Unit Weight

Table 17 shows values obtained for moisture content. The total unit weight is 16.4 $kN/m^3$.

**Table 17. Moisture Content.**

| Sample | Moisture Content (%) |
|---|---|
| 1 | 10.6 |
| 2 | 9.9 |
| 3 | 10.5 |
| 4 | 11.0 |
| Average | 10.5 |

#### 4.2.1.7.2. Grain Size Distribution

Figure 75 shows the grain size distribution of the soil sample taken, and Table 18 shows the index grain size parameters, such as $d_{10}$, $d_{30}$, $d_{50}$, and $d_{60}$, along with the coefficients of uniformity and curvature. The soil can be classified as a fine sand with 4 percent fine particles.

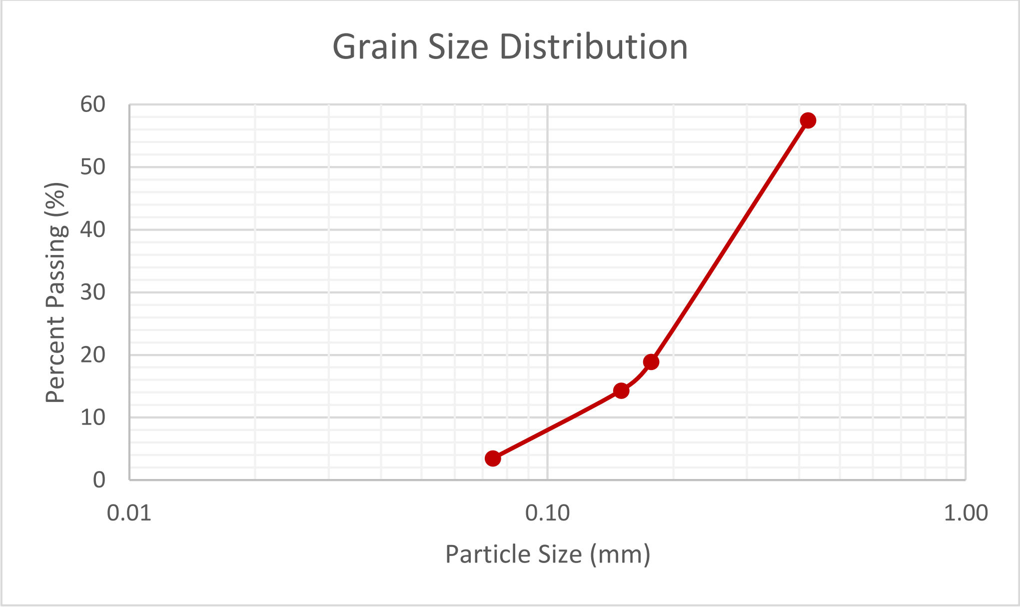



**Figure 75. Grain Size Distribution.**

**Table 18. Index Grain Size Parameters.**

| Parameter | Value |
|---|---|
| D10 (mm) | 0.12 |
| D30 (mm) | 0.22 |
| D50 (mm) | 0.35 |
| D60 (mm) | 0.45 |
| Coefficient of Uniformity | 3.75 |
| Coefficient of Curvature | 0.90 |

### 4.2.1.7.3. Direct Shear Test

A direct shear test was performed on a sample using three loads of 32.5, 63.5, and 94.5 kPa. The rate of displacement was 0.005 mm/s. Figure 76 shows the plots of shear force versus time, and Figure 77 shows the maximum shear stress as a function of normal stress. Using this plot and the Mohr Coulomb equation, $\tau = \sigma' * tan(\phi) + c$, the shear strength parameters can be obtained: the friction angle equals 32 degrees, and the cohesion equals 5 kPa (Table 19). The cohesion may be due to the water tension since a true cohesion in this fine sand is unlikely.

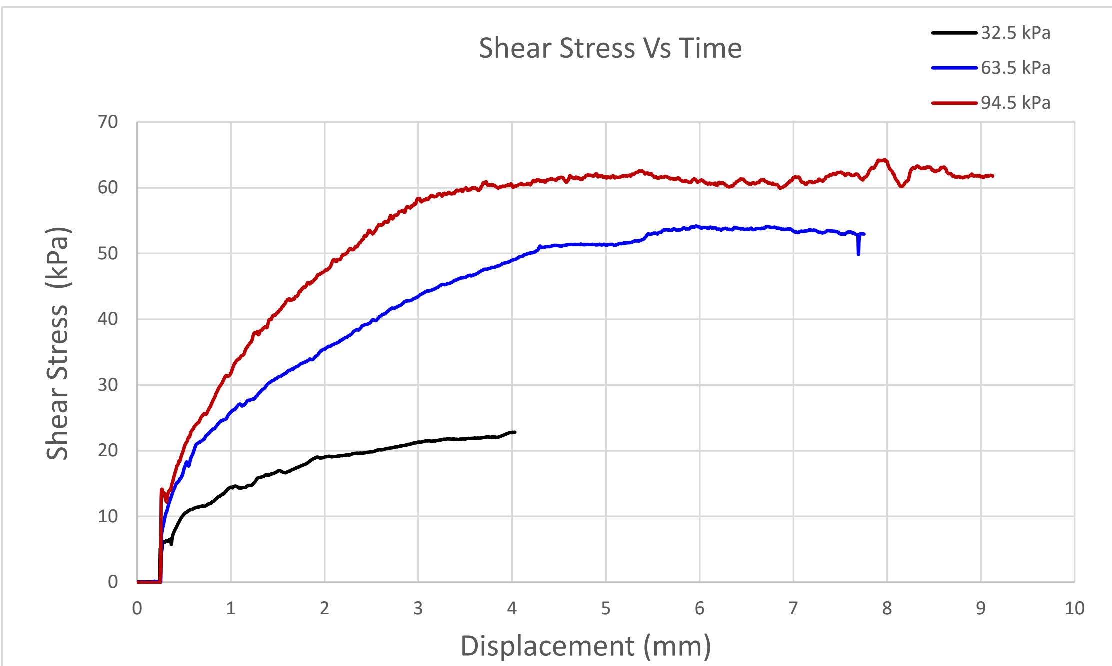



**Figure 76. Direct Shear Test Shear Stress vs. Displacement.**

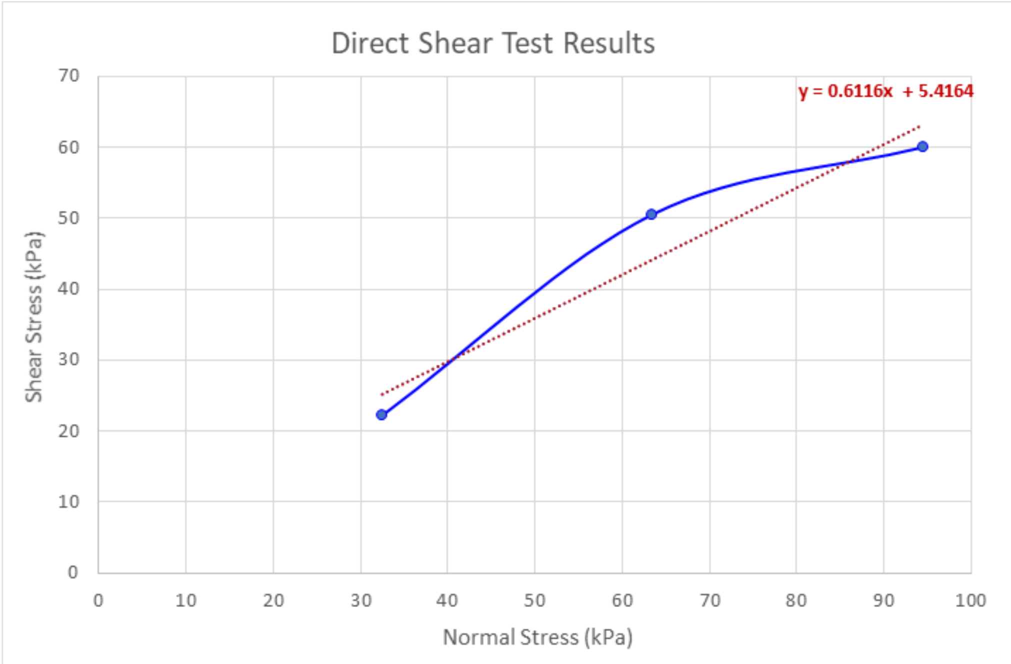



**Figure 77. Direct Shear Test Result.**

**Table 19. Shear Strength Parameters.**

| Shear Strength Parameter | |
|---|---|
| Friction Angle (°) | 32 |
| Cohesion (kPa) | 5.4 |

### 4.2.1.7.4. EFA Testing

In order to assess the erodibility of the soil, a sample was tested in the EFA (Briaud, 2013) (Figure 78 and Figure 79). The results obtained are shown in Figure 80 and Figure 81 and plotted on the Briaud erosion charts in Figure 82 and Figure 83. The soil is in the high to medium erodibility category, which is consistent with the observation at the site that abutment scour took place at the bottom of the riprap-covered slope.

