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Preliminary Procedure to Predict Bridge Scour in Bedrock

DEPARTMENT OF TRANSPORTATION

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16. Abstract

The primary goal of this study is to develop a procedure to predict scour depths in bedrock that accounts for both the hydraulic conditions at the bridge site and the bedrock's ability to resist erosion. A methodology for determining material erodibility resulting from the erosive power of water has been presented by Annandale (1993, 1995). He introduced a relationship between stream power and a geomechanical material classification system known as the erodibility index. This report applies his findings to bridge scour analysis and presents an interim procedure for estimating bridge scour depths in bedrock and other materials defined by the erodibility index.

Implementation:

The recommendations in this report are currently being applied as an interim method for evaluating bridge scour at the Colorado Department of Transportation. Further laboratory study is planned to refine the relationship between energy dissipation at bridge piers and pier geometry. Findings from this study will be made available in a subsequent report.

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ABSTRACT

Scour at bridge crossings can lead to undermining of foundations and potentially to structure collapse. Bridge foundations must be designed to withstand the effects of scour from flooding events that can reasonably be expected to occur during a structure's life. Many equations are available to assist in the prediction of scour at bridge crossings. However, few account for the effects of gradation and none account for the effects of cohesion and consolidation. Currently, no quantitative procedure for determining bridge scour in bedrock or cohesive and consolidated material is in practice. The primary goal of this paper is to develop a procedure to predict scour depths in bedrock at bridges that accounts for both the hydraulic conditions at the bridge site and the bedrock's ability to resist erosion. A methodology for determining material erodibility resulting from the erosive power of water has been presented by Annandale (1993; 1995). He introduced a relationship between stream power and a geomechanical material classification system known as the Erodibility Index (Annandale, 1993; 1995; and Kirsten, 1982). This paper applies his findings to bridge scour analysis and presents an interim procedure for estimating bridge scour depths.

This study involved a review of conventional scour prediction methods and available data. Preliminary methods for determining stream power at bridge crossings are presented and an interim procedure for predicting bridge scour is outlined. This preliminary procedure will require refinement and calibration with additional laboratory data and field correlation. Although the main goal of this paper is to provide a method to predict scour at bridges in bedrock, this procedure is equally applicable to scour prediction in all naturally occurring materials defined by the Erodibility Index classification system.

Notation

A - net area of orifice (bridge opening)

b - pier width

C_o - orifice coefficient

D_{50} - median particle diameter

ΔE - energy loss per unit weight of water

g - gravitational acceleration

H - change in energy gradient through bridge contraction

J_a - joint alteration number

J_c - number of joints per cubic meter

J_n - joint set number

J_r - joint roughness number

K_1 - correction factor for pier shape

K_2 - correction factor for approach flow angle

K_b - particle/block size factor

K_d - interparticle bond strength factor

K_m - mass strength factor

K_n - Erodibility Index number

K_s - relative shape and orientation factor

l - unit channel length

L - pier length

P - stream power per unit channel width

P_a - stream power in approach section

P_p - stream power at base of bridge pier

$P_p^{/}$ - stream power at pier base adjusted for pier shape and flow attack angle

q - unit discharge of water

RQD - rock quality designation

S_f - slope of energy grade line

V - mean channel velocity

y - flow depth

γ - unit weight of water

τ - shear stress

Φ - equivalent residual friction angle

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1. INTRODUCTION

Flood related scour is a major threat to bridges and the travelling public. Analysis of scour requires an understanding of the interaction between the hydraulic forces and the variable properties of the channel bed and foundation materials that are found at bridge crossings. A large amount of literature is available regarding the analysis of bridge scour in non-cohesive materials but no procedure is currently in practice which relates the erosive power of water to the properties of bedrock or cohesive and consolidated material at bridges.

A quantitative method of scour prediction is needed to determine scour depths in bedrock and other cohesive and consolidated materials. Such a method will provide an increased level of confidence in locating bridge foundations at depths which will withstand scour, but not be excessively conservative as to needlessly increase foundation costs. The purpose of this report is to provide a practical procedure which combines an assessment of both the unique hydraulic conditions at a bridge site and the erodibility of its channel bed and bedrock foundations to predict potential scour depths. This report discusses the basic concepts of scour at bridges, the processes of erosion, and the relationship between the erosive power of water and material erodibility as presented by Annandale (1993; 1995). It provides a preliminary method to predict bridge scour in bedrock and other materials based on this relationship.

1.1 Scour

Richardson (1993) provided the following general definition of scour:

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams.

Scour occurs at a bridge crossings because the erosive potential of flowing water can be greatly increased at the constricted section of the bridge and in the vicinity of bridge piers and abutments. Scour at bridge crossings is a complex process and requires determination of the cumulative effect of its three main components: aggradation and degradation; contraction scour; and local scour.

Aggradation and degradation are long term changes in channel bed elevation that are generally independent of the bridge structure. They can be caused by human practices within the watershed or naturally occurring changes in basin climate or basin hydrologic characteristics. Aggradation is an increase in channel bed elevation caused by an increase in sediment load or decrease in sediment transport capabilities in a watercourse. Degradation is a lowering of channel bed elevation that can be caused by reductions in sediment load or increases in a stream's sediment transport potential. Degradation is common in urbanizing basins and in channels subject to aggregate mining.

Contraction scour is a lowering of the channel bed due to localized narrowing of a channel as caused by a constriction at a bridge. A contraction will cause an increase in velocity and a reduced flow area through the channel section. This will cause a local increase in the erosive potential of water at the contraction and can lead to scour.

Local scour occurs at bridge piers and abutments due to complex flow and turbulent conditions that develop in the vicinity of these obstructions. Pier scour is caused by strong turbulence and the complex flow pattern known as the horseshoe vortex that occurs at the pier. Abutment scour is caused by a turbulent mixing of flows in the main channel with flows obstructed by the abutment.

The process of scour at bridges is dependant on the site specific effects of hydrologic and hydraulic conditions, the geometry of the bridge, the geometry of the channel, and the characteristics of channel bed and foundation material. All these considerations are subject to considerable variation during a structure's life and can vary greatly even during a single flood event. This variability makes scour prediction a difficult task.

1.2 Existing Methods of Scour Prediction

In April, 1987, the New York State Thruway bridge over Schoharie Creek collapsed as a result of scour during flooding. A total of five vehicles and ten persons were lost in the failure. This tragedy became the impetus for an increased effort to evaluate and protect the nation's bridges from the effects of scour.

The Federal Highway Administration (FHWA) led this effort by publishing a technical advisory with guidance for state and local transportation agencies in establishing scour evaluation programs. The FHWA also published a manual with equations and recommendations for predicting scour at new and existing bridges.

The FHWA has published Technical Advisory - T 5140.23, "Evaluating Scour at

Bridges", which mandates that state and local agencies establish scour evaluation programs. This advisory requires that every new and existing bridge over water be evaluated for its vulnerability to scour. It recommends that bridge scour be analyzed by a multi-disciplinary team consisting of hydraulics, geotechnical and structural engineers. The technical advisory specifically states that bridge foundations should be designed to withstand the effects of scour without failing for the worst condition resulting from floods equal to or less than the 100-year flood and that bridges should be checked to ensure that they will not fail due to scour from the superflood (500-year flood).

The Federal Highway Administration has also published equations for computing scour in Hydraulic Engineering Circular 18 (HEC 18), "Evaluating Scour at Bridges" (Richardson et al., 1993). These equations have been selected as the most reliable equations currently available for predicting bridge scour. The equations in HEC 18 were developed in laboratory studies of non-cohesive, granular material and do not account for the variable ability of different materials to resist erosion. It is commonly thought that when applied at bridges with cohesive and bedrock foundation materials, these equations provide unreasonably excessive scour depths.

Interim guidelines for evaluating scourability of bedrock are presented in the FHWA publication "Scourability of Rock Formations" (Gordon, 1991). These guidelines provide advice for evaluating scour of bedrock material and relate a number of geotechnical index properties to a material's potential to scour. Rock quality designation (RQD), unconfined compressive strength, slake durability, abrasion and soundness of core samples are determined and results of these tests are evaluated against predetermined limiting values below which the material is assumed to be scourable.

The recommendations in these guidelines provide a qualitative assessment of relative scourability but do not provide a method to predict potential scour depths. The FHWA guidelines do not consider the hydrologic or hydraulic conditions at the bridge site nor provide an estimate of total scour depths. These guidelines only distinguish between an erodible and non-erodible material as defined by the indexed parameters. They do not provide recommendations to account for the integrated effect of material properties or attempt to relate them to the erosive power of water. The FHWA guidelines stress the use of subjective engineering judgement and experience in assessing material erodibility at bridge sites.

1.3 Scope of Study

This study presents a preliminary procedure for estimating the depth of scour into bedrock and other materials at bridges. This procedure relates the hydrologic and hydraulic conditions at the bridge site to the geomechanical properties of the channel bed and foundation material. The procedure is based on the relationship between the erosive power of water as defined by stream power and the ability of the channel bed and foundation material to resist erosion. This approach to erosion analysis and the basic relationships required for its application to bridge scour is explicitly suggested by Annandale (1993; 1995). The same approach is currently being investigated by the United States Bureau of Reclamation (Wittler et al., 1993) to estimate the extent of erosion of dam spillways and foundations.

This study presents a preliminary quantitative method for estimating bridge scour in sedimentary bedrock and other materials. It will aid engineers in determining optimal foundation elevations with respect to safety and cost.

Preparation of this paper involved review and summary of available literature and data relating to scour at bridges and material erodibility. Information accumulated during the literature review was used to develop a procedure for predicting bridge scour in bedrock and other material. This method is a stepped procedure that relates stream power to the erodibility of material at incremental scour depths. At some depth, an equilibrium will be reached where the material strength exceeds the erosive power of water. This quantitative procedure is recommended as a preliminary method to determine ultimate design scour depth at bridge crossings.

2. SCOUR AT BRIDGES

Erosion due to the effects of flooding at bridges is dependent on both the properties of the material being eroded and the hydraulic conditions of the flow. At bridges over waterways, these material properties and hydraulic conditions can vary considerably during the life of the structure or during a single flood event.

Conventional bridge scour prediction methods assume that with sufficient time, predicted scour depths will ultimately be reached regardless of channel bed and foundation material properties. Annandale (1993) asserts that erodibility is a threshold condition dependent on the magnitude of the erosive power of water. He states that if the erodibility threshold of a material is exceeded, scour will occur, otherwise scour will not occur.

Any approach to evaluating scour at bridge crossings requires a basic understanding of the process of erosion and the properties which relate to a material's susceptibility to scour. This chapter discusses the basic material properties which affect scour at bridges and the process of erosion from flowing water. It also discusses the application of an empirical material classification system developed by Kirsten (1982) and recommended by Annandale (1993; 1995) for scour and erosion analysis.

2.1 Material Erodibility

Hydraulic erosion occurs when the erosive power of flowing water exceeds a material's ability to resist erosion. It is dependent on many site specific properties of channel bed and foundation material. Scour is an example of hydraulic erosion and any procedure to determine scour at bridges requires consideration of those material properties which influence hydraulic erosion.

Material properties such as particle size and shape, material density, degree of cementation, and material gradation all affect scour of granular materials. The characteristics of rock which influence scour and erodibility are identified by Moore (1991) as rock material properties and rock mass properties. He describes rock material properties as those which define the rock type, color, particle size, texture, hardness and strength. Moore (1991) described rock mass properties as those macroscopic features of a rock mass which affect erosion such as joints and fractures.

Kirsten (1982) described the material properties which affect excavation. These properties also characterize a material's ability to resist hydraulic erosion and include material strength, density, degree of weathering, block size, block shape, block orientation, joint roughness, joint gouge, and joint separation. Kirsten (1982) developed an index to classify materials based on these properties.

2.2 The Process of Erosion

The process of erosion is defined by Annandale (1993) as a process of progressive dislodgement involving jacking, dislodgement and transportation. This process is caused by pressure fluctuations originating from the turbulence of flowing water. The greater the turbulence, the greater the pressure fluctuations and the higher the likelihood of erosion.

Figure 2.1 illustrates the process of erosion. Annandale describes the process of erosion as follows:

> *The jacking effect is caused by pressure fluctuations in water. These fluctuations originate from turbulence which is generated as water discharges over or incident to a boundary. The higher the turbulence intensity, the greater the fluctuations in pressure. Research has shown that the pressure fluctuations largely affect the pressure at the upper surface of the boundary. Pressure at the boundary surface can be as low as vapor pressure, while at the same time, the pressures within the cracks and crevices of the material are still at hydrostatic pressure. These large pressure differentials essentially result in fluctuating net forces which cause the material to progressively move out of its position of rest. Once the material is at the threshold of stability, it is dislodged by the power of the discharging water and finally transported in a downstream direction.*
>
> *The process of erosion for other earth material types can be described in the same generic manner.*

2.3 The Erodibility Index

Kirsten (1982) presented a classification system for indexing the effort required for material excavation. This classification system is based on in situ material properties derived from common laboratory tests and field observations. His classification index is defined by the product of the basic parameters which influence excavation and summarizes the most important variables into a single dimensionless number.

