

SCOUR AROUND BRIDGE PIERS

G.R. Hopkins, R.W. Vance, and B. Kasraie



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Interim Report

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16. ^{Abstract} Available theories and prediction formulas on scour at bridge waterways are reviewed. Formulas that offer potential for prediction of scour around bridge piers are compared by reducing each formula to a non-dimensional form that includes Froude Number, the ratio of scour depth to pier width, and the ratio of stage to pier width. A field study to gather data on scour and related variables is described. The study is aimed at collecting field data in order to furnish a basis on which to compare scour prediction formulas. Four test sites are included in the study. These sites are located on: 1) the Red River in Shreveport, Louisiana 2) the Brazos River in Richmond, Texas 3) the Homochitto River near Brookhaven, Mississippi 4) the Ohio River in Lawrenceburg, Indiana An automatic instrumentation system that measures scour depth at three points around a bridge pier as well as river stage is used in this study. The system is based on a depth measuring fathometer. The test protocol and study philosophy are discussed. Data gathered from the field sites are presented and compared to values predicted from scour formulas. Recommendations are made on ways to improve the scour research effort. These include improved instrumentation systems, additional field studies, laboratory studies and computer program development.		
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PREFACE

This interim report is submitted to the Federal Highway Administration in compliance with Article II of Modification Number 7 to U. S. Department of Transportation Contract Number DOT-FH-11-7759. Included in this report is the progress made in the study of "Scour Around Bridge Piers" during the first three years of contract which covers the period from July 1, 1971 to July 1, 1974.

This report was prepared by Dr. Gordon R. Hopkins, Project Principal Investigator, Mr. Richard W. Vance, Project Research Engineer, and Mr. Behzad Kasraie, Graduate Assistant, Engineering Experiment Station, West Virginia University, Morgantown, West Virginia.

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LIST OF SYMBOLS

a	= length of the obstruction (abutment) normal to the flow direction
b	= width of the pier
b*	= width of the pier projected on a plane normal to undisturbed flow
B	= average width between banks under regime condition
C _c	= factor defining cross-sectional shape of channel
C _d	= coefficient of drag
C _g	= factor defining plain geometry of channel
C _p	= parameter representing mechanical properties and specific gravity of the sediment
C _r	= parameter representing stream roughness
C _w	= parameter representing stream geometry
d	= diameter of bed material
d _r	= regime depth
D	= scour depth measured from mean bed elevation
D*	= scour depth measured from water surface
f	= Lacey silt factor = $1.76\sqrt{d_m}$
F	= Froude No. of the channel = $V/\sqrt{gH}$
F _b	= bed factor
F _p	= pier Froude No.
g	= gravitational acceleration
G	= weight of the bedload transported per unit time and width
h	= river bed height above a reference level
H	= flow depth (stage)
k/k'	= ratio of the coefficient of roughness of the bed and the grain roughness of the surface.
L	= length of the control volume section
N _s	= sediment number
q	= discharge intensity of fluid
q _s	= bedload discharge intensity after the obstruction has been added
Q	= discharge of the fluid
r	= ratio of the depth of scour at pier or abutment to the depth of scour in a fictitious long constriction
R	= pier Reynolds number
Re	= flow Reynolds number
Re*	= particle or shear Reynolds number
R _h	= hydraulic radius
s	= specific gravity of the sand
S	= slope of the channel
t	= time
T	= temperature
V	= flow velocity
V*	= shear velocity
V _f	= fall velocity of particle
V̄	= volume
W	= width of the channel
W _{ws}	= width of the water surface
α	= particle shape factor
γ	= specific weight of the fluid

SYMBOLS (Continued)

η = factor representing channel roughness
 μ = viscosity of fluid
 ρ = density of fluid
 σ = sediment concentration
 τ = shear stress between flowing fluid and loose boundary

Subscripts:

cr = signifies the critical value of the quantity
e = signifies the equilibrium value of the quantity
m = signifies the mean value of the quantity
max = signifies the maximum value of the quantity
n = signifies the normal value of the quantity
s = signifies the value of the quantity evaluated for bed load sediment
ss = signifies the value of the quantity evaluated for suspended sediment
st = signifies the total value of the quantity (bed load the suspended load)
in = signifies the quantity entering a section
out = signifies the quantity leaving a section
1 = signifies the value of the quantity in unconstricted section
2 = signifies the value of the quantity in constricted section

SUMMARY

The need for field data to validate existing scour prediction formulae has been recognized by many researchers. The National Cooperative Highway Research Program, Synthesis of Highway Practice No. 5, "Scour at Bridge Waterways," states, "the first priority in research on scour problems should be given to field measurements." West Virginia University undertook this study in response to the demonstrated need for field data on scour around bridge piers. Existing theories and scour prediction formulae have been critically reviewed. Comparisons have been made and the parameters essential to describing scour around bridge piers have been identified. The instrumentation necessary to conduct a field study and measure essential scour parameters was developed. This instrumentation included an automated scour monitoring system that takes periodic measurements on scour depth and stage at a bridge pier.

Existing highway bridges, located on alluvial rivers were selected as field sites. The bases for the selection were that a potential for pier scour existed and the river chosen had good probability of a flood occurrence during the study. Sites were selected and instrumentation was installed on bridges on the Brazos River, Texas; the Red River, Louisiana; the Homochitto River, Mississippi; and the Ohio River, Kentucky.

A large portion of this three-year period was devoted to instrumentation development and field installation. The scour monitoring instrumentation system is designed to be permanently attached to bridge piers. After installation, instrumentation systems at two of the four bridge sites were severely damaged during river flooding. These two instrumentation systems were to be more adequately protected from debris damage and reinstalled at the study sites.

Although some important data has been lost due to flood damage and instrumentation malfunction, data thus far collected during the year the scour monitoring systems have been operational has been significant. In addition to general data collected for each of the four sites, specific data necessary to evaluate the scour prediction formulae has been collected at two of the sites. These data are not yet sufficient to prove or disprove any of the existing theories. However, the large discrepancies among the various scour prediction formulae have been demonstrated.

I. INTRODUCTION

1.1.0 Introduction

It is the nature of alluvial streams to undergo changes and adjust their boundaries. This mobile nature of river boundaries presents many problems to the bridge engineer who must anticipate changes if his designs are to be effective. Failure to do so often results in catastrophic consequences. Examples abound where bridges have failed during floods. The result is the loss of money, convenience, and regrettable, at times, the loss of life.

In the United States, it is estimated that the annual bridge damage resulting from floods is in excess of \$20 million in the Federal Highway Emergency Relief Funds alone. The figure must surely be doubled when considering all the state, county and privately owned bridges.

A major portion of bridge damage is caused by the erosive action of the water running in the streams. This erosive action of water in excavating and carrying away material from the bed is generally referred to as Scour.

Scour is a natural phenomenon that occurs in streams, but is generally aggravated by the presence of obstructions such as bridge waterways construction, piers, spur dikes and the like. The undermining of bridge piers, abutments and approaches caused by scour is a constant aggravation to the bridge structures.

What is generally accepted as scour may be categorized as follows:

1) Natural Scour: In alluvial channels there is a tendency for channel changes or meanders that are the natural result of sediment transport caused by the flowing water. This channel shifting often results in river bed elevation changes and may be considered as natural scour. Included in what can be considered as natural scour is channel degradation that is part of the natural fluvial processes. Often, however, the aggregation and degradation tendencies of rivers are the result of man-made regime changes either upstream or downstream of the site.

2) Contraction Scour: Contraction scour may be associated with a constriction in a stream that results in sediment being transported out of an area faster than it is resupplied. Of course, this constriction can be natural but man-made changes are of specific interest. This scour is thought of as occurring over a relatively large area. As an example, it may be associated with the constriction in a bridge waterway opening and would result in the general lowering of the bed in the vicinity of the bridge crossing.

3) Local Scour: Local scour results from localized flow changes. These changes may be natural or man-made, but particular attention is

placed on the local scour resulting around piers, abutments, embankments, etc. that are associated with bridge structures. It is generally felt that local scour results from changes in flow patterns induced by these obstructions.

Scour at bridge piers is the sum of the natural scour that would be present if the bridge was not present, the contraction scour that results from the flow restriction caused by the bridge waterway opening and the local scour that is generated by a pier itself.

A good hydraulic design of a bridge waterway is predicated on a knowledge of the mechanics of scour. To this end researchers have attempted for nearly a century to furnish a reliable formula to predict scour at bridge waterways. In spite of intensive research, no single formula is available that bridge engineers can use with confidence.

Scour is a very complex manifestation of sediment transport that defies a simple mathematical explanation. At a bridge pier, it is a three-dimensional problem involving the interaction of an unsteady fluid flow and a loose boundary. The fundamental laws of mobile hydraulics have been used in many cases as a basis for an understanding of scour. Unfortunately, the understanding of transport of sediment is not yet complete. Because of the complexity of the problem and randomness of many of the variables a concise mathematical description of scour based on physical fundamentals of sediment transport is not available.

It is anticipated that a complex description of scour must consider a large number of variables and would, indeed, be complex. Investigators have sought to simplify the problem and thus make it manageable by reducing the number of variables considered and using empirical or semi-empirical methods to arrive at a formula. In most cases, laboratory models were used to gather the data on which to base the empirical formula.

Field data on bed movement and the mechanics of scour during floods is extremely difficult to collect. Consequently, little reliable field data is available. This is unfortunate because correlation between laboratory results and field data is still far from satisfactory as far as mobile bed hydraulics is concerned.

Available field data on scour at bridge waterways demonstrates that available scour prediction formulae are seldom in agreement. This situation leaves the bridge designer little better off than if he had no scour prediction formula at all. Consequently, most bridge and hydraulic engineers report that experience is their best guide in pier design.

Lack of specific and definite knowledge of scour around bridge piers surely leads to less than optimum pier design and minimum cost. An under-designed bridge pier of insufficient depth is likely to fail and an over-designed pier may waste money.

This problem has been recognized for some time. The Highway Research Board in their 1970 publication "Scour at Bridge Waterways" points out that "The first priority in research on scour problems should be given to field measurements." They further emphasize that "no relationship for predicting scour can be used with confidence until proof has been shown that it does predict what happens with reasonable accuracy and reliability."

This study is in response to this need. It aims to furnish field data on scour around bridge piers collected during flooding conditions. These data will be used as a basis for validating scour prediction formula. Realization of this goal should have a focusing effect on scour research and hopefully give better directions to future efforts to develop a formula for predicting scour around bridge piers. In particular, this study is aimed at supplying field data to validate or modify theories related to local scour.

II. THEORIES RELATED TO SCOUR

2.1.0 Introduction

Scour is a consequence of sediment continuity. It is the change in bed elevation resulting from the erosive action of flowing water over mobile beds. To more clearly see this, consider a small control section located in an alluvial channel as shown schematically in Figure 1.

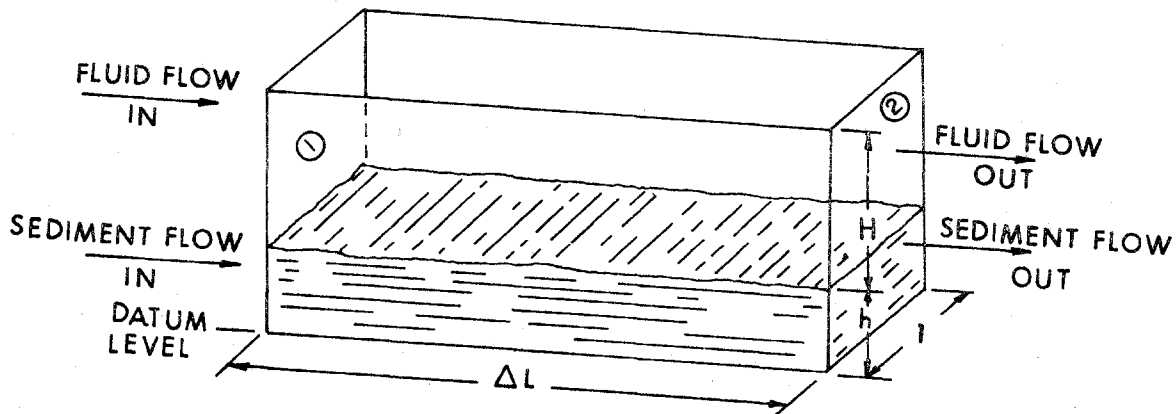


Figure 1. Control Section (Schematic)

This unit width section includes flowing water of depth H and a portion of the river bed of height h above a reference level. A decrease in the height h represents scour in the section, and an increase indicates a deposition of material in the section.

For simplicity, the control section is considered to be located normal to the flow. That is, flow enters face 1 and leaves through face 2, and no flow crosses the other face. Thus, the discussion is limited to a two-dimensional model but would apply equally well to a three-dimensional case with more mathematical complexity.

Loose boundary hydraulics recognizes that there is sediment flow in conjunction with the fluid flow. Although sediment flow depends on the fluid flow, it is meaningful to discuss the continuity of the fluid flow and sediment flow separately.

Scour is a direct consequence of sediment flow. Thus, material continuity is an important consideration. To illustrate this consideration, the relationship between the influx of sediment and efflux of sediment for the control section is

$$\frac{d\bar{V}_{st}}{dt} = q_{st_{in}} - q_{st_{out}}. \quad (2-1)$$

It follows that if $q_{st_{in}} > q_{st_{out}}$, there will be an increase in the material volume within its control section with time. Conversely, if $q_{st_{in}} < q_{st_{out}}$, there will be a decrease in the material volume within the section.

Total sediment load is comprised of the suspended load and the bedload, or

$$q_{st} = q_s + q_{ss}. \quad (2-2)$$

It has been shown analytically and experimentally that suspended load has a less significant role in the mechanics of scour if the particle size of the suspended load is less than that of the bed material. It is reasonable to assume that for water flows above normal, when scour becomes of interest to the highway designer, the suspended sediment carried into the control section will be carried out and has no appreciable effect on the mechanics of scour.

By neglecting the effects of suspended sediment on scour, a reasonable model for the material balance in a control section is

$$\frac{d\bar{V}_s}{dt} = q_{s_{in}} - q_{s_{out}}. \quad (2-3)$$

The volume of material in the control section is

$$\bar{V}_s = (\Delta L)(h)(1). \quad (2-4)$$

Therefore,

$$\frac{d\bar{V}}{dt} = \frac{dh}{dt} \quad (2-5)$$

and the bed elevation above some reference level is

$$\frac{dh}{dt} = q_{s_{in}} - q_{s_{out}}. \quad (2-6)$$

Equation 2-6 is a statement of the time rate of change in bed elevation in a control section. Since scour is a degrading of the bed, this may be accepted as the governing equation for scour in a control section.

It is well to note that scour where q_{sin} is negligible is frequently referred to as clear water scour and scour where q_{sin} is of significant magnitude is referred to as sediment-transport scour.

To further illustrate the mechanics of scour consider a two-dimensional, alluvial channel as shown schematically in figure 2.

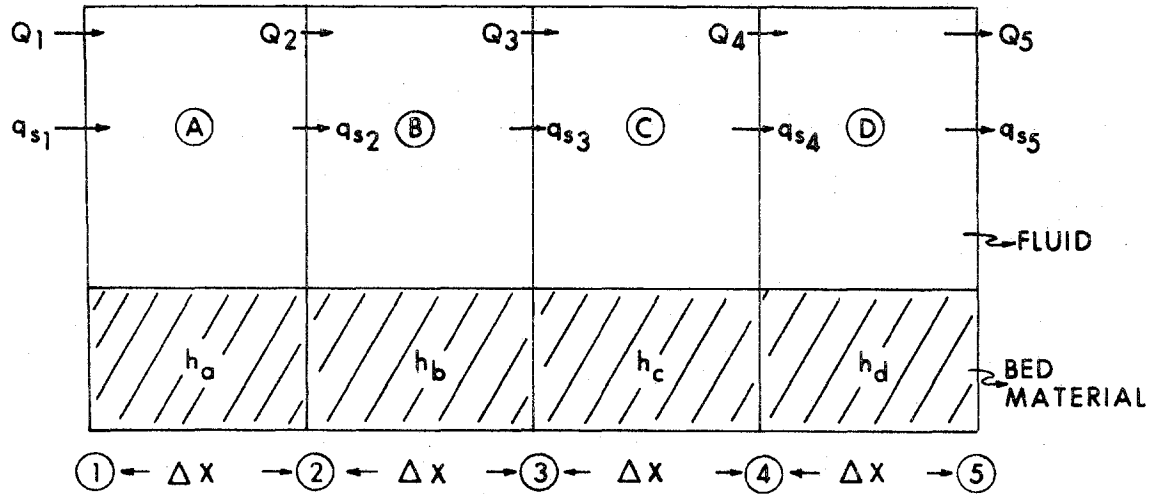


Figure 2. Two-Dimensional Model of Channel Scour (Schematic)

The channel model has been broken up into discrete sections A through D in order to discuss scour. It follows from equation 2-6 that the depth in section A is given by

$$\frac{dh_a}{dt} = q_{s1} - q_{s2} \quad (2-7)$$

and that in section B is given by

$$\frac{dh_b}{dt} = q_{s2} - q_{s3} \quad (2-8)$$

By similar reasoning, expressions may be stated for sections C and D. If the bed in each section contains the same bed material, these bed material parameters will be the same for each section. Furthermore, for steady flow, the volumetric flow rate of fluid through each section will be the same. For a given slope, it is valid to assume that the bedload material transport through each section will be the same.

$$q_{s1} = q_{s2} = q_{s3} = q_{s4} = q_{s5} \geq 0 \quad (2-9)$$

Even though there may be bed movement corresponding to a value of q_s greater than zero, equations 2-7, 2-8 and 2-9 show that the bed elevation will remain constant,

If an obstruction were to be placed in section C, this could be a narrowing of the section or an obstacle such as a bridge pier, it is expected that hydraulic parameters for section C would change. There would be a corresponding change in the bedload material transport rate q_s in section C. For instance, an increase in the shear velocity would increase the bedload material transport rate,

Let the bedload material transport in section C after the obstruction has been added be denoted as $\bar{q}_s$ where

$$\bar{q}_s = q_s + \Delta q_s \quad (2-10)$$

if

q_s = bedload material transport rate before obstruction

Δq_s = change in bedload material transport rate resulting from obstruction

It follows that

$$\Delta q_s = f(\text{change in hydraulic parameters}) \quad (2-11)$$

Since no changes took place in section A or B

$$q_{s1} = q_{s2} = q_{s3} \quad (2-12)$$

and

$$h_a = h_b = \text{constant} \quad (2-13)$$

However, in section C there would be an increase in the bedload material transport rate and

$$\Delta q_{s4} > 0. \quad (2-14)$$

Then

$$\bar{q}_{s4} > q_{s4} \quad (2-15)$$

and

$$\bar{q}_{s4} > q_{s3}. \quad (2-16)$$

Thus,

$$\frac{dh_c}{dt} = q_{s3} - \bar{q}_{s4} < 0 \quad (2-17)$$

and the bed elevation in section C will decrease with time. This process will not continue indefinitely. In fact, it will continue only until the decrease in bed elevation adjusts the hydraulic parameters so that once again $\bar{q}_{s4} = q_{s3}$ and

$$\frac{dh_c}{dt} = 0 \quad (2-18)$$

This is a rather crude description of the scour process but serves as a reasonable model to visualize the "scour hole" that results from a stream bed adjustment that takes place to establish sediment transport continuity throughout the stream.

Since scour is principally the result of bed movement it is well to briefly review the mechanics of bedload material transport as a basis for establishing pertinent parameters in any empirical scour prediction formula.

Bedload transport in alluvial streams has been explained to result from:

1. The impact of the liquid on solid particles (critical velocity criteria)
2. The drag of the fluid flow on the solid particles (shear stress criteria)
3. The lift force created by the pressure differences due to the velocity gradient (lift force criteria)

In all probability, no one mechanism is sufficient to explain the mechanics of bed movement. Rather, all must be assumed to play a role. The prevailing bedload formulas will be briefly reviewed to demonstrate the views held by the investigators as to what forms a basis for bedload material transport. Many of these bedload equations have been proposed as an "initial scour" criteria.

There are essentially three different approaches to describing bed movement; one based on critical velocity, one based on critical shear stress and one based on a lift force criteria. It follows that bedload equations similarly may be grouped into three categories. These are:

1. Equations considering a discharge or critical velocity relationship (Schoklitsch type)
2. Equations considering a shear stress relationship (DuBoys type)
3. Equations based upon statistical consideration of the lift forces (Einstein type)

Some frequently used bedload transport equations are shown in Table 1. Along with each equation, presented in a representative form, is an acceptable functional representation of the equation to demonstrate a set of independent variables for the equation. Although this table does not list all bedload transport equations, those listed certainly represent the modern thinking on bedload material transport theory.

It is concluded from the equations of Table 1 that a general functional representation for the bedload material transport rate may be expressed as:

$$q_s = f(d, g, \rho, \rho_s, \gamma, \gamma_s, S, Re_*, V_*, C_d, C_r, C_w, H, C_p, q) \quad (2-19)$$

This functional representation of q_s serves to illustrate the complete collection of variables considered important to bedload transport.

It is interesting to note that most of these sediment transport formulas may be expressed in the form

$$q_s = \rho \rho_s V_*^3 f(Re_*, \frac{\gamma}{\gamma_s}, \frac{V_*^2}{gd}) \quad (2-20)$$

regardless of whether the critical velocity, tractive force, or lift force criteria was used in its development. It may be concluded that bedload material transport rate is dependent on:

1. hydraulic parameters
2. fluid properties
3. sediment properties
4. geometric parameters

In order to speculate on the complete collection of variables that may influence scour, it is necessary to include those that influence bedload material transport rate, that is, variables from each of the four categories above, along with those parameters that influence flow and stream geometry. Equation 2-21 is a functional representation of a supposition of the parameters influencing river bed scour as an accumulation of variables from the bedload transport equations and variables that influence the flow as predicted from the theory of open channel hydraulics.

$$D_s = f(H, b, \tau, \tau_{cr}, \rho, \rho_s, Re_*, Re, \gamma, \gamma_s, S, \sigma, d, \alpha, \mu, T, q, C_c, C_g, \dots, V_*, V, n, F, C_d, V_F, V_m, W) \quad (2-21)$$

No effort has been made to identify independent and dependent variables. Rather, equation 2-21 is intended to include all distinct parameters that may influence scour to establish a basis for any theory or empirical relationship that describes the mechanics of scour.

EQUATION NAME	TYPICAL FORM	FUNCTIONAL REPRESENTATION	PHYSICAL MODEL
DuBoys Bedload Transport Equation	$q_s = C_s \tau (\tau - \tau_{cr})$	$q_s = f(d, \tau, \tau_{cr})$ or $q_s = f(d, \gamma, \gamma_s, H, Re_*, S)$	Shear Stress
Meyer-Peter Bedload Transport Equation	$\gamma H \left(\frac{k}{\gamma}\right)^{3/2} S - 0.047 (\gamma_s - \gamma) d =$ $0.25 \sqrt[3]{\rho} G \left[\frac{\gamma_s - \gamma}{\gamma_s} \right]^{2/3}$	$q_s = f(d, \gamma, \gamma_s, H, Re_*, S)$	Shear Stress
Sheilds Bedload Transport Equation	$q_s = \frac{\gamma}{\gamma_s} s \cdot q \left[\frac{\tau - \tau_{cr}}{(\gamma_s - \gamma) d} \right]$	$q_s = f(d, \gamma, \gamma_s, q, Re_*, S)$	Critical Tractive Force
Schoklitsch and MacDougall Bedload Transport Equations	$q_s = (C_p / \sqrt{d}) S^{3/2} (q - q_{cr})$	$q_s = f(d, S, q, H, C_r, C_w, C_p)$	Critical Flow
Kalinske Bedload Transport Equation	$q_s = V_* d f \left[\frac{\tau_{cr}}{\tau} \right]$	$q_s = f(d, \gamma, \gamma_s, H, S, Re_*)$	Shear Stress
Einstein-Brown Transport Equation	$G = 40 \left[\frac{V_*^3}{\gamma_s} \right]^2 \left[\frac{\rho^5 (\gamma_s - \rho) g}{d^3} \right]^{1/2}$	$q_s = f(\gamma_s, d, \rho, V_*)$	Hydrodynamic Life Probability

Table 1. Bedload Transport Equations.

It is obvious that the task of gathering sufficient data to develop an empirical scour prediction formula from equation 2-21 and determining which variables or dimensionless numbers should be considered as independent is a very formidable job. This is exactly the task facing an investigator trying to develop a "simple" scour prediction formula. To further complicate the problem, a useful scour prediction formula must be expressed in terms of "measurable" variables that are available to the bridge engineer. It is obvious that in any attempt to develop a scour prediction formula, simplification is desirable from a utilization standpoint and justifiable from an economic standpoint.

Over the past century many investigators have attempted to develop a "simple" scour prediction formula. Instead of systematically collecting laboratory and field data to determine the relative importance of the many variables shown in equation 2-21, it appears that a "set" of variables were arbitrarily selected and data collected over a limited range to determine their relationship to scour depth. However, this approach, understandably due to economic limitations, has left us with a large number of sometimes conflicting formulas to predict scour. Without reliable field data on which to qualify these formulas there is no means of judging which is better or over what range of field parameters they are valid.

The aim of this study is to collect sufficient field data to qualify the existing scour prediction formulas. The following section briefly reviews existing scour prediction formulas that were considered in this study.

2.2.0 Scour Prediction Formulas

Scour prediction formula, or theories, may be grouped into three broad categories. These are:

1. Regime Theory (Natural Scour)
2. Long Contraction Theory (General Scour)
3. Secondary Flow Theory (Local Scour)

Regime theory was introduced first in India about a half a century ago, as a result of measurements made on different stable canals. The relations between channel parameters, such as depth, width, slope, discharge and property of boundary material were empirically determined when the channel was in its regime and used to describe natural scour. A channel is said to be in regime when there is no deposition, erosion or change in its boundary over a hydrological cycle. This approach has been employed by some investigators to determine the depth of scour in rivers and bridge crossings.

A reduction in river width is known to be an important factor in scour phenomenon. Laursen and others made use of long contraction model to explain the general scour due to a decrease in the waterway width. Long contraction theory has been used to predict scour at bridge crossings.

Secondary flow has been used to explain local scour. It is the opinion of some that the secondary flows caused by obstructions in the flow, generate scour holes. Bridge piers are an important example.

A good many scour prediction formulas are empirical and are the direct result of using experimental data to generate a formula rather than the use of any particular theory. Many of these do not fall into either or the three categories mentioned above and are considered here simply as empirical formulas.

The following is a brief review of the literature in scour. The intent of this review is to establish the state-of-the-art in scour prediction formulas as a basis for the field work performed in this study.

2.2.1 Natural and Contraction Formulas

Kennedy, R. G. (1895)

Kennedy believed that the vertical component of constant eddies in flowing water keeps the sediment in suspension.

He introduced the equation

$$V_{cr} = CH^m \quad (2-22)$$

where

$$m = 0.64$$

$$C = 0.84$$

Further studies indicated that C and m are not constant, but are different for different channels.

Lindley, E. S. (1919)

Lindley reviewed Kennedy's work and from his own observations on the Upper Chenab canal he derived the following equations that relate scour depth to mean flow velocity and average width of the channel:

$$V_m = 0.95 H^{0.57} \quad (2-23)$$

$$V_m = 0.57 B^{0.355} \quad (2-24)$$

Lacey, Gerald (1929)

Lacey discussed the applications of Lindley's theorem and introduced the equation

$$W_{ws} = 2.67 Q^{1/2} \quad (2-25)$$

where

W_{ws} is in feet and Q is in ft^3/sec

He also presented an equation to determine the maximum depth of scour below the water surface at bridge crossings and other constructions. This equation is

$$D_{\max}^* = CR_h \quad (2-26)$$

where

C is a function of the cross-section of the river at the bridge.

Lacey, G. (1934)

It was observed from field data that for uniform flow in an alluvium

$$V \sim \sqrt{H} \quad (2-27)$$

$$S \sim \frac{1}{\sqrt{H}} \quad (2-28)$$

This observation prompted Lacey to suggest

$$V = KR_h^{3/4} S^{1/2}, \quad (2-29)$$

an equation like the Chezy equation.

In his 1945 paper, Lacey calculated the scour depth upstream and downstream of barrages and presented

$$D^* = 1.40 (q/f)^{1/3} H_{cr} \quad (\text{upstream}) \quad (2-30)$$

$$D^* = 3.0 H_{cr} \quad (\text{downstream}) \quad (2-31)$$

where

$$q = \frac{1}{3} Q^{1/2}$$
$$H_{cr} = (q^2/f)^{1/3}$$

Khosla, Rai, Bahaduv, A. N., Bose, N. K., and Taylor, E. M. (1936)

Using Lacey's formula and assuming that scour depth is proportional to regime depth, this paper proposed that

$$D_{\max}^* = 0.9 KH_{cr} \quad (2-32)$$

where

K = a factor dependent on geometry

Laursen, E. M. (1950-73)

During the last two decades, the work of E. M. Laursen has had the largest impact of any American researcher on the thinking of other researchers and engineers working on the scour problem. His earlier work with the Iowa Institute of Hydraulic Research enables him to form opinions on sediment transport and scour around bridge piers and abutments. This research included laboratory experiments and limited amounts of field data.

During the early 1950's and subsequent years Laursen published many important works on his observations and ultimately presented design curves and formulas for predicting scour. He defined scour as the enlargement of a flow section by the removal of material composing the boundary through the action of the fluid in motion and gave four general characteristics on which an analysis of local scour should be based. These are:

1. By the principle of conservation of matter the local rate of scour is equal to the difference between the rate of removal and rate of supply.
2. The rate of scour will decrease as the flow section is enlarged.
3. There will be a limiting extent of scour.
4. This limit will be approached asymptotically.

Feeling that scour is a consequence of an imbalance between the capacity of the flow to transport sediment out of an area and the rate of supply of sediment to that area, Laursen approached the problem of scour around bridge piers and abutments by making use of the long contraction analysis. Long contraction analysis describes the conditions in uniform reaches upstream and in a long contraction by a discharge and a sediment-transport equation. It is reasoned that the scour depth can be determined from the depth ratio of unconstricted to constricted sections. Laursen derived the following formulas:

For the sediment transporting scour

$$\frac{b}{H} = 5.5 \frac{D}{H} \left[\left(\frac{D}{rH} + 1 \right)^{1.7} - 1 \right] \quad (2-33)$$

and for clear water scour

$$\frac{b}{H} = 5.5 \frac{D}{H} \left[\frac{\left(\frac{D}{rH} + 1 \right)^{7/6}}{\left(\frac{V^2}{120d^{2/3}H^{1/3}} \right)^{1/2}} - 1 \right] \quad (2-34)$$

where the units are in feet.

Laursen also presented a basic design curve to evaluate scour depth as $\frac{\text{depth of scour}}{\text{pier width}}$ vs $\frac{\text{flow depth}}{\text{pier width}}$. Later on, C. R. Neill fits an equation to the design curve. This equation is:

$$D = 1.5 b^{0.7} H^{0.3} K_1 K_2 \quad (2-35)$$

where

K_1 = shape factor

K_2 = flow direction factor

Ahmad, M. (1953-62)

Through dimensional analysis, Ahmad determined the nondimensional parameters affecting scour depth at a spur dike. Experimental studies on the effect of discharge intensity, sand grade, flow concentration and flow direction resulted in Ahmad's 1953 formula to predict scour depth. This equation is:

$$D^* = Kq^{2/3} \quad (2-36)$$

where

K is a corrective factor that depends on the geometry of the spur dike. It ranges from 1.5 to 2.25.

Subsequent field and model studies of bridges crossing in Pakistan enabled him to present the same formula to measure the scour depth around bridge piers. For bridge pier scour, the value of K varies from 1.3 to 2.3.

Chitale, S. V. (1960)

Based on the concept that equilibrium scour depth depends on the relative depth of flow, shape of the pier and the angle of attack and not on velocity and sediment size, Chitale conducted laboratory experiments using scale models of the Hardinge Bridge to reach the conclusion that:

1. With axial flow the maximum depth of scour is always at the nose of a pier.
2. The ratio of scour at the nose of a pier and the depth of flow in the channel bears a simple relationship with the approach velocity in the channel.
3. The upstream depth of flow has an influence on the depth of scour at the pier nose.

From data furnished by the flume experiments, Chitale presented the following empirical formula for predicting scour depth:

$$\frac{D}{H} = -0.51 + 6.65F - 5.49F^2 \quad (2-37)$$

where $F = V/\sqrt{gH}$ (Froude No. of approaching flow)

Breusers (1964)

Based on model tests of drilling spuds in waves and currents, Breusers derived the following equation to predict the depth of scour.

$$D_{\max} = 1.4b^* \quad (2-38)$$

Saburo Kamura (1966)

This theoretical analysis classifies the longitudinal and cross-sectional equilibrium profiles of alluvial channels into a dynamics equilibrium theory (Regime theory) and a static equilibrium theory (Tractive Force theory). The first theory requires that the sediment load passing through the cross-section of the upstream end to be equal to that of all points along the channel. The second theory requires that the critical shear stress on the bed be constant at all points along the channel.

For the case of dynamic equilibrium, Kamura suggests that scour depth can be obtained from

$$\frac{H_2}{H_1} = \frac{D_e}{H_1} + 1 = \left(\frac{\tau_1}{\tau_2}\right)^{\frac{3m+1}{14}} \cdot \left(\frac{w_1}{w_2}\right)^{\frac{m+5}{7}} \quad (2-39)$$

where m is a dimensionless exponent pertaining to the tractive force.

It is assumed that the sediment supply is practically zero for the static equilibrium case. Thus, when scour is fully developed it is assumed that the boundary shear stress in the constricted part of the channel will be the critical tractive force and that the critical boundary shear stress will also exist in the unconstricted part of the channel. Then the scour depth is found to be

$$\frac{H_2}{H_1} = \frac{D_e}{H_1} + 1 = \left(\frac{\tau_{cr1}}{\tau_{cr2}}\right)^{2/7} \cdot \left(\frac{w_1}{w_2}\right)^{6/7} \quad (2-40)$$

2.2.2 Local Scour

Several investigators have looked at secondary flow as a contributing factor in local scour. As early as 1932 Keutner used flume experiments to demonstrate the effects of secondary flow. He observed that

1. Deepest scour occurs in front of the pier.
2. Scour depends on the pier shape.
3. The shape of the pier tail has no influence on the depth and extent of scour at the nose.
4. Semi-circular nosed pier causes the greatest scour.
5. Scour depth when angle of attack is 27° is twice as large as that at 0° .

6. Backwater height for streaming flow is practically independent of the slope of both the nose and tail of the pier.
7. Scour process occurs due to the existence of a secondary flow together with the primary flow. Secondary flow is generated from the slope of the lateral water surface.

In 1938, Ishihara discussed scour and concluded that velocity, discharge and sand characteristics are important factors when considering scour in an alluvial river bed. He observed that the shape of a pier has an effect on scour hole depths, and recommends that circular piers not be used, but concluded that the length of a pier has no effect on the depth of the scour hole in front of the pier. In 1942, T. Ishihara discussed the effect of the angle of attack and stated that scour will increase with an increase in the angle.

Ishihara believes that scour results from horizontal eddies caused by flow eccentricity and non-uniform velocity along a vertical section. He assumes that there is a similarity between scour at a river bend and scour around piers and concludes that a two-dimensional flow model is not sufficient to study local scour caused by obstructions.

Following Ishihara, Sugimoto and Imda in 1950, Toomre in 1959 and Tison in 1961 continued the investigation of changes in flow on local scour. Their efforts supported the idea that local changes in flow caused by bridge piers or other obstructions influence the depth of scour at the pier.

Inglis, C. C. (1949)

Model studies in connection with the Hardinge Bridge furnished the data for the following:

$$\frac{D_{\max}^*}{b} = 1.73 \left(\frac{H}{b} \right)^{0.78} \quad (2-41)$$

and also

$$\frac{D_{\max}^*}{b} = 1.7 \left(\frac{q}{b} \right)^{2/3} \quad (2-42)$$

The study considered bed material ranging from 0.06 to 0.37 in diameter and concluded that material size has little or no effect on the maximum depth of scour. Inglis suggests that the effect of a bridge pier on flow can be approximated as a bend and that the maximum scour depth around the bridge pier is proportional to Lacey's regime depth.

After gathering data from 17 bridges with discharge varying from 29,063 to 2,250,000 ft³/sec, scour depth below the water surface varying from 25 to 117 feet, and size of bed material varying from .17 to 0.39mm, scour depth was found to be twice the "Lacey depth." This is known as

Inglis-Lacey equation

$$D_{\max}^* = 0.946 (Q/f)^{1/3} \quad (2-43)$$

Blench, T. (1960)

Blench simplified the Inglis-Poona equation to give

$$\frac{D^*}{d_r} = 1.8 \left(\frac{b}{d_r}\right)^{0.25} \quad (2-44)$$

and by fitting a curve to model and field data

$$D^* = 1.8 d_r \quad (2-45)$$

where

$$d_r = (q^2/F_b)^{1/3}$$

$$F_b = \text{bed factor defined as } \frac{(\text{average velocity})^2}{\text{flow depth}}$$

In 1964 Blench introduced an equation in which " d_r " had been replaced by H , the depth of local approaching flow, to give

$$\frac{D^*}{H} = 1.8 \left(\frac{b}{H}\right)^{0.25} \quad (2-46)$$

with the condition that $b < H$.

Varzeliotis, A. N. (1960)

As a result of laboratory experiments using a 44" wide, 130' long flume with gravel as bed material, Varzeliotis reached the following conclusions:

1. size, width, and angle of attack of a pier are of great importance in estimating scour depth
2. the effect of the pier length on maximum scour depth is not significant
3. the effect of flow contraction has little effect on scour depth if the contraction ratio is less than 10%

Using these experimental data he plotted

$(q^{2/3}/b)$ vs (D^*/b) and determined a formula similar to Inglis' equation.

This empirical equation is

$$\frac{D_{\max}^*}{b} = 1.43 \left(\frac{q^{2/3}}{b}\right)^{0.72} \quad (2-47)$$

Bata, G. (1960)

It is Bata's opinion that the vertical velocity in the downward direction resulting from backwater pressure is the cause of local scour at bridge piers. The scour hole exists upstream of the pier for a distance of 3 to 6 times the radius of the pier.

Results from his prototype and model studies show that the influence of the Reynolds Number is negligible and that there is a linear relationship between $\frac{D}{H}$ & $\frac{V}{\sqrt{gH}}$ (Froude Number). He presents a curve of $\frac{D}{H}$ vs $\frac{V}{\sqrt{gH}}$ that shows an initial velocity is necessary to start erosion around the pier. Bata modified the Jaroslavcev formula to give

$$D = K_1 K_2 K_3 V^2 - 30d \quad (2-48)$$

by assuming $K_1 K_2 K_3 \doteq 1$ this reduces to

$$\frac{D}{H} = 10 \left(\frac{V^2}{gH} - \frac{3d}{H} \right) \quad (2-49)$$

Bata concluded that the diameter of sediment particles did not have a significant influence on this magnitude of scour in model experiments.

Knezevic, Bogic (1960)

Knezevic's contribution to the knowledge of scour results from an extensive investigation conducted in the hydraulic laboratory of the Civil Engineering Department in Belgrade, Yugoslavia. Flume experiments were performed with 2 piers at the same time; one was rectangular and the other circular. Different types of sand and different values of flow discharge, depth, and velocity were used. As a result of the experiment, it was concluded that the maximum depth of scour is a function of discharge, flow depth, sediment size, and pier shape.

From the experimental data, Knezevic presented the following formula to describe scour at bridge piers:

$$D_{\max} = \frac{A(q - CH\sqrt{gd})^{3/2}}{H^{5/4} g^{3/4}} \quad (2-50)$$

where C and A are constants. Curves were presented for evaluating the constants A and C.

Liu, H. K., Chang, F. M., Skinner, M. M. (1961)

This work considered scour to develop with three distinct periods:

1. Transition of scour establishment; an early stage where depth of scour increases rapidly with time and its rate decreases rapidly as scour depth increases.

2. Establishment of scour, the second stage in which the average depth of scour increases with the logarithm of time.
3. Transition to maximum scour in which scour depth approaches a maximum when there is no bedload supply into the scour hole, this period can not be defined easily by experiments.

From the experimental data on scour at bridge abutments it is concluded that

$$\frac{D_e}{H_n} = 2.15 \left[\frac{a}{H_n} \right]^{0.4} F_n^{1/3} \quad (2-51)$$

and for maximum depth of scour

$$\frac{D_{\max}}{H_n} = 0.3 + 2.15 \left[\frac{a}{H_n} \right]^{0.4} F_n^{1/3} \quad (2-52)$$

where F_n is the Froude Number evaluated at normal flow.

It is the opinion of these investigators that scour caused by flow with bedload material transport is a local phenomenon. On the other hand, scour occurring in flows with little bedload material transport is general scour and is dependent on the overall geometry of the constriction. It was concluded that:

1. The effect of flow velocity is significant.
2. The effect of opening ratio is not fully understood. In cases of appreciable bedload, the opening ratio seems to have little effect on scour. On the other hand, if no bedload is being carried by the flow, limiting maximum scour is found to be effected by the opening ratio.
3. Scour depth is a function of flow depth.

Tarapore, Z. S. (1962)

Based on the sediment continuity equation, Tarapore shows that the rate of scour in a given section is equal to the gradient of the sediment carrying capacity in the direction of the flow. Potential theory is employed together with the Du Boys bedload formula to obtain a relationship for maximum depth of scour. Equations were left in integral form with solutions presented as working design curves.

Tarapore concluded that:

1. Velocity of flow and sediment size are not important so long as the rate of bedload transport is advanced.

2. If the flow depth is large, the scour depth is 2.7 times the radius of the cylinder.
3. For low flow depth, the dimensionless ratio of the scour depth to the cylinder ratio is a function of the flow depth to cylinder radius.
4. Suspended particles have no effect on scour depth.

Larras, Jean (1963)

Larras presents an analysis of data obtained from various French rivers and different model studies on maximum depth of scour around bridge piers. Scour depth is studied in stable river beds and in unstable river beds.

In stable river beds, enough material is transported to maintain a level bottom. In unstable streams, the transported material is inadequate to maintain the river bed at constant level.

Larras concluded that maximum scouring is independent of the water depth and bed material size if the bed is stable, water depth is greater than 30 to 40 times the size of the bed material, and the channel constriction is less than 10% at the bridge site. The scour depth is a function of the maximum width of the pier, its shape, and flow direction.

Larras introduces the following equation to calculate the maximum scour depth:

$$D_{\max} = 1.42 K b^{3/4} \quad (2-53)$$

where

K = a coefficient to compensate for pier shape and angle of attack, given in a table. (D and b are in feet)

It was explained that scour depth is greater in unstable river beds than stable beds due to the inadequate supply of bed material to the scoured area.

Jose Antonio Maza Alvarez and Jose Luis Sanchez Bribiesca (1964)

A study was conducted to explain the discrepancy between Laursen's and Yaruslavtsiev's theory. Laursen suggests that scour depth depends on the depth of flow and pier shape while Yaruslavtsiev suggests that scour depth is a function of the square of the mean velocity, the bed sediment size, and the geometry of the pier. Flume experiments showed that for a given flow depth, the scour depth varies with velocity as suggested by Yaruslavtsiev if the design curve proposed by Laursen is not exceeded. In other words, there exists a limiting value for which scour depth does not increase with an increase in velocity for a given depth of flow. Laursen's design curve is a limit to the velocity dependent region.

A modified Yaroslavtsev's formula was suggested as:

$$\frac{D_{\max}}{b} = K_1 K_5 \frac{V_m^2}{gb} - \frac{30d_m}{b} \quad (2-54)$$

where

K_1 = function of H/b , determined from Laursen's simple design curve for zero angle and rectangular pier.

K_5 = function of H/b , V^2/gb , grafically given

Carstens, M. R. (1966)

Carstens made an extensive study to develop similarity criteria for sediment transport rate and for local scour depth. He assumed that in an area of localized scour the velocity and velocity distribution resulted from the obstruction around which the scour was occurring. He explained that localized scour occurs where the water is accelerated as it moves past an obstruction in a stream, or where large vortices are generated as the flow separates from the obstruction. He analyzed localized scour using the following experiments:

1. A defined scour hole (Lefevre experiment)
2. Scour associated with dunes
3. Two-dimensional jet scour
4. Scour around vertical cylinder
5. Scour around cylinder lying on the bed

From his analysis, Carstens suggests the following equation to determine local scour depth around a cylindrical pier:

$$\frac{D_e}{b} = 0.546 \left[\frac{N_s^2 - 1.64}{N_s^2 - 5.02} \right]^{5/6} \quad (2-55)$$

where

$$N_s \text{ (the sediment number)} = \frac{V}{[(s-1)gd_m]^{1/2}}$$

Shen, H. W., Schneider, V. R., & Karaki, S. (1969)

A series of papers and reports by these investigators explain a mechanism for local scour. It is pointed out that the dominant feature of the flow near a pier is a large-scale eddy-structure, or the system of vortices, which develops about the pier. This eddy-structure is felt to form the basic mechanism for local scour. By analyzing the flow in the neighborhood of a blunt-nosed cylinder it was concluded that a horse-shoe vortex system is the mechanism of local scour and the strength of the horseshoe vortex system is a function of the pier Reynolds Number.

To obtain the strength of the horseshoe vortex a flow with an unspecified but continuous velocity distribution is considered. Therefore, the equilibrium depth of scour is a function of the piers Reynolds Number.

Since the strength of a horseshoe vortex system is a function of pier Reynolds Number and the horseshoe vortex is the mechanism of scour, it was assumed that the scour properties, i.e., scour depth, rate of scour, etc. could be determined from properties of the vortex system.

By using data from other investigators, such as Chabert and Engeldinger, Maza and Sanches, Bata, Knezevic, Tison, Chitale, Laursen and Toch, Tarapore and Varzeliotis and field data from Neill, Laursen and Toch and Larras, the functional relationship between scour depth and Reynolds Number was determined by a least squares fit of the data, to be:

$$D_e = 0.00073R^{0.619} \quad (2-56)$$

By considering $\frac{D_e}{b}$ vs. Froude Number two other formulas were also presented. These are:

$$\frac{D_e}{b} = 11.0F_p^2 \quad (2-57)$$

and

$$\frac{D_e}{b} = 3.4F_p^{0.67} \quad (2-58)$$

For continuous sediment motion the equilibrium depth of scour obtained by equation 2-56 may be conservative. In this case scour depth seems to be independent of velocity and it is recommended that equations presented by Breusers and Larras be used.

2.3.0 Comparison of Scour Prediction Formulas

The lack of a scour prediction formula that can be used with any reasonable degree of confidence was the primary motivation for this study. This fact is well illustrated by noting the variation in scour depths as predicted by different scour prediction formulas.

The foregoing sections demonstrate the diversity of approaches used to explain the mechanics of sediment transport and scour. On first inspection, there seems to be little agreement in any of the scour prediction formulas, except that they do predict scour depth. It is seen that different investigators relate scour depth with different variables. Consequently, since their formulas do not always include the same variables, it is difficult to compare them in their primitive state.

The major difficulty here evolves from the fact that the equations are empirical. The investigators "fit" the data gathered and this

fitting varies, depending on the type of experiment and the intent of the investigator. Consequently, the equations agree with the experiment and are not really applicable beyond the range of the experiment or for other variables.

A review of the scour prediction formulas shows that, with few exceptions, these formulas may be expressed in terms of three variable groupings. That is,

$$\frac{\text{scour depth}}{\text{pier width}} = f \left(\text{Froude Number}, \frac{\text{stage}}{\text{pier width}}, \text{sediment property} \right)$$

where scour depth is measured from a mean bed elevation. A. G. Anderson has suggested that the proper sediment property variable is V_m/V_{*cr} .

All of the scour prediction formulas do not necessarily depend on all three variable groups. Some depend on only one, some two and some all three. One exception is Breuser's formula, which is equal to a constant in the nondimensional comparison format.

Tables 2, 3, and 4 show the scour prediction formulas in their original and comparison format. The assumptions used to reduce the formulas to the comparison format are also given in these tables.

Table 2 lists the scour prediction formulas that can be reduced to the form $\frac{\text{scour depth}}{\text{pier width}} = f \left(\frac{\text{stage}}{\text{pier width}} \right)$.

Table 3 shows the formulas that have the form $\frac{\text{scour depth}}{\text{pier width}} = f \left(\text{Froude Number}, \frac{\text{stage}}{\text{pier width}} \right)$.

