A-JACKS CONCRETE ARMOR UNITS
Channel Lining and Pier Scour
Design Manual

Prepared for

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1. INTRODUCTION

1.1 General

This manual provides technical information and guidelines for the hydraulic design of stable open-channel conveyanceways and pier scour countermeasures using A-JACKS® concrete armor units. The interlocking A-JACKS system is assembled into a continuous yet flexible matrix that provides protection against high-velocity flow. The matrix of A-JACKS units has a high void ratio and can be backfilled with topsoil and vegetated to increase the stability of the system. The system provides a nonerodible boundary between the channel subgrade and the potentially damaging flow of water.

The A-JACKS system may be used for bank toe stabilization in combination with "softer" techniques to secure the upper bank. A-JACKS can also be used as an armoring countermeasure to minimize the amount of local pier scour at bridges. The ability of the A-JACKS system to dissipate energy and resist the erosive forces of flowing water allows the system to protect channel boundaries from scour and erosion.

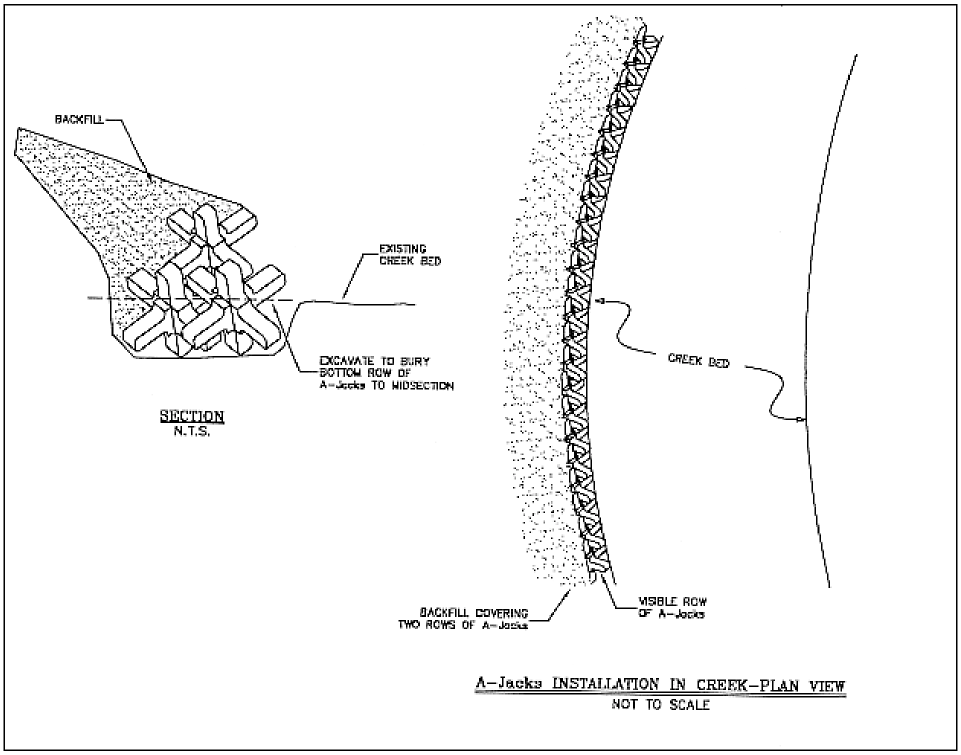


Figure 1.1. A-JACKS used for bank toe stabilization.

The A-JACKS system is presented as an alternative to dumped rock riprap, grouted rock, gabions, and other heavy duty channel and pier scour protection systems. In some geographic locations the size, quality and gradation of rock required for a particular application may be difficult to obtain. The cost of transporting rock over large distances can be quite expensive. Rock of poor quality can potentially break into smaller pieces and become mobilized by the flow, and therefore not provide the required protection. Riprap performance is also dependent on an appropriate gradation of material. The ability to manufacture the A-JACKS system at a consistent size and quality (durability) allows it to maintain a positive interlock and function reliably at the required level of performance.

The design relations presented in this manual are developed from physical principles of open channel flow supported by extensive laboratory testing. They represent semi-empirical, dominant-process models that are internally consistent and well suited as design tools. Because the relations represent a simplification of a complex process, the underlying assumptions of the methods, areas of applicability, and limits of the techniques are also described.

1.2 Background

Manufactured concrete armor units have been used in coastal applications since the 1800s. They have been utilized successfully as breakwaters, groins, seawalls and other coastal revetments. Armor units are preferred over riprap in certain ocean environments where the availability of large rock that is suitable to resist the impact forces of waves is limited. In cases were the cost of large rock is excessive, manufactured concrete armor units have been a preferred alternative. The advantages of armor units are that they are available in large sizes, the interlocking characteristics provide a high level of stability, and they can be produced at a consistent level of quality.

The use and consideration of armor units for erosion protection in the river environment has grown in the last century. The U.S. Army Corps of Engineers has employed concrete armor units for channel protection on the Mississippi River. Japanese engineers have used armor units to protect bridge piers at highway and railway crossings. Several laboratories have tested concrete armor units for use in open channel conveyance systems. The U.S. Army Corps of Engineers' Waterways Experiment Station, Turner Fairbanks Laboratory of the Federal Highway Administration, Colorado State University's Engineering Research Center, and University of Minnesota's St. Anthony Falls Hydraulic Laboratory have tested many different types of concrete armor units for riverine applications.

1.3 Organization of this Manual

This manual provides design information for the A-JACKS system and is organized based upon the type of application being considered:

Chapter 2 provides design information for bed and bank protection applications. The fundamental principles of uniform flow in open channels are introduced in <u>Section 2.1</u>. This section includes necessary assumptions, descriptions and equations used in standard engineering practice which are required to understand and quantify the forces generated on a channel boundary by flowing water.

<u>Section 2.2</u> provides hydraulic performance characteristics of the A-JACKS system determined from laboratory testing. This section includes Manning's roughness coefficients, equivalent roughness heights, and design shear stresses for the model scale units as well as the four commercially-available prototype A-JACKS sizes. The principles of Froude scaling are discussed to provide the background for utilizing laboratory data for field scale applications.

<u>Section 2.3</u> contains a design procedure for the selection of appropriate sizes of A-JACKS for channel lining applications. Charts and worked examples for commonly used trapezoidal channel geometries are provided. Corrections for computing local stress effects of bends, constrictions, and expansions are presented.

Section 2.4 provides general installation procedures and recommendations, along with typical construction details that apply in the most commonly occurring channel lining situations. Subjects covered in this section include subgrade preparation and testing, geotextile considerations, and system termination recommendations.

Chapter 3 provides design information for pier scour applications. The mechanics of local scour are briefly developed in Section 3.1. This section describes the physical processes that cause local scour and presents generally accepted methods for predicting the local scour depths and scour hole geometry. In addition, the methodology for pier scour countermeasure stability analysis is presented.

Section 3.2 provides performance characteristics for the A-JACKS system placed at bridge piers. Included are results from scale model laboratory tests on local scour reductions for various A-JACKS placement patterns.

Section 3.3 presents a design procedure for the selection of appropriate sizes of A-JACKS for local pier scour mitigation. Charts and worked examples for common pier configurations are provided. Corrections for pier shape and angle of attack are presented.

Product Information is provided in the next section of this manual. This section contains information on dimensions, weights, and standard installation details for the various sizes of A-JACKS armor units. Regional and international product distribution contacts are also provided.

The **References and Appendices** section contains relevant technical citations referenced in this manual, as well as descriptions of related testing reports, field studies, and other sources of data regarding the A-JACKS concrete armor system.

1.4 Disclaimer

Ayres Associates was commissioned by Armortec, Inc. to develop this hydraulic design manual for the A-JACKS system in open channel bank protection and pier scour countermeasure applications. Ayres Associates utilized laboratory data obtained from the Hydraulics Research Laboratory at Colorado State University's Engineering Research Center for this purpose. Armortec and CSU established the scope of laboratory testing. Ayres utilized this data to develop hydraulic stability criteria based on accepted engineering principles. The selection and design criteria presented in this manual assume that installation procedures and quality standards replicate that which was achieved in the hydraulic testing laboratory. The criteria are based on the high-density interlocked configuration of the armor units, as opposed to randomly placed elements.

This manual is intended for use as an analysis and design aid by engineering professionals having a background in hydrology and open-channel flow hydraulics. Although the design charts presented in this manual could be used in "cookbook" fashion, an understanding of free-surface flow behavior and boundary stresses by the practitioners is warranted. There is no substitute for experience and good engineering judgement; given these, designs based on the use of the charts and tables in this manual will, with very few exceptions, result in reasonably conservative installations. It is to be expressly understood that the responsibility for the success or failure of an engineering design rests with the engineer of record; use of the information contained in this manual in no way implies review or approval of a specific design by Armortec, its agents, or its consultants.

2. CHANNEL BED AND BANK PROTECTION

The selection and design of A-JACKS for channel bed and bank applications requires an understanding of open channel hydraulics and stable channel design. A review of these fundamentals is presented to provide the designer with the theory behind the development of design parameters. Specific design parameters for A-JACKS are presented to facilitate hydraulic analyses and selection of appropriate sizes of armor units for the design of stable channels.

2.1 Fundamentals of Open Channel Flow

2.1.1 Basic Concepts

The hydraulic conditions of open channel flow are a function of the channel geometry, discharge, roughness, and slope. The degree of erosion protection required can only be determined after the hydraulic conditions of flow are known. Typically, a design discharge is selected using appropriate hydrologic techniques; this discharge usually corresponds to a storm event of specified frequency, such as a 10-, 25-, or 100-year storm, as required by the regulatory authority having jurisdiction over the project. The channel size and configuration are largely dictated by physical constraints of the site, although the designer usually has some latitude for refinement of cross-section, slope, and alignment during the design process. Several trials are usually required before arriving at a final design.

Open-channel flow can be classified according to three general conditions:

1. Uniform or nonuniform flow
2. Steady or unsteady flow
3. Subcritical or supercritical flow

In <u>uniform</u> flow, the depth, cross-sectional flow area, and velocity along the reach of channel remain constant. <u>Nonuniform</u> flow is characterized by accelerations or decelerations caused by changes in slope or cross-sectional geometry of the channel; as a result the depth, velocity or cross sectional area can vary from one location to the next. In <u>steady flow</u>, discharge does not change over time. Steady uniform flows are rare in natural streams; however this condition is frequently used for open channel design. <u>Unsteady</u> flows are usually characterized by a discharge hydrograph that rises, peaks, and falls as dictated by the inflow pattern. In many cases, the runoff hydrograph varies gradually, so that for practical purposes, the flow can be described as a series of intervals, with each interval assumed to exhibit steady flow characteristics. In practice, the peak flow rate of the hydrograph is typically used as the design discharge, and, for purposes of hydraulic analysis and channel design, is usually treated as steady flow.
<u>Subcritical</u> flow is described as tranquil, and is characterized by relatively deep flow with slow velocity. <u>Supercritical</u> flow, on the other hand, is described as rapid or "shooting" flow, with shallow depths and high velocity. The dimensionless number known as the Froude number, F_r, is used to distinguish between subcritical and supercritical flow. The Froude number is defined as the ratio of inertial forces to gravitational forces in the flow field:

$$F_r = \frac{V}{\sqrt{gy}} \tag{2.1}$$

where:

V	=	average velocity of flow (ft/s or m/s)
g	=	acceleration due to gravity (32.2 ft/s^2 or 9.81 m/s^2)
y	=	hydraulic depth, defined as the flow area divided by the top width of the water surface (ft or m)

A Froude number less than 1.0 indicates that subcritical flow is occurring, whereas a Froude number greater than 1.0 indicates that the flow is supercritical. In the transition range ($0.8 < F < 1.2$) the flow field is highly unstable and tends to oscillate rapidly and unpredictably between the subcritical and supercritical regimes. Channel designs that result in a Froude number in the transition range should be avoided.

2.1.2 Hydraulics of Steady Flow

For design purposes, steady uniform flow is often assumed. Uniform flow conditions result when the slope of the energy grade line is equal to the slope of the channel bed. The Manning equation provides a reliable estimate of uniform flow conditions, and is expressed as:

$$Q = \frac{C}{n} A R^{2/3} S_f^{1/2} \tag{2.2}$$

where:

Q	=	design discharge, in cubic feet per second (cfs or cms)
C	=	coefficient for unit type, C = 1.486 for English units and C = 1.0 for SI units
n	=	Manning's roughness coefficient
A	=	cross-sectional flow area (ft^2 or m^2)
R	=	hydraulic radius, equal to the cross-sectional area A divided by the wetted perimeter P (ft or m)
S_f	=	slope of the energy grade line (approximated by the average bed slope)

Given the design discharge, cross-sectional geometry, roughness coefficient, and bed slope, numerically solving the Manning equation for flow depth, y_o, typically requires an iterative procedure, because both the area A and the hydraulic radius R are functions of the unknown depth. **Figure 2.1** provides a nomographic solution to the Manning equation for trapezoidal channels, and solves for the depth of flow, y_o, under uniform flow conditions. This depth is termed normal depth, and is representative of flow conditions wherein the resistance to flow is exactly balanced by the gravitational force.

Once the depth of flow is known, the mean (cross-sectional average) velocity V of the flow can be calculated as:

$$V = \frac{Q}{A} \tag{2.3}$$

where $A = by_o + Zy_o^2$ for trapezoidal channels with side slope ratio z horizontal to 1 vertical, and base width b.

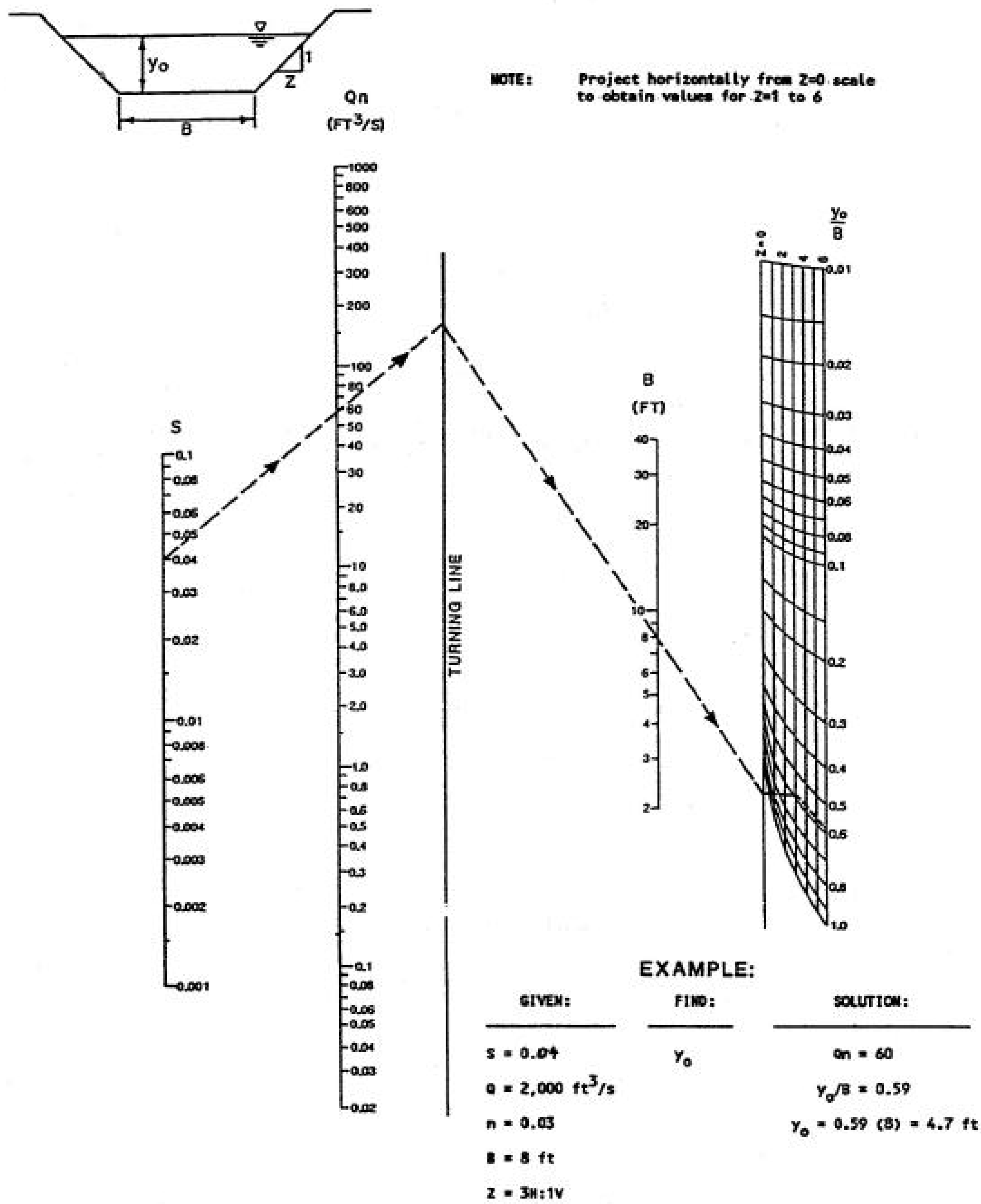


Figure 2.1. Nomographic solution of Manning's equation for trapezoidal channels.

Nonuniform flow conditions exist due to changes in flowrate, channel geometry, slope, or some combination of these variables. When flow is nonuniform, the depth will be either greater than or less than the normal depth computed with Manning's equation. Nonuniform flow can be described as gradually varied flow or rapidly varied flow. Rapidly varied flow occurs over relatively short distances where local accelerations or decelerations are more important than friction. Gradually varied flow occurs over longer distances and friction losses are more important than acceleration. Nonuniform flow is computed with the energy and momentum equations. The energy equation is expressed as:

$$Z_1 + y_1 + \frac{V_1^2}{2g} = Z_2 + y_2 + \frac{V_2^2}{2g} + h_L \qquad (2.4)$$

where:

$Z_{1,2}$	=	elevation of the channel bed at two adjacent cross sections (ft or m).
$y_{1,2}$	=	depth of flow at two adjacent cross sections (ft or m).
$V_{1,2}$	=	mean velocity at two adjacent cross sections (f/s or m/s).
h_L	=	frictional losses = $S_f L$ (ft or m).
L	=	effective length between two adjacent cross sections (ft or m).