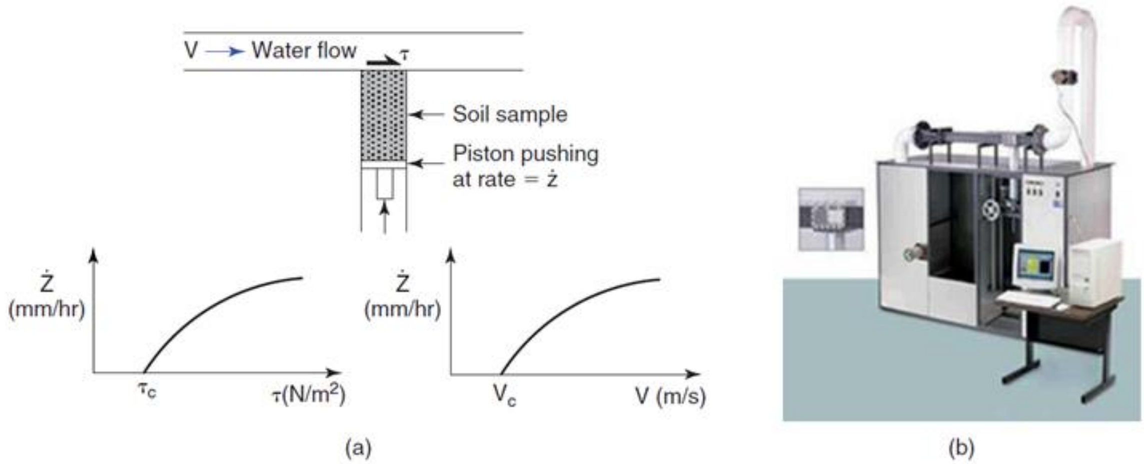



**Figure 78. Erosion Function Apparatus (Briaud, 2013).**

**Figure 79. Sample during EFA Test.**

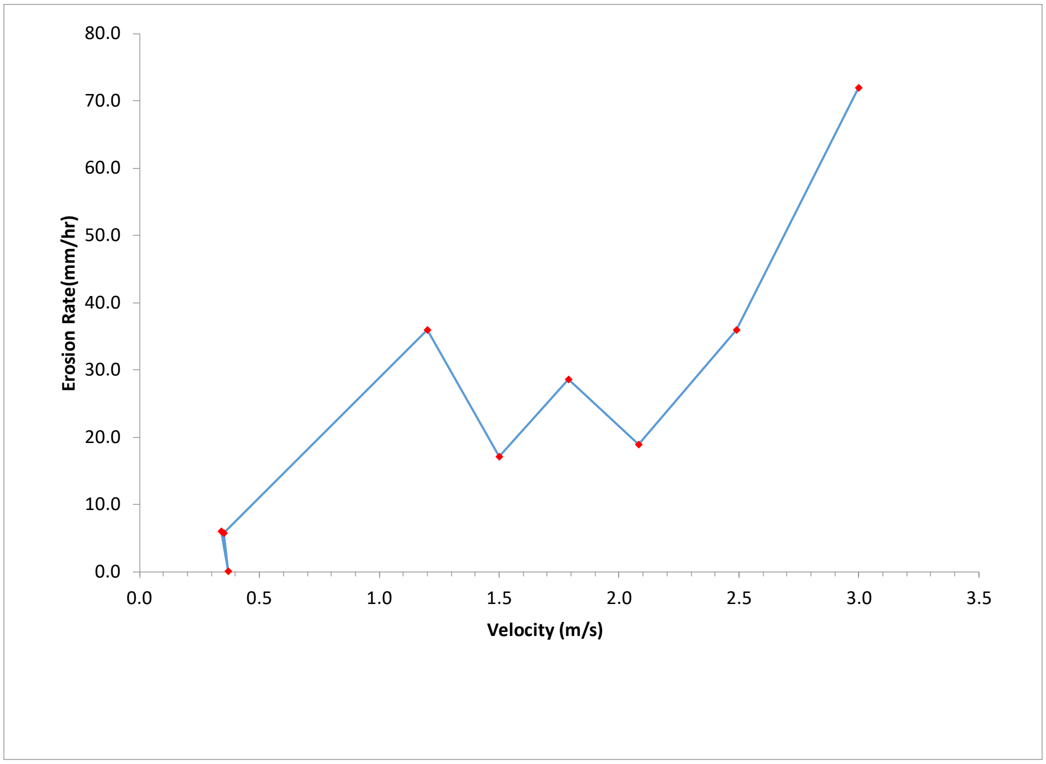



**Figure 80. Erosion Rate vs. Velocity in Natural Scale.**

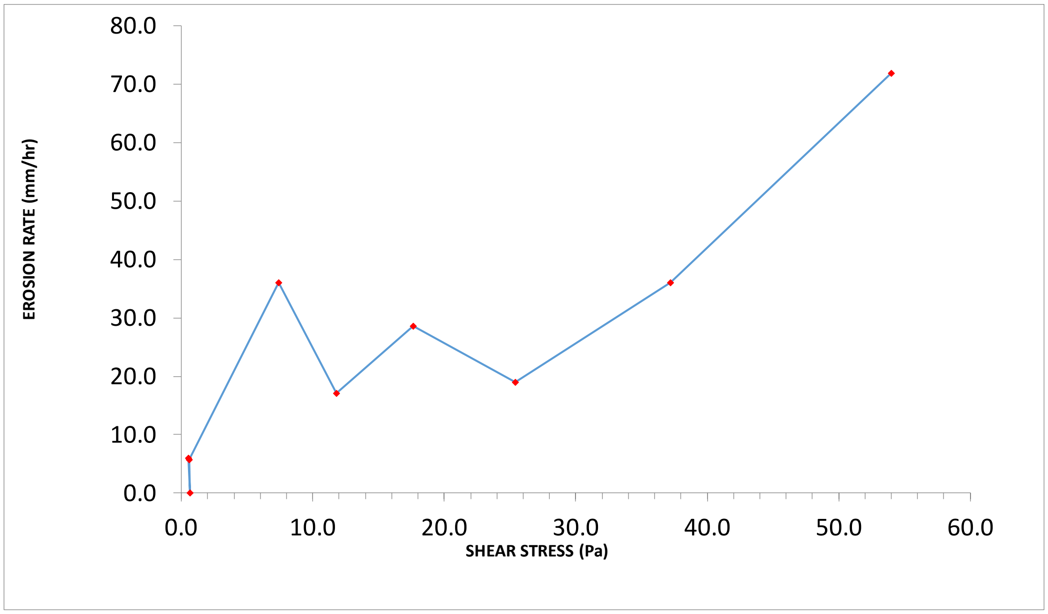



**Figure 81. Erosion Rate vs. Shear Stress in Natural Scale.**

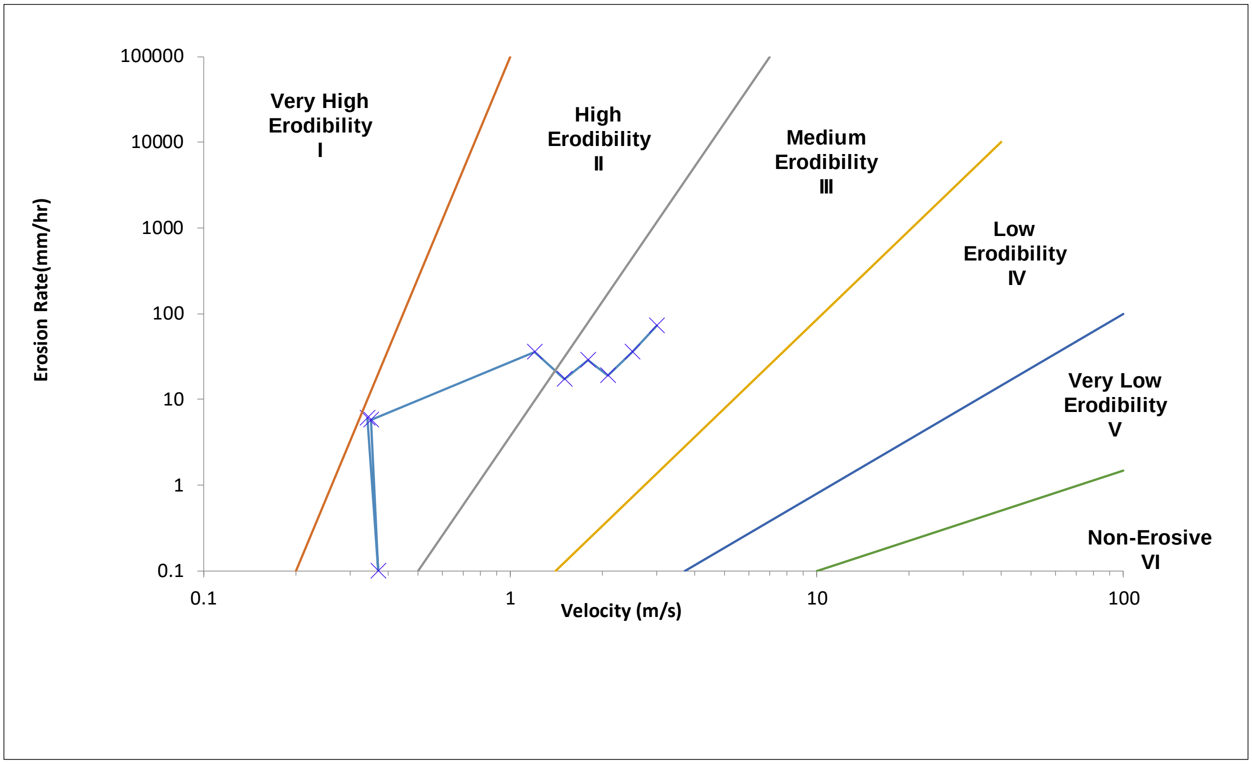



**Figure 82. Erosion Classification in Log-Log Scale (Erosion Rate vs. Velocity).**

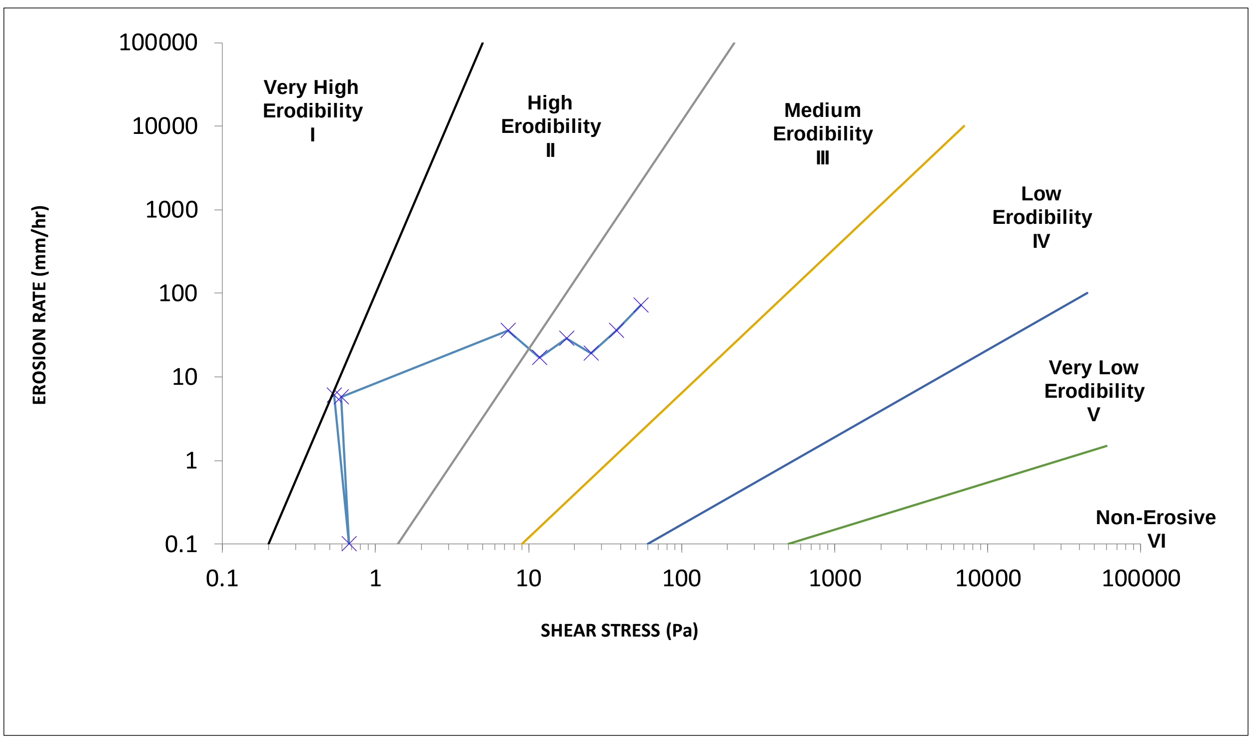



**Figure 83. Erosion Classification in Log-Log Scale (Erosion Rate vs. Shear Stress).**

#### *4.2.1.8 Conclusion*

Considering that the underlying soil is classified as being of high to medium erodibility, the most probable event causing the failure is that abutment scour started to develop at the toe of the slope, which led to a steeper slope angle in the bottom half of the slope. The slope angle may have reached a value that became too steep for the riprap to resist. From previous slope stability analyses, it has been found that the steepest slope to safely place riprap on would be a 2:1 slope. However, the slope angle in this case is 1.4:1, which is not recommended based on previous analysis.

It can be concluded that the failure in this case is due to abutment scour that modified the angle at the bottom of the slope and induced failure of the riprap-covered slope.

### 4.2.2. SH 74 over Skeleton Creek, Oklahoma

This case history was obtained from the Oklahoma DOT as part of the distributed survey. During a bridge construction on SH 74 over Skeleton Creek, Oklahoma (7.4 mi north of Crescent, Oklahoma), riprap was placed in order to line the termini of a USGS blue-lined unnamed channel running south into Skeleton Creek. Design plans show riprap thickness to be 0.45 m, with a $d_{50}$ equal to 12 inches. The riprap was to be installed on top of a 6-inch granular filter. Construction was still underway in May 2015 when heavy rainfall occurred, and the riprap washed away. A site investigation took place after this failure, and it was concluded that the slope was too steep for this high-velocity channel, and this factor caused failure of the riprap (Figure 84 and Figure 85). Therefore, the concerned entity decided to use an alternative solution instead of riprap and lined the embankment with sod and Armormax B2 erosion Control Mat (Figure 86). Monitoring of this solution was regular, and it displayed great performance. In 2019, an extreme flood event took place in Oklahoma, and the countermeasure held well during this event, which corresponded approximately to a 100-year event.

Given that the $d_{50}$ of the riprap was equal to 12 inches, the critical velocity would be around 4.5 m/s, according to Figure 74 this factor should have been sufficient. However, it may be that the critical velocity of the riprap on that steep slope was not as high as given by Figure 74 for a flat surface. This result indicates that a relationship might exist between the slope angle and the critical velocity of riprap stones for a given $d_{50}$. This case history appears to not be a slope stability failure but rather that the riprap on the slope was displaced by high flow velocities.

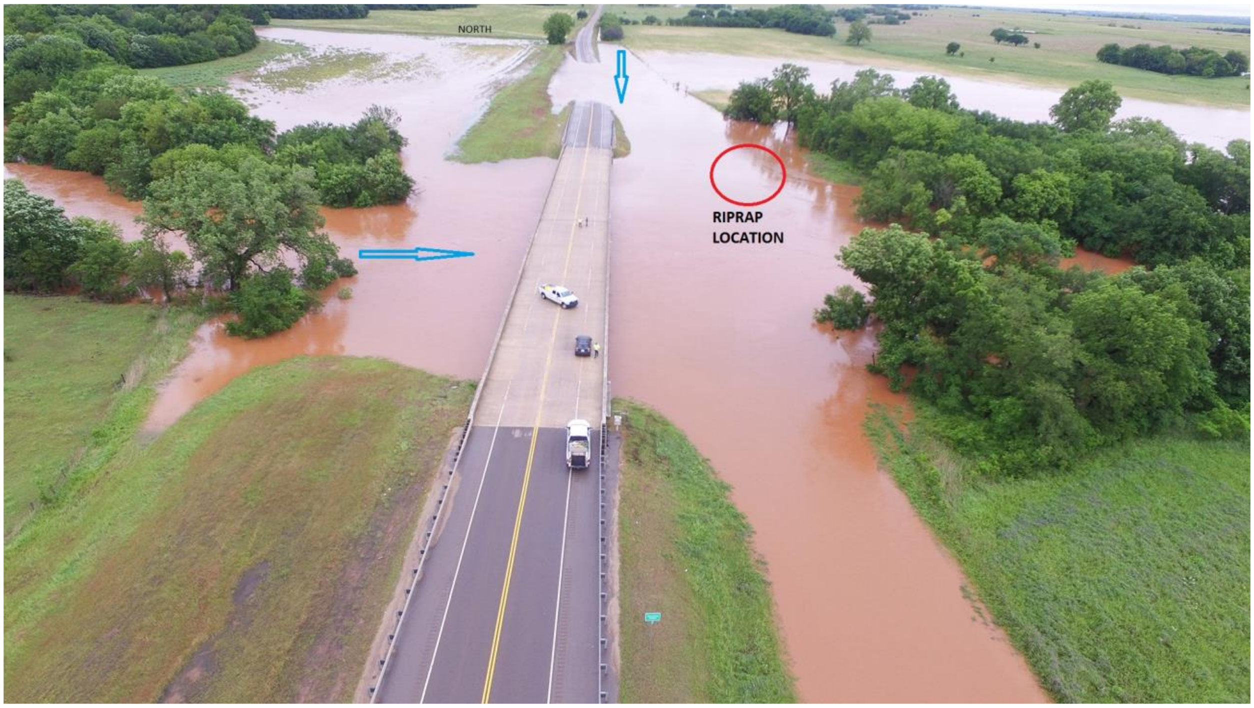