Table 4 shows the formulas that have the form $\frac{\text{scour depth}}{\text{pier width}} = f \left(\text{Froude Number}, \frac{\text{stage}}{\text{pier width}}, \text{sediment characteristic} \right)$.

By reducing all the formulas to a common functional format that includes the same variable groups, the scour prediction formulas may be meaningfully compared. A comparison was made using

- 1) $\frac{\text{scour depth}}{\text{pier width}}$ vs Froude Number for constant $\frac{\text{stage}}{\text{pier width}}$ and constant sediment characteristics.

and

- 2) $\frac{\text{scour depth}}{\text{pier width}}$ vs $\frac{\text{stage}}{\text{pier width}}$ for constant Froude Number and constant sediment characteristics.

No effort was made to make a comparison on sediment characteristics. So many variables are involved it would be necessary to analyze the

NAME	ORIGINAL FORMULA	ASSUMPTION	COMPARISON FORMAT
Breusers	$D_{\max} e = 1.4b^*$	None	$\frac{D_{\max} e}{b} = 1.4$
Blench (2)	$\frac{D^*}{H} = 1.8 (b/H)^{0.25}$	None	$\frac{D}{b} = 1.8 \left(\frac{H}{b}\right)^{0.75} - \frac{H}{b}$
Inglis-Poona (2)	$\frac{D_{\max}^*}{b} = 1.73 (H/b)^{0.78}$	None	$\frac{D_{\max}}{b} = 1.73 (H/b)^{0.78} - H/b$
Laursen	$D = 1.5 K_1 K_2 b^{0.7} H^{0.3}$	None	$\frac{D}{b} = 1.5 K_1 K_2 (H/b)^{0.3}$

Table 2. Scour Prediction Formulas Reduced To: Scour Depth/Pier Width = f (stage/pier width)

NAME	ORIGINAL FORMULA	ASSUMPTION	COMPARISON FORMAT
Shen (1)	$\frac{D_e}{b} = 11.0 F_p^2$	None	$\frac{D_e}{b} = 11 (H/b) F^2$
Shen (2)	$\frac{D_e}{b} = 3.4 F_p^{0.67}$	None	$\frac{D_e}{b} = 3.4 (H/b)^{0.335} F^{0.67}$
Chitale	$\frac{D}{H} = 6.65F - 5.49 F^2 - 0.51$	None	$\frac{D}{b} = (6.65F - 5.49F^2 - 0.51) H/b$
Liu	$\frac{D_{\max}}{H} = 0.3 + 2.15 (a/H)^{0.4} F^{1/3}$	None	$\frac{D_{\max}}{b} = 0.3 \frac{H}{b} + 2.15 (H/b)^{0.6} F^{1/3}$
Bata	$\frac{D}{H} = 10 \left(\frac{V^2}{gH} - \frac{3d}{H} \right)$	$\frac{d}{H} = 0$	$\frac{D}{b} = 10 \frac{H}{b} F^2$
Varzeliotis	$\frac{D_{\max}^*}{b} = 1.43 (q^{2/3}/b)^{0.72}$	$q = (V) (H)$	$\frac{D_{\max}}{b} = 1.43 (g)^{0.24} \left(\frac{H}{b} \right)^{0.72} F^{0.24} - \frac{H}{b}$
Inglis-Poona (1)	$\frac{D_{\max}^*}{b} = 1.7 (q^{2/3}/b)^{0.78}$	$q = (V) (H)$	$\frac{D_{\max}}{b} = 1.7 (g)^{0.26} \left(\frac{H}{b} \right)^{0.78} F^{0.26} - \frac{H}{b}$
Ahmad	$D^* = K q^{2/3}$	$q = (V) (H)$	$\frac{D}{b} = K (g)^{1/3} \left(\frac{H}{b} \right) F^{2/3} - \frac{H}{b}$

Table 3. Scour Prediction Formulas Reduced To: Scour Depth/Pier Width = f (Froude No., stage/pier width)

NAME	ORIGINAL FORMULA	ASSUMPTION	COMPARISON FORMAT
Blench (1)	$\frac{D^*}{d_r} = 1.8 (b/d_r)^{1/4}$	$q = (V) (H)$	$\frac{D}{b} = 1.8 \left(\frac{g}{F_b}\right)^{1/4} \left(\frac{H}{b}\right)^{3/4} (F)^{1/2} - \frac{H}{b}$
Blench (3)	$D^* = 1.8 (d_r)$	$q = (V) (H)$	$\frac{D}{b} = 1.8 \left(\frac{g}{F_b}\right)^{1/3} \left(\frac{H}{b}\right)^{2/3} (F) - \frac{H}{b}$
Carstens	$\frac{D_e}{b} = 0.546 \left[\frac{N_s^2 - 1.64}{N_s^2 - 5.02} \right]^{5/6}$	None	$\frac{D_e}{b} = 0.546 \left[\frac{F^2(H/b)/(d/b) - 1.65(s-1)}{F^2(h/b)/(d/b) - 5.02(s-1)} \right]^{5/6}$
Knezevic	$D_{max} = A(q - CH\sqrt{gd})^{3/2} / H^{5/4} g^{3/4}$	$q = (V) (H)$	$\frac{D_{max}}{b} = A \left(\frac{H}{b}\right)^{1/4} \left[F \left(\frac{H}{b}\right)^{1/2} - c \left(\frac{d}{b}\right)^{1/2} \right]^{3/2}$
Inglis-Lacey	$D_{max}^* = 0.946 (Q/f)^{1/3}$	$Q = VHW_{ws}$ $*W_{ws} = 2.67Q^{1/2}$	$\frac{D_{max}}{b} = (1.82) \left(\frac{g}{f}\right)^{1/3} \frac{H}{b} F^{2/3} - \frac{H}{b}$

Table 4. Scour Prediction Formulas Reduced To: Scour Depth/Pier Width = f (Froude No., stage/pier width sediment characteristics)

*derived by Lacey

original data in order to arrive at a common sediment parameter. Twelve of the seventeen formulas considered are not dependent on sediment characteristics. The sediment parameters were kept constant in the others for comparisons. It was assumed that:

Bed sediment mean diameter = 0.1mm
 Sediment specific gravity = 2.65
 $\frac{\text{sediment diameter}}{\text{pier width}} = 2.3 \times 10^{-5}$
 Pier shape = semi-circular
 Angle of attack = zero

A digital computer program was coded and used to evaluate the scour prediction formulas. The output from the computer program was fed into a CalComp plotter and the comparison curves were generated. These curves are shown in Figures 3 through 10. Figures 3 through 6 give $\frac{\text{scour depth}}{\text{pier width}}$ vs Froude Number for stage to pier width ratio of 0.5, 1, 2 and 3. Figures 7 through 10 give curves for $\frac{\text{scour depth}}{\text{pier width}}$ vs $\frac{\text{stage}}{\text{pier width}}$ for Froude Numbers of 0.05, 0.1, 0.5 and 0.8.

It can be seen from the comparison curves that there is little agreement in the scour depth predicted by the formulas. The best agreement occurs at conditions that could be considered normal river flow conditions. In Figures 3 through 6 as the Froude Number increases the disagreement in the predicted scour depth also increases. Similarly, in Figures 7 through 10 as $\frac{\text{stage}}{\text{pier width}}$ increases the variation in the predicted scour depth increases. It may be reasoned that this is the result of extrapolating curves that fit the limited available data. With little field data available and essentially no flood data available investigators have had to rely on laboratory data. Consequently, the shape of the curves, that is the functional relationship for scour depth, for large flow conditions, large Froude Numbers, has been mostly speculation.

The discrepancies in the predicted scour depths have prevented bridge engineers from using any of the formulas with confidence. To illustrate what can result from the use of these formulas consider this hypothetical case. Assume the following design case with the conditions:

velocity = 3 ft/sec
 stage = 28 ft
 bridge pier width = 14 ft diameter

then the

$$\text{Froude Number} = \frac{V}{\sqrt{gH}} = 0.1$$

and

$$\frac{\text{stage}}{\text{pier width}} = 2$$

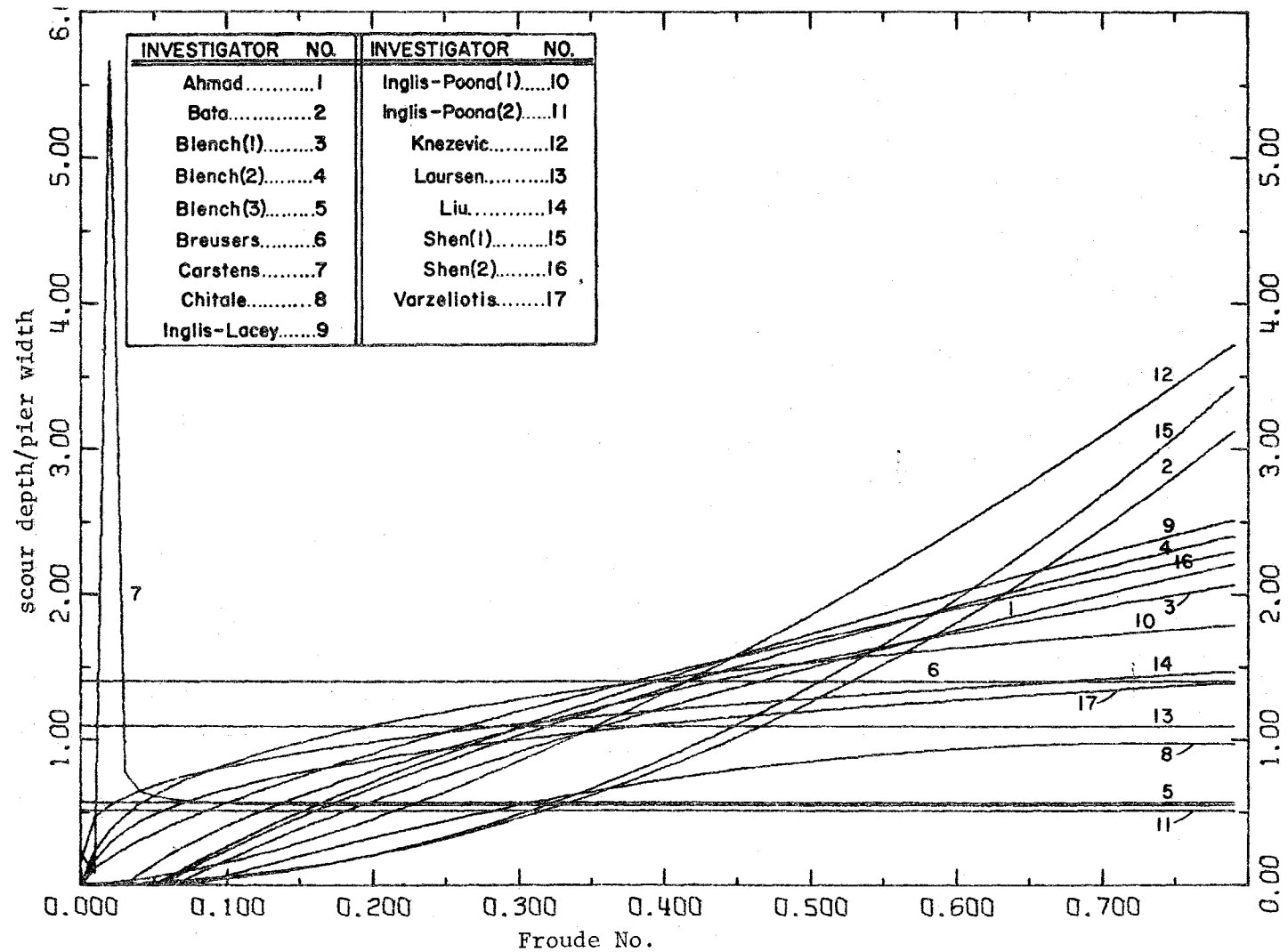


Figure 3. Scour prediction curves: $\frac{D}{b}$ vs Froude No. for stage/pier width = 0.5.

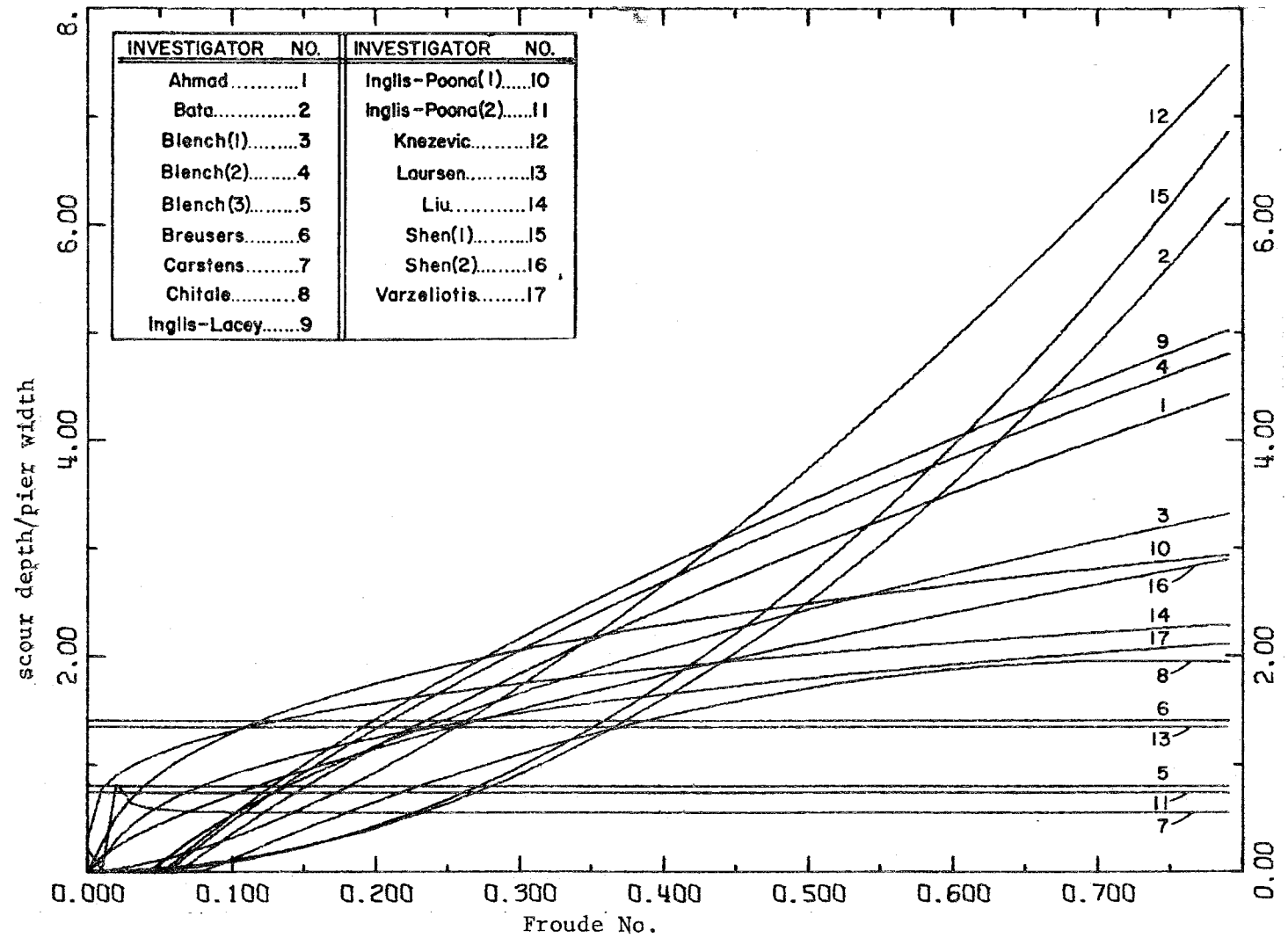


Figure 4. Scour prediction curves: $\frac{D}{b}$ vs Froude No. for stage/pier width = 1.

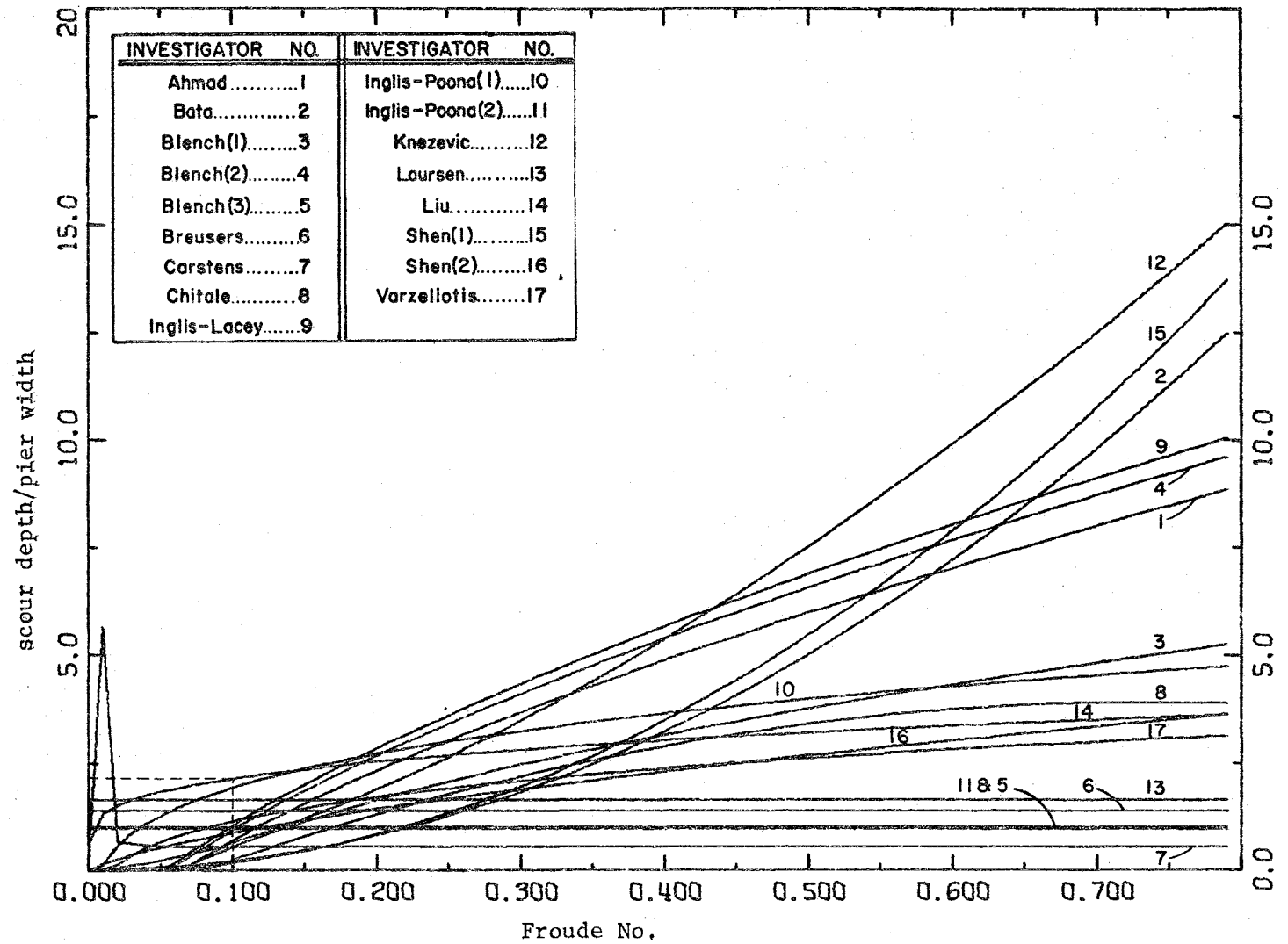


Figure 5. Scour prediction curves: $\frac{D}{b}$ vs Froude No. for stage/pier width = 2.

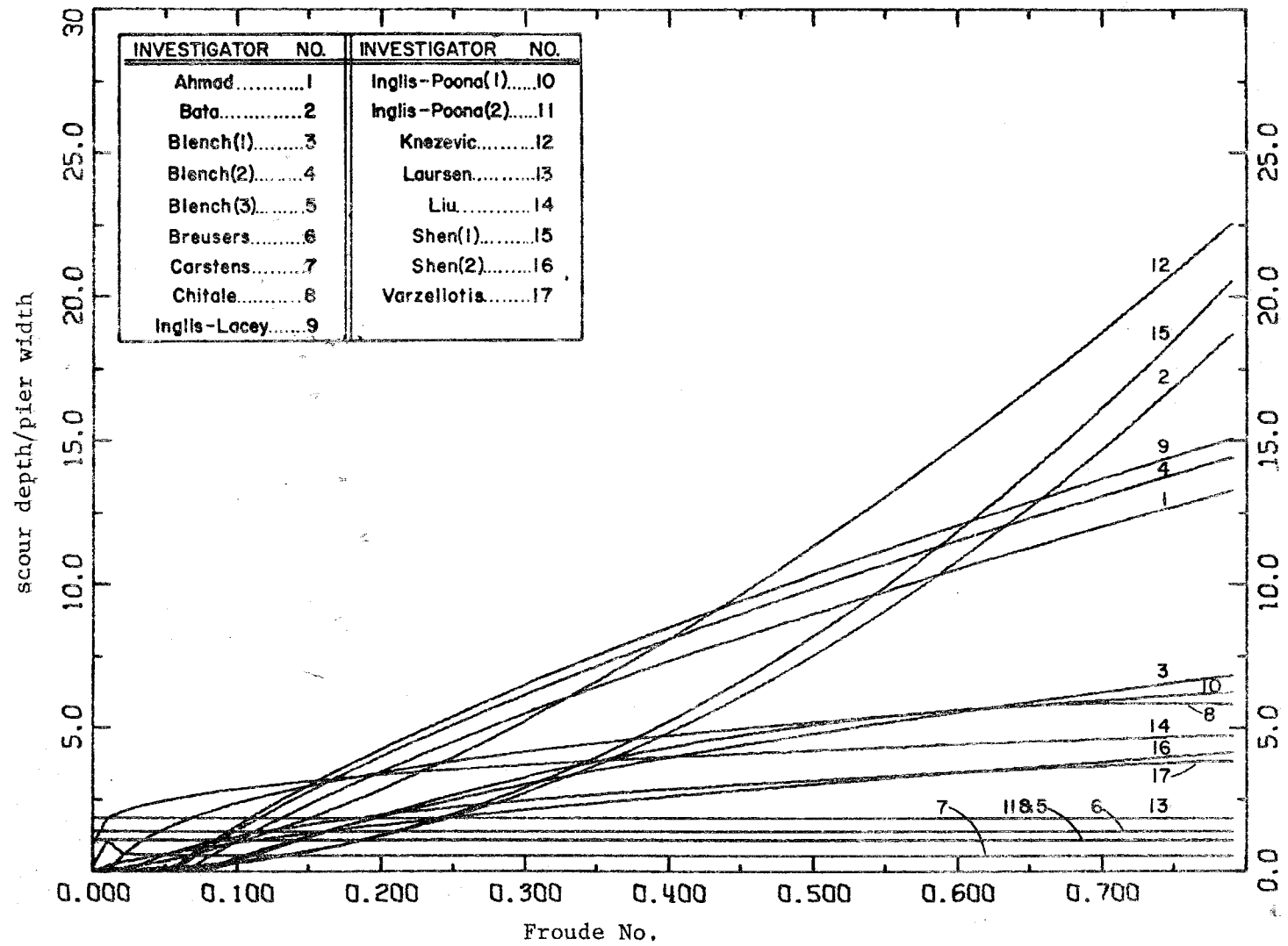


Figure 6. Scour prediction curves: $\frac{D}{b}$ vs Froude No. for stage/pier width = 3.

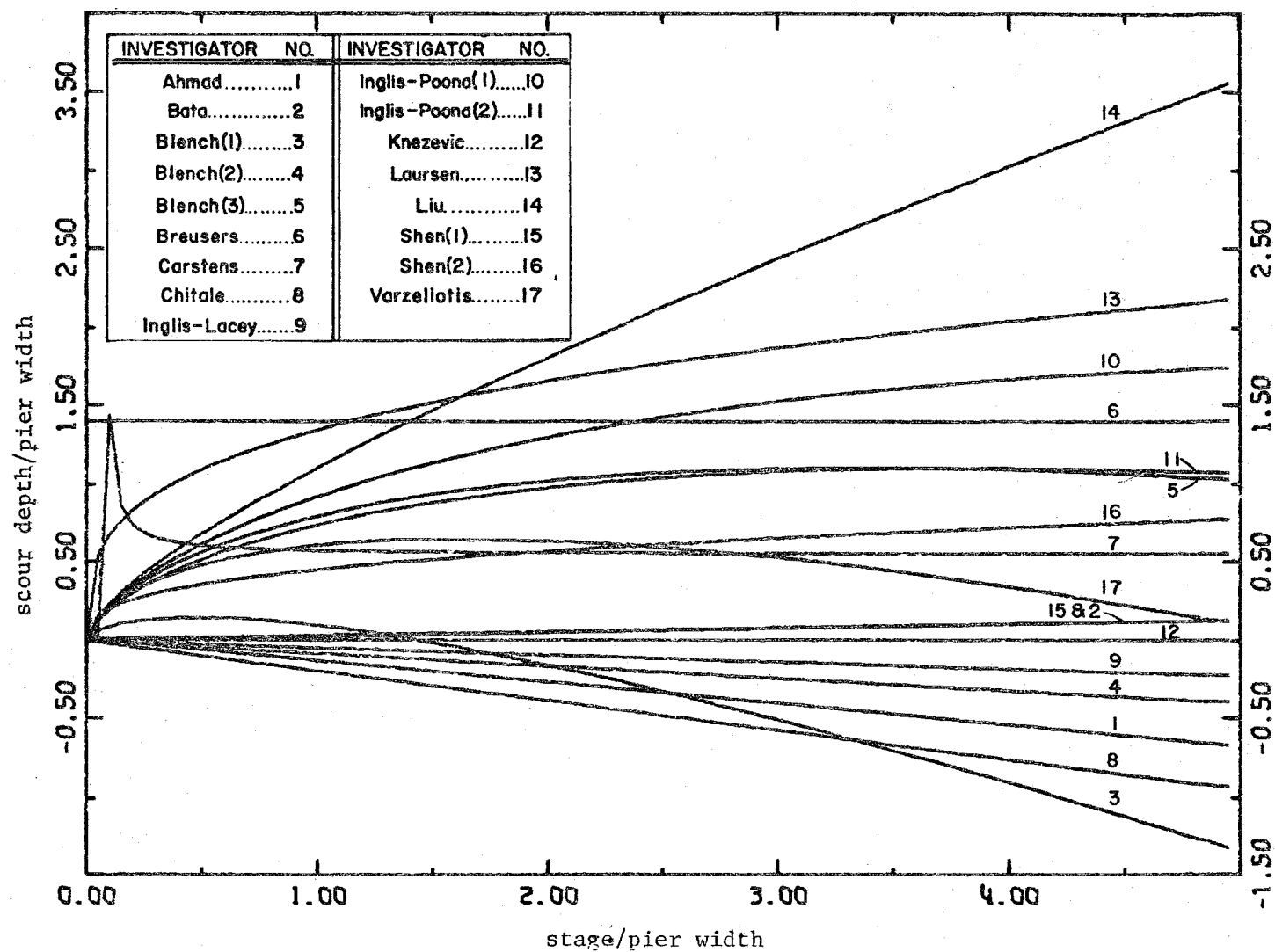


Figure 7. Scour prediction curves: $\frac{D}{b}$ vs $\frac{\text{stage}}{\text{pier width}}$ for Froude No. = 0.05.

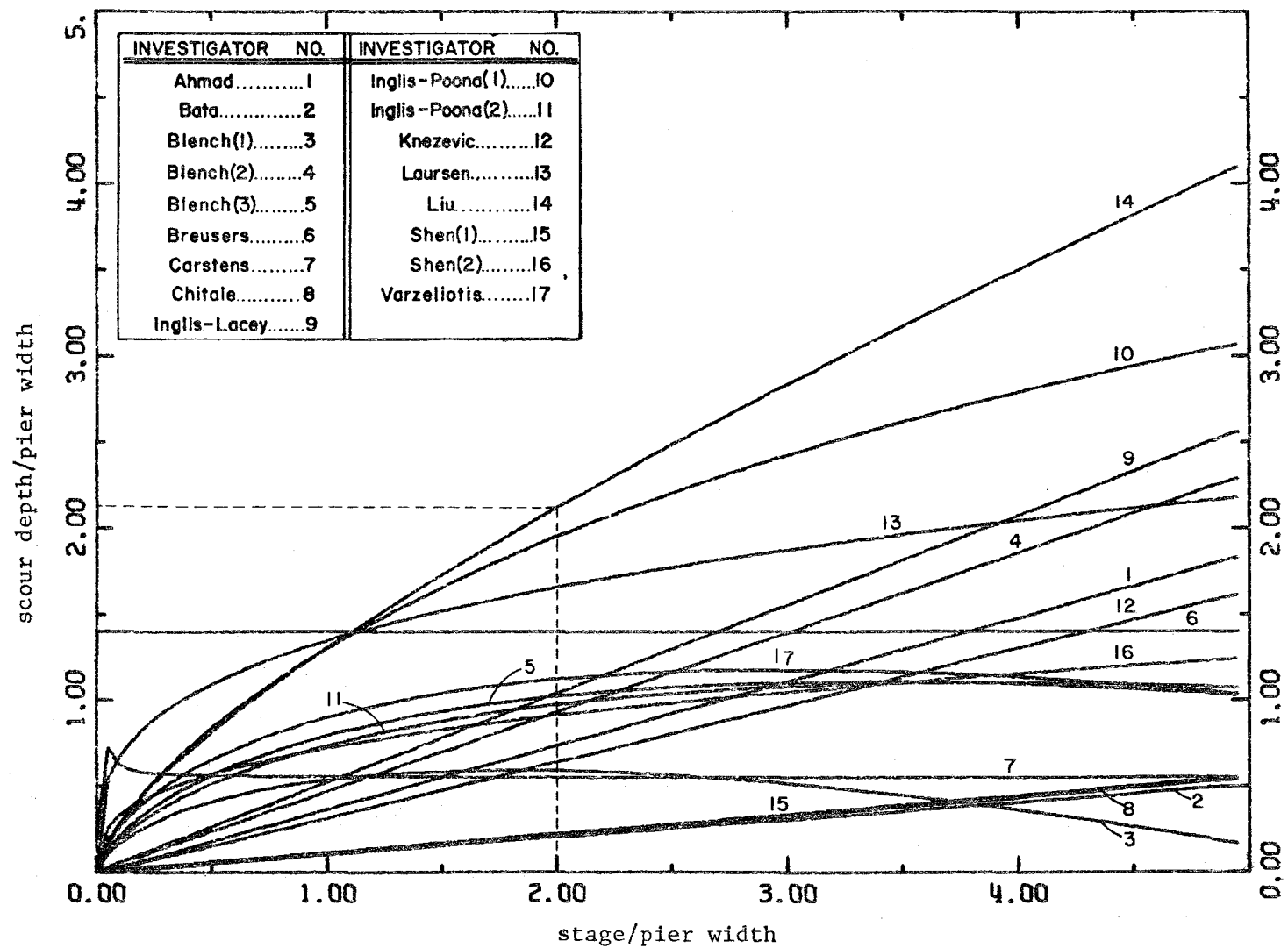


Figure 8. Scour prediction curves: $\frac{D}{b}$ vs $\frac{\text{stage}}{\text{pier width}}$ for Froude No. = 0.1.

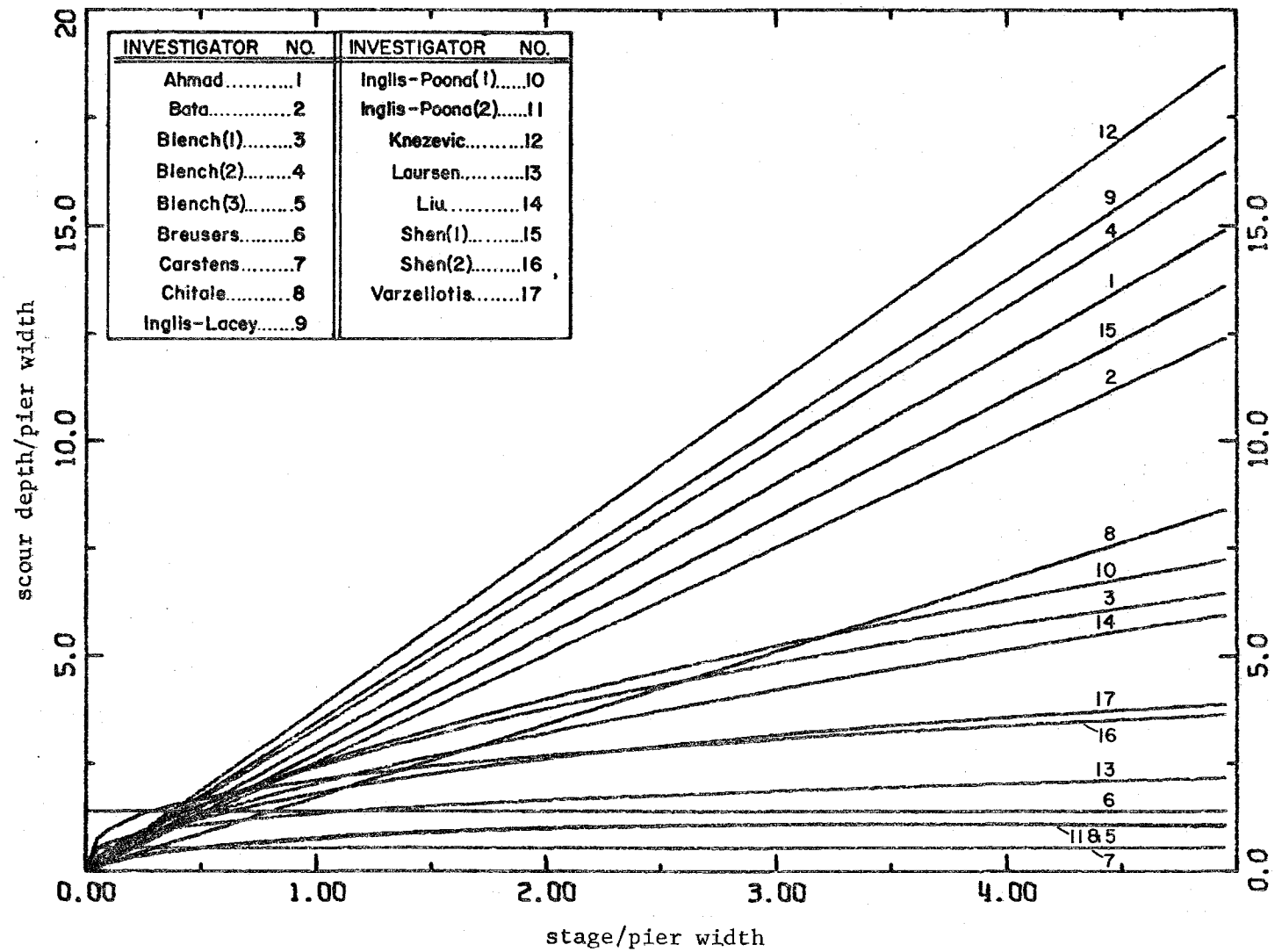


Figure 9. Scour prediction curves: $\frac{D}{b}$ vs $\frac{\text{stage}}{\text{pier width}}$ for Froude No. = 0.5.

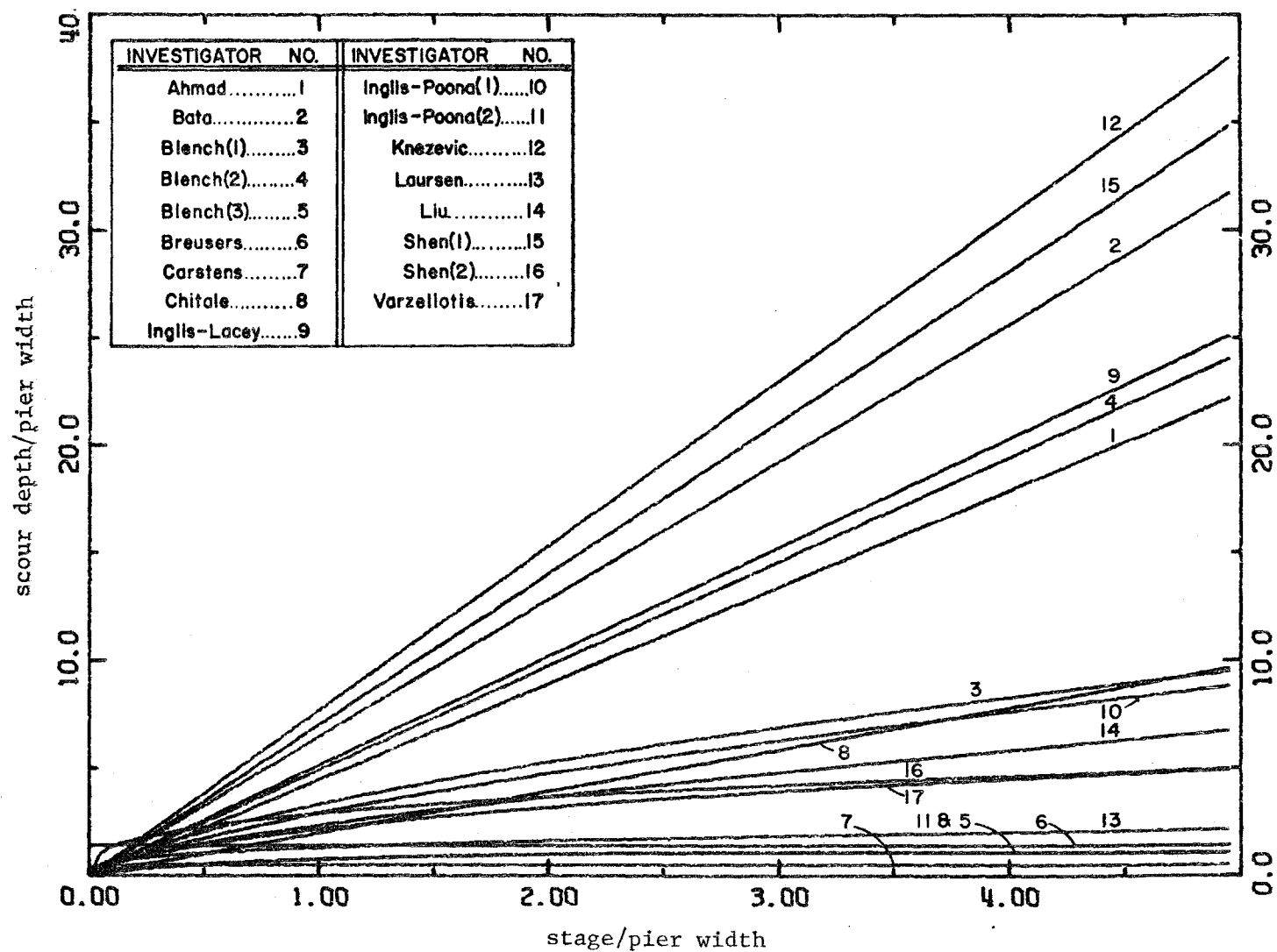


Figure 10. Scour prediction curves: $\frac{D}{b}$ vs $\frac{\text{stage}}{\text{pier width}}$ for Froude No. = 0.8.

These values are plotted as a dashed line on the comparative curves in Figure 5 and Figure 8. From these curves, a prediction of scour depths based on these examples range from zero to 29.7 feet. It is obvious that the discrepancy is even worse for flood conditions.

At this time it is not possible to qualify scour prediction formulas according to their accuracy in predicting scour depths. Field data will give us confidence in laboratory data and define the ranges of applicability of the existing formulas. If this study reaches a successful completion, data will be available to make this judgment. Until that time, little can be done.

III. STUDY OBJECTIVES

3.1.0 Objectives

As discussed in the preceeding chapter, the primary aim of this study is to validate existing scour prediction formulas with field data. Specifically, this study will:

1. Measure the depths of scour around bridge piers.
2. Supplement the scour measurements with other pertinent field data.
3. Evaluate the effectiveness of state-of-the-art scour theories in predicting scour.

3.2.0 Parameters

Before planning a field study on scour around bridge piers, it is first necessary to determine which parameters are pertinent to scour, and, therefore, which are to be measured.

Based on the scour prediction formulas, the scour parameters are summarized in Table 5. With the exception of scour depth, the parameter most often taken into consideration is PIER GEOMETRY, which includes the general pier shape and pier dimensions.

VELOCITY is the second most used parameter in the prediction formulas and the least clearly defined. Some formulas use what they define as an undisturbed velocity, others use an upstream velocity, but don't specify how far upstream, and still others use a velocity without defining its location.

Both STAGE and DISCHARGE appear in the formulas. In general, stage and discharge have a very definite relationship to each other and discharge versus stage rating curves are widely accepted to express one in terms of the other.

CHANNEL GEOMETRY appears in many formulas and includes such parameters as:

- channel width and width change
- contraction at the bridge crossing
- channel slope
- channel bed shape and overbank (flood plan) dimensions
- channel curvature

BED MATERIAL PROPERTIES are included in many of the scour prediction formulas selected for study. They usually include size, size distribution and specific gravity of the bed material. Other properties often considered are bed roughness and vegetation; bed critical shear stress; cohesive or non-cohesive material; and material composition.

AUTHOR	PIER GEOMETRY	VELOCITY	STAGE	DISCHARGE	CHANNEL GEOMETRY	BED MATERIAL PROPERTIES	FLOW DIRECTION	VISCOSITY
Liu								
Maza-Sanches								
Carstens								
Shen 1								
Shen 2								
Shen 3								
Inglis-Poona 1								
Inglis-Poona 2								
Varzeltotis								
Laursen								
Ahmad								
Blench 1								
Blench 2								
Breusers								
Larras								
Bata								
Chitale								
Knezevic								
Blench 3								
Inglis-Lacey								

Table 5. Scour Prediction Formulas Parameters

FLOW DIRECTION is considered in many of the formulas since it has been demonstrated that the angle of attack of the pier influences depth of scour.

Fluid VISCOSITY is considered in one equation and since viscosity is temperature dependent, temperature is a significant parameter to be considered.

It follows that to effectively evaluate the scour prediction formulas, a field study must gather data on:

1. Scour depth
2. Pier geometry
3. Flow velocity
4. River stage
5. Discharge
6. Channel geometry
7. Bed material properties
8. Flow direction
9. River temperature

3.3.0 West Virginia University Field Study

The objectives of the field study are met in the following manner. Stage and scour depth at the pier are measured by an automated scour monitoring system. This system is fixed to the pier and periodically measures and records both stage and scour depth at the pier. Discharge is determined from rating curves for the resulting values of stage. Frequent updating of these curves are made from U.S.G.S. discharge measurements.

Pier geometry is obtained from highway department bridge construction plans.

The remaining parameters are periodically measured by field personnel on routine visits. Channel geometry and flow direction are determined for various river stages using survey techniques. Topographic mapping of the channel and bridge crossing is made using fathometer-type survey instruments.

Collections of bottom samples and water temperature measurements are made during site visits.

Velocity measurements are made in two ways. Two-dimensional velocity vectors are measured in the immediate vicinity of the scour hole by velocity meters permanently installed on the bridge pier. Undisturbed flow velocity measurements, called for in some scour prediction formulas, are made using mobile instrumentation from a boat.

Field sites have been selected that are representative of different types of rivers and bridge crossings. These sites display scour potential and have a probability of frequent flooding.

IV. SITE SELECTION

4.1.0 Criteria

The aim of West Virginia University was to select existing highway bridges in such a manner that maximum useful data could be gathered. The two primary considerations influencing the choices were: 1) assurance that the bridges be located on alluvial rivers where pier scour is experienced and 2) sufficiently high river stages were likely to occur during the monitoring period so that meaningful pier scour data could be collected.

In order not to add additional parameters or make the existing parameters more complicated the following criteria were also considered in the selection of bridges:

- 1) Standard pier geometry with no scour protection device
- 2) Straight channel
- 3) Limited flow angle
- 4) Pier located in the water near middle of river
- 5) No added flow conditions due to adjacent bridges or structures

For reasons of convenience additional considerations in the bridge selection were as follows:

- 1) Ease in which the bridge could be safely and economically instrumented
- 2) Access to the pier and river during major floods
- 3) Availability of an airport for flood alert purposes
- 4) Minimizing travel

4.2.0 Bridges Selected

Many bridges were reviewed through the cooperation of the Federal Highway Administration and State Highway Departments. Although many of the bridges reviewed were found acceptable, those not found acceptable usually failed to meet several of the criteria conditions. In the final selection, a great amount of emphasis was placed on minimizing travel, by selecting sites in the same general section of the country.

The five bridges selected are shown in figures 11 through 20. Four out of the five are currently being used as scour monitoring sites. The I-35E Bridge over the Mississippi River was found to be too difficult to instrument, due to the inability to cope with the large currents around the pier.

In addition to meeting the basic criteria conditions, the four bridges selected were located in different watershed areas. This was to insure the lowest probability of encountering two or more flood conditions simultaneously. Finally, the four sites selected were on

ivers of different sizes. Peak discharges for the selected sites are as follows:

- | | |
|---------------------|------------------------------|
| 1) Homochitto River | 30,000 ft ³ /sec |
| 2) Brazos River | 90,000 ft ³ /sec |
| 3) Red River | 300,000 ft ³ /sec |
| 4) Ohio River | 900,000 ft ³ /sec |

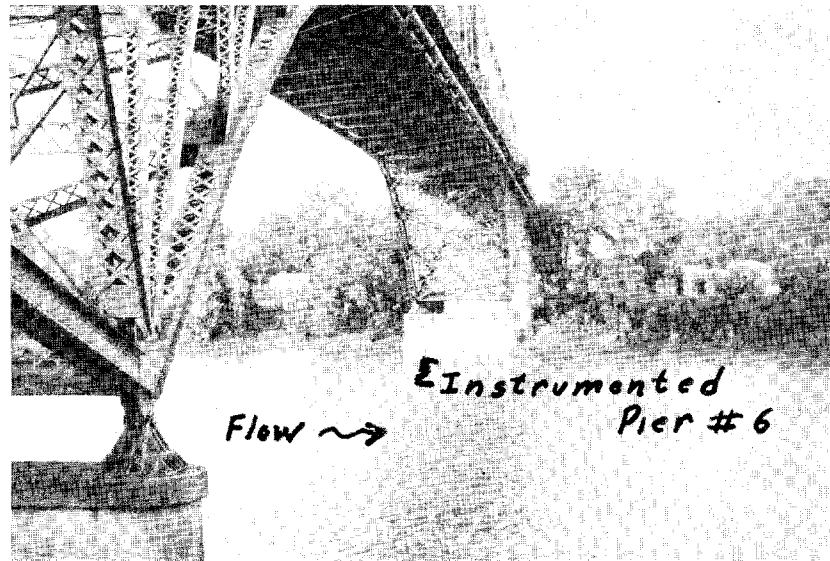


Figure 11. Texas Street Bridge over the Red River at Shreveport, La.

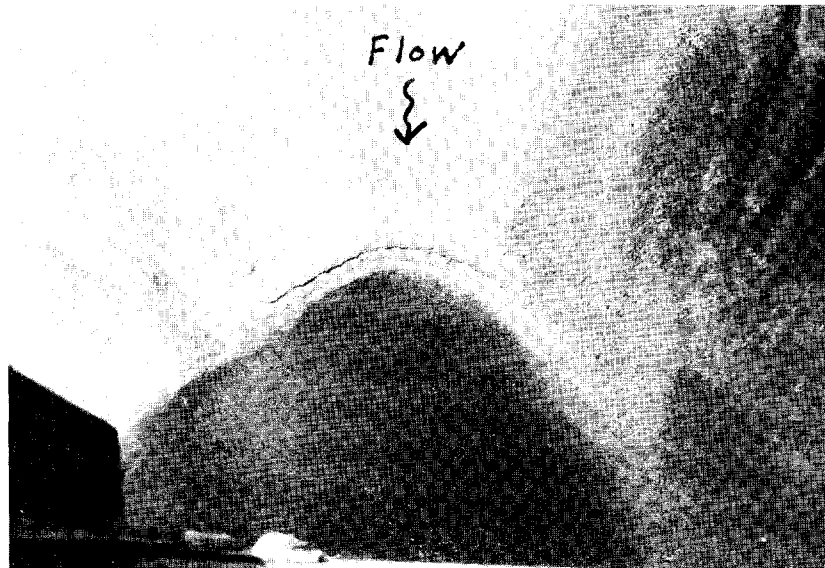


Figure 12. Scour hole around front of Pier 6, Red River.