S_f can be computed using the Manning roughness coefficient. The energy equation is used to solve for flow depths along a channel reach. When the friction slope, S_f, equals the channel bed slope the energy equation will yield the same results as Manning's equation. Therefore, Manning's equation can be used if changes in channel geometry and slope are relatively small from one cross section to the next. The momentum equation should be considered for rapidly varied flow conditions, but the energy equation may applicable for rapidly varied flow. Computer programs such as HEC-RAS are used to solve the energy and momentum equations to determine hydraulic conditions for open channel design with nonuniform flow.

2.1.3 Surface Roughness

The Manning's roughness coefficient n is approximately constant for channels where the turbulence at the channel boundary is small with respect to the total depth of flow. The Manning's roughness coefficient is, however, also dependent on depth in channels where the boundary turbulence is a significant portion of the flow depth. For the A-JACKS system, when used as a channel or bank lining, the Manning roughness coefficient will vary depending on the flow conditions.

A relationship that describes the Manning's roughness coefficient as a function of flow depth can be represented using the *roughness height* concept. Roughness height describes the linear dimension of turbulence above the channel bed. This height is assumed constant for a given channel lining material. The roughness height is dependent on the magnitude of surface irregularities of the channel lining material. Roughness height is related to the size of the bed material in alluvial channels. For the A-JACKS system, the roughness height is related to a characteristic linear dimension of the individual units that comprise the system. The roughness height relationship is based on the Prandtl-von Karman universal velocity distribution law. Using this law Keulegan (1938) derived equations for velocity profiles that have the following form:

$$\frac{V}{V_*} = 5.75 \log\left(a\frac{R}{K_s}\right) \qquad (2.5)$$

where:

V_*	=	shear velocity = (τ_o/ρ)^0.5 (ft/s or m/s).
τ_o	=	boundary shear stress (lb/ft^2 or Pa).
ρ	=	density of water, 1.94 slug/ft^3 or 1000 kg/m^2 for clear water.
K_s	=	roughness height (ft or m).
a	=	coefficient which is dependent on channel shape.

The coefficient a = antilog (A_o/5.75) where A_o is used when equation 2.5 is written as V/V_* = A_o + 5.75log(R/K_s) (Chow 1959). Typical values of A_o = 6.25 and a = 12.2 are commonly used in practice. Substituting equation 2.2 into 2.5 Manning's n can be related to the roughness height and hydraulic radius in the following equation:

$$n = \frac{CR^{1/6}}{\sqrt{g}\, 5.75 \log\left(a\frac{R}{K_s}\right)} \qquad (2.6)$$

where g is the acceleration due to gravity and all other variables are previously defined. It is common to replace the hydraulic radius, R, in equations 2.5 and 2.6 with the depth of flow, y_o, to determine the Manning roughness coefficient for a point in the flow field or on the channel bed. Laboratory data from A-JACKS tests performed at Colorado State University were used to determine K_s values for the A-JACKS system for use as a channel or bank lining. Plots of the variation in Manning n with respect to depth for the A-JACKS systems are provided in section 2.2.

2.1.4 Stable Channel Design Concepts

Stable channel design concepts focus on evaluating and defining a channel configuration that will perform within acceptable limits of stability. In the case of static equilibrium, stability is achieved when the material forming the channel boundary effectively resists the erosive forces of the design flow.

In a dynamic system, some change in the channel bed or banks is to be expected if erosive forces of the flow are sufficient to detach and transport the materials comprising the boundary. Stability in a dynamic channel reach is generally achieved when the sediment supply rate from upstream equals the sediment-transport rate through the reach. This condition is referred to as dynamic equilibrium. Dynamic equilibrium evaluations and techniques are most often applied to natural streams and rivers in areas remote from urbanization or other man-made improvements, where a certain amount of natural lateral and/or vertical changes to the channel can be accommodated.

For most development projects, bridges, culverts, roadway drainage applications, and other designs where nearby infrastructure is involved, bed and/or bank instability (with potential lateral migration) cannot be tolerated. In these situations, the establishment of static equilibrium through the utilization of erosion-resistant channel boundaries is preferred over dynamic equilibrium concepts.

2.1.5 Hydraulic Forces

Investigations by the U.S. Bureau of Reclamation in the 1950s led to the development of the permissible tractive force procedure. This methodology provides a more fundamental basis for relating the erosion resistance of boundary materials to the erosive force of the flow. Less empirical in nature than the permissible velocity approach, the tractive force procedure is more easily extended to various channel linings. The average tractive force per unit area (or shear stress) over the channel boundary is given by:

$$\tau_o = \gamma \; R \, S_f \tag{2.6}$$

where:

τ_o	=	average tractive force per unit area (or shear stress) (lbs/ft^2 or Pa)
γ	=	unit weight of water, 62.4 lbs/ft^3 or 9810 N/m^3 for clear water
R	=	hydraulic radius = Area of flow divided by wetted perimeter; (A/P) (ft or m)
S_f	=	slope of the energy line (approximated by the bed slope for uniform flow)

The maximum shear stress on the boundary of a straight channel occurs on the channel bed, and is determined by substituting the depth of flow y_o for the hydraulic radius R in the above equation, yielding:

$$\tau_o = \gamma \; y_o \, S_f \tag{2.7}$$

where:

τ_o	=	maximum shear stress on channel bed
y_o	=	maximum depth of flow
		other terms as defined previously

Shear stresses in channels are not uniformly distributed along the wetted perimeter. Shear stress varies with velocity and depth across the channel. The shear stress at a point in the flow (i.e., on the channel bed or bank) can be computed using the logarithmic velocity distribution defined in equation 2.5. Solving for shear stress and substituting the depth of flow, y_o, for the hydraulic radius, R, yields:

$$\tau_o = \frac{\rho V^2}{\left[5.75 \log\left(a \frac{y_o}{K_s}\right)\right]^2} \tag{2.9}$$

An assessment of the vertical velocity distribution is required to apply equation 2.9. A distribution of shear stress based on the logarithmic velocity distribution in a straight reach of a trapezoidal channel is shown in **Figure 2.2a**.

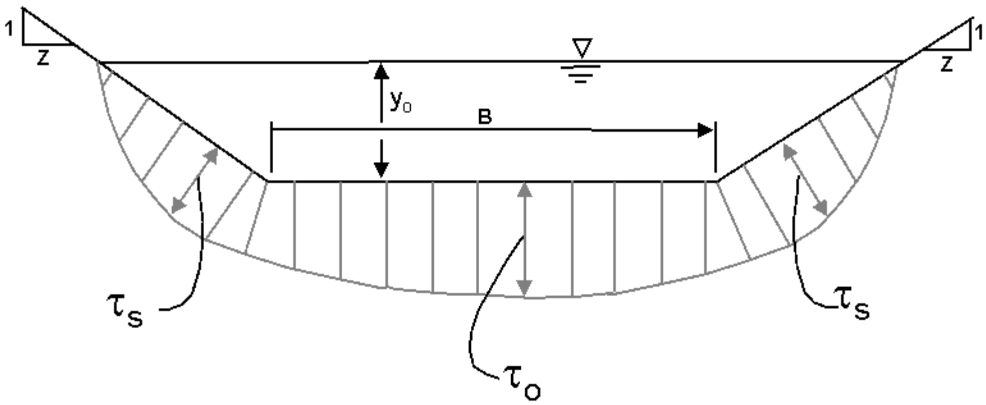


Figure 2.2a. Shear stress distribution on the boundary of a trapezoidal channel in a straight reach.

Flow around a bend creates secondary currents, which impose higher than normal shear stresses on the channel sides and bottom in localized areas, as shown in **Figure 2.2b**.

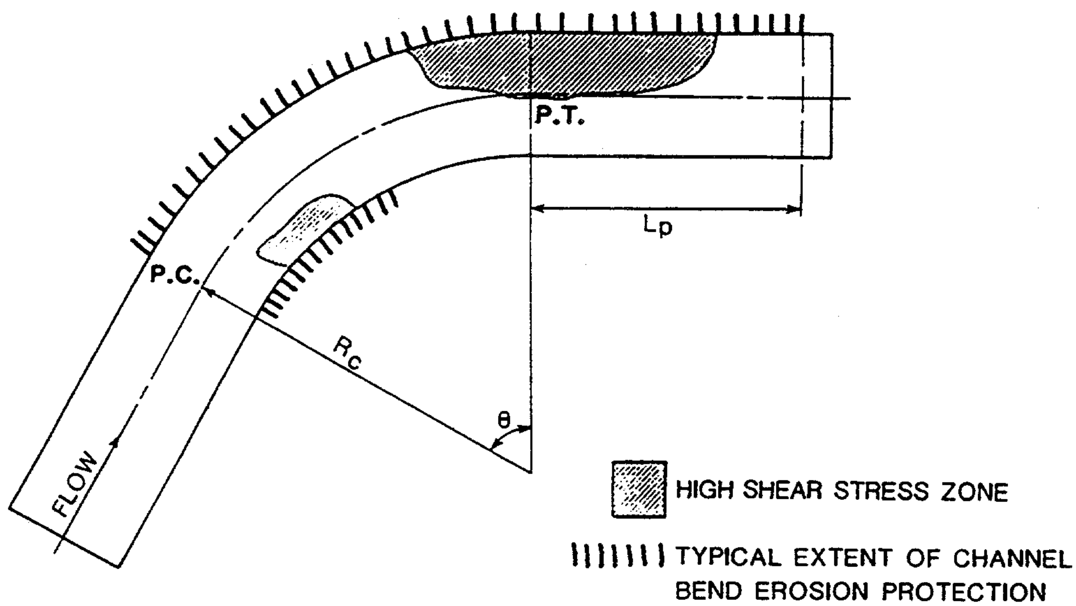


Figure 2.2b. Shear stress concentration areas in channel bend.

At the entrance to the bend, the maximum shear stress is located near the inside of the curve. Near the exit of the bend, the zone of high shear stress is located near the outside of the curve, and persists a distance L_p downstream from the point of tangency. The amount of increase in the shear stress due to a channel bend is related to the ratio of channel radius of curvature to the bottom width, R_c/b. The sharper the bend (small R_c), the higher the amount of shear stress increase. The bend shear stress, τ_b, is expressed by the dimensionless factor, K_b, from **Figure 2.3** multiplied by the maximum shear stress for an equivalent straight reach:

$$\tau_b = K_b \ \tau_o \tag{2.10}$$

It can be seen from Figure 2.4 that for relatively sharp bends, the effective shear stress can nearly double in magnitude compared to straight reaches.

The distance L_p over which the high shear stresses persist downstream from the bend is a function of the roughness of the boundary in the bend, n_b, and the hydraulic radius of the channel, R:

$$\frac{L_p}{R} = \frac{0.604 R^{1/6}}{n_b} \tag{2.11}$$

Figure 2.4 provides the relationship of L_P/R to n_b, for typical ranges of the hydraulic radius R. From this chart, it is seen that the effect of increasing the bend roughness serves to decrease the downstream distance over which the shear stress is influenced This is due to the ability of a rougher boundary to more rapidly dissipate the secondary currents created by the bend. Because the A-JACKS system provides more roughness than traditional revetments, this infers that the A-JACKS system can reduce the length of the high shear stress zone downstream of a bend.

2.1.6 Hydraulic Stability of A-JACKS Concrete Armor Units

The stability of the A-JACKS is contingent on the system being installed in a continuous matrix that spans the extent of the surface to be protected. Individual A-JACKS armor units surrounded by a matrix of identical blocks are subjected to the forces of lift and drag under the action of flowing water. The lift force acts in a direction normal to the plane of the bed, and is typically comprised of the buoyant force and differential pressure across the matrix due to local accelerations. The drag force acts in the direction of flow, and is comprised of frictional drag and form drag. The lift force and the drag force combine to produce an overturning moment, which is resisted by the submerged weight of the interlocking matrix as shown in **Figure 2.5**.

2.1.7 Definition of Failure

Loss of "intimate contact" between the channel lining and the subgrade which it protects has been identified as the primary indicator of incipient failure for many types of channel revetment. Given the nature of revetment installation in typical channel applications, failure due to slipping or sliding of the revetment matrix along the plane of the bed is remote. The loss of intimate contact, therefore, is the result of overturning or uplift of a unit or group of units from the matrix. Incipient uplift occurs when the overturning moments equal resisting moments about the downstream contact point of the unit. Once one or more armor units becomes destabilized the system may continue to degrade if the flow conditions that caused initial failure persist. The A-JACKS design criteria relies on their ability remain interlocked in a closely-packed matrix.

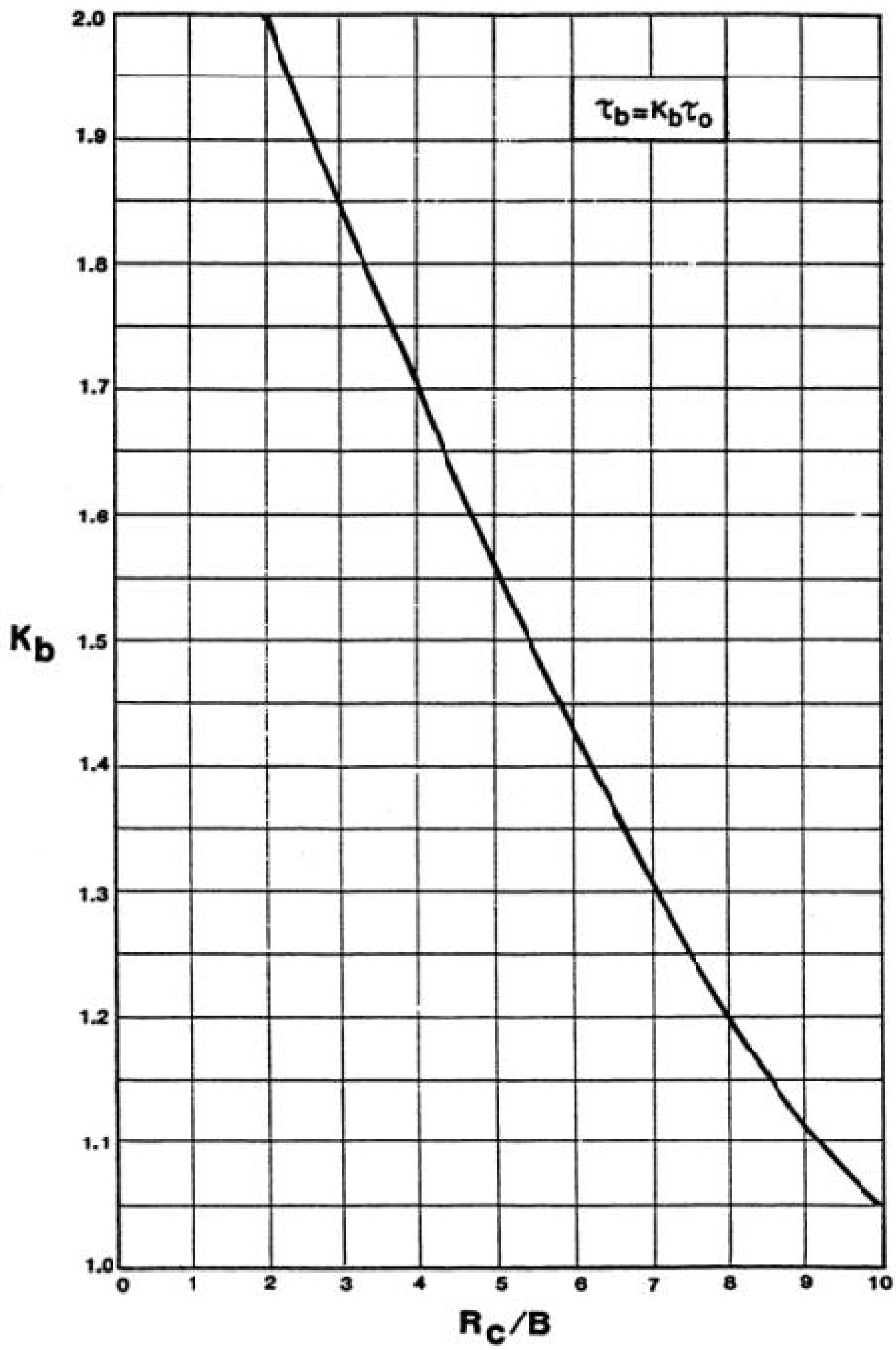


Figure 2.3. K_b factor for computing shear stresses at channel bends.

