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MECHANICS OF LOCAL SCOUR
DISCUSSION AND BIBLIOGRAPHY

PART I

<u>Discussion</u> - Status of Knowledge on Local Scour

by

S. S. Karaki

and

R. M. Haynie

Prepared for

U. S. Department of Commerce
Bureau of Public Roads
Division of Hydraulic Research
under Contract CPR 11-8022

Civil Engineering Section
Colorado State University
Fort Collins, Colorado

November 1963 CER63SSK46

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FOREWORD

The mechanics of local scour is a study undertaken with the sponsorship of the U. S. Department of Commerce, Bureau of Public Roads, Division of Hydraulic Research, for the purpose of developing mathematical equations for scour from basic principles of fluid mechanics. The study is planned under a three-year program. In the first year a thorough review of literature was to be made pertaining to local scour, along with a study of the fundamental equations for three-dimensional flow and scour process. The experimental facilities were to be designed and constructed and if possible, testing of some simple geometric shapes of obstructions were to be undertaken. During the second and third years, development of the basic equations for scour are to be completed.

This report constitutes the terminal report for the first year. The accomplishments are listed as follows:

1. Review of available literature, domestic and foreign, has been completed and an evaluation of the state of knowledge on local scour is presented.
2. Study of the theoretical approach to local scour is underway, but no equations are presented in this report. Several technical discussions have been held with the co-project leader and advisors concerning the theoretical study. From these discussions a method of approach to the theoretical study has evolved. The theoretical study will begin with development of equations describing the flow and distribution of velocity around a circular cylindrical pier in horizontal flow for conditions of fixed boundary geometry. The scour pattern will define the boundary geometry, and experimental measurements of velocity and pressure distributions will be made to supplement the theoretical study. These velocity and pressure measurements will be made by "fixing" the boundary at successive periods of time during the scour process until equilibrium conditions are established. From

the resulting theoretical equation for scour depth as a function of time, various other obstruction geometries will be studied in similar manner, and through successive adaptation, evolve a generalized equation for local scour.

3. The experimental flume has been designed and is practically completed, the pump unit for the flume will be delivered by the manufacturer in January. The facility consists of a 60 ft long flume 6 ft wide and 2 ft deep with a recessed test section 15 ft long located 30 ft downstream from the upstream end. The approach distance of 30 ft was required to enable establishment of proper flow and sediment transport conditions at the test section. The equipment is powered with a multiple speed pump and is designed in such a way as to enable control of discharge, depth and water surface level in the flume so that the starting and stopping of flow at various times can be effected with no disturbance of the boundary geometry.
   Colorado State University has participated in the construction of the equipment by providing funds and materials.
4. Tests of simple obstruction geometries were not conducted in this first year, however, these tests shall be undertaken as soon as the equipment is completed.

The authors wish to express their appreciation to the following individuals for their contribution and direct assistance in the conduct of this study in this first year; to Drs. Yevdjevich, Simons and Cermak for their advice and guidance, and to Mr. Markovic, Dr. Binder and Mr. Yano for their assistance in translation of foreign literature. Appreciation is expressed also to the Library Staff at Colorado State University, the staff of the U. S. Geological Survey at Colorado State University, the U. S. Bureau of Reclamation Technical Library in Denver, the

Denver Public Library, the Libraries at the Universities of Colorado, California at Berkeley and Stanford, and the Bureau of Public Roads offices in Washington and Denver, for assistance in acquisition of many references.

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NOMENCLATURE

Note: This Nomenclature applies only to the first part of this report. It does not apply to the Bibliography and Abstracts.