**Figure 84. Flood during 2019 (Courtesy of Oklahoma DOT).**

**Figure 85. Riprap-Covered Slope Failure (Courtesy of Oklahoma DOT).**


**Figure 86. Armormax B2 Erosion Control Mat (Courtesy of Oklahoma DOT).**

### 4.2.3. I-55 over Hickahala, Mississippi

This case history was obtained from the Mississippi DOT and regards a bridge on I-55 over Hickahala Creek. The soil at this site was classified as a hard clay according to the geotechnical report. Riprap was installed on a 2:1 slope. In 2014, the toe of the slope was undermined, and riprap stones were displaced from areas of the bank slope (Figure 87). The toe of the slope was scoured away, which modified the geometry of the slope into a steeper angle that induced riprap movement. Toe stabilization is a common and efficient technique to avoid such failures.

**Figure 87. Riprap Slope Failure (Courtesy of Mississippi DOT).**

### 4.2.4. SR 487 over Tuscolameta Creek (North Canal), Mississippi

This case history was obtained from the Mississippi DOT. The bridge is located in Leake County on SR 487 over Tuscolameta Creek. The upper soil layer was classified as dense sand, according to the geotechnical report. Scour had been detected at the bridge, which was built in 1961, and a plan of action was developed. The countermeasure was to install an 18-inch thick riprap blanket on the north and south banks of the creek. The blanket extended from the top of the bank to the creek bed and uniformly surrounded the piles in the bents. A flood monitoring program was implemented; however no fixed monitoring device nor increased inspection frequency was planned. In time, the riprap washed away from the bank (Figure 88); no filter was used, and the bank slope was steep (1:1).

Failure of the riprap was observed because the riprap washed away. This failure was attributed to the steep 1:1 slope.

**Figure 88. Riprap Slope Failure (Courtesy of Mississippi DOT).**

### 4.2.5. Pinole Creek, California

This case history was obtained from the document *Rock Riprap Design for Protection of Stream Channels Near Highway Structures* (Blodget & McConaughy, 1986). This riprap was designed by the U.S. Army Corps of Engineers and was constructed in 1965. Movement of the riprap stones during high flow was the main reason behind the failure of the riprap in 1982 following a flood event (Figure 89). The stone movement occurred on the lower part of the bank slope, while riprap stones remained intact toward the upper side of the bank. Table 20 shows the available parameters involved in the riprap failure, which indicate that even if the lower support was eroded, the upper portion of the riprap did not erode because the side slope of the bank was smaller than the angle of repose. Nevertheless, going back to Figure 74, using the given $d_{50}$ = 0.6 ft = 180 mm, the critical velocity would be 3.6 m/s, but in Table 20 the mean flow velocity is shown to be 7.7 ft/s (2.3 m/s), and the riprap was placed on a 2:1 slope. This design is also a problematic issue.

It can be concluded that the main reason behind the failure of the riprap was undersizing of the riprap stones, which is mentioned in the literature as one of the common reasons behind riprap failure. The riprap stones themselves can be susceptible to erosion if not made of durable rock.