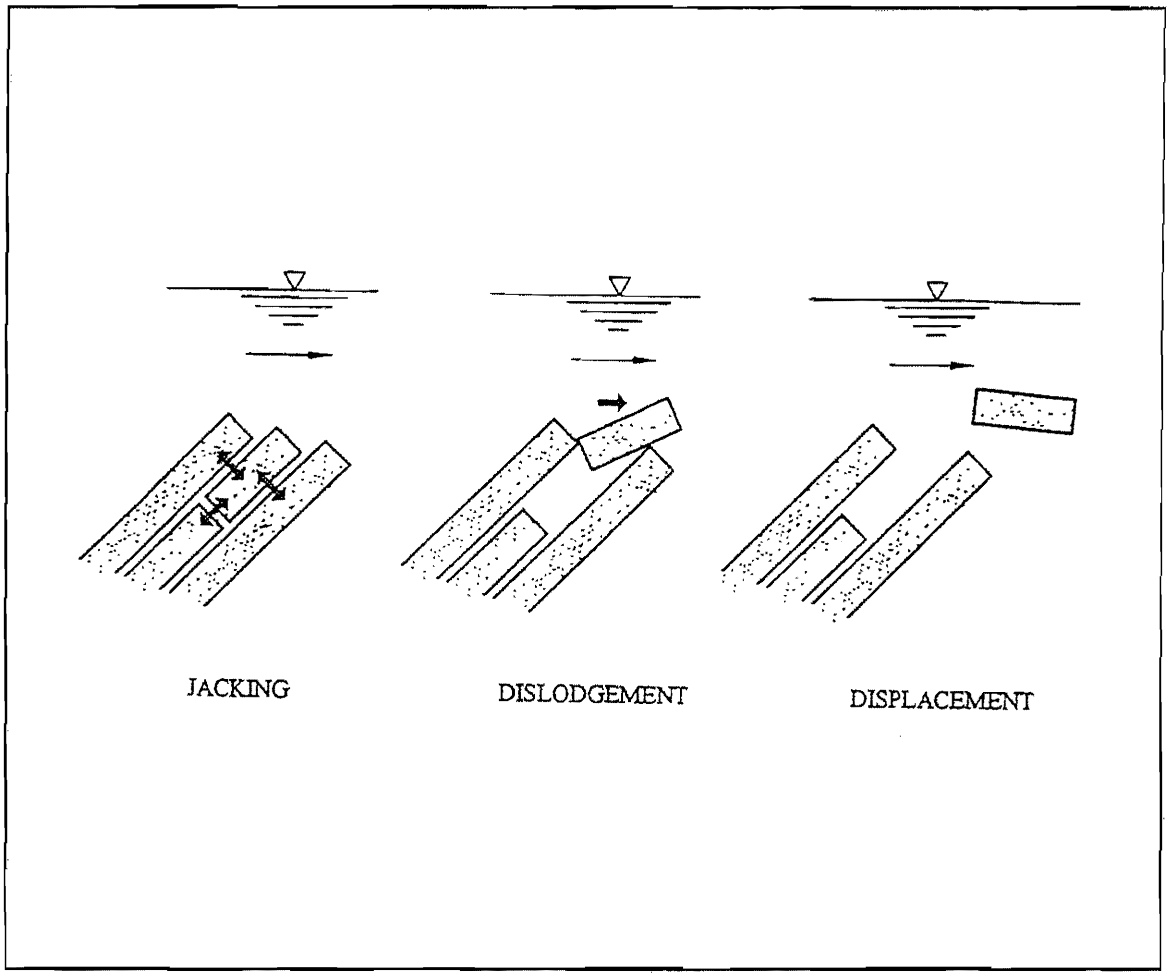

Figure 2.1 The Process of Erosion (Annandale, 1993)

These same parameters which indicate the effort required for excavation represent a material's ability to resist hydraulic erosion. Annandale (1993; 1995) presented Kirsten's classification system as a method for quantifying material erodibility subject to flowing water. He called it the Erodibility Index and expressed it in terms of the following equation:

$$K_n = K_m K_b K_d Ks \qquad \text{(Equation 2.1)}$$

where:

K_n = Erodibility Index

K_m = Mass strength factor

K_b = Particle/block size factor

K_d = Interparticle bond strength factor

K_s = Relative shape and orientation factor

These parameters can be readily attained from bore hole information and standard laboratory tests. The indexing system is suitable for nearly all naturally occurring materials including rock, granular soils, cohesive soils, and detritus. This section defines the factors which comprise the Erodibility Index and presents guidelines for its determination.

2.3.1 Mass Strength Factor

The mass strength factor (K_m) is the dominant Erodibility Index parameter which represents a material's strength and therefore its ability to resist erosion. It is a measure of a material's consistency and is determined by various field and laboratory procedures, depending on material type. It has been related to the standard penetration test (SPT) for granular materials, the vane shear strength for cohesive soils, the unconfined compressive strength for rock and the in situ deformation modulus for detritus. Mass strength numbers for these materials are tabulated in Appendix A in Tables A-1 to A-4, as derived by Kirsten (1982). Where geotechnical tests can not be performed, the mass strength factor can be estimated based on observations of the materials relative consistency.

2.3.2 Particle/Block Size Factor

The particle/block size factor (K_b) is the material parameter which represents rock mass quality or the median particle diameter for granular material. Larger block and particle sizes will provide greater resistance to movement. The particle/block size factor is determined for rock by the ratio of rock quality designation (RQD) to the number of different joints, the joint set number (J_n). Therefore for rock:

$$K_b = RQD \,/\, J_n \qquad \text{(Equation 2.2)}$$

where: $5 \le RQD \le 100$

If RQD data is unavailable, Kirsten (1982) recommended that K_b be represented as:

$$K_b = (115 - 3.3J_c)/J_n \qquad \text{(Equation 2.3)}$$

where: J_c = number of joints per cubic meter

For granular material (Wittler et al.,1993) K_b can be expressed as:

$$K_b = 1000\ (D_{50})^3 \qquad \text{(Equation 2.4)}$$

where: D_{50} = median particle diameter (meters)

A RQD of 5 should be used for intact granular soils and gravel and a RQD of 100 should be used for cemented material. A J_n of 5 should be used for soils and granular materials. This indicates a K_b value of 1.0 for uncemented, intact granular soils. Table A-5 in Appendix A provides a relationship for joint count number (J_c) and rock quality designation (RQD). Table A-6 provides information for determining the joint set number (J_n) based on observation of field samples.

2.3.3 Interparticle Bond Strength Factor

Kirsten (1982) called the interparticle bond strength factor (K_d), the joint strength number. It is the parameter which represents the relative strength of discontinuities in rock and the strength of particle bonding in granular materials. It is determined by the ratio between joint wall roughness and joint wall alteration in rock material. It is expressed by the following equation:

$$K_d = J_r\ /\ J_a \qquad \text{(Equation 2.5)}$$

where: J_r = joint roughness

J_a = joint alteration number

Values for J_r and J_a are provided in Tables A-7 and A-8 respectively. Joint roughness and joint alteration number are determined by observation of joint tightness, condition, alteration material and joint separation. Tighter and rougher joints with more sound alteration material within the joints will result in a composite material that is more resistant to erosion. Rock with tighter joints is less erodible than rock with open joints.

In granular materials and materials for which J_r and J_a can not be clearly defined, Kirsten (1982) recommends estimating the interparticle bond strength factor by the following equation:

$$K_d = \tan \Phi \qquad \text{(Equation 2.6)}$$

where: Φ = equivalent residual friction angle

2.3.4 Relative Shape and Orientation Factor

Kirsten (1982) called this parameter (K_s) the relative ground structure number. K_s is used to relate the relative shape of material particles or blocks and the orientation and spacing of the structural features to the direction of effort during excavation. Direction of excavation effort is analogous to the direction of flowing water. Kirsten (1982) developed a table from which K_s can be determined from the dip angle and direction of the least favorable discontinuity relative to stream flow and the ratio of joint spacing, r. Strike and dip of bedding planes or discontinuities is ideally obtained during drilling but this is not always practical. Information on orientation of discontinuities is sometimes available on geologic maps or through observation of local outcrops.

If a material has no identifiable structure, the relative ground number is assumed to have a value of one (K_s= 1.0). In cases where structure is present but its orientation is not definable a relative shape and orientation factor of 0.5 is suggested. Table A-9 is provided to determine K_s.

2.3.5 Summary of Erodibility Index

The Erodibility Index (Annandale, 1993; 1995)(Kirsten, 1982) provides a quantitative classification system representing the strength of materials. It can be used to determine a material's ability to resist erosion when subject to flowing water. The Erodibility Index is incorporated into a procedure of scour prediction at bridges as recommended by Annandale (1993; 1995).

The discussion presented above provides practical equations and tabular relationships generated by Kirsten (1982), but it does not provide a complete theoretical derivation or literature referencing from which Kirsten's classification system was developed. Table 2.1 summarizes the Erodibility Index factors and the geotechnical parameters required for its determination.

Although the Erodibility Index provides a thorough classification of a material's erodibility, there are certain aspects of hydraulic erosion which it can not implicitly consider.

Abrasion, armoring and chemical weathering are all factors which affect scour at bridges. Abrasion can occur when bed load material impacts the channel bed and initiates mechanical weathering. This acts to increase the rate of erosion during flooding. The influence of abrasion on scour at bridges is extremely difficult to quantify. Although the susceptibility of a material to mechanical weathering can be to some extent accounted for

Table 2.1 Determining the Erodibility Index

Step	Task	Material Type			
		Rock	Granular Soil	Cohesive Soil	Detritus
1	Evaluate Mass Strength Number, K_m	Determine Unconfined Compressive Strength, **Get K_m from Table A-3**	Determine Standard Penetration Test, **Get K_m from Table A-1**	Determine Vane Shear Strength, **Get K_m from Table A-2**	Determine in situ Deformation Modulus, **Get K_m from Table A-4**
2	Evaluate Particle/Block Size Number, K_b	Determine RQD, J_n, and J_c. J_n and J_c from tables A-4 and A-5 **K_b = RQD/J_n or $K_b = \frac{(115-3.3J_c)}{J_n}$**	Determine D_{50} of material in meters **$K_b = 1000D_{50}^3$**	For intact soils and detritus **K_b = 1.0**	
3	Evaluate Interparticle Bond Strength Number, K_d	Determine J_r and J_a from Tables A-7 and A-8, **$K_d = J_r / J_a$**	Determine residual friction angle, Φ **$K_d = \tan \Phi$**		
4	Evaluate Relative Shape and Orientation Number, K_s	Determine effective dip of material, **Get K_s from Table A-9**	If no identifiable structure, **K_s = 1.0**		
5	Calculate Erodibility Index, K_n	**$K_n = K_m K_b K_d K_s$**			

in the Erodibility Index, it is possible that with further analysis the effects of abrasion can be directly determined.

Armoring occurs when large particles which can not be transported are deposited and form a sorted layer of cobble and boulder size material. An armoring layer is typically formed during low magnitude flooding events. Scourability of armored layers can be evaluated by treating the layer as a distinct unit and defining it with the Erodibility Index classification system.

The effects of chemical weathering on the erodibility of channel bed and foundation material is extremely difficult to assess and its extent can vary over the life of a bridge structure. While some have suggested (Lewis, 1993) that chemical weathering is a relatively slow process which is negligible during a structure's design life, review of Colorado Department of Transportation bridge inspection files indicate that under some circumstances chemical weathering can be significant. The effects of chemical weathering might be qualitatively predicted with a laboratory test for soundness. If weathering is expected to be significant, a reduction in the Erodibility Index values can be made for an upper incremental thickness of bedrock material to that of weathered rock. This modified Erodibility Index number can be used in the scour prediction procedure.

Stress history can also affect scour. Lewis (1993) describes scour at a Yellowstone River bridge where scour occurred around and below the bridge pier footing leaving a column of material directly below the footing. The foundation material's resistance to erosion was increased by the stress applied at the footing. The consolidation of material caused by the bearing weight of the footing may have prevented the physical failure of the bridge. Although a vertical compressive stress applied by a bridge pier appears to increase a material's ability to resist erosion, it is not proposed that the Erodibility Index be modified to reflect this localized increase in material strength. If a pier is undercut it should be considered a critical situation which could rapidly fail with additional scour or chemical and mechanical weathering.

2.4 Erodibility Threshold

Annandale (1993; 1995) presented the results of a comparative analysis between the erosive power of water and the erodibility of emergency spillways. A relationship between stream power and the Erodibility Index (Annandale, 1993;1995; and Kirsten, 1982) was developed based on Soil Conservation Service (SCS) field data from more than 150 observations of scour at emergency spillways and published data on incipient motion of granular materials. Materials ranged from cohesionless soil to bedrock. By comparison of material and flow conditions under which spillway erosion occurred and those under which erosion did not occur, a limit was established between erosive and non-erosive conditions. Annandale (1993; 1995) observed that this erodibility threshold limit showed a distinct trend and recommended that it be used to predict whether material erosion will occur for a wide variety of flow conditions and material types.

The results of this analysis are presented in Figures 2.2(a) and (b). These figures can be used to predict whether channel bed or foundation material will scour from the erosive power of flows of specified flood frequencies. If a material's critical stream power at the erodibility threshold is exceeded, erosion will occur, if it is not exceeded, no scour is predicted. This threshold relationship can be used to predict the erodibility of incremental thicknesses of channel bed and foundation material at bridges. An iterative application of the erodibility threshold criterion will provide an estimate of total scour depth at bridges.