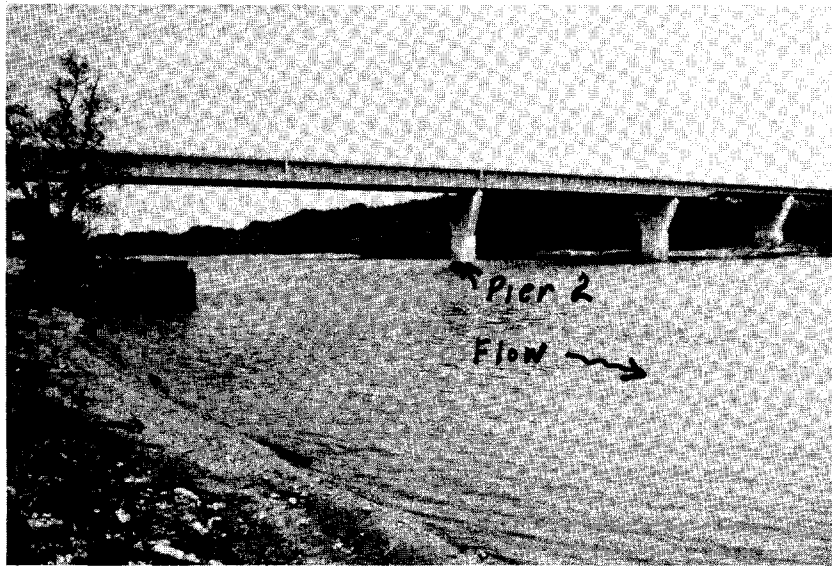


Figure 13. I-35E Bridge over the Mississippi River at St. Paul, Minn.

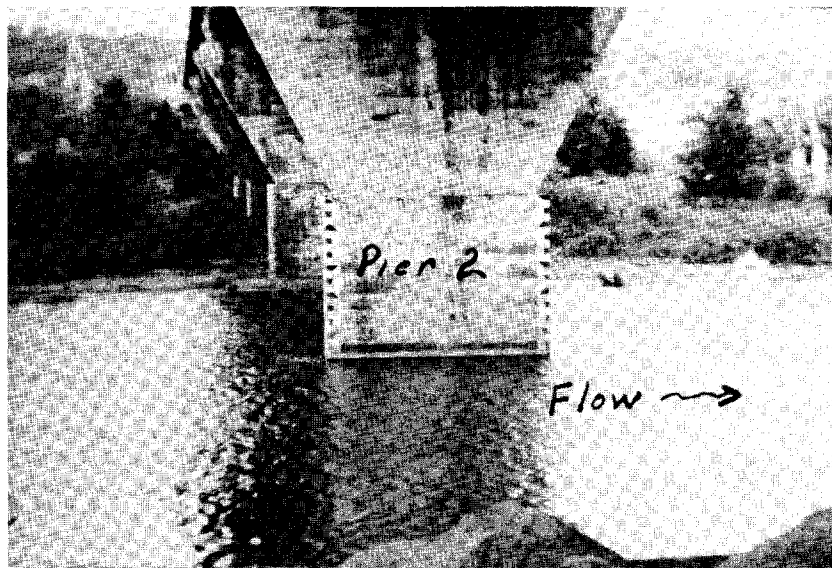


Figure 14. Pier #2, I-35E Bridge.

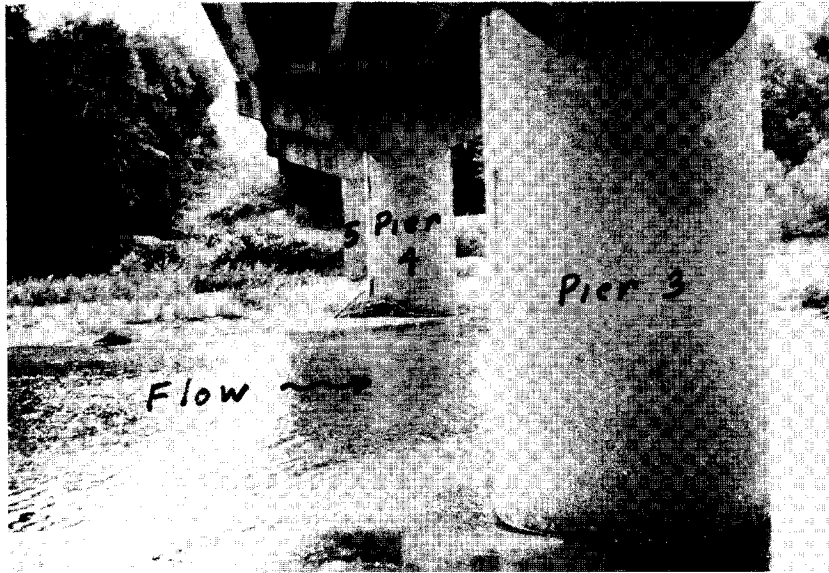


Figure 15. U. S. Highway 84 Bridge over the Homochitto River.

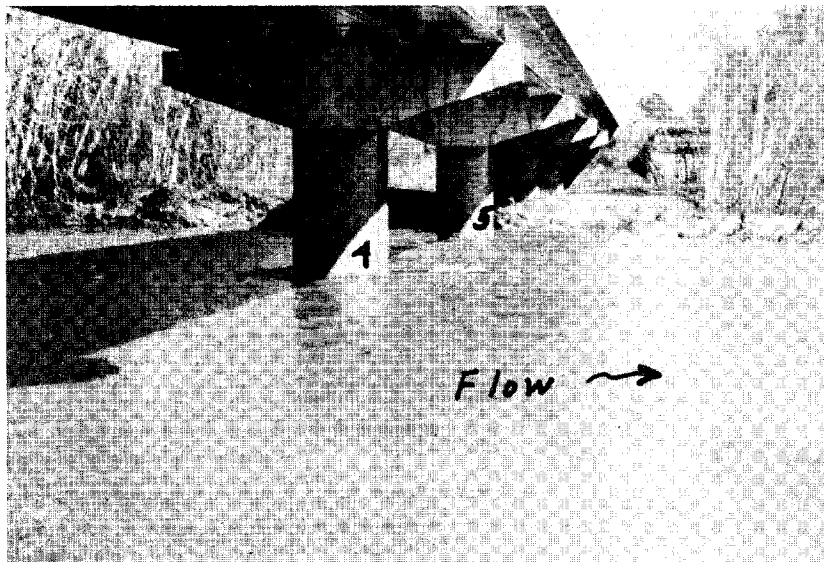


Figure 16. Homochitto River during high flow.

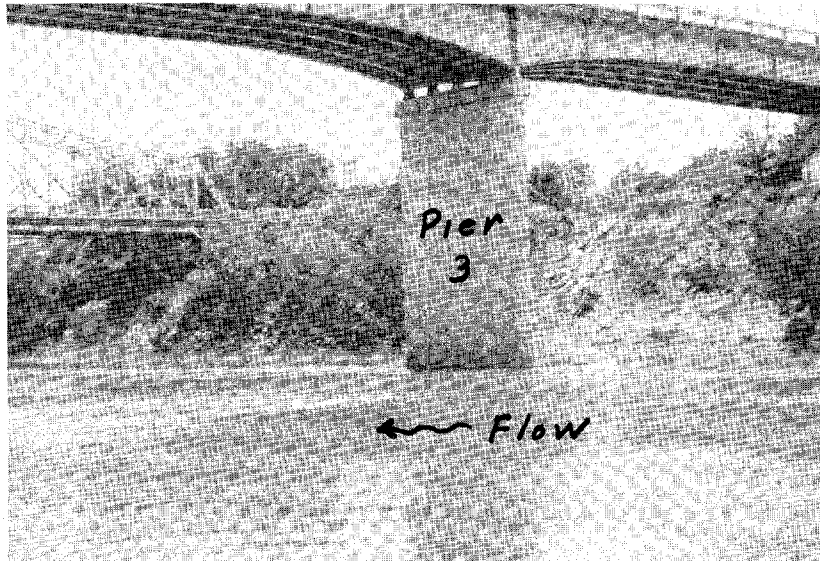


Figure 17. U. S. Highway 59 West Bridge over the Brazos River at Richmond, Texas.

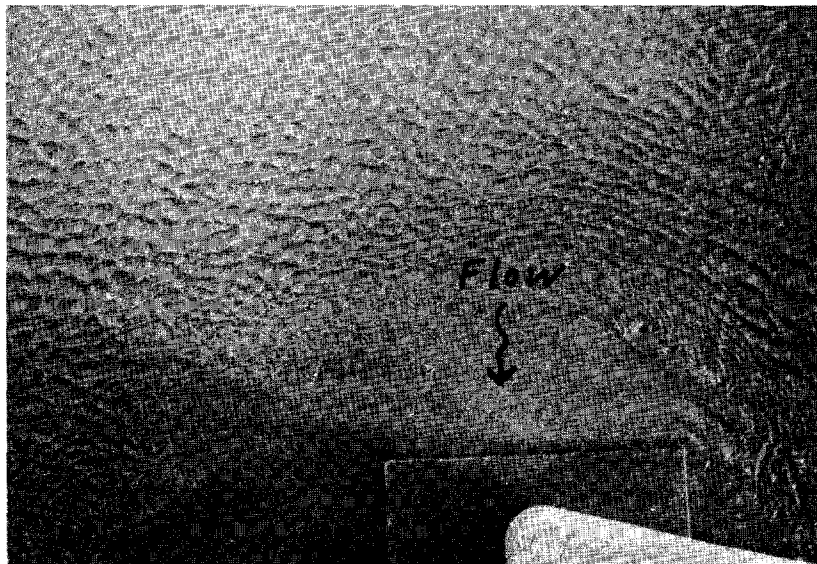


Figure 18. Bow wave on footing of instrumented pier 3, Brazos River.

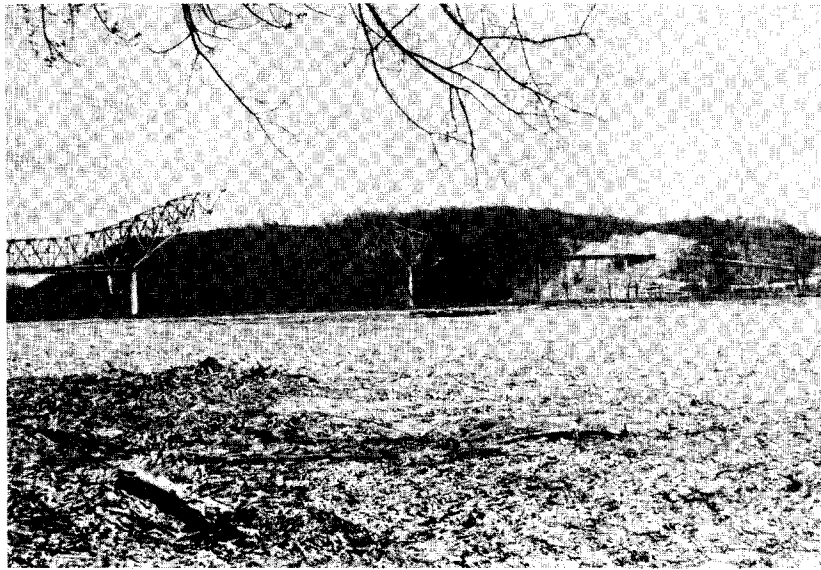


Figure 19. I-275 Bridge over the Ohio River at Lawrenceburg, Indiana
(between Dearborn County, Indiana and Boone County, Kentucky).

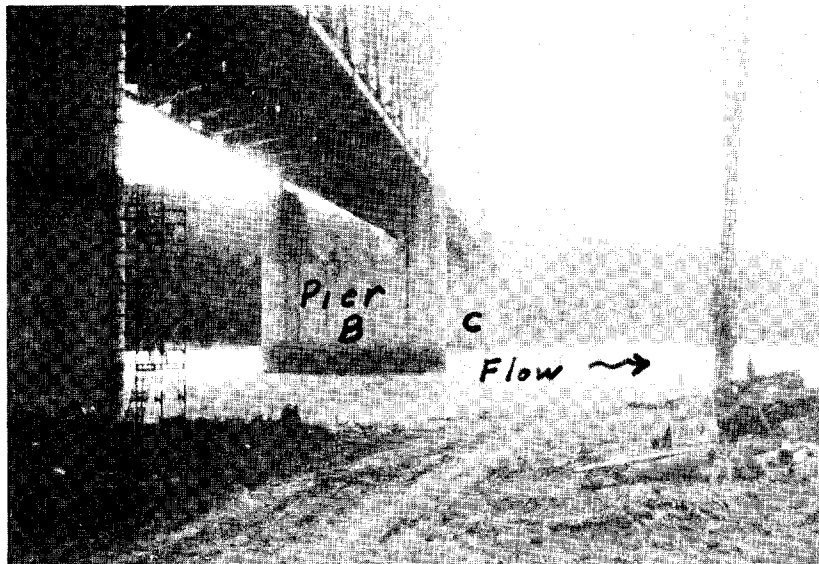


Figure 20. Instrumented Pier B, I-275 Bridge over the Ohio River.

V. DATA

5.1.0 Introduction

The data collected for each scour monitoring site falls into four classifications. First, there is that data which remains constant for a particular site, such as pier geometry and stage versus discharge relations. It has also been determined that for a particular stage the area of the bridge opening and average velocity through the bridge opening fall into a well-defined envelope. This is shown in the data for the Red River at Shreveport, Louisiana. Also falling into this classification is angle of attack vs. stage. That is, once the orientation of the pier with respect to flow direction has been determined for particular stages it will not change significantly. This is due to the fact that the river banks at each site are well-defined at higher stages and have remained stable for many years. In addition, the bridge crossings at all four sites are located in comparatively straight channels.

The second classification of data is that collected periodically by a field team. This data includes, for example, bottom samples, river topography, and velocity profiles. Since many of the parameters needed to investigate the scour prediction formulas are contained in data collected upstream from the bridge crossing, a field team with a boat and mobile instrumentation are required. Most often this data is limited by river conditions.

The automatic recording of data with the scour monitoring instrumentation is the third classification of data. This data includes scour hole depths at the bow of the pier and river stage.

The fourth classification of data is that obtained from various agencies such as the U.S.G.S., the Army Corps of Engineers, River Forecast Centers, etc. This usually includes historical data about the river; such as yearly hydrographs from which flood recurrence intervals are determined. U.S.G.S. and Army Corps of Engineers discharge measurements at bridge sites include depth profiles at the bridge crossing. It is hoped that eventually enough scour monitoring data can be collected to correlate maximum scour depth with the bridge crossing profiles. The present method of discharge measurement using a weighted bomb does not permit accurate depth readings near the bow of the pier.

5.1.1 Data

The presentation of data in this chapter is given for each site in a similar format. A general description of the bridge, instrumented pier and waterway is first given for each site. This is followed by information on U.S.G.S. gaging stations and historical data such as average and extreme discharges. Finally, collected data is presented which gives bottom sample analysis, topographic maps of the bridge crossing and waterway, cross-sectional views showing both general and local scour at different river stages, and flood histories. Due to the installation of

the sites at different periods and problems encountered keeping the instrumentation operable in adverse river conditions, there is a variation in amounts of data available from each of the sites.

5.2.0 Brazos River, Texas

The Scour Monitoring Instrumentation for this site is located on the U. S. Highway 59 West Bridge over the Brazos River at Richmond, Texas. The bridge is 942 feet long and is supported by ten piers, figure 21(a). Pier shape and dimensions of the instrumented pier 3 are shown in figure 21(b). During low and normal flow (stages 1 to 6, U.S.G.S. gage, datum 40.94' M.S.L.) the river is approximately 250 to 300 feet wide with the main channel about half way between piers 2 and 3. During higher flows (stage 24) the river is approximately 360 feet wide with the main channel shifting to pier 3.

A U.S.G.S. gaging station (No. 08114000) is located on the downstream U. S. 59 East Bridge. The following information about that gaging station is reproduced from "Water Resources Data for Texas" prepared by the United States Department of the Interior - Geological Survey, in cooperation with the state of Texas and with other agencies.

Location: Lat. 29°34'56", long. 95°45'27", Fort Bend County, on right bank at downstream side of downstream bridge on U. S. Highway 59 in Richmond, 925 ft. downstream from Texas and New Orleans Railroad Co. bridge, and at mile 92.0.

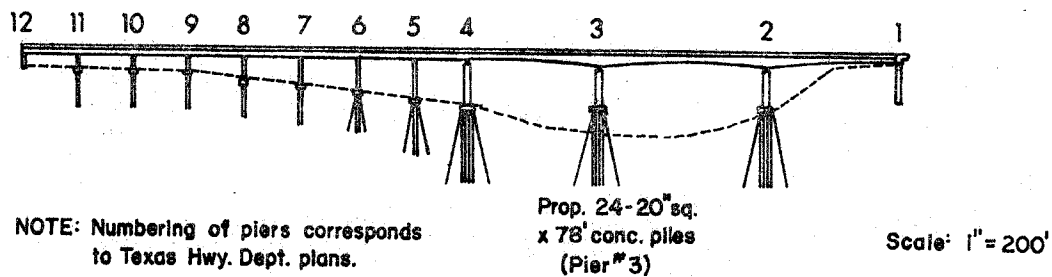
Drainage Area: 44,020 sq. mi. approximately, of which 9,240 sq. mi. is probably noncontributing.

Period of Record: January 1903 to June 1906 and October 1922 to current year. Published as "at Rosenberg" October 22 to September 1931 and equivalent except for diversion by Richmond Irrigation Co.'s canal. June to November 1901 and June to September 1902 in U. S. Department of Agriculture, Office of Experiment Stations, Bulletin Nos. 119 and 133. Gage-height records collected in this vicinity since 1914 are contained in reports of the National Weather Service (formerly U. S. Weather Bureau).

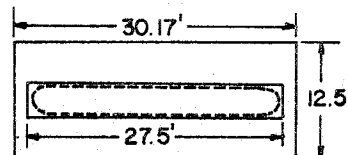
Gage: Water-stage recorder. Datum of gage is 40.94 ft. above mean sea level. Prior to Oct., 1922, various types of nonrecording gages at railroad bridges 925 ft. upstream at different datums. Oct. 1 to Sept. 30, 1922, nonrecording chain gage at Rosenberg 7.6 miles upstream at datum 4 ft. higher.

Average Discharge: 20 years (1903-5, 1922-40) unregulated, 7,209 cfs (5,223,000 acre-ft. per year); 30 years (1940-72) regulated, 7,173 cfs (5,197,000 acre-ft. per year).

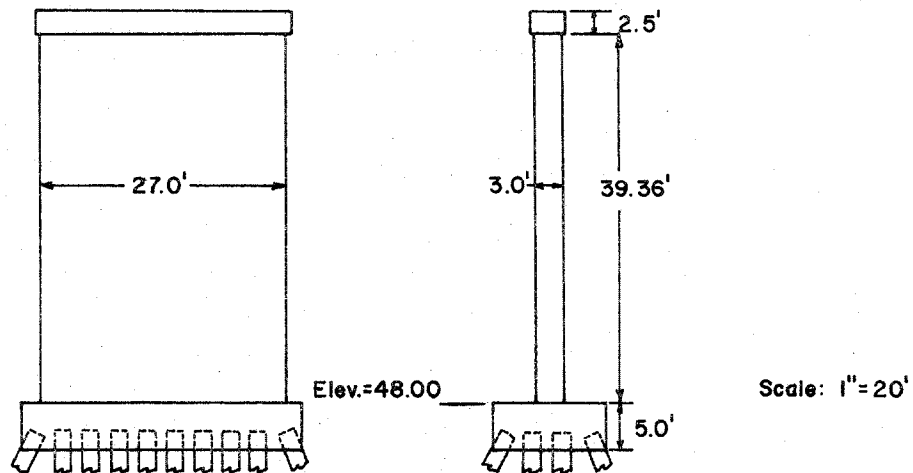
Extremes: Current year (1972): Maximum discharge, 24,400 cfs May 12 (gage height, 14.14 ft); minimum daily, 525 cfs Oct. 4. Period of



a) ELEVATION (Looking downstream)



NOTE: Elevations shown are M.S.L.
Subtract 40.94' to obtain
gage height elevations.



b) INSTRUMENTED PIER 3 DETAILS

Figure 21. (a) U.S. Highway 59 West Bridge over the Brazos River.
(b) Instrumented Pier 3.

record: Maximum discharge, 123,000 cfs June 6, 1929 (gage height, 40.6 ft., from flood marks), present site and datum; minimum daily, 35 cfs Aug. 23, 1934. Maximum stage since at least 1852, 48.2 ft. Dec. 10, 1913, present datum, from floodmarks on right bank 1,000 ft. upstream from gage. From information by Southern Pacific Railroad, stages of other floods at railroad bridges, present datum, are as follows: May 1884, 43.7 ft.; June 13, 1885, 44.7 ft.; July 1889, 45.6 ft.; May 2, 1915, 43.3 ft.; May 9, 1922 40.9 ft.

In Figure 22 are core borings taken in the vicinity of the instrumented pier. These borings show the river bottom to be made of silty sands or sand-silt mixture. Below this is a layer of inorganic clays of high plasticity, fat clay. A bottom sample was collected on the scour hole along the centerline of the pier, between the most upstream pilings. A grain size analysis of that sample showed $d_{50} = 0.16\text{mm}$, figure 23. Core boring blow counts are tabulated in Table 6.

A discharge rating curve for this site is shown in figure 24. The rating curve was reproduced from a U.S.G.S. Rating No. 7, dated 1 Oct., 1971. The discharge intensity rating curve is produced from the discharge rating curve and field measurements of river width (Dis. Int. = discharge/river width).

Figure 25 is a topographic map of the Brazos River and bridge crossing at stage 12.1 (datum 40.9). The instrumented bridge is approximately 575' downstream of the railroad bridge and approximately 350' upstream of the U. S. 59 East Bridge. The contour map shows the river bottom profile and general flow direction. The cross-sectioned view of the river was taken at the upstream edge of the pier.

A flood history from October 14, 1973 to November 1, 1973 for the Brazos is shown in figure 26. The elevation of the river bottom in front of the pier during low flow (stage 4) is approximately 23.0 ft. M.S.L. The elevation during high flow (stage 24) is approximately 18.0 ft. M.S.L. So, for a rise in stage of 20 ft. the depth of the scour hole increased 5 ft. This is a combination of both local and general, since general degradation of the river bottom was observed. Figure 27 shows this general degradation.

5.3.0 Homochitto River, Mississippi

The U. S. Highway 84 Bridge, 0.8 miles east of Eddiceton, Mississippi is the Scour Monitoring location for the Homochitto River Site. The bridge is 562' 3" long and supported by 6 piers, figure 28(a). Pier shape and dimensions of the instrumented piers are shown in figure 28(b). During normal flow (stages 0.5 to 1.5) the river is approximately 100 feet wide. The main channel during low flow may be anywhere between piers 3 and 5 due to the meandering of the river. During higher flows (stage 10) the river is approximately 350 feet wide with the main channel between piers 3 and 5. Consequently, three piers (3, 4 and 5) were instrumented.

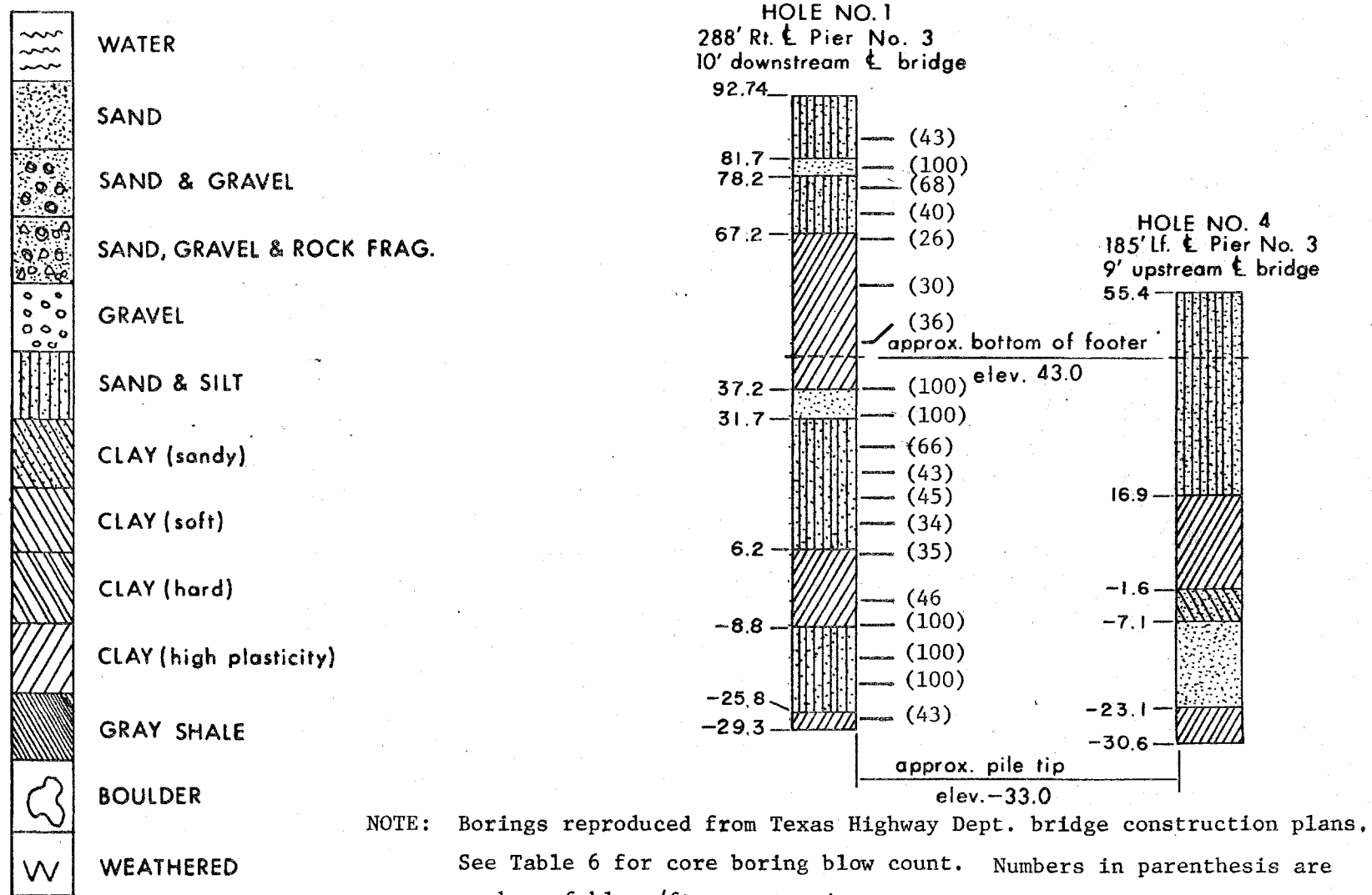
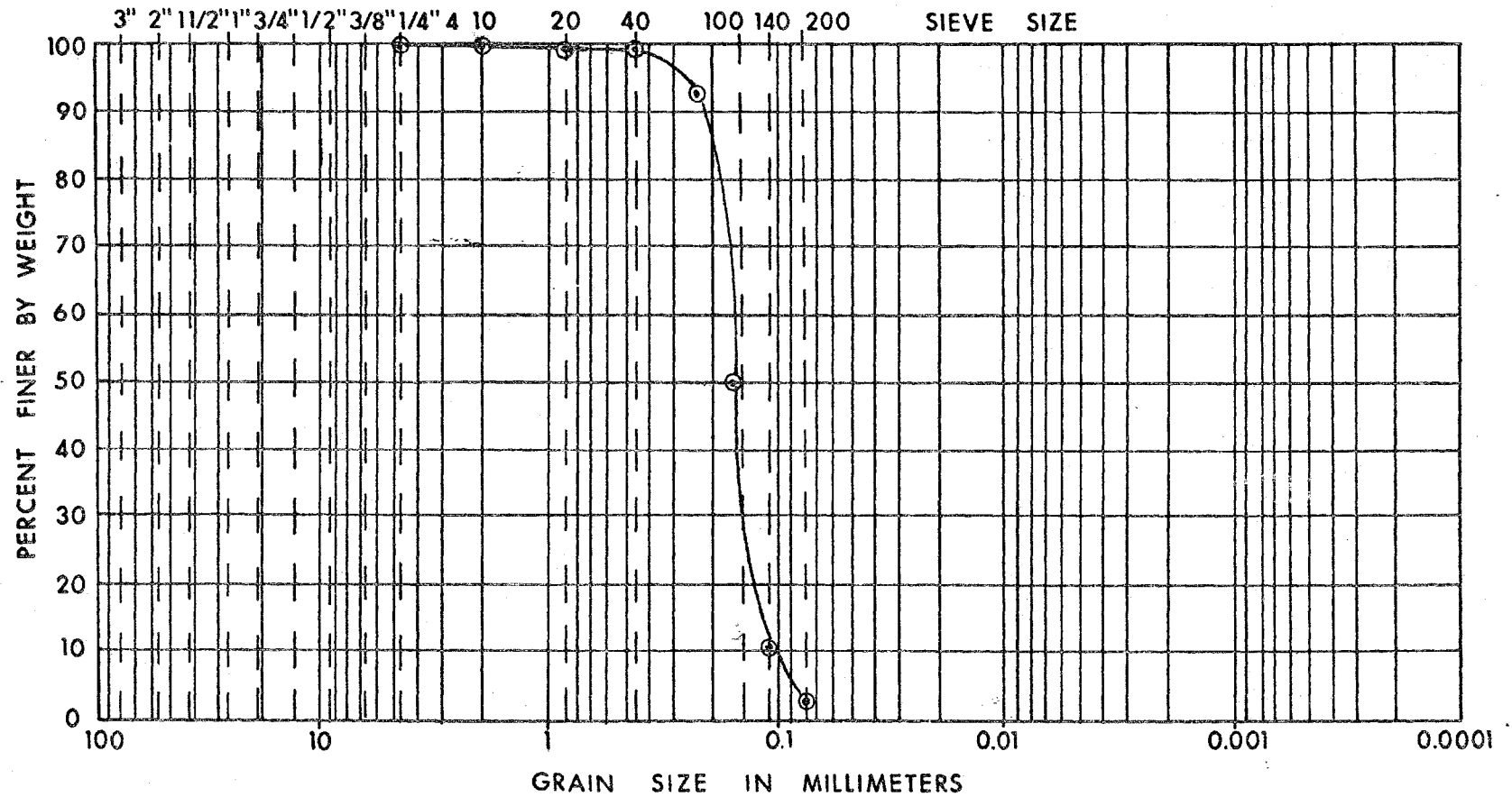


Figure 22. Core borings taken in the vicinity of instrumented pier 3, Brazos River.

Approx. Elev. M.S.L. (ft.)	Weight of Drill Stem (lbs.)	Penetrometer Test	
		No. of Blows/Penetration (in.)	
85	45	25/6	18/6
80	90	50/4.25	50/3
76	90	27/6	41/6
71	135	17/6	23/6
66	135	13/6	13/6
57	180	14/6	16/6
46	225	17/6	19/6
37	270	50/4.38	50/1.25
32	315	50/2.5	50/1.5
26	315	34/6	32/6
21	360	25/6	18/6
16	360	15/6	30/6
11	405	15/6	19/6
5	405	16/6	19/6
-4	450	20/6	26/6
-9	495	50/2.25	50/1.75
-15	495	50/3.0	50/2.13
-20	540	50/2.63	50/2.13
-27	585	19/6	24/6

NOTE: Numbers in the third and fourth columns are added to get number of blows/ft. penetration.

Table 6. Core boring blow count, hole no. 1, Brazos River.



AASHO	GRAVEL			SAND		SILT	CLAY	COLLOIDAL CLAY
	CRS.	MED.	FINE	COARSE	FINE			
PROJECT		DATE	SAMPLE		DEPTH	DESCRIPTION		
BRAZOS		10/30/73	Scour Hole		Elev. 20.0	Grab Sample @ Bow of Pier No. 3		

Figure 23. Grain Size analysis curve, Brazos River.

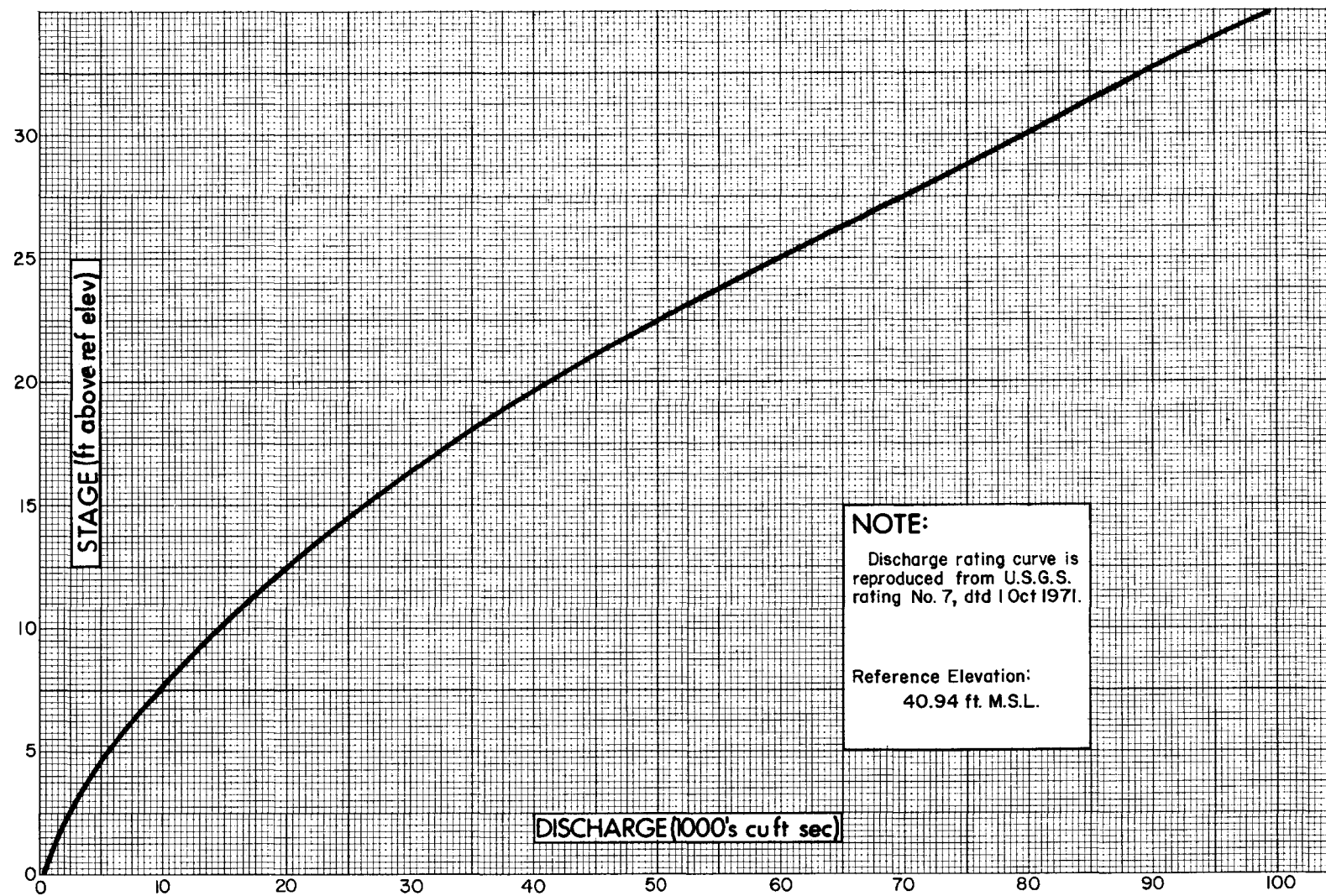


Figure 24. Discharge Rating Curve for the Brazos River at Richmond, Texas.

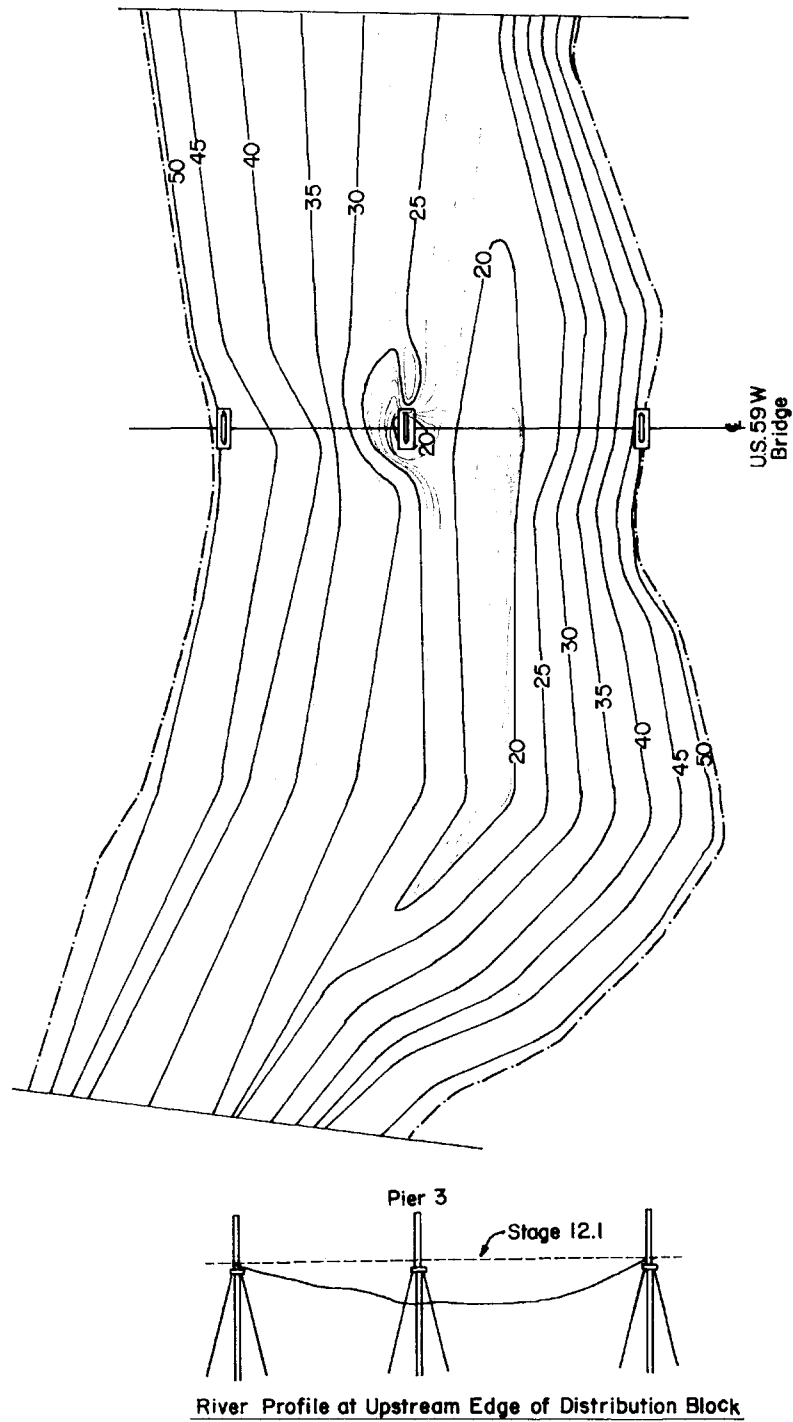


Figure 25. Topographic map of Brazos River in vicinity of U. S. Highway 59 West Bridge.

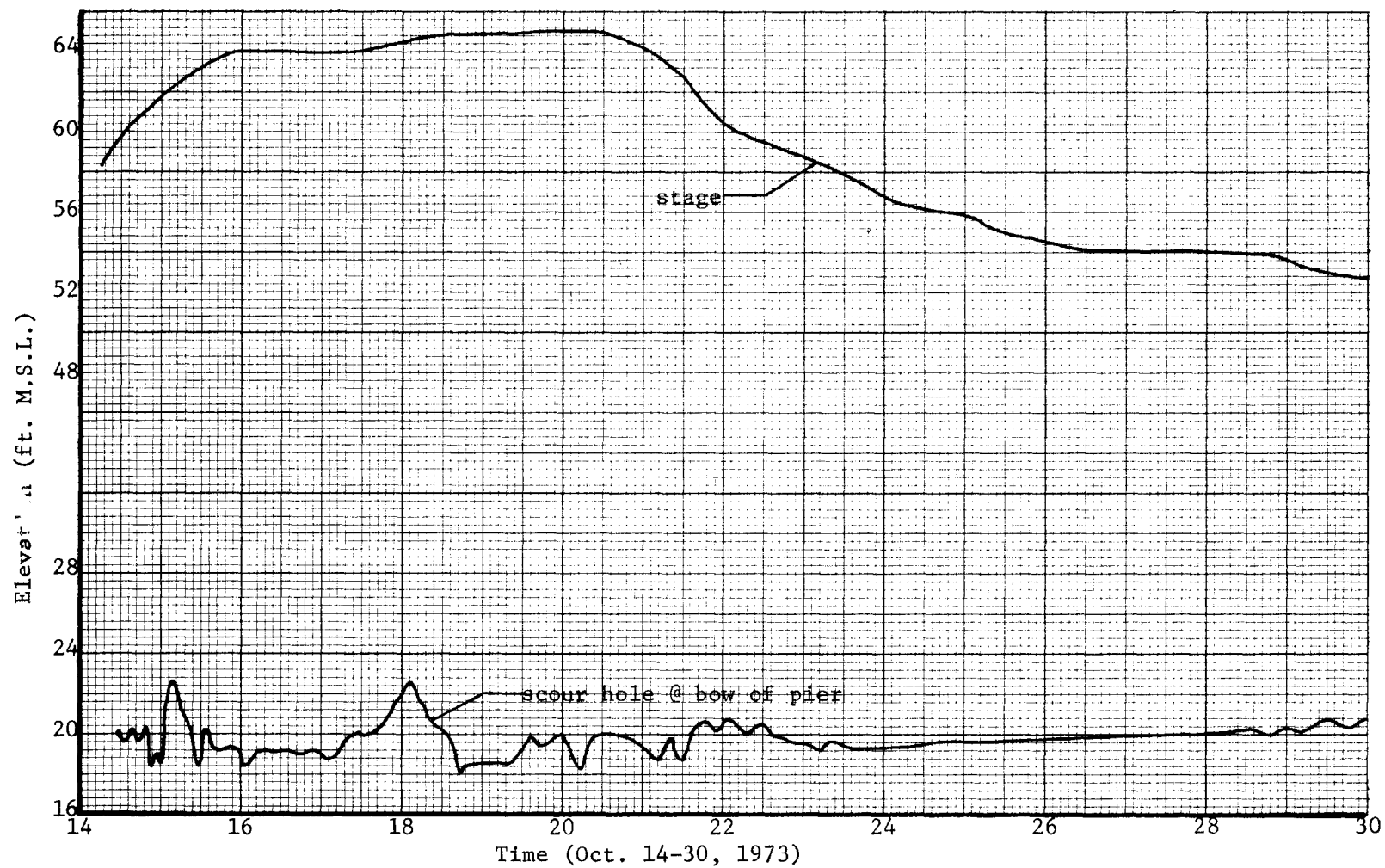


Figure 26. Flood history of Brazos River for period October 14-30, 1973.

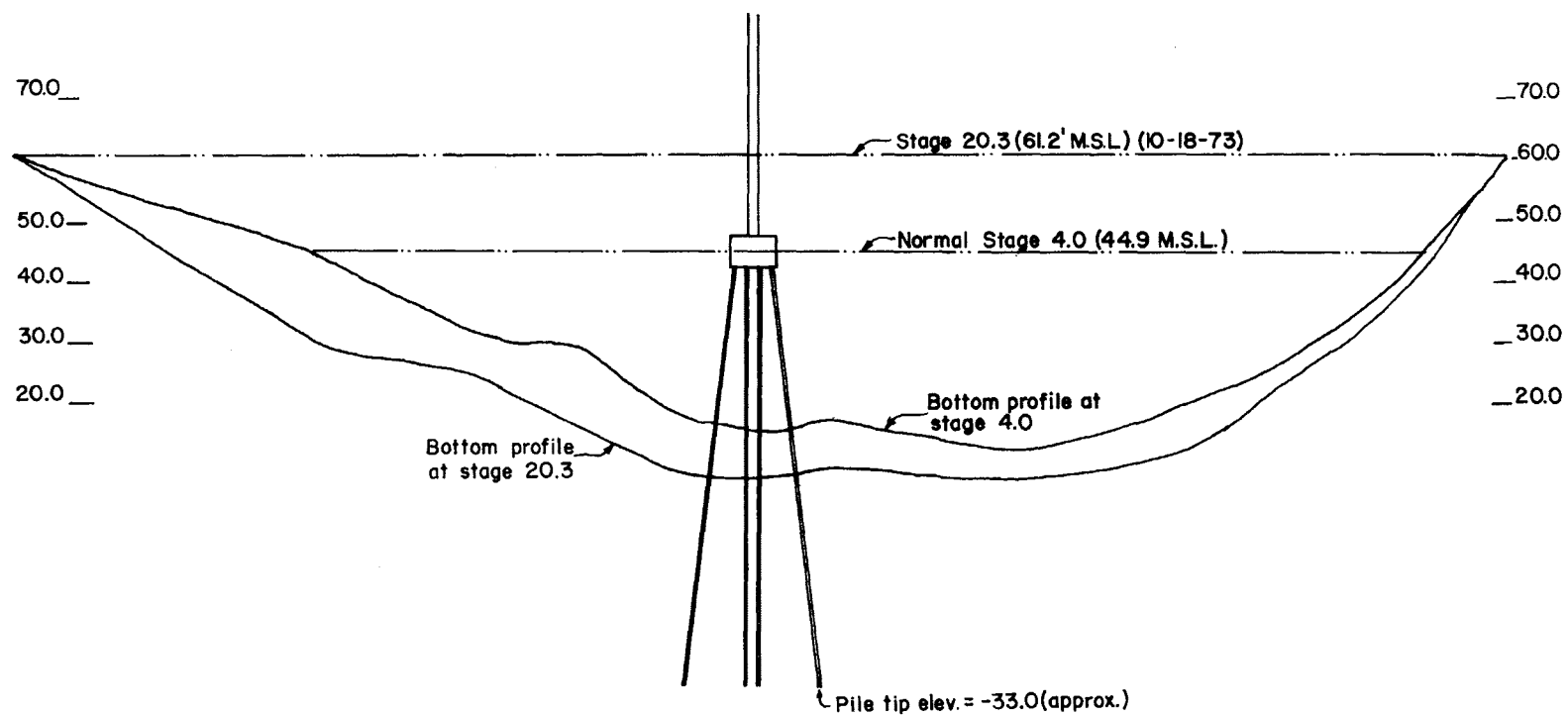
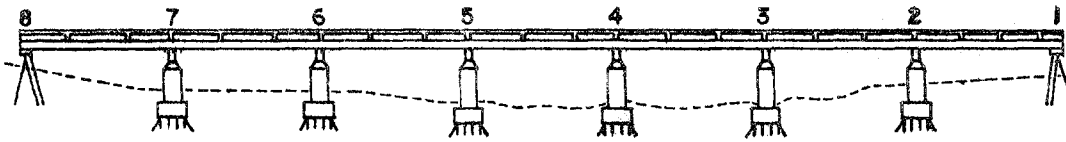


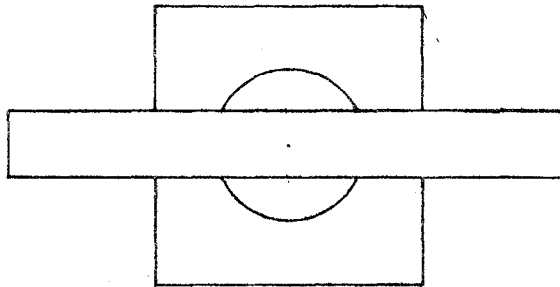
Figure 27. Bottom profiles at Brazos River Bridge crossing.



NOTE: Numbering of piers corresponds to Mississippi State Hwy. Dept. plans.