**Figure 89. Pinole Creek Riprap Failure (Blodget & McConaughy, 1986).**

**Table 20. Pinole Creek Riprap Failure Parameters.**

| Location | | Pinole Creek at Pinole, CA |
|---|---|---|
| Date | | 1/4/1982 |
| Hydraulic Properties | Discharge (ft3/s) | 2250 |
| | Water-Surface Slope | 0.0054 |
| | Manning's n | 0.03 |
| | Mean Velocity (ft/s) | 7.7 |
| | Maximum Depth (ft) | 7.7 |
| | Depth of Flow above toe (ft) | 7.7 |
| | Mean Depth (ft) | 4.9 |
| | Curvature Angle (°) | 66 |
| | Curvature Radius (ft) | 150 |
| | Flow Contraction Ratio | 0.87 |
| | Shear Stress (lb/ft$^2$) | 2.59 |
| | Froud Number F | 0.61 |
| Revetment Material | $D_{85}$ (ft) | 0.84 |
| | $D_{50}$ (ft) | 0.6 |
| | $D_{15}$ (ft) | 0.42 |
| | Ratio $D_{85}/D_{50}$ | 1.4 |
| | Specific Gravity | 2.85 |
| | Design Side Slope | 2:1 |
| D50 According to different guidelines | HEC-11 | 0.43 |
| | HEC-15 | 0.98/0.5 |
| | Cal- B & SP | 0.85 |
| | EM-1601 | 0.6 |
| Cause of Failure | | Riprap Movement |

### 4.2.6. Site E-10 over Sacramento River, California

This case history was obtained from *Rock Riprap Design for Protection of Stream Channels Near Highway Structures* (Blodget & McConaughy, 1986). This particular site is known for having a low flood plain, which makes it a good candidate for prolonged inundation. The entire riprap slope is subjected to the flow during extreme events. Riprap failure was reported in three locations at this site during 1983 (Figure 90). The unique flow contraction condition that was observed was in the vertical direction, with the channel bed rising rather than having contraction in the width of the channel. This condition was caused by a delta built up in the riverbed. The displacement of individual stones in different locations at different time intervals due to cyclic water flow led to the failure of the riprap. For the given $d_{50}$ = 0.51 ft = 153 mm, the critical velocity should be 3.3 m/s (Figure 74), but as seen in Table 21, the flow event velocity was 6.7 ft/s (2 m/s), and the riprap was placed on a 2:1 slope. Failure of the riprap was attributed to undersizing of the riprap stones and also brought into question the influence of the slope angle on the riprap critical velocity.

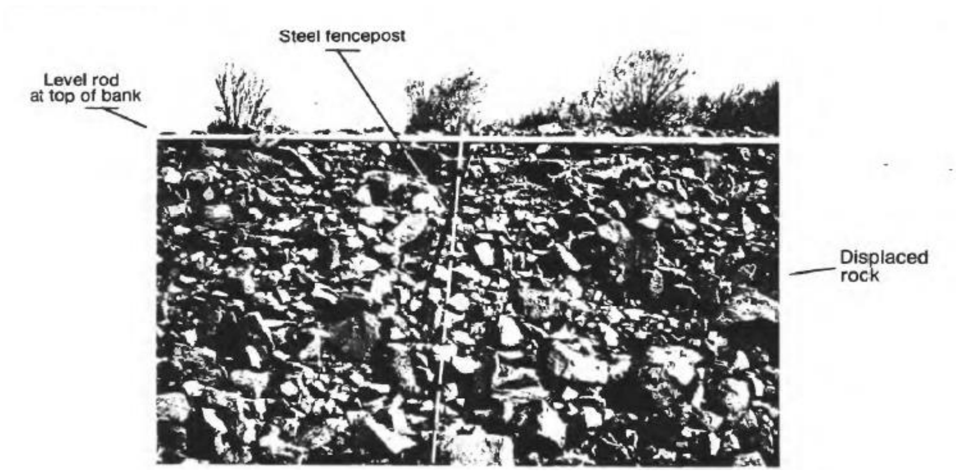



**Figure 90. Site E-10 over Sacramento River Riprap Failure (Blodget & McConaughy, 1986).**

**Table 21. Site E-10 over Sacramento River Riprap Failure Parameters.**

| | | |
|---|---|---|
| Location | | Sacramento River E-10, California |
| Date | | 1/27/1983 |
| Hydraulic Properties | Discharge (ft3/s) (Main Channel) | 98000 |
| | Water-Surface Slope | 0.000556 |
| | Manning's n | 0.033 |
| | Mean Velocity (ft/s) | 6.7 |
| | Maximum Depth (ft) | 34.5 |
| | Depth of Flow above toe (ft) | 13 |
| | Mean Depth (ft) | 20.2 |
| | Curvature Angle (°) | 11 |
| | Curvature Radius (ft) | 4280 |
| | Flow Contraction Ratio | 0.63 |
| | Shear Stress ($lb/ft^2$) | 0.9 |
| | Froud Number F | 0.26 |
| Revetment Material | $D_{85}$ (ft) | 0.66 |
| | $D_{50}$ (ft) | 0.51 |
| | $D_{15}$ (ft) | 0.31 |
| | Ratio $D_{85}/D_{50}$ | 1.29 |
| | Specific Gravity | 2.6 |
| | Design Side Slope | 2:1 |
| D50(ft) According to different guidelines | HEC-11 | 0.3 |
| | HEC-15 | - |
| | Cal- B & SP | 0.7 |
| | EM-1601 | 0.23 |
| Cause of Failure | | Riprap Movement |

### 4.2.7. Hoh River at Site 1 Forks, Washington

This case history was obtained from *Rock Riprap Design for Protection of Stream Channels Near Highway Structures* (Blodget & McConaughy, 1986). The design parameters used for this riprap installed in 1982 are not available. Riprap movement at two different locations was observed after several floods took place (Figure 91). Undermining by scour at the toe caused the riprap slope to fail in a modified slump behavior, which means the slope failure plane is within the riprap and does not involve the underlying soil. The two main causes behind this failure were riprap stone undersizing, which caused the stone movement, and placing the riprap on too steep a slope (1.2:1). These two issues are considered the typical reasons behind riprap failure. However, the ratio of $D_{85}$ to $d_{50}$ was 1.92, which is higher than the recommended value in all design procedures. Table 22 shows the available parameters involved in the riprap failure. Using the given $d_{50}$ = 1.3 ft = 390 mm and Figure 74, the critical velocity should be 5.1 m/s, which is highly unlikely in a river environment, but in Table 22 the mean flow velocity is shown to be 7.94 ft/s (2.4 m/s), and the riprap was placed on a 1.2:1 slope, which is a very steep slope. This highlights the possibility of the decrease of the critical velocity of the riprap with an increasing slope angle.

**Figure 91. Hoh River at Site 1 Forks Riprap Failure (Blodget & McConaughy, 1986).**

**Table 22. Hoh River at Site 1 Forks Riprap Failure Parameters.**

| Location | | Hoh River at Site 1, Near Forks, WA |
|---|---|---|
| Date | | 10/22/1982 |
| Hydraulic Properties | Discharge (ft3/s) (Main Channel) | 22000 |
| | Water-Surface Slope | 0.0014 |
| | Manning's n | 0.035 |
| | Mean Velocity (ft/s) | 7.94 |
| | Maximum Depth (ft) | 19.1 |
| | Depth of Flow above toe (ft) | 19.1 |
| | Mean Depth (ft) | 3.52 |
| | Curvature Angle (°) | 80.5 |
| | Curvature Radius (ft) | 991 |
| | Flow Contraction Ratio | 0.76 |
| | Shear Stress (lb/ft$^2$) | 1.67 |
| | Froud Number F | 0.76 |
| Revetment Material | $D_{85}$ (ft) | 2.5 |
| | $D_{50}$ (ft) | 1.3 |
| | $D_{15}$ (ft) | 0.58 |
| | Ratio $D_{85}/D_{50}$ | 1.92 |
| | Specific Gravity | 2.59 |
| | Design Side Slope | 1.2:1 |
| D50(ft) According to different guidelines | HEC-11 | 0.4 |
| | HEC-15 | - |
| | Cal- B & SP | 1.4 |
| | EM-1601 | 0.4 |
| Cause of Failure | | Translational Slide |

### 4.2.8. Cosumnes River at Site 3 near Dillard Road Bridge near Sloughhouse, California

This case history was obtained from *Rock Riprap Design for Protection of Stream Channels Near Highway Structures* (Blodget & McConaughy, 1986). The manual used for the design of this riprap in 1983 is unknown. Six months after installation, a modified slump failure was observed after flood events (Figure 92). Riprap stones were displaced downward. The reported cause of this failure is failure of the interface between the underlying soil and the riprap layer. It was not mentioned if a filter blanket was at the interface between the two layers. Excess of hydrostatic pressure in the base material was said to have probably occurred and triggered the failure mechanism. Table 23 shows the available parameters involved in the riprap failure.

The possibly low hydraulic conductivity of the riprap may have contributed to the failure. When designing riprap, the purpose is not to allow native soil particles to erode but to allow water to properly flow in the system. If water is not allowed to flow properly, water pressure will build up, weaken the soil, and lead to failure. It must also be mentioned that the slope where the riprap was placed was 1.8:1, which is considered a relatively steep slope for riprap placement.


**Figure 92. Cosumnes River at Site 3 Riprap Failure (Blodget & McConaughy, 1986).**

**Table 23. Cosumnes River at Site 3 Riprap Failure Parameters.**

| Location | | Cosumnes River at Site 3, near Slough house, CA |
|---|---|---|
| Date | | 3/13/1983 |
| Hydraulic Properties | Discharge (ft3/s) (Main Channel) | 26100 |
| | Water-Surface Slope | 0.0007 |
| | Manning's n | 0.03 |
| | Mean Velocity (ft/s) | 4.08 |
| | Maximum Depth (ft) | 31 |
| | Depth of Flow above toe (ft) | 10.2 |
| | Mean Depth (ft) | 18.6 |
| | Curvature Angle (°) | 99 |
| | Curvature Radius (ft) | 458 |
| | Flow Contraction Ratio | 1.25 |
| | Shear Stress (lb/ft$^2$) | 0.812 |
| | Froud Number F | 0.17 |
| Revetment Material | $D_{85}$ (ft) | 1 |
| | $D_{50}$ (ft) | 0.78 |
| | $D_{15}$ (ft) | 0.5 |
| | Ratio $D_{85}/D_{50}$ | 1.28 |
| | Specific Gravity | 2.92 |
| | Design Side Slope | 1.8:1 |
| D50(ft) According to different guidelines | HEC-11 | 0.2 |
| | HEC-15 | - |
| | Cal- B & SP | 0.3 |
| | EM-1601 | 0.195 |
| Cause of Failure | | Modified Slump |

### 4.2.9. Truckee River at Sparks, Nevada

This case history was obtained from *Rock Riprap Design for Protection of Stream Channels Near Highway Structures* (Blodget & McConaughy, 1986). The bridge location, the design manual used, and the site characteristic data are not available for this case. However, the site location was known for facing impinging flow. Failure of the riprap due to riprap movement was seen at a location where the channel has an 18-degree curve. It was reported that the failure of the riprap occurred after heavy flood events in 1983 (Figure 93). The riprap was installed on a 1.8:1 slope. Table 24 shows the available parameters involved in the riprap failure. Given the $d_{50}$ = 0.71 ft = 213 mm, the critical velocity should be 3.9 m/s (Figure 74), but in Table 24, the mean flow velocity is shown to be 5.2 ft/s (1.5 m/s). The slope angle of 1.8:1 slope and the presence of the bend angle combined with frequent flooding events are probably the main contributors to this failure.