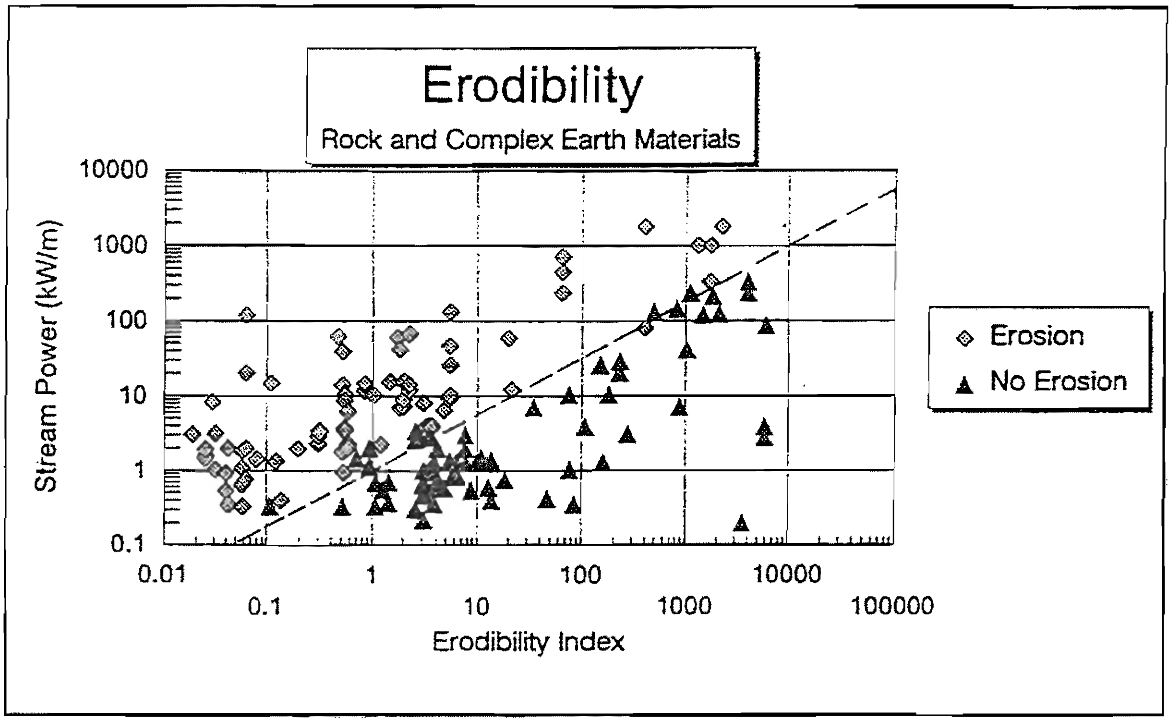

Figure 2.2(a) Erodibility Threshold (Annandale, 1995)

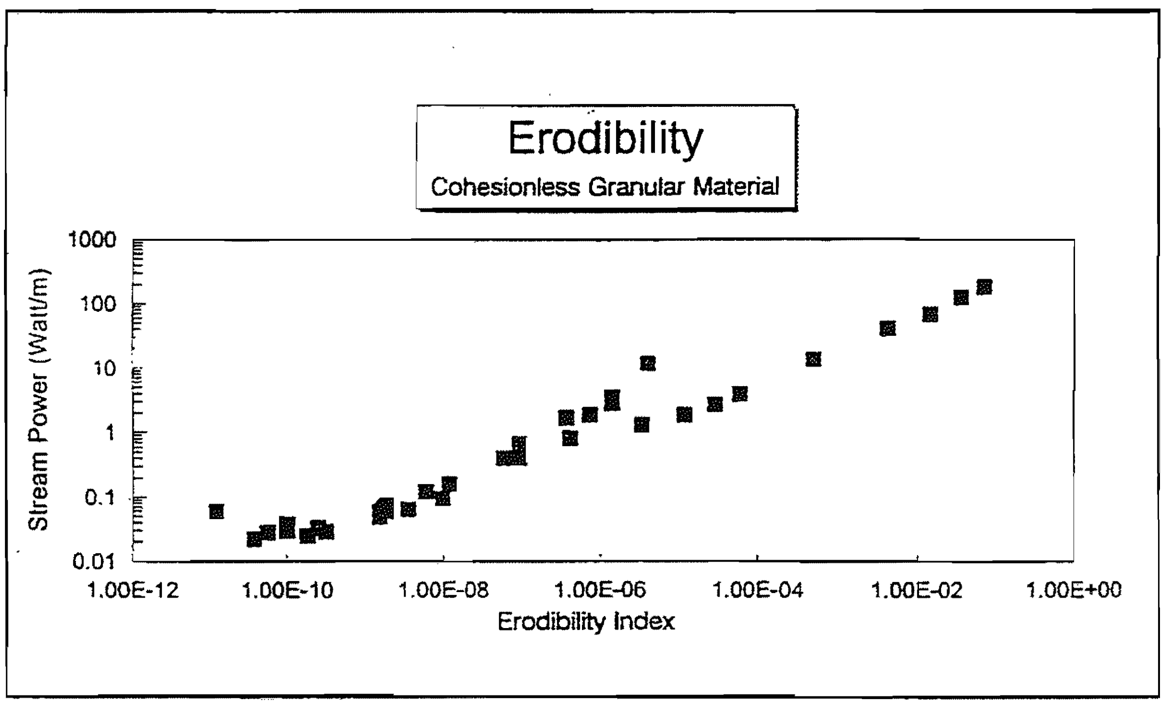

Figure 2.2(b) Erodibility Threshold (Annandale, 1995)

3. STREAM POWER

Many equations have been developed which relate alluvial and scour processes to the erosive and sediment transport potential of flowing water. Velocity, discharge, flow depth, slope, shear stress, and Froude number have all been used to characterize the erosive power of water for such applications as sediment transport (Yang, 1977), scour analysis, riprap design, and stable channel design. Studies have indicated (Yang, 1977) these parameters provide reasonable results in some situations but do not provide universally applicable correlations to the stability of channel bed and bridge foundation material. They do not provide a complete characterization of the erosive power of water.

Annandale (1993; 1995) states that pressure fluctuations resulting from turbulence intensity are the primary cause of erosion in flowing water. A parameter which represents the magnitude of turbulent pressure fluctuations was identified by Annandale (1993; 1995). He analyzed the findings of Fiorotto and Rinaldo (1992) and concluded that the magnitude of fluctuating turbulent pressures in discharging water is strongly related to the rate of energy dissipation and that these fluctuating pressures are the primary cause of hydraulic erodibility. Greater turbulence intensity in the vicinity of bridge contractions, piers and abutments will result in flow conditions with greater strength and a higher rate of energy dissipation. Scour potential will increase. Therefore, a flow parameter which represents the rate of energy dissipation should be used to characterize the power of flowing water. The rate of energy dissipation can be expressed as stream power using the following expression (Annandale, 1993) :

$$P = \gamma \, q \, \Delta E \qquad \text{(Equation 3.1)}$$

where:

- P = stream power per unit width, [kW/m]
- γ = unit weight of water, [kN/m^3]
- q = unit discharge of water, [$m^3/s \cdot m$]
- ΔE = energy loss per unit weight of water, [m]

Stream power was used by Annandale (1993) to develop the erodibility threshold criterion which he has recommended as a tool in erosion analysis.

The stream power approach to analysis of fluvial processes is well established. It has been discussed by Bagnold (1966), Yang (1972) and Chang (1984; 1988). They all conclude that a relationship exists between the rate of energy dissipation and the rate of sediment transport. Bagnold (1966) described stream power as the rate of energy dissipation per unit area. He expressed stream power as:

$$P = \tau V \qquad \text{(Equation 3.2)}$$

where: τ = shear stress, [kN/m^2]

V = mean channel velocity, [m/s]

For a unit channel width,

$$\tau = \gamma\, y\, S_f \qquad \text{(Equation 3.3)}$$

$$V = q/y \qquad \text{(Equation 3.4)}$$

where: y = flow depth [m]

substituting Equations 3.3 and 3.4 into Equation 3.2, yields:

$$P = \gamma\, q\, S_f \qquad \text{(Equation 3.5)}$$

where: S_f = slope of energy grade line

Equation 3.5 is equivalent to Equation 3.1 for a unit channel length.

Yang (1972) developed the concept of unit stream power. He analyzed data on sediment transport rates and concluded that the erosion rate in alluvial channels is related to the rate of energy dissipation per unit weight of water. Yang defined unit stream power as the product of flow velocity and channel slope.

Chang (1984; 1988) presented the concept of minimum stream power per unit channel length. He states that an alluvial channel will attempt to reduce the spatial variation in the rate of energy dissipation by changes in the channel geometry. These changes tend to reduce the boundary resistance and establish uniform power expenditure along a channel reach.

At bridge sites, where constrictions and flow obstructions are present, backwater will result. Upstream storage of potential energy and the rate of energy dissipation that occurs at a bridge is reflected as bridge backwater. This potential energy is dissipated through the bridge structure. The energy losses through the bridge result in changes to the channel geometry in the direction of a uniform and minimum rate of energy dissipation. The channel will attempt to increase its area and scour will result. Isolation and determination of the energy losses caused by the individual bridge components will allow estimation of resulting local stream power. Stream power through the contracted section, in the vicinity of piers and adjacent to abutments can be compared with the stream power required to initiate erosion of the channel bed and foundation material using the erodibility threshold criterion presented by Annandale (1993; 1995).

This chapter recommends procedures for estimating stream power in contracted sections and at bridge piers. Because of the complex nature of turbulent flows around bridge piers, an interim empirical relationship between stream power in the upstream channel section and at the pier is recommended for use in this scour prediction procedure. A discussion of abutment scour is provided but no correlation or method for estimating stream power at abutments is presented.

3.1 Scour at Bridge Contractions

Contraction scour occurs when the flow area of a channel is reduced by the encroachment of a bridge. An increased rate of energy dissipation occurs as a result of increased friction, flow contraction and flow expansion. Flows will have a greater capacity to erode and transport channel bed material at the bridge. Chang (1988) states that the channel geometry will adjust until an equilibrium rate of energy dissipation is attained for a given channel reach. The scouring process will continue until a non-erodible material is encountered or until the rate of energy dissipation through the bridge is equal to that of the upstream channel.

Richardson (1993) identifies two types of contraction scour situations. Live bed scour occurs when the upstream flow has sufficient power to transport channel bed material into the section at the bridge. Clear water scour occurs when no significant sediment transport occurs from the upstream channel into the bridge section. From a practical standpoint, using the erodibility threshold criterion in the scour prediction procedure is analogous to clear water scour analysis. In clear water scour, erosion of channel bed material will occur when its erodibility threshold (critical stream power) is exceeded.

Richardson (1993) pointed out that under flood conditions, live bed scour should be expected in most cases. Further research is needed to determine the influence of sediment transport on stream power at bridge contractions. It is not known if the bed load will tend to increase scour depths through abrasive action, or reduce scour by limiting available stream power.

This section discusses the effect of bridge contractions on the rate of energy dissipation. It provides recommendations for determining stream power in free surface and pressure flow conditions at bridges. A discussion of the applications of the Erodibility Index (Annandale, 1993; 1995; and Kirsten, 1982) methodology for contraction scour analysis is also presented.

3.1.1 Stream Power at Bridge Contractions

Contraction of flow at bridges will cause an increase in the rate of energy dissipation through the bridge section. This increase in the rate of energy dissipation is reflected by the upstream storage of potential energy in the form of backwater. In an open channel section, the rate of energy dissipation in terms of stream power per unit width, is expressed as:

$$P = \gamma\, q\, \Delta E \qquad \text{(Equation 3.1)}$$

where: $\Delta E = S_f\, L$ (Equation 3.7)

and: $S_f =$ slope of the energy grade line

$L =$ unit channel length, [m]

Substituting Equation 3.7 into Equation 3.1 yields (Annandale, 1993; 1995):

$$P = \gamma\, q\, S_f\, L \qquad \text{(Equation 3.8)}$$

The slope of the energy grade line through the bridge is readily determined from WSPRO (FHWA, 1986) or HEC 2 (United States Corps of Engineers (C.O.E.), 1990) output.

When scour at a bridge develops, the flow area through the bridge increases. If material is free to move, scour will continue until stream power in the bridge contraction approaches that in the upstream channel. Concurrently, backwater will be reduced and the rate of energy dissipation will approach that which occurs in the unconstricted channel.

Bradley (1978) performed model scour experiments and observed that reductions in backwater are directly related to the cross sectional area of scour. He developed a correction factor to adjust backwater for progressive scour depths. This backwater correction factor is related to the ratio of the area removed by scour to the total area of flow through the bridge prior to initiation of scour. Bradley (1978) recommends that this correction factor be multiplied by the total backwater prior to initiation of scour to calculate the reduced backwater as scour increases.