a - experimental coefficient

b - width of obstruction measured normal to the mean flow path

c - coefficient

$\overline{c}$ - mean concentration

c' - coefficient

d_s - scour depth measured below normal bed level

e - base of natural logarithm

f - Lacey silt factor

g - gravitational acceleration

h - tailwater depth

j - portion of bed area exposed to shear

k - coefficient of velocity distribution

k_o - velocity head downstream from the pier

ℓ - mixing length of the jet

m - exponent

n - Manning's roughness coefficient

o - rates between longest and shortest diameter of sand particles

p - fraction of bed material of diameter ∂

q - unit discharge

q_c - critical discharge causing sediment movement

r - radius of stream line curvature

s - specific gravity of sand particles

t - time

v - point velocity

$\overline{v}$ - mean velocity in a vertical

$\overline{v}_s$ - mean sediment velocity

w - fall velocity of the sediment particles

y - variable flow depth

z - afflux at the obstruction

A -

B - canal width

C - empirical constant

D - depth of flow

D_L - Lacey flow depth

D_s - scour depth measured from the water surface

F - Froude number

G - transport rate of sediment per foot of width

H - total height of fall

K - multiplier to regime depth greater than 1

L - sediment load

M - uniformity modulus

P - wetted perimeter

Q - total discharge

Q_c - total discharge at which transport begins

R - hydraulic radius

S - canal slope

S_e - slope of the energy gradient

T - hydraulic force

V - average velocity

V_B - bottom velocity

V_* - shear velocity - $\sqrt{g R S}$

V_{*c} - critical shear velocity for sediment movement

W - weight of stone

δ - thickness of laminar sublayer

α - channel contraction ratio

β - pier shape coefficient

γ - specific weight of water

γ_s - specific weight of sediment

∂ - sediment diameter

ρ - fluid density

ρ_s - sediment density

σ - standard deviation of the time variation of v

ν - kinematic fluid viscosity

τ_o - shear at the bed

$\bar{\tau}_o$ - average shear of the bed in a reach

τ_c - critical shear at the bed for incipient motion

Ψ - a coefficient depending on sediment size

ϕ_f - friction angle of sand grains

θ - angle of repose of sediment

ϕ - channel side slope angle

Φ - boundary description

MECHANICS OF LOCAL SCOUR
STATE OF KNOWLEDGE AND BIBLIOGRAPHY

by

S. S. Karaki and R. M. Haynie

INTRODUCTION

Progress towards complete understanding of water and sediment flow has been relatively slow, considering that man through the ages has been concerned with changing and controlling water courses to his benefit. Available historical evidence indicates that it was not until near the end of the nineteenth century before systematic methods were utilized to solve water-sediment problems. Nor is it accidental that this should correspond with the beginning of a period of rapid development in fluid mechanics and the science as we know it today.

The science of river hydraulics, although historically originating from ancient times, received much interest and thus impetus in the late nineteenth century. Early methods involved appraising existing structures in the field, noting the successes as well as failures and assembling from collections of data, satisfactory solutions to describe the various phenomena. The procedure is basically sound but inherently requires long periods of time for data collection and analysis. An example is the regime theory. Although limitations exist in its application to water courses in general, the regime theory contributed fundamentally to the concept that alluvial channels transporting given amounts of water and sediment, and given sufficient time, will establish equilibrium conditions of width, depth and slope.

The turn of the twentieth century brought forth a surge of interest in quantitative analysis of alluvial channel problems. The number and variety of solutions applicable to sediment transport, channel stabilization and local scour problems are almost as numerous as the investigators themselves. Conscious of the excessively long periods of time required to collect sensible quantities of field data, researchers concentrated on small scale models to reduce the time element and very rapidly, field methods were superceded by improved laboratory flumes and dimensional techniques. This method was practiced almost exclusively into the mid-twentieth century, and indeed it is still a popular approach to water-sediment problems. The current trend in analysis is to utilize the knowledge developed in fluid mechanics, analyze the problem mathematically and derive proportionality and other constants from laboratory data. Laboratory techniques, particularly instrumentation for collection of data, are also rapidly advancing and will help fulfill the need for more accurate data to assist in the analysis. Use of dimensional analysis and laboratory data alone does not always produce useful solutions to sediment problems, primarily because proper model-prototype relationships are not established for alluvial channels.

It is the purpose of this report to assemble historical and recent references to publications on local scour and prepare brief abstracts of these references, and from this survey of literature it is the aim to establish a current status of knowledge on local scour. The report therefore is prepared in two parts, Part I Discussion - Status of Knowledge on Local Scour and Part II Bibliography. No new theory will be advanced in this report, for it is intended that an investigation be conducted to study the mechanics of local scour in the immediate period ahead.

II. DISCUSSION ON LOCAL SCOUR

Any list of references to local scour has the option of including or excluding references to other water-sediment problems. Local scour however is not an isolated problem, and hence, the option has been exercised in this report to include some references, albeit arbitrary, to sediment transport, channel stabilization and general scour.

A. Local Scour Process

Local scour is defined as the process of removing more alluvial material at the bed or banks of channels, in a relatively small area within the channel, than that which is supplied to the same area. Local scour is caused by increase in local velocities and turbulence which increases local transport rate from the area to be greater than the supply rate. Local scour is a time-variant phenomenon.

If there is no sediment transport to the region of local scour, removal of material will continue until the change in boundary geometry reduces the shear forces and turbulence and a balance is achieved between the active and resistive forces on the sediment particles within the area of local scour. In other words, with zero sediment supply, changes in geometry of the boundary will occur until the sediment output is zero. For this condition, many investigators have shown experimentally that the depth of scour increases as an exponential function of time. Theoretically then, the scour hole never reaches a condition where sediment output is zero. Of course, in practical terms, beyond a certain period of time when increase in scour depth is insignificant, the scour can essentially be considered as having reached a limit. Neither the period of time nor the increment of scour depth from the limit are defined. Also, under natural conditions, stream discharge and velocity vary with time, and these cause additional changes of scour depth with time.