**Table 24. Truckee River at Sparks Riprap Failure Parameters.**

| Location | | Truckee River at Sparks, NV |
|---|---|---|
| Date | | 3/13/1983 |
| Hydraulic Properties | Discharge (ft3/s) (Main Channel) | 7340 |
| | Water-Surface Slope | 0.003 |
| | Manning's n | 0.035 |
| | Mean Velocity (ft/s) | 5.19 |
| | Maximum Depth (ft) | 17.5 |
| | Depth of Flow above toe (ft) | 17.5 |
| | Mean Depth (ft) | 10.5 |
| | Curvature Angle (°) | 18 |
| | Curvature Radius (ft) | 646 |
| | Flow Contraction Ratio | 0.88 |
| | Shear Stress (lb/ft$^2$) | 3.27 |
| | Froud Number F | 0.28 |
| Revetment Material | $D_{85}$ (ft) | 1.14 |
| | $D_{50}$ (ft) | 0.71 |
| | $D_{15}$ (ft) | 0.46 |
| | Ratio $D_{85}/D_{50}$ | 1.61 |
| | Specific Gravity | 2.68 |
| | Design Side Slope | 1.8:1 |
| D50(ft) According to different guidelines | HEC-11 | 0.2 |
| | HEC-15 | - |
| | Cal- B & SP | 0.4 |
| | EM-1601 | 0.82 |
| Cause of Failure | | Riprap Movement |


**Figure 93. Truckee River at Sparks Riprap Failure (Blodget & McConaughy, 1986).**

#### 4.2.10. Willamette Highway over Salmon Creek, Oregon

This case history was obtained from *Riprap Design Criteria, Recommended Specifications, and Quality Control* (Lagasse et al., 2006). The location is Salmon Creek on the Willamette highway (Route 58), Lane County, Oregon. The riprap failure was attributed to impinging flow that caused undermining at the toe of the slope (Figure 94). The riprap was designed by the USACE Portland District following EM 1110 guidelines. One of the assumptions made during the design was that parallel flow conditions would be occurring. Post-failure analysis showed that the main causes of failure were a combination of the steepness of the slope, the riprap stones being rounded (which reduces the global frictional resistance), and no filter layer being present. Again, the common issue of the steepness of the slope was mentioned in this case history. Moreover, it was mentioned that the failure occurred during turbulent flow conditions, which are not typically taken into consideration in design. Finally, friction is a major contributor to riprap movement resistance, and using rounded stones lowers this resistance because rounded rocks have smaller global friction than angular rocks; indeed, rounded stones do not provide interlocking resistance during shearing.

View upstream of west bank just upstream of Route 58 bridge
(photographed May 1992)

**Figure 94. Willamette Highway over Salmon Creek Riprap Failure (Lagasse et al., 2006).**

### 4.2.11. Route 30 over Grizzly Creek, California

This case history was obtained from *Riprap Design Criteria, Recommended Specifications, and Quality Control* (Lagasse et al., 2006). Riprap designed according to Bank and Shore Protection (State of California Department of Public Work) was installed on the bank of Grizzly Creek at Route 30 in Lake County, California. Failure of this riprap occurred due to many reasons (Figure 95). Under sizing of the riprap was the main issue at the design stage. Other reasons included a small permittivity filter fabric, a steep slope angle (2:1), and riprap stones not properly installed, which led to poor interlocking between stones. Some of the lessons learned from this failure are that installation of the riprap must promote interlocking and that proper selection of the filter fabric permittivity is important.

View is looking upstream. Woven-tape geotextile (slit-film) marked with arrows is inappropriate as RSP-fabric on banks (photographed May 1995)

**Figure 95. Route 30 over Grizzly Creek Riprap Failure (Lagasse et al., 2006).**

### 4.2.12. I-90 Bridge over Schoharie Creek, New York

This case history was obtained from *Riprap Design Criteria, Recommended Specifications, and Quality Control* (Lagasse et al., 2006). A major bridge failure occurred in 1987 on the I-90 bridge over Schoharie Creek near Albany, New York, costing 10 lives. The reason of failure was scour at the bridge piers due to riprap failure (Figure 96). The peak flow was 1,838 $m^3/s$, with a 70- to 100-year return period. The footings were set 1.5 m into the stream bed in glacial till, which was assumed during the design stage to be non-erodible. However, flume studies of samples of the stratified drift showed that some material does erode at a velocity of 1.5 m/s. At a velocity of 2.4 m/s, the erosion rates were high, which called for the need of riprap as a countermeasure. From 1953 to 1987, high-velocity flows were able to displace some of the riprap, which led to rapid erosion of the native soil in 1987. During an inspection in 1979, it was documented that a good portion of the riprap around the pier was missing; however, during another inspection in 1985, absence of the riprap was not detected and led to continuous erosion and failure of the piers. It can be concluded from this case history that another critical variable in the performance of riprap is proper inspection; inspectors must be trained to recognize when riprap has washed away, especially underwater, because this is a critical stage of the failure mobilization, not only in regard to the failure of the riprap but also in regard to the failure of the structure, which could lead to fatalities.

Pier 2 in the foreground with Pier 3 in the background.

**Figure 96. I-90 Bridge over Schoharie Creek (Lagasse et al., 2006).**

### 4.2.13. FM 1155 Little Sandy Creek, Washington County, Texas

This case history was obtained from TxDOT. It occurred in Washington County beside the bridge on FM 1155 over Little Sandy Creek. Riprap was installed on a 3:1 slope (Figure 97) as a scour countermeasure, but then it washed away along the stream. A geotextile blanket was present (Figure 98). One possible reason failure occurred is that the riprap stones used on the site were smaller than the size indicated in the design. When it was replaced by grouted riprap, it worked well. Riprap stone sizing is a crucial parameter for the resistance of the riprap against hydraulic stresses caused by the flow.

**Figure 97. FM 1155 Little Sandy Creek Riprap Failure (Courtesy of TxDOT).**

**Figure 98. Geotextile underneath the Riprap (Courtesy of TxDOT).**

### 4.2.14. SH 78 over Price Creek, Dallas District, Texas

This case history was obtained from TxDOT. It occurred in Dallas District and deals with the bridge built in 1972 on SH 78 over Price Creek. The riprap was placed on a 2:1 slope (Figure 99). Failure of riprap occurred when the limestone rock riprap moved downhill (Figure 100) and ended up resting on the interior bents, becoming piled higher around each column as time went on. The recommended action was to reconstruct the north and south banks with large (300 to 500 lb) concrete rubble, or with gabions. The last time it was inspected was September 12, 2019.

**Figure 99. SH 78 over Price Creek Riprap-Covered Slope (Courtesy of TxDOT).**

**Figure 100. SH 78 over Price Creek Riprap Slope Failure (Courtesy of TxDOT).**

### 4.2.15. FM 982 over Tickey Creek, Dallas District, Texas

This case history was obtained from TxDOT. It occurred in the Dallas District and relates to the bridge built in 1974 on FM 982 over Tickey Creek. The riprap was placed on a 2:1 slope. Failure of the riprap occurred when the limestone rock riprap washed away from the western side of the northern bank, resulting in exposed earth and drilled shaft (Figure 101 and Figure 102). The repair consisted of backfilling with cement-stabilized sand under the abutment cap, then armoring the channel bank with a combination of rocks and gabions. The last time it was inspected was on August 1, 2019. This failure brings up again the issue of riprap resistance to movement when placed on slopes.

**Figure 101. FM 982 over Tickey Creek Riprap Slope Failure (Courtesy of TxDOT).**

**Figure 102. FM 982 over Tickey Creek Riprap Slope Failure (Courtesy of TxDOT).**

### 4.2.16 US 59 over Morgan Creek, Lufkin District, Texas

This case history was obtained from TxDOT. It occurred in the Lufkin District and relates to the bridge built on US 59 over Morgan Creek. Slope stability failure occurred in early 2019. The repair operation consisted of reworking the slope and then installing stone riprap on the repaired slope. The underlying soil was erodible, and the slope was undermined later that year due to erosion, which led to a second failure of the stone riprap (Figure 103).

**Figure 103. US 59 over Morgan Creek Riprap-Covered Slope Failure (Courtesy of TxDOT).**

### 4.2.17. IH 35 NBFR over Cobb Creek, Waco District, Texas

This case history was obtained from TxDOT. It occurred in the Waco District and relates to the bridge on IH 35 NBFR over Cobb Creek (Figure 104). The riprap layer was installed on a 2:1 slope. The stone riprap slope failed, and then the contractor reshaped the slope and installed stone riprap again. The exact cause of failure is unknown for this case.

**Figure 104. IH 35 NBFR over Cobb Creek Riprap Slope Failure (Courtesy of TxDOT).**

### 4.2.18. FM 218 over Cowhouse Creek, Waco District, Texas

This case history was obtained from TxDOT. It is located in the Waco District and relates to the bridge built on FM 218 over Cowhouse Creek. A concrete cover was installed on the 3:1 slope. This countermeasure was undermined, and the concrete riprap failed in 2007. It was replaced by stone riprap in the form of crushed rock up to 6 inches in size that was used to fill all the voids caused by scour (Figure 105). This stone riprap also failed, and it was repaired by re-installing stone riprap. The exact failure mechanism is not known, but a 3:1 slope does not typically lead to slope stability problems; however, undersized riprap stones might be a potential issue triggering the failure of the riprap when subject to water flow.

**Figure 105. FM 218 over Cowhouse Creek Riprap-Covered Slope (Courtesy of TxDOT).**

### 4.2.19. FM 667 at Cobb Hollow Creek, Wichita Falls District, Texas

This case history was obtained from TxDOT. It occurred in the Wichita Falls District and relates to the bridge built on FM 667 at Cobb Hollow Creek. A layer of 18-inch stone riprap was installed on top of a filter fabric on a 2:1 slope. The failure consisted of the riprap stones sliding down into the channel (Figure 106). It was observed that the underlying soil was a highly erodible geomaterial and that high-velocity flow was taking place when the riprap failed. It may be that the filter fabric provided a sliding plane that contributed to the failure of the riprap-covered slope.