This same approach could potentially be used to adjust the energy slope in Equations 3.8 for progressive scour depths if a relationship between energy slope and scour depth can be confirmed. Once the adjusted energy gradient is computed, this slope can be used to compute the stream power for incremental scour depths below the original channel bed elevation. When compared with the critical stream power determined from the erodibility threshold, an incremental process can be implemented to predict ultimate scour depths. Determining the reduction in energy gradient as scour develops can be accomplished by repeating step backwater analysis with cross sections at the bridge modified to show progressive scour depths until such a relationship can be developed.

3.1.2 Pressure Flow Conditions

Many bridges will be inundated or overtopped during major floods. Flow conditions through the bridge will change from open channel flow to pressure flow. The flow conditions and scour mechanism will be considerably altered and potential scour depths can increase (Richardson, 1993).

Pressure flow is calculated using the common orifice equation:

$$Q = C_o A \, (2 \, g \, H)^{0.5} \qquad \text{(Equation 3.9)}$$

where:
- Q = discharge through the orifice
- C_o = orifice coefficient
- A = net area of orifice, bridge opening
- g = gravitational acceleration
- H = change in energy gradient elevation upstream and tailwater elevation downstream

HEC 2 (C.O.E, 1990) and WSPRO (FHWA, 1986) will both provide energy slope and energy losses through the bridge structure for pressure flow based on the orifice equation. The slope determined for the energy grade line at the bridge provided for orifice conditions can be used in Equation 3.8 to calculated stream power due to pressure flow. This is a preliminary recommendation and confirmation of its applicability is required.

3.1.3 Contraction Scour Summary

Interim recommendations are provided to compute stream power for both free surface and pressure flow conditions at bridge contractions. These stream power values can then be compared to the critical stream power of channel bed and foundation material as estimated from the erodibility threshold criterion (Annandale, 1993).

A detailed procedure outline for determining the total scour depth at a bridge contraction is presented in Chapter 4.

3.2 Scour at Bridge Piers

Pier scour is the removal of channel bed material from the base of a bridge pier during flooding events. Excessive pier scour can lead to undermining of footings, exposure of piling and potentially to bridge failure. Engineers must provide estimates of scour to ensure that bridge foundations are safe from scour events that may reasonably be expected during a structure's life.

Many equations have been developed to compute pier scour. Most of these pier scour prediction methods were empirically derived from laboratory studies with little or no field verification. They relate maximum scour depths to pier geometry and approach flow conditions including depth, velocity, and Froude number. Few consider the effects of channel bed material gradation and none consider the effects of cohesion and consolidation.

Hopkins et al. (1980) compared some of the more commonly used pier scour equations and reported considerable variation in predicted scour depths. He concluded that each prediction method's validity is limited to the range of flow conditions in the experiments from which the method was derived.
Hopkins et al.(1980) state:

> *Over the past century many investigators have attempted to develop a simple scour prediction formula... It appears that a set of variables were arbitrarily selected and data collected over a limited range to determine their relationship to scour depth... This approach has left us with a large number of sometimes conflicting formulas to predict scour.*

It is argued that these pier scour equations represent maximum scour potential for the design flow conditions and that predicted scour depths will be ultimately reached with sufficient time regardless of channel bed material. Annandale (1993; 1995) suggests that erodibility is a threshold condition dependent on the magnitude of the erosive power of water. If the erodibility threshold of a material is not exceeded, scour will not occur.

The following sections discuss the relationship between the horseshoe vortex and pier scour, and present a preliminary relationship between scour depth and the erosive power of water associated with the horseshoe vortex. Application of this relationship to a method of scour prediction is discussed. This method of scour prediction couples the hydrologic and hydraulic conditions at the pier with the geomechanical properties of the channel bed, and provides a quantitative method for estimating scour in cohesive, consolidated and bedrock materials.

3.2.1 The Horseshoe Vortex

Strong turbulence and flow acceleration occurs at the base of bridge piers during flooding. A complex flow pattern is established that causes turbulent pressure fluctuations and changes in shear stress distribution at the base of the pier. This flow pattern is known as the horseshoe vortex and the magnitude of pressure fluctuations associated with it is directly related to the development of scour at bridge piers.

Johnson et al. (1993) suggest that the size and strength of the horseshoe vortex can be characterized in terms of the flow conditions at the base of the pier. Greater turbulence intensity at the base of the pier will result in a horseshoe vortex with greater strength. A higher rate of energy dissipation and greater scour potential will result. The rate of energy dissipation can be expressed as stream power.

A method to determine the stream power at piers is needed to permit incorporation of the erodibility threshold criterion into a procedure for estimating pier scour of bedrock or cohesive and consolidated channel bed and foundation materials.

3.2.2 Stream Power/Pier Scour Relationship

The complex flow patterns that form the horseshoe vortex and cause scour at bridge piers are not completely understood. These patterns and the magnitude of pressure fluctuations associated with the vortex are affected by elements of the pier geometry including pier width, pier shape, and alignment to flow. The vortex is also related to the depth and velocity of the approach flow and the depth and shape of the scour hole itself. Developing a theoretical equation to compute stream power at a pier, which accounts for the effects of pier geometry, approach flow, and scour hole geometry would be extremely difficult. Direct measurement of stream power within the horseshoe vortex is presently not possible. An empirical relationship of pier scour depths to stream power is required.

Data presented by Parola (1990) and Johnson et al. (1993) was analyzed to assess the relationship between scour hole depth at the base of a pier to the rate of energy dissipation of water flowing within the hole. Both investigators performed model experiments to determine the relationship of boundary shear stress in a scour hole to the stability of material placed in the hole. Boundary shear was determined indirectly by placing uniform granular material with known critical shear stress in preformed scour holes and exposing it to flowing water until movement was observed. Parola (1990) and Johnson et al. (1993) measured both pier geometry and the approach flow conditions. Sufficient data was presented to compute stream power within the pier scour hole and in the upstream channel. The relationship derived from Johnson's data between relative scour depth (scour depth/pier width) and stream power ratio (stream power at the pier base/stream power in the upstream channel) is expressed graphically in Figure 3.1. A relationship between the two is apparent. The figure indicates that stream power decreases as the scour hole develops and increases with greater pier width.

Shen et al. (1969) observed that under uniform flow conditions, the shear stress in the channel is equal to that at the bottom of the scour hole after maximum scour depth is reached. This implies that maximum relative scour depth is an equilibrium condition at which the rate of energy dissipation within the scour hole is approximately equal to the rate of energy dissipation in the upstream channel. A similar observation was made by Richardson et al. (1993). They state that the equilibrium scour depth does not exceed approximately 3.0 times the pier width. The curve plotted on Figure 3.1 represents the 95% upper confidence limits of the stream power ratio/relative scour depth relationship within the limits of Johnson's data based on linear regression analysis (Appendix B). The curve is extrapolated asymptotically to the point with a stream power ratio value of 1.0 and a relative scour depth value of 3.0, consistent with Richardson's observations.

3.2.3 Application of Preliminary Stream Power/Pier Scour Relationship

Figure 3.1 provides a provisional upper limit correlation between stream power and scour depth. This relationship can be used as an interim method to estimate stream power values as a scour hole develops for specific design flow conditions and pier widths. These stream power values can then be compared to the erodibility threshold of the channel bed and foundation material at incremental depths. The ultimate scour depth is determined to be where the stream power in the scour hole is less than the threshold stream power that is required to initiate erosion. Recommendations for applying the preliminary stream

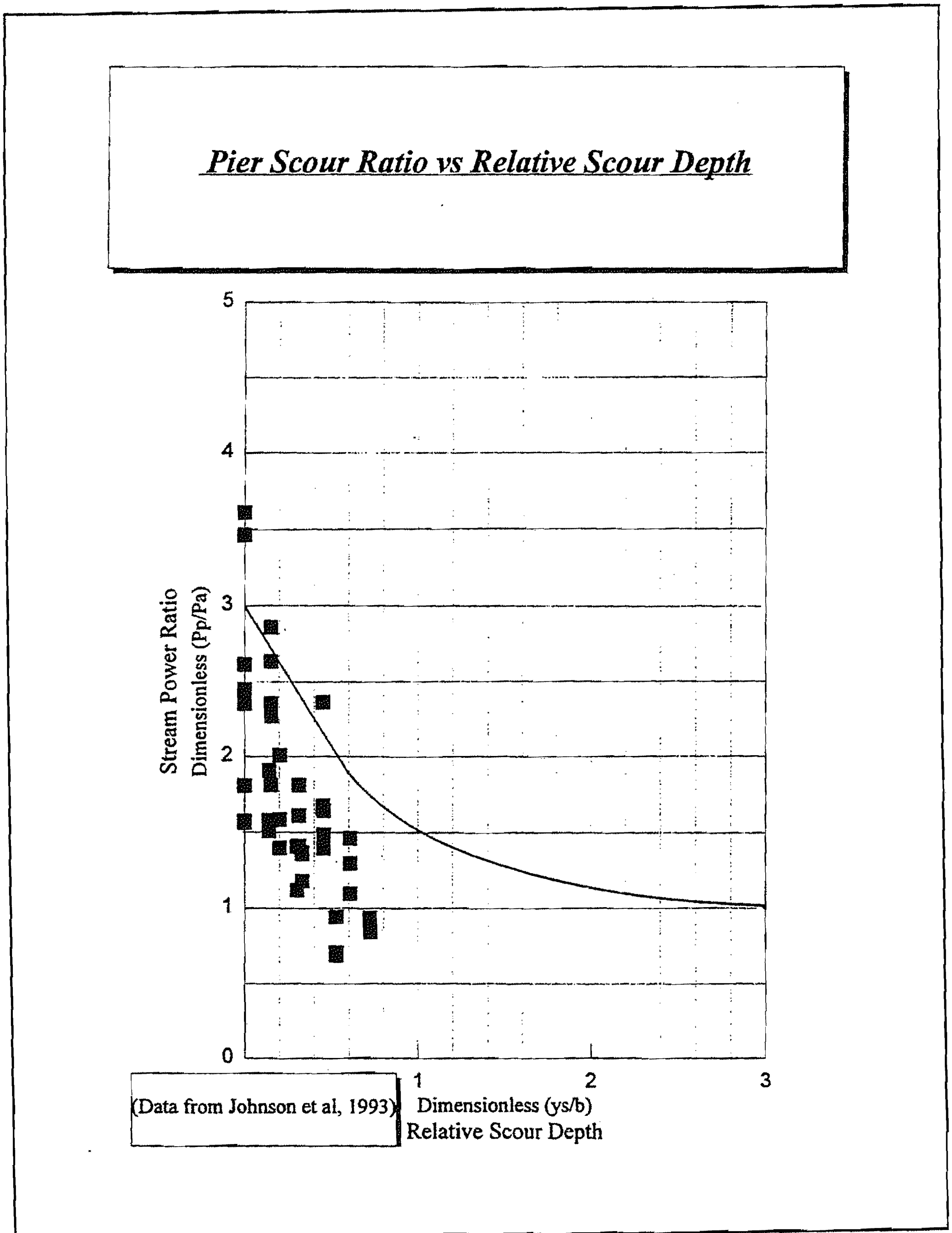

Figure 3.1 Preliminary Stream Power/Pier Scour Relationship

power/pier scour relationship to a procedure for predicting pier scour in bedrock and cohesive material are presented in Chapter 4.

The relationship developed is for circular or rounded piers and adjustment is required for other pier shapes and adverse approach flow angles. It is recommended that the influence of pier shape and angle of attack of flow be considered in the same way as recommended by Richardson et al. (1993) in HEC 18. Adjustment for the pier can be accomplished by multiplying the stream power estimated from Figure 3.1 by the appropriate correction factors. The adjusted stream power at the base of piers can be expressed as:

$$P_p' = K_1 K_2 P_p \qquad \text{(Equation 3.10)}$$

where:

P_p' = stream power adjusted for pier shape and flow attack angle

K_1 = correction factor for pier shape

K_2 = correction factor for approach flow angle

P_p = stream power at pier determined from Figure 3.1

These correction factors are provided in Tables 3.1 and 3.2.

The data from which the relationship between the stream power ratio and relative scour depth was derived is limited and may be dependent on the subjectivity inherent in the experiments. Scour holes were preformed with slopes at the angle of repose of sand. This may introduce some inaccuracy into the relationship as it may not accurately represent scour shapes and slopes that form in cohesive and bedrock material. Additional data is needed, particularly at relative scour depths between 1.0 and 3.0. Further laboratory data and field calibration will help refine this relationship. Correction factors for pier shape and flow attack angle should also be analyzed to confirm their applicability to this scour prediction procedure.

Although the primary goal of this paper is to determine scour in bedrock and cohesive material, this methodology is equally applicable to determination of pier scour in noncohesive channel bed material.

Table 3.1 Correction Factor K_1 for Pier Nose Shape

Shape of Pier Nose	K_1
(a) Square Nose	1.1
(b) Round Nose	1.0
(c) Circular Cylinder	1.0
(d) Sharp Nose	0.9
(e) Group of Cylinders	1.0

Source: Richardson, E.V., Harrison, L.J., and Richardson, J.R., Revised 1993, *Evaluating Scour at Bridges,* FHWA-IP-90-017,U.S Department of Transportation, Washington, D.C.