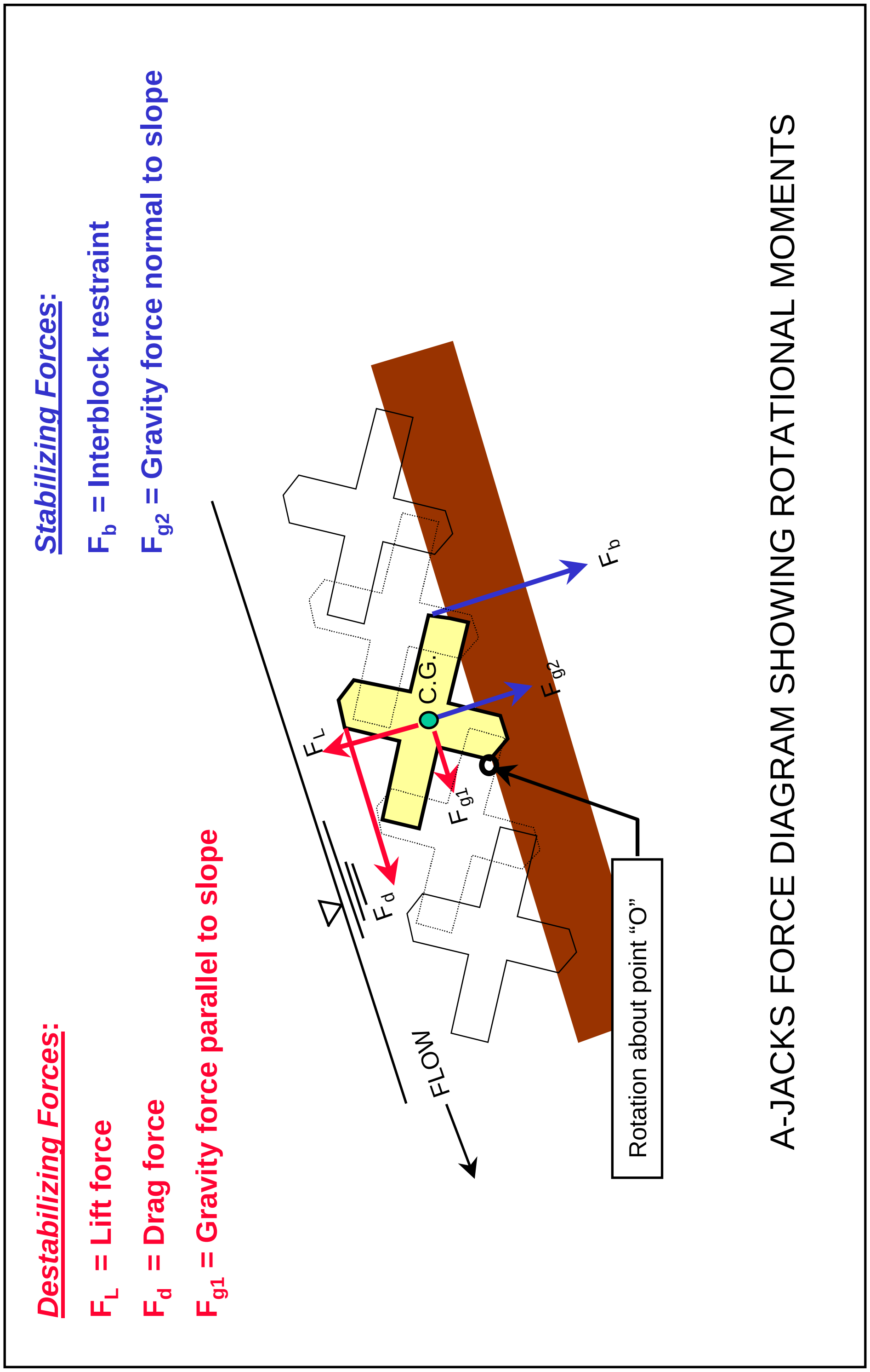


Figure 2.4. A-JACKS force diagram showing rotational moments.

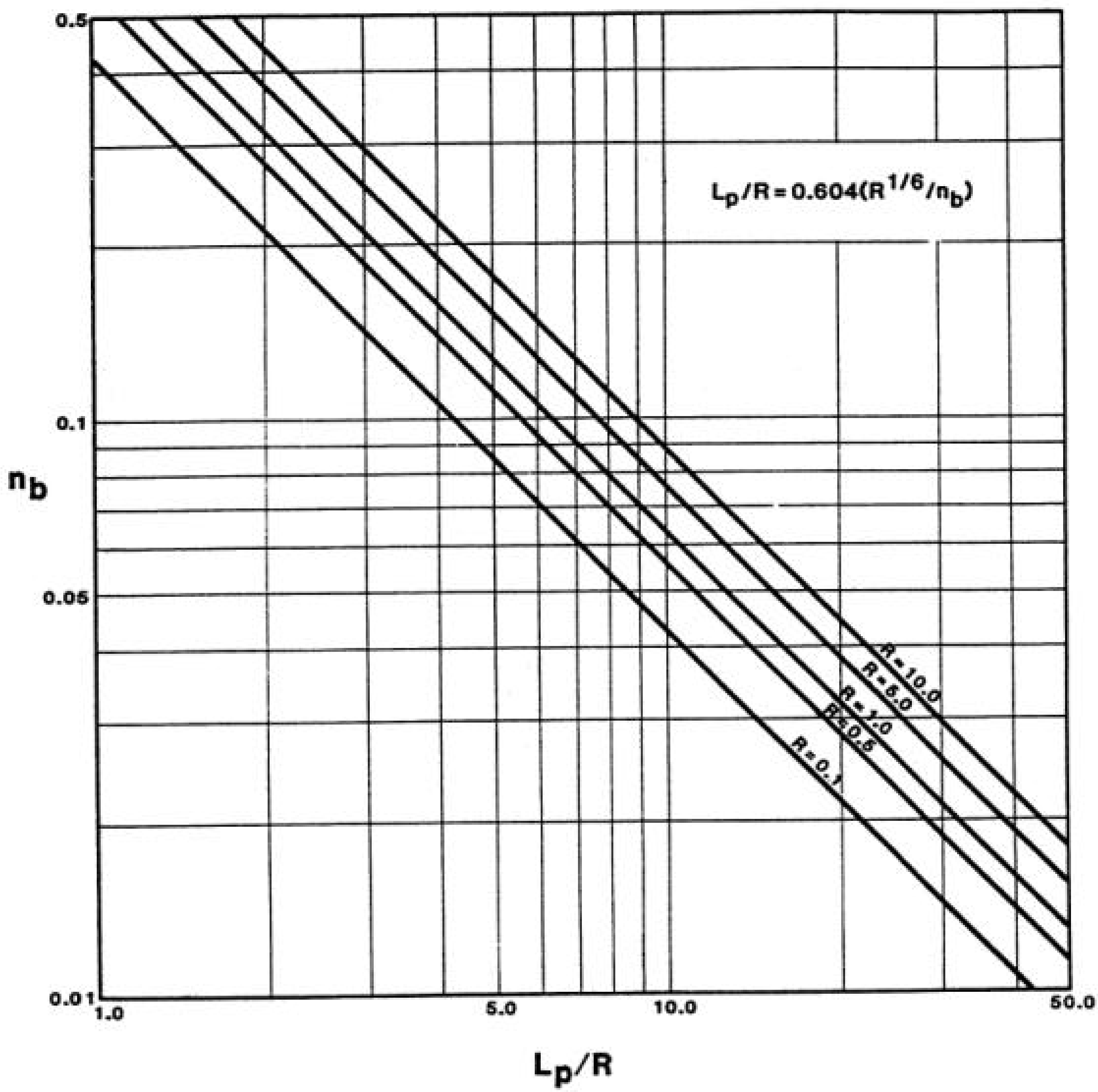


Figure 2.5. Protection length L_p downstream of channel bends.

2.1.8 Hydraulic Stability - The Discrete Particle Method

The method for quantifying the hydraulic stability of an A-JACKS matrix utilizes the "discrete particle" approach. Adapted to manufactured revetment systems, this approach is similar to that introduced by Stevens (1968) for sizing rock riprap. The stability of A-JACKS is dependent on the interlocking characteristics of individual units to neighboring units in the matrix. The design analysis method compares the ratio of the overturning moments due to uplift and tractive forces caused by the flow to the resisting moments due to weight and interlock unique to the A-JACKS system. The ratio of resisting forces to tractive forces is known as the **factor of safety** against failure as described by the following equation.

$$FS = \frac{\tau_c (K_s)}{\tau_o} \tag{2.12}$$

where:

FS	=	factor of safety
τ_c	=	critical shear stress of the A-JACKS matrix (lb/ft^2 or Pa)
τ_o	=	the design hydraulic shear stress caused by the flow (lb/ft^2 or Pa)
K_s	=	side slope reduction factor

The resisting forces are dependent on the on the size and weight of an individual unit within the matrix, as well as the restraint provided by adjacent units. This method is calibrated to the results obtained from laboratory model tests, and can be extended to larger A-JACKS units through Froude-law scaling, providing the individual units are geometrically similar to those tested.

2.1.9 Selection of Factor of Safety for Channel Revetments

It is the designer's responsibility to determine the appropriate factor of safety to be used for a particular design. Some variables that affect the factor of safety are (1) the frequency of the design event, (2) risks associated with a failure of the project, (3) the uncertainty of hydraulic values used in the design, and (4) uncertainties associated with the quality of subgrade preparation or revetment installation. Typically, a minimum factor of safety against failure during the design hydrologic event of 1.2 is used for revetment design, with values of 3.0 or greater specified for areas of complex flow fields or geometric irregularities. Values of 5.0 or more may be used in cases where the consequences of failure are dire.

2.2 A-JACKS Hydraulic Performance Characteristics

2.2.1 General

A comprehensive physical model testing program of A-JACKS revetment in channel bed and bank lining applications was conducted by Colorado State University researchers at CSU's Engineering Research Center in late 1998 and early 1999. The purpose of the laboratory tests was to document the hydraulic characteristics and performance capability of A-JACKS under various conditions of open-channel flow.

Both 6-inch and 24-inch A-JACKS units were studied in a variety of flume environments under controlled laboratory conditions. Three placement configurations were examined: random (loose), low density interlocked, and high density interlocked. From these studies, it was determined that the high-density interlocked configuration resulted in the highest and most consistent stability performance. This configuration forms the basis for the design procedures developed in this chapter. A complete and detailed description of CSU's testing program, data, observations and conclusions is provided in the CSU research documents entitled, "A-JACKS Hydraulic Property Documentation" (Holmquist-Johnson, et al., February 1999), and "A-JACKS Full-Scale Testing" (Thornton et al., February 1999).

Ayres Associates utilized the data from the CSU testing program to develop design parameters for the range of A-JACKS sizes commercially available for field installations (24, 48, 72, and 96-inch tip-to-tip dimensions, corresponding to individual unit weights of 78, 265, 2,120, and 5,020 pounds, respectively). The following sections present the theory and results of this procedure.

2.2.2 Froude-Law Scaling

The results of the hydraulic tests on the 6-inch and 24-inch A-JACKS are extrapolated to the larger sized units by means of hydraulic similitude using the dimensionless Froude number. This scaling techniques acknowledges that in open channel flow, the forces due to gravity and inertia are of paramount importance. It also assumes that the effects of scaling on viscosity, pressure, and surface tension are either negligible or irrelevant to the stability of the prototype (i.e., field-scale) units.

All pertinent testing parameters may be defined in terms of the model scale ratio λ, defined in terms of a representative length unit as:

$$\lambda = L_m/L_p$$

where:

L_m	=	linear dimension in model
L_p	=	linear dimension in prototype

For example, when testing a matrix of 6-inch A-JACKS and extrapolating the test results to the 48-inch units, λ = (6")/(48"), or 1/8. In accordance with the principles of dimensionless Froude number scaling, the pertinent hydraulic relationships are listed in **Table 2.1**.

2.2.3 Hydraulic Resistance: Manning's n

The A-JACKS high-density interlocked configuration was tested in CSU's 2-foot indoor flume at two different bed slopes, 0.017 ft/ft and 0.04 ft/ft. At each bed slope, a series of discharges was run and Manning's n was determined for each discharge from measured data. The Manning's n value is related to the Darcy friction factor f by the relation

$$f = 116\, n^2 y^{-1/3}$$

where:

y	=	depth of flow

Table 2.1. Hydraulic Relationships According to Froude Number Scaling.			
Parameter		Dimensions	Scale factor
Dimensional properties:	Length	L	λ
	Area	L^2	λ^2
	Volume	L^3	λ^3
Kinematic properties:	Time	T	$\lambda^{1/2}$
	Velocity	L/T	$\lambda^{1/2}$
	Discharge	L^3/T	$\lambda^{5/2}$
Dynamic properties:	Mass	M	λ^3
	Force	MLT^{-2}	λ^3
	Shear stress	$ML^{-1}T^{-2}$	λ
Resistance:	Manning's n		$\lambda^{1/6}$
Note: Assumes undistorted model with fresh water used for both model and prototype.			

An effective roughness height K_s can be calculated from this data by plotting the quantity $f^{-0.5}$ as a function of y/L, where L is the representative length of the tested A-JACKS unit, in this case 6 inches. As seen in **Figure 2.6**, the CSU flume data closely matches the Prandtl-von Karman velocity distribution law (equation 2.6) using a best-fit K_s value of 5.7 times the representative length, or 34 inches, as determined by linear regression. Using Froude-law scaling, the Manning n values for various sizes of A-JACKS prototype units are presented in **Figure 2.7** as a function of depth of flow. Also on this figure are shown the Manning n values calculated from tests performed on the full-scale AJ-24 A-JACKS units (24 inch length scale) at CSU's outdoor flume. As seen from this figure, the full-scale tests provide independent validation of the Froude-law scaling technique.

2.2.4 Critical Shear Stress

Due to limited discharge capacity and available bed slopes, indoor flume tests at CSU did not result in destabilization of the 6-inch model A-JACKS in the high-density interlocked configuration. Therefore, additional testing on the 6-inch A-JACKS was performed in a 4 foot wide outdoor flume at CSU's Engineering Research Center. This flume is part of the test facility referred to as the Tarbela flume in recognition of its original development and use for dam and spillway studies to support the design of the Tarbela Dam, Pakistan's largest water supply and hydroelectric facility.

A-JACKS were placed in the high-density interlocked configuration along a 30-foot length of flume at a bed slope of 0.13 ft/ft (13 percent). Six tests were conducted at increasing discharges, up to a maximum of 46 ft^3/s during Test Run 6. An applied shear stress of 9.5 lb/ft^2 and a peak velocity of 11.0 ft/s were calculated from measured data on the 6-inch A-JACKS units at the highest discharge investigated. No movement of the A-JACKS was observed in any of these tests. A plot of bed elevation, water surface elevation, and energy grade line corresponding to Test Run 6 is provided in **Figure 2.8**.

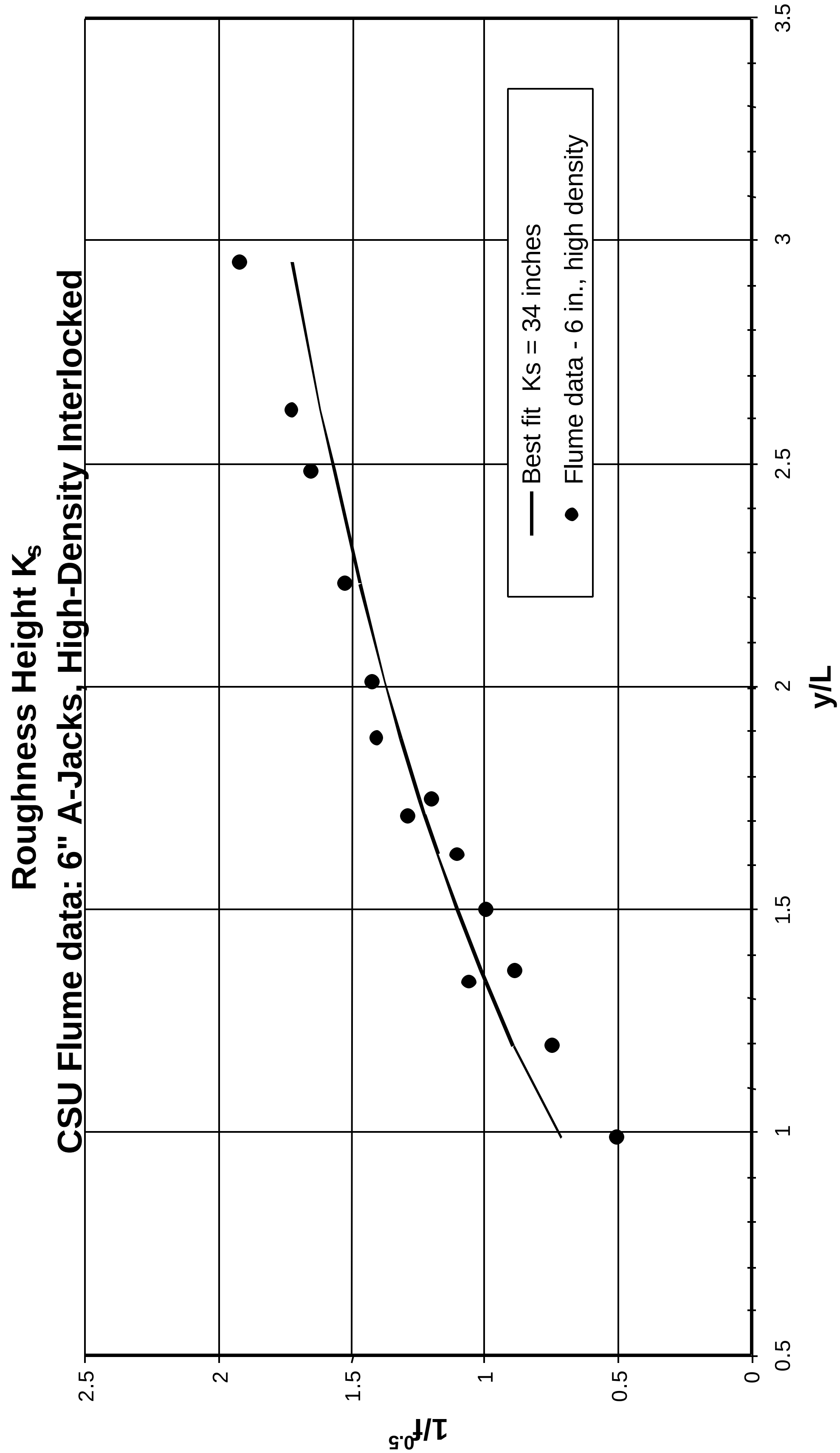


Figure 2.6. Roughness height K_s from 6-inch model tests.

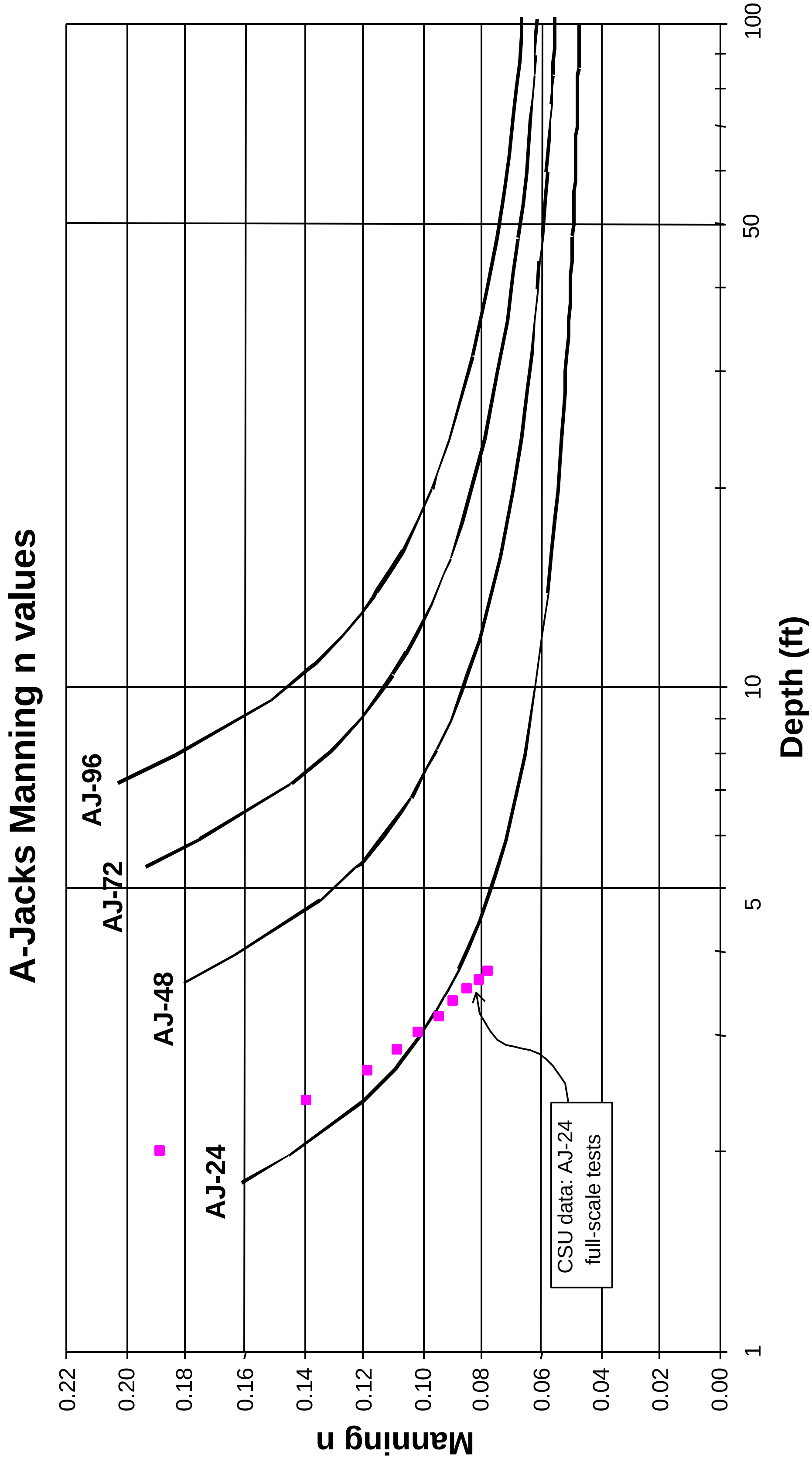


Figure 2.7. Recommended Manning's n values versus depth of flow.

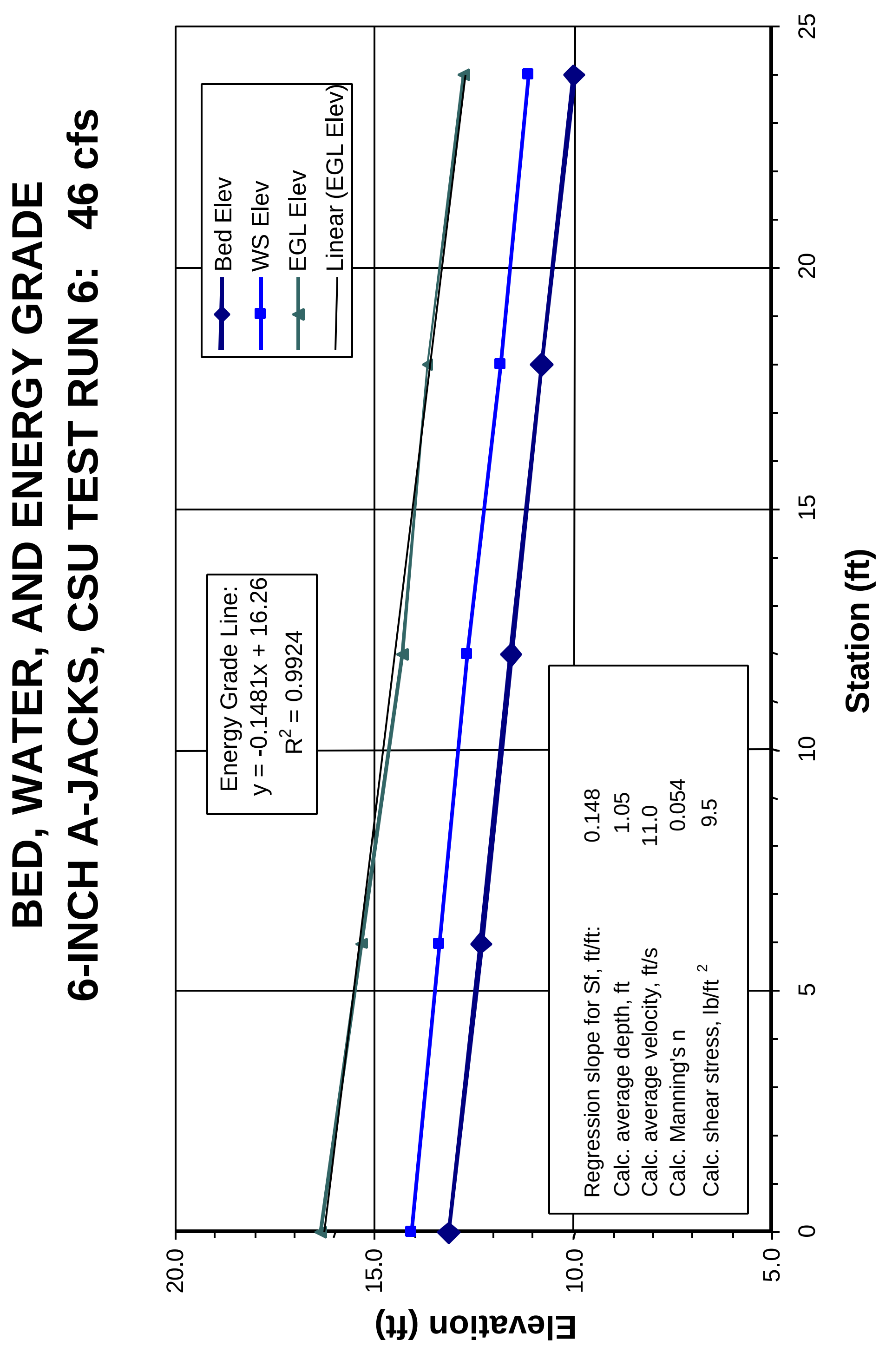


Figure 2.8. Hydraulic data from 6-inch model test (CSU Run No. 6).

Although no failures occurred during this series of tests, the hydraulic conditions associated with Test Run 6 at facility capacity are conservatively assumed to correspond to threshold values for purposes of estimating hydraulic stability. Froude-law scaling provides a means of extrapolating the design values of shear stress and velocity from the 6-inch model scale to the prototype units, as summarized in **Table 2.2**. In section 2.3, these values are used to develop design curves to extend the application of A-JACKS to a wide range of bed slope and side slope combinations.

Table 2.2. Scaled Values of Shear Stress and Velocity for A-JACKS Systems at 13 Percent Slope.

A-JACKS System	Observed Shear Stress (lb/ft^2)	Observed Velocity (ft/s)	Calculated Design Shear Stress (lb/ft^2)	Calculated Design Velocity (ft/s)
6-inch model	9.5	11.0	-	-
AJ - 24	-	-	38	22.0
AJ - 48	-	-	76	31.1
AJ - 72	-	-	114	38.1
AJ - 96	-	-	152	44.0

Note: Values shown correspond to the high-density interlocking configuration.

2.3 A-JACKS Design Procedure for Bed and Bank Protection

2.3.1 General

This chapter describes the procedure for using A-JACKS armor units for establishing bed and bank stability in open channels. Typical applications include rivers, spillways, sloping drop structures, and stilling basins. Included in this chapter are examples of this procedure using design charts developed from the scale model testing program at Colorado State University. **Figures 2.9 and 2.10** present recommended limiting values of shear stress and velocity, respectively, as a function of bed slope for the various sizes of A-JACKS armor units. **Figure 2.11** presents correction factors which are used to adjust these values when the A-JACKS are placed on the side slope of a channel. All charts reflect a factor of safety of 1.0.

2.3.2 Use of Design Charts and the Factor of Safety Method

This section provides the basic procedure for designing with A-JACKS armor units and worked examples. The two examples present a direct use of the design charts for bed and bank protection, respectively. In general, the design procedure for the A-JACKS armor units in open channel flow consists of the following steps:

1. Determine the depth y_0 and velocity of flow V for the design event using standard hydraulic analysis techniques.

2. Calculate the maximum bed shear stress τ_0, including the effect of channel bends, if any:

$$\tau_0 = K_b \gamma y_0 S_f$$

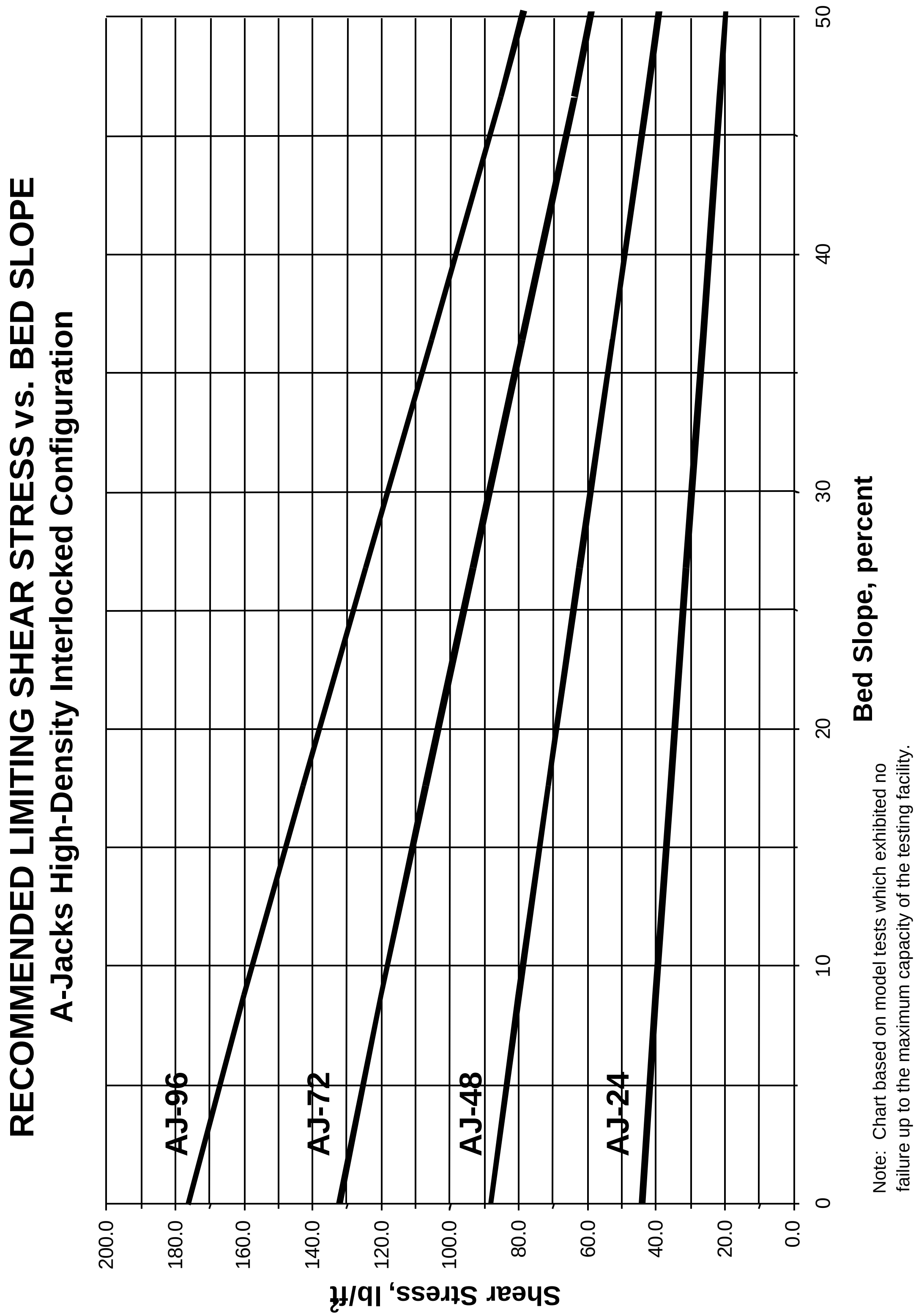


Figure 2.9. Recommended limiting values of shear stress versus bed slope.

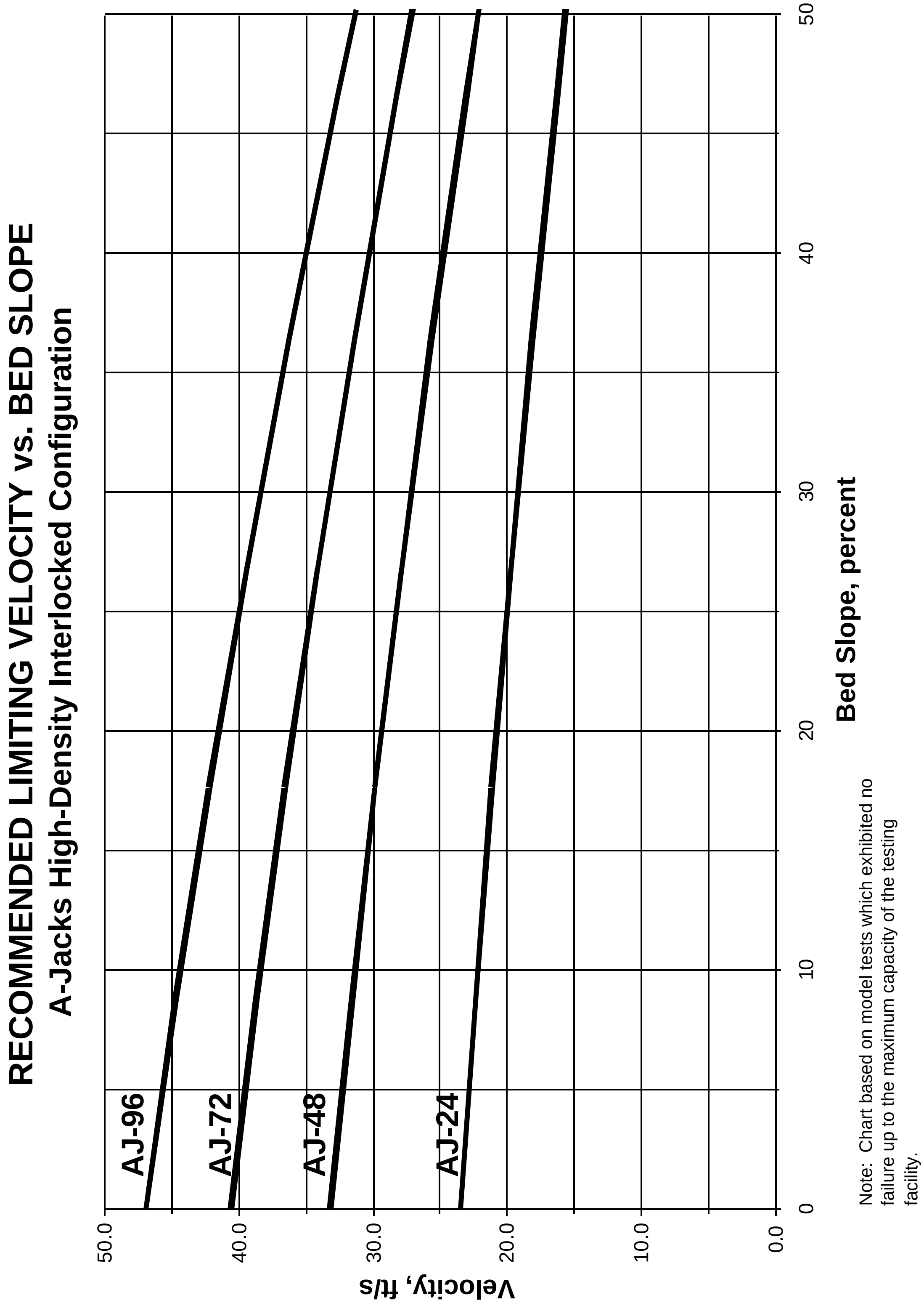


Figure 2.10. Recommended limiting values of velocity versus bed slope.