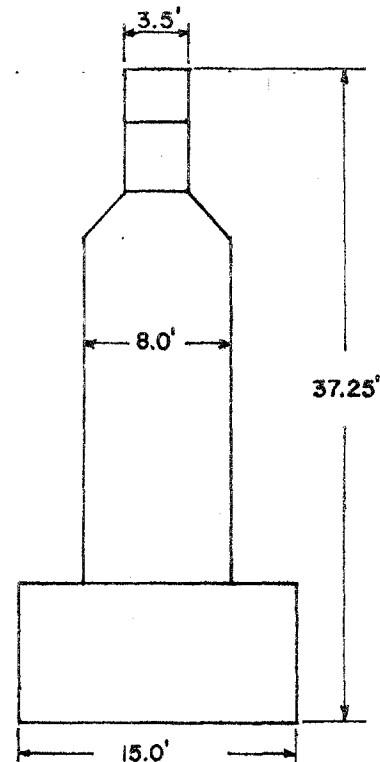
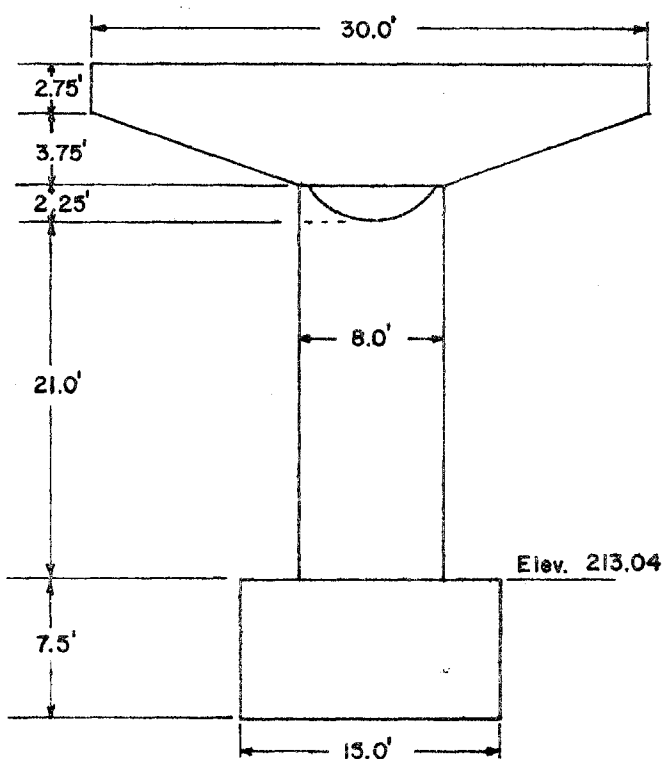
Scale: 1" = 100'

a) ELEVATION VIEW—Looking downstream.



PLAN VIEW

NOTE: Elevations shown are M.S.L.
Subtract 217.22' to obtain
gage height elevations.



Scale: 1" = 10'

b) INSTRUMENTED PIERS 3, 4 & 5 DETAILS

Figure 28. (a) U.S. Highway 84 Bridge over the Homochitto River.
(b) Instrumented piers 3, 4 and 5 details.

A U.S.G.S. gaging station (No. 07291000) is located 900 feet downstream on the Mississippi Central Railroad Bridge. Information on that gaging station from the U.S.G.S. "Water Resources Data for Mississippi" is as follows:

Location: Lat. 31°30'10", long. 90°46'35", in SE 1/4 NE 1/4 sec. 11, T.6 N., R.4 E., Washington meridian, Franklin County, on left bank at upstream side of Mississippi Central Railroad Bridge, 900 ft. downstream from bridge on U. S. Highway 84, 0.4 miles upstream from McCall Creek, and 0.8 miles east of Eddiceton.

Drainage Area: 180 sq. mi., approximately.

Period of Record: October 1938 to current year.

Gage: Water-stage recorder. Datum of gage is 217.22 ft. above mean sea level. Prior to May 26, 1942, nonrecordings gage at site 900 ft. upstream at present datum.

Average Discharge: 33 years, 241 cfs (18.18 inches per year)

Extremes: Current year: Maximum discharge, 7,980 cfs Mar. 2 (gage height, 8.10 ft); minimum daily, 37 cfs Oct. 5. Period of Record: Maximum discharge 30,900 cfs Mar. 29, 1939; maximum gage height, 16.82 ft. Oct. 4, 1964; minimum discharge, 26 cfs Sept. 10, 1952 (temporary regulation); minimum unregulated, 28 cfs Sept. 2, 4, 1954, Sept. 29, 1956.

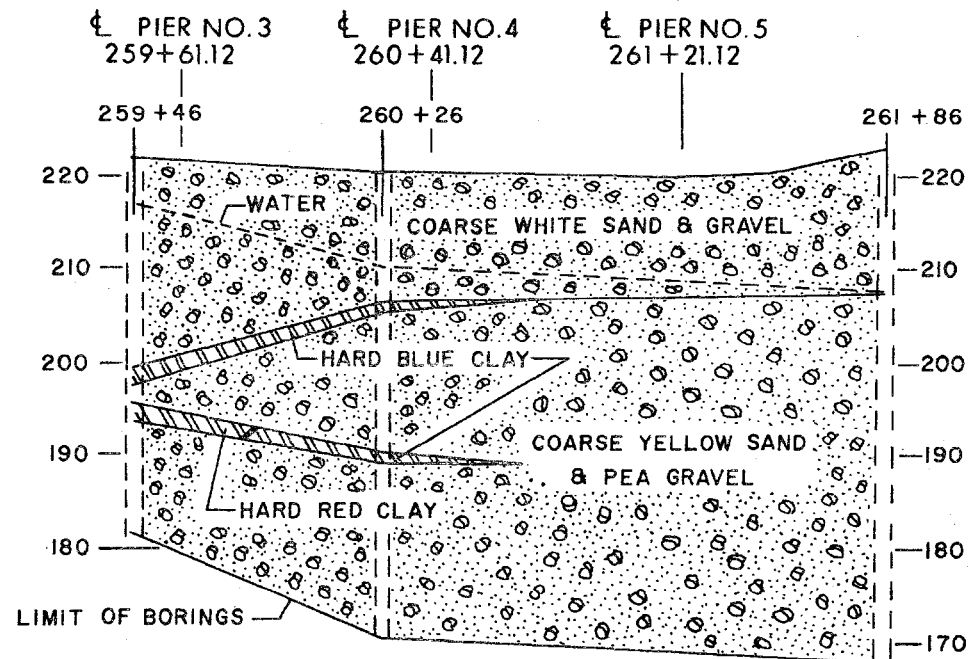
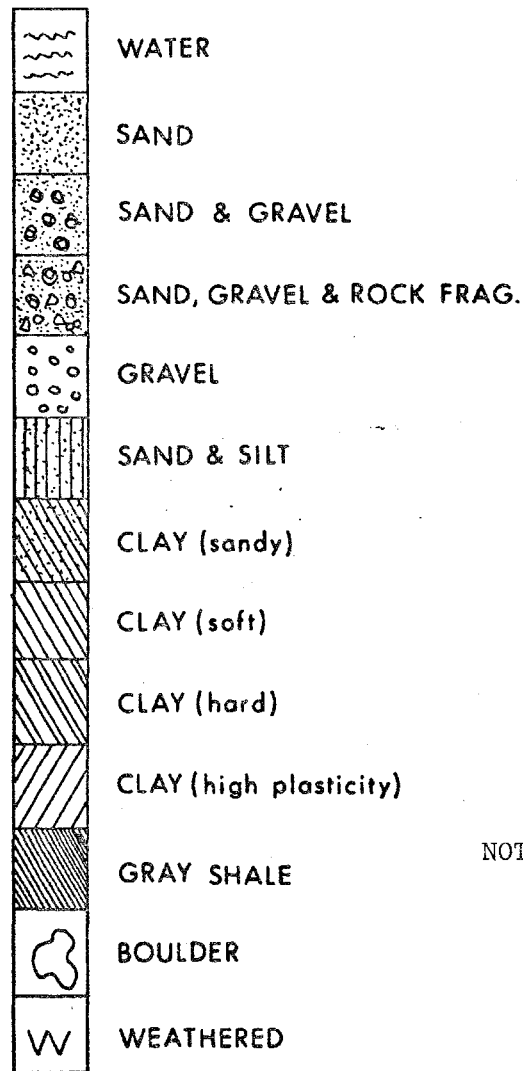
Figure 29 shows core borings taken in the vicinity of the three instrumented piers. These borings show the river bottom to be a sand and gravel mixture. A grain size analysis of the bottom sample taken from the scour hole at pier 3 gave $d_{50} = 0.49\text{mm}$, figure 30. A grain size analysis of a bottom sample taken from between piers 3 and 4 gave $d_{50} = 7.5\text{mm}$, figure 31. Core boring blow counts are tabulated in Table 7.

A discharge rating curve for this site is shown in figure 32. The rating curve was produced from U.S.G.S. rating tables 12 through 22, dated 12 March, 1962 to 30 September, 1973.

5.4.0 Ohio River, Kentucky

The Interstate 275 Bridge over the Ohio River between Kentucky and Indiana near Lawrenceburg, Indiana is approximately 3500 feet long and is supported by 15 piers, figure 33(a). Pier shape and dimensions are shown in figure 33(b). Core borings in the vicinity of pier B are shown in figure 34.

Figures 35, 36 and 37 show the discharge at the bridge crossing; the area of the bridge opening; and the mean velocity through the bridge opening, respectively. These figures were reproduced from the "Prelimi-



NOTE: Borings reproduced from Mississippi State Highway Department Plans.

See Table 7 for blow count on test pile report for pier 5.

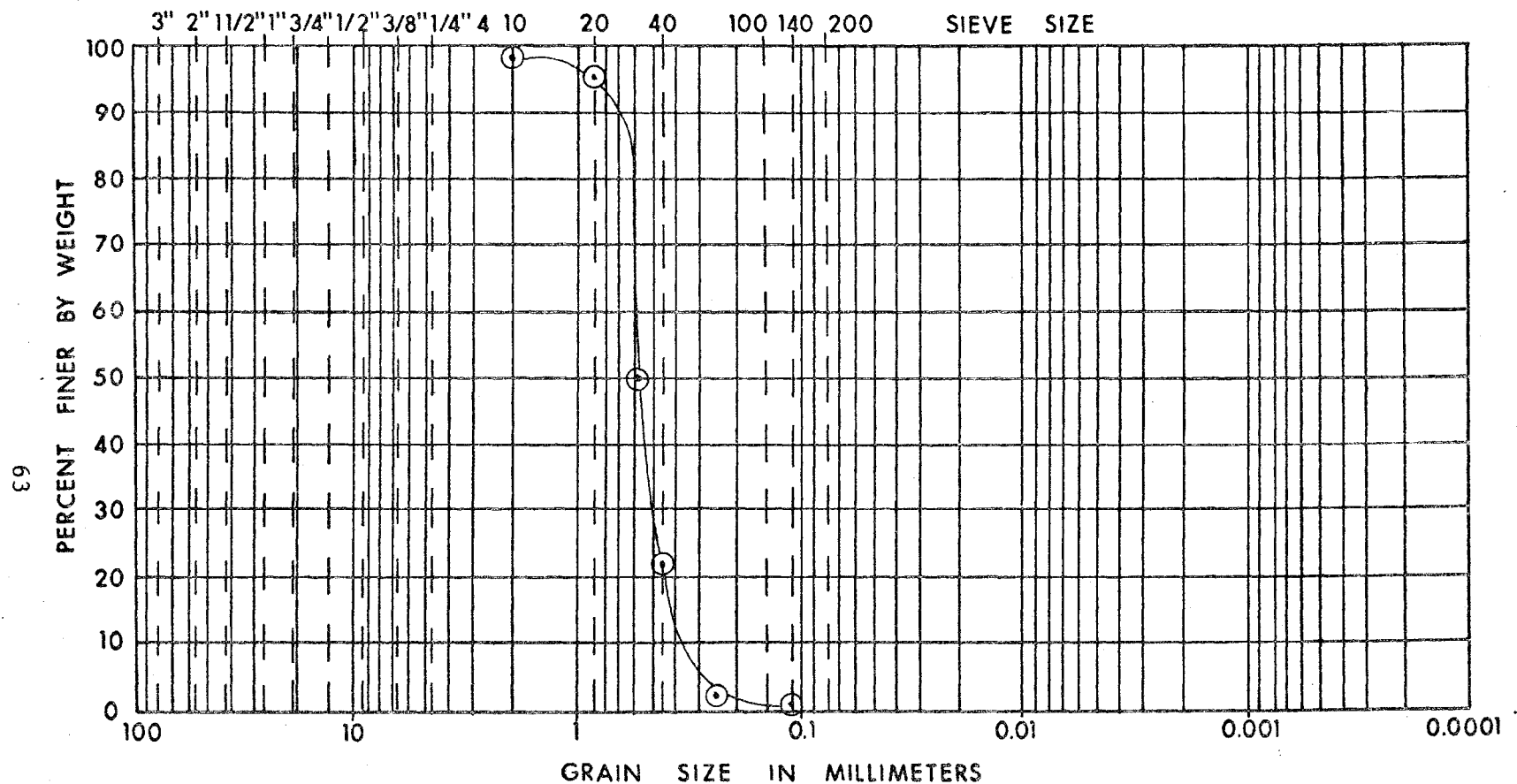
Figure 29. Core borings taken in the vicinity of instrumented piers 3, 4 and 5, Homochitto River.

Length of Pile 45 ft. Butt Size 14 in. Tip Size 14 in.
 Ground Elevation at Test Pile 210.58
 Make & Size of Hammer McKiernan-Terry 10B3

Elevation of Pile Tip	Total Ground Penetration	Blows Per Minute	Energy Per Blow	Blows Per Foot Penetration	Av. Penet. Per Blow Feet	Bearing in Tons
200.58	10	105	13100	0	0	0
199.58	11	"	"	26	0.0385	19.79
198.58	12	"	"	23	0.0435	18.14
197.58	13	"	"	24	0.0417	18.70
196.58	14	"	"	23	0.0435	18.14
195.58	15	"	"	52	0.0192	30.44
194.58	16	"	"	51	0.0196	30.10
193.58	17	"	"	161	0.0062	47.74
192.58	18	"	"	314	0.0032	54.95
191.58	19	"	"	137	0.0073	45.55
190.58	20	"	"	110	0.0091	42.37
189.58	21	"	"	136	0.0074	45.36
188.58	22	"	"	128	0.0078	44.62
187.58	23	"	"	87	0.0115	55.04
186.58	24	"	"	151	0.0066	46.92
185.58	25	"	"	201	0.0050	50.38
184.58	26	"	"	147	0.0068	46.52
183.58	27	"	"	102	0.0098	41.25
182.58	28	"	"	60	0.0167	32.72
181.58	29	"	"	59	0.0169	32.52
180.58	30	"	"	68	0.0147	34.80
179.58	31	"	"	75	0.0133	36.43
178.58	32	"	"	77	0.0130	36.80
177.58	33	"	"	85	0.0118	38.35
176.58	34	"	"	102	0.0098	41.25
175.58	35	"	"	315	0.0032	54.95
174.58	36	"	"	220	0.0045	51.57
173.58	37	"	"	191	0.0052	49.92
172.58	38	"	"	200	0.0050	50.38
171.58	39	"	"	199	0.0050	50.38
170.58	40	"	"	115	0.0087	43.04
169.58	41	"	"	119	0.0084	43.55
168.58	42	"	"	51	0.0196	30.10
167.58	43	"	"	81	0.0123	37.69
166.58	44	"	"	80	0.0125	37.43
166.05	44.53	"	"	48	0.0110	39.46

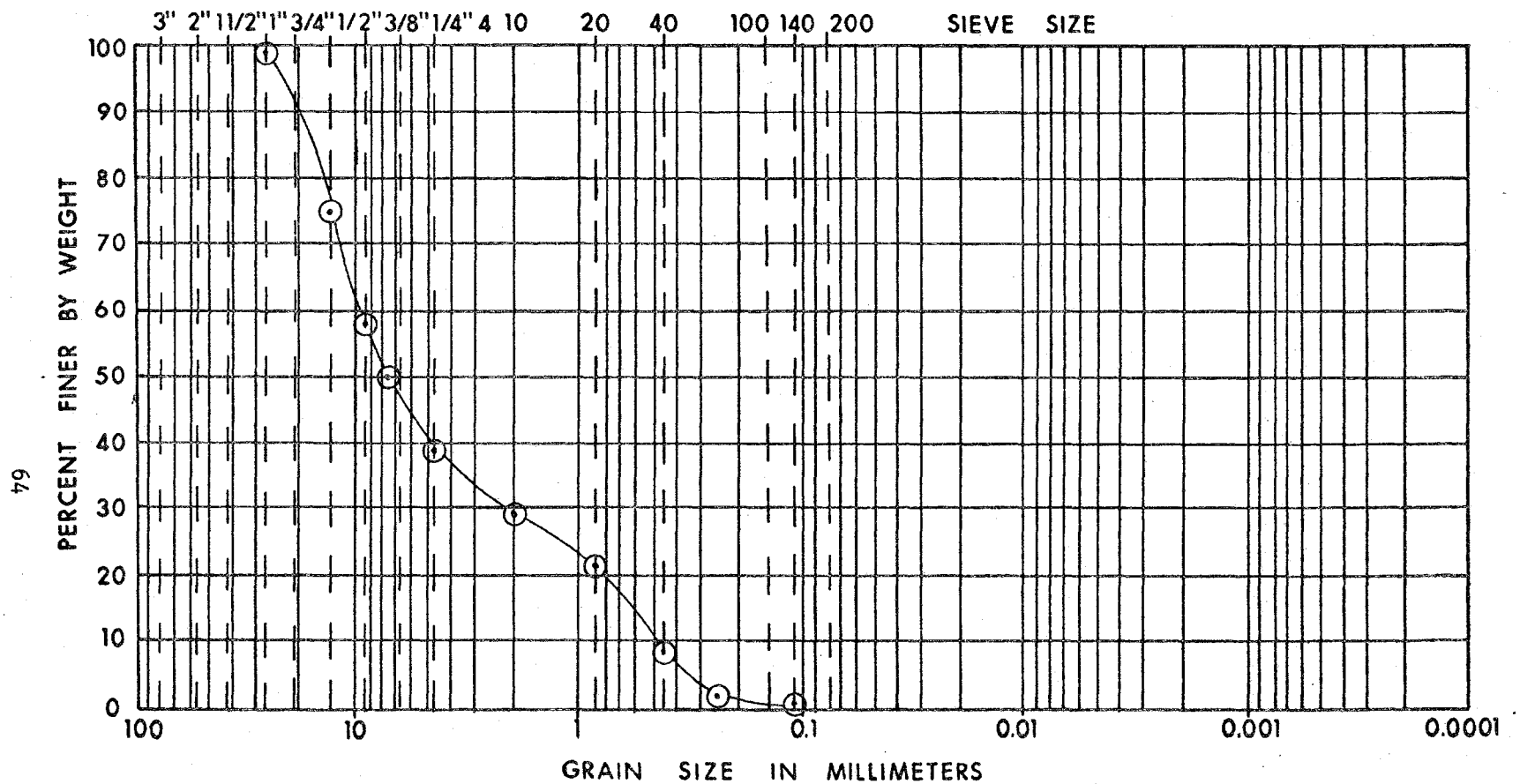
*Note: Test pile report reproduced from Mississippi State Highway
 Department Report, Dated 7-20-59, Project F-015-1(7)

Table 7. Test pile report* for pile #13, pier #5, Homochitto River.



AASHTO	GRAVEL			SAND		SILT	CLAY	COLLOIDAL CLAY
	CRS.	MED.	FINE	COARSE	FINE			
PROJECT		DATE		SAMPLE	DEPTH	DESCRIPTION		
HOMOCHITTO		9/23/74		Scour Hole	Elev. 216.0	Grab Sample @ Bow of Pier No.3		

Figure 30. Grain size analysis curve, Homochitto River.



AASHTO	GRAVEL			SAND		SILT	CLAY	COLLOIDAL CLAY
	CRS.	MED.	FINE	COARSE	FINE			
PROJECT		DATE	SAMPLE	DEPTH		DESCRIPTION		
HOMOCHITTO		9/23/74	between piers	Elev. 218.0		Grab Sample between Piers No.3 + No.4		

Figure 31. Grain size analysis curve, Homochitto River.

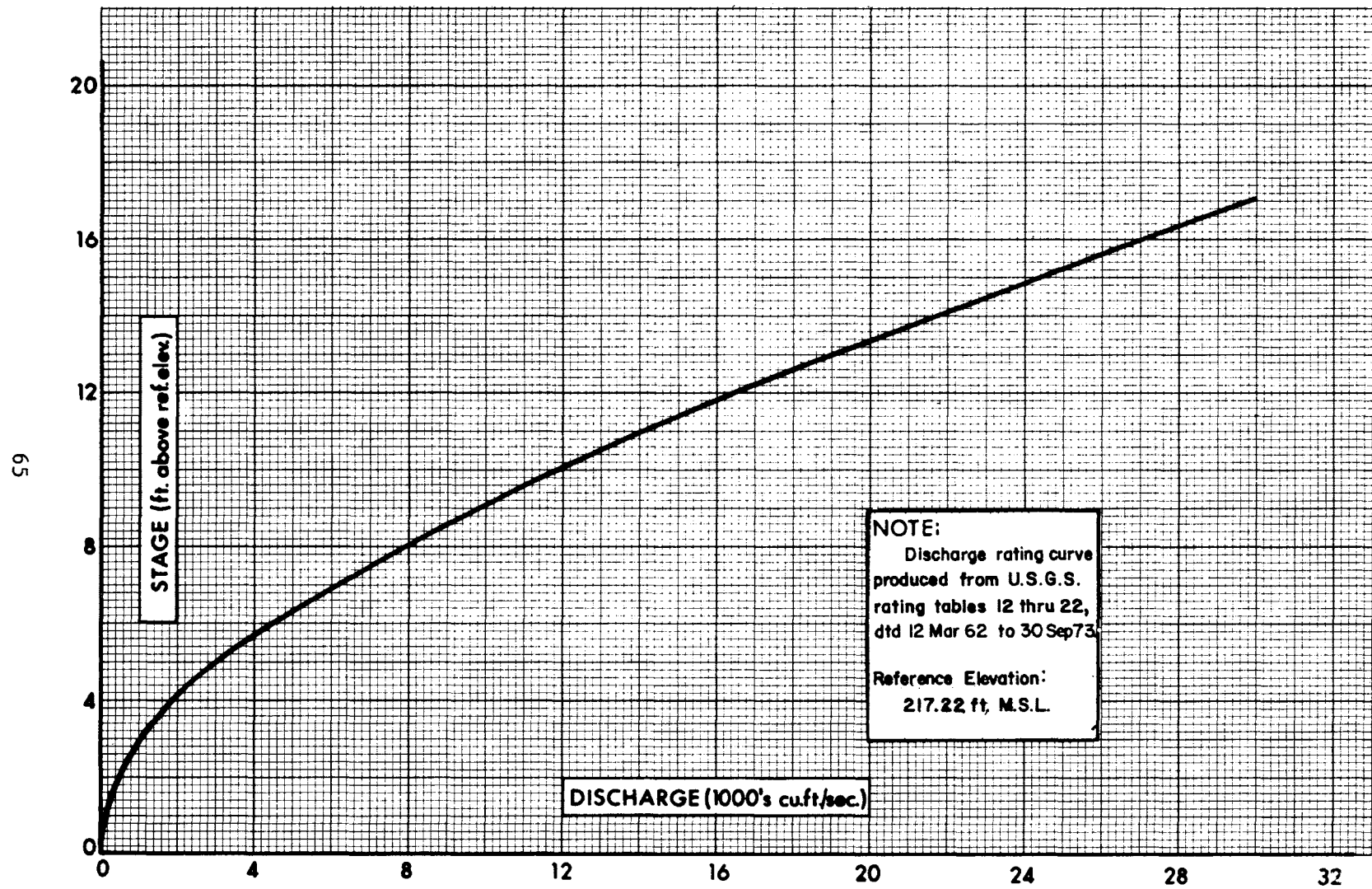
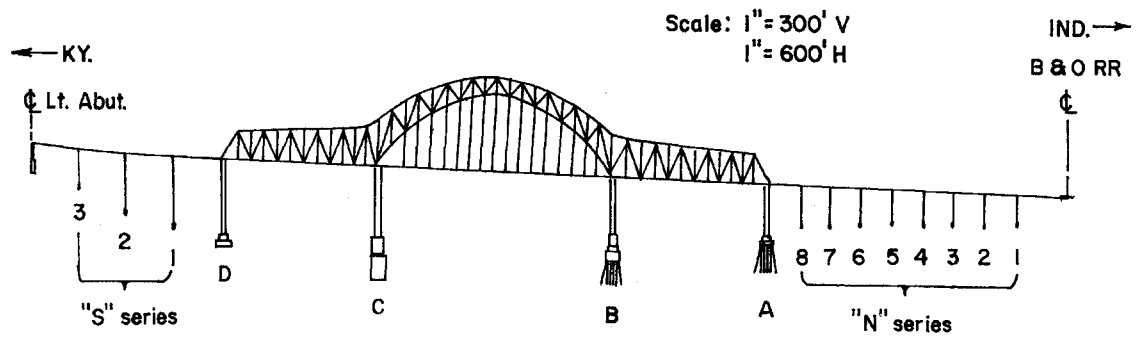
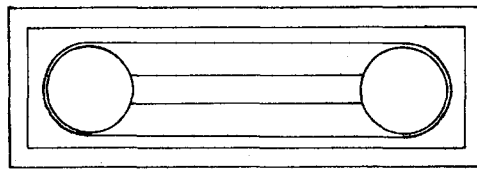


Figure 32. Discharge Rating Curve for the Homochitto River at the U. S. Highway 84 Bridge.



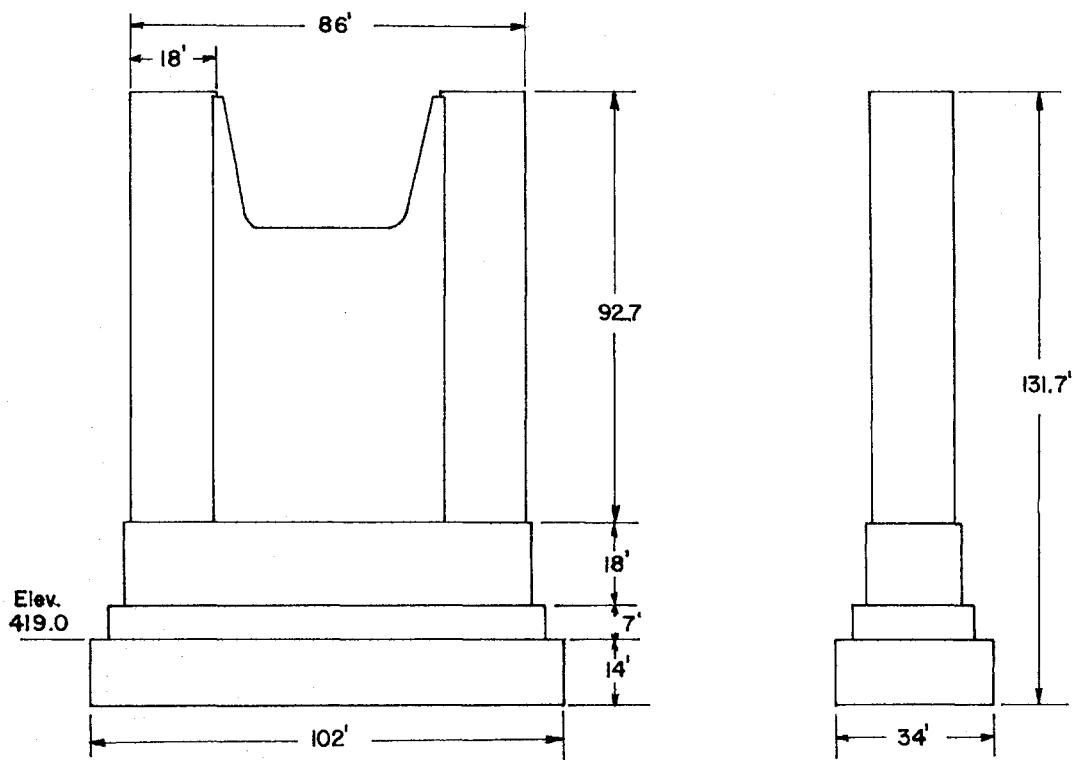
a) ELEVATION VIEW (Looking downstream)

NOTE: Numbering of piers corresponds to Ky. Dept. of Hyvys.—Ind. St. Hwy. Comm. plans.



PLAN VIEW

NOTE: Elevations shown are M.S.L.
Subtract 429.61' to obtain gage height elevations.



Scale: 1" = 40'

b) INSTRUMENTED PIER "B" DETAILS

Figure 33. (a) I-275 Bridge over the Ohio River.
(b) Instrumented Pier B.

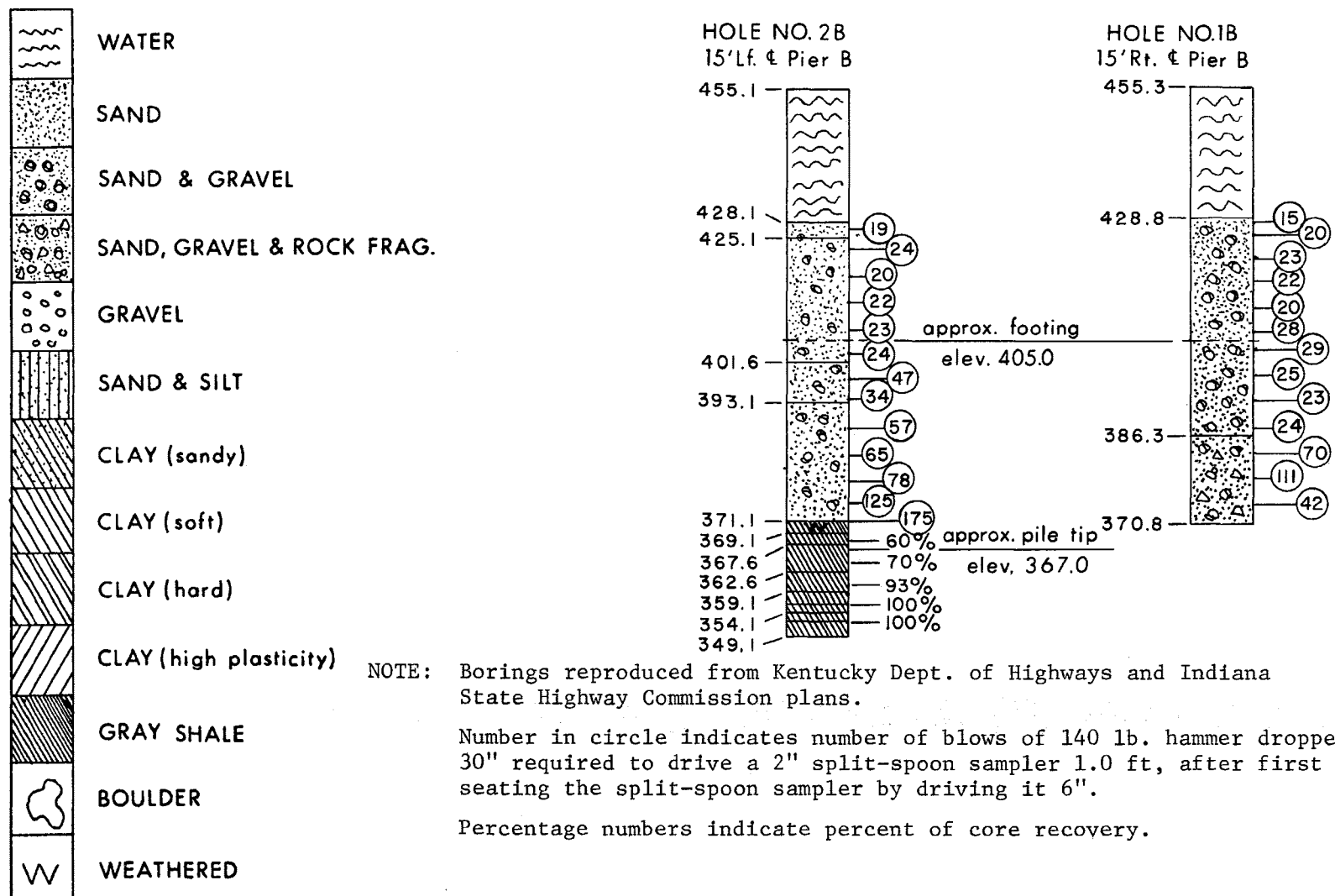


Figure 34. Core borings taken in the vicinity of instrumented pier B, Ohio River.

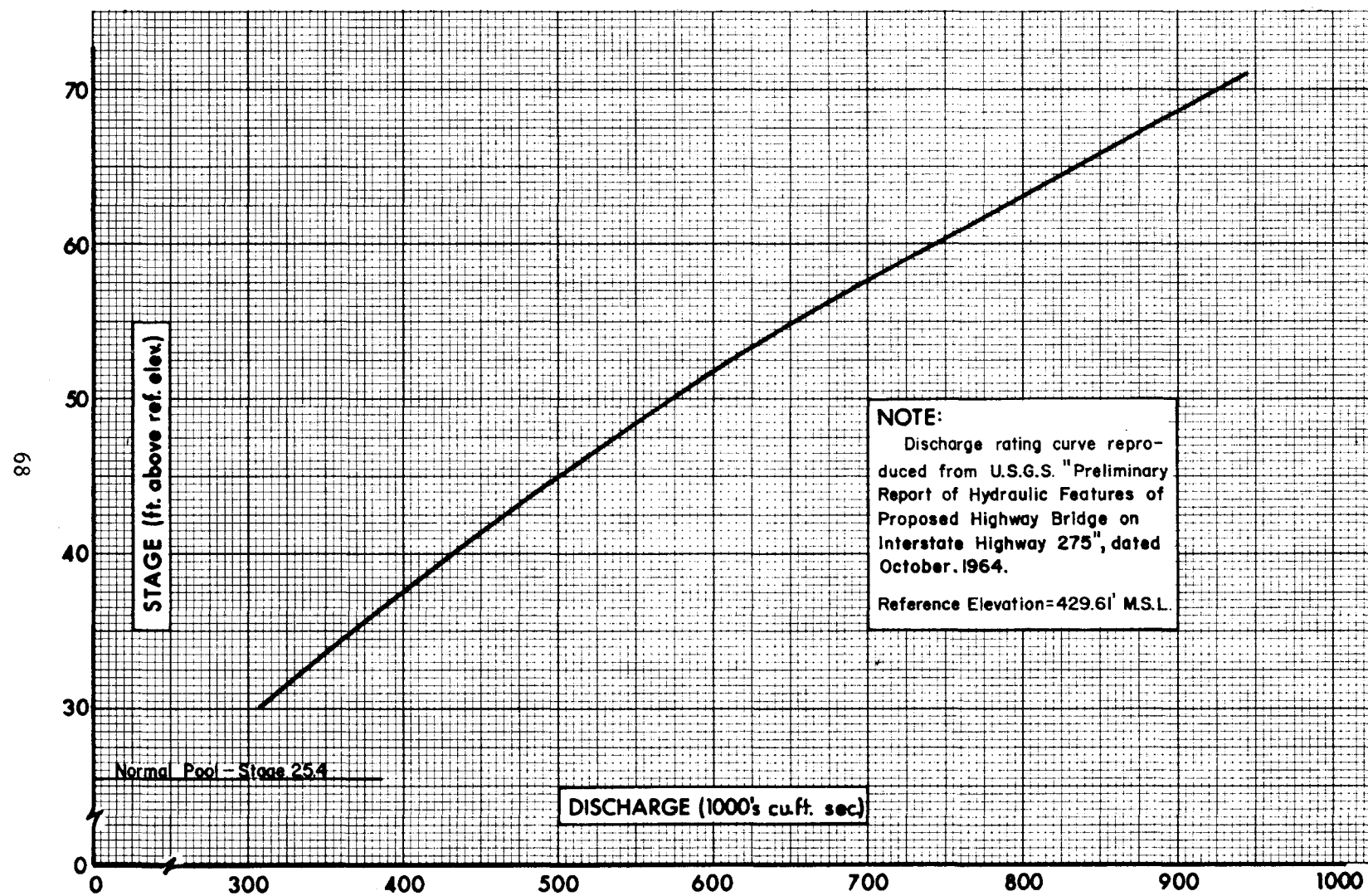


Figure 35. Discharge Rating Curve for the Ohio River at the I-275 Bridge.

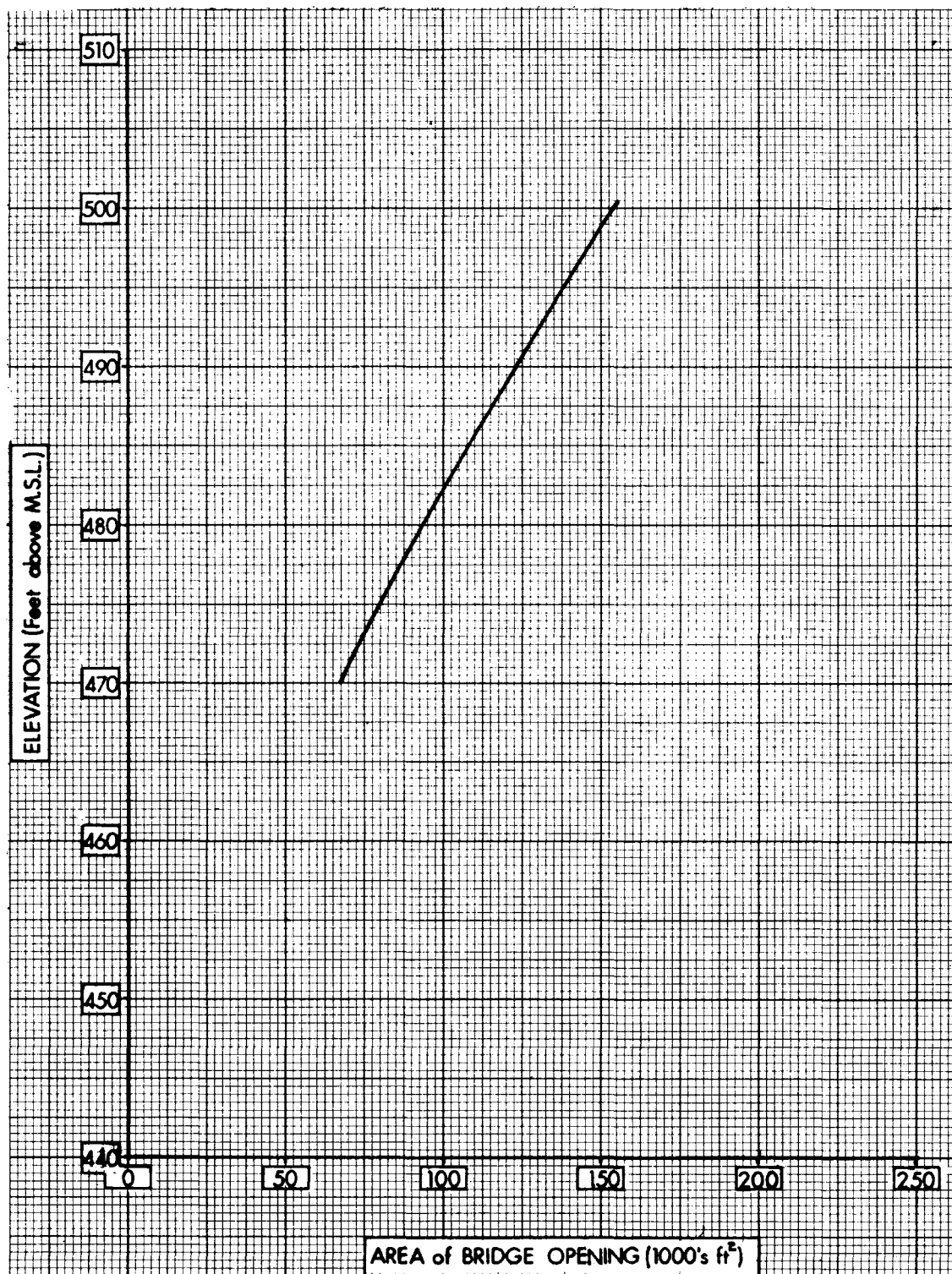


Figure 36. Area of I-275 Bridge opening for a change in river stage.

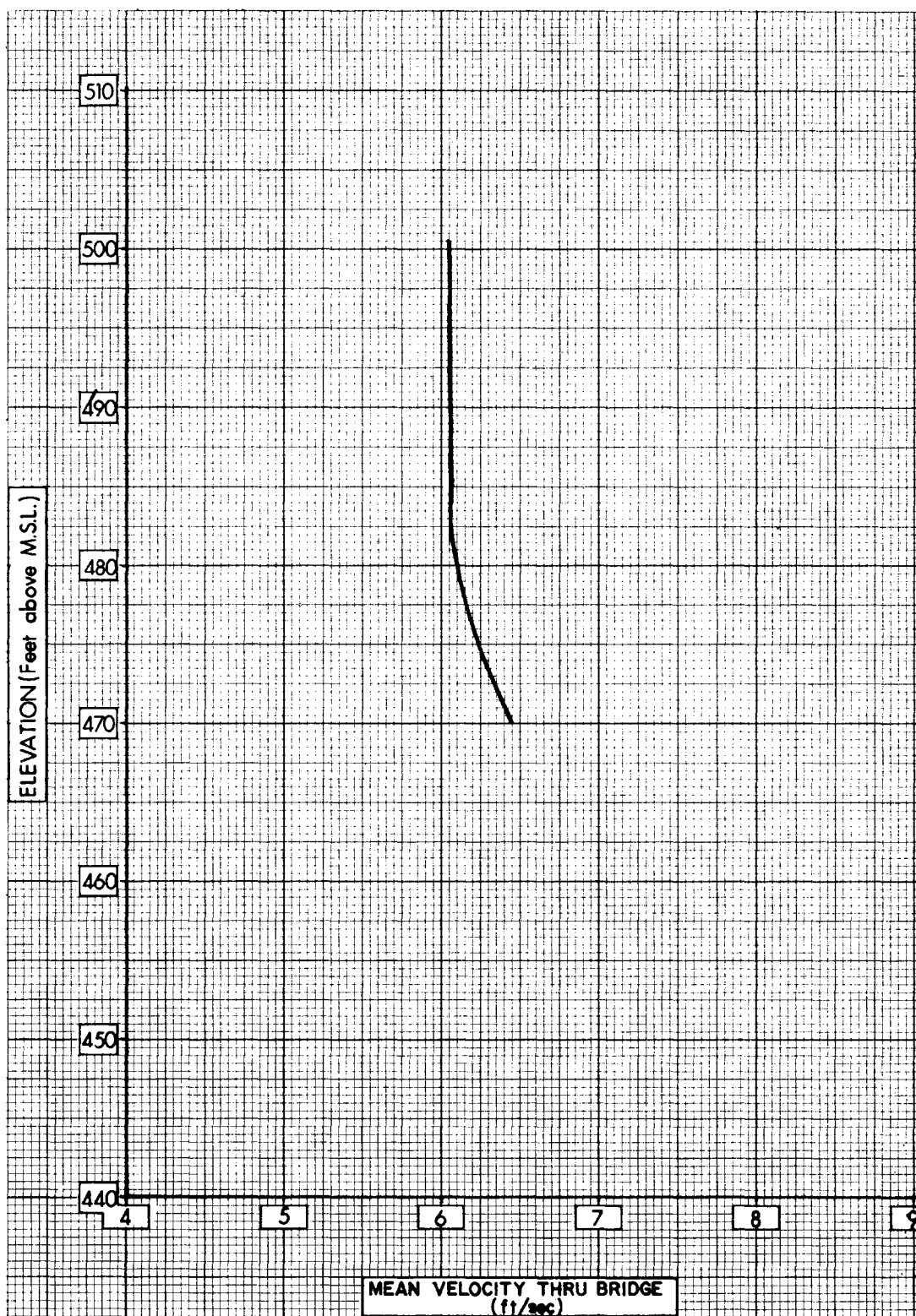


Figure 37. Mean velocity through I-275 Bridge opening for a change in stage.

nary Report of Hydraulic Features of Proposed Highway Bridge on Interstate Highway 275," a report prepared for the Kentucky Department of Highways by the U. S. Geological Survey.

5.5.0 Red River, Louisiana

The Long-Allen (Texas Street) Bridge over the Red River at Shreveport, Louisiana is approximately 3100 feet long and is supported by ten piers, figure 38(a). Pier shape and dimensions of the instrumented pier 6 are shown in figure 38(b).

A U.S.G.S. gaging station (No. 4) is located downstream on the I.C. Railroad Bridge. Information on that gaging station from the U.S.G.S. Water Resources Data for Louisiana is as follows:

Location: Lat. 32°30'55", Long. 93°44'25". Illinois Central Railroad Bridge at mile 302.4.

Drainage Area: 54,677 sq. miles.

Period of Record: Stage, May 1873 to date. Stages are published also by U. S. Weather Bureau. Zero of gage prior to 1926 was ten feet higher than present datum. Discharge, 1872 intermittently to 1905 and Aug. 1929 to date. Computed daily since July 31, 1928. Also in reports of U. S. Geological Survey.

Gage: Automatic recorder with telemark attachment and wire-weight.

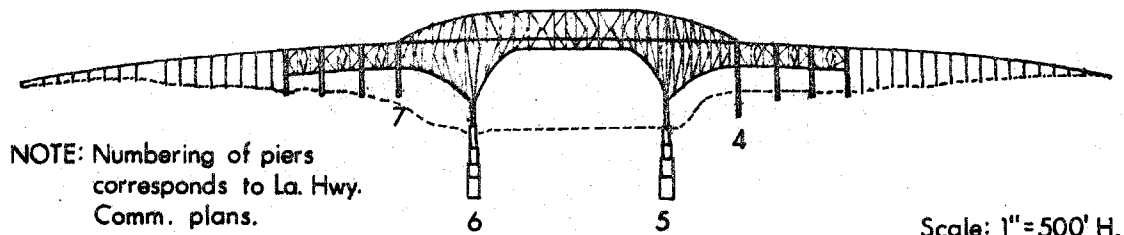
Average Discharge: U. S. Highway 80 Bridge at mile 302.7.

Extremes: Highest, 45.9 feet (present datum) from watermark in Aug. 1849 (affected by huge log raft). Lowest, 0.2 foot on Nov. 8 and 9, 1939. Maximum, 354,000 cfs estimated for June 15, 1908 (stage, 45.1). A discharge of 303,000 cfs was observed on Apr. 5, 1945 (stage, 37.4). Minimum, 690 cfs computed for Oct. 30, 1956 (stage, 2.72).

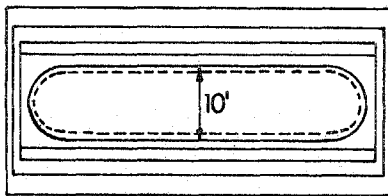
Remarks: Gage maintained by U. S. Weather Bureau. Read by New Orleans district, except on weekends and holidays. Drainage area, 60,613 sq. miles (54,677, surface runoff). Bankfull stage 30 feet. Mean high water stage, 25.0 feet, and mean low water stage, 4.5 feet (1938-62). Flow partly regulated by denison, texarkana, and ferrells bridge dams.

Core borings in the vicinity of the instrumented pier are shown in figure 39. These borings show the river bottom to be fine sand. A grain size analysis of a bottom sample collected from the scour hole gave $d_{50} = 0.1\text{mm}$, figure 40.