Table 3.2 Correction Factor K_2 for Angle of Attack of Flow

Angle	L/b = 4	L/b = 8	L/b = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of approach flow L = length of pier b = width of pier			

Source: Richardson, E.V., Harrison, L.J., and Richardson, J.R., Revised 1993, *Evaluating Scour at Bridges,* FHWA-IP-90-017,U.S Department of Transportation, Washington, D.C.

Most of the variables affecting pier scour identified by previous investigators are considered in this procedure. Pier width and approach flow conditions are accounted for directly in stream power ratio versus relative scour depth relationship. Impacts from ice or debris buildup can be considered by increasing the width of the pier appropriately. Pier shape and flow attack angle are considered by using the correction factors presented in Tables 3.1 and 3.2. Channel bed and foundation material properties are accounted for in the Erodibility Index developed by Kirsten (1982) and Annandale (1993; 1995).

3.3 Abutment Scour

Abutment scour occurs when flows from the main channel mix with flows obstructed by the abutment. The convergence of these flows will create a highly turbulent flow pattern which is known as the primary vortex. The turbulent strength and erosive potential of the primary vortex can be characterized by the erosive power of water in the same manner as is recommended for the horseshoe vortex at bridge piers.

Conventional abutment scour equations are empirically related to the abutment shape, velocity and depth of flows in the main channel and overbank, and the abutment's alignment to the flow (Richardson, 1993; Melville 1991). Richardson (1993) states that these methods for determining scour at abutments will generally provide excessive scour estimates. He observed that this results from the fact that the equations were developed from laboratory data that failed to accurately model typical field conditions.

Practical application of abutment scour equations supports this observation because unreasonable abutment scour depth estimates are frequently obtained. Field observations suggest that in many conditions, energy dissipation at abutments is more likely to cause channel widening than vertical scouring. Richardson (1992) related this in his observations of a bridge failure in Virginia:

> *A 1972 flood on the James River in Virginia destroyed part of the approach and spill through sections of two abutments (C.F. Boles III, personal communication, 1991). There was little if any vertical scour of the bridge abutments and no collapse of either bridge span. The flood peak was around 360,000 cfs, which was*

in excess of the 100 year flood. For this failure, the L/y ratio was 23.8 and calculated (Melville's method) scour depths would be 166 feet.

(L represents the abutment length extended into the flow and y represents average depth of the approach flow)

This is a dramatic example of overly conservative vertical scour estimates and although excessive, it is consistent with many bridge abutment scour conditions that have been observed.

Consideration of Chang's concept of minimum stream power suggests that adjustments in channel geometry will result from the increased rate of energy dissipation at the abutment. These channel adjustments would be in the direction of least resistance. In the case of a spill abutment, channel widening would tend to dominate the scour process if riprap or embankment material were more easily eroded that the channel bed material. A rigid vertical wall abutment will be more resistant to lateral erosion than most naturally occurring materials. In this case, vertical scouring is expected to dominate.

No method for determining the rate of energy dissipation at bridge abutments is proposed at this time. It is hoped that an empirical relationship or equation can be developed to estimate stream power at bridge abutments. Application of the erodibility threshold criterion to abutment scour analysis will then be incorporated into the proposed scour prediction procedure. In the mean time, the reader is encouraged to follow the guidelines recommended by Richardson (1993) in HEC 18 for assessment of scour at abutments.

4. PRELIMINARY PROCEDURE TO PREDICT SCOUR IN SEDIMENTARY BEDROCK

This chapter presents detailed guidelines for predicting scour depths at bridge crossings. This preliminary scour prediction procedure is an incremental approach to determining scour depths. It relates the channel bed and foundation material properties which influence erodibility to the erosive power of water as characterized by stream power. It is simple and directly applied, and can be used to predict scour depth for bedrock and naturally occurring materials.

Implementation of this procedure requires basic hydraulic and geotechnical information. A subsurface investigation and collection of geotechnical data is needed to determine the Erodibility Index. Bridge geometry, pier dimensions, and a hydraulic analysis of the proposed structure is also needed.

A general outline of the scour prediction procedure and a detailed outline of the technique for estimating the Erodibility Index, contraction scour and pier scour are presented. An example problem is also provided to illustrate application of the procedure.

4.1 Procedure Outline

Table 4.1 shows the basic steps required for application of this scour prediction procedure. Major tasks are identified and shown in their logical sequence within the framework of the bridge design process.

4.2 Determination of the Erodibility Index

Defining the erodibility of channel bed and foundation materials is the first major step in the scour prediction procedure and accurately determining the factors which make up the Erodibility Index is essential. The data required to determine the four constitutive Erodibility Index factors is readily obtained from standard geotechnical field and laboratory tests. These field tests and collection of samples for laboratory tests can be accomplished during the subsurface investigation for bridge foundation design.

Worksheet #1 is provided in Appendix C to assist in collection and processing information required in the Erodibility Index. Rock material properties and conditions may indicate that significant weathering and degradation of Erodibility Index values will occur during the bridge's functional life. If this situation occurs, Erodibility Index values of rock material expected to degrade can be reduced to reflect the expected degree of weathering.

Table 4.1 Scour Prediction Procedure Outline

Step	Task	Comments
1	Hydrologic Analysis	Determine design discharges.
2	Preliminary Bridge Sizing	Includes preliminary assessment of contraction scour assuming sand bed channel.
3	Preliminary Structural Design	Determine structure type, location and size of piers.
4	Subsurface Investigation	Collect data and laboratory samples required to determine Erodibility Index.
5	Determine Erodibility Index	Use Worksheet #1. Erodibility Index values should be shown on boring logs.
6	Final Hydraulic Design	Confirm bridge size and reevaluate bridge hydraulics.
7	Compute Contraction Scour	Use Worksheet #2.
8	Compute Pier Scour	Use Worksheet #3.
9	Compute Abutment Scour	Refer to HEC 18, Richardson et al.(1993)
10	Final Structural and Foundation Design	Place foundations below depth at which they are susceptible to scour
11	Design Scour Countermeasures and Abutment Protection	Refer to HEC 18, Richardson et al.(1993)

4.3 Determination of Contraction Scour

After the Erodibility Index has been determined for each distinguishable layer of channel bed and foundation material and a hydraulic analysis of the proposed bridge structure has been completed, scour calculations can be performed.

Table 4.2 outlines the basic steps required for predicting ultimate contraction scour depths. Worksheet #2 is provided in Appendix C to assist in documentation of contraction scour calculations.

Table 4.2 Contraction Scour Procedure

Step	Task	Comments
1	Determine Initial Flow Conditions	Design discharge, flow depth, velocity and energy gradient through the bridge should be determined.
2	Determine Flow Conditions at Progressive Scour Depths	Rerun hydraulic analysis to determine flow depth, velocity and energy gradient for progressive scour at 0.5 m increments. Initial attempt should be 2.0 m into bedrock. If scour below this depth is predicted hydraulic analysis of deeper scour is required.
3	Estimate Stream Power for Initial and Progressive Scour Depths	$P = \gamma \, q \, S_f$
4	Determine Erodibility Threshold Values for Channel Bed and Foundation Material	Determine Erodibility Index for channel bed and foundation material at 0.5 m increments.
5	Determine Scourability at Incremental Depths	If stream power value exceeds erodibility threshold values for an increment then scour is predicted for that interval.
6	Determine Ultimate Contraction Scour Depth	Ultimate scour depth is predicted to occur at the lowest incremental layer which is considered erodible for design flow conditions.

4.4 Determination of Pier Scour

If the proposed bridge structure will have a pier, pier scour must also be predicted so that foundation elevations can be determined. Information regarding the pier's shape, width and length, and the angle of attack of approach flow is needed to initiate pier scour calculations. Pier scour calculations should be performed assuming piers will be located in the channel thalweg. This will account for lateral stream migration of the main channel which might occur during the structure's life. Table 4.3 outlines the basic steps required for predicting ultimate pier scour depths. Worksheet #3 is provided in Appendix C to assist in documentation of pier scour calculations.

Table 4.3 Pier Scour Procedure

Step	Task	Comments
1	Determine Initial Flow Conditions	Design discharge, flow depth, velocity and energy gradient through the bridge should be determined
2	Estimate Initial Stream Power	$P = \gamma\, q\, S_f$
3	Determine Pier Geometry	Pier shape, width and length and angle of attack of approaching flow should be determined.
4	Determine Pier Shape and Attack Angle Correction Factors	See Tables 3.1 and 3.2
5	Determine Erodibility Threshold Values for Channel Bed and Foundation Material	See Figure 2.2
6	Compute Relative Scour Depth for Incremental Depths	Relative scour depth is the ratio of scour depth to pier width.
7	Determine Pier Stream Power Ratio	Pier stream power ratio is the ratio of stream power at the bridge pier to initial steam power at the bridge. See Figure 3.1.
8	Determine Pier Stream Power	Pier Stream Power ratio is multiplied by initial stream power.
9	Apply Pier Shape and Attack Angle Correction Factors	Multiply Pier Stream Power by pier shape and attack angle correction factors.
10	Determine Scourability at Incremental depths	If pier stream power value exceeds erodibility threshold values for an increment then scour is predicted for that interval.
11	Determine Ultimate Pier Scour Depth	Ultimate scour depth is predicted to occur at the lowest incremental layer which is considered erodible for design flow.

4.5 Example Problem

A hypothetical bridge situation is presented to illustrate the application of the preliminary bridge scour prediction procedure. A spill abutment bridge is proposed to cross a waterway. A bridge channel section with a 15 meter bottom width and 2:1 side slopes is being considered. Preliminary structural design indicates that a single pier is required. Plans show that the pier will have a square face.

This structure will be designed to withstand the effects of flood flows up to 225 cubic meters/second. Preliminary hydraulic analysis was performed and flow velocity, depth and energy slope were determined to be 4.2 m/s, 3.5 m, and 0.025 m/m respectively.

A subsurface investigation and collection of channel bed and foundation samples has been performed. Subsurface conditions consisted of a 0.5 meter thick silty sand unit overlying a sandy gravel. The sandy gravel unit is approximately 0.5 meters thick and is overlying a 0.5 meter silty clay deposit and a 1.0 meter thickness of weathered shale. Soft shale bedrock was encountered at a depth of 2.5 meters below the channel bed surface.

A summary of geotechnical properties of channel bed and foundation materials is provided in Table 4.4. Calculation for the Erodibility Index, contraction scour and pier scour are shown in Table 4.5, Table 4.6 and Table 4.7, respectively.

Table 4.4 Example Problem - Summary of Geotechnical Properties

Depth (m)	Material Description	Observation/Test Results
0.0	Silty sand	SPT = 20 blows d_{50} = 0.002 m $\Phi = 33^0$
0.5	Sandy gravel	SPT = 40 blows d_{50} = 0.02 m $\Phi = 45^0$
1.0	Silty clay	No vane shear performed, silty clay has soft consistency $\Phi = 27^0$ No identifiable structure
1.5	Weathered shale	UCS = 2.0 MPa RQD = 6 Two joint fissure sets/plus random Joint Conditions - smooth, planer, tight, unaltered
2.5	Soft Shale	UCS = 5.0 MPa RQD = 50 Single joint set Joint Conditions - smooth, planer, tight, unaltered Dip Direction - towards flow Dip Angle = 30^0

Table 4.5 Example Problem - Erodibility Index Calculations

CDOT Scour Prediction Procedure
Erodibility Index - Worksheet #1

Project: Example
Boring No.: B-1
Date: 12/94
Geologist: SPS
Checked By: SPS

		Mass Strength Factor, Km				Particle/Block Size Factor, Kb				
(1) Depth	(2) Material Type	(3) Unconfined Compressive Strength (MPa)	(4) Standard Penetration Test	(5) Vane Shear Strength (KPa)	(7) Mass Strength Factor Km	(8) Rock Quality Designation RQD	(9) Joint set number Jn	(10) Joint Count Number Jc	(11) D50 (m)	(12) Particle/Block Size Factor Kb
0.00	silty sand		20.00		0.09				0.0020	8.00E-06
0.50	sandy gravel		40.00		0.15				0.02	8.00E-03
1.00	silty clay			soft	0.04					1.00
1.50	weathered shale	2.00			0.84	6.00	2.24			2.67
2.00	weathered shale	2.00			0.84	6.00	2.24			2.67
2.50	shale	5.00			4.50	50.00	1.25			40.00
3.00										
3.50										
4.00										

		Interparticle Bond Strength Factor, Kd				Relative Shape and Ground Structure Factor, Ks				
(1) Depth	(2) Material Type	(13) Joint Roughness Number	(14) Joint Alteration Number	(15) Residual Friction Angle	(16) InterparticleBond Strength Factor Ks	(17) Dip Direction (degrees)	(18) Dip Angle (degrees)	(19) Ratio of Joint Spacing r	(20) Relative Shape and Ground Str. Factor	(21) Erodibility Index Kn
0.00	silty sand			33.00	0.65				1.00	4.67E-07
0.50	sandy gravel			45.00	1.00				1.00	0.0012
1.00	silty clay			27.00	0.51				1.00	0.0204
1.50	weathered shale	1.00	1.00		1.00				1.00	2.2428
2.00	weathered shale	1.00	1.00		1.00				1.00	2.2428
2.50	shale	1.00	1.00		1.00	180.00	30.00	1:1	0.49	88.2000
3.00										
3.50										
4.00										