If the alluvial channel transports sediment to the scour region, changes in boundary geometry in the area of local scour will take place until the velocity and depth, hence shear, and turbulence reaches magnitudes sufficient to substantially balance incoming and outgoing sediment. For this condition, the term "equilibrium scour depth" is applied. The rate of sediment transport in the channel is generally expressed as a time-averaged value. The total sediment load should include both suspended and bed loads as their is a constant exchange of particles between the modes of transport, and the rate of exchange depends upon local shear and turbulence characteristics of the flow. The transport mode of suspended sediment is easily visualized. Bed load however, is more difficult to fisualize, and hence, to define, but for the purpose herein, bed load shall be loosely distinguished as the sediment transported in close proximity to and in contact with the bed of the channel. The configuration of the channel bed, in longitudinal section, may contain waves of various amplitudes and lengths or it may be a plane, depending upon flow, fluid and sediment characteristics. If the bed load is transported in waves, the rate of incoming sediment load to the area of local scour would vary appreciably depending upon whether the sand wave reaching the area was a crest or a trough. If the sediment supply to the scour hole is large as it would be if a crest of a sand wave were to cascade into the scour hole, then over a short span of time the rate of supply would exceed the rate of outflow and the scour hole would fill. Conversely, if the sand wave at the edge of the scour hole was a trough, the sediment supply would be less and the transport rate out of the scour hole would exceed the transport rate into the scour hole and a deepening of the scour hole would result. This constant imbalance between sediment inflow and outflow from an area of local scour and corresponding fluctuations in scour depth leads to the concept of equilibrium depth of scour, or a time-averaged depth of scour.

Should the bed load be transported as plane bed, which is probably more generally the condition for rivers in floods, the transport rate of sediment to an area of local scour may not fluctuate nearly so radically as it does with sand waves.

B. Regime Theory

The concept and definition of the term, regime, as applied to alluvial channels is attributed to R. G. Kennedy as expressed in his publication in 1895. His regime formula was based upon observations and data taken from 22 selected canals of the Punjab, in India. The formula is

$$V = 0.84\, D^{0.64} \,, \tag{1}$$

where V is the mean canal velocity and D is the depth of flow in foot-pound-second units. A channel is said to be in regime when for a given quantity of water and sediment, an equilibrium geometry of the canal in terms of width, depth and slope is achieved. The term equilibrium implies time-average mean values which permits certain finite changes about a mean.

The Kennedy regime equation has been altered to be more adaptable for canal design. One change was expressed by E. S. Lindley (1919). While Kennedy believed that bed width had no relationship to depth except with discharge, Lindley offered the following formulae using the same data,

$$V = 0.95\,D^{0.57} \tag{2}$$

$$V = 0.57\,B^{0.355}\ , \tag{3}$$

and consequently, that

$$B = 3.8\,D^{1.61}\ , \tag{4}$$

where B is canal width in the foot-pound-second units. The difference between equations (1) and (2) is defended with the fact that under a given set of flow conditions and sediment discharge, there is some latitude of velocity between incipient scour and incipient deposition. In establishing equation (3) and hence, equation (4), Lindley believed that two canals carrying the same water discharge but different sediment discharge would assume different regime geometry. The canal transporting the larger sediment load would be wider and shallower and the canal slope would be steeper.

Other refinements are made, not only to Kennedy's equation but also to the formulae by Lindley. Having made the observation that different average velocities are required to scour the alluvial channel bed and to maintain sediment particles in motion, Fortier and Scobey (1926) propounded that silting and scouring involve two fundamentally different processes. In the first instance the opposing force was believed to be due only to gravity and in the second instance the movement of the particles was believed to be dependent on gravitational force and geometry of the particles. In his publication, G. Lacey (1929) introduces a silt factor to account for sediment discharge in a canal as being a dominant factor in determining the canal geometry. Later (1934) he expresses formulae in terms of the silt factor;

$$V = 1.151\,(fR)^{1/2} \tag{5}$$

$$P = 2.67\,Q^{1/2} \tag{6}$$

$$S = \frac{f^{5/3}}{1.788\,Q^{1/6}} \tag{7}$$

where f is silt factor, P is wetted perimeter, Q is total discharge and S is canal slope in foot-pound-second units. Further modifications and refinement were offered by Bose, Blench, Griffith, Inglis, Wood and others.

The complexity of alluvial channel behavior precludes either simplification of conditions or statement of precise conditions for which the regime equations apply. In order to use the equations satisfactorily, the engineer must be acquainted with the variable conditions of the channels from which the theory originated and be able to visualize similarities for design and compare conditions for analysis. These abilities are generally included in the meaning of the term experience, and in order to expect any degree of competence in the use of regime equations, this experience is mandatory.

C. Application of the Regime Theory to Local Scour

In 1927 W. M. Griffith applied the regime formulae, somewhat controversially, to rivers, reasoning that the coefficient of 0.84 in equation (1) is dependent upon the quantity and character of the sediment being transported and assumed that it may vary from 0.84 to 1.09, the larger values applying to rivers in flood. Assuming the coefficient is equal to 1 for sake of discussion, equation (1) leads to

$$V = D^{0.64}\ .$$

If velocity for non-scouring condition cannot safely exceed say 5 ft per sec., then D is limited to about 12 ft, and rivers must adopt wide shallow sections. Where the normal width of the river is unduly contracted, for instance at bridge sites, the water surface slope increases locally because of the afflux, or backwater, created and the velocity will increase and exceed the safe scour velocity, hence, scour must occur. Or alternatively from the Kennedy equation, an increase in V must mean an increase in depth. The scour depth, d_s, at contractions then can be estimated from

$$D_2 = \left(\frac{B_1}{B_2}\right)^{0.61} D_1 , \tag{8}$$

where B_1 and B_2 are widths of the normal river and contraction respectively and D_1 is regime depth for the normal river. The depth of scour is then

$$d_s = D_2 - D_1 . \tag{9}$$

The Lacey formulae expressed in a slightly different manner yields

$$D = 0.47\left(\frac{Q}{f}\right)^{1/3} \tag{10}$$

for regime depth D, or alternatively

$$D = C\left(\frac{q^2}{f}\right)^{1/3} , \tag{11}$$

in which C is a constant and q is unit upstream discharge.

Equation (11) was applied to results of model experiments of the Hardinge bridge piers, (1939). In this study, models with different geometric scales of 1:40, 1:65, 1:105 and 1:210 were used. The study was motivated by failure of the bridge piers due to abnormally deep scour. From the studies, it was concluded that scour depth could be calculated from the equation

$$\frac{D_s}{b} = 1.70\left(\frac{q^{2/3}}{b}\right)^{0.78} \tag{12}$$

where scour depth D_s is measured from the water surface and b is the width of the pier. The difference is exponent of q between equations (11) and (12) is explained by the fact that depth of scour is influenced by the width of the pier in relation to depth of flow. Equation (12) however, was not used extensively by bridge designers. They resorted instead to relating scour depth with Lacey's regime depth, equation (10), by a multiple factor. That is,

$$D_s = K D_L \quad , \quad K > 1 \tag{13}$$

in which D_s and D_L are both measured from the water surface. Lacey in 1945 expresses scour depth downstream from barrages in this manner, and Inglis who developed equation (12) recommended the following criteria for design:

$$D_s = 2\, D_L \, , \quad \text{for scour at bridge piers} \tag{14}$$

$$D_s = 4\, D_L \, , \quad \text{downstream of bridges} \tag{15}$$

$$D_s = 3.6\, D_L \, , \quad \text{at spur heads} \tag{16}$$

$$D_s = 2.75\, D_L \, , \quad \text{at long radius spur} \tag{17}$$

D. Critical Tractive Force Theory

The concept of shear force, acting on an alluvial channel bed was introduced by du Boys (1897). He assumed that the sediment moved by sliding in layers due to uniform shear or tractive force, and determined a relationship for rate of sediment transport (bed load),

$$G = \frac{\Psi}{\gamma^2} \tau_o (\tau_o - \tau_c) \tag{18}$$

where
G = the transport rate of sediment per foot of width (lbs/sec),
Ψ = a coefficient dependent on sediment size,
τ_o = shear at the bed (lb/ft^2)
τ_c = critical shear at the bed for beginning of motion (lb/ft^2)
γ = specific weight of water.

The average value of shear in a reach was expressed as

$$\bar{\tau}_o = \gamma RS \tag{19}$$

where
$\bar{\tau}_o$ = average shear
R = hydraulic radius (ft)
S = hydraulic slope (ft/ft)

While admittedly the assumption upon which it is based is oversimplified, and neglects the mechanisms of turbulence and boundary layer flow, equation (18) has been essentially substantiated by Straub (1935), Chang (1939) and others. Straub utilized du Boys tractive force concept together with the Manning formula and developed the relationship

$$G = \frac{n^{1.2}\, S^{1.4}\, q^{0.6}}{1.609}\, \gamma \left(q^{0.6} - q_c^{\ 0.6}\right) , \tag{20}$$

wherein n is Manning's roughness coefficient, q is unit discharge and q_c is critical discharge for sediment movement.

A modification of the tractive force concept was introduced by Kalinske in the light of improved knowledge of turbulent flow. In considering bed movement, it is important to recognize not only the fluid force to which a particle must be subjected in order to cause it to move, but the fluctuation of the force due to turbulence. Through rational analysis, utilizing experimental data to determine certain coefficients, the expression for transport rate of sediment per unit of width is given as

$$G = \frac{2}{3}\, \bar{v}_s\, j\, \partial\, \gamma_s \ . \tag{21}$$

In equation (21), $\bar{v}_s$ is the mean sediment velocity and is approximately equal to the difference between flow velocity and critical velocity for movement, j is the portion of the bed area exposed to shear, ∂ is sediment diameter and γ_s is the specific weight of sediment. By assuming that the time variation of flow velocity v follows the normal-error law, it is shown that $\frac{\bar{v}_s}{\bar{v}}$ is a function of $\frac{\tau_c}{\tau_o}$ and $\frac{\sigma}{\bar{v}}$, where $\frac{\sigma}{\bar{v}}$ is the relative intensity of turbulence and σ is the standard deviation of the time variations in v. From the boundary-layer theory, $\bar{v} = 11\sqrt{\tau_o/\rho}$, where ρ is mass fluid density, and re-arrangement of equation (21) gives

$$G = \frac{2}{3}\, f\left(\frac{\tau_c}{\tau_o}\right) j\, \partial\, \gamma_s \sqrt{\frac{\tau_o}{\rho}} \tag{22}$$

where $f\left(\frac{\tau_c}{\tau_o}\right)$ is read as "function of" and is given graphically as a function of $\frac{\bar{v}_s}{\bar{v}}$ and $\frac{\sigma}{\bar{v}}$.

Shields (1936) determined that the relationship of the tractive force to the resistive force of the bed is a universal function of the ratio of grain size to thickness of the laminar boundary layer and developed the relationship

$$\tau_o = f\left(\frac{V_* \partial}{\nu}\right)(\gamma_s - \gamma)\partial \quad , \tag{23}$$

in which $\left(\frac{V_* \partial}{\nu}\right)$ is the shear Reynold's number, ν is kinematic viscosity of water, $(\gamma_s - \gamma)$ is the submerged weight of the sediment particles, and $V_* = \sqrt{gRS}$ is the shear velocity, where g is gravitational acceleration. The functional relationship is expressed graphically using experimental data.

In 1950 H. A. Einstein published the results of a long term study on sediment transport in alluvial channels. The analysis assumes uniform flow in a reach of stable alluvial channel. The resistance to flow is divided into two parts; (1) resistance due to sand grain roughness and (2) resistance due to form roughness of the irregular bed forms. He assumes that the roughness of the sand grains, or shear at the bed, transforms flow energy to turbulence which affects the individual grains, hence, bed load transport, but that part of the total flow energy which is affected by shape resistance causes turbulence at a considerable distance away from the grains, and does not affect the bed load transport. By using available knowledge on turbulent fluctuations and the boundary layer he developed a relationship for calculating suspended load, believed to be valid from the water surface to a close proximity of the bed. By using added knowledge of probability theory on the exchange of particles from the bed-layer to the stationary bed, and from large amounts of flume and field data, he developed a bed load equation which requires use of empirical graphical functions to solve. The bed load equation especially is believed to be valid for a specific range of sediment sizes but deviates from measured data when sediment size is large. There have been numerous modifications to the Einstein bed load function, aimed at unifying solutions for all sediment sizes and to be applicable to suspended load transport as well. These studies will not be included in this discussion. Einstein believed that the complex nature of sediment transport defied a unified solution.

A further modification of the tractive force theory was expressed by Iwagaki and Tsuchiya in 1956. Not unlike Kalinske's analysis, the shear force was considered from consideration of equilibrium forces on spherical grains utilizing modern concepts of turbulence and boundary layer theories. A dimensionless function of critical tractive force is given as

$$\frac{V_{*_c}{}^2}{\left(\frac{\rho_s}{\rho} - 1\right) g\, \partial \tan \phi_f} = f\left(\frac{V_{*_c}\, \partial}{\nu}, n\right) \tag{24}$$

where V_{*_c} is critical shear velocity, ρ_s is sediment density and ϕ_f is the friction angle of the sand grains.

Laursen developed an equation for total sediment load carried by streams using experimental and available field data. From qualitative analysis of the mechanics of sediment transport he selects the factors which are most likely to determine the rate of sediment transport. These are, (1) the shear velocity/fall-velocity ratio, $\sqrt{\frac{\tau_o}{\rho}}\ w$, (2) tractive force $\tau_o = \frac{V^2\, \partial^{1/3}}{30\, D^{1/3}}$, (3) critical tractive force, $\tau_c = c\, \partial$ where c is a coefficient dependent on sediment properties, (4) ratio of velocity of sediment particles in motion on the bed to fall velocity. The equation is

$$\bar{c} = \Sigma \rho \left(\frac{\partial}{D}\right)^{7/6} \left(\frac{\tau_o}{\tau_c} - 1\right) f\left(\frac{\sqrt{\frac{\tau_o}{\rho}}}{w}\right) \tag{25}$$

where $\bar{c}$ is mean concentration ρ = fraction of bed material of diameter ∂ and $f\left(\frac{\sqrt{\tau_o/\rho}}{w}\right)$ was presented graphically. The method presumes to give both quantity and quality of the total, suspended and bed loads as functions of the basic hydraulic characteristics of the stream and characteristics of the bed material.

E. Application of the Critical Tractive Force Theory to Local Scour

The critical tractive force theory has not been used extensively to describe local scour. The reason lies principally in determining the direction and magnitude of the velocity in the scour region. The three dimensional nature of local scour presents considerable difficulty in developing a reliable mathematical expression for the flow in the region.

Tsuchiya utilized the dimensionless concept of critical tractive force in his study of scour downstream from the end of a horizontal apron. His relationship for scour criterion is

$$\frac{V_{*c}^{2}}{\left(\frac{\rho_s}{\rho} - 1\right) g\, \partial \tan \phi_f} = \frac{4}{3}\, \phi_i \qquad i = 1,2,3\ldots, \tag{26}$$

where $\phi_i = f\, \frac{V_{*c}\, \partial}{\nu}$. The analysis depends on experimental determination of ϕ_i .

Laursen applies equation (25) to develop scour depth equations at abutments and piers of bridges by considering the flow through a long contraction. His expression for scour depth in the contracted region is

$$\frac{d_s}{D} = \left(\frac{Q_t}{Q_c}\right)^{6/7} - 1 \;, \tag{27}$$

where Q_t is total discharge, Q_c is discharge confined to the main channel and D is depth of flow in the upstream main channel. By assuming that scour in a long contraction is a fraction $1/r$ of the scour at the abutment d_s , equation (27) is rewritten

$$\frac{Q_o}{Q_w}\,\frac{w'}{D_o} = 2.75\,\frac{d_s}{D_o}\left[\left(\frac{1}{r}\,\frac{d_s}{D_o} + 1\right)^{7/6} - 1\right] , \tag{28}$$

where Q_o is the combined flow at the overbank, $w' = 2.75\, d_s$ assumed to be the lateral extent of the scour hole, and D_o is the average flow depth in w' . The equation requires a trial and error solution for d_s . The depth of scour at the pier was presented graphically.

Some measure of success was achieved by Z. S. Tarapore (1962) who utilized the horizontal jet diffusion concept in potential flow to derive the magnitude of velocity in the scour region. Together with the equation of continuity and du Boys transport equation the expression for the depth of scour with time is

$$e^{4k\frac{\eta}{\ell}} = \left(\frac{16\,k^2 K_2}{\ell X_d}\ \frac{\Psi}{100\,\gamma^2}\ C_f^{\,2}\ \frac{\rho^2}{4}\ V^4\ e^{-4k\,\xi/\ell}\right) t + \ell\,, \qquad (29)$$

where k = coefficient of velocity diffusion,

η = depth of scour at any arbitrary point on the bed,

ℓ = mixing length from the edge of the scour hole,

K_2 = constant for describing scour hole profile,

X_d = horizontal distance of the scour hole at the plane of the bed,

Ψ = transport characteristic of the bed,

γ = specific weight of water,

C_f = friction coefficient,

ρ = density of water,

V = mean stream velocity,

e = base of natural logarithm,

ξ = distance the jet diffuses normal to flow direction,

t = time.

The scour-time relationship is described as occurring in three stages:

Stage 1. The rate of scour is rapid and non-logarithmic with time

Stage 2. The scour hole expands logarithmically with time

Stage 3. The scour depth reaches equilibrium with or without sediment supply. The solution of equation (29) is not entirely independent of the experimental data, since K and ξ/ℓ requires experimental determination. Sample computations are given for limiting depths at circular and elliptical piers.

Equation (29) is the most recent formula available for computation of local scour depth based on the tractive force concept. Some of the limitations of the equation are:

1. The velocity-depth relationship in the scour hole is expressed in terms of a diffusing jet, while other investigations have shown the existence of secondary circulation and vorticity in the neighborhood of scour, around bridge piers in particular.

2. Application has been only to very small models.

3. Even after determination of the scour depth, since it applies to models specifically, a time-sediment scale relationship as required before it can be applied confidently to the prototype.

Nevertheless, the approach to the problem of local scour is based on rational concepts and is consistent with the recent trend for solution of water sediment problems.

F. Other Theoretical Approaches

There is no theoretical approach to transport of sediment and scour significantly different from the tractive force concept as modified by turbulence and boundary layer theories. However, some theoretical expressions for the velocity function has been presented which are worthy of mention. Tison, (1937) visualized scour as resulting from development of secondary currents due to curvature of the stream lines around an obstruction in the stream. Utilizing the vertical velocity distribution along a stream line and assuming potential flow, he derived an expression similar to the Bernoulli equation

$$D_B + \frac{p_B}{\gamma} + \frac{1}{g}\int_A^B \frac{V^2}{r}\, d_s = D_A + \frac{p_A}{\gamma} \tag{30}$$

where $\frac{V^2}{gr}$ is the lateral water surface slope in the zone of curvilinear flow due to centrifugal force, and r is the radius of curvature of the stream lines. By applying the expression to the flow at the surface and bed, he concludes that a downward velocity component must exist at converging stream lines and an upward velocity must prevail at diverging stream lines. He does not relate the magnitude of the downward velocity to shear at the bed, but instead presents experimental results of the scour pattern to verify his theory. Subsequently, Ishihara utilized Tison's analysis to develop an expression for scour force,

$$K = \phi_c \frac{\gamma}{g} \frac{1}{\alpha(\alpha + 2)} \int_{r_1}^{r_2} \frac{V_\theta^2 D}{r} \, dr \; , \tag{31}$$

where K = scour force, and

α = is obtained from Laval-Rapp's equations for vertical velocity distribution.

If scour is proportional to scour force, then scour depth must be proportional to the radius of curvature of the stream lines. He does not elaborate further to develop expressions for scour for specific structures.

A theoretical analysis is presented by Bata based on the Eulerian equation of flow. The analysis was prepared principally to develop a physical interpretation of local scour around bridge piers. A logarithmic distribution of velocity in the approach flow is assumed and the partial differential Euler equation is solved by successive approximation, to describe the velocity distribution around a circular cylinder. The successive approximation approach is utilized in effect to adapt potential theories to three-dimensional flow. Having developed a quasi-three-dimensional equation for the velocity distribution, the author observes that the vertical velocity component possesses sufficient magnitude to cause scour around a pier and subsequently performs laboratory experiments, not so much to verify his flow equation as to develop an empirical relationship for scour at bridge piers using as others, standard flow properties.

From concepts generated along somewhat a novel approach, Moore and Masch also conclude that the downward velocity component along the front face of bridge piers is the principal cause of scour. Utilizing observations in wind tunnel experiments that a long cylinder placed in the flow region developed measurable vertical components of velocity along the stagnation line, and verifying this observation in a water tunnel with circular cylinders, they calculate possible magnitudes of vertical velocity at the front face of a bridge pier and find that it is approximately equal to the surface velocity of the upstream flow. Although scour relationships are not developed, presumably the tractive force theory can be utilized with the vertical velocity component. The investigators realized too, that secondary currents and vorticity generated at the face of the pier added to the total scour depth.

G. Experimental Methods

The number of investigators utilizing the experimental approach has exceeded by far those who have utilized a theoretical approach to the problem of local scour. The reason obviously is the complexity of a theoretical solution to water-sediment problems and the relative simplicity of deriving solutions from observed flow and geometric phenomena.

1. Scour prevention

Early experimental methods were concerned only with measures to prevent scour at the base of structures. In 1893 Engels conducted model experiments to determine the proper placement of rip rap around bridge piers. In this study he discovered that maximum scour depth occurred at the upstream nose of the pier, where formerly it had been assumed that scour was predominant at the downstream end. As a result of this observation, he recommended that rip rap be placed in a horseshoe shaped pattern around the pier, with the open end downstream.

Rip rap protection has not been entirely successful as for instance at a bridge across the Ganges in India, boulder pitching with sizes varying from 80 to 130 pounds was washed away during a flood. Increasing the gradation of the rip rap has improved its stability to some degree. If varying sizes of rock can be placed in successive layers beginning with the smallest sizes at the bottom and with the larger rocks exposed to the flow, further improvement in protection capability can be expected.

Flexible willow mats were used at the Memphis bridge across the Mississippi River and downstream from a concrete apron beneath a bridge across the Fraser River. Posey and Appel investigated use of flexible rock mats around the base of bridge piers. At the bridge site across the Ganges where failure of rip rap protection was experience, large concrete blocks were chained together, essentially to form a flexible mat.

Durand-Claye in his early experiments (1873) was concerned with reshaping the pier to prevent scour. Various simple geometric forms were tested and it was observed that a rectangular shaped pier created deepest scour and a triangular-nose pier with the apex pointed upstream resulted in a significant reduction of scour depth. Lenticular-shaped piers, circular, elliptical and parabolic shapes of various forms have also been tested in the laboratory for actual structures. A pier of novel shape was suggested by Tison, where the surface would be formed approximately according to the velocity distribution of the approaching flow. In this manner he contended that the vertical component of velocity would not be created and hence, scour arrested or prevented entirely. Investigations in France and Mexico have suggested installation of a pile or battery of piles upstream of each bridge pier so that material scoured from the base of the piles would deposit downstream against the piers. This idea would function moderately well for a short period of time if we consider that local scour occurs because the sediment capacity exceeds the sediment supply. By increasing the sediment supply temporarily from the material scoured around the piles, the scour at the pier should be retarded. However, as the scour around piles ceases to produce sufficient sediment quantities, scour would then proceed around the bridge piers.

An interesting method of scour prevention was studied in a laboratory in Yugoslavia where the flow at the the stagnation line of the pier was siphoned off. They used the concept that the driving flow at the front of the pier was responsible for scour to a large degree, and if the diving flow could be prevented, then scour should be reduced. Indeed, in the experiments actual reduction of scour was observed. However, the relatively large quantity of flow which needed to be siphoned made application to actual piers less practical.

Concrete aprons around the base of piers have not met with any great success, for scour at the downstream end of the apron undermines the concrete and very rapidly the apron is broken up and the effectiveness lost.

2. Experimental Studies on Scour

The experimental studies on local scour are concerned with developing criteria for designing depth of footings for structures in rivers. Systematic studies were made by varying the parameters of flow, fluid, geometry, and sediment. The general method involves assessing the properties most likely to be significant to the scour phenomenon and from tests, develop an equation for probable scour depth as a function of those properties. Existing theoretical knowledge and qualitative observations by previous investigators were used as guides to select the significant variables.

Among the earliest systematic study conducted was that in India in connection with model studies of the Hardinge bridge. From that study evolved an equation for calculating the scour depth which was previously discussed in Section C and presented as equation (12). Two different sizes of sand were used in three flumes of different widths with various discharges and depths.

An intensive study of scour around bridge piers and abutments was conducted at Iowa University beginning in 1948. The laboratory experiments concerned first a qualitative study to determine relative effects of different shapes of piers and type of abutment on the scour depths, followed by detailed study on the prediction of maximum scour depth as a function of velocity and depth of flow, geometry of the channel and structure and characteristics of the sediment material. Some studies were included to ascertain the effect of angle of flow with respect to the axis of the bridge. The conclusion from this study was that maximum scour depth was a function of the size and shape of piers and depth of flow, and there were no discernable effects of velocity of approach flow nor grain size of the material composing the bed. The conclusions are later qualified by Laursen (1961) to be applicable where general bed movement in the channel occurs. With no general bed movement involved, he states that velocity of flow and sediment size become important variables. Similar observations were made by Schwartz in 1925. While his experiments were not as extensive as those conducted at Iowa University, Schwartz discovered that initial scour rates varied for different sand sizes but the maximum depth of scour was about the same for all sand sizes.

A laboratory study of scour around bridge piers was conducted in Chatou, France (1956). Studies included roughness of the bed and distribution of velocity in addition to the other flow, sediment and geometric properties. Attempts were made to establish similitude relationships between laboratory models and protogype without much success. Some effort was given to establish a relationship between scour and shear as described by the Meyer-Peter bed load formula, again without success. No quantitative conclusions are given but the significant conclusions of the study were:

1. Scour depth reaches a maximum at a velocity near that corresponding to beginning of bed transport.
2. Angle of the flow with respect to the axis of the pier has appreciable effects on location and depth of maximum scour.

Studies conducted at Colorado State University on scour at bridge piers and abutments yielded an empirical equation

$$d_s = d_{s_M} \frac{1 - e^{-\frac{c_t}{t}}}{1 - \frac{t}{t_m}} \tag{32}$$

where d_s = depth of scour,

d_{s_M} = maximum depth of scour

e = base of natural logarithm

c_t = empirical coefficient

t_o = time factor = e^{-B}

t_m = time required for d_s to achieve d_{s_m}.

B = channel width

The maximum depth of scour is biven by

$$\frac{d_{s_M}}{D} = 0.3 + 2.15 \left(\frac{a}{D} \right)^{0.4} \tag{33}$$

where a is the width of obstruction measured normal to the approach flow.

Although model data can be represented fairly well by equations (32) and (33) no definite conclusions are reached for its applicability to field structures, again because of the lack of appropriate similitude relationships.

The most recent experimental solution for the problem of local scour is presented by Garde (1961). Through dimensional analysis he establishes that a non-dimensional scour depth can be related to

$$\frac{d_s}{D} = A\, \eta_1\, \eta_2\, \eta_3\, \frac{1}{\alpha}\, F^n \tag{34}$$

where A is an experimental constant = 4.

$\eta_1 = f(C_D)$, C_D = drag coefficient of the sediment

$\eta_2 = f(F , \frac{\ }{b})$, = pier length, b = pier width

$\eta_3 = f(F , C_{D_1})$, C_{D_1} = drag coefficient of the pier

F = Froude number of the approach flow.

α = opening ratio, $\frac{B - b}{B}$

The equation is shown to fit flume data but field verification is lacking.

III. SUMMARY

The mechanics of local scour is a complex phenomenon not subject to easy mathematical or laboratory solution. It involves knowledge of the hydrodynamics of flow around objects placed in open channel streams, and the interrelationships between fluid turbulence, boundary layer and sediment movement. Each experimental study has met with limited success chiefly because of the lack of appropriate model-prototype similitude relationships for alluvial channels. Some correlate well with other laboratory data, but none are confidently applicable to actual structures. Theoretical approaches have been few in the main, because of the complexity of the problem.

Collectively however, there are several significant observations and conclusions which can be stated. They are:

1. Local scour results where local shear forces exceed the resistive forces of the particles on the bed of alluvial channels.
2. Local scour continues until the change in boundary geometry is sufficient to reduce the shear to a value where capacity to remove sediment equals sediment input to the scour area.
3. Deepest scour at the base of piers occurs at the upstream nose. This is caused by the large vertical component of velocity created there in the immediate neighborhood of the stagnation line.
4. The development of the strong downward component of velocity and convergence of stream lines in the region creates secondary circulation on each side of the pier and the intensity of shear there is sufficient to remove the particles from the bed.
5. The turbulence and divergence of stream lines at the downstream end of a pier is not materially important to the scour phenomenon.