**Figure 106. FM 667 Cobb Hollow Riprap Slope Failure (Courtesy of TxDOT).**

### 4.2.20. US 84 over Navasota River E Relief, Waco District, Texas

This case history was obtained from TxDOT. It occurred in the Waco District and relates to the bridge built on US 84 at Navasota River. Concrete riprap was installed on a 2:1 slope. The concrete riprap failed and was replaced by stone riprap (Figure 107). However, the riprap stones were too small and did not stay in place.

**Figure 107. US 84 over Navasota River E Relief Riprap-Covered Slope (Courtesy of TxDOT).**

### 4.2.21. Skookumchuck River, Lewis County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 (1970) and installed in 1971. Failure occurred in 1989 due to debris accumulation; it was repaired using a 34-inch-thick layer of Class II riprap with a gravel filter. It was evaluated three years later, and it worked well.

### 4.2.22. Puyallup River, Pierce County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 (pre-1948) and installed in 1948. Failure occurred due to scour at the toe and was repaired with a 24-inch-thick layer of riprap in 1970. It was last evaluated in 1992, and it was still working well.

### 4.2.23. Cedar River, Orchard Grove, King County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 (1970) and installed in 1975 on a 1.3:1 slope. The riprap failure occurred due to toe scour. It was repaired by flattening the slope to a 2.67:1 angle using a 30-inch-thick layer of

Class III riprap. The repair took place in 1977; it was last inspected in 1992 and was working well.

### 4.2.24. Cedar River, Rainbow Bend, King County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 (1970) and installed in 1975 on a 1.3:1 slope. The riprap failure occurred due to toe scour. It was repaired by flattening the slope to a 2.67:1 slope angle and using a 30-inch-thick layer of Class III riprap. The repair took place in 1977; it was last inspected in 1992 and was working well.

### 4.2.25. South Fork Skagit River, Areas A and B, Skagit County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 (1970). Riprap failure occurred due to an inadequate filter leading to the underlying soil eroding through the large riprap stones. It was repaired by flattening the slope to a 3.3:1 angle and using a 48-inch-thick layer of Class II riprap. The repair took place in 1991; it was last inspected in 1992 and was working well.

### 4.2.26. North Fork Skagit River, Area C, Skagit County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 manual (1970). The riprap failure occurred due to bed and toe scour. It was repaired using a 60-inch-thick layer of Class V riprap. The repair took place in 1991; it was last inspected in 1992 and was working well.

### 4.2.27. North Fork Skagit River, Areas D and E, Skagit County, Washington

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the CORPS Seattle EM-1110 manual (1970). The riprap failure occurred due to toe scour. It was repaired by flattening the slope to a 3.3:1 angle and using a 24-inch-thick layer of Class II riprap. The repair took place in 1991, was last inspected in 1992, and was working well.

### 4.2.28. South Platte River, 88th Ave. Drop Structure, Adams County, Colorado

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the Denver *UDFCD Vol. 2 Drainage Manual* (1984). The riprap failed due to scour development. It was repaired, last inspected in 1992, and was working well.

### 4.2.29. Sanderson Gulch Tributary of S. Platte, Denver County, Colorado

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The riprap was designed according to the Denver *UDFCD Vol. 2 Drainage Manual* (pre-1975) and installed in 1970. The riprap failed in 1975 due to toe scour. The riprap stones were undersized, and no filter layer existed, which might have triggered the failure. It was repaired by concreting existing Type L riprap, last inspected in 1992, and was working well.

### 4.2.30. Van Duzen River by Grizzly Creek State Campground, California

This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The site is located in the Van Duzen River at Grizzly Creek State Campground, Route 36, mile 16.9, California. The riprap was designed according to the *California Bank & Shore Manual* (1960) and installed in 1968. Failure of the riprap occurred due to impinging flow and to riprap stones that were too small, coupled with the absence of a filter layer. It was repaired in 1970 and was still working well when it was inspected in 1996.

### 4.2.31. US 101 over South Fork Eel River, California

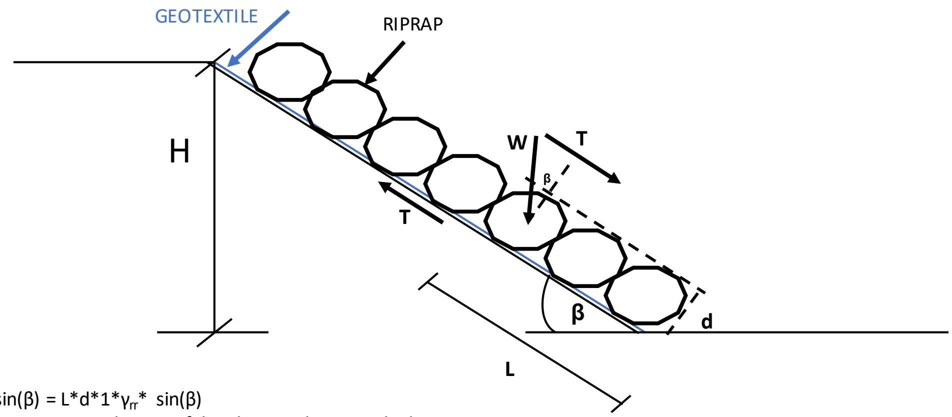
This case history was obtained from *California Bank and Shore Rock Slope Protection Design* (Racin et al., 2000). The site is located in Humboldt County at South Fork Eel River Crossing on US 101, mile 21.7-22, California. The riprap was designed according to *ASCE Manual 54*, a precursor of *HEC-11*, and according to *California Bank & Shore*. The riprap failure occurred in 1964 due to floods in the area; no filter was installed below the riprap. The riprap layer was thin in the upper and lower area; internal erosion occurred and caused failure of the riprap. It was repaired in 1965 and was still working fine when last inspected in 1996.

### 4.2.32. Geotextile Failure in Tension and Potential Sliding Plane

No case histories were reported on geotextile failure in tension and potential sliding plane; however these two scenarios are theoretically possible and should be taken into consideration.

First, when the geotextile is loaded with riprap, tension forces will develop in the geotextile, especially when the geotextile is anchored. Figure 28 shows how these tension forces are generated and how to estimate their magnitude.

**Tension in Geotextile with Riprap**




$T = W*\sin(\beta) = L*d*1*\gamma_{rr}* \sin(\beta)$

So T is maximum at the top of the slope and zero at the bottom

$T_{max} = L*d*\gamma_{rr}* \sin(\beta) = \frac{H}{\sin(\beta)}*d*\gamma_{rr}* \sin(\beta) = H*d*\gamma_{rr}$

Ex: H=7m, d=0.5m, $\gamma_{rr}$= 25 kN/$m^3$ → $T_{max}$= 7 kN/m

This is very high for a geotextile and likely to cause tension failure.

NOTE: This derivation assumes that no force is transferred between the geotextile and he soil below. This would be the case if the slope fails.

**Figure 108 Geotextile Tension Failure Derivation**

Second, the riprap stones can slide along the geotextile or the geotextile can slide over the the soil of the embankment fill ; this was also mentioned by some survey responders as one of the drawbacks of using geosynthetic filters instead of granular filters. There are two interfaces: the riprap to geotextile interface and the geotextile to embankment soil interface. Both interface friction forces would have to be checked to ensure that a proper factor of safety is satisfied. One problem is that the friction coefficient for both interfaces is not well known and tests have to be conducted to ensure reasonable values. Figure 109 shows a possible solution to minimize such a failure mechanism.

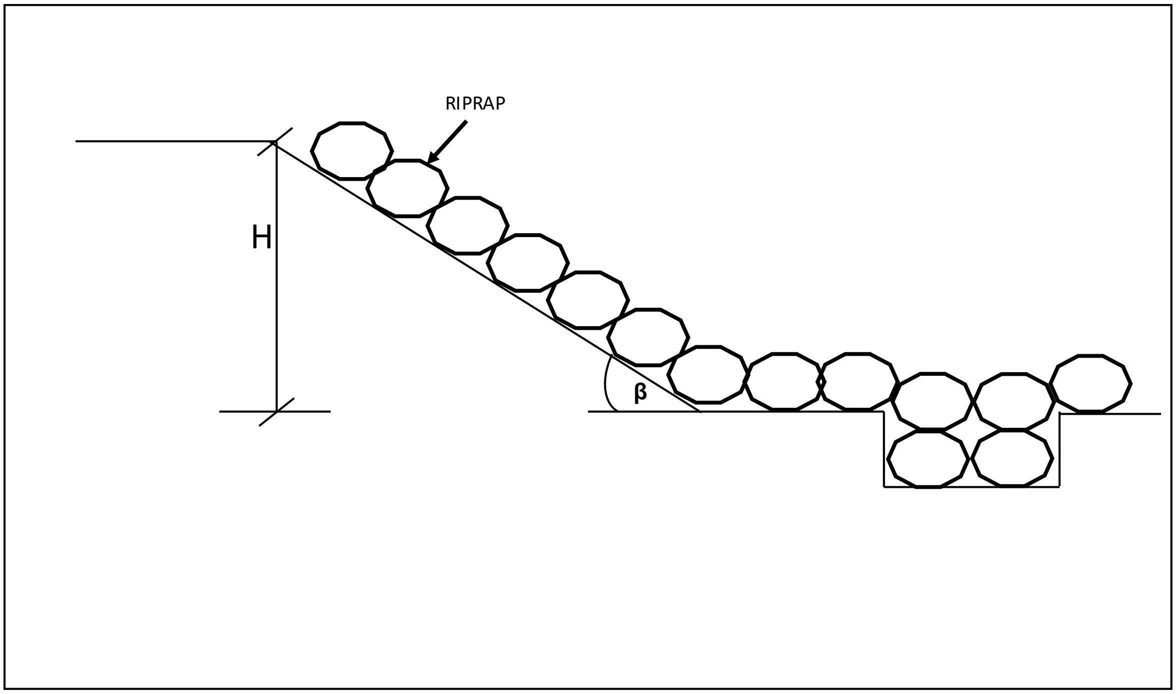



**Figure 109 Riprap Sliding Stability Countermeasure**

## 4.3. CONCLUSIONS

Appendix A presents a summary table showing all the riprap failure case histories collected for this study. Some of the cases listed in the table are not described in the text because they were not directly relevant to the study. The following is a list of causes of riprap failures, with recommendations given to avoid such failures.

- **The size of the riprap stones is too small**. This common reason for riprap failure occurred 48.4 percent of the time for the failure cases reviewed. The solution is to increase the size of the riprap stones according to the design flow velocity. However, the size was found to depend on the slope angle, and a new chart is needed to select riprap size on slopes for a given velocity.
- **The toe of the riprap-covered slope scours and triggers riprap movement**. This occurs quite often (38.7 percent of the cases reviewed). It goes back to the failure mechanism associated with too steep a slope, but in this case the steep slope is created by scour at the bottom of the slope. This problem can be mitigated by reinforcing the toe of the riprap-covered slope. One way is to extend the riprap far beyond the toe of the slope so that if scour occurs the riprap falls into the scour hole and slows down the process.
- **The riprap-covered slope is too steep**. This issue is also a major cause of failure that occurred in 35.5 percent of the cases reviewed. In these cases, the stability of the riprap itself on a steep slope was weakened by the steep angle. Lowering the slope to a 2:1 slope or flatter is recommended.

- **The natural soil erodes through the riprap and the riprap sinks**. This situation is not uncommon (16.1 percent of the case reviewed) but is often related to the absence of a filter between the natural soil and the riprap layer. It can be mitigated by using a proper filter material between the riprap and the underlying soil to stop seep-through erosion.
- **Failures are occurring during extreme events**. This event occurred in 12.9 percent of the cases reviewed. The design must be considered at the flood stage.
- **The soil slope fails because of water pressure buildup**. This situation occurred in 6.5 percent of the case reviewed. This type of failure is often generated by poor drainage in the filter, which leads to buildup of water pressure in the soil below. Mitigation requires selecting the hydraulic conductivity of the filter material to allow proper drainage of water out of the slope.
- **Poor installation and shape of the riprap**. This factor occurred in 6.5 percent of the cases reviewed. Proper installation will lead to better interlocking properties between riprap stones and increase stability. Although dumping riprap is common, the interlocking of stones should be evaluated after placement. The use of angular stones is recommended instead of rounded stones.
- **Absence of frequent and proper monitoring/inspection**. This cause of failure occurred in 6.5 percent of the cases reviewed. Early detection of instabilities in the system and taking proper corrective measures will minimize the impact of the problems.
- **The natural soil is too weak to handle the riprap loading**. This factor occurred in 3.2 percent of the cases reviewed. It can lead to a soil slope failure, but it is a rare failure mechanism for riprap-covered slopes.
- Tension failure of the geotextile should be taken into consideration. Sliding of riprap stones on the geotextiles should also be taken into account.

Note that the percentages of the cases add up to more than 100 percent because in several cases multiple reasons for the failure were present.

# CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

## 5.1. RECOMMENDATIONS FROM LITERATURE REVIEW ON FILTERS FOR RIPRAP AND STABILITY OF RIPRAP-COVERED SLOPES

Some of the existing knowledge on filters under riprap layers and on the stability of riprap-covered slopes was collected and analyzed. The salient points are summarized in the following set of bullet points.

- Riprap Filters:
  - Two types of riprap filters exist: granular filters and geosynthetic filters.
  - A filter enhances the efficiency of the riprap by not allowing erosion of the underlying soil through the riprap.
  - The main parameters in geosynthetic filter design are permeability, transmissivity, porosity, AOS, POA, thickness, grab strength, tear strength, and puncture strength.
  - The main parameters in granular filter design are particle size distribution, permeability, porosity, thickness, and durability.
  - In most cases, a geosynthetic filter is preferred to a granular filter.
  - Many techniques for filter installation exist; the type used depends mainly on the type of filter, contractor, and environment. Placing riprap underwater is the biggest challenge.
- Riprap-Covered Slopes:
  - The water velocity applied to the riprap must be carefully evaluated to properly select the riprap size that will resist the flow.
  - The angle of the slope where the riprap is being placed is the major parameter because steep slopes are the major cause of failure for riprap-covered slopes. A threshold value for the steepest allowable slope must be determined.
  - The four main types of failure mechanisms for riprap-covered slopes are particle erosion, translational slide, slump, and modified slump.
  - Stabilization methods for riprap-covered slopes include toe trenching or excavation at the toe with backfilling; flexible launching apron; and toe wall.

A survey prepared by the Texas A&M Transportation Institute was distributed by TxDOT to all U.S. DOTs and a few additional entities in the world. The questions and the answers are summarized below.

- Riprap Filters:
  1. What is the most commonly used guideline for riprap filter design? HEC 23 (Lagasse et al., 2009).
  2. What percent of the time do you use filters below riprap? Most of the time (>80 percent of the time).

3. Do you prefer a granular filter or a geosynthetic filter? Geosynthetic.
4. How is the filter installed underwater? It is very difficult and often left to the contractor.
5. What is your experience in the comparison of granular and geosynthetic filter? Practically no experience was shared.
6. What is your experience in the comparison of riprap with and without filter? Riprap performance is best with a filter.
7. Can you share case histories of riprap failure? Five case histories were mentioned.

- Riprap-Covered Slopes:
  1. What is the most commonly used guideline for riprap-covered slope design? HEC 23 (Lagasse et al., 2009).
  2. What are your typical stone riprap toe details? Trenching to anticipated scour depth and back filling with erosion resistant material.
  3. How often do you experience riprap-covered slope failure? Rarely (1 to 5 percent of the time).
  4. What is the main cause of failure? Extreme event leading to undermining the toe of the slope and associated riprap stone movement.
  5. What measures are taken to avoid riprap-covered slope failures? Three components: proper design (riprap size and stone quality), proper construction/placement, and regular monitoring.
  6. What is the maximum slope angle you would use for riprap? Most answers stated a 2:1 slope.
  7. Can you share case histories of riprap failure? Only one case history was mentioned.

## 5.2. RECOMMENDATIONS ON FILTER SELECTION, DESIGN, AND PLACEMENT

Following are the conclusions covering what has been studied on granular and geosynthetic filters:

- Two types of riprap filters exist: granular filters and geosynthetic filters.
- A filter enhances the efficiency of the riprap by not allowing erosion of the underlying soil through the riprap.
- Using a filter between the riprap and the underlying soil improves the erosion resistance of the soil and of the riprap.
- Using a filter is better than using multiple layers of riprap without a filter.
- The main parameters in granular filter design are particle size distribution, permeability, porosity, thickness, and durability.
- The main parameters in geosynthetic filter design are opening size, permeability, porosity, thickness, and durability.

- The commonly used design approach was described in a step-by-step procedure and an example was given.
- Sometimes the design of a filter may require a combination of a granular filter and a geosynthetic filter.
- Many techniques for filter installation exist; the type used depends mainly on the type of filter, the contractor, and the environment. Placing riprap and filters underwater is the biggest challenge.
- Geosynthetic filters are easier to install than granular filters on land but are harder to install underwater.
- Granular filters are more expensive than geosynthetic filters.

## 5.3. RECOMMENDATIONS ON RIPRAP-COVERED SLOPES: STABILITY ANALYSIS

Two different approaches were taken for the stability analysis: an effective stress analysis with rapid drawdown and a total stress undrained analysis, both using Bishop's method of slices.

Conclusions from the results of the analyses are as follows:

- For a 1:1 slope and for a 1.5:1 slope, the riprap is very likely to fail.
- The steepest slope wherein riprap can be safely installed while meeting the TxDOT design requirements is a 2:1 slope (26.7 degrees).

Riprap on slopes has less erosion resistance than riprap on flat ground. The steeper the slope, the more likely the riprap is to move during high flows. Further research is needed to quantify this phenomenon.

## 5.4. RECOMMENDATIONS FROM RIPRAP FAILURE CASE HISTORIES

The following is a list of causes of riprap failures with recommendations on how to avoid such failures:

- **The size of the riprap stones is too small**. This common reason for riprap failure occurred 48.4 percent of the time for the failure cases reviewed. The solution is to increase the size of the riprap stones according to the design flow velocity. However, the size was found to depend on the slope angle, and a new chart is needed to select riprap size on slopes for a given velocity.
- **The toe of the riprap-covered slope scours and triggers riprap movement**. This occurs quite often (38.7 percent of the cases reviewed). It goes back to the failure mechanism associated with too steep a slope, but in this case the steep slope is created by scour at the bottom of the slope. This problem can be mitigated by reinforcing the toe of

the riprap-covered slope. One way is to extend the riprap far beyond the toe of the slope so that if scour occurs the riprap falls into the scour hole and slows down the process.

- **The riprap-covered slope is too steep**. This issue is also a major cause of failure that occurred in 35.5 percent of the cases reviewed. In these cases, the stability of the riprap itself on a steep slope was weakened by the steep angle. Lowering the slope to a 2:1 slope or flatter is recommended.
- **The natural soil erodes through the riprap and the riprap sinks**. This situation is not uncommon (16.1 percent of the case reviewed) but is often related to the absence of a filter between the natural soil and the riprap layer. It can be mitigated by using a proper filter material between the riprap and the underlying soil to stop seep-through erosion.
- **Failures are occurring during extreme events**. This event occurred in 12.9 percent of the cases reviewed. The design must be considered at the flood stage.
- **The soil slope fails because of water pressure buildup**. This situation occurred in 6.5 percent of the case reviewed. This type of failure is often generated by poor drainage in the filter, which leads to buildup of water pressure in the soil below. Mitigation requires selecting the hydraulic conductivity of the filter material to allow proper drainage of water out of the slope.
- **Poor installation and shape of the riprap**. This factor occurred in 6.5 percent of the cases reviewed. Proper installation will lead to better interlocking properties between riprap stones and increase stability. Although dumping riprap is common, the interlocking of stones should be evaluated after placement. The use of angular stones is recommended instead of rounded stones.
- **Absence of frequent and proper monitoring/inspection**. This cause of failure occurred in 6.5 percent of the cases reviewed. Early detection of instabilities in the system and taking proper corrective measures will minimize the impact of the problems.
- **The natural soil is too weak to handle the riprap loading**. This factor occurred in 3.2 percent of the cases reviewed. It can lead to a soil slope failure, but it is a rare failure mechanism for riprap-covered slopes.
- Tension failure of the geotextile should be taken into consideration. Sliding of riprap stones on the geotextiles should also be taken into account.

Note that the percentages of the cases add up to more than 100 percent because in several cases multiple reasons for the failure were present.

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# APPENDIX A: LIST OF CASE HISTORIES

| Number | Source | Location | Cause of Failure | Picture |
|---|---|---|---|---|
| 1 | TTI | SH 80 Over San Antonio River, Texas | Slope Failure |  |
| 2 | Oklahoma DOT | SH 74 over Skeleton Creek, Oklahoma 2015 | Steep slope |  |
| 3 | Mississippi DOT | 0 I-55 over Hickahala, Mississippi | Toe undermining |  |
| 4 | Mississippi DOT | SR 487 at Tuscolameta (North Canal), Mississippi | Riprap Movement |  |
| 5 | USGS | Pinole Creek at Pinole, CA | Riprap Movement |   |
| 6 | USGS | Sacramento River E-10, California | Riprap Movement |  |
| 7 | USGS | Hoh River at Site 1, Near Forks, WA | Translational Slide |  |

| | | | | |
|---|---|---|---|---|
| **8** | USGS | Cosumnes River at Site 3, near Slough house, CA | Modified Slump |  |
| **9** | USGS | Truckee River at Sparks, NV | Riprap Movement |  |
| **10** | NCHRP | Salmon Creek, Oregon | toe undermining |  |
| **11** | NCHRP | Grizzly Creek California | Under sizing, Heavy flow |  |
| **12** | NCHRP | Scholarie Creek, Albany, New York | Riprap Movement |  |
| **13** | CABS | Skookumchuck River, Lewis County, Washington | Debris Impinged | - |
| **14** | CABS | Puyallup River, Pierce County, Washington | Toe Scour | - |
| **15** | CABS | Cedar River, Orchard Grove, King County, Washington | Toe Scour (3V:4H) | - |

| | | | | |
|---|---|---|---|---|
| **15** | CABS | Cedar River, Orchard Grove, King County, Washington | Toe Scour (3V:4H) | - |
| **16** | CABS | Cedar River, Rainbow Bend, King County, Washington | Toe Scour (3V:4H) | - |
| **17** | CABS | South Fork Skagit R. Areas A&B Skagit County, Washington | Inadequate Filter, saturated bank, stage dropped fast, material piped out | - |
| **18** | CABS | North Fork Skagit R. Area C Skagit County, Washington | Bed and Toe scour | - |
| **19** | CABS | North Fork Skagit R. Areas D&E Skagit County, Washington | Toe Scour | - |
| **20** | CABS | South Platte River, 88th Av drop structure, Adams County, Colorado | Scoured type VL Riprap | - |
| **21** | CABS | Sanderson Gulch tributary of S.Platte, Denver County, Colorado | Toe Scour, no filter, rock too small | - |
| **22** | CABS | Van Duzen River by Grizzly Creek State Campground Rte. 36mile 16.9 California | River Bank Failure Flood, No filter and undersized rock | - |
| **23** | CABS | South Fork Eel River US 101 mile 21.7-22 Humboldt County, California | River Bank Failure Flood, no filter, thin lower and upper zones, fill piped out | - |

| | | | | |
|---|---|---|---|---|
| **24** | TxDOT | FM1155 Little Sandy Creek - BRYAN | Washed out |  |
| **25** | TxDOT | SH 30 Carter's Creek- Bryan | Gabions were undermined |  |
| **26** | TxDOT | SH 78 Price Creek- Dallas | Riprap Floated Downhill |  |
| **27** | TxDOT | FM982 Tickey Creek - Dallas | Riprap Washed away |  |
| **28** | TxDOT | FM 428 Aubrey Branch - Dallas | Scour exposed 3' of bottom slab wall toe wall and upstream end of culvert |  |
| **29** | TxDOT | US 77 SB Judy Creek - Dallas | Masonry Channel Liners rotated toward channel at southeast , southwest and northwest |  |
| **30** | TxDOT | FM 2164 Clear Creek - Dallas | Heavy Bank Erosion & Scour exposing drilled shafts at Bent 7 |  |
| **31** | TxDOT | IH 45 Drainage Ditch - Dallas | Moderate Scour and channel degradation exposed up to 3' of concrete apron slab toe wall at downstream end of culvert; riprap stones still in place |  |
| **32** | TxDOT | FM 85 Hobb Branch, - Dallas | Moderate scour at downstream end of culvert been repaired with concrete and stone apron |  |

| | | | | |
|---|---|---|---|---|
| 33 | TxDOT | FM 85 Draw - Dallas | Moderate scour washed out large portion of stone riprap downstream channel |  |
| 34 | TxDOT | FM 813 Draw, Dallas | Moderate Scour created 6' deep and 15' diameter hole at downstream end, riprap failed |  |
| 35 | | FM 945 San Jacinto River - Lufkin | Undermining of concrete riprap causing erosion of underlying soil |  |
| 36 | TxDOT | US 59 At Morgan Creek- Lufkin | Slope Failure due to riprap settlement |  |
| 37 | TxDOT | FM 218 Cowhouse Creek - Waco | Riprap Failed |  |
| 38 | TxDOT | IH 35 NBFR Cobb Creek - Wako | Stone Riprap Failed, Slope reshaped and installed riprap again |  |
| 39 | TxDOT | US 84 Navasota River Relief - Waco | Concrete Riprap failed. Stone riprap installed instead but riprap stones are washing away because they are too small |  |
| 40 | TxDOT | SH 159 Over Rocky Creek, Yoakum | Concrete Riprap failed and was replaced with stone riprap without filter |  |

| | | | | |
|---|---|---|---|---|
| **41** | TxDOT | SH 71 over Baylor Creek - Yoakum | Sheet piling and filled with rock as scour countermeasure |  |
| **42** | TxDOT | SH 71 EB over Rocky Creek - Yoakum | Concrete riprap being repaired by re-installing concrete riprap, but failed again. Rock riprap will be installed instead. |  |
| **43** | TxDOT | FM 667 Cobb Hollow - Wichita Falls | Riprap Washed away |  |

# APPENDIX B: VALUE OF RESEARCH

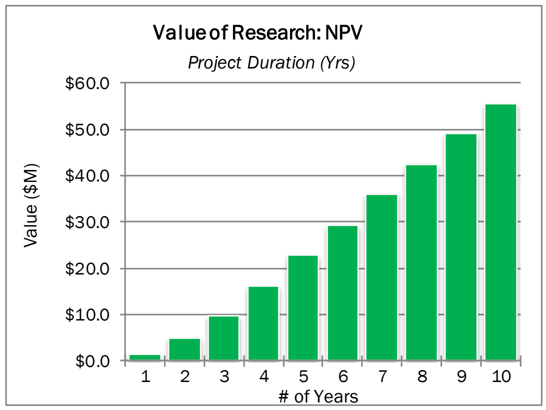
| | | |
|---|---|---|
| Project # | 0-7091 | |
| Project Name: | Riprap For Scour Countermeasures | |

| | | | |
|---|---|---|---|
| Agency: | TTI | Project Budget | $ 65,000 |
| Project Duration (Yrs) | 1 | Exp. Value (per Yr) | $ 5,052,844 |
| Expected Value Duration (Yrs) | 10 years, conservatively | Discount Rate | 0% |

**Economic Value**

| | | | |
|---|---|---|---|
| Total Savings: | $ 55,581,288 | Net Present Value (NPV): | $ 55,581,288 |
| Payback Period (Yrs): | 0.012864 | Cost Benefit Ratio (CBR) | 855 |

| Years | Expected Value |
|---|---|
| 0 | -$65,000 |
| 1 | $1,636,656 |
| 2 | $3,273,311 |
| 3 | $4,909,967 |
| 4 | $6,546,622 |
| 5 | $6,546,622 |
| 6 | $6,546,622 |
| 7 | $6,546,622 |
| 8 | $6,546,622 |
| 9 | $6,546,622 |
| 10 | $6,546,622 |




**Variable Justification**

The project goal is to evaluate the design of riprap including the need for filters and the stability of riprap covered slopes. From the Report on Texas Bridges (2016), TxDOT spent a total of 287.4 million dollars that year for new construction of on-system and off-system bridges. This number is assumed to be 300 million dollars in 2020. A total number of 50183 bridges in Texas was assumed. Then it was estimated that 80% of these bridges were build over rivers and that 50% of the bridges built over rivers needed riprap for erosion protection. It was further assumed that 300 cubic yards of riprap (50 ft wide bridge deck, 2:1 abutment slope 30 ft high, 60 ft long) is needed on every bridge. The TxDOT panel indicated that 127670 cy of stone riprap was used in 2020. That gives 426 bridges (127670/300) or 2.12% of the bridges over water in Texas needing riprap repair every year. After contacting a few contractors and checking with the TxDOT panel, the installed cost of riprap was chosen as 200$/cy of riprap. That leads to almost 25.5 Million dollars as the cost of riprap per year (200 x 127670). Considering an average cost for a new bridge equal to 1 million dollars and using 300 million dollars for new bridges in 2020 gives 300 new bridges built in 2020 in Texas. Using previous assumptions 120 (300 x 0.8 x 0.5) of those 300 new bridges will need riprap, costing around 7.2 million dollars every year (120 x 300 cy x 200 $/cy). Combining these numbers, TxDOT is assumed to be paying 32.7 million dollars per year for riprap repair of existing bridges and riprap protection for new bridges. After our study, it is assumed that TxDOT will be saving 20% of that cost per year in the future with a transition period for years 1, 2, and 3 at 5%, 10% and 15% respectively. The Numbers above show that the payback period will be less than a year, and have a net present value of 55.58 million dollars assuming 0% discount rate. This represents a cost to benefit ratio equal to 855 (55.58/0.065) and an expected yearly value of 5.05 million dollars in return for the 65000 dollars invested into this study.

**Qualitative Value**

| Benefit Area | Value |
|---|---|
| Increased Service Life | The value is not in an increase in service life but in maintaining the same service life for less money. |
| Traffic and Congestion Reduction | Keeping bridges open during high flood events will decrease traffic congestion. |
| Infrastructure Condition | The infrastructure condition will be extended because it will remain serviceable during flood events. |
| Engineering Design Improvement | TxDOT management faces a challenge in how to optimize the design of riprap including filters, and the stability of riprap on slopes. This study will help TxDOT to optimize the design of riprap at bridge piers and abutments. |