(1) Depth below channel bed surface.
(2) As described on boring log.
(3) Unconfined compressive strength from laboratory test ASTM D-2938.
(4) Standard penetration test
(5) Vane shear strength
(6) In situ deformation modulus
(7) Mass strength factor Km as determined from Tables A-1, A-2, A-3, and A-4.
(8) Rock quality designation
(9) Joint set number, from Table A-5
(10) Joint count number, from Table A-4
(11) Average particle diameter for granular soils
(12) Particle/block size factor, column (8) divided by column (9)
(13) Joint roughness number, from Table A-7
(14) Joint alteration number, from Table A-8
(15) Residual friction angle in degrees
(16) Interparticle/Bond strength Factor, from Equations 2.5 or 2.6
(17) Dip direction relative to flow
(18) Dip angle relative to flow
(19) Ratio of joint spacing
(20) Relative shape and Orientation Factor
(21) Erodibility index, product of columns (7), (12), (16) and (20)

Bridge Scour Prediction Procedure - Contraction Scour
Worksheet #2

Project:	Example
Date:	12/94
Engineer:	SPS

Bridge Hydraulic and Hydrologic Data :

Design Discharge -	Q =	225 m^3/s
Bridge Width -	w =	15 m
Unit Weight of Water-	γ =	9.8 kN/m^3
Unit Discharge -	q =	15 m^/s*m
Flow Depth -	y =	3.5 m
Flow Velocity -	V =	4.2 m/s
Initial Energy Slope -	Sf =	0.025 m/m
Initial Flow Area -	An =	55 m^2
Initial Stream Power -	P =	3.6 kW/m

(1) Scour Depth (m)	(2) Erodibility Index, Kn	(3) Erodibility Threshold (kW/m)	(4) Energy Slope	(5) Stream Power (kW/m)	(6) Scour Yes/No
0.00					
	4.67E-07	0.00008	0.025	3.6	Y
0.50					
	0.0012	0.005	0.021	2.41	Y
1.00					
	0.02	0.10	0.017	2.05	Y
1.50					
	2.24	2.00	0.014	1.76	N
2.00					
	2.24	2.00	0.012	1.51	N
2.5					
	88.20	30.00	0.01	1.37	N
3					
3.5					
4					
	Estimated Contraction Scour =			1.5 m	

(1) Depth Below stream bed, incremental depths should be adjusted to reflect changes in material Erodibility Index.
(2) From Worksheet #1.
(3) From Figure 2.2.
(4) Reduced energy slope through the bridge due to cumulative scour.
(5) Adjusted stream power using energy slope from column (4).
(6) If stream power in column (5) exceeds the material's erodibility threshold in column (3), the increment is predicted to scour. Continue analysis until a non-erodible increment is reached.

Table 4.6 Example Problem - Contraction Scour Calculations

Bridge Scour Prediction Procedure - Pier Scour
Worksheet #3

Project:	Example Problem
Date:	12/94
Engineer:	SPS

Bridge Hydraulic and Hydrologic Data :

Design Discharge -	Q =	225.00	m^3/s
Channel Width -	w =	15.00	m
Unit Weight of Water-	γ =	9.80	kN/m^3
Unit Discharge -	q =	15.00	m^3/s*m
Flow Depth -	y =	3.50	m
Flow Velocity -	V =	4.20	m/s
Initial Energy Slope -	Sf =	0.025	m/m
Initial Stream Power -	P =	3.60	kW/m

Pier Geometry:

Shape -	Square		
Angle of Attack -	θ =	15.00	degrees
Pier Length -	L =	8.00	m
Pier Width -	b =	1.00	m

Correction Factors :

Pier Nose Shape -	K1 =	1.10	(From Table 3.1)
Attack Angle -	K2 =	2.00	(From Table 3.2)

(1) Scour Depth ys (m)	(2) Erodibility Index, Kn	(3) Erodibility Threshold (kW/m)	(4) Relative Scour Depth, ys/b	(5) Pier Stream Power Ratio	(6) Pier Stream Power (kW/m)	(7) Corrected Pier Stream Power (kW/m)	(8) Scour Yes/No
0.00	4.67E-07	8.00E-05	0.50	2.30	8.28	18.22	Y
0.50	0.0012	5.00E-03	1.00	1.50	5.40	11.88	Y
1.00	0.020	0.10	1.50	1.30	4.68	10.30	Y
1.50	2.240	2.00	2.00	1.20	4.32	9.51	Y
2.00	2.240	2.00	2.50	1.10	3.96	8.72	Y
2.50	88.200	30.00	3.00	1.00	3.60	7.92	N
3.00							
3.50							
4.00							

Predicted Pier Scour Depth = 2.50 m

(1) Depth Below stream bed, incremental depths should be adjusted to reflect changes in material Erodibility Index.
(2) From Worksheet #1.
(3) From Figure 2.2.
(4) Ratio of scour depth, ys, to pier width, b.
(5) From Figure 3.2.
(6) Pier stream power, divide stream power ratio in column (5) by initial stream power.
(7) Adjusted stream power pier correction factors K1 and K2.
(8) If adjusted stream power in column (8) exceeds the material's erodibility threshold in column (3), the increment is predicted to scour. Continue analysis until a non erodible increment is reached.

Table 4.7 Example Problem - Pier Scour Calculations

4.5.1 Example Problem Discussion

Analysis using the proposed scour prediction procedure follows a logical sequence and a solution is readily provided. The example problem although hypothetical, does illustrate application of the procedure.

Table 4.5 shows calculation of the Erodibility Index using Bridge Scour Prediction Procedure Worksheet #1. Its completion requires the performance of a number of common geotechnical field and laboratory tests. Results from these tests are used to determine the major erodibility factors of mass strength number, particle/block size number, interparticle bond strength factor, and the relative shape and ground structure number. These factors are estimated either from calculations or directly from relationships tabulated by Kirsten (1982) and the product of these factors provides the material's Erodibility Index value. Collection of data for the Erodibility Index is easily accomplished at the same time as the subsurface investigation for foundation design and should not cause significant increases in design cost. Subsurface materials are subdivided at distinct changes in lithology and Erodibility Index values at increments of no greater than 0.5 meters. Observations of the data presented in Table 4.5 suggest that the most dominant factor in the Erodibility Index is the mass strength factor.

Scour Prediction Procedure Worksheet #2 is shown in Table 4.6. This worksheet is used to predict ultimate contraction scour depths. Prediction of ultimate contraction scour requires computation of energy slope and resulting stream power for progressive, incremental scour depths. These stream power values are compared against erodibility threshold values and ultimate contraction scour is predicted to occur where the erodibility threshold exceeds the stream power. Table 4.7 shows Scour Prediction Procedure Worksheet #3 with calculation of pier scour depths. Pier scour depths are determined much the same as for contraction scour. Stream power at the pier is estimated from Figure 3.1 as a function of scour depth, pier width, and initial flow conditions. Correction factors for pier shape and flow attack angle are multiplied by the stream power predicted from Figure 3.1 and compared against erodibility threshold values.

Abutment scour is not calculated. Equations and guidelines provided in HEC 18 (Richardson et al., 1993) can be used to estimate vertical abutment scour depths. Consideration of the relative erodibility of abutment material and channel bed material should be given in predicting if vertical scouring or channel widening will predominate.

5. SUMMARY AND CONCLUSIONS

There is currently no equation or method of bridge scour prediction in practice which implicitly considers the ability of bedrock or cohesive and consolidated materials to resist erosion.

A preliminary procedure is presented for predicting scour depths at bridge crossings. This procedure relates the ability of channel bed and foundation material to resist erosion due to the erosive power of water. It can be used to predict scour depths for bedrock or cohesive and consolidated materials. The basic methodology used in this scour prediction procedure is proposed by Annandale (1993; 1995). He recommended the use of Kirsten's material classification system, the Erodibility Index (Kirsten, 1982), for rating erodibility of materials subject to flowing water. This classification system quantifies the properties which influence a material's ability to resist erosion into a single representative value.

Annandale (1993; 1995) related stream power to Kirsten's Erodibility Index. He presented the results of a comparative analysis between the erosive power of water and the erodibility of emergency spillways. A relationship between stream power and Kirsten's Erodibility Index (Annandale, 1993; 1994; and Kirsten, 1982) was developed based on Soil Conservation Service (SCS) field data from more than 150 observations of scour at emergency spillways. Materials ranged from cohesionless soil to bedrock. By comparison of material and flow conditions under which spillway erosion occurred and those under which erosion did not occur, an erodibility threshold limit was established between erosive and non-erosive conditions. This erodibility threshold is the critical stream power above which a given material is predicted to scour.

Preliminary methods for calculating stream power at bridge contractions under free surface and pressure flow conditions are presented. An interim empirical relationship is provided for estimating the rate of energy dissipation and scour at bridge piers. No method for estimating stream power or predicting scour depths at bridge abutments is provided.

The procedure for estimating scour at bridges is an incremental approach. Stream power is predicted for incremental depths of scour. This stream power is compared against the erodibility threshold values for the material which is encountered at each incremental depth. Ultimate scour depths are predicted to occur where material is encountered with an erodibility threshold exceeding the stream power.

It is very difficult to explicitly account for every variable which influences the physical process of bridge scour. It is essential though, to ensure that the dominant variables are considered. This scour prediction method does consider the two most important parameters in scour analysis; a measure of material erodibility and a parameter characterizing the erosive power of water.

The scour prediction method presented is based on preliminary review of available data and literature. It has not yet been field verified or calibrated with laboratory data. There is a need to collect and plot experimental and field scour depths against stream power and Kirsten's Erodibility Index. Use of this preliminary procedure should consider these factors and judgement should be exercised in its application. Additional data will help to improve the interim stream power/pier scour relationship, develop a relationship between stream power and abutment scour, and determine the level of efficiency of stream power in the scour process. Numerical analysis may be useful in simultaneously determining flow patterns and the mechanics of erosion and aid in procedure calibration. The priority of any research will be verification of the procedure with field data. It is hoped that verification and calibration of the scour prediction procedure will be accomplished. With refinement of this procedure and experience in its use, this procedure should develop into a valuable tool in scour analysis of bedrock and naturally occurring channel bed and foundation materials.

Experiments to initiate calibration and refinement of the proposed procedure are planned for summer 1995, at the FHWA Hydraulics Laboratory in McClean, Virginia. Analysis of scour data is also proposed and results of these studies will be incorporated into a final report.

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Appendix A - Erodibility Index

Sources: Kirsten, H.A.D., *A Classification System for Excavation in Natural Materials,* The Civil Engineer in South Africa, pp. 292 to 308, July, 1982.

and Annandale, G.W., *Analysis of Complex Scour Problems in Rock Other Earth Materials,* 1993.

Note: Any minor changes to tables in Appendix A from Kirsten's original tables are to reflect nomenclature used in Annandale's discussions.

Table A-1: Mass Strength Number for Granular Soil (K_m)

Consistency	Identification in Profile	SPT blow count	Mass strength number (K_m)
Very loose	Crumbles very easily when scraped with geological pick	0 - 4	0.02
Loose	Small resistance to penetration by sharp end of geological pick	4 - 10	0.04
Medium Dense	Considerable resistance to penetration by sharp end of geological pick	10 - 30	0.09
Dense	Very high resistance to penetration of sharp end of geological pick - requires many blows of pick for excavation	30 - 50	0.19
Very Dense	High resistance to repeated blows of geological pick - requires power tools for excavation	50 - 80	0.41

Note:A granular material in which the SPT blow count is larger than 80 shall be taken as rock, for which the hardness can be obtained from Table A-3.

Table A-2: Mass Strength Number for Cohesive Soil (K_m)

Consistency	Identification in Profile	Vane shear strength (kPa)	Mass strength number (K_m)
Very soft	Pick head can easily be pushed into the shaft of handle. Easily molded by fingers	0 - 80	0.02
Soft	Easily penetrated by thumb; sharp end of pick can be pushed in 30mm - 40mm; molded by fingers with some pressure	80 - 140	0.04
Firm	Indented by thumb with effort; sharp end of pick can be pushed in up to 10mm; very difficult to mould with fingers. Can just be penetrated with an ordinary hand spade	140 - 210	0.09
Stiff	Penetrated by thumbnail; slight indentation produced by pushing pick point into soil; can not be molded by fingers. requires hand pick for excavation	210 - 350	0.19
Very Stiff	Indented by thumbnail with difficulty; slight indentation produced by blow of pick point. Requires power tools for excavation	350 - 750	0.41

Note: A cohesive material of which the vane shear strength is larger than 750 kPa shall be taken as rock, for which the hardness can be obtained from Table A-3.