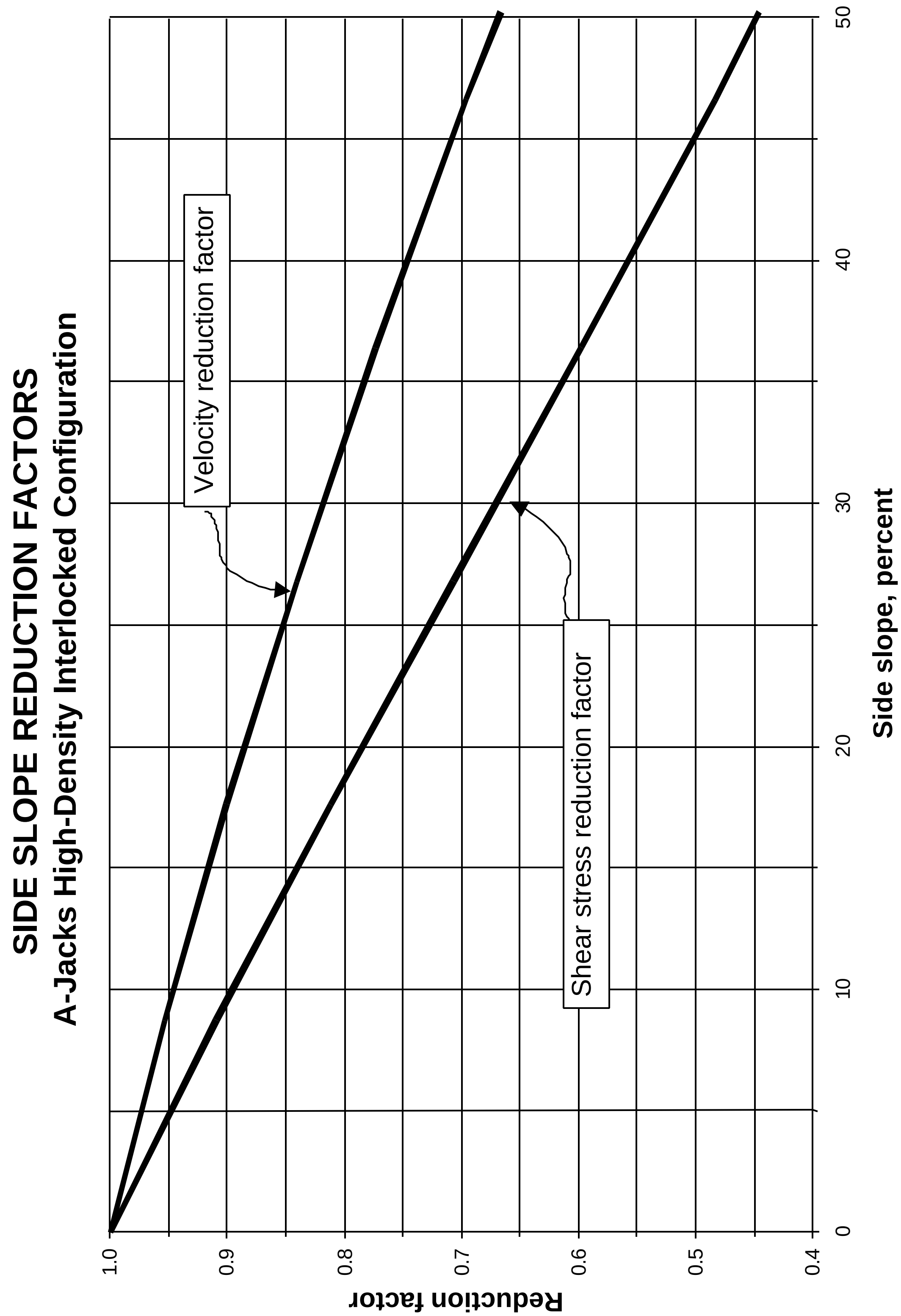


Figure 2.11. Side slope reduction factors for channel bank applications.

where:

K_b	=	bend coefficient for shear stress (1.0 for straight channels)
γ	=	unit weight of water, 62.4 lb/ft^3
y_0	=	depth of flow, ft
S_f	=	slope of energy grade line, ft/ft

3. Determine the limiting values of shear stress and velocity for the desired A-JACKS system from Figures 2.9 and 2.10, respectively, corresponding to the channel bed slope.

4. Multiply the limiting values of shear stress and velocity from Step 3 by the side slope reduction factors found in Figure 2.11, corresponding to the channel side slope.

5. Calculate the safety factors for shear stress and velocity as the ratio of limiting design values to actual (project-specific) values as shown below:

Shear stress safety factor	$SF_\tau = \tau_p(K_t)/\tau_0$
Velocity safety factor	$SF_V = V_p(K_v)/V$

where:

τ_p and V_p are limiting values of shear stress and velocity from Figures 2.9 and 2.10, respectively

K_t and K_v are side slope correction factors from Figure 2.11

Example 1:

Given: Sloping drop structure on the channel bed is to be formed of A-JACKS. Channel side slopes are lined with cast-in-place concrete.

100-year design discharge:	Q = 500 ft^3/s
Slope of drop structure:	S_0 = 0.10 ft/ft (10 percent)
Channel bottom width:	b = 20 ft
Channel side slope:	Z = 2H:1V
Required factor of safety:	F.S. = 2.0

Solution:

Step 1

The determination of hydraulic conditions will be iterative, since the Manning n value for A-JACKS is highly dependent on depth. The solution procedure will evaluate the AJ-24 armor units (24-inch length, 78 pounds per unit) and determine their factor of safety under the given conditions.

Trial 1:
a. Assume depth of flow is 2 ft
b. Determine Manning n of 0.14 for AJ-24
c. Enter nomograph of Fig. 2.1 with Qn = 500 x 0.14 = 70
d. From nomograph determine y_0/b = 0.16
e. Calculate y_0 = 0.16 x bottom width = 0.16 x 20 = 3.2 feet

Since the computed depth is greater than the depth we initially assumed, we will have to make a second trial with a greater depth.

Trial 2:

a. Assume depth of flow is 2.75 ft
b. From Figure 2.1, determine Manning n of 0.105 for AJ-24
c. Enter nomograph of Fig. 2.1 with Qn = 500 x 0.105 = 52.5
d. From nomograph determine $y_0/b = 0.14$
e. Calculate $y_0 = 0.14 \times$ bottom width $= 0.14 \times 20 = 2.8$ feet

The calculated value of flow depth, 2.8 feet, is close enough to the assumed value of 2.75 feet to allow continuation to the next step.

<u>Step 2</u>

Determine velocity and bed shear stress (including bend correction):

a. Velocity: $V = Q/A = Q/y_0(b+Zy_0) = (500\ ft^3/s) / (71.7\ ft^2) = 7.0$ ft/s

b. Bend coefficient: No bend in channel section, so $K_b = 1.0$

c. Maximum shear stress: $\tau_0 = K_b \gamma y_0 S_f = 1.0 \times 62.4\ lb/ft^3 \times 2.8\ ft \times 0.10$ ft/ft

$\tau_0 = 17.5\ lb/ft^2$

<u>Step 3</u>

Determine the suitability of A-JACKS AJ-24 armor units:

Enter the charts, Figures 2.9 and 2.10, at a bed slope of 0.10 ft/ft (10 percent). Determine the limiting values of shear stress and velocity for AJ-24 armor units on the channel bed:

τ_p (bed) = 40 lb/ft^2
V_p (bed) = 22 ft/s

Since the A-JACKS will not be placed on the side slopes, no side slope corrections need to be made.

<u>Step 4</u>

Determine the safety factors associated with shear stress and velocity:

F.S. (shear stress) = $(\tau_p) / (\tau_{actual})$ = $(40\ lb/ft^2) / (17.5\ lb/ft^2)$ = 2.3
F.S. (velocity) = $(V_p) / (V_{actual})$ = (22 ft/s) / (7.0 ft/s) = 3.1

Conclude that safety factors for both shear and velocity criteria exceed the required value of 2.0 for this particular application, using A-JACKS AJ-24 armor units.

Step 5

Summarize results for the A-JACKS AJ-24 system:

100-year discharge Q, ft^3/s	500
Bed slope, percent	10
Bottom width b, ft	20
Side slope Z (not lined with A-JACKS)	2H:1V
Manning's n	0.105
Depth y, ft	2.8
Velocity V, ft/s	7.0
Shear stress τ_0, lb/ft^2	17.5
Safety factor (shear stress)	2.3
Safety factor (velocity)	3.1

Example 2:

Given: A wide natural channel is to have its banks stabilized with A-JACKS. The channel bed is to be left as a soft-bottom channel with an n-value of 0.035. The channel has a sweeping bend with a radius of curvature R_c of 200 feet.

100-year design discharge:	Q	= 2,800 ft^3/s
Slope of drop structure:	S_0	= 0.012 ft/ft (1.2 percent)
Channel bottom width:	b	= 40 ft
Channel side slope:	Z	= 4H:1V
Required factor of safety:	F.S.	= 1.5

Solution:

Step 1

The determination of hydraulic conditions will be straightforward, since the roughness of the A-JACKS along the banks will not appreciably affect the flow conditions of the soft-bottom channel. The solution procedure will evaluate the AJ-24 armor units (24-inch length, 78 pounds per unit) and determine their factor of safety under the given conditions.

a. Enter nomograph of Fig. 2.1 with Qn = 2,800 x 0.035 = 98
b. From nomograph determine $y_0/b = 0.12$
c. Calculate y_0 = 0.12 x bottom width = 0.12 x 40 = 4.8 feet

Step 2

Determine velocity and bed shear stress (including bend correction):

a. Velocity: $V = Q/A = Q/y_0(b+Zy_0) = (2{,}800\ ft^3/s) / (284\ ft^2) = 9.9\ ft/s$

b. Bend coefficient: From Figure 2.3 with $R_c/b = 200/40 = 5$, find $K_b = 1.55$

c. Maximum shear stress: $\tau_0 = K_b \gamma y_0 S_f = 1.55 \times 62.4\ lb/ft^3 \times 4.8\ ft \times 0.012\ ft/ft$

$\tau_0 = 5.6\ lb/ft^2$

$$\tau_0 = 5.6 \text{ lb/ft}^2$$

Step 3

Determine the suitability of A-JACKS AJ-24 armor units:

Enter the charts, Figures 2.9 and 2.10, at a bed slope of 0.012 ft/ft (1.2 percent). Determine the limiting values of shear stress and velocity for AJ-24 armor units on the channel bed as:

τ_p (bed) = 44 lb/ft^2
V_p (bed) = 23.5 ft/s

Enter the side slope correction chart at a side slope of 4H:1V (25 percent) and find reduction factors of 0.725 for shear stress and 0.85 for velocity. Multiply the limiting values of shear stress and velocity for the bed by these factors to find the corresponding design values for A-JACKS placed on the side slopes.

τ_p (side slope) = 44 lb/ft^2 x 0.725 = 31.9 lb/ft^2
V_p (side slope) = 23.5 ft/s x 0.85 = 20.0 ft/s

Step 4

Determine the safety factors associated with shear stress and velocity:

F.S. (shear stress) = $(\tau_p) / (\tau_{actual})$ = (31.9 lb/ft^2) / (5.6 lb/ft^2) = 5.7
F.S. (velocity) = $(V_p) / (V_{actual})$ = (20.0 ft/s) / (9.9 ft/s) = 2.0

Conclude that safety factors for both shear and velocity criteria exceed the required value of 1.5 for this particular application, using A-JACKS AJ-24 armor units.

Step 5

Summarize results for the A-JACKS AJ-24 system:

100-year discharge Q, ft^3/s	2,800
Bed slope, percent	1.2
Bottom width b, ft	40
Side slope Z (lined with A-JACKS)	4H:1V
Manning'[s n	0.035
Depth y, ft	4.8
Velocity V, ft/s	9.9
Shear stress τ_0, lb/ft^2	5.6
Safety factor (shear stress)	5.7
Safety factor (velocity)	2.0

Note: On side slope installations, it is always prudent to toe down the A-JACKS armor units into the soft-bottom stream bed to a depth equal to the maximum anticipated depth of scour. This will prevent undermining of the armor units at the toe.

2.3.3 Bedding Considerations

When using A-JACKS on channel beds or banks where the native soil is fine enough to be pulled through the voids between the armor units, the system should be placed on a geotextile or granular bedding material which has been selected for compatibility with the native soils. The bedding must retain the soil particles of the subgrade while allowing water to freely infiltrate and exfiltrate through the system for the entire service life of the structure under the site-specific design gradients anticipated. When using a geotextile, the apparent opening size (AOS) and its permeability are therefore the primary design characteristics considered in selecting the appropriate fabric for compatibility with the soil subgrade. Additionally, protection against long-term potential for clogging involves the fabric's percent open area (woven geotextiles) or porosity (nonwoven geotextiles).

The bedding stone or geotextile filter should be selected based on the site-specific soil characteristics, physical boundary conditions, and hydraulic conditions. For erosion protection using A-JACKS armor units, the bedding layer typically governs the passage of water through the protection system, since the block system itself maintains a relatively large drainage area as a result of voids between the legs of the individual armor units. The purpose of the bedding layer is to prevent soil loss due to piping or washout through the armor units, while preventing excessive hydraulic uplift pressures from developing beneath the system. In some cases, select imported granular fill can be used in conjunction with a geotextile. This section does not apply to other types of geosynthetic soil erosion control or stabilizing systems, such as erosion control revegetation mats, turf reinforcement mats, or geogrids.

If placement of a geotextile below the waterline is anticipated, a bedding layer of suitably-sized crushed rock, large gravel, or small cobbles may be placed directly on top of the geotextile prior to installing the block system. The bedding layer serves both as ballast for the geotextile and as a means of achieving closer tolerances to the design lines and grades, particularly when an irregular surface exists.

Appropriate geotextile strength requirements, corresponding to installation survivability considerations, are recommended in **Table 2.3**. In general, design guidelines recommend that the geotextile exhibit a permeability at least 10 times greater than the underlying subsoil. In addition, a typical requirement is that the Apparent Opening Size (AOS) of the geotextile should be selected to retain at least 30 percent, but not more than 70 percent, of the particle sizes present. However, defining specific geotextile selection and design methods, and the resulting property values for geotextiles in this application, is beyond the scope of this manual. Other sources of information and guidance for this application have been developed which are referenced below:

1. TC Mirafi, 1998. "Geotextile Filter Design Manual." Prepared by Geosyntec Consultants, Norcross, Georgia.

2. Industrial Fabrics Association, 1990. "A Design Primer: Geotextiles and Related Materials." St. Paul, Minnesota.

3. American Association of State Highway Transportation Officials (AASHTO), 1995. "Standard Specification for Geotextiles." AASHTO Designation M 288-96 DRAFT. February.

Table 2.3. Recommended Geotextile Strength Requirements.

	Test Methods	Units	Class 1		Class 2		Class 3	
			Elongation <50%[2]	Elongation >50%[2]	Elongation <50%[2]	Elongation >50%[2]	Elongation <50%[2]	Elongation >50%[2]
Grab Strength	ASTM D 4632	N	1400	900	1100	700	800	500
Sewn Seam Strength [3]	ASTM D 4632	N	1200	810	990	630	720	450
Tear Strength	ASTM D 4533	N	500	350	400[4]	250	300	180
Puncture Strength	ASTM D 4833	N	500	350	400	250	300	180
Burst Strength	ASTM D 3786	N	3500	1700	2700	1300	2100	950
Permittivity	ASTM D 4491	sec^{-1}	Property requirements for permittivity, AOS and UV stability are based on geotextile application and site-specific soil characteristics. Refer to Section 3.3 for geotextile selection for permanent erosion control design using ACB systems.					
Apparent Opening Size	ASTM D 4751	mm						
Ultraviolet Stability	ASTM D 4355	%						

Notes:

1. Required geotextile class for permanent erosion control design is designated below for the indicated application. The severity of installation conditions generally dictates the required geotextile class.

   Class 1 is recommended for harsh or severe installation conditions where there is a greater potential for geotextile damage, including irregular sections where repeated armor unit realignment and replacing is expected, or when vehicular traffic on the installation is anticipated.

   Class 2 is recommended for installation conditions where placement in regular, even reaches is expected and little or no vehicular traffic on the installation will occur, or when hand-placing on a graded surface of native soils.

   Class 3 is specified for the least severe installation environments, typically hand-placed systems (zero drop height) on a bedding layer of graded sand, road base aggregate, or other select imported material.

2. As measured in accordance with ASTM D-4632.

3. When seams are required.

4. The required Minimum Average Roll Value (MARV) tear strength for woven monofilament geotextiles is 250 N.

2.4 A-JACKS Installation Guidelines

Recommended guidelines for the installation of A-JACKS armor units in channel bed and bank applications are provided in the following sections and accompanying figures.

2.4.1 Subgrade Preparation

Subgrade soil should be prepared to the lines, grades, and cross sections shown on the contract drawings. Termination trenches and transitions between slopes or slopes and embankment crests, benches, berms, and toes should be shaped and uniformly graded to facilitate the development of intimate contact between the A-JACKS system and the underlying grade.

Subgrade soil should be approved by the Engineer to confirm that the actual subgrade soil conditions meet or exceed the required material standards and conform to the design calculations and assumptions. Soils not meeting the required standards should be removed and replaced with acceptable material.

Care should be exercised so as not to excavate below the grades shown on the contract drawings, unless directed by the Engineer to remove unsatisfactory materials, and any excessive excavation should be filled with approved backfill material and compacted. Where it is impractical, in the opinion of the Engineer, to dewater the area to be filled, over-excavations should be backfilled with crushed rock or stone conforming to the grading and quality requirements of 19 mm (3/4 inch) maximum size coarse aggregate for concrete.

The areas above the waterline which are to receive the A-JACKS system should be graded to a smooth surface to ensure that intimate contact is achieved between the subgrade surface and the bedding layer (granular materials and/or geotextile), and between the bedding layer and the bottom surface of the A-JACKS system. Unsatisfactory soils and soils having a natural in-place moisture content in excess of 40 percent, and soils containing roots, sod, brush, or other organic materials, should be removed, backfilled with select material, and compacted. The subgrade should be uniformly compacted to a minimum of 95 percent of Standard Proctor density (ASTM D-698). Should the subgrade surface for any reason become rough, corrugated, uneven, textured, or traffic marked to the extent that voids beneath the armor system are created, such unsatisfactory portion should be scarified, reworked, recompacted, or replaced as directed by the Engineer.

Excavation of the subgrade, above the water line, should not be more than 100 mm (4 inches) below the grade indicated on the contract drawings. Excavation of the subgrade below the water line should not be more than 200 mm (8 inches) below the grade indicated on the contract drawings. Where such areas are below the allowable grades, they should be brought to grade by placing thin layers of select material and compacted. Where such areas are above the allowable grades they should be brought to grade by removing material or reworking existing material and compacting as directed by the Engineer.
Immediately prior to placing the bedding and A-JACKS system, the prepared subgrade should be inspected and approved by the Engineer.

2.4.2 Placement of Geotextile

When a geotextile underlayer is used, the geotextile should be placed directly on the prepared area, in intimate contact with the subgrade, and free of folds or wrinkles. The geotextile shall be placed in such a manner that placement of the overlying A-JACKS system will not excessively stretch or tear the geotextile. After geotextile placement, the work area should not be disturbed so as to result is a loss of intimate contact between the armor units and the geotextile, or between the geotextile and the subgrade. The geotextile should not be left exposed longer than the manufacturer's recommendation to minimize damage potential due to ultraviolet radiation.

The geotextile should be placed so that the upstream strips of fabric overlap downstream strips, and so that upslope strips overlap downslope strips. Overlaps should be in the direction of flow wherever possible. The longitudinal and transverse joints should be overlapped at least 1m (3 ft.) for below-water installations, and at least half that amount for dry installations. If a sewn seam is to be used for the seaming of the geotextile, the thread used shall consist of high strength polypropylene or polyester and shall be resistant to ultraviolet radiation. The geotextile should extend at least 0.3m (1 ft.) beyond the top, toe, and side termination points of the revetment. If necessary to expedite construction and to maintain the recommended overlaps, 450 mm (18 in.) anchoring pins and/or 11gauge, 6"x1" U-staples may be used.

2.4.3 Placement of the A-JACKS System

Individual A-JACKS armor units should be assembled with an epoxy-based grout in accordance with the manufacturer's specifications. **Figure 2.12** provides a recommended A-JACKS assembly detail. The armor units should be placed on the bedding layer (granular materials and/or geotextile) in such a manner as to produce a densely-interlocked matrix in intimate contact with the geotextile. Care shall be taken during installation so as to avoid damage to the geotextile or armor units during the installation process. System placement shall begin at the toe termination trench and proceed up the slope.

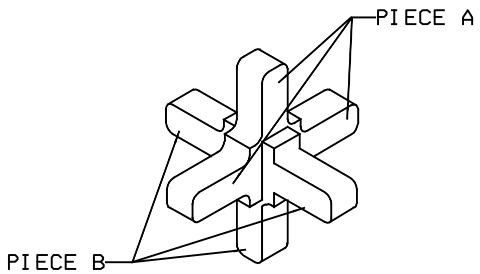
2.4.4 Finishing

Sediment excavated from the toe or termination trenches should be used to fill the voids of the installed A-JACKS system. Trenches should be backfilled and compacted flush with the top of the system. The integrity of a soil trench backfill must be maintained so as to ensure a surface that is flush with the top surface of the system for its entire service life. Top, toe and side termination trenches should be backfilled with suitable material and compacted immediately after the system has been placed. Upper banks may be vegetated, often in combination with lightweight erosion control materials or engineered biostabilization materials suitable to the area. **Figures 2.13 and 2.14** provide typical installation and finishing details for the A-JACKS system.

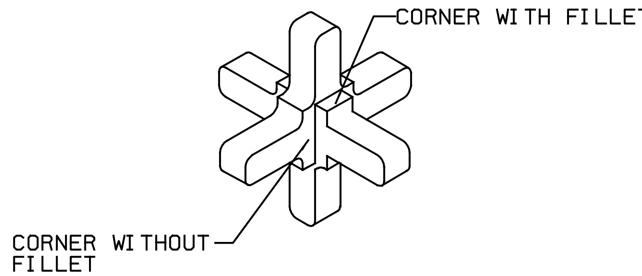
2.4.5 Inspection

The subgrade preparation, bedding layer (granular materials or geotextile) placement and A-JACKS system installation, and overall finished condition including termination points, should be inspected and approved by the Engineer.

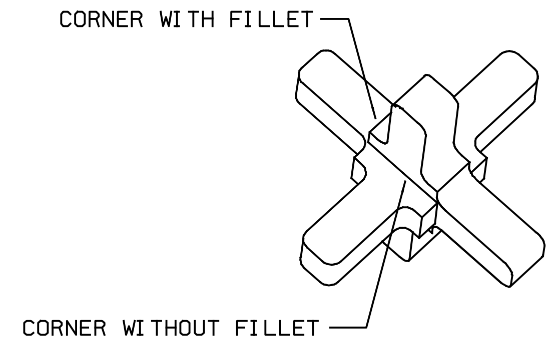
1. IDENTIFY A-JACKS COMPONENTS.



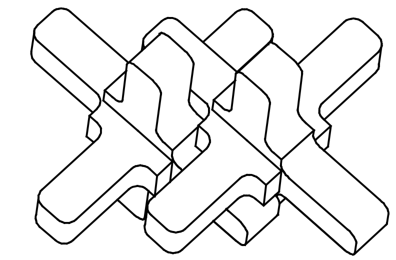
2. IDENTIFY CORNERS WITH AND WITHOUT FILLETS.



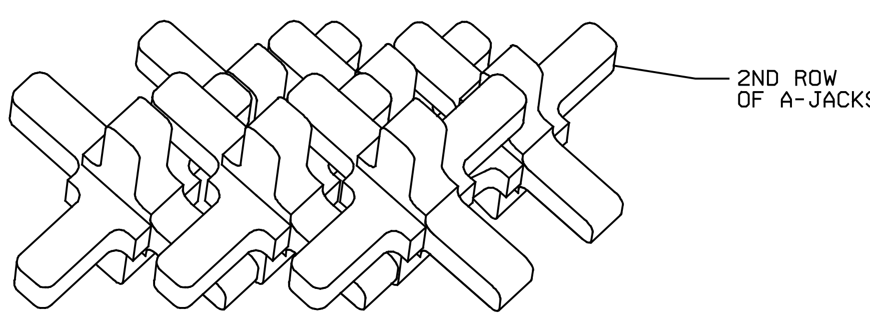
3. PROPER ROTATION OF A-JACKS.

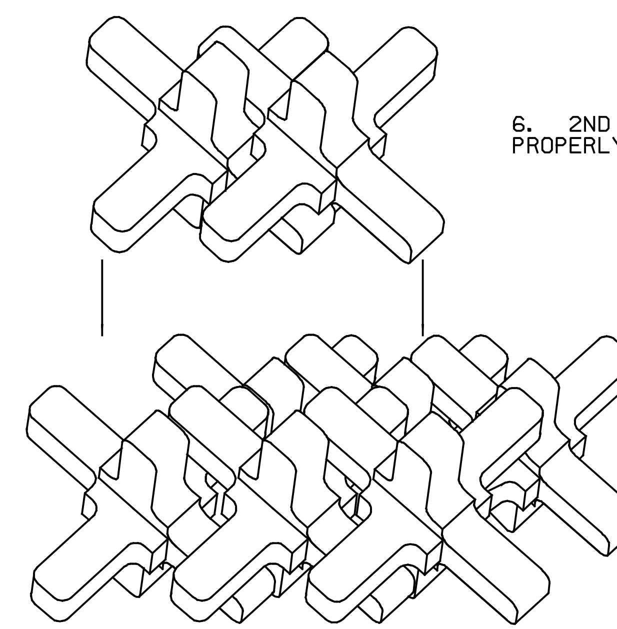


4. ALIGN ALL A-JACKS IN SAME DIRECTION. CORNERS WITHOUT FILLET MUST LINE UP.



5. INSTALL 2ND ROW OF A-JACKS USING SAME ALIGNMENT.





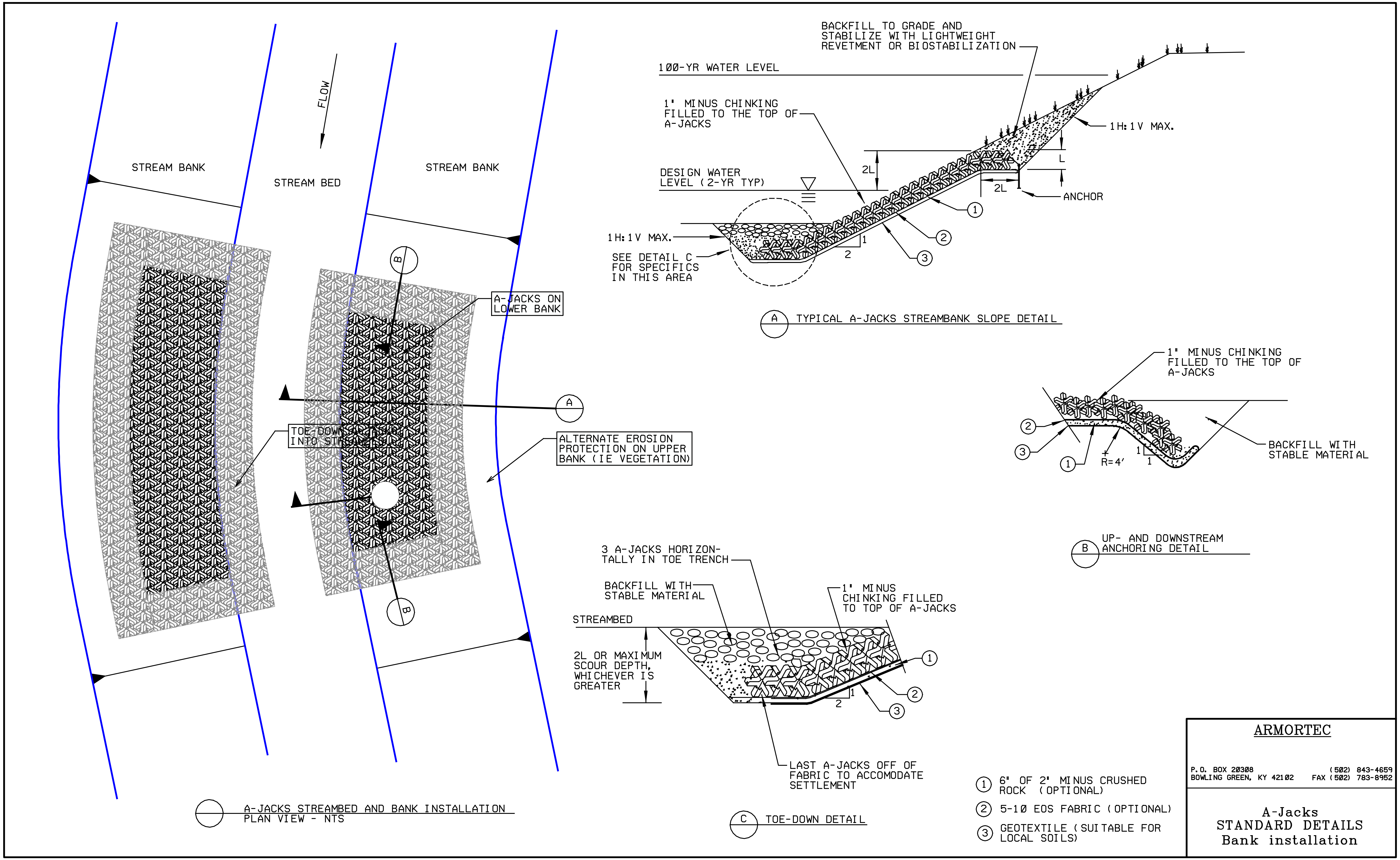
6. 2ND LEVEL OF A-JACKS WILL FIT WHEN PROPERLY ALIGNED WITH 1ST LEVEL.

ARMORTEC

P.O. BOX 20308 (502) 843-4659
BOWLING GREEN, KY 42102 FAX (502) 783-8952

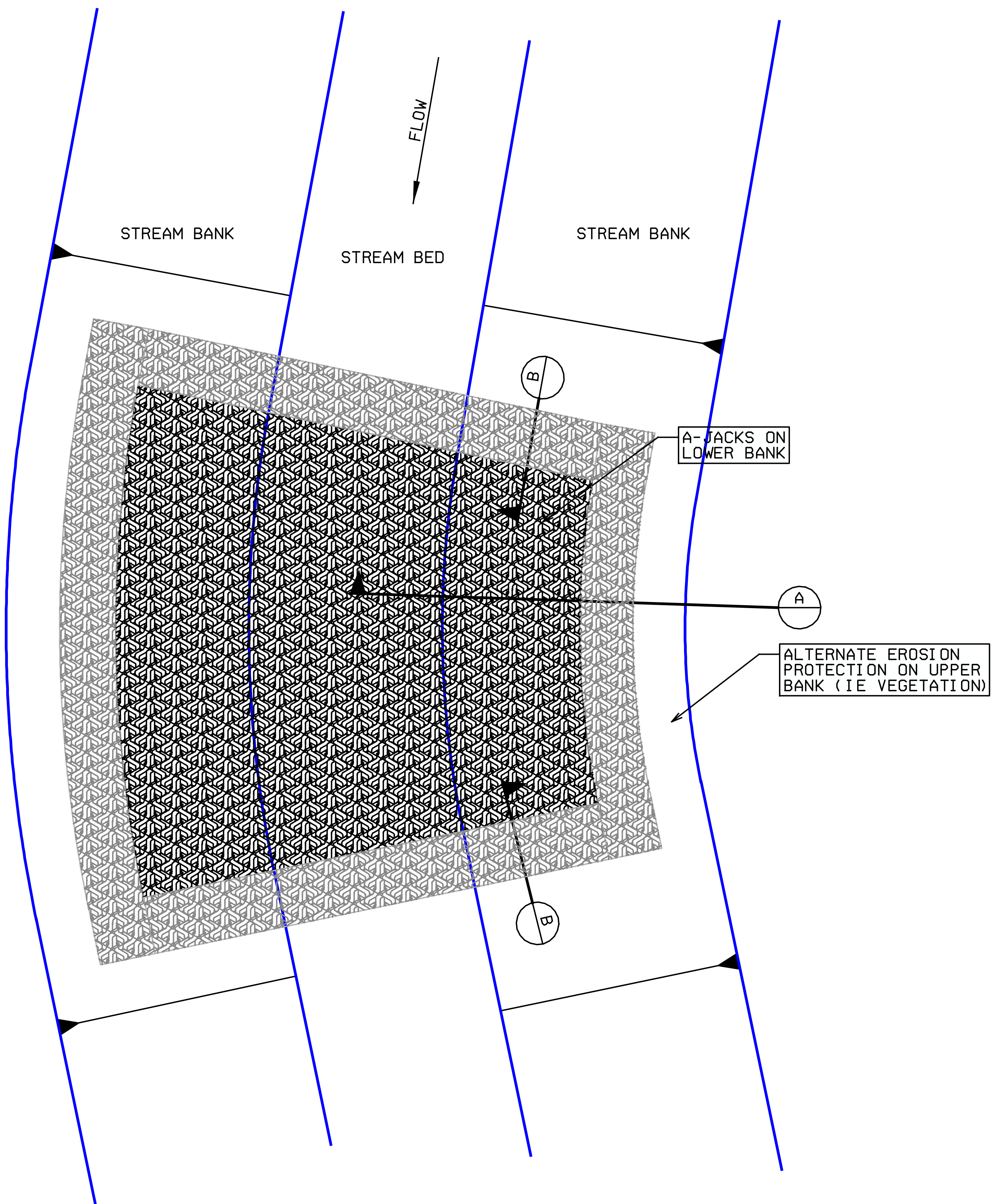
A-Jacks
STANDARD DETAILS
Configuration of A-Jacks for proper fit

Figure 2.12

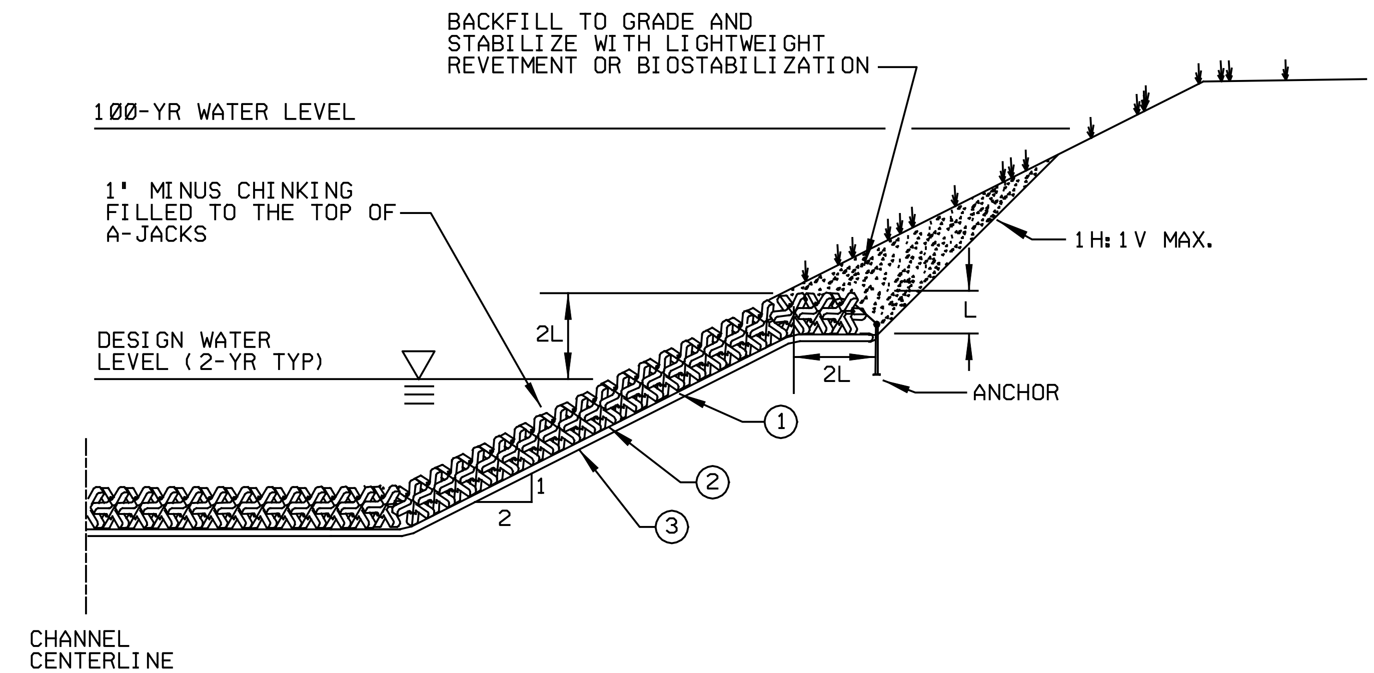


FLOW
STREAM BANK
STREAM BED
STREAM BANK
A-JACKS ON LOWER BANK
TOE-DOWN
INTO ST
ALTERNATE EROSION PROTECTION ON UPPER BANK (IE VEGETATION)
A
B
B
A-JACKS STREAMBED AND BANK INSTALLATION
PLAN VIEW - NTS
BACKFILL TO GRADE AND STABILIZE WITH LIGHTWEIGHT REVETMENT OR BIOSTABILIZATION
100-YR WATER LEVEL
1" MINUS CHINKING FILLED TO THE TOP OF A-JACKS
DESIGN WATER LEVEL (2-YR TYP)
2L
L
2L
1H:1V MAX.
ANCHOR
1H:1V MAX.
SEE DETAIL C FOR SPECIFICS IN THIS AREA
1
2
1
2
3
A
TYPICAL A-JACKS STREAMBANK SLOPE DETAIL
1" MINUS CHINKING FILLED TO THE TOP OF A-JACKS
BACKFILL WITH STABLE MATERIAL
R=4'
1
1
B
UP- AND DOWNSTREAM ANCHORING DETAIL
3 A-JACKS HORIZONTALLY IN TOE TRENCH
BACKFILL WITH STABLE MATERIAL
1" MINUS CHINKING FILLED TO TOP OF A-JACKS
STREAMBED
2L OR MAXIMUM SCOUR DEPTH, WHICHEVER IS GREATER
LAST A-JACKS OFF OF FABRIC TO ACCOMODATE SETTLEMENT
C
TOE-DOWN DETAIL
1 6" OF 2" MINUS CRUSHED ROCK (OPTIONAL)
2 5-10 EOS FABRIC (OPTIONAL)
3 GEOTEXTILE (SUITABLE FOR LOCAL SOILS)
ARMORTEC
P.O. BOX 20308
BOWLING GREEN, KY 42102
(502) 843-4659
FAX (502) 783-8952
A-Jacks
STANDARD DETAILS
Bank installation

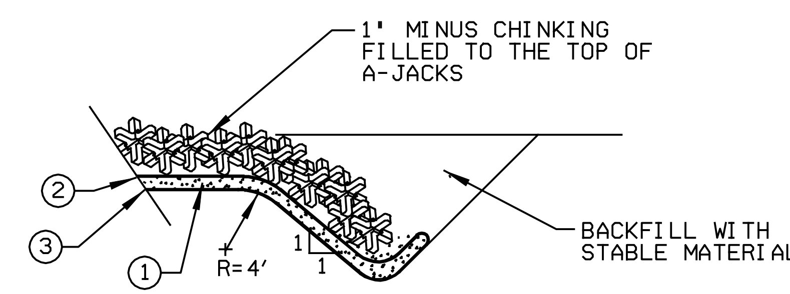
Figure 2.13



A-JACKS STREAMBED AND BANK INSTALLATION
PLAN VIEW - NTS



A TYPICAL A-JACKS STREAMBANK SLOPE DETAIL



B UP- AND DOWNSTREAM ANCHORING DETAIL

1. 6" OF 2" MINUS CRUSHED ROCK (OPTIONAL)
2. 5-10 EOS FABRIC (OPTIONAL)
3. GEOTEXTILE (SUITABLE FOR LOCAL SOILS)

ARMORTEC

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Bed & Bank installation

Figure 2.14

3. PIER SCOUR APPLICATIONS

The use of A-JACKS for pier scour applications requires an understanding of local scour processes. A review of the fundamentals is presented to provide the designer with the theory behind the development of design parameters. Specific design parameters for A-JACKS are presented to facilitate the selection of appropriate sizes of armor units for pier scour applications.

3.1 Mechanics of Pier Scour

To design adequate countermeasures for minimizing local scour at bridge piers, the physical forces causing the problem should be understood. The basic mechanism causing local scour at piers is the formation of vortices at their base. The action of the vortices removes bed material from around the pier nose at the base of the obstruction. The transport rate of sediment away from the vicinity of the base is greater than the transport rate into the region and consequently a scour hole develops (Richardson 1995). **Figure 3.1** illustrates the hydraulic vortices causing local scour at a pier.

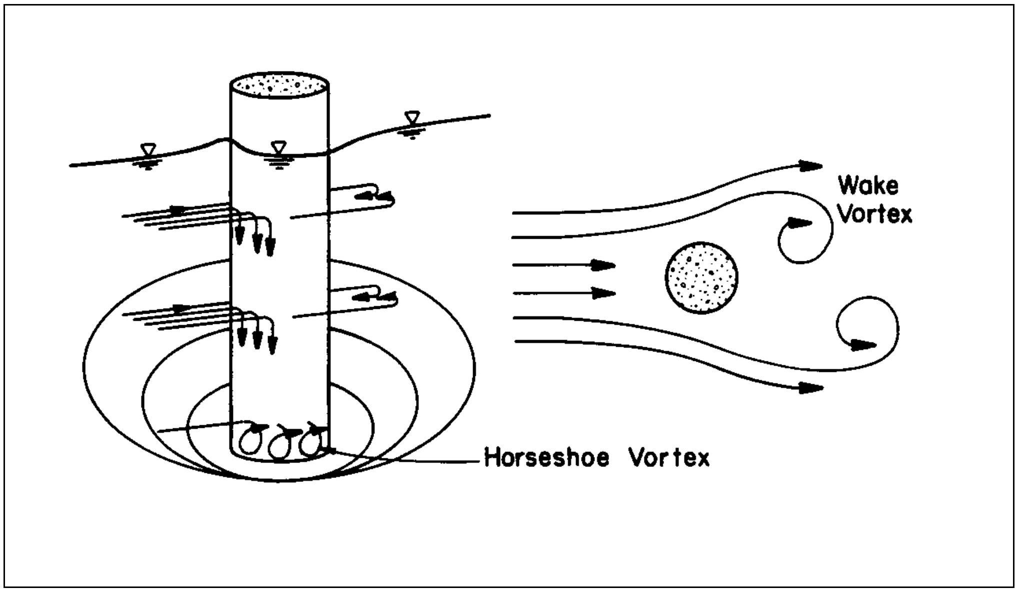

Figure 3.1. Schematic representation of scour at a Cylindrical Pier (Richardson and Davis 1995).

Pier scour countermeasures are generally placed in the disturbance zone created by the vortices described above. The lateral and horizontal extent of a scour hole depends on the hydraulic characteristics of the flow, the pier shape and size, and character of the bed material. **Figure 3.2** illustrates the area of the horseshoe vortex at a cylindrical pier.

Some of the factors that affect the magnitude of local scour at bridge piers are:

- velocity of the approach flow
- depth of flow
- geometric shape and size of the pier
- angle of attack
- size and gradation of the bed material

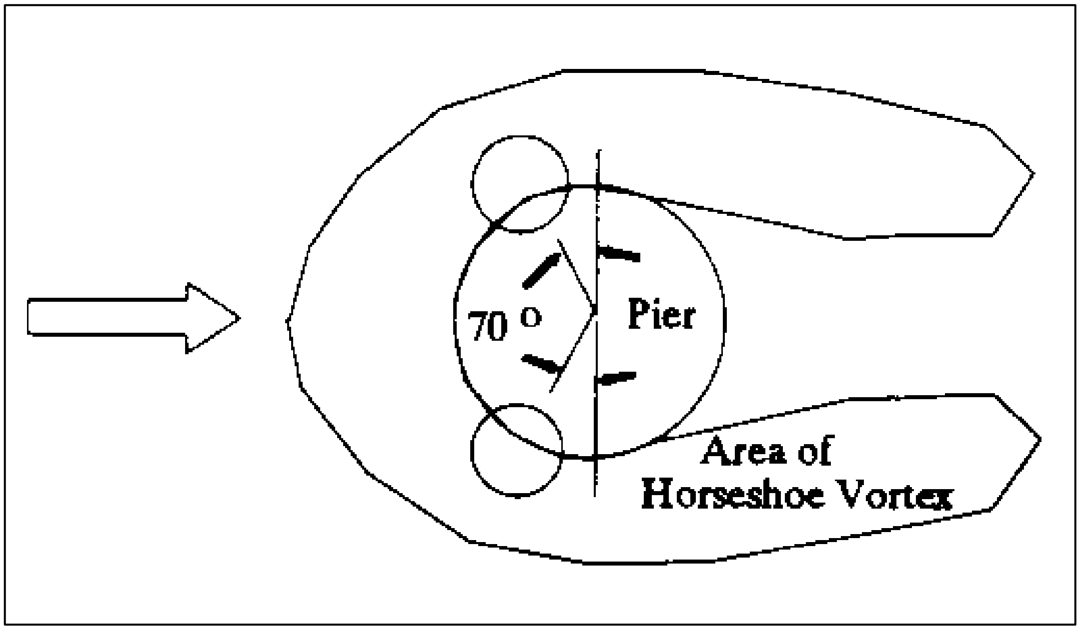

Figure 3.2. Schematic representation of the area of the Horseshoe Vortex (Fotherby 1995).

The magnitude of local scour at piers has been studied extensively in many laboratory studies. The Federal Highway Administration's Hydraulic Engineering Circular No. 18 (HEC-18) recommends the use of the CSU equation for predicting maximum pier scour depths. The CSU equation is:

$$\frac{y_s}{a} = 2.0 K_1 K_2 K_3 K_4 \left[\frac{y_1}{a}\right]^{0.35} Fr_1^{0.43} \tag{3.1}$$

where:

y_s	=	scour depth (ft or m)
K_1, K_2, K_3, K_4	=	correction factors for pier shape, angle of attack, bed condition, and armoring, respectively
a	=	pier width (ft or m)
y_1	=	flow depth directly upstream of pier (ft or m)
Fr	=	Froude number = $V_1/(gY_1)^{1/2}$
V_1	=	mean velocity of flow directly upstream of pier (ft/s or m/s)

The CSU equation will predict the maximum depth of local scour around a pier. The lateral dimensions of the scour hole are primarily dependent of the depth of scour and the bed material. HEC-18 provides a method for predicting the width of the scour hole in cohesionless bed material from one side of a pier with the following equation and as illustrated in **Figure 3.3**:

$$W = y_s(K + \cot q) \tag{3.2}$$

where:

W	=	top width of the scour hole from each side of the pier or footing (ft or m)
K	=	bottom width of scour hole as a fraction of scour depth (0 – 1.0)*y_s (ft or m)
θ	=	angle of repose of the bed material

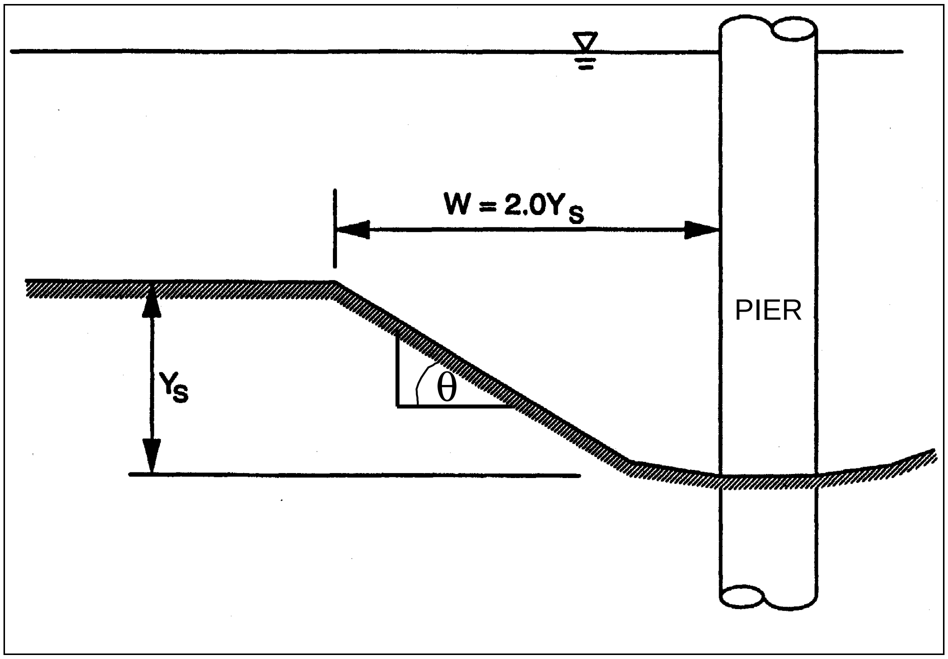

Figure 3.3. Schematic of Scour Hole Topwidth (Richardson and Davis 1995).

The prediction methods for scour hole geometry (depth and width) described above are commonly used to determine the extent of local pier scour. The next section will describe the principles used in designing armor units to protect piers from local scour.

3.2 Stability Paradigms

Extensive research has been conducted on methods for protecting piers from local scour. In lieu of structural modifications, armoring countermeasures have most commonly been used. The FHWA Hydraulic Engineering Circular No. 23 (HEC-23 1997) provides a summary of local scour armoring countermeasures (i.e., riprap, concrete armor units, articulated blocks, etc.) and design guidelines for installing selected countermeasures at piers. For the purpose of discussion, local scour armoring countermeasures will be referred to simply as armor units for the rest of this section. The criteria for selection of appropriate sizes of armor units are determined by their stability in flowing water. The most common stability paradigms are the Shields and Isbash frameworks. The Shield's criteria assumes that the required size of an armor unit for a particular flow condition is a function of shear stress and Shield's parameter:

$$D_u = \frac{\tau_c}{\rho_w g (S_g - 1) SP} \tag{3.3}$$

where:

D_u	=	equivalent spherical diameter of stone (ft or m)
τ_c	=	shear stress required to move a particle (lb/ft^2 or Pa)
ρ_w	=	density of water (slug/ft^3 or kg/m^3)
g	=	gravitational acceleration (ft/s^2 or m/s^2)
S_g	=	specific gravity of the particle
SP	=	Shields parameter which is a function of the flow conditions

The equivalent spherical diameter, D_u, represents diameter of a sphere with the same volume as the armor unit. When rock is used D_u represents the D_{50} of the material gradation. The difficulty in applying the Shield's relationship to local scour countermeasures is in determining a representative shear stress. Therefore, methods which incorporate velocity (a variable which can be directly measured) are more commonly used to select local scour countermeasures. The Isbash approach relates particle stability to velocity and a stability number, N:

$$D_u = \frac{V_c^{\ 2}}{g (S_g - 1) N} \tag{3.4}$$

where:

V	=	velocity acting on the stone required to initiate movement (ft/s or m/s)
N	=	stability number = $2E^2$ (1.5 for loose particles and 2.9 for seated particles)
E	=	Isbash's coefficient (0.86 for loose particles and 1.2 for seated particles)

The stability number N and Isbash coefficient E are indicative of the shape and interlocking characteristics of the particular armor units. The Isbash and Shields relations were developed for unobstructed flow conditions; therefore, these relationships require adjustments for application to flow fields at piers. Normally, the approach velocity is used with a correction factor for the acceleration around a pier. The HEC-18 equation for designing riprap at piers uses the Isbash equation with a stability number of 2.9 and a correction factor of 1.5 applied to the approach velocity for round nose piers. The assumption is that the velocity at the base of the pier is approximately 1.5 times the mean velocity of the approach velocity to the pier.

Investigators, including but not limited to Quazi and Peterson (1973), Neill (1975), Parola (1993), Bertoldi (1995), and Fotherby (1995) made contributions to the Isbash framework considering the effects of bridge piers on the stability of armor units. The later research considers the effect of pier width and flow depth on the stability number.

The foregoing developments have concentrated on the size, shape, and weight of individual armor units, whether randomly placed or in stacked or interlocked configurations. However, the basic construction element of A-JACKS for pier scour applications is a "module" comprised of 14 individual A-JACKS banded together in a densely-interlocked cluster, described as a 5x4x5 module. The banded module thus forms the individual design element. **Figures 3.4a and 3.4b** illustrate the concept.

In late 1998 and early 1999, a series of 54 tests of 6-inch model scale A-JACKS was conducted at Colorado State University to examine their effectiveness in pier scour applications. This program is described in detail in CSU's test report entitled, "Laboratory Testing of A-JACKS Units for Inland Applications: Pier Scour Protection Testing" (Thornton et al., February 1999).

The CSU tests were conducted in an 8-foot wide indoor flume with a sand bed, and examined a variety of conditions, including no protection (baseline conditions), banded 5x4x5 modules arrayed in several different configurations, and individual (unbanded) A-JACKS armor units. Both round and square piers were used in the program. The results indicated that, when used in combination with a bedding layer (either granular bedding stone or a properly selected geotextile), the A-JACKS 5x4x5 modules reduced scour at the pier from 70 percent to more than 95 percent (scour depths were from 30 percent to less than 5 percent of that in the unprotected baseline condition).

Hydraulic stability of a 5x4x5 A-JACKS module can be estimated by setting the overturning moment due to the total drag force equal to the resisting moment due to the submerged weight of the 5x4x5 module:

$$F_d H_d = W_s L_w \qquad (3.5)$$

where:

F_d	=	drag force, equal to $0.5C_d\rho AV^2$, lb
C_d	=	drag coefficient (dimensionless)
ρ	=	unit weight of water, 1.94 slugs/ft^3
A	=	frontal area of 5x4x5 A-JACKS module, ft^2
V	=	flow velocity immediately upstream of 5x4x5 A-JACKS module, ft/s
H_d	=	moment arm through which the drag force acts, ft
W_s	=	submerged weight of 5x4x5 A-JACKS module, lb
L_w	=	moment arm though which the submerged weight acts, ft

As a first estimate, the coefficient of drag C_d on a 5x4x5 A-JACKS module can be assumed to be similar to that of a disc oriented normal to the flow velocity, with flow occurring over the top and around the sides. This value is approximately 1.2 (Vennard and Street 1975). A conservative estimate for the location of the drag force would place it at the full height of the module, providing the greatest moment arm for overturning.

In July 1999, Ayres Associates conducted an additional series of 5 tests at CSU in a steep (13 percent slope), fixed-bed flume. The purpose of these tests was to determine the hydraulic stability of the 5x4x5 A-JACKS modules in a typical pier scour configuration. Discharge was gradually increased until overturning of the module was achieved. Both submerged and unsubmerged conditions were examined (**Figures 3.5a and 3.5b**).

Figure 3.4a. A-JACKS 5x4x5 modules in a typical pier scour installation.

Figure 3.4b. Frontal view showing 5x4x5 A-JACKS modules.

Figure 3.5a. Stability test of 5x4x5 A-JACKS modules, unsubmerged conditions. Note flow shedding around protected pier.

Figure 3.5b. Stability test of 5x4x5 A-JACKS modules, submerged conditions.

Hydraulic conditions measured at the threshold of overturning allows both the coefficient of drag, C_d, and the height of the drag force, H_d, to be determined directly from measured data. The other variables in Equation 3.5 are determined from the physical characteristics of the 5x4x5 A-JACKS module. **Table 3.1** provides a summary of the hydraulic analyses performed on data collected during the 5x4x5 A-JACKS module stability tests at threshold overturning conditions.

Table 3.1. Results of 6-Inch 5x4x5 Module A-JACKS Hydraulic Stability Tests.

Run Number	Hydraulic Condition	Velocity at Front of Module (ft/s)	Coefficient of Drag (C_d)	Height of Drag Force as Fraction of Total Module Height
1	Unsubmerged	11.9	0.54	0.66
2	Unsubmerged	11.7	0.68	0.76
Average of Unsubmerged Tests			**0.61**	**0.71**
3	Submerged	5.0	1.25	1.03
4	Submerged	6.0	0.86	1.03
5	Submerged	6.5	1.04	0.73
Average of Submerged Tests			**1.05**	**0.93**

From Table 3.1, it can be seen that, for purposes of developing a design method for 5x4x5 A-JACKS modules, the results of the tests under submerged conditions yield more conservative values for both the drag coefficient and the length of the overturning moment arm associated with the drag force.

Using a drag coefficient C_d of 1.05 for the 5x4x5 A-JACKS modules from Table 3.1, and assuming that the drag force acts at the full height the module (somewhat more conservative than the average value obtained from the submerged tests reported in Table 3.1), the hydraulic stability of prototype scale A-JACKS 5x4x5 modules can be determined. **Table 3.2** provides the results of this hydraulic stability analysis.

Table 3.2. Hydraulic Stability of Prototype Size 5x4x5 A-JACKS Modules.

5x4x5 A-JACKS Modules					Equivalent Rock Riprap		
A-JACKS system	Tip-to-tip length of armor unit (ft)	Module dimensions (H x W x L) (ft)	Weight in air (lbs)	Limiting velocity (ft/s)	Equivalent riprap D_{50} (ft)	Weight in air (one rock) (lbs)	Riprap blanket thickness (ft)
AJ-24	2.0	1.3 x 4.3 x 3.3	1,030	10.7	2.2	920	6.6
AJ-48	4.0	2.7 x 8.7 x 6.7	8,270	15.1	4.3	6,870	12.9
AJ-72	6.0	4.0 x 13.0 x 10.0	27,900	18.5	6.4	22,640	19.2
AJ-96	8.0	5.3 x 17.3 x 13.3	66,200	21.4	8.6	55,000	25.8

Notes:

1. Volume of concrete in ft^3 for a 14-unit module is 14 x 0.071 x L^3 where L is tip-to-tip dimension of armor unit in feet.
2. Values in table assume a unit weight of 130 lbs/ft^3 for concrete
3. Equivalent rock size from Isbash relationship (Hydraulic Engineering Circular No. 18, Equation 83, K = 1.7 for square-nosed pier)

3.3 Layout and Installation

3.3.1 Geometry

The movable-bed tests conducted at CSU indicate that a chevron-style A-JACKS placement around a bridge pier does not improve performance beyond that afforded by simple rectangular geometries. As the rectangular shape accommodates the basic 5x4x5 A-JACKS module design unit, this geometry provides the recommended style for layout and placement of the armor units. **Figure 3.6** provides recommended minimum dimensions for the placement of modules around a pier of width "a" and having an unprotected depth of scour y_s as determined by the CSU equation.

It should be noted that the CSU stability tests were conducted on a fully-exposed module; partial burial will result in a more stable installation. Also, the orientation of the modules in the stability tests exposed the maximum frontal profile to the flow (i.e., long axis perpendicular to the flow direction). Placement of the 5x4x5 modules with the long axis parallel to the flow will result in a more stable arrangement than indicated by the recommended values in Table 3.2.

3.3.2 A-JACKS Placement

A-JACKS modules can be constructed onsite in the dry and banded together in 5x4x5 clusters in place around the pier, after suitable bedding layers have been placed. Alternatively, the modules can be pre-assembled and installed with a crane and spreader bar; this arrangement may be more practical for placement in or under water.

Bands should be comprised of cables made of UV-stabilized polyester, galvanized steel, or stainless steel, as appropriate for the particular application. Crimps and stops should conform to manufacturer's specifications. When lifting the modules with a crane and spreader bar, all components of the banding arrangement should maintain a minimum factor of safety of 5.0 for lifting.

Where practicable, burial or infilling of the modules to half-height is recommended so that the voids between the legs are filled with appropriate sized stone. Stone sizing recommendations are provided in the next section.

3.3.3 Bedding Considerations

The movable-bed tests conducted at CSU indicate that a bedding layer of stone, geotextile fabric, or both, can appreciably enhance the performance of A-JACKS in terms of limiting the depth of scour at the pier nose. The purpose of a bedding layer is to retain the finer fraction of native bed material that could otherwise be pumped out between the legs of the A-JACKS armor units.

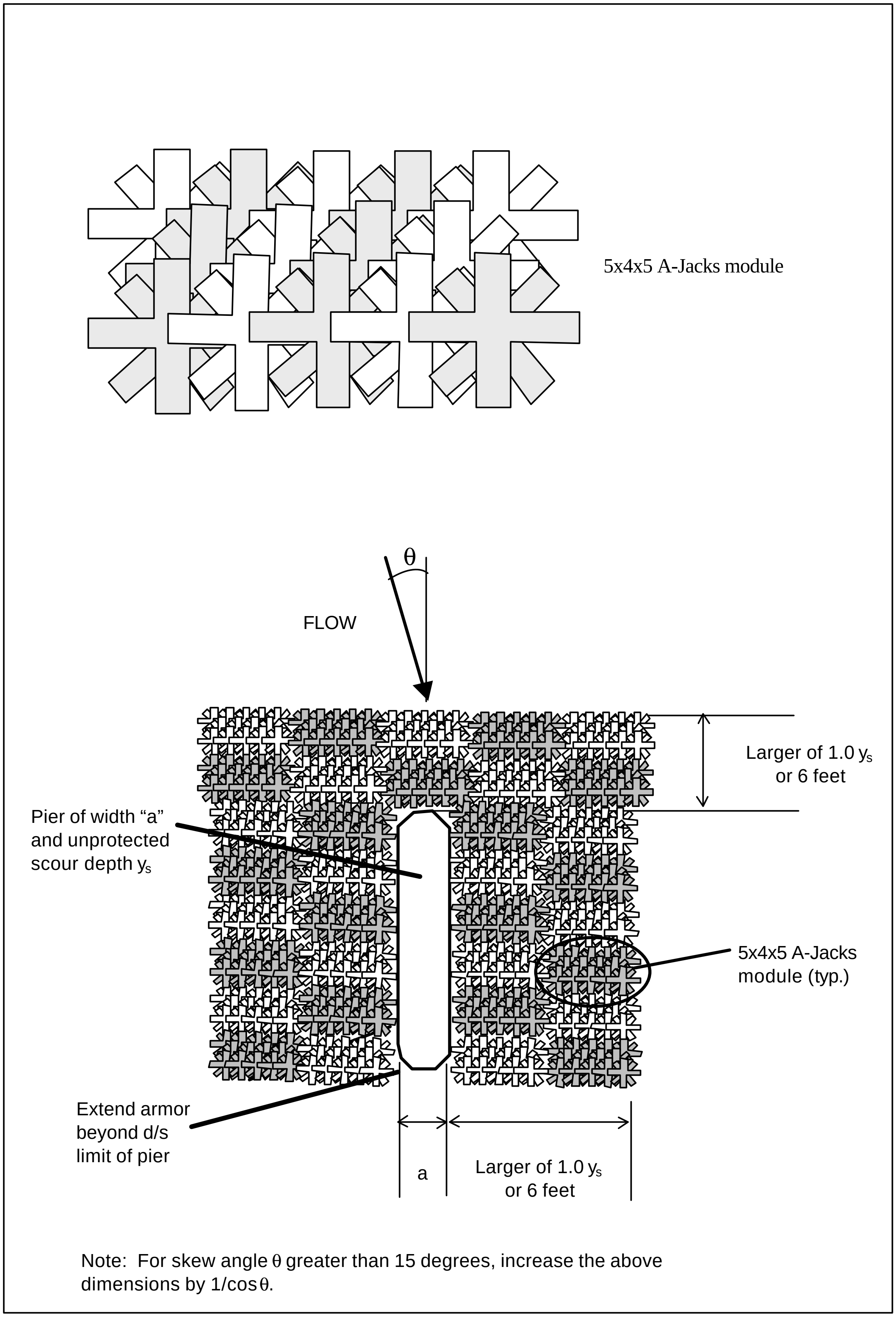

Figure 3.6. Placement of 5x4x5 A-JACKS modules for pier scour protection.

When bedding stone is placed directly on the streambed material at a pier, it must meet certain size and gradation requirements to ensure that it not only retains the bed material, but that it is permeable enough to relieve potential pore pressure buildup beneath the installation. In addition, the size of the bedding stone must be large enough to resist being plucked out through the legs of the A-JACKS by turbulent vortices and dynamic pressure fluctuations. In some cases, two or more individual layers of bedding stone, graded from finer in the lower layers to coarsest at the streambed, must be used to satisfy all the criteria.

Recommended sizing criteria for bedding stone (Escarameia 1998) are as follows:

Retention: $D_{85(Lower)} > 0.25D_{15(Upper)}$
$D_{50(Lower)} > 0.14D_{50(Upper)}$
Permeability: $D_{15(Lower)} > 0.14D_{15(Upper)}$
Uniformity: $D_{10(Upper)} > 0.10D_{60(Upper)}$

In the above relations, D_x is the particle size for which x percent by weight are finer, and the designations Upper and Lower denote the respective positions of various granular bedding layers in the case when multiple layers are used. Each layer should be at least 6 to 8 inches thick, with the exception of uppermost layer which should be thicker, in accordance with **Table 3.3**. Note that the lowest layer of the system corresponds to the native streambed material.

Table 3.3. Recommended Properties of Uppermost Layer of Bedding Stone for use with A-JACKS Armor Units.

A-JACKS System	D_{50} Size of Uppermost Layer (in.)	Recommended Minimum Thickness of Uppermost Layer (in.)
AJ-24	2-3	8
AJ-48	4-6	12
AJ-72	6-9	24
AJ-96	8-12	30

In lieu of multiple layers of granular bedding, it is often desirable to select a geotextile which is compatible with the native streambed material. However, placement of a geotextile may not always be practical, particularly when installing the system under flowing water. If a geotextile is used, it is recommended that a layer of ballast stone, with characteristics in accordance with Table 3.3, be placed on top prior to installing the A-JACKS modules.

When a geotextile is used, selection criteria typically require that the fabric exhibit a permeability at least 10 times that of the native streambed material to prevent uplift pressures from developing beneath the geotextile. In addition, the Apparent Opening Size (AOS) of the apertures of the geotextile should typically retain at least 30 percent, but not more than 70 percent, of the grain sizes present in the bed. Selected references for determining geotextile properties are provided in Chapter 2. Lastly, the geotextile must be strong enough to survive the stresses encountered during placement stone and armor units. Table 2.3 in Chapter 2 of this manual provides recommended guidelines for strength requirements.

Figures 3.7a and 3.7b illustrate the bedding options discussed in this section.

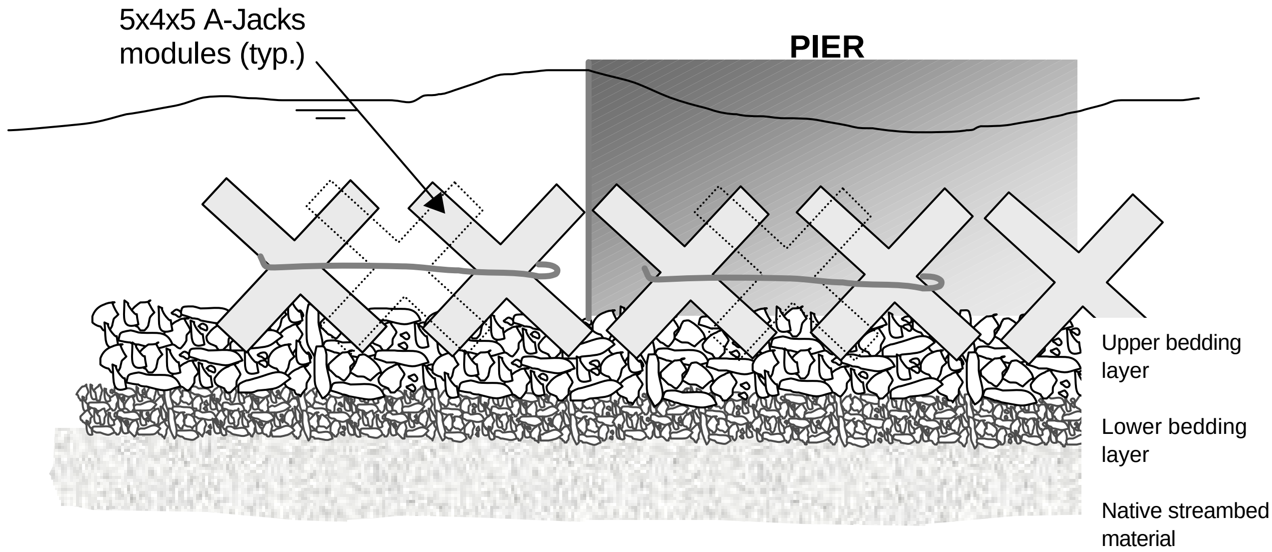

Figure 3.7a. Bedding detail showing two layers of granular bedding stone above native streambed material.

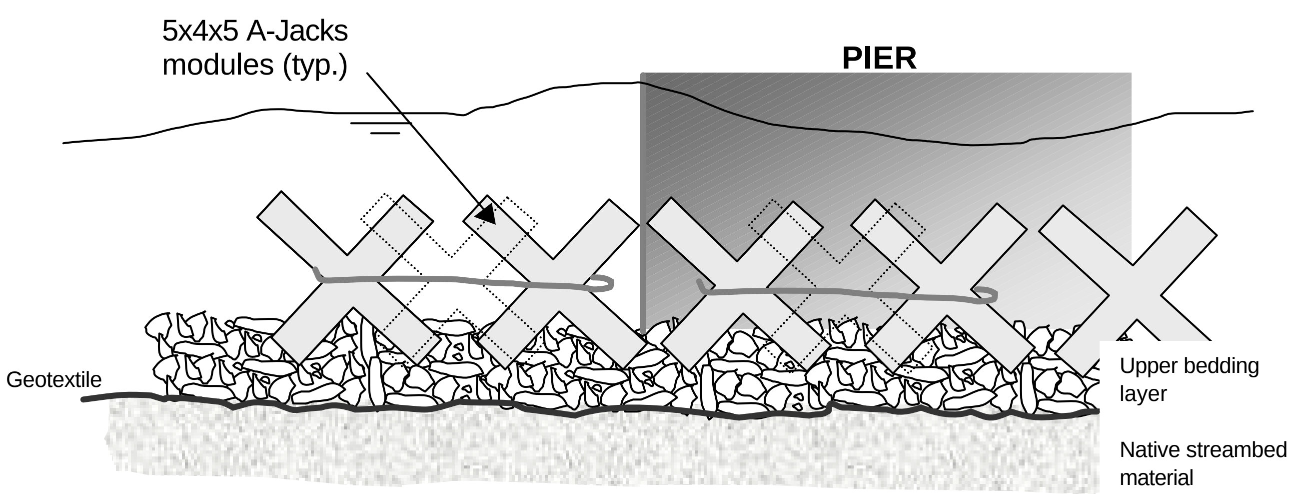

Figure 3.7b. Bedding detail showing ballast stone on top of geotextile.

5. REFERENCES AND APPENDICES

American Association of State Highway Transportation Officials (AASHTO), 1995. "Standard Specification for Geotextiles." AASHTO Designation M 288-96 DRAFT. February.

Bertoldi, D.A., et al., 1996. "An Experimental Study of Scour Protection Alternatives at Bridge Piers." U.S. Federal Highway Administration Publication No. FHWA-RD-95-187.

Escarameia, M., 1998. "River and Channel Revetments: A Design Manual." Thomas Telford Publications, London.

Fotherby, L.M., and J.F. Ruff, 1995. "Bridge Scour Protection Using Toskanes, Phase 1." Pennsylvania Department of Transportation, Report 91-02, June.

Holmquist-Johnson, C.L., et al., 1999. "Laboratory Testing of A-JACKS Units for Inland Applications: Hydraulic Property Documentation." Colorado State University research report for Armortec Inc., February.

Industrial Fabrics Association, 1990. "A Design Primer: Geotextiles and Related Materials." St. Paul, Minnesota.

Lagasse, P.F., et al., 1995. "Stream Stability at Highway Structures." U.S. Federal Highway Administration Hydraulic Engineering Circular No. 20, Publication No. FHWA-IP-90-014, 2nd edition, November.

Lagasse, P.F., et al., 1997. "Bridge Scour and Stream Instability Countermeasures." U.S. Federal Highway Administration Hydraulic Engineering Circular No. 23, Publication No. FHWA-HI-97-030, July.

TC Mirafi, 1998. "Geotextile Filter Design Manual." Prepared by Geosyntec Consultants, Norcross, Georgia.

Neill, C.R., 1975. "Guide to Bridge Hydraulics." Roads and Transportation Association of Canada, University of Toronto Press, 2nd edition.

Parker, et al., 1998. "Countermeasures to Protect Bridge Piers from Scour." University of Minnesota draft final report No. NCHRP 24-7, prepared for the National Cooperative Highway Research Program, Transportation Research Board, May.

Parola, A.C., 1993. "Stability of Riprap at Bridge Piers." Journal of Hydraulic Engineering, ASCE, Vol. 119, No.10.

Quazi, M.E., and A.W. Peterson, 1973. "A Method for Bridge Pier Riprap Design." First Canadian Hydraulics Conference, Edmonton, Canada. Proceedings.

Richardson, E.V., and S.R. Davis, 1995. "Evaluating Scour at Bridges." U.S. Federal Highway Administration Hydraulic Engineering Circular No. 18, Publication No. FHWA-IP-90-017, 3rd edition, November.

Thornton, C.I., et al., 1999. "Laboratory Testing of A-JACKS Units for Inland Applications: Pier Scour Protection Testing." Colorado State University research report for Armortec Inc., February.

Thornton, C.I., et al., 1999. "Laboratory Testing of A-JACKS Units for Inland Applications: Full-Scale Testing." Colorado State University research report for Armortec Inc., February.

Vennard, J.K., and R.L. Street, 1975. "Elementary Fluid Mechanics." John Wiley & Sons, New York.