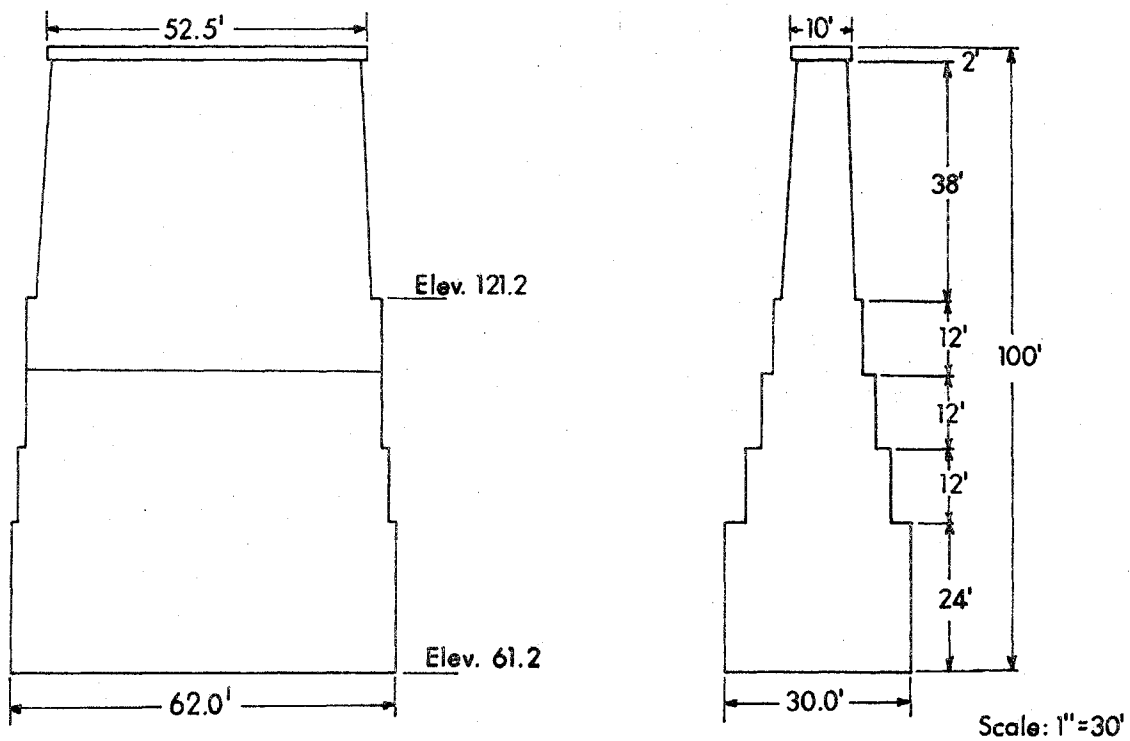
A discharge and discharge intensity rating curve for this site is given in figure 41. The rating curves are reproduced from U.S.G.S.



a) ELEVATION (Looking downstream)



NOTE: Elevations shown are M.S.L.
Subtract 131.48' to obtain
gage height elevations.



b) INSTRUMENTED PIER #6 DETAILS

Figure 38. (a) Texas Street Bridge over the Red River,
(b) Instrumented Pier 6,

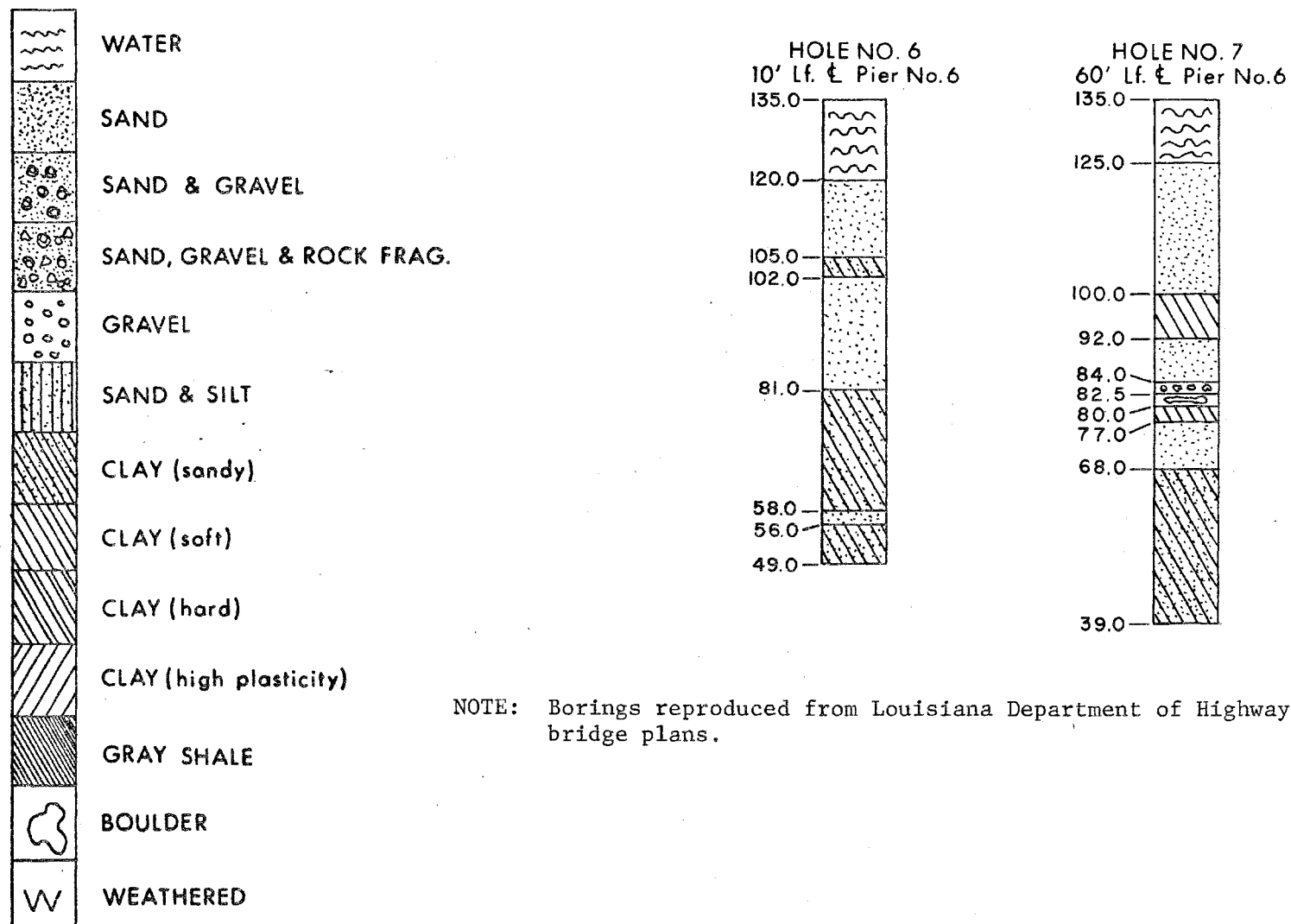
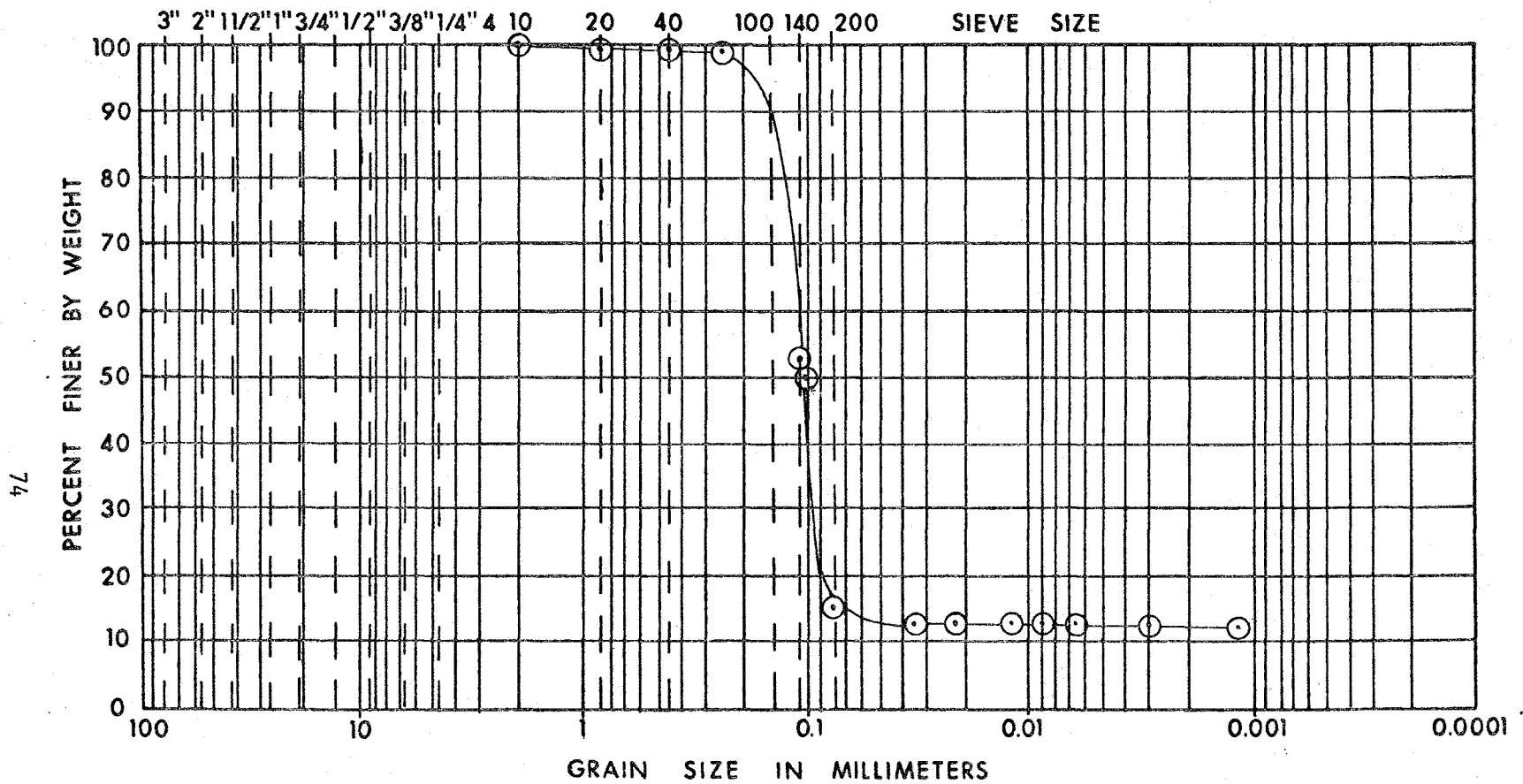


Figure 39. Core borings taken in the vicinity of instrumented pier 6, Red River.



AASHO	GRAVEL			SAND		SILT	CLAY	COLLOIDAL CLAY		
	CRS.	MED.	FINE	COARSE	FINE					
PROJECT		DATE		SAMPLE		DEPTH			DESCRIPTION	
RED		8/1/72		Scour Hole		Elev. 118.0			Grab Sample @ Bow of Pier No.6	

Figure 40. Grain size analysis curve, Red River.

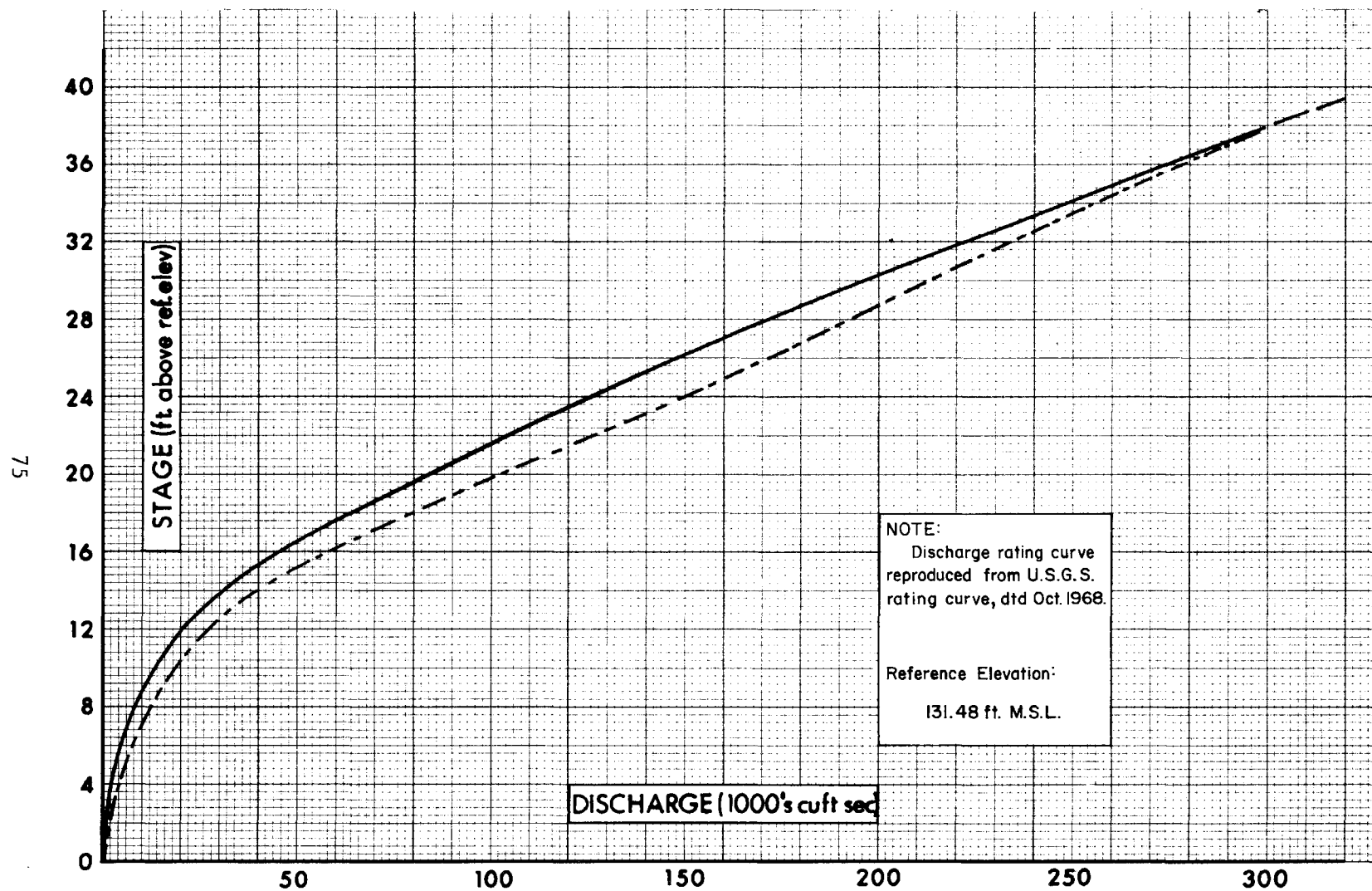


Figure 41. Discharge Rating Curve for the Red River at Shreveport, Louisiana.

ratings, dated September, 1968. Figure 42 is a flood frequency curve, computed from daily mean values of discharge for the period 1929-71.

Figure 43 shows a flood history of the Red River for the period Dec. 16-30, 1972. Figure 44 illustrates the very rapidly changing scour hole with respect to time. It can be seen from Figure 44 that the scour hole changed five feet in depth during a time period of less than one hour.

Weekly measurements of discharge for the Red River at Shreveport are made by the Army Corp. of Engineers. Those measurements include depth and velocity readings which in turn yield area of the bridge opening and discharge. From data supplied by the Corp. of Engineers and the U.S.G.S. for January 1971 through June 1973, data points for 130 days were plotted. They are area of the bridge opening, figure 45 and maximum velocity through the bridge opening, figure 46.

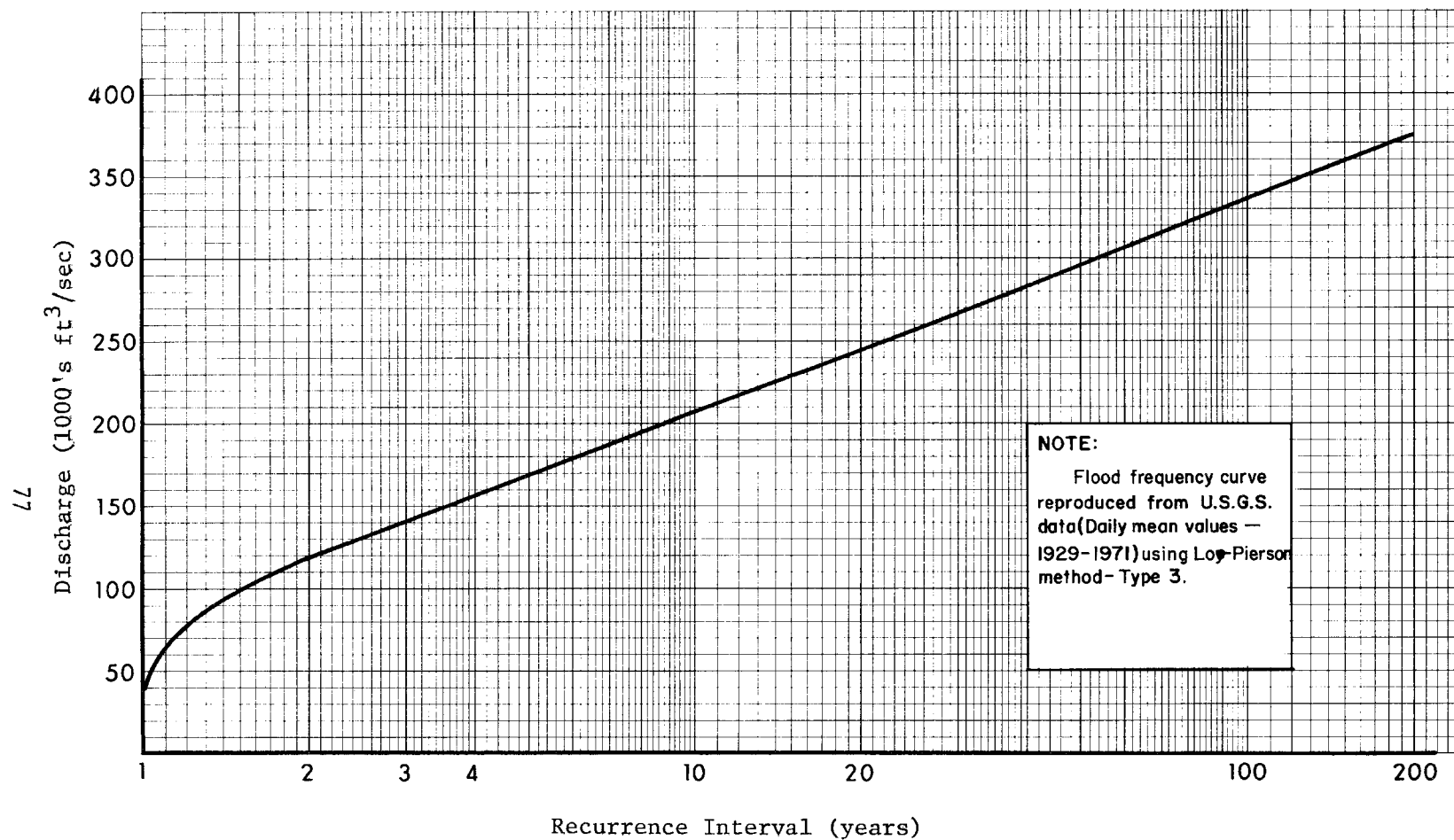


Figure 42. Flood Frequency Curve for the Red River at Shreveport, Louisiana.

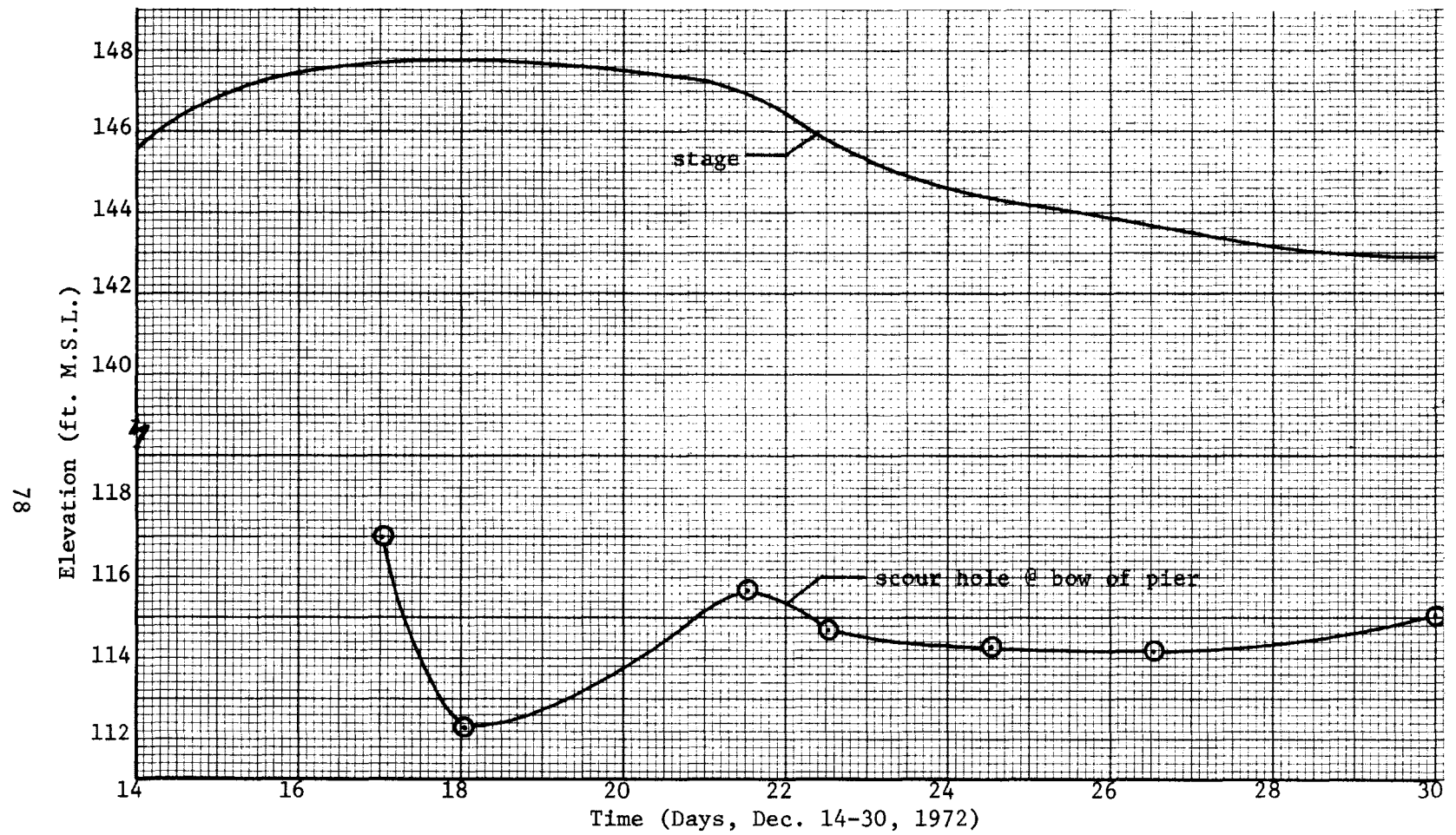


Figure 43. Flood History of Red River for Dec. 16-30, 1972

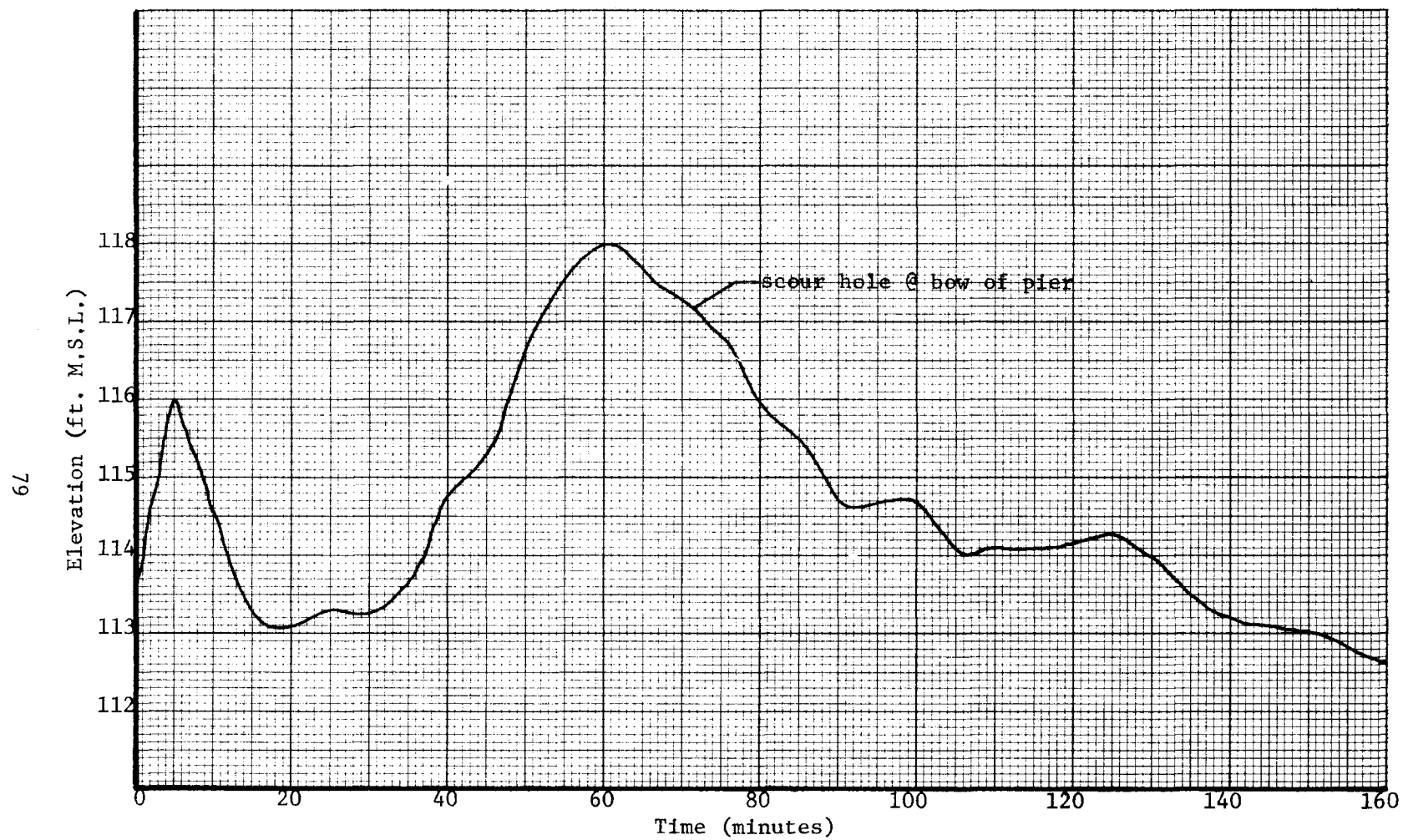


Figure 44. Change in scour hole with time, Red River.

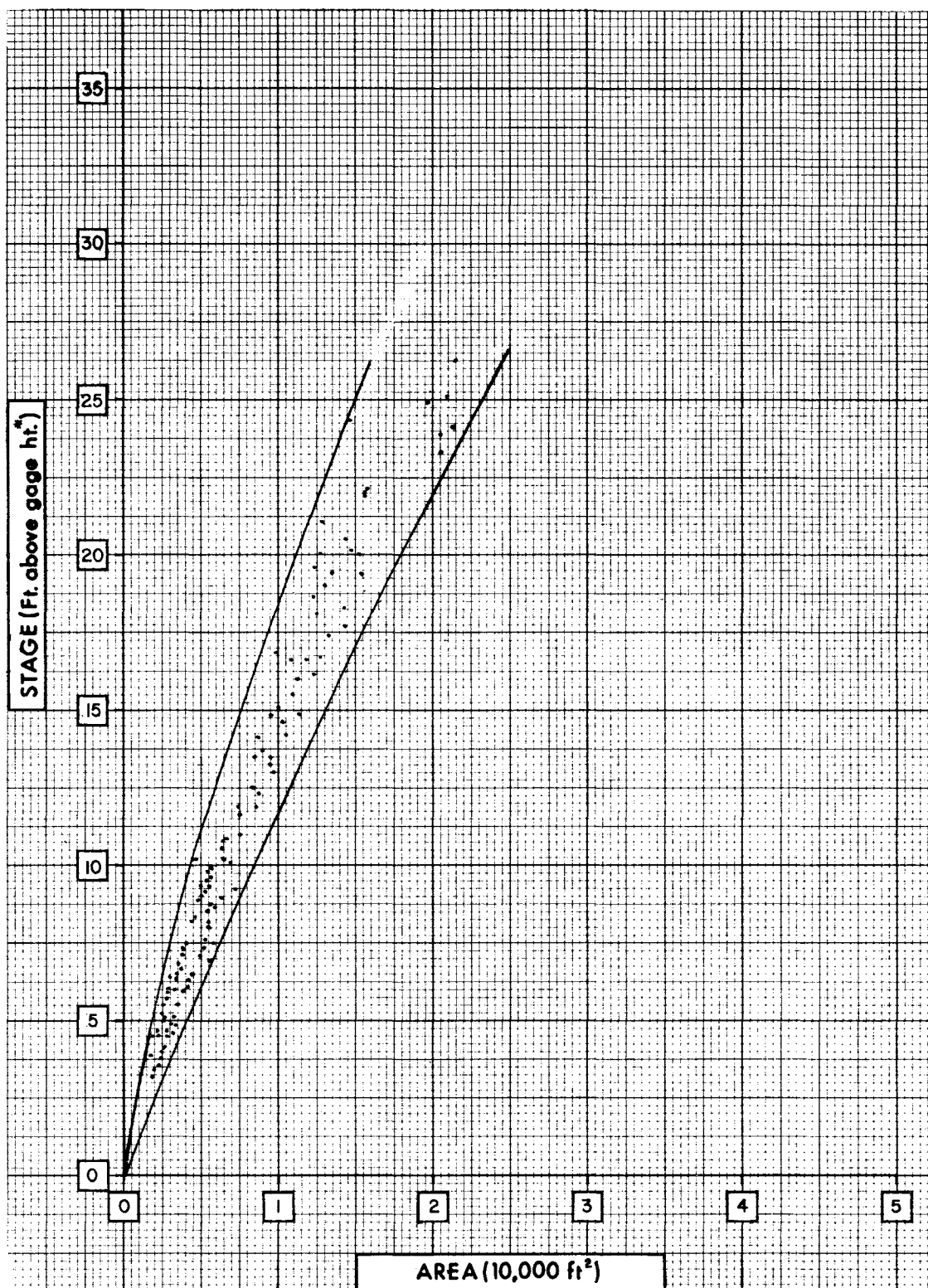


Figure 45. Area of bridge opening, Red River.

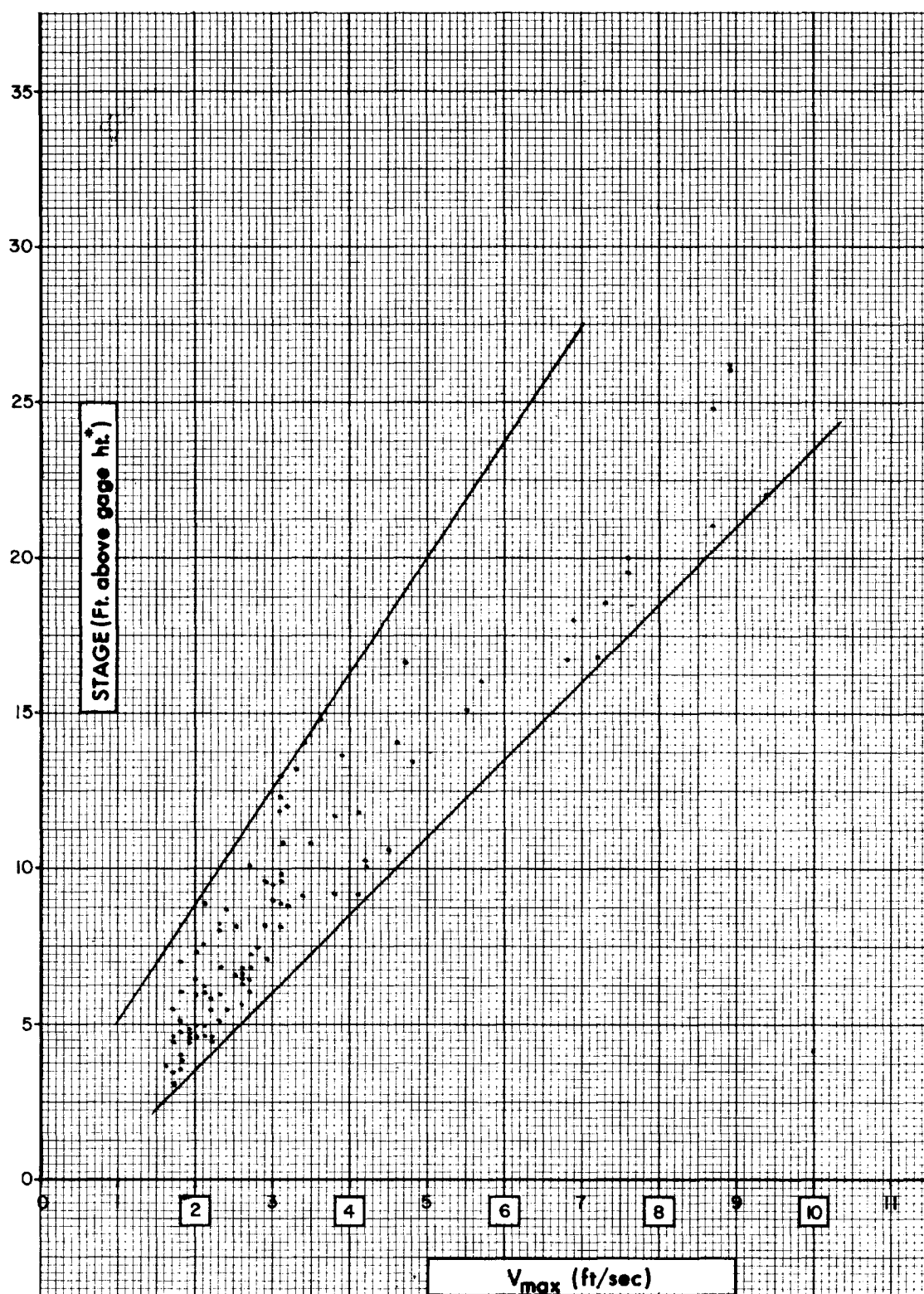


Figure 46. Maximum velocity through bridge opening, Red River.

VI. ANALYSIS

6.1.0 Data Analysis

Since the purpose of this study is to evaluate the formulas that predict the depth of scour around bridge piers, a computer program was coded to evaluate these formulas based on measured field data. Using field data as input to the program, calculations are made, and scour depths are computed using the twenty formulas listed in table 8. The printout for the program gives the field data input, which includes the measured scour depths and the depths predicted by each formula. From this output, the accuracy of each of the formulas in predicting scour depths can be examined.

<u>Name of the Formula</u>	<u>Equation Number</u>
Laursen	2-35
Ahmad	2-36
Chitale	2-37
Breusers	2-38
Inglis-Poona (2)	2-41
Inglis-Poona (1)	2-42
Inglis-Lacey	2-43
Blench (1)	2-44
Blench (3)	2-45
Blench (2)	2-46
Varzeliotis	2-47
Bata	2-49
Knezevic	2-50
Liu	2-52
Larras	2-53
Maza & Sanches	2-54
Carstens	2-55
Shen (3)	2-56
Shen (1)	2-57
Shen (2)	2-58

Table 8. List of the prediction formulas considered in this study.

Comparisons of the scour prediction formulas using data from the Red River and the Brazos River (flood condition), are shown in tables 9 and 10, respectively. As can be seen from these tables, some of the formulas predict the scour depth accurately for both sites. An example is the Inglis-Poona (2) formula. Some of the formulas predict the scour depth closely for one site but not for the other one. The Shen (1) formula is an example. Many of the formulas predict scour depths that are in more than 100% error. A better perspective is gained by comparing the predicted scour depths, as shown on the comparison curves in section 2.3.0 of this report, to the measured data from the field study. Figures 47, 48 and 49 show this comparison for the Red River and the Brazos River data.

FIELD DATA:

width of the pier	= 12.00
undisturbed flow velocity	= 2.00
flow depth approaching	= 11.80
discharge	= 0.900E 04
ave. discharge intensity	= 12.50
sediment number	= 11.04
channel Froude number	= 0.08
pier Froude number	= 0.10
pier Reynolds number	= 0.240E 07
scour depth measured from mean-bed elevation	= 10.00
scour depth measured from water surface	= 21.00

SCOUR DEPTH PREDICTED FROM MEAN-BED ELEVATION:

<u>Investigator</u>	<u>Description</u>	<u>Scour Depth</u>
Bata		0.69
Breusers	Equilibrium	16.80
Carstens	Equilibrium	6.71
Knezevic	Maximum	1.29
Larras	Max. Equilib.	9.16
Laurson	Equilibrium	16.12
Liu	Maximum	7.74
Maza-Sanches	Not Applicable	
Shen-1	Equilibrium	6.50
Shen-2	Equilibrium	1.37
Shen-3	Equilibrium	8.82

SCOUR DEPTH PREDICTED FROM WATER SURFACE:

Ahmad	Maximum	10.77
Blench-1		13.45
Blench-2		21.33
Blench-3		11.49
Chitale	Maximum	11.44
Inglis-Poona-1	Maximum	10.92
Inglis-Poona-2	Maximum	20.49
Inglis-Lacey		23.92
Varzeliotis	Maximum	9.64

Table 9. Scour depths as predicted by various formulas for the Red River.

FIELD DATA:

width of the pier	= 12.50
undisturbed flow velocity	= 3.60
flow depth approaching	= 22.00
discharge	= 0.170E 05
ave. discharge intensity	= 56.25
sediment number	= 10.41
channel Froude number	= 0.08
pier Froude number	= 0.14
pier Reynolds number	= 0.788E 07
scour depth measured from mean-bed elevation	= 12.00
scour depth measured from water surface	= 33.00

SCOUR DEPTH PREDICTED FROM MEAN-BED ELEVATION:

<u>Investigator</u>	<u>Description</u>	<u>Scour Depth</u>
Bata		1.22
Breusers	Equilibrium	30.65
Carstens	Equilibrium	7.01
Knezevic	Maximum	5.68
Larras	Max. Equilib.	18.68
Laursen	Equilibrium	31.10
Liu	Maximum	18.67
Maza-Sanches	Equilibrium	0.30
Shen-1	Equilibrium	13.56
Shen-2	Equilibrium	4.43
Shen-3	Equilibrium	19.52

SCOUR DEPTH PREDICTED FROM WATER SURFACE:

Ahmad	Maximum	29.36
Blench-1		30.42
Blench-2		39.55
Blench-3		27.90
Chitale	Maximum	21.09
Inglis-Poona-1	Maximum	24.09
Inglis-Poona-2	Maximum	33.61
Inglis-Lacey		26.34
Varzeliotis	Maximum	20.07

Table 10. Scour depths as predicted by various formulas for the Brazos River.

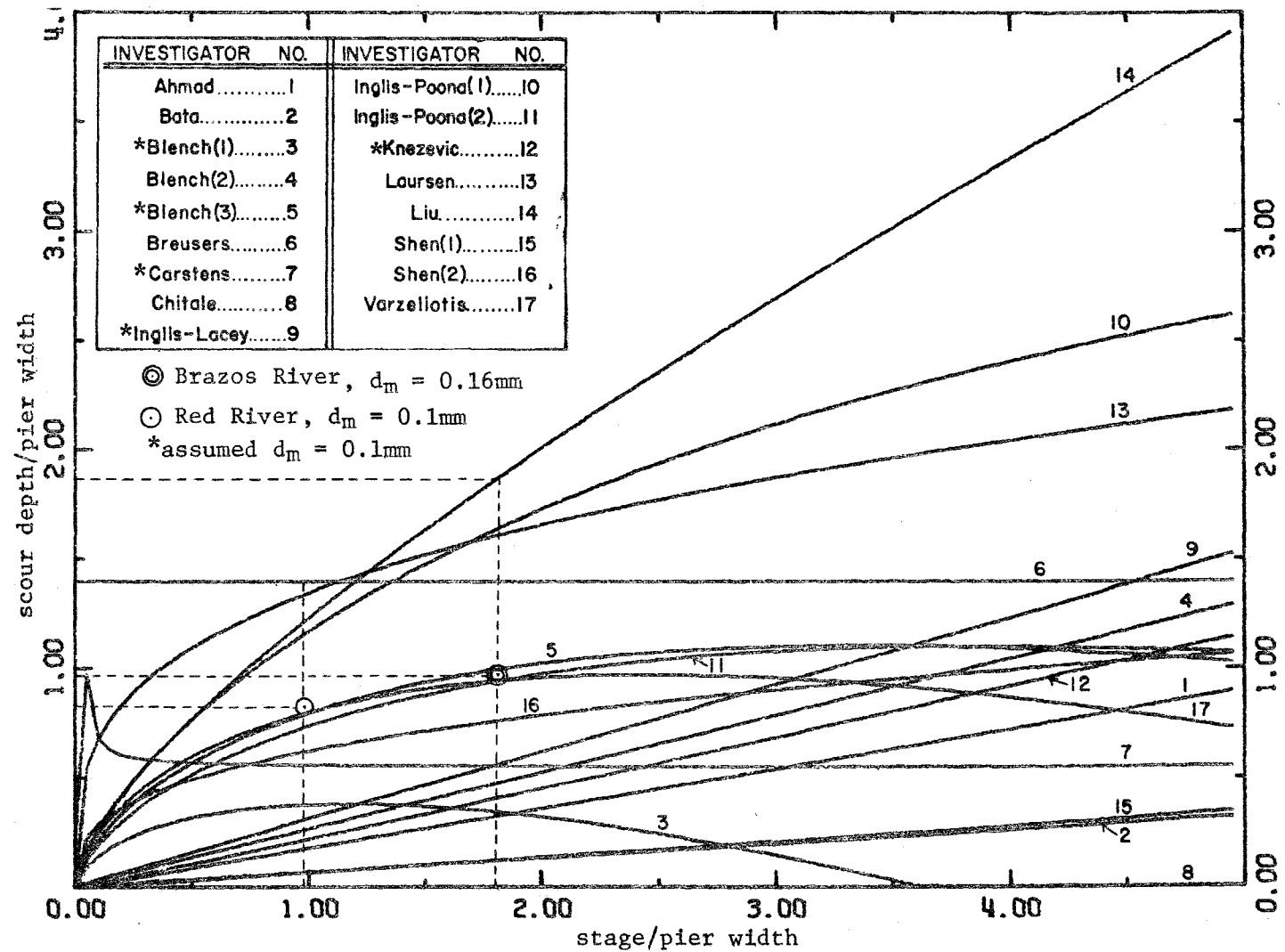


Figure 47. Comparison of the predicted depth of scour with the measured scour depth at Red River and Brazos River for Froude No. = 0.08.

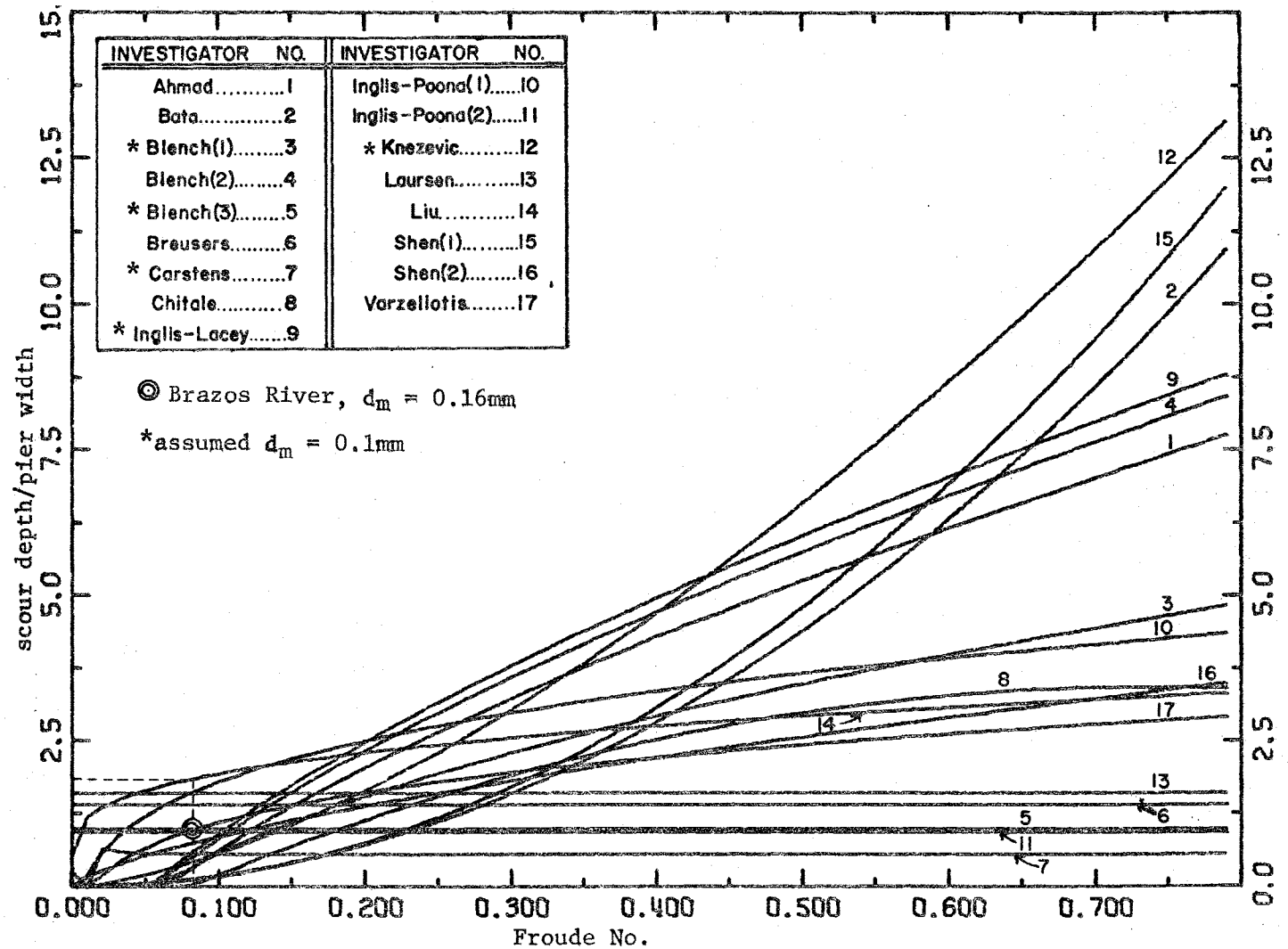


Figure 48. Comparison of the predicted depth of scour with the measured scour depth at Brazos River for stage/pier width = 1.75.

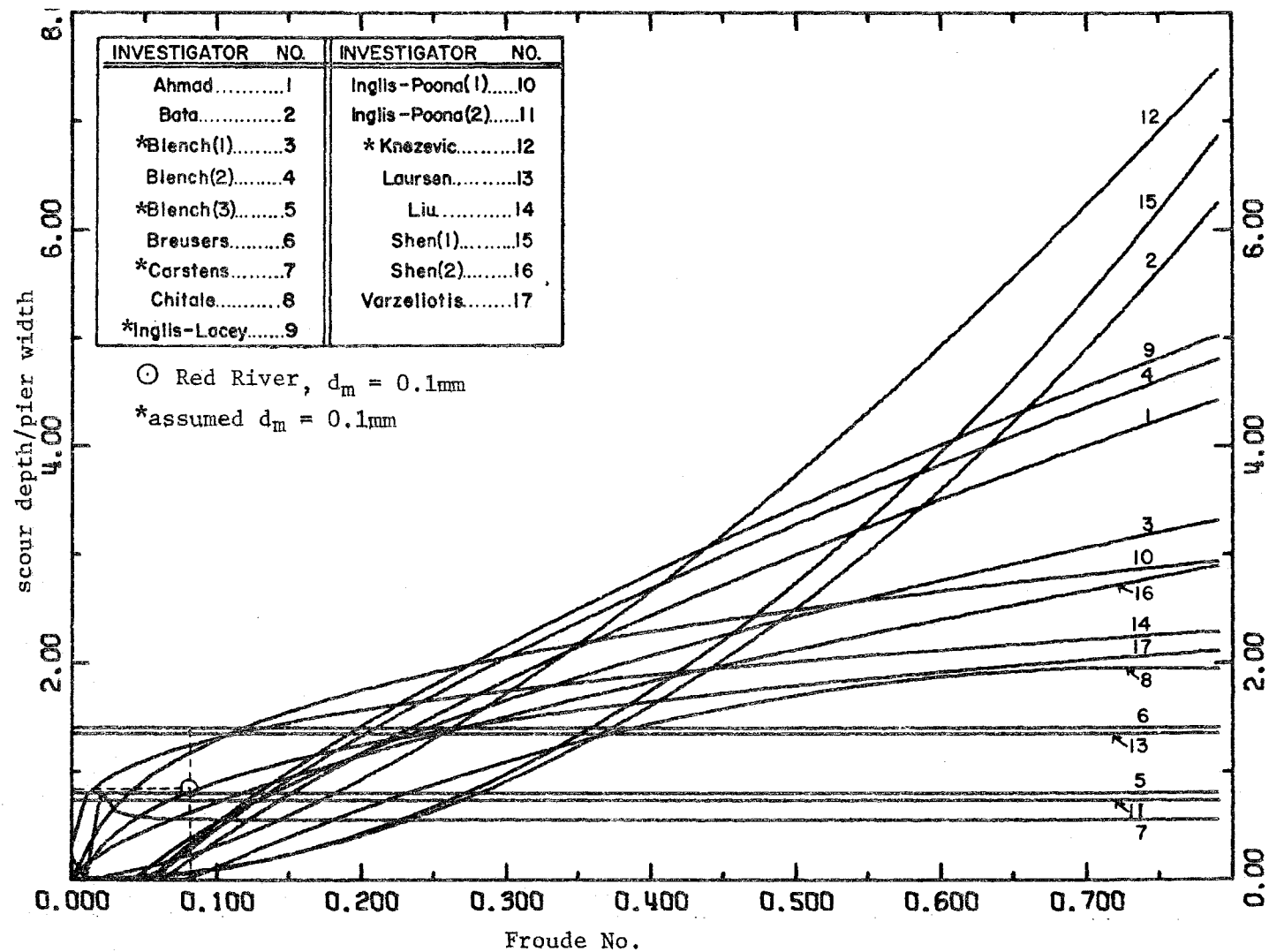


Figure 49. Comparison of the predicted depth of scour with the measured scour depth at Red River, for stage/pier width = 1,

VII. CONCLUSIONS AND RECOMMENDATIONS

7.1.0 Conclusions

Basically, all scour prediction formulas are empirical and as such are limited. This limitation is due to the large amount of time and money needed to collect sufficient data on flow, scour depth, material properties, etc. Consequently, available formulas are "fit" to select sets of data. While there is little doubt that each scour prediction formula describes the experiment or field situations on which it was based, difficulties arise in trying to describe scour for situations beyond the range of the original experiment. This fact is graphically illustrated in figures 3 through 10.

Past investigations have done much to illuminate the scour problem. However, more data are needed before a reliable scour prediction formula is available to the bridge engineer.

Certain conclusions may be drawn concerning scour around bridge piers. These are:

- 1) Laboratory experiments have been beneficial in illustrating scour around piers in a controlled situation. But due to the lack of scaling factors for sediment transport, laboratory experiments are insufficient to quantitatively describe scour in the field.
- 2) Scour is a dynamic process. This has been shown in the laboratory as well as in the field. Results of this study, illustrated in figures 26 and 43 for the Brazos and Red Rivers, respectively, show that scour depth around bridge piers increases and decreases with the stage. Indications are that maximum scour depth occurs approximately at maximum stage. This fact makes scour measurements difficult and has resulted in erroneous conclusions when scour depth was measured after floods. Consequently, it is concluded that accurate data on maximum scour depth must be collected during peak flow conditions.
- 3) Scour at a bridge pier is the sum of three separate scouring actions. These are:
 - a) the general scour occurring from the natural changes in the stream
 - b) scour resulting from the contraction caused by the narrowing of the waterway at the bridge site
 - c) local scour that results from flow disturbances introduced by the presence of the bridge pier

- 4) State-of-the-art in the hydraulics of sediment transport is sufficiently advanced so that an analytical treatment of contraction scour seems plausible. Local scour, on the other hand, which results from secondary flows generated by piers, abutments or other obstructions is probably not amenable to direct analysis. At this time experimental investigation of local scour seems to be the best approach.
- 5) The most significant contribution to total scour at a bridge pier comes from general or contraction scour. It is concluded that local scour, by itself, typically would not be of sufficient magnitude to explain bridge failures.
- 6) Field data are needed for two specific purposes:
 - a) First, to delineate those efforts that are most successful in describing scour in the field situation and, thus, serve as a basis for directing future research efforts. In other words, we need to determine which of the past efforts have been fruitful before investing more time and money in a new effort. Because of the transient nature of scour these data must be gathered during floods to determine the maximum effect of scour. This, of course, is difficult because of the instrumentation damage due to floods.
 - b) Additionally, field data are needed to form a more complete basis for modeling sediment transport in laboratory experimentation of scour around bridge piers. These field data would not necessarily need to be taken during floods.
- 7) The type of instrumentation used for taking scour measurements are either mobile or stationary. Each type has some advantages and some disadvantages.

The stationary type of scour measuring instrumentation, such as used in this study, has the ability to more precisely measure scour depth, and can be made to operate remotely and automatically. It is fixed at one location and, therefore, is limited to the amount of data that may be collected.

A mobile instrumentation system, such as that used by Sanden in Canada, overcomes many of the limitations of a fixed system by being able to move from site to site. Data from more sites may be collected in this manner but the system can not be operated remotely nor automatically. The correlation of measured data to locations relative to the stream or bridge pier is difficult. However, mobile instrumentation has been used successfully by the U.S.G.S. during floods.

Both types of instrumentation systems are subject to flood damage. However, because of the installation costs, damage to a "fixed" type of system is usually more expensive.

7.2.0 Recommendations

Based on the experience gained in this study, the following recommendations are made:

- 1) Field studies should be continued to collect data on maximum scour depth until sufficient flood data are collected to correlate maximum scour depth with the independent variables and qualify the existing prediction formulas to determine their limits and ascertain their accuracy in predicting maximum scour depth.

Selected sites currently used in the Scour Around Bridge Piers study should be maintained in operational status for this purpose. Since the major expense of these scour monitoring sites is in the development and installation, maintenance costs by comparison are small. Consequently, the most efficient and inexpensive means to collect data on scour around bridge piers during floods will be to keep these systems operational for an extended period of time. Support from local state highway departments would greatly reduce the costs of this effort.

During this study valuable field data was lost as a result of system malfunctions that occurred between maintenance. This problem could be eliminated with more frequent maintenance trips to each field site. However, a more reasonable way is to use local state highway department personnel to assist in this maintenance responsibility.

- 2) Additional field studies should be initiated to gather larger quantities of data on scour depth and related variables. These data are needed to support efforts to:
 - a) develop reliable modeling parameters between natural sediment transporting streams and laboratory experiments,
 - b) understand the relative magnitude of natural, contraction and local scour,
 - c) establish a more definite relationship between scour and its independent variables.

Most of these data do not necessarily need to be taken during major floods and, therefore, the instrumentation requirements are reduced.

Data collected on sediment transport and stream behavior by agencies such as the U.S.G.S. and the U. S. Army Corp. of Engineers could be useful. It is suggested that a study be initiated to determine if these data can be developed to aid the study of scour around bridge piers.

- 3) Consideration should be given to developing new, inexpensive instrumentation systems that can be used to more efficiently gather field data on scour parameters. These systems could be either fixed or mobile. However, the objective should be to furnish an inexpensive instrument that is reliable, easy to operate and could be used by state highway departments in collecting data on scour.

An inexpensive fixed type instrumentation system could be developed so that only the depth and stage measuring transducers would have to remain fixed at the site. The peripheral instrumentation, such as recorders, amplifiers, etc. could be made portable. Thus, cost of the fixed portion of the instrumentation system could be held to a minimum. Such a system would offer the advantage of being able to monitor a number of important sites within an area with only one recorder and amplifier system.

A second, and more attractive, approach would be to develop a mobile scour monitoring system. It is visualized that such a system would be self-contained and transportable by truck. This approach offers considerable advantage over a fixed site in that a larger number of stream sites could be covered. The instrumentation could be taken to a flood, rather than waiting for a flood to occur at a chosen stream. The probability of gathering useful data is, therefore, increased.

Several schemes could be used for implementing the measuring transducers. Some of these are:

- a) A guide rail could be placed on a number of bridges such that the transducer could be powered into and out of the water on this attachment.
- b) The transducers could be lowered into the water by a cable suspended from a boom truck, such as those used by the U.S.G.S.
- c) The transducers could be introduced into the stream from a powered maneuverable, floating platform that could be remotely controlled from either a bridge or stream bank.

It is recommended that a cost and effectiveness analysis be conducted on these alternative scour monitoring systems.

- 4) It is recommended that an effort be initiated to develop a computer simulation of contraction scour. The state-of-the-art is sufficiently advanced in sediment transport to where such an effort is feasible. Such a computer program would aid the bridge engineer in predicting the magnitude of contraction scour

that would occur at a bridge site, and allow him to test alternate design approaches.

- 5) A laboratory study should be initiated to investigate the local hydraulics associated with obstructions in a stream. The mechanism for local scour is intimately related to the secondary flows generated by obstructions such as abutments and piers. This phenomenon is not clearly understood and should be pursued with vigor. Much can be learned from such experiments without a complete modeling of the sediment transport.

It is felt that the limiting magnitude of local scour can be defined in this manner and can be included as an added scale factor to contraction scour. The result would be a very useful tool to the bridge engineer. It would allow him to inexpensively and expeditiously simulate various waterway designs prior to the site construction.

- 6) It is recommended that one of the Scour Around Bridge Piers sites be selected for model studies. Much could be learned concerning the relationship between prototype and model for future studies if model studies were conducted simultaneously with ongoing field studies.

APPENDIX A. SCOUR MONITORING INSTRUMENTATION SYSTEM

A.1.0 Present Instrumentation System

The Scour Monitoring Instrumentation System in its most present stage of development has evolved through many design stages. Due to installation difficulties and due to extreme river conditions during flooding, the mounting and protecting of the transducer have undergone the greatest change. That part of the system used to record the data has remained the same due to the little change in its present state-of-the-art and high original cost. The power supply and controls for the monitoring system have necessarily become more sophisticated along with the supplementary field equipment.

The basic monitoring system includes both depth-stage measuring and velocity measuring instrumentation, figure 50. The depth-stage system is completely automated and requires minimal servicing. The velocity system must be manually operated. Transducers for the complete system are mounted on the bridge pier as near to the river bottom as possible. It is believed that a mounting location in the scour hole provides the maximum transducer protection from floating debris.

The controls and recorders for both systems are located in a steel housing, for their protection and for easy access by operator personnel. The instrumentation housing is located either on the cap of the instrumented pier, on the bridge deck above the instrumented pier, or on the abutment nearest the instrumented pier. Cables provide the connection between the pier-mounted transducers and the remote control-recorder system.

A.1.1 Depth-Stage System

Typically, there are depth transducers at three locations on the bow of the pier to determine the shape and maximum depth of the scour hole, figure 51. One is located on the center line of the pier. The other two are located on either side of the pier at a distance of two to six feet behind the bow. The transducer supports have been designed in an effort to make them sufficiently strong without creating additional flow disturbances.

The stage transducer is located on the rear of the pier where it is more easily protected from debris. In addition, the relatively calm water surface at the rear of the pier provides the optimum reflecting surface for the echo sounding stage transducer. When located on the rear of the pier comparative data recordings have shown the stage transducer to be unaffected by extreme river conditions which presents high velocities with large amounts of debris.

A level indicator switch is located near the rear of the pier, also for protection. The closing of the switch by flood stage waters causes more frequent recordings of the depth-stage data.

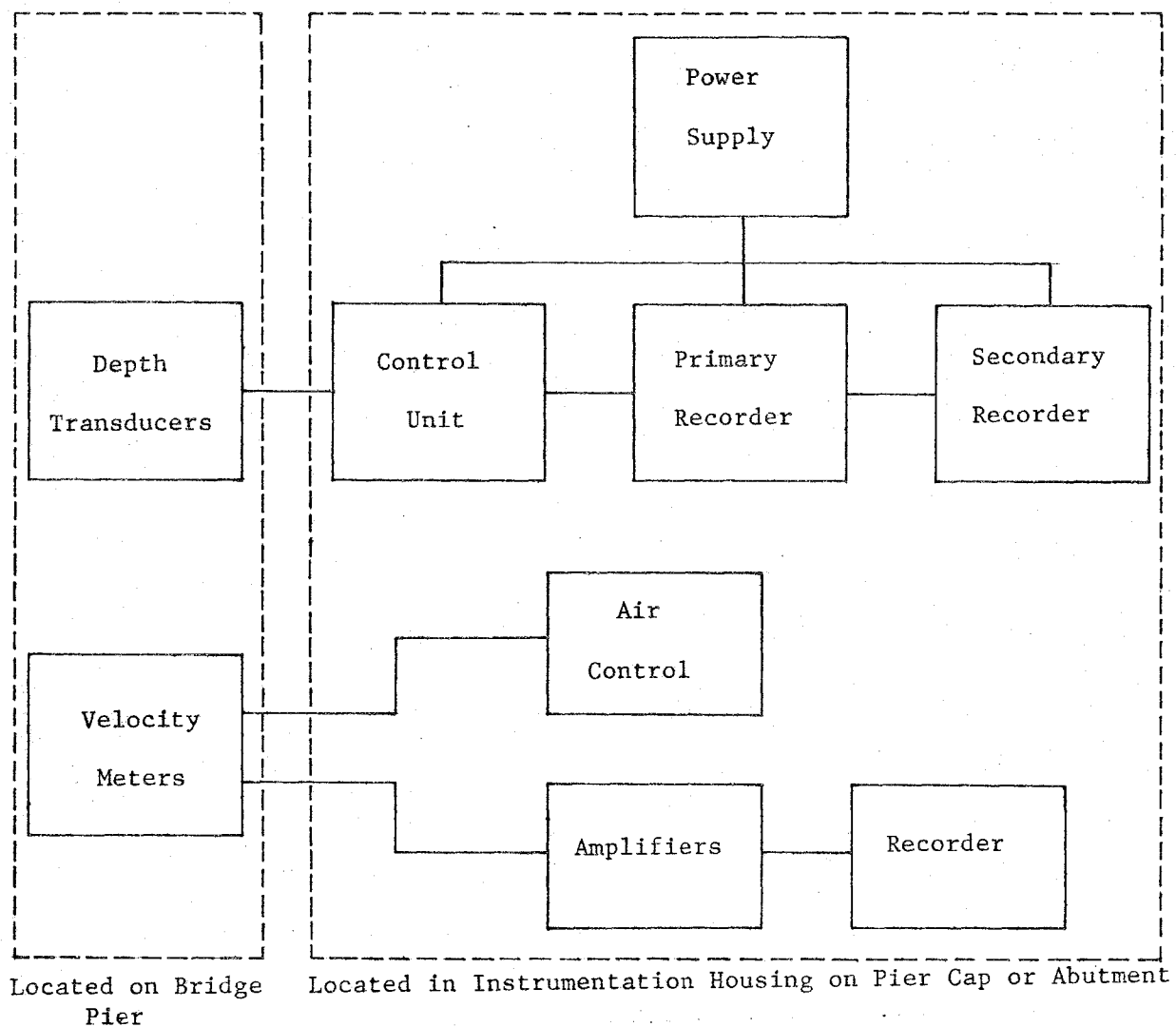


Figure 50. General Schematic of "Scour Around Bridge Piers" Instrumentation System.

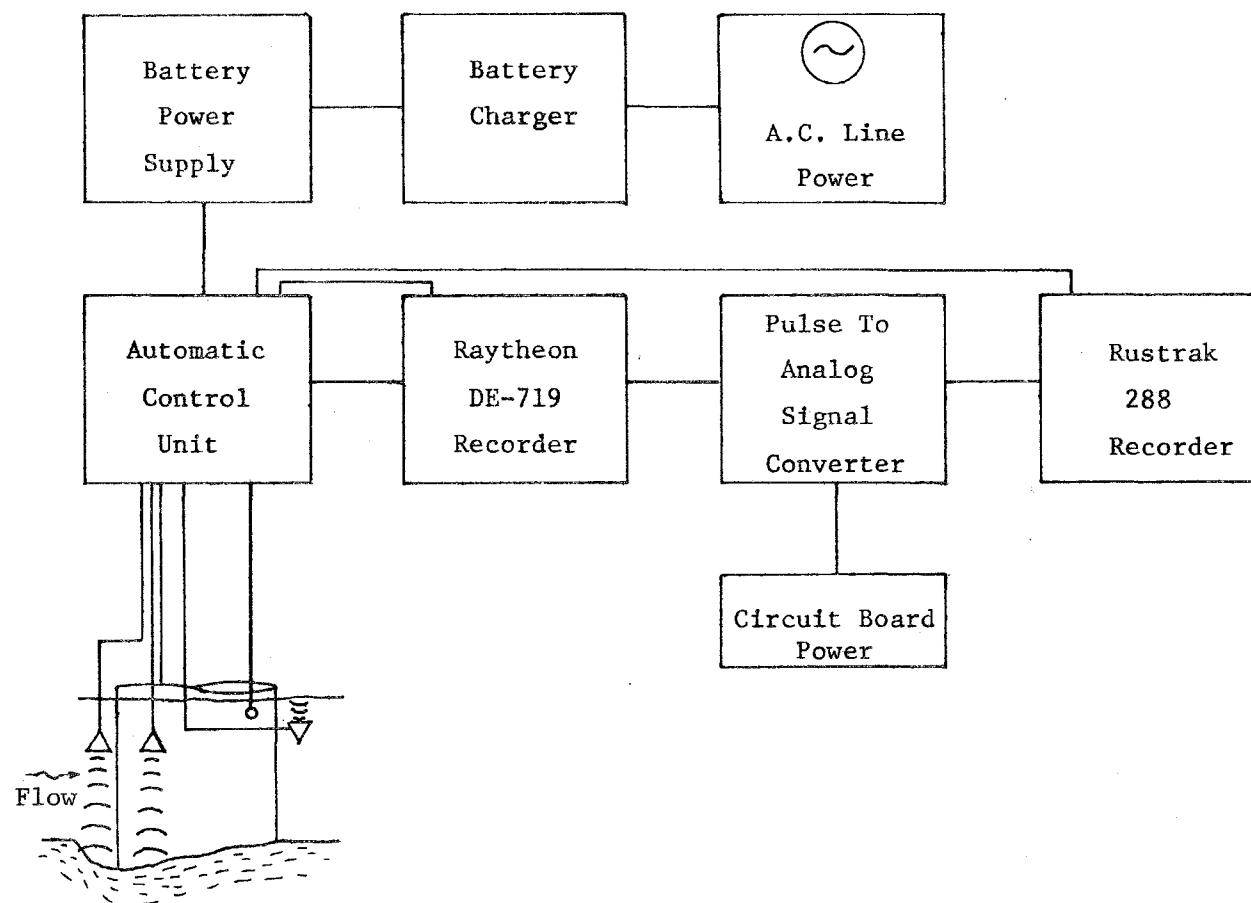


Figure 51. Scour-depth and Stage Instrumentation System.

Data from the stage and depth transducers is transmitted to an Automatic Control Unit which allows the alternate recording of the data from each transducer. Recordings are simultaneously produced on two recorders. Data transmission is first received by an echo-sounding transmitter/receiver recorder. Here the data signal is recorded, converted to an analog output and delivered to a back-up recorder.

The battery power supply which operates the entire depth-stage system is distributed to the system through the control unit. An a.c. line voltage operates a charger to prolong the life of the battery power supply.

A.1.2 Velocity System

Similar to the depth transducers, the velocity meters are also located at three locations on the pier, figure 52. Present designs allow mounting of the velocity meters on the supports with the depth transducers. However, individual meters may be mounted at any location.

Signals from the two-dimensional measuring, velocity meters are transmitted to two bridge amplifiers and recorded on a two-channel recorder. Each of the two recordings represents a component of a velocity vector. The air supply and air control unit are used to operate the velocity meter's protective cover.

Cables from both the depth and velocity transducers are mated and brought up the pier to the instrumentation housing. Protection of the cables from debris may be accomplished by bringing them up the rear of the pier or by bringing them up the bow of the pier and covering them with a durable steel or aluminum conduit.

The following sections will describe the improvements and design changes of each part of the scour monitoring system. From the original concepts of a basic system to the present system, the necessity of each development will be discussed.

A.2.0 Depth Instrumentation Development

The basic scour-depth measuring instrumentation has remained essentially the same during the length of the project. Depth transducers using an echo-sounding principle have been utilized with the only change being different methods of transducer mounting.

There are many brands of depth sounding instrumentation and most of them have generally the same specifications. The one used with this system has an operating frequency of 200 khz. The high operating frequency permits optimum transmission in highly concentrated sediment conditions and allows a narrow beam transducer to be used. With the mounting of the transducers to a pier, a narrow sounding beam was desired to prevent echo reflections from the pier. The sounding recorder chosen for this project could also be adapted for remote recordings.

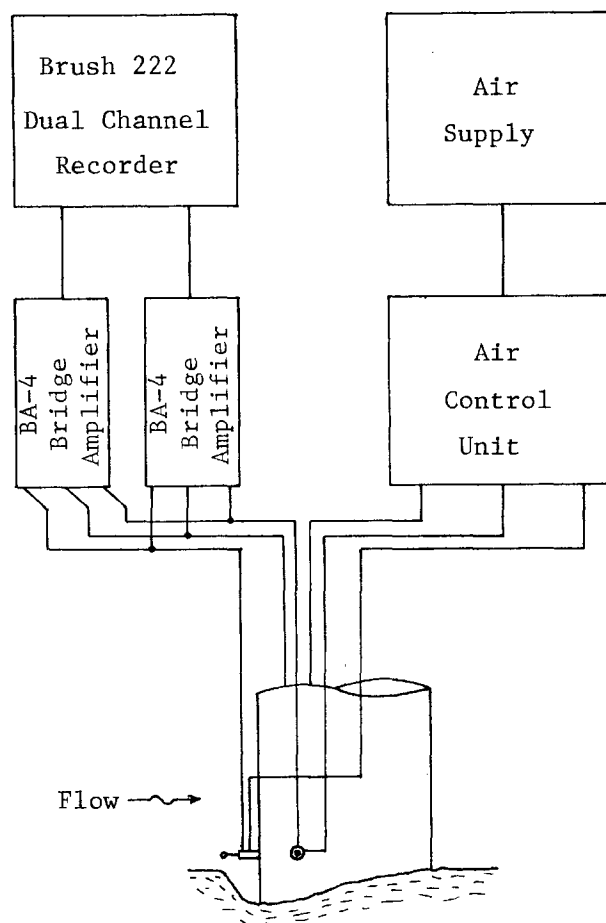


Figure 52. Velocity Instrumentation System

This would permit back-up recording of the data in the event a malfunction were to occur with the basic recording system, i.e. a broken or over-used writing stylus. Since the instrumentation would be operating automatically for a period of four to six weeks, it was realized that it was very possible for a writing stylus to become unusable.

A.2.1 Principle of Operation

The principle of operation 'echo depth sounding' is a method of measuring water depth by computing the time interval required for sound waves to travel, at a known velocity, from a known point to a reflecting surface and return. If the time is measured between the transmission of a sound and the reception of its echo, the distance may be computed by multiplying one-half of this time interval by the velocity of sound in water.

From fresh water at the freezing point to the warmest water of the highest saline content likely to be encountered, there can be a variation

in the Speed of Sound from approximately 4550 to 5050 feet per second; the speed increasing with both higher temperature and salinity. The division of change is roughly three fifths for temperature and two fifths for salinity. The Speed of Sound also increases slightly with increasing pressure; however, for water depths measured with this equipment, the correction for pressure would be negligible, therefore, will not be considered.

The foregoing shows that under widely differing water conditions the sounding accuracy could be affected as much as plus or minus 5% due to the Speed of Sound in water variation. Under normal conditions of operation in a given area, variations of the Speed of Sound in water will usually be less than plus or minus 0.5%.

A.2.2 Transducers

The transducer most used for mounting to a pier was the narrow beam type, figure 53. It is a two-way energy conversion device which converts electrical energy pulses into supersonic energy pulses during transmission and then converts the echos of supersonic energy pulses into electrical signals during reception. Because of a larger radiating area and greater directivity index the transmission response, figure 54 of the narrow beam transducer shows an improvement in echo strength over the standard wide beam transducer. The U. S. Coast and Geodetic Survey has used the narrow beam transducer for measuring wave heights, where the narrow beam width of the transducer can differentiate between the peak and the trough of the waves.

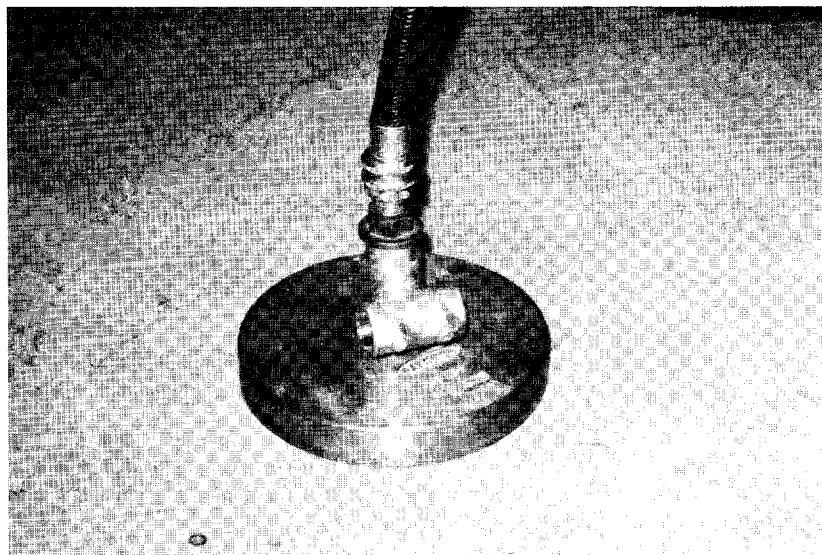


Figure 53. Narrow beam type depth transducer.

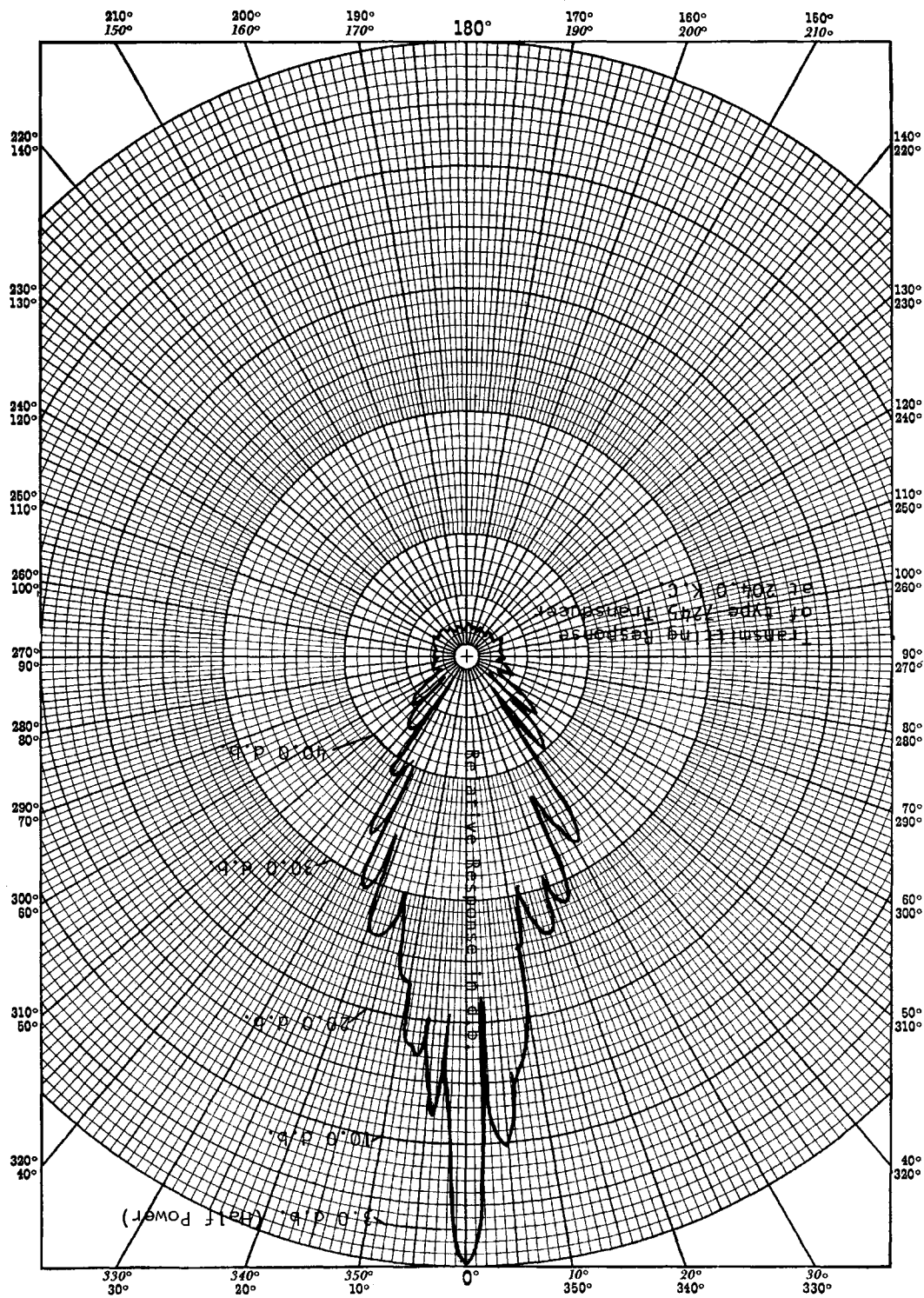


Figure 54. Transmitting response of type 7245 transducer at 204,0 khz.

To protect the transducer cables against abrasion from sand particles, initially, plastic garden hose was used to cover the cables from the transducer to a point above the highest flood water encountered. Present instrument package designs and cable protection with conduit eliminates the need for the garden hose.

A.2.3 Cage-type Support

The first design used to support and protect a depth transducer was a cage support as used in the first installation at Shreveport, Louisiana, figure 55. The one unit cage and its base plate were studded to the pier with the depth transducer in place. Due to problems encountered in the diving installation (covered in more detail in the installation section), later designs incorporated a two-piece unit consisting of a cage and baseplate. The baseplate was first affixed to the pier, then the cage with depth transducer in place was attached to the baseplate. The cage design as shown in figure 55 has not changed except for minor alterations to the method of clamping the transducer to the cage. The original design used a machined clamp with nuts and bolts to secure it to the cage. All subsequent designs use standard pipe clamps with wing nuts. As previously indicated many design changes were brought about to make the installations easier. Also, many changes were made because of problems encountered by debris.

The cage type depth transducer supports are presently being used at the Texas and Mississippi monitoring sites with no apparent difficulties. Both of these installations were done during very low river stages and, as a result, a very secure and adequate fastening of the cages to the pier was accomplished.

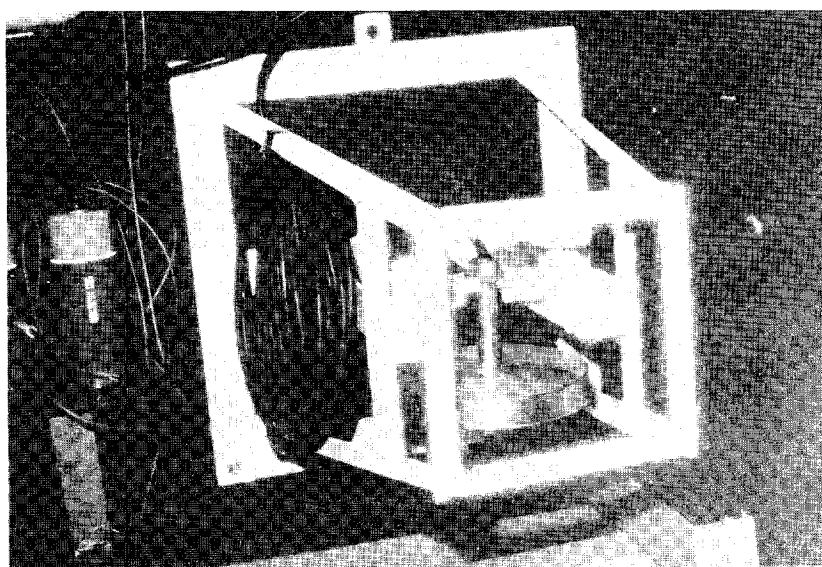


Figure 55. Cage-type Depth Transducer Support

Another cage-type transducer support, figure 56, was designed for specific mounting to a pier ledge. This support utilizes the same method of enclosing the transducer for protection. It differs from the previously mentioned cage in that it is conveniently mounted to a horizontal pier ledge, whereas the original design mounted to a vertical surface of the pier.

A major drawback in the cage design, besides its awkwardness to handle during installation, is that it creates enough flow disturbances to cause flow separation from the face of the depth transducer at high velocities. It was realized from data received from the Shreveport site, which utilized the cage design on the first two installations, that at high river stages the depth readings were either faintly recorded or not at all. At that time three possible factors were considered to be possible causes. They were 1) the flow separation just discussed, 2) a changing slope scour hole surface, and 3) a high sedimentation concentration. It has since been determined that all three possible causes can and do affect the readings with an additional problem of debris hang-up. Debris, caught on the instrument can give a zero depth or faint reading depending on the amount and size of debris.

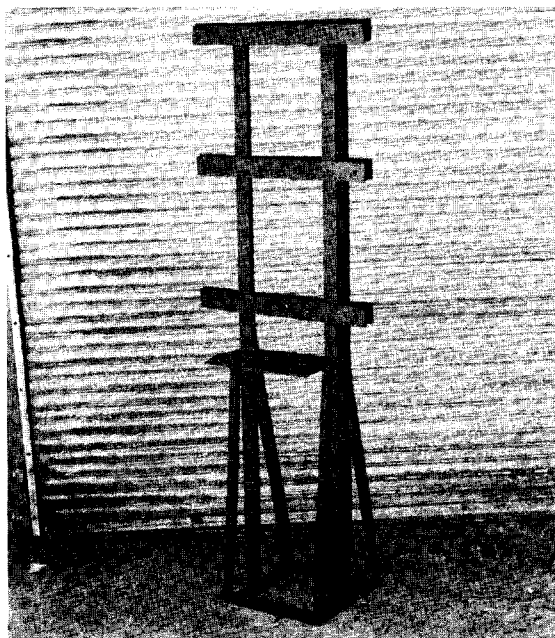


Figure 56. Ledge mounted, cage-type depth transducer support.

A.2.4 Testing of Cage Supports

To determine the effects of flow separation, tests were conducted with the depth transducers and their supports in the U.S.G.S. operated tow tank at the National Space Technology Laboratories, formerly the N.A.S.A., Mississippi Test Facility. These tests showed flow separation to occur at flow velocities exceeding 8 feet per second. It was determined that the cage-type supports created sufficient flow disturbance to cause the separation of flow from the face of the transducer. Attempts to streamline the flow with the transducer mounted within the cage support were unsuccessful.

Tests conducted with a tear-drop ferring attached to the transducer without the cage support, figure 57, allowed a usable velocity range of 15 feet per second. Additional tests were made with the transducer (without ferring) mounted toward the direction of flow at an approximate angle of one degree with respect to the reflecting surface. This slight incident angle was sufficient to increase the usable velocity range to 15 feet per second without the ferring. This test proved very beneficial in connection to the mounting of the transducers to the pier. Due to the slope of the scour hole, it is desirable to mount the transducer at a slight angle. This produces the needed incident angle with respect to the flow and allows the face of the transducer to be more parallel to the sloping surface of the scour hole.

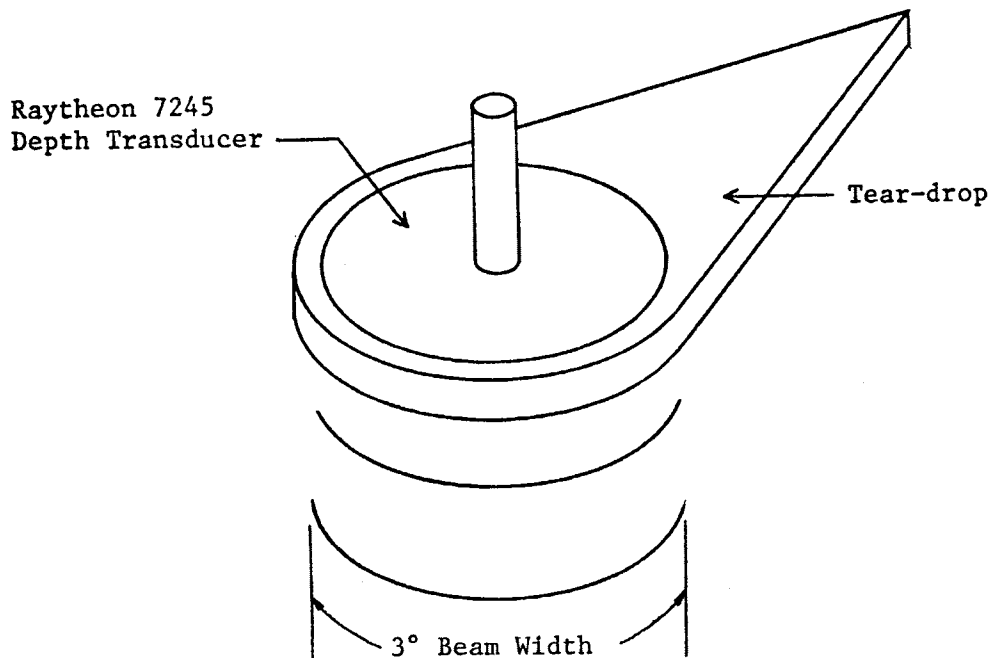


Figure 57. Mounting of tear-drop ferring on narrow beam transducer to improve flow characteristics.

As a result of the testing conducted with the transducer and cage-type support, a redesign of the depth transducer support was initiated. The support structure currently used consists of a single aluminum channel with a bolt-on housing to protect the transducer.

The narrow beam depth transducer was again tested using a cylindrical protecting cover, figure 58(a), in the following three configurations:

1. The depth transducer completely enclosed within the cylindrical cover.
2. The depth transducer protruding from the cover.
3. The depth transducer protruding from the cover, but with the teardrop ferring installed.

Configurations 1 and 2 showed no deterioration in the signal below a velocity of 10 feet per second. Above 10 feet per second some deterioration was observed, but complete loss of the signal was not encountered below 15 feet per second, the upper limit of the tow tank facility. Configuration 3 showed an improvement in operation. Velocities could be increased to 12 feet per second without loss of signal strength. It was observed during these tests that further improvement could be made by inclining the transducer at a slight positive angle relative to the flow. In this configuration, velocities of 15 feet per second were obtainable without any loss in signal strength.

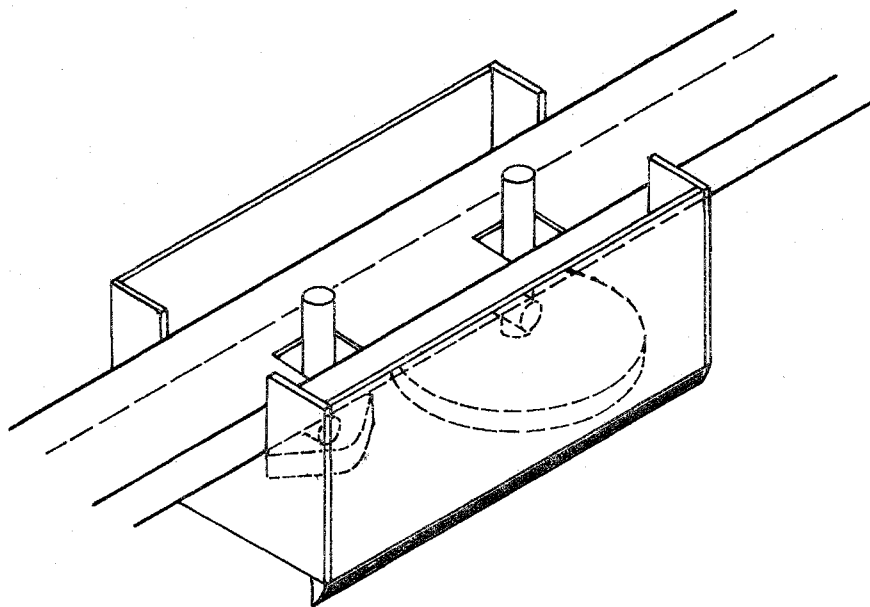
The box-shaped protective cover, figure 58(b), was tested in the following configurations:

1. With the transducer completely enclosed.
2. With the transducer protruding.
3. With the transducer protruding but protected by the streamlining cowling.

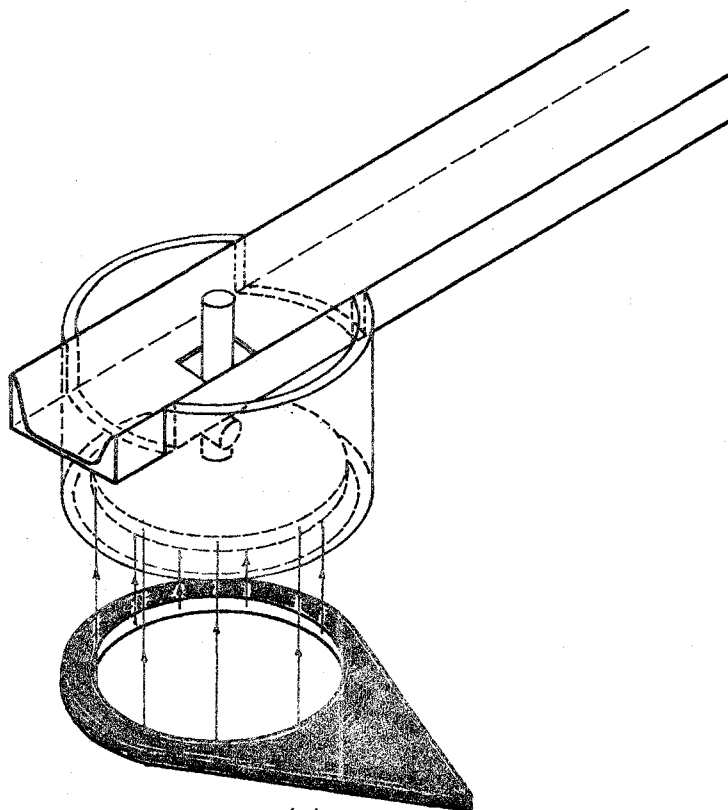
Configurations 1 and 2 for the box-shaped protective cover again showed that velocities of 10 feet per second could be achieved without loss of return signal. Configuration 3 permitted the velocities to be increased above 10 feet per second without significantly reducing the signal strength.

It has been concluded that cavitation caused from the sharp edges on the transducer body and the sharp edges on the original cage design was at least partially responsible for the loss of the signal at higher velocities. These tests showed that the redesigned support structure with both cylindrical and box-shaped covers has improved the situation. The transducer can now be depended upon to operate effectively in flows with velocities exceeding 10 feet per second without signal loss due to flow separation. In cases where the transducer face is inclined at a slight positive angle with respect to the flow, i.e., the bow position on a pier, the effective velocities will be higher.

The diamond-shaped 15° transducer was tested and shown to operate satisfactorily at velocities up to 15 feet per second.



(b)



(a)

Figure 58. Depth transducer protecting covers; (a) cylindrical cover to be used with 3° transducer, (b) box-cover used with combined 3° and 15° transducer,

A.2.5 Latest Depth System Designs

As previously indicated it became necessary to redesign the depth transducer support instrumentation due to the problems encountered in the original design. The all steel constructed cage-type support, in addition to creating flow disturbances, was heavy and cumbersome. Two divers were required for the underwater installation of the cage.

The latest support design, figure 59, is an all aluminum construction making it lighter and easier to handle. The transducer is mounted three to four inches below the support and is protected without completely enclosing it. This method of supporting the transducer protects it without the support itself creating additional flow disturbances to the face of the transducer.

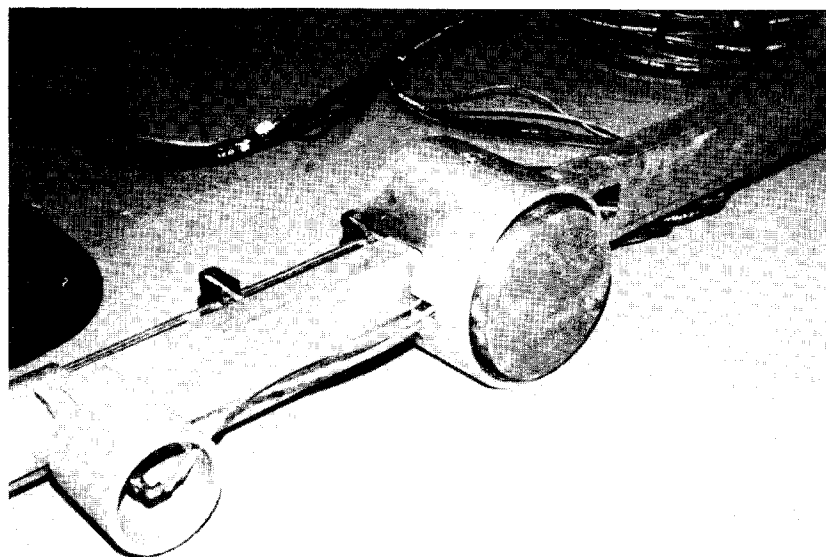


Figure 59. Current method to support and protect depth transducer.

A.3.0 Stage Instrumentation

River stage measuring instrumentation is incorporated in the depth measuring system. Stage measurements are made by directing the sound waves from a narrow beam transducer toward the river surface. In the principle of echo-sounding the reflecting surface need only have a different density than that of the fluid in which the waves are transmitted. In fact, the strength of the reflected sound waves from the stage transducer produce a better recording than those from the depth transducers. This is a result of the sharply defined interface between the water-air surface. Where as the interface between the water-soil bottom may be in a highly concentrated mixture, especially during floodings.

The stage transducer supports have followed the same development as the depth transducer supports. The early supports were steel cages and the latest supports are aluminum channels with cylindrical protecting covers, figure 60.

A.4.0 Depth-stage System Control

The first controls used with the depth measuring system included two separate control units. They were a timing unit which automatically turned on the recorders and a transducer sequencing unit which permitted manual selection of the depth or stage transducers, figure 61.

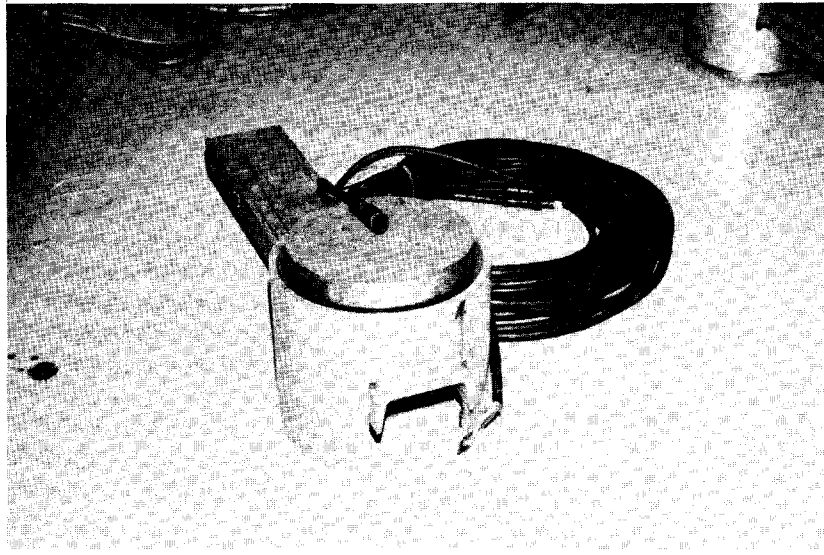


Figure 60. Stage transducer support and protecting cover.

A.4.1 Timing Unit

The timing unit, shown schematically in figure 62, utilized both a 12 volt and 24 volt power supply. The 24 volt source was necessary to operate the twenty-four position stepping switch. This particular type of stepping switch was needed to obtain a variety of system turn on intervals, since the frequency with which depth measurements should be monitored could not be initially determined. In addition to supplying power to the timing unit clock and relays, the 12 volt source is channeled through the timing unit to the depth recording system. The timing unit has the capability of turning the system on for three minutes at preset intervals of 1/hr., 1/4 hr., 1/6 hr., 1/12 hr., or 1/36 hr.

A.4.2 Transducer Sequencing Unit

The manual sequencing unit, shown schematically in figure 63, provided receptacles for the outputs from the depth and stage transducers and a single input receptacle into the recording system. Consequently,

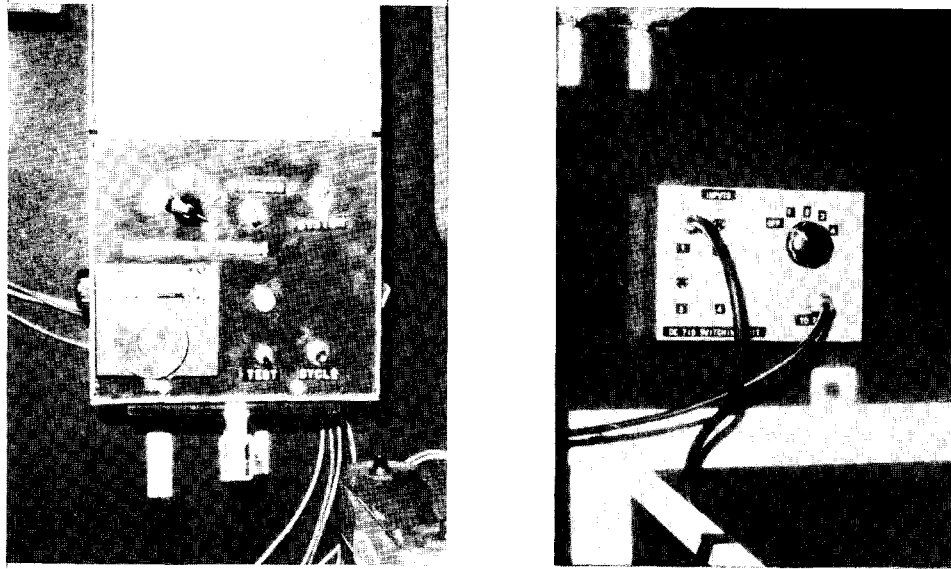


Figure 61. Depth system timing unit (left) and manual transducer sequencing unit (right).

simultaneous recordings from each of the depth transducers and the stage transducer could only be made when an operator were available to manually switch the transducers. This was a major drawback in the depth control system.

A.4.3 Automatic Control Unit

The present depth system control unit, figure 64, has overcome the need for any manual operator functions. By combining the original timing unit with an automated sequencing unit, a completely automated depth control system has been developed.

The Automatic Control Unit (A.C.U.) has the same system turn-on capabilities as the original timing unit. Through the use of an additional stepping switch and a more complex relay system the A.C.U. alternately switches from one transducer to another letting each have a recording time of thirty to forty-five seconds, figure 65. Presently the control unit has receptacles to handle four transducers (3 depth and 1 stage) but could be modified to accept a total of eleven transducers.

Receptacles for transducer output and recorder input are located on the right side of the control unit. Battery power to the entire depth system is supplied at single input receptacles along with output receptacles for the various recorders on the left side of the control unit. Additional receptables are located on the left side of the control unit for power to the analog circuit board and for the level indicator connection.

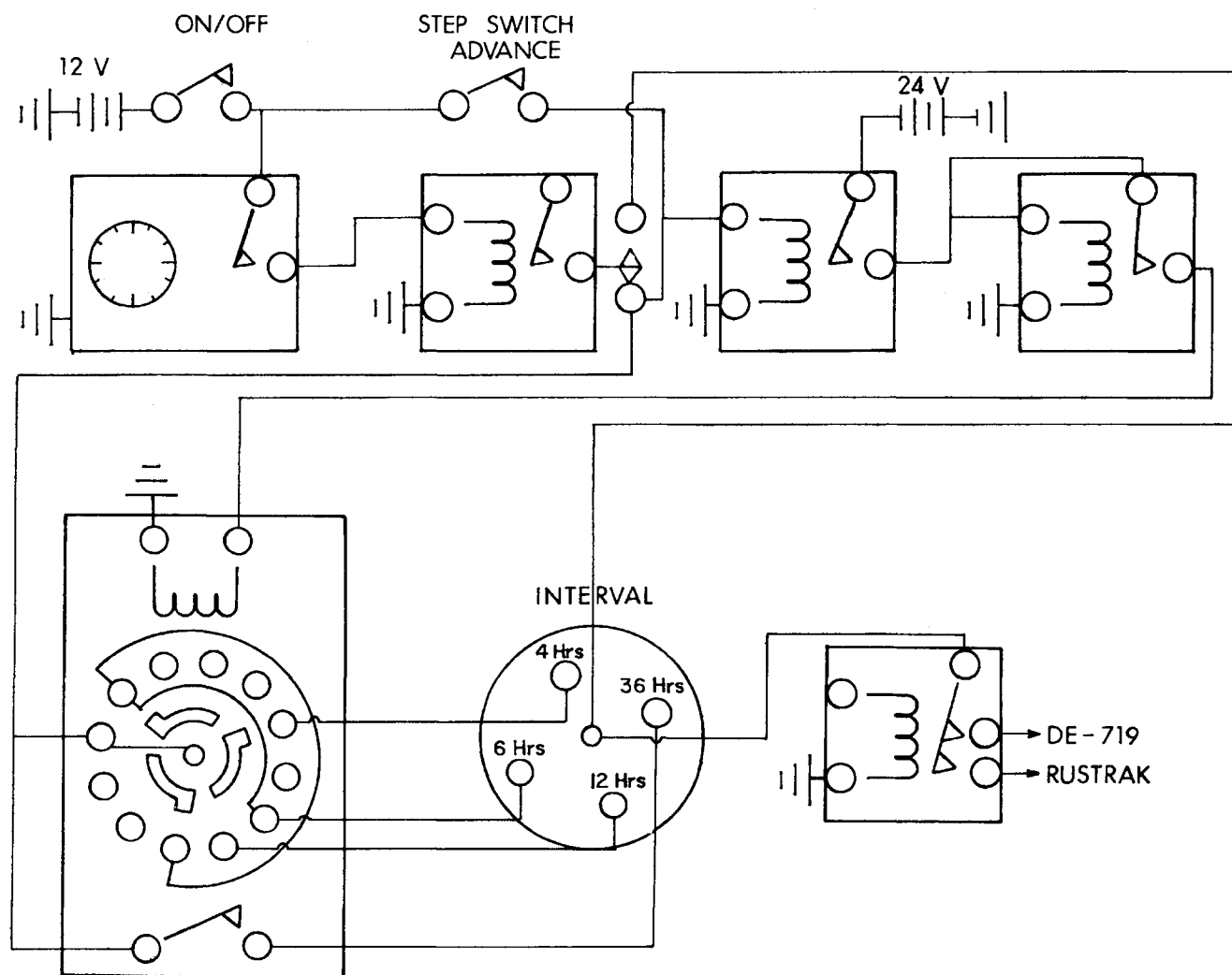


Figure 62. Depth system timing unit (schematic).

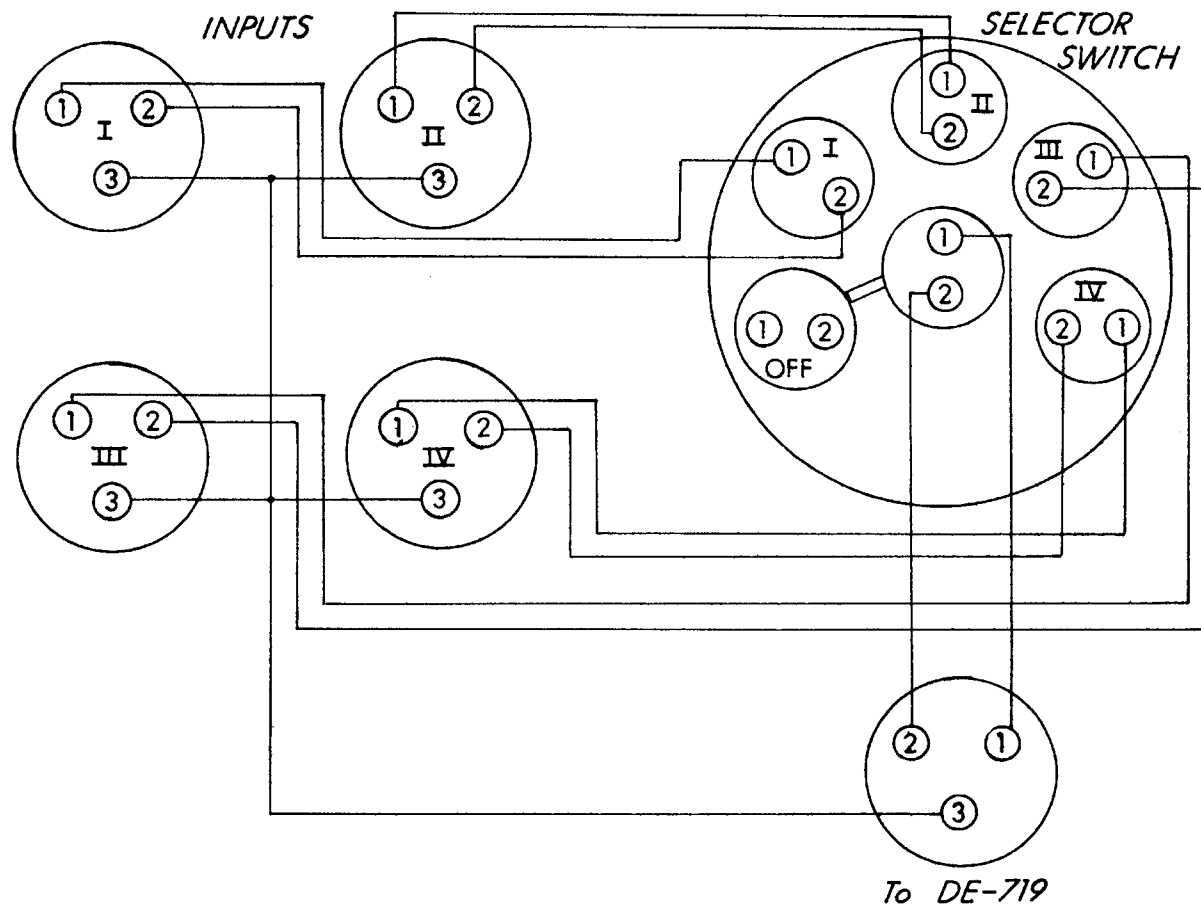


Figure 63. Manual Sequencing Unit (schematic).

A.4.4 Level Indicator Switch

A level indicator switch is mounted on the bridge pier, figure 66, at a flood stage elevation. Closing of the indicator switch by high waters causes the turn-on interval of the A.C.U. to change from a normal 1/36 hr. to a 1/4 hr. recording interval. This allows more frequent readings during periods that the scour hole depth may be rapidly changing.

A.5.0 Recorders

Permanent recordings of data from the depth and stage instrumentation are simultaneously produced on chart paper from two separate recorders. The secondary or back-up recorder is used to prevent the loss of data in the event the stylus of the primary recorder wears out.

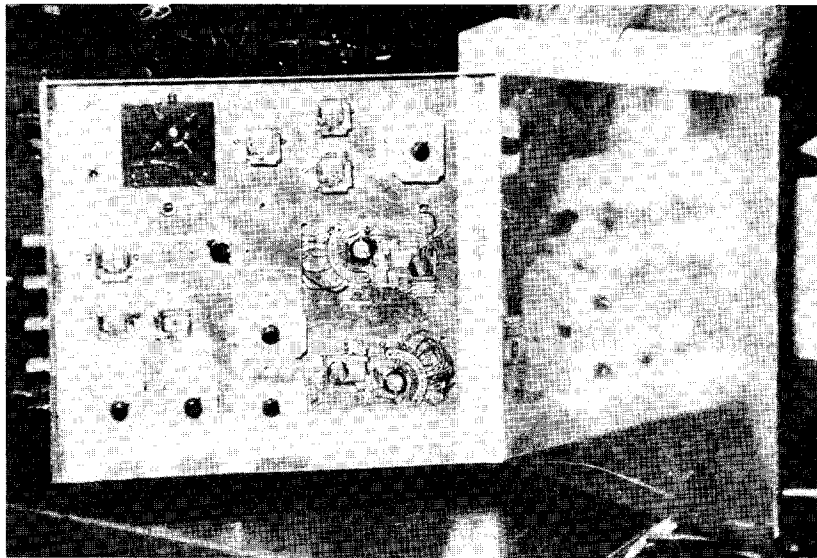


Figure 64. Automatic Control Unit

A.5.1 Primary Recorder

The primary recorder contains completely solid-state transmitter and receiver circuitry in addition to being a basic recording system. The transmitter is basically a Master-Oscillator Power-Amplifier operating at a frequency of 200 kHz. A built-in inverter oscillator which operates from the 12 volt D.C. power supply, furnishes the operating voltage to the transmitter amplifier. The pulses of electrical energy from the transmitter are converted by the transducer into pulses of supersonic energy which travel through the water toward the bottom or surface. The reflected echoes of supersonic energy are then received by the transducer,

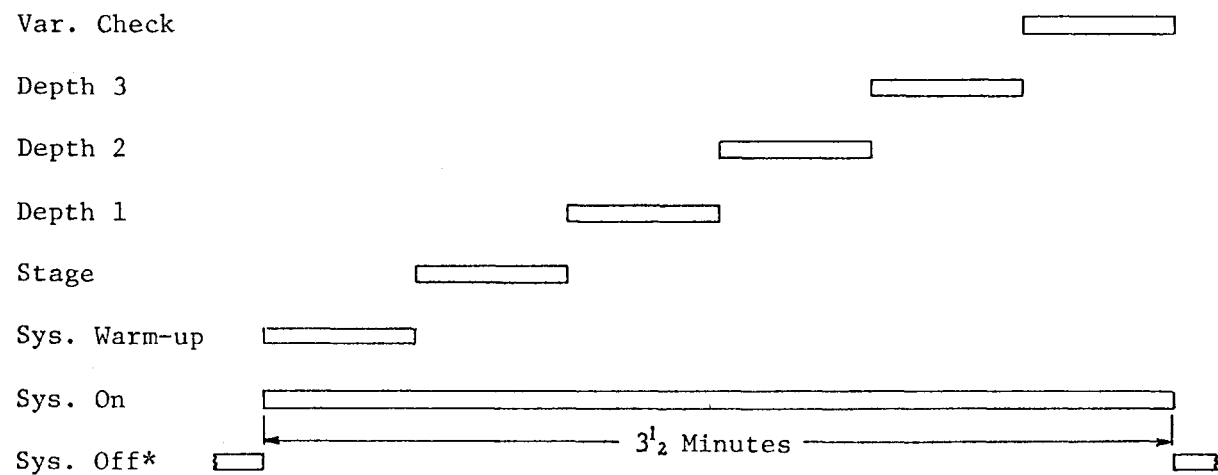


Figure 65. Depth System, Automatic Control Unit Operation Schedule

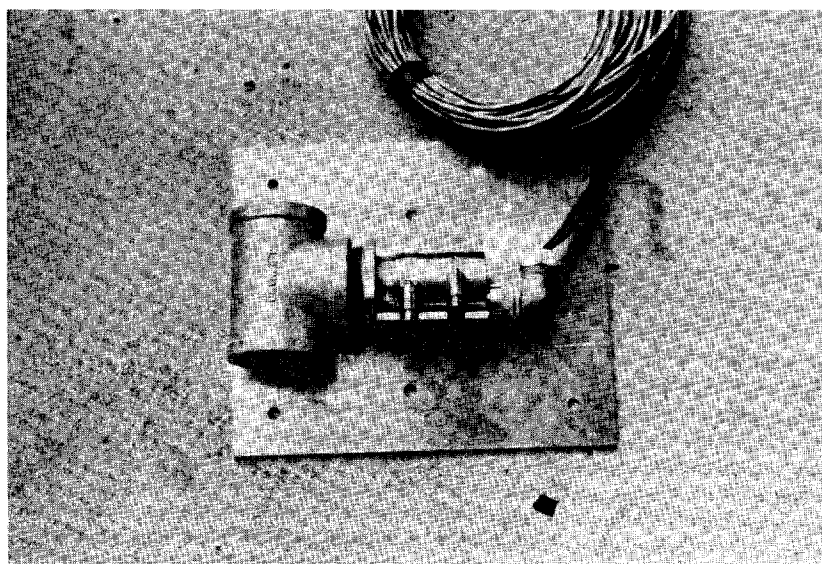


Figure 66. Level Indicator Switch

converted back to electrical signals and received by the recorder. The output of the receiver is rectified and filtered to remove the 200 kHz. component and delivered to the stylus. By coupling the speed of the stylus drive motor with the speed of the signal transmission and reception, a D.C. voltage, which is compared to the motor speed, is delivered to the stylus. Consequently, the D.C. voltage across the stylus burns a trace on the chart paper at intervals which are indications of depth.

A.5.2 Analog Circuit Board

The primary recorder receives a pulse-type signal. This signal is transmitted through the analog circuit board, located on the back of the recorder and converted to an analog signal. The millivolt analog signal is then transmitted to the secondary recorder.

A.5.3 Secondary Recorder

The back-up recorder is a basic 12V D.C. recorder. It prints through the impinging action of its stylus against pressure-sensitive paper. Recording on the cord of the stylus arc allows presentation of the data as a continuous line.

Writing speed varies with motor speed. Chart speed and trace density depend on the ratio of the gear box, which couples the paper drive to the motor. The controls are located on the front of the recorder.

A.6.0 Power Supply

Operation of the depth and stage measuring instrumentation system depends on both a 12 and 24 volt D.C. power supply. The recorders operate

from the combined voltage sources. The comparatively large current demand from the recorders requires a large 90 amp hr. capability, while the control unit requires only a 15 amp hr. rating.

During periods of flooding, more frequent measurements are needed, which results in heavy use of the depth system. Due to the large drain on the battery power supply during this time a constant charge on the batteries was necessary. As a result a 110 volt A.C. operated battery charger was built to constantly supply a .9 amp charge to the main power supply.

A.7.0 Velocity Meter

Due to its recognized, two-dimensional measurement capacity the velocity meter used in the scour monitoring system has undergone extensive development. Unlike most state-of-the-art velocity instrumentation used in this type of field research, a velocity vector may be measured with this meter.

A.7.1 Principle of Operation

The basic principle of operation of the velocity meter is that the drag on the sphere is proportional to the fluid velocity, figure 67. The drag force on the sphere causes a bending moment to occur at the base of the connecting rod. This bending moment is measured with strain gages. Therefore, by calibrating the meter over a particular velocity range, the strain measurements are indications of velocity. This type of calibration also allows for the very small drag force along the length of the connecting rod. The calibration facility is shown in figure 68.

A.7.2 Meter Protection

The velocity meters major drawback is that it is sensitive to environment and, consequently, difficult to protect. Current methods to waterproof the strain gages, using a silicone rubber compound have not been completely satisfactory. The life expectancy of the meter is presently only two or three months.

Keeping the meter from being hit by floating debris requires a second form of protection. The initial design used an air cylinder to slide the meter in and out of a protective housing, figure 69. Due to difficulties encountered with the strain gage transmission cables this design had to be altered. The present design uses the same air cylinder to slide a protective covering over the stationary meter, figure 70. A second smaller air cylinder locks the cover into the closed position.

A.7.3 Air Control

The air control system, figure 71, is used to extend and retract the velocity meter protecting cover. Two air hoses, per velocity meter,

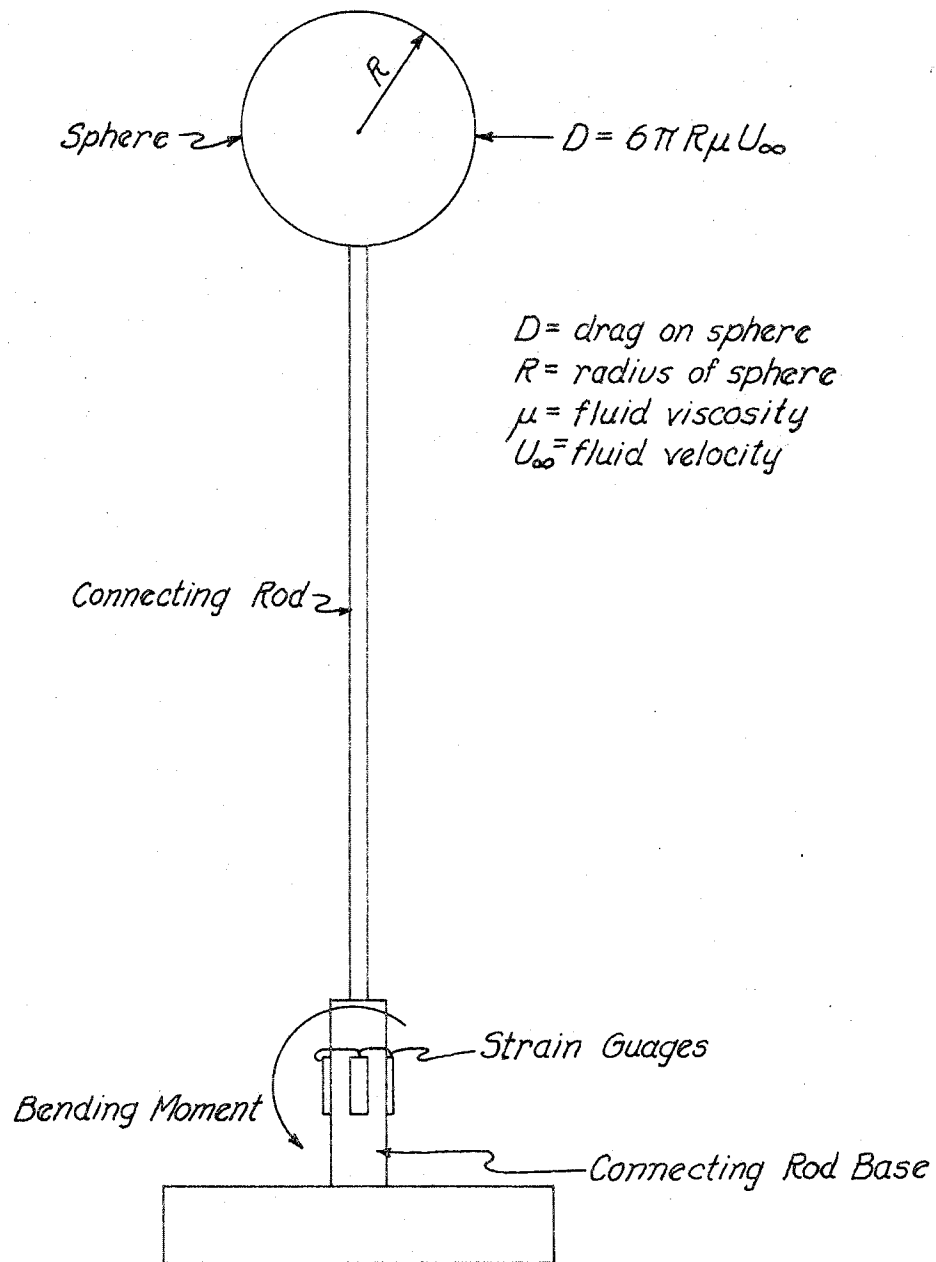


Figure 67. Velocity meter principle of operation.

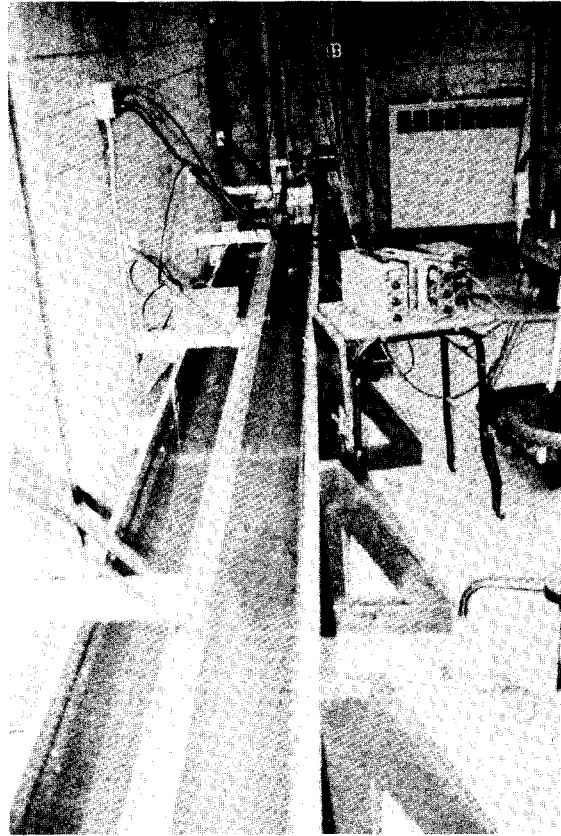


Figure 68. Velocity meter calibration facility.

terminate at the instrumentation housing with quick-connect fittings. Both air hoses from any velocity meter are plugged into the air control unit.

The air control unit consists of a system of three-way valves, pressure reduction valves, and manifolds. Air at 1600p.s.i. from an air cylinder (vol. = 26 cu. ft.) is reduced to 60 p.s.i. in the air control unit and directed to either the extend or retract air hose.

A.7.4 Future Development

In effect the velocity meter is not rugged enough in its present stage of development. Although its application in this field of research could be very valuable, further development may be cost prohibitive. An important consideration concerning further development of the meter is its state-of-the-art usefulness. Existing theories do not require a velocity measurement of this exactness.

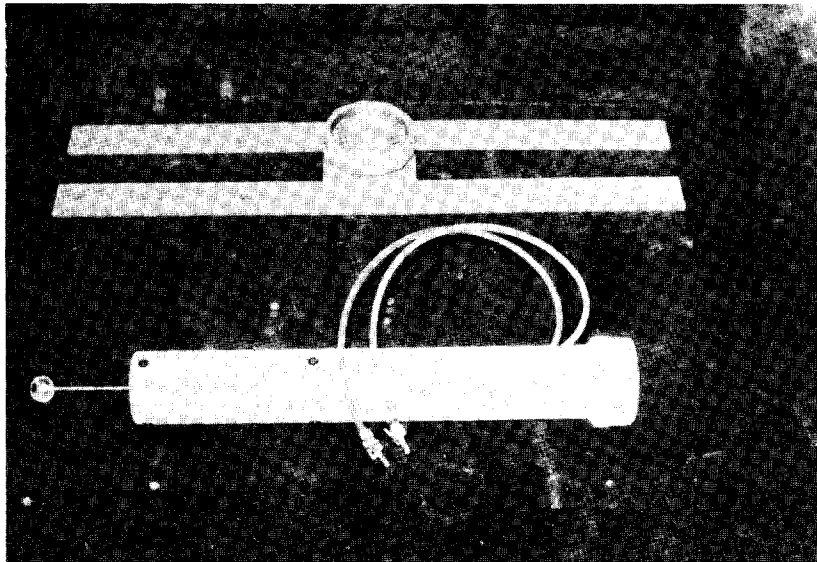


Figure 69. Early design for the protection of the velocity meter.

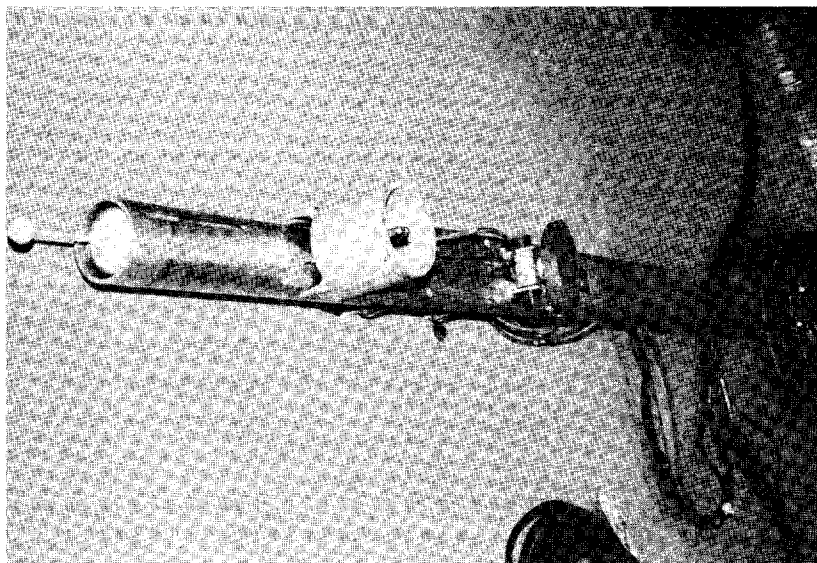


Figure 70. Latest velocity meter protection design (movable protecting cover).

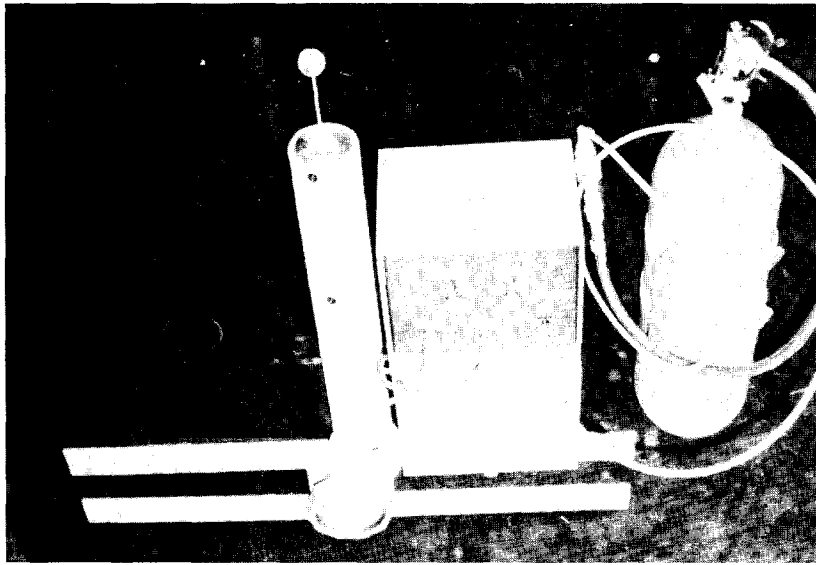


Figure 71. Air control for velocity meter protection

A.8.0 Field Equipment

The field equipment for a study of this nature is essential to its outcome. To perform the underwater portion of an installation requires a great many special tools as well as diver support equipment. A boat and motor capable of handling many different requirements is a major part of that support equipment.

In addition to installation of equipment, data collecting equipment is needed.

A.8.1 Mobile Research Vehicle

The research vehicle, figure 72, is used to transport the scour monitoring instrumentation as well as the field equipment. As a consequence the van was equipped to meet the needs of this type of field study. A special rack was constructed on the top to carry a disassembled 8' x 8' work platform along with a ladder, cable-protection conduit and at various times other paraphernalia. Tool cabinets, equipment racks and a work bench were built into the inside of the van. During data collecting it must serve as a field office.

A.8.2 Inflatable Boat

An inflatable boat, figure 73, was chosen due to its portability and stability. When not in use the entire boat required a 1' x 2 1/2' x 3 1/3' storage space. During installations it was possible to conduct diving operations from the boat while carrying a diving compressor, installation equipment and instruments, and two or three crew members.



Figure 72. West Virginia University Research Vehicle.

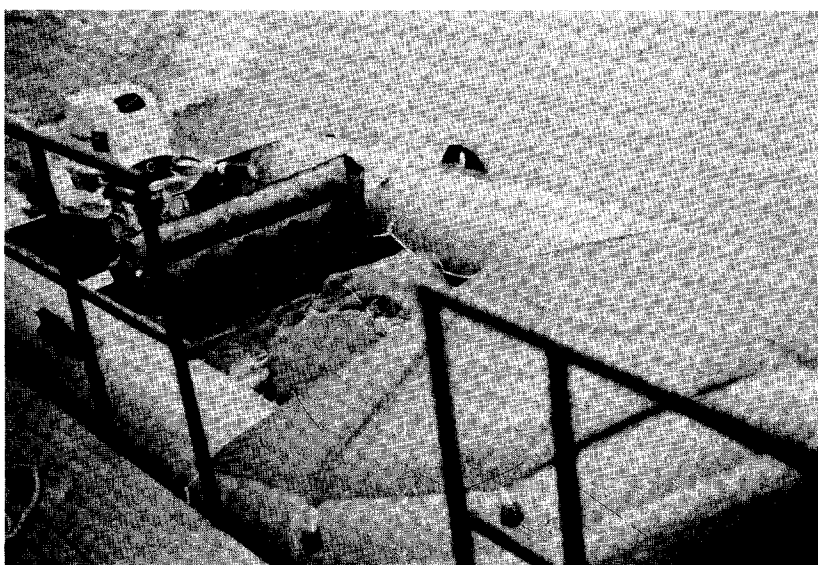


Figure 73. Portable Inflatable Boat.

The boat's stability was also of great convenience during the collection of data. Field instruments like a survey fathometer or velocity meter could be clamped to the stern of the boat.

A.8.3 Special Tools

To affix instruments to the pier required the use of a 44 caliber underwater stud driver, figure 74. Diver opinion was that it was one of the more effective stud drivers they had ever used. It could drive 1 3/4" long threaded stud through a 1/4" thick piece of steel into a concrete pier with 1 1/4" penetration. The pull strength of one of these studs shot directly into concrete was 3,000 pounds force.

A 27 caliber stud driver and a concrete drill were used for the surface installation of cables and protective conduit.

5.8.4 Diving Equipment

Diving equipment included wet suits, air tanks, a communications system, an air compressor and all kinds of specialized small tools. Part of the diving was conducted using scuba tanks and part using a compressor on the surface. When the surface air supply was used, communications could be made between diver and surface crew, figure 75.

5.8.5 Data Collecting Equipment

To determine channel geometry, survey equipment was needed. A transit and elevation rod was used to determine the location of the bridge crossing with respect to the channel along with general surface topography of the channel. A portable survey fathometer was used to measure channel cross-sections.

A vane-type current meter was used to obtain vertical velocity profiles approaching the pier from an undisturbed upstream location. This is the type of current meter generally accepted and used by most agencies to measure discharge.

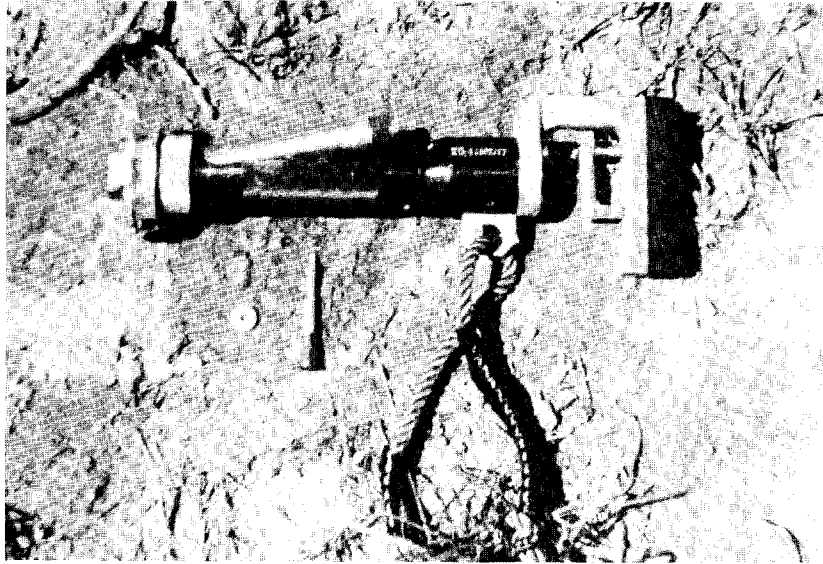


Figure 74. 44 caliber stud driver.

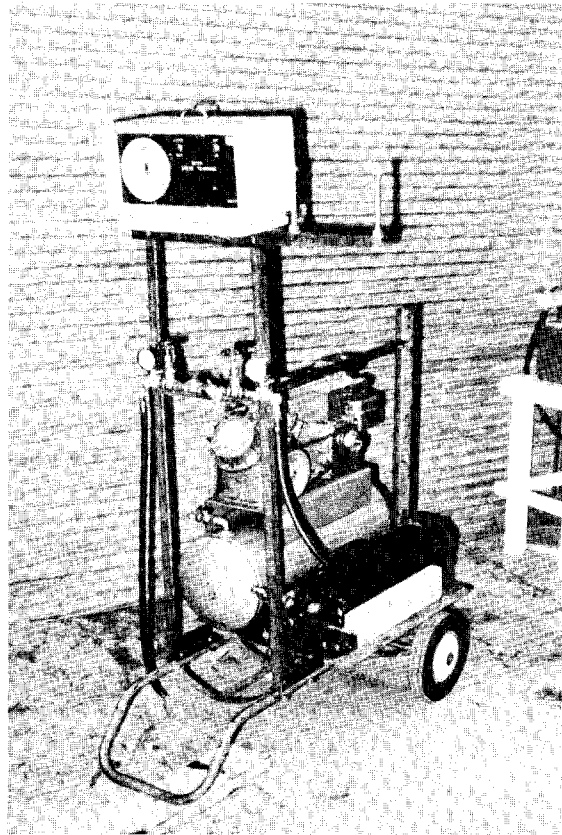


Figure 75. Diving compressor with diver communications system.

B. SITE INSTALLATION

B.1.0 Red River at Shreveport, Louisiana

The first scour monitoring instrumentation system was installed on the Texas Street Bridge over the Red River in Shreveport, Louisiana during April 26 - May 5, 1972. The river receded from stage 10 to stage 8 during the installation. At stage 10 the instrumented pier 6 is approximately 350 feet from the left bank and 1200 feet from the right bank, figure 76. Pier 6 at low flow, stage 4 is shown in figure 77.

Two depth transducers, one staging transducer and three velocity meters were attached to the pier as shown in figure 78. The depth transducers were protected in four foot long cage supports; the stage transducer in a two foot long cage support. The velocity meters were base mounted and protected by a removable plexiglas bubble. Cables were clamped to the pier as shown in figure 78. From the pier cap the cables were fed through a vertical beam of the bridge superstructure to the bridge deck, figures 79 through 83.

The instrumentation housing, figure 84, was bolted to the bridge deck on the sidewalk directly over pier 6.

B.1.1 Installation and Diving Problems

As this was the first bridge to be instrumented, several unforeseen difficulties were encountered. Access to the river in the vicinity of the bridge could only be accomplished by lowering the boat and equipment down a 45 degree bank, figure 85.

Diving conditions were extremely difficult due to zero visibility and high current velocities around the pier, figure 86. Five feet below the water surface all light was blanked out. In typical low visibility rivers the diver can see light if not objects. However, in the Red River at stage 10, it was impossible for the diver to see the inside of his face mask. It was necessary for the diver to do everything by touch.

During free S.C.U.B.A. diving, the diver cannot orient himself and quickly fatigues from combating the current. The diver is normally oriented by watching the bubbles from his S.C.U.B.A. system. It was necessary for the diver to be heavily weighted so he could walk on the river bottom to combat the current and get some sense of direction. Consequently, a surface air system (S.A.S.) was used, figure 87.

Extended use of the Underwater Stud Driver for fastening of the cage supports to the piers was found to be very uncomfortable for the divers. The pressure wave produced from firing the stud driver was fatiguing and extremely uncomfortable to the divers ears. In addition, the high bacteria count in the Red River resulted in the divers having ear infections.



Figure 76. Instrumented Pier #6, Texas Street Bridge over Red River.

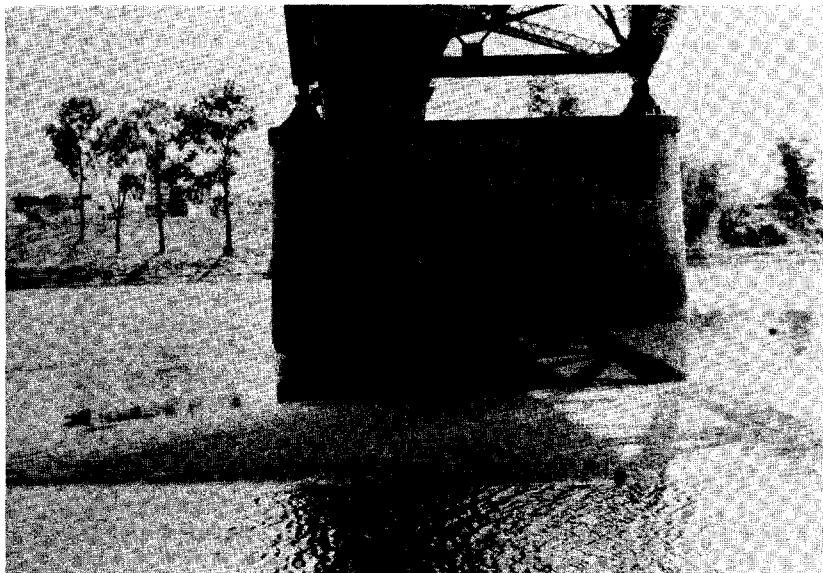


Figure 77. Red River at low flow, stage 4.

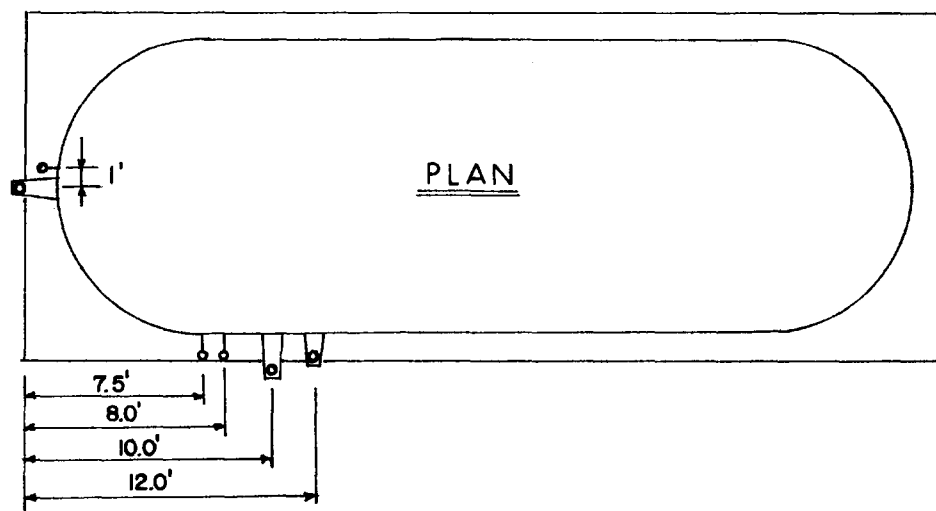
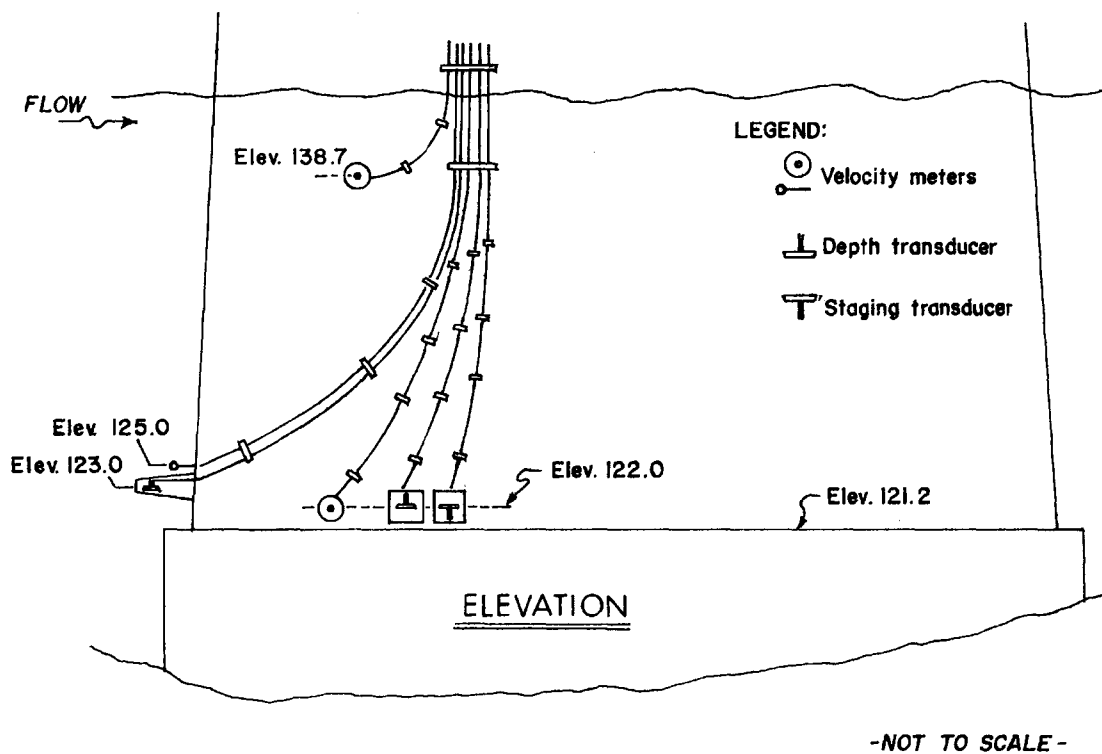


Figure 78. Instrument location, Pier 6, Red River, April-May, 1972, installation.



Figure 79. Cable being prepared to be lowered over bridge deck.

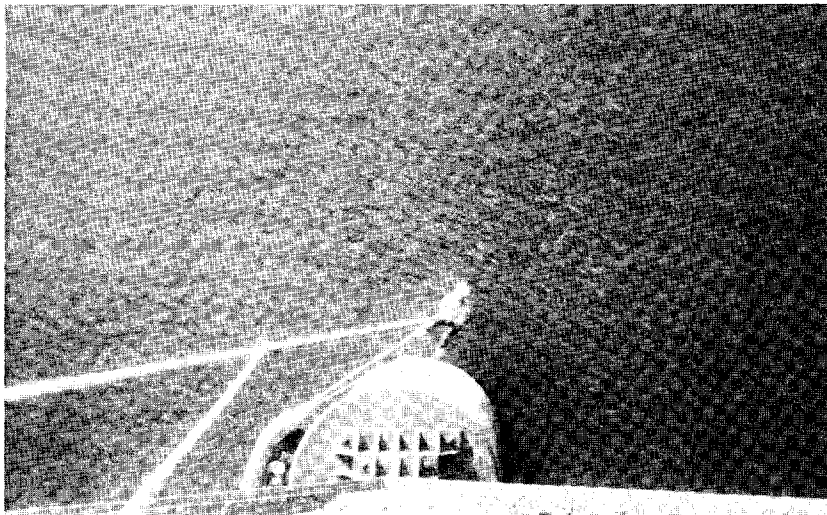


Figure 80. Worker attempting to gain access to pier superstructure by lowering himself in boatswain chair from bridge deck.

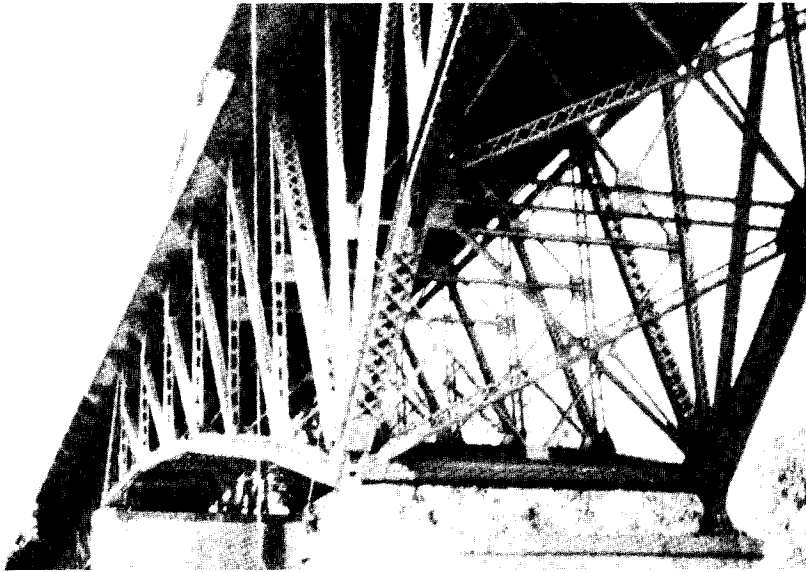


Figure 81. Cable lowered from bridge deck, to be fed into superstructure.

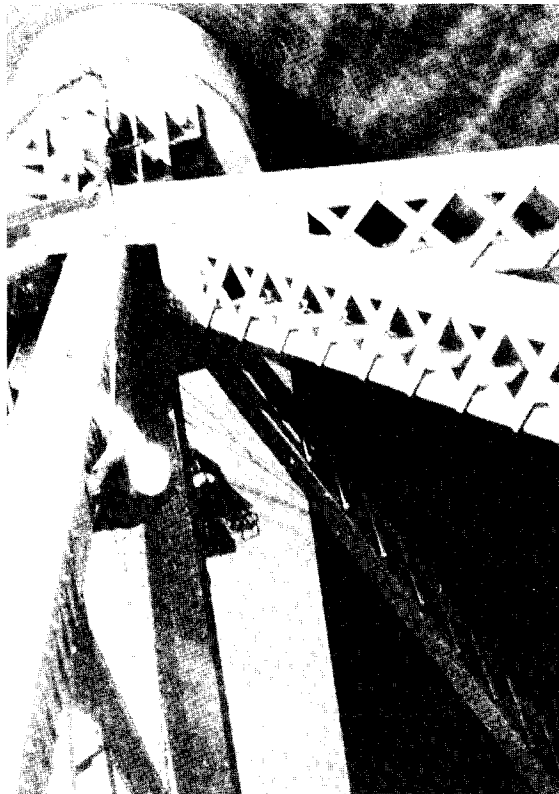


Figure 82. Cable being fed into vertical beam of superstructure.

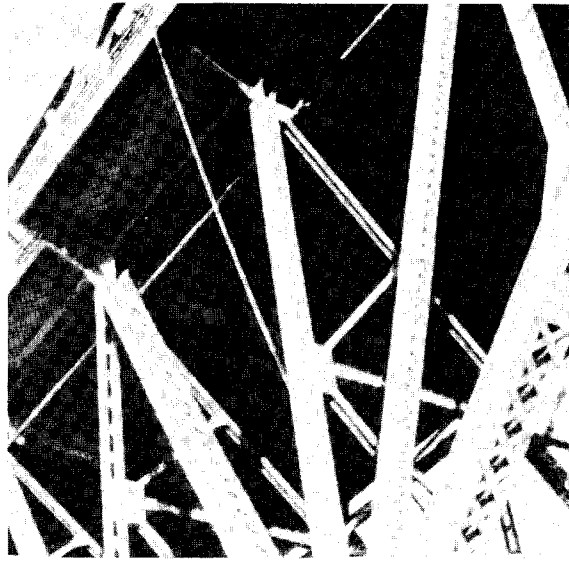


Figure 83. Completed cable installation in bridge superstructure.



Figure 84. Instrumentation housing on sidewalk over instrumented Pier 6.

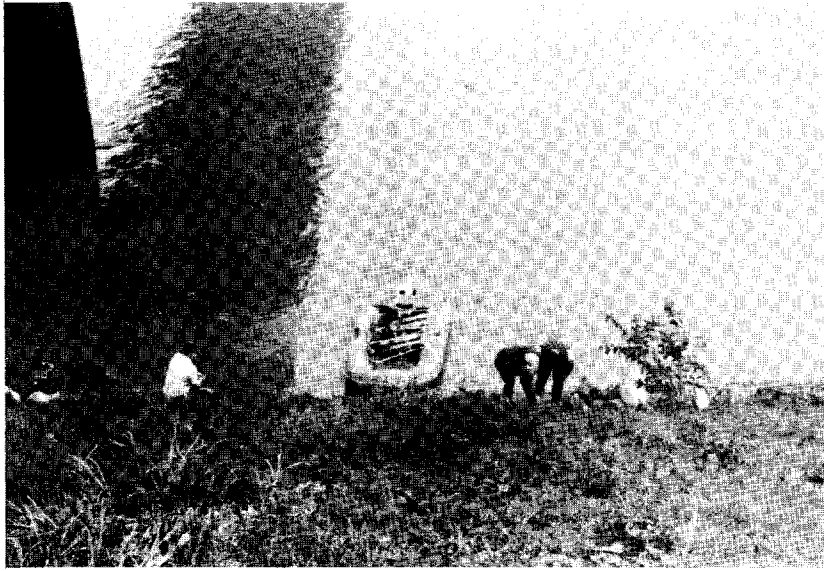


Figure 85. Access to Red River from left bank.



Figure 86. Bow wave on Pier 6 during river stage 10.

It was determined, several months after the first installations, that a problem involving water-borne deposits existed. Larvae live in the water of the Red River and attach themselves to underwater surfaces. The entire pier and the instruments affixed to the pier are covered by the larvae. Figures 88 through 91 show the effects of these larvae to the instrumentation. Moving parts become inoperable and the sounding face of the depth transducers worked less effectively.

To combat this problem, moving parts were constructed with greater tolerances and an anti-fouling paint was used to prevent the build-up of these larvae on the transducer.

B.1.2 Update Installation, Red River

The scour monitoring system for the Texas Street Bridge was updated during September 27 - October 10, 1972. An additional depth transducer was attached to the left side of the pier, in the same plane of symmetry as the transducer on the right side. All of the depth transducers and staging transducer were temporarily removed for cleaning.

All of the velocity meters from the first installation were completely removed and replaced with ones having the extender/retractor method of protection. These velocity meters were installed on the bow of the pier above the depth cage, and on each side (tangent point) of the pier at the corner of the vertical pier wall and the top of the footing.

Cables were again attached to the right side of the pier up to the pier cap. To provide more adequate protection the cables were secured to the pier cap, figures 92 and 93 and covered with plastic conduit from the water surface to the pier cap, figure 94.

A new cable assembly was installed from the pier cap to the instrumentation housing. All cable connections at the pier cap were made with plugs and fittings for convenience in future repairs or changes.

The original timer and sequencing unit were replaced with the automatic control unit (A.C.U.). A level indicator switch was attached to the pier at an elevation of 146.48 (stage 15) to convert the A.C.U. operation interval to a once per hour mode during rises in river stage.

The instrumentation housing was equipped with a weather canopy and a vibration isolator, figure 95. Heavy traffic on the bridge caused extreme vibration which could be harmful to the recorders.

B.1.3 Flood Damage To The Installation

Depth and stage data were collected from the Red River monitoring site from the end of the update installations until February 1, 1973. On April 11, 1973 a flood alert trip was made to the site to gather data. At this time it was learned that all of the transducers were inoperable. Although visual inspection indicated that the cables were still attached

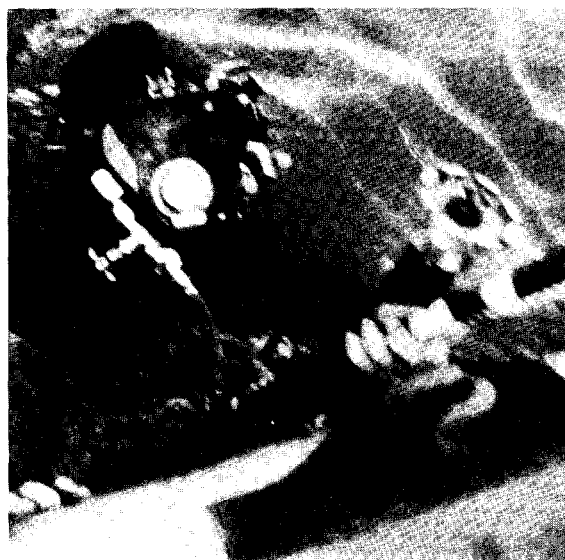


Figure 87. Diver using surface air system during installation.

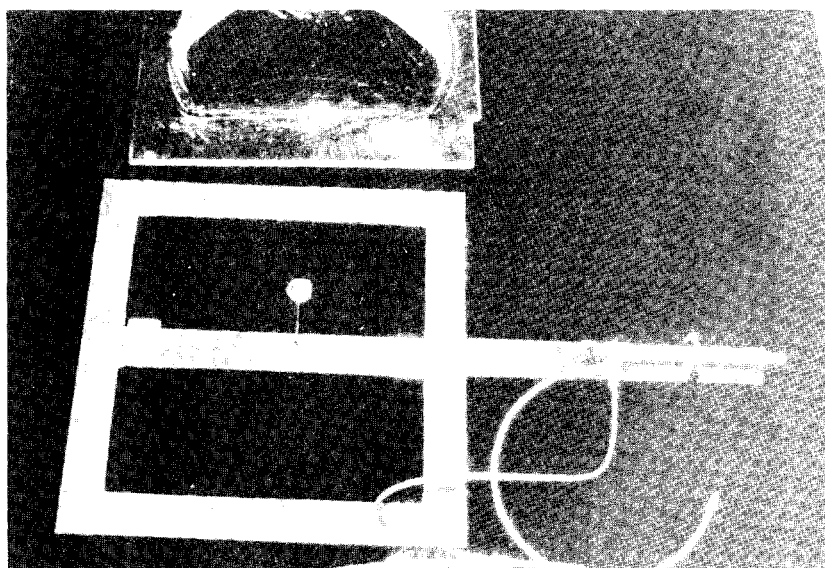


Figure 88. Velocity meter prior to installation.



Figure 89. Velocity meter covered with larvae from the Red River.

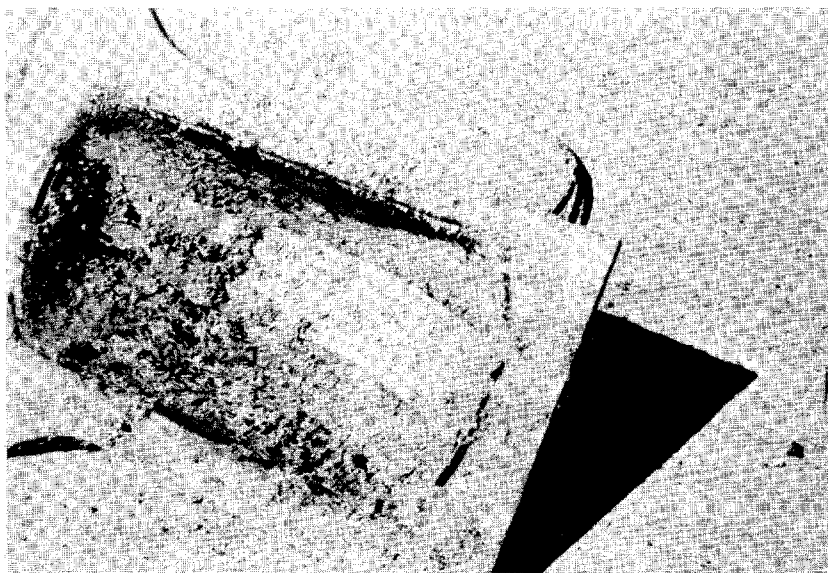


Figure 90, Protective bubble covered with larvae.



Figure 91. Depth transducer covered with larvae.

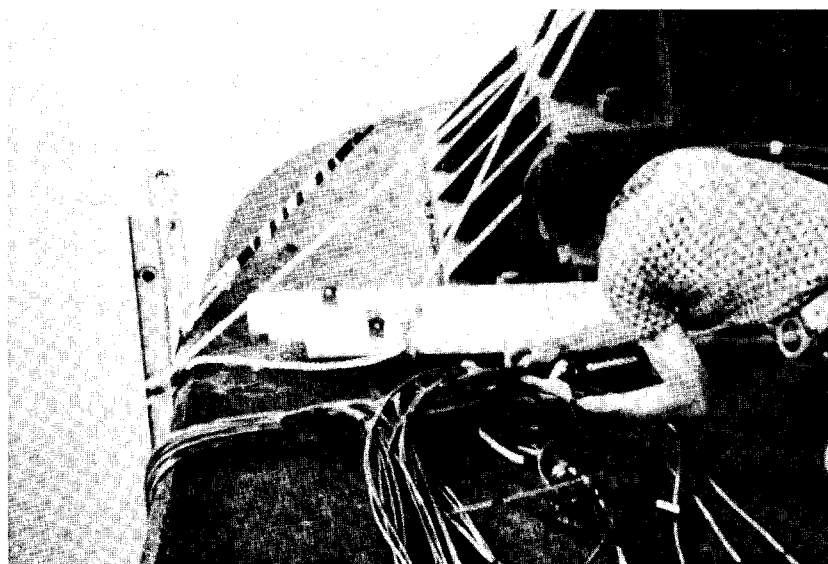


Figure 92. Cable connections on pier cap.

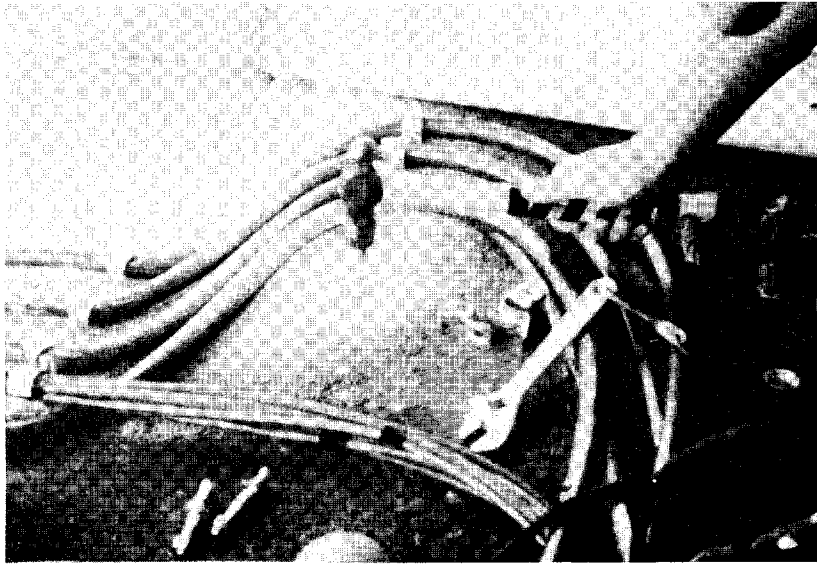


Figure 93. Securing cables to pier cap.

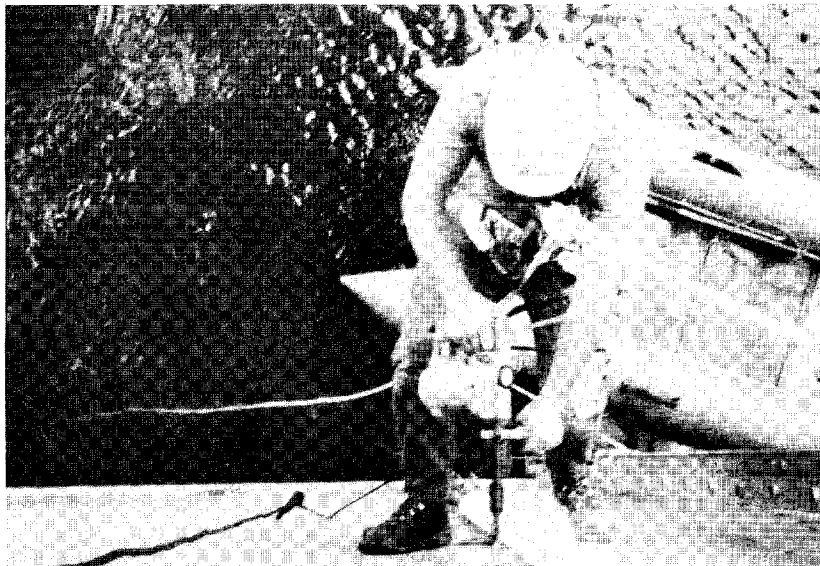


Figure 94. Method of covering cables with plastic conduit.

to the side of the pier it was felt that damage due to debris had occurred below the water surface. Since, at that time, the river was in flood stage, an inspection dive could not be made to assess the damages.

From the time that recorded data was available until the April 11, 1973 alert trip there had been two significant rises in river stage. They were; stage 19.8 on February 12, 1973, and stage 23.9 on March 17, 1973. The average stage from the time of installation until the crest on March 17 was 13.64. Either of the highs (19.8 or 23.9) were sufficiently above the average stage to produce excessive amounts of debris.

The next maintenance trip to the Red River was conducted on June 3, 1973. From the time of the April alert trip to the June maintenance a high stage of 26.5 occurred on April 29. This high stage left the instrumented pier 6 completely surrounded by debris, figure 96. At this time the stage was 12.3 and it could be seen that the entire cable assembly had been severed by debris, figure 97.

New instrumentation with better methods of cable protection have been developed for the Red River site. The installation of this system is to be made in August, 1974.

B.2.0 Mississippi River, at St. Paul, Minnesota

The second bridge chosen to be instrumented was the I-35E Bridge, figure 98, over the Mississippi River at St. Paul, Minnesota, during November 9-16, 1972. The pier to be instrumented was Pier 2 located approximately 320 feet from the right bank and 230 feet from the left bank during normal pool stage. Since it was anticipated that water temperatures in the 30 degree range and air temperatures of 20 to 40 degrees would be encountered, considerable attention was given to developing a diving plan. As a consequence, a twenty-two foot houseboat, figure 99, was rented for the diving operations.

Prior to diving, velocity measurements and special pier rigging were conducted. The free stream velocity varied from four and one-half to five feet per second and a high degree of turbulence was created by the pier, figure 100. To aid the diver an anchor was dropped about seventy-five or eighty yards upstream from the pier and the attached rope was secured to the work boat. This line gave the diver a means of securing himself on descent and ascent.

After preparations had been completed, figures 101 and 102, the diver descended through the boundary layer, figure 103. At a depth estimated to be ten feet, the anchor line that he was following caused him to descend into the rollers (horizontal vortices) being shed off the exposed ledge at the front of the pier. These rollers were of such a magnitude that they immediately turned the diver head over heels and slammed him against the pier wall. Fortunately, he was able to maintain his grasp on the anchor line and was able to ascend safely to the work boat.

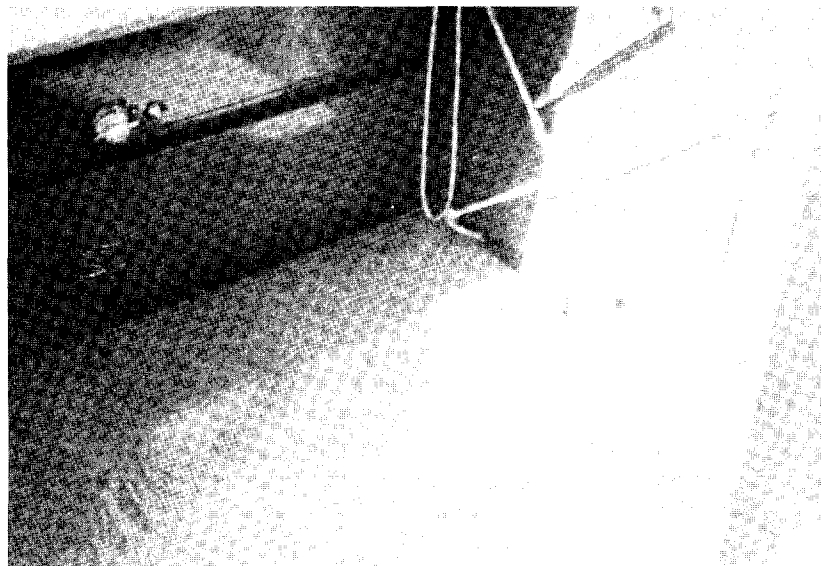


Figure 95. Instrumentation housing with vibration isolator.

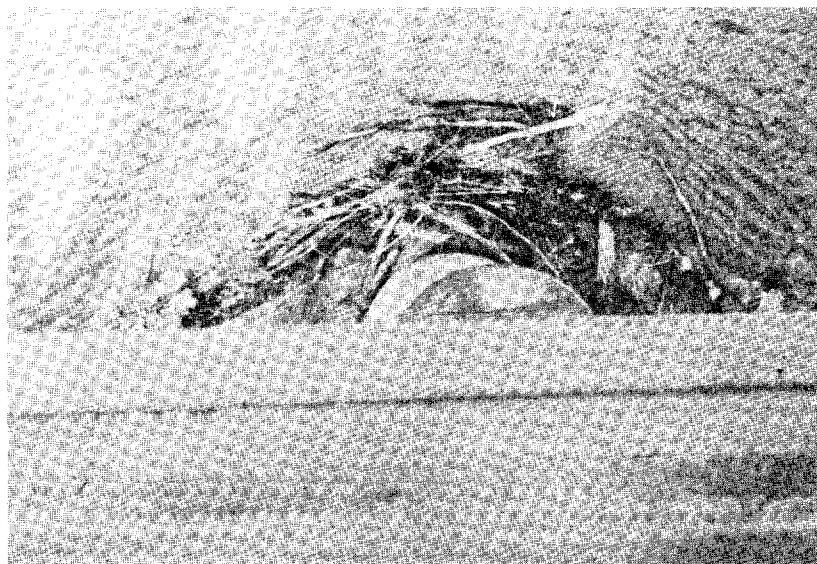


Figure 96. Debris build-up around instrumented Pier 6.

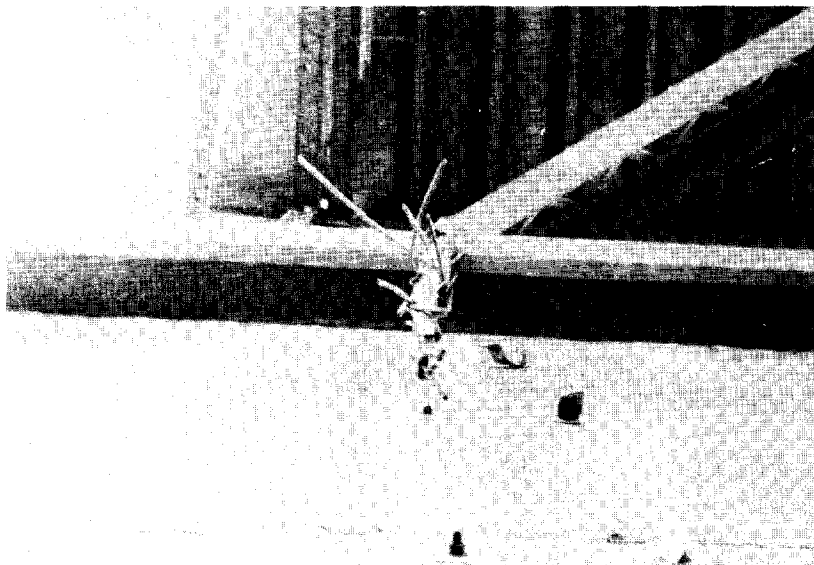


Figure 97. Cable assembly torn from pier by debris.



Figure 98. I-35E Bridge over the Mississippi River at St. Paul, Minn.

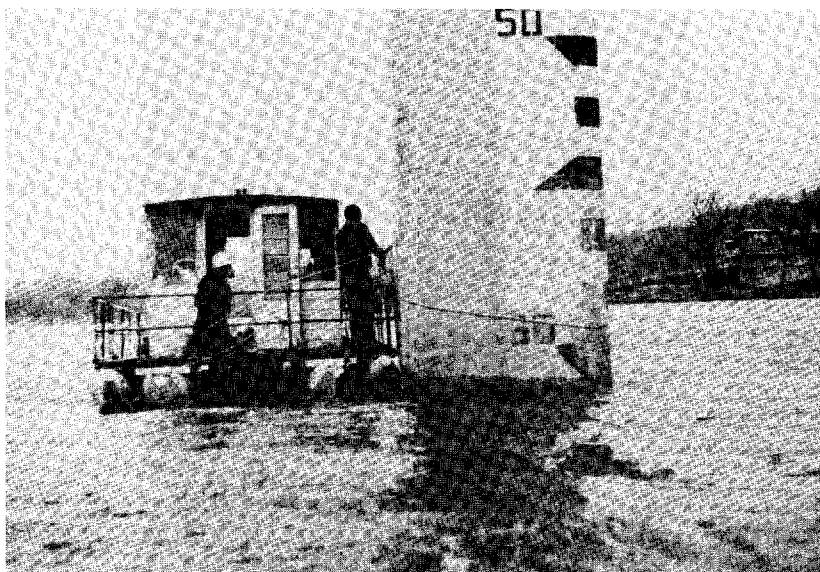


Figure 99, Houseboat used to facilitate diving operations.

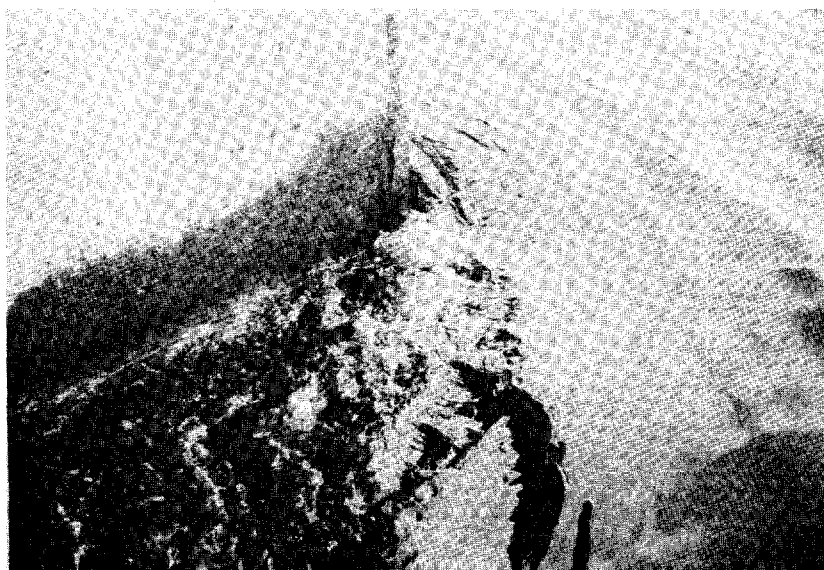


Figure 100, Turbulence created at bow of pier.



Figure 101. Mooring houseboat to pier.



Figure 102. Preparing for diving operations.

It was decided that diving on the sides of the pier was not possible due to the high level of turbulence. It was concluded that even with hard hat diving equipment, working in this very turbulent region of the pier could not be assured. Consequently, a diving attempt was made to put instrumentation on the nose and rear of the pier, where the turbulence was minimal.

A ladder was placed on the nose of the pier and secured above water. The work boat was positioned so that the diver could descend directly down the ladder. Once in the water, the diver quickly determined that even with the help of the ladder, it was not possible to work in these very strong currents. The slightest deviation from the centerline of the pier caused him to be swept around to the side of the pier by the strong currents. This made it necessary for him to hold onto the ladder with one hand at all times. This left only one free hand to work with. Because of the small area in front of the pier where a diver could remain stable, it was impossible for two divers to work in the same area. It was impossible for a diver to hold the cages in this strong current and at the same time use the stud driver to secure them.

After reviewing the situation it was decided that the diving capabilities that existed on the project were not adequate for the job at hand and the decision was made to terminate the installation.

B.3.0 Homochitto River, near Brookhaven, Mississippi

A scour monitoring system was installed on the U. S. Highway 84 Bridge, figure 104, over the Homochitto River near Brookhaven, Mississippi during December 7-15, 1972. The piers on the U. S. 84 Bridge are circular with an eight foot diameter. During high flows there are three piers in the main flow; piers 3, 4 and 5, figure 104. Due to the erratic and fast rise and fall of the Homochitto River, scour could occur at any of the three piers or all of them.

Depth transducers, protected in the four foot long cage supports were attached to the upstream surface of each of the three piers. The staging transducer was attached to the right side of pier 3 in a two foot long cage. Due to the fast rise and fall of the Homochitto River it was not feasible for field personnel to be on site during flooding. Consequently, a velocity measuring system was not included in the installation since it must be manually operated. The installation is shown pictorially in figures 105 through 115. Due to past experience with debris it was realized that the instrumentation could be torn from the piers during flooding. It was therefore decided that an additional securing procedure would be used in conjunction with the stud fastening of the cages by wrapping a steel cable around the pier and around the base plates of the cages, figure 109. Then, in the event the instruments were torn from the piers, they would still be salvageable.

A level indicator switch was placed on the downstream edge of pier 3, figure 110. All cables were also attached to the downstream edge of the

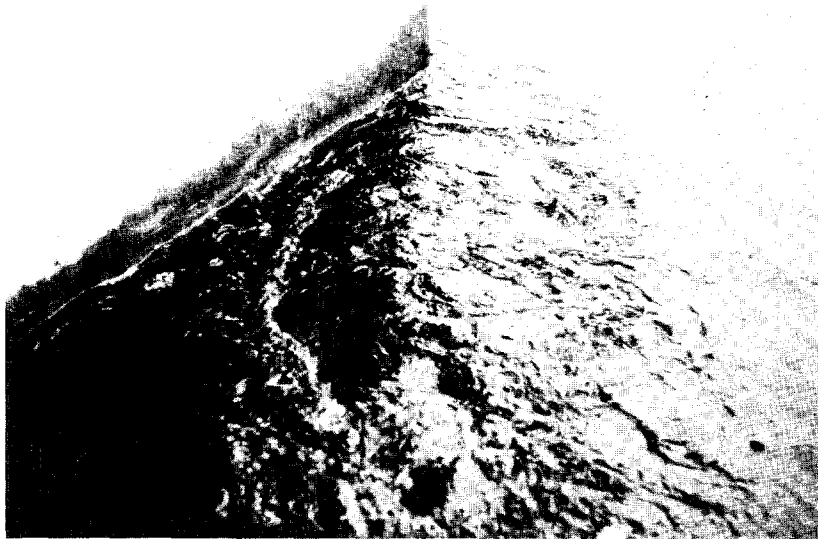


Figure 103. Turbulent area at bow of pier which prevented diver descent.

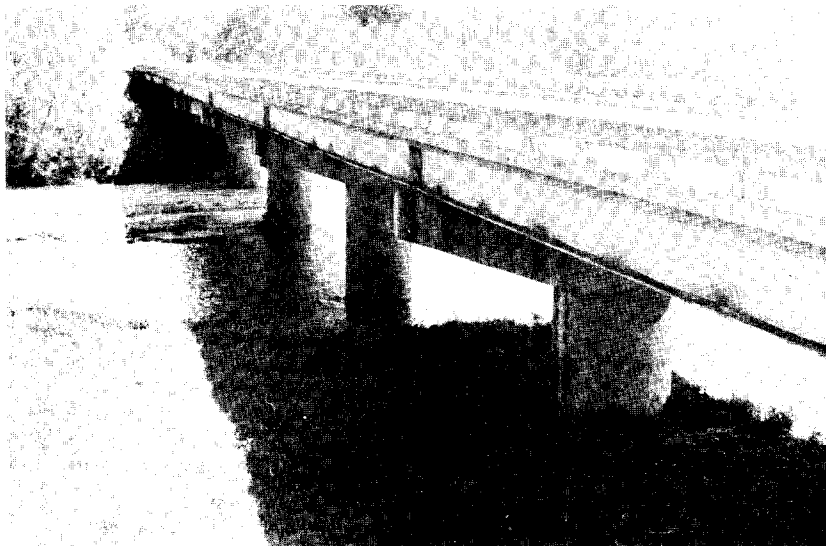


Figure 104. U. S. Highway 84 Bridge over the Homochitto River, showing the instrumented Piers 3, 4 and 5.

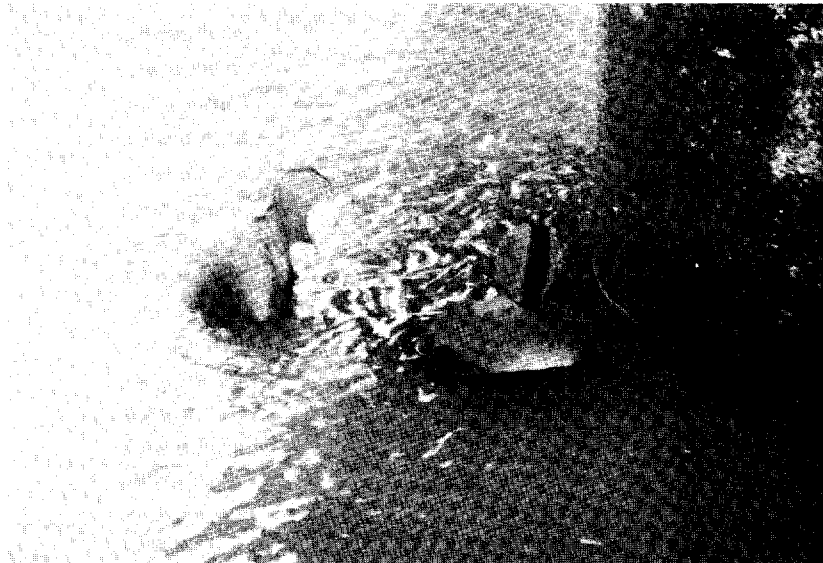


Figure 105, Underwater installation of Pier 3.



Figure 106, Depth transducer support on Pier 3,

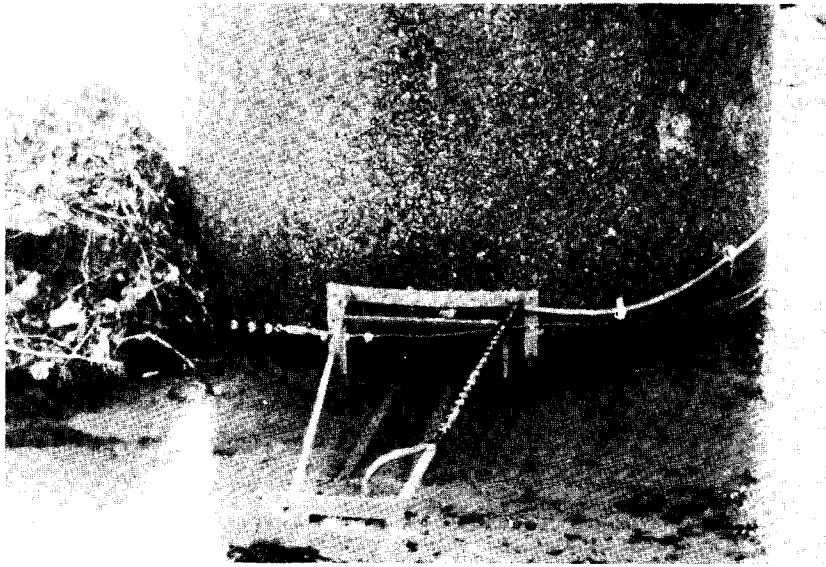


Figure 107. Completed installation of Pier 4.



Figure 108. Installation of instruments on Pier 5.

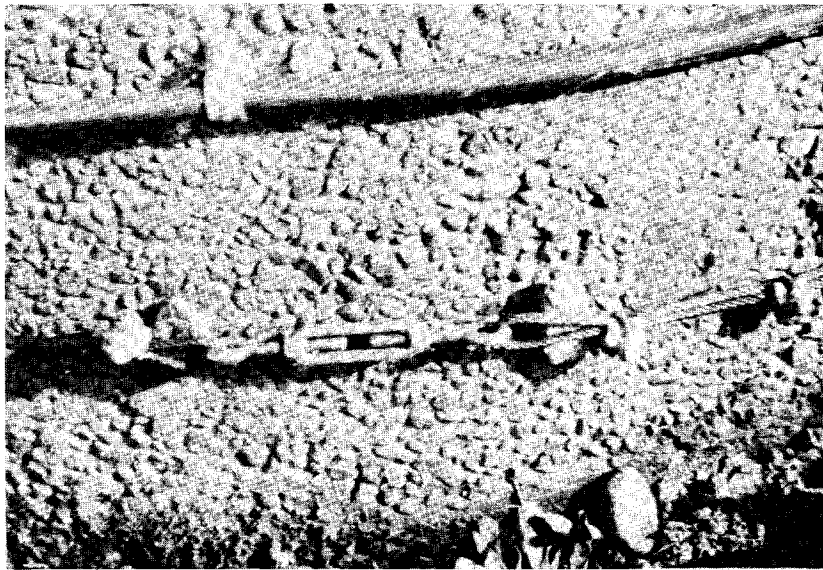


Figure 109. Additional securing of cage supports with steel cable.

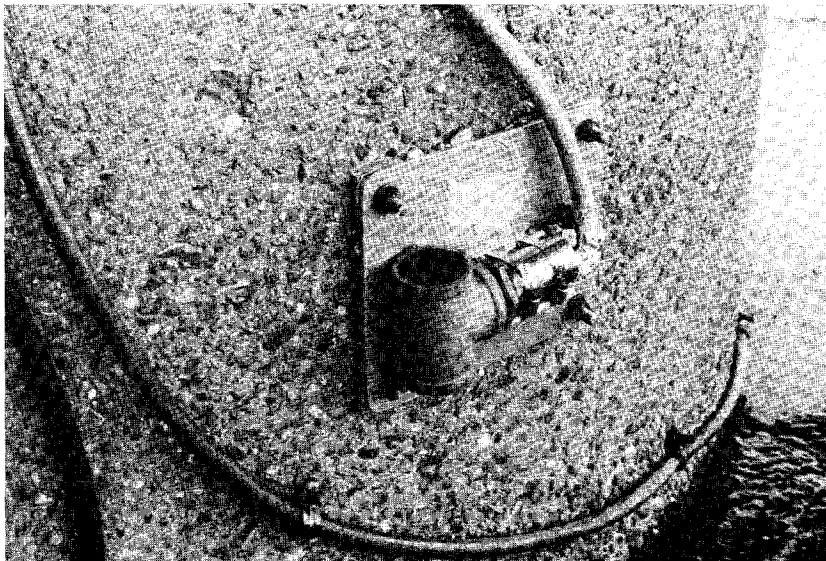


Figure 110. Level Indicator Switch attached to downstream surface of Pier 3.

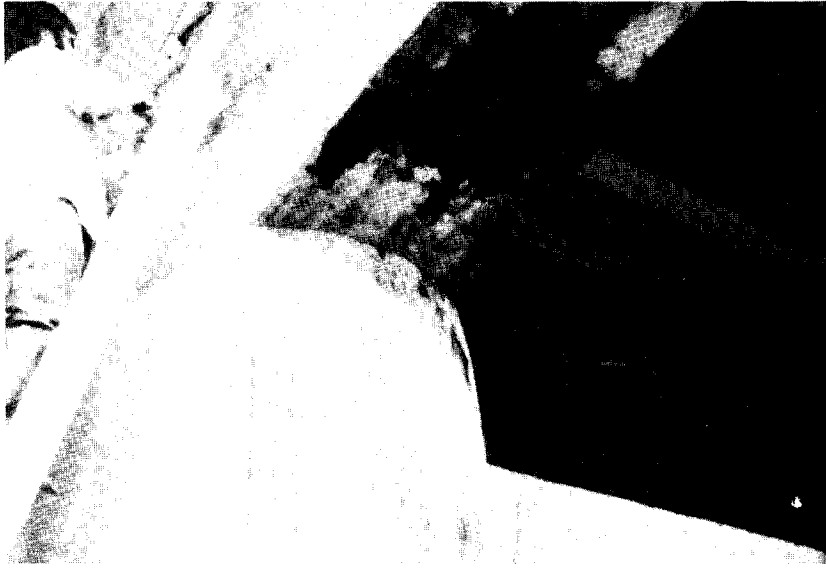


Figure 111. Attachment of cables to the downstream surface of piers.

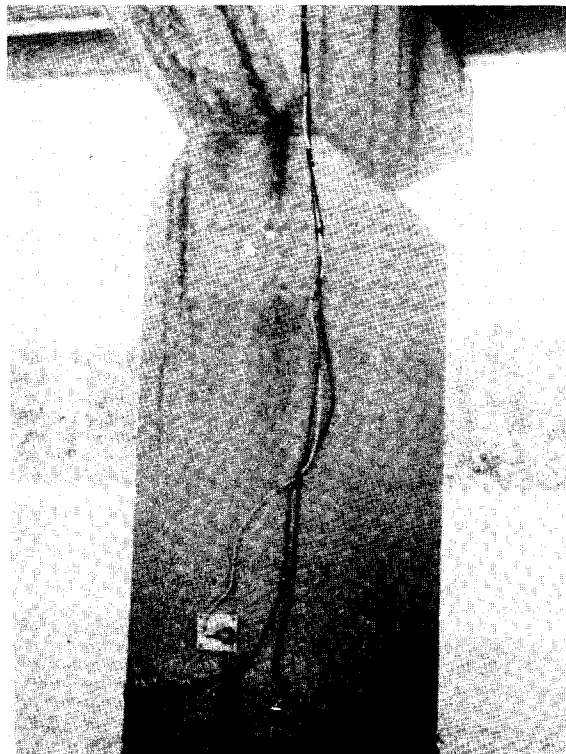


Figure 112. Completed cable installation of Pier 3.

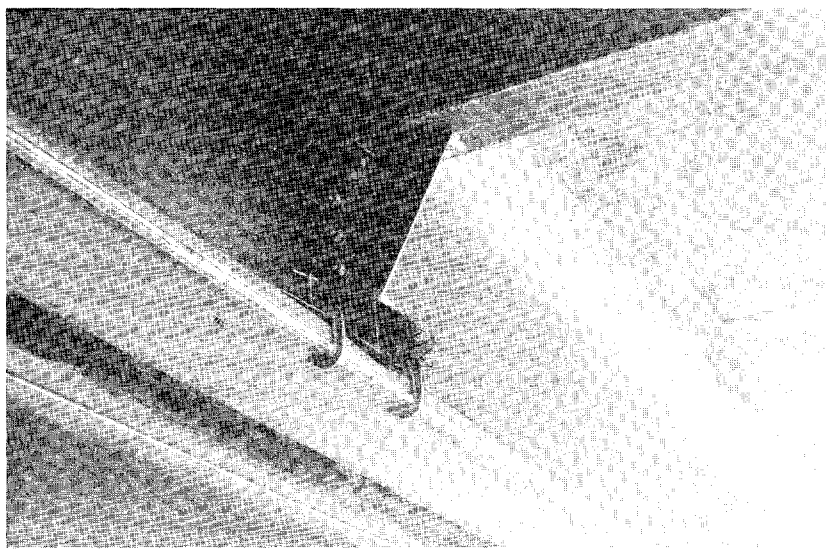


Figure 113. Method of C-clamping cables to bridge span,



Figure 114. Location of instrumentation housing on downstream side of right abutment.

piers, figures 111 and 112. From the pier cap of each pier the cables were run underneath the bridge deck and fastened to the I-beams with C-clamps, figure 113. Cables terminated at the instrumentation housing, located on the downstream side of the right bank abutment, figure 114 and 115.

B.3.1 Flooding of the Homochitto River

In an eighteen month period since the December 1972 installation of the Homochitto River there have been several significant rises in the river stage. Each of these rises have left various amounts of debris around the instrumented piers, figures 116, 117 and 118. Due to the location of the instruments on the upstream surface of the piers, they have not been damaged.

The most recent spring 1974 flood on the Homochitto River resulted in the loss of a pier and span of the U. S. 33 Bridge, located approximately thirty miles downstream from our study site, Figure 119 shows the large amount of debris that was caught by the bridge piers, producing a highly constricted flow.

B.4.0 Brazos River at Richmond, Texas

Installation of a scour monitoring system on the U. S. 59 West Bridge, figure 120, over the Brazos River at Richmond, Texas was completed during May 18 - June 1, 1973. The combined depth-velocity instrumentation packages were attached to the footing of pier 3, figure 121.

The complete installation is shown pictorially in figures 122 through 135. Figures 122 through 125 show the instrument package and its installation. Figures 126 through 135 illustrate the method of cable attachment and location of the instrumentation housing.

B.4.1 Flood Damage to Instrumentation

In the years time since the installation of the Texas site there have been three significant rises of river stage. They were: crest 31.5, June 15, 1973; crest 23.1, October 18, 1973; crest 19.2, January 28, 1974. Normal flow is approximately stage 5. During the October 1973 rise, the right side instrumentation package was torn from the pier by debris. Although the package was damaged, it was recovered and the depth transducer was salvageable.

B.5.0 Ohio River at Lawrenceburg, Indiana

The final bridge chosen for the installation of scour monitoring instrumentation was the I-275 Bridge, figure 136, over the Ohio River near Lawrenceburg, Indiana, between Boone County, Kentucky and Dearborn County, Indiana. The installation was conducted during December 8 - 20, 1973. Pier B, figure 137, located approximately 1,230 feet from the left bank and 340 feet from the right bank during normal pool (stage 25), was the pier instrumented.

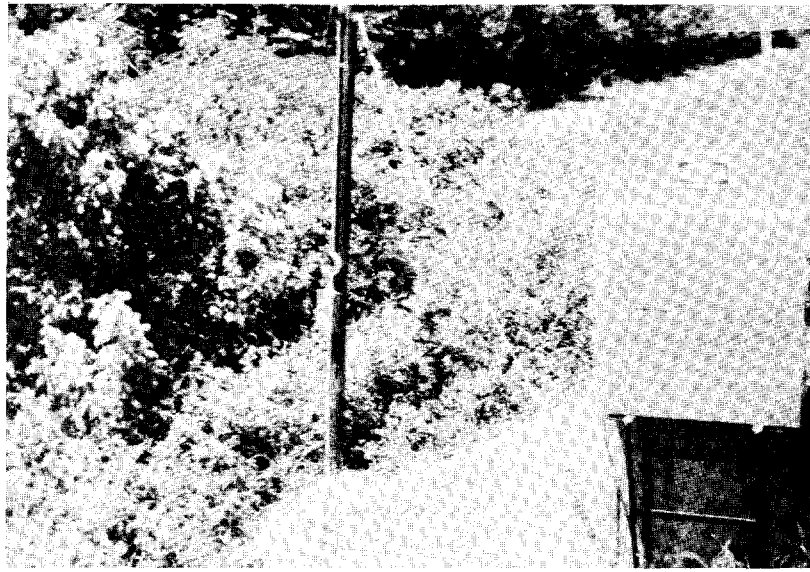


Figure 115. Instrumentation housing braced to wall of abutment (line power to battery charger located at left of picture).

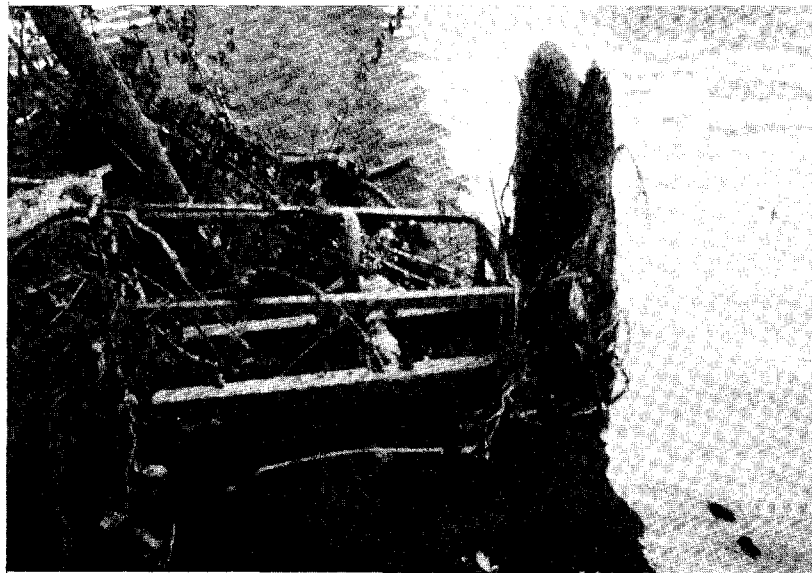


Figure 116. Debris collected on the instrumentation of Pier 4.

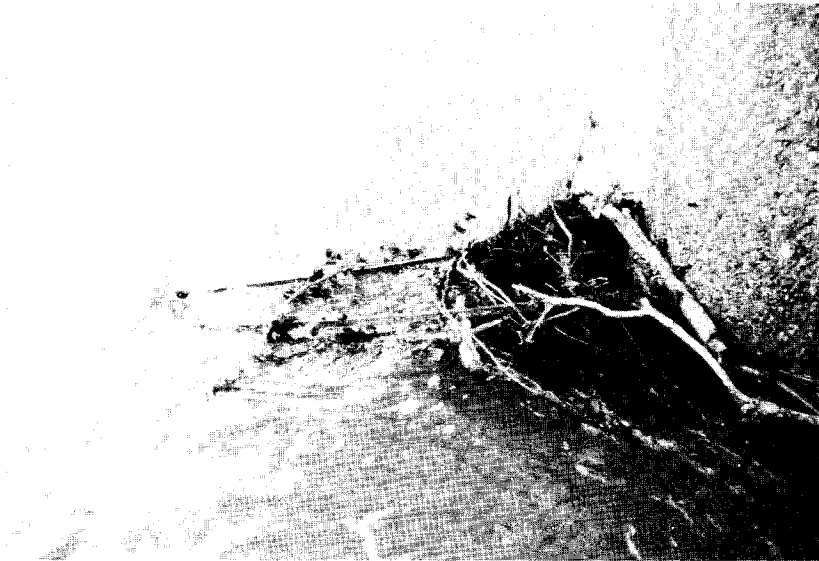


Figure 117. Debris entangled in cage support on Pier 3.

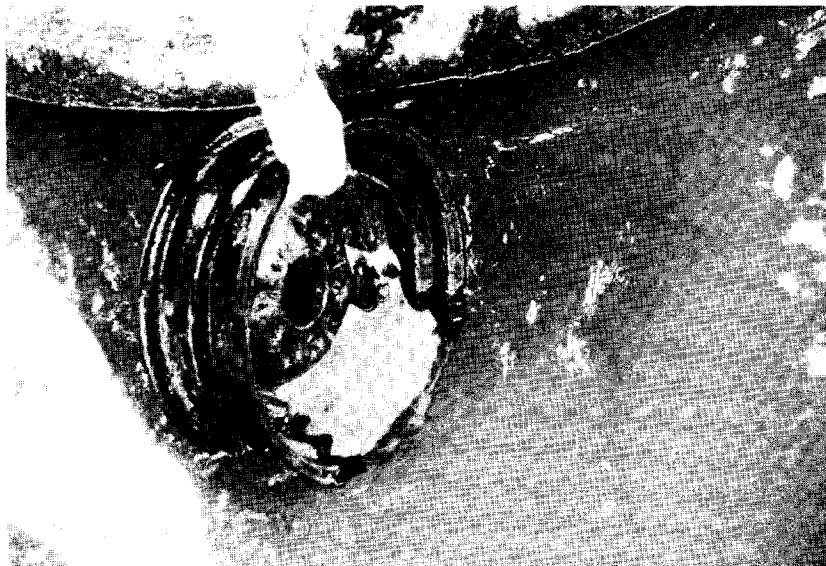


Figure 118. Tire rim removed from the interior of the cage support on Pier 3.



Figure 119. Damaged U. S. Highway Bridge over the Homochitto River at Rosetta, Mississippi after spring 1974 flood.



Figure 120. U. S. Highway 59 West Bridge over the Brazos River at Richmond, Texas.