Table A-3: Mass Strength Number for Rock (K_m)

Consistency	Identification in Profile	Unconfined compressive strength (MPa)	Mass strength number (K_m)
Very soft rock	Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife; it is too hard to cut a triaxial sample by hand	1.7 1.7 - 3.3	0.87 1.86
Soft rock	Can just be scraped and peeled with a knife; indentations 1mm to 3mm show in the specimen with firm (moderate) blows of the pick point	3.3 - 6.6 6.6 - 13.2	3.95 8.39
Hard rock	Can not be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of a geological pick with a single firm (moderate) blow	13.2 - 26.4	17.7
Very hard rock	Hand-held specimen breaks with hammer end of pick under more than one blow	26.4 - 53.0 53.0 - 106.0	35.0 70.0
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material	106.0 - 212.0 212.0	140.0 280.0

Note: For UCS < 10 MPa

$$K_m = 0.78\ (UCS)^{1.09}$$

For UCS > 10 MPa

$$K_m = UCS$$

Table A-4: Mass Strength Number for Detritus (K_m)

Consistency	Identification in Profile	In situ deformation modulus[1] (MPa)	Mass strength number (K_m)
Very loose	Detritus very loosely packed. High percentage of voids and very easily dislodged by hand. matrix crumbles very easily when scraped with a geological pick. ravelling often occurs in excavated faces	0 - 4	0.02
Loose	Detritus loosely packed. some resistance to being dislodged by hand. Large number of voids. matrix shows small resistance to penetration by sharp end of geological pick	4 - 10	0.05
Medium dense	Detritus closely packed. Difficult to dislodge individual particles by hand. Voids less apparent. matrix has considerable resistance to penetration by sharp end of geological pick	10 - 30	0.10
Dense	Detritus very closely packed and occasionally very weakly cemented. can not dislodge individual particles by hand. The mass has a very high resistance to penetration by sharp end of geological pick - requires many blows to dislodge particles	30 - 80	0.21
Very Dense	Detritus very densely packed and usually cemented together. The mass has a high resistance to repeated blows with a geological pick - requires power tools for excavation	80 - 200	0.44

Note: 1. determined by plate bearing test of diameter 760 mm
2. A detritus of which the in situ deformation modulus exceeds 200 Mpa shall be taken as the lowest boulder formation

Table A-5: Joint count number (J_c)

Number of joints per cubic meter (J_c)	Ground quality designation (RQD)	Number of joints per cubic meter (J_c)	Ground quality designation (RQD)
33	5	18	55
32	10	17	60
30	15	15	65
29	20	14	70
27	25	12	75
26	30	11	80
24	35	9	85
23	40	8	90
21	45	6	95
20	50	5	100

Table A-6: Joint set number (J_n)

Number of joint sets	Joint set number (J_n)
Intact, no or few joint/fissures	1.00
One joint/fissure set	1.22
One joint/fissure set plus random	1.50
Two joint/fissure sets	1.83
Two joint/fissure sets plus random	2.24
Three joint/fissure sets	2.73
Three joint/fissure sets plus random	3.34
Four joint/fissure sets	4.09
Multiple joint/fissure sets	5.00

Note: For intact granular material take $J_n = 5.00$

Table A-7: Joint roughness number (J_r)

Joint Separation	Condition of Joint	Joint roughness number (J_r)
Joints/fissures tight or closing during excavation	Discontinuous joint/fissures Rough or irregular, undulating Smooth undulating Slickensided undulating Rough or irregular, planar Smooth planar Slickensided planar	4.0 3.0 2.0 1.5 1.5 1.0 0.5
Joint/fissures open and remain open during excavation	Joints/fissures either open or containing relative soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation Shattered or micro-shattered clays	1.0 1.0

Note: For intact granular material take $J_r = 3.0$

Table A-8: Joint alteration number (J_a)

Description of gouge	Joint Alteration Number (J_a) for joint separation (mm)		
	< 1.0[1]	1.0-5.0[2]	> 5.0[3]
Tightly healed, hard, non-softening impermeable filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or rock crushed filling	2.0	4.0	6.0
Non-softening, slightly clayey, non cohesive filling	3.0	6.0	10.0
Non-softening strongly over consolidated clay mineral filling, with or without crushed rock	3.0*[4]	6.0*	10.0*
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0	13.0
Softening moderately over consolidated clay mineral filing, with or without crushed rock	4.0*	8.0*	13.0*
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0	18.0

Note: 1. Joint walls effectively in contact.
2. Joint walls come into contact after approximately 100mm shear.
3. Joint walls do not come into contact at all upon shear.
4. Values asteriked added to Barton's data.

Table A-9: Relative ground structure number (K_s)

Dip direction[1] of closer spaced joint set (degrees)	Dip angle[2] of closer spaced joint set (degrees)	Ratio of joint spacing, r			
		1:1	1:2	1:4	1:8
180/0	90	1.00	1.00	1.00	1.00
0	85	0.72	0.67	0.62	0.56
0	80	0.63	0.57	0.50	0.45
0	70	0.52	0.45	0.41	0.38
0	60	0.49	0.44	0.41	0.37
0	50	0.49	0.46	0.43	0.40
0	40	0.53	0.49	0.46	0.44
0	30	0.63	0.59	0.55	0.53
0	20	0.84	0.77	0.71	0.68
0	10	1.22	1.10	0.99	0.93
0	5	1.33	1.20	1.09	1.03
0/180	0	1.00	1.00	1.00	1.00
180	5	0.72	0.81	0.86	0.90
180	10	0.63	0.70	0.76	0.81
180	20	0.52	0.57	0.63	0.67
180	30	0.49	0.53	0.57	0.59
180	40	0.49	0.52	0.54	0.56
180	50	0.53	0.56	0.58	0.60
180	60	0.63	0.67	0.71	0.73
180	70	0.84	0.91	0.97	1.01
180	80	1.22	1.32	1.40	1.46
180	85	1.33	1.39	1.45	1.50
180/0	90	1.00	1.00	1.00	1.00

Note:
1. Dip direction of closer spaced joint set relative to direction of rip
2. apparent dip angle of closer spaced joint set in vertical plane containing direction of ripping
3. For intact material take $K_s = 1.0$
4. For values of r less than 0.125 take K_s as for r = 0.125

Appendix B - Analysis of Data

B-1 Analysis of Laboratory Data for Shear Stress at Base of Bridge Pier (Johnson et al., 1993)

Data presented by Johnson et al. (1993) was analyzed to assess the relationship between scour hole depth at the base of a circular pier to the rate of energy dissipation of flowing water within the hole. Johnson performed model experiments to determine the relationship of boundary shear stress in a scour hole to the shear stress in the approach flow. Boundary shear was determined indirectly by placing uniform diameter marbles with known critical shear in preformed scour holes and exposing it to flowing water until movement was observed. Johnson measured both pier geometry and the characteristics of the approach flow. Sufficient data was presented to derive stream power within the pier scour hole and for the approach flow. This data is tabulated in table B-3 and the relationship between relative scour depth and the stream power ratio is expressed graphically in figures B-1 and B-2. B-2 is provide to extrapolate from Johnson's data to scour/pier width ratios between 1.0 and 3.0. This extrapolation is base on the observation by Richardson et al. (1993) that equilibrium scour is reached when the ratio of scour depth to pier width approaches 3.0.

A description of the calculations performed in each column of table B-3 follows:

Data for columns (1), (2), (5), and (6) were provided in Johnson's data.

Relative scour depth is provided in column (4). This is the ratio of the preformed scour hole depth (y_s) to the pier width (b).

Stream power per unit boundary area of the approach flow is shown in column (7). It is computed using the following equation -

$$P_a = \tau_a V_a$$

where: P_a = [(ft lb/sec)/ft^2]
τ_a = approach flow shear stress, [lb/ft^2]
V_a = approach flow velocity, [ft/sec]

Column (8) shows the ratio of stream power estimated to occur in the scour hole at failure to the stream power of the approach flow -

Stream Power ratio = P_p / P_a

The stream power in the vicinity of the pier is the product of the critical shear stress (τ_c) and the critical velocity (V_c) of the material used in the experiment and is represented by -

$$P_p = \tau_c V_c$$

where: τ_c = 0.052 lb/ft^2, computed by Johnson
V_c = 2.0 ft/sec

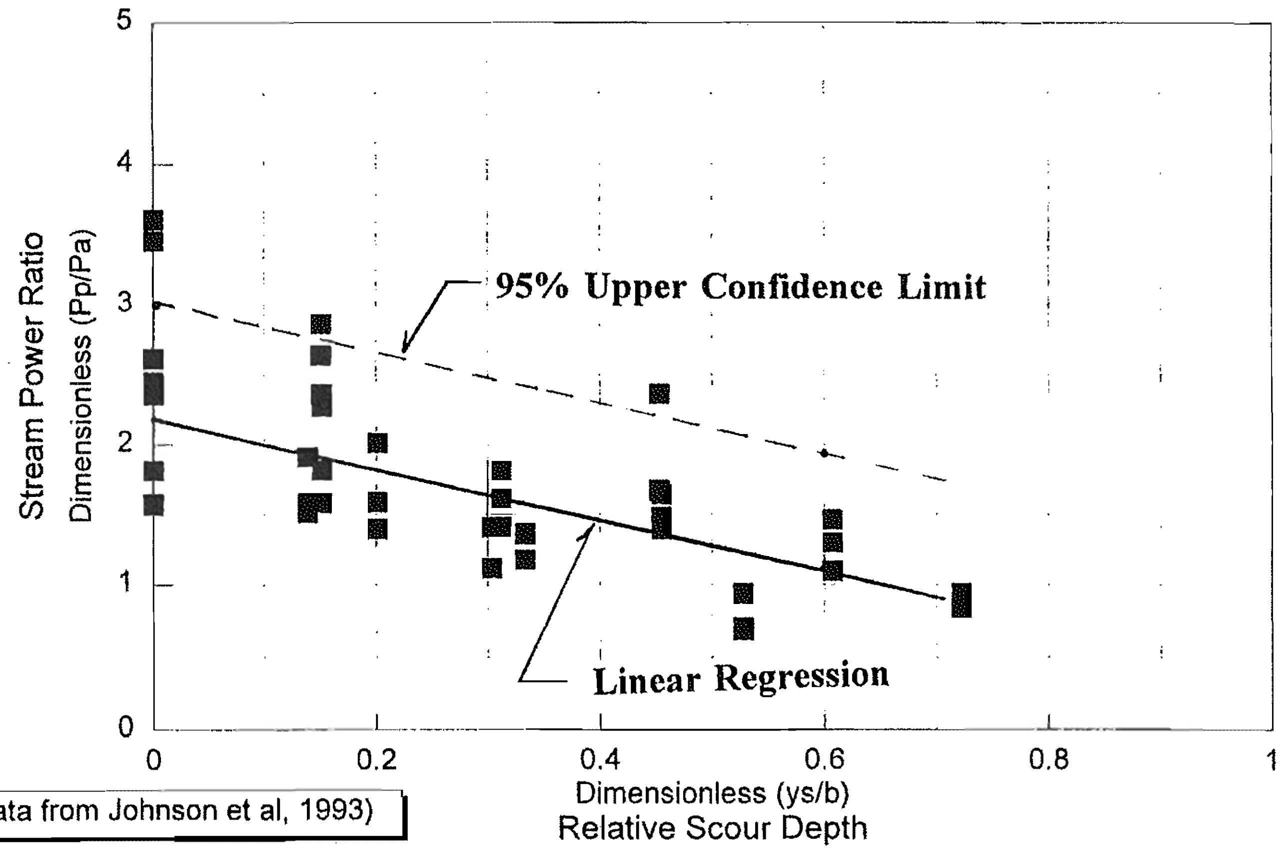
Critical velocity of the marbles was not documented in Johnson's study. It was computed by trial and error.

The relationship between relative scour depth and the stream power ratio is shown in figure B-1. A relationship between the two is indicated that confirms that stream power decreases as the scour hole increases. Observations of the data plotted in figure B-1 indicate a relationship between relative scour depth (y_s/b) and stream power ratio (P_p/P_a) but there is insufficient data to provide a conclusive relationship for practical application to scour analysis.

An interim relationship is proposed until additional data can be plotted and the relationship refined and verified. This interim relationship is shown on figure B-2. The proposed limit utilizes the 95% upper confident limit based on a linear regression analysis within the limits of Johnson's data. Beyond the limits of Johnson's data the limit was extrapolated to the point where an equilibrium rate of energy dissipation (stream power) is expected.

Table B-1

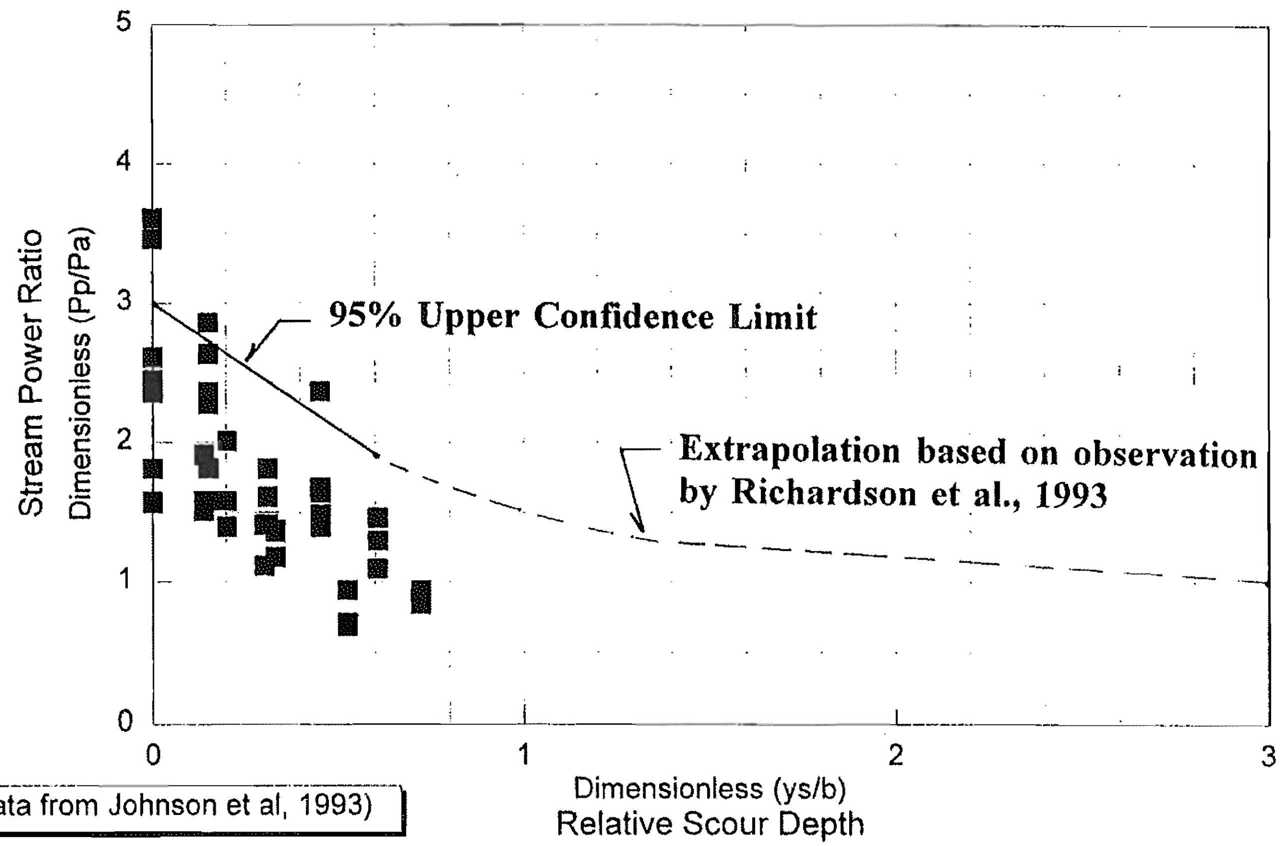
Pier Scour Ratio vs Relative Scour Depth



(Data from Johnson et al, 1993)

Table B-2

Pier Scour Ratio vs Relative Scour Depth



(Data from Johnson et al, 1993)

Table B-3 Laboratory Data for Shear Stress Within a Scour Hole
(Information acquired or derived from Johnson et al., 1993)

(1) Pier Width b (feet)	(2) Scour Depth ys (inches)	(3) Scour Depth ys (feet)	(4) Relative Scou ys/b	(5) Approach Velocit Va (fps)	(6) Approach Shear Stre to (lb/sq. ft.)	(7) Approach Stream Po Pa (ft lb/sec)	(8) Stream Power Ratio
0.3750	0.0000	0.0000	0.0000	1.8330	0.0360	0.0660	1.5760
0.3750	0.0000	0.0000	0.0000	1.7960	0.0370	0.0665	1.5650
0.3750	0.0000	0.0000	0.0000	1.6900	0.0340	0.0575	1.8100
0.3750	0.6250	0.0521	0.1389	1.8290	0.0360	0.0658	1.5795
0.3750	0.6250	0.0521	0.1389	1.7010	0.0320	0.0544	1.9106
0.3750	0.6250	0.0521	0.1389	1.7680	0.0390	0.0690	1.5083
0.3750	1.5000	0.1250	0.3333	1.9140	0.0400	0.0766	1.3584
0.3750	1.5000	0.1250	0.3333	1.9610	0.0450	0.0882	1.1785
0.3750	1.5000	0.1250	0.3333	1.8510	0.0410	0.0759	1.3704
0.3750	2.3750	0.1979	0.5278	2.1280	0.0520	0.1107	0.9398
0.3750	2.3750	0.1979	0.5278	2.2750	0.0650	0.1479	0.7033
0.3750	2.3750	0.1979	0.5278	2.2640	0.0670	0.1517	0.6856
0.3750	3.2500	0.2708	0.7222	2.1310	0.0520	0.1108	0.9385
0.3750	3.2500	0.2708	0.7222	2.1410	0.0560	0.1199	0.8674
0.3750	3.2500	0.2708	0.7222	2.1300	0.0580	0.1235	0.8418
0.5490	0.0000	0.0000	0.0000	1.6150	0.0270	0.0436	2.3850
0.5490	0.0000	0.0000	0.0000	1.5740	0.0270	0.0425	2.4472
0.5490	0.0000	0.0000	0.0000	1.5760	0.0280	0.0441	2.3568
0.5490	1.0000	0.0833	0.1518	1.6340	0.0280	0.0458	2.2731
0.5490	1.0000	0.0833	0.1518	1.7790	0.0370	0.0658	1.5800
0.5490	1.0000	0.0833	0.1518	1.6860	0.0340	0.0573	1.8142
0.5490	2.0000	0.1667	0.3036	1.8950	0.0390	0.0739	1.4072
0.5490	2.0000	0.1667	0.3036	1.8450	0.0400	0.0738	1.4092
0.5490	2.0000	0.1667	0.3036	1.9390	0.0480	0.0931	1.1174
0.5490	3.0000	0.2500	0.4554	1.8110	0.0350	0.0634	1.6408
0.5490	3.0000	0.2500	0.4554	1.8660	0.0400	0.0746	1.3934
0.5490	3.0000	0.2500	0.4554	1.7990	0.0390	0.0702	1.4823
0.5490	4.0000	0.3333	0.6072	1.8750	0.0380	0.0713	1.4596
0.5490	4.0000	0.3333	0.6072	1.9110	0.0420	0.0803	1.2958
0.5490	4.0000	0.3333	0.6072	1.9800	0.0480	0.0950	1.0943
0.8280	4.5000	0.3750	0.4529	1.6290	0.0270	0.0440	2.3645
0.8280	4.5000	0.3750	0.4529	1.7730	0.0350	0.0621	1.6759
0.8280	4.5000	0.3750	0.4529	1.7390	0.0360	0.0626	1.6612
0.8280	3.1000	0.2583	0.3120	1.8910	0.0390	0.0737	1.4102
0.8280	3.1000	0.2583	0.3120	1.7940	0.0360	0.0646	1.6103
0.8280	3.1000	0.2583	0.3120	1.6890	0.0340	0.0574	1.8110
0.8280	2.0000	0.1667	0.2013	1.9080	0.0390	0.0744	1.3976
0.8280	2.0000	0.1667	0.2013	1.6690	0.0310	0.0517	2.0101
0.8280	2.0000	0.1667	0.2013	1.7750	0.0370	0.0657	1.5836
0.8280	1.5000	0.1250	0.1510	1.6340	0.0270	0.0441	2.3573
0.8280	1.5000	0.1250	0.1510	1.5150	0.0240	0.0364	2.8603
0.8280	1.5000	0.1250	0.1510	1.5180	0.0260	0.0395	2.6350
0.8280	0.0000	0.0000	0.0000	1.4410	0.0200	0.0288	3.6086
0.8280	0.0000	0.0000	0.0000	1.4310	0.0210	0.0301	3.4608
0.8280	0.0000	0.0000	0.0000	1.5300	0.0260	0.0398	2.6144

LINEAR REGRESSION RESULTS

Independent (x) variable: Rel Sco
Dependent (y) variable: SP Rati

Source	Sum of Squares	Deg. of Freedom	Mean Squares	F-ratio
Model	3.92153	1	3.92153	25.284
Error	5.27331	34	.15510	
Total (- mean)	9.19483	35		

Coefficient of Determination (R^2): .426
Correlation Coefficient (R): .653
Standard Error of Estimate (s): .3938
Coefficient of Efficiency (E): .581

Linear Regression Equation is: Y = -1.7972 * X + 2.1859

Appendix C - Bridge Scour Prediction Procedure Worksheets

CDOT Scour Prediction Procedure
Erodibility Index - Worksheet #1

Project:
Boring No.:
Date:

Geologist:
Checked By:

		Mass Strength Factor, Km				Particle/Block Size Factor, Kb				
(1) Depth	(2) Material Type	(3) Unconfined Compressive Strength (MPa)	(4) Standard Penetration Test	(5) Vane Shear Strength (KPa)	(7) Mass Strength Factor Km	(8) Rock Quality Designation RQD	(9) Joint set number Jn	(10) Joint Count Number Jc	(11) D50 (m)	(12) Particle/Block Size Factor Kb
0.00										
0.50										
1.00										
1.50										
2.00										
2.50										
3.00										
3.50										
4.00										

		Interparticle Bond Strength Factor, Kd				Relative Shape and Ground Structure Factor, Ks				
(1) Depth	(2) Material Type	(13) Joint Roughness Number	(14) Joint Alteration Number	(15) Residual Friction Angle	(16) InterparticleBond Strength Factor Ks	(17) Dip Direction (degrees)	(18) Dip Angle (degrees)	(19) Ratio of Joint Spacing r	(20) Relative Shape and Ground Str. Factor	(21) Erodibility Index Kn
0.00										
0.50										
1.00										
1.50										
2.00										
2.50										
3.00										
3.50										
4.00										

(1) Depth below channel bed surface.
(2) As described on boring log.
(3) Unconfined compressive strength from laboratory test ASTM D-2938.
(4) Standard penetration test
(5) Vane shear strength
(6) In situ deformation modulus
(7) Mass strength factor Km as determined from Tables A-1, A-2, A-3, and A-4.
(8) Rock quality designation
(9) Joint set number, from Table A-5
(10) Joint count number, from Table A-4
(11) Average particle diameter for granular soils
(12) Particle/block size factor, column (8) divided by column (9)
(13) Joint roughness number, from Table A-7
(14) Joint alteration number, from Table A-8
(15) Residual friction angle in degrees
(16) Interparticle/Bond strength Factor, from Equations 2.5 or 2.6
(17) Dip direction relative to flow
(18) Dip angle relative to flow
(19) Ratio of joint spacing
(20) Relative shape and Orientation Factor
(21) Erodibility index, product of columns (7), (12), (16) and (20)

Bridge Scour Prediction Procedure - Contraction Scour
Worksheet #2

Project:
Date:
Engineer:

Bridge Hydraulic and Hydrologic Data :

Design Discharge -	Q =	m^3/s
Bridge Width -	w =	m
Unit Weight of Water	γ =	kN/m^3
Unit Discharge -	q =	m^/s*m
Flow Depth -	y =	m
Flow Velocity -	V =	m/s
Initial Energy Slope -	Sf =	m/m
Initial Flow Area -	An =	m^2
Initial Stream Power -	P =	kW/m

(1) Scour Depth (m)	(2) Erodibility Index, Kn	(3) Erodibility Threshold (kW/m)	(4) Energy Slope	(5) Stream Power (kW/m)	(6) Scour Yes/No
0.00					
0.50					
1.00					
1.50					
2.00					
2.5					
3					
3.5					
4					
	Estimated Contraction Scour =				**m**

(1) Depth Below stream bed, incremental depths should be adjusted to reflect changes in material Erodibility Index.
(2) From Worksheet #1.
(3) From Figure 2.2.
(4) Reduced energy slope through the bridge due to cumulative scour.
(5) Adjusted stream power using energy slope from column (4).
(6) If stream power in column (5) exceeds the material's erodibility threshold in column (3), the increment is predicted to scour. Continue analysis until a non-erodible increment is reached.

Bridge Scour Prediction Procedure - Pier Scour
Worksheet #3

Project:
Date:
Engineer:

Bridge Hydraulic and Hydrologic Data :

Design Discharge -	Q =	m^3/s
Channel Width -	w =	m
Unit Weight of Water-	γ =	kN/m^3
Unit Discharge -	q =	m^3/s*m
Flow Depth -	y =	m
Flow Velocity -	V =	m/s
Initial Energy Slope -	Sf =	m/m
Initial Stream Power -	P =	kW/m

Pier Geometry:

Shape -	Square	
Angle of Attack -	θ =	degrees
Pier Length -	L =	m
Pier Width -	b =	m

Correction Factors :

Pier Nose Shape -	K1 =	(From Table 3.1)
Attack Angle -	K2 =	(From Table 3.2)

(1) Scour Depth ys (m)	(2) Erodibility Index, Kn	(3) Erodibility Threshold (kW/m)	(4) Relative Scour Depth, ys/b	(5) Pier Stream Power Ratio	(6) Pier Stream Power (kW/m)	(7) Corrected Pier Stream Power (kW/m)	(8) Scour Yes/No
0.00							Y
0.50							Y
1.00							Y
1.50							Y
2.00							Y
2.50							N
3.00							
3.50							
4.00							
				Predicted Pier Scour Depth =			m

(1) Depth Below stream bed, incremental depths should be adjusted to reflect changes in material Erodibility Index.
(2) From Worksheet #1.
(3) From Figure 2.2.
(4) Ratio of scour depth, ys, to pier width, b.
(5) From Figure 3.2.
(6) Pier stream power, divide stream power ratio in column (5) by initial stream power.
(7) Adjusted stream power pier correction factors K1 and K2.
(8) If adjusted stream power in column (8) exceeds the material's erodibility threshold in column (3), the increment is predicted to scour. Continue analysis until a non erodible increment is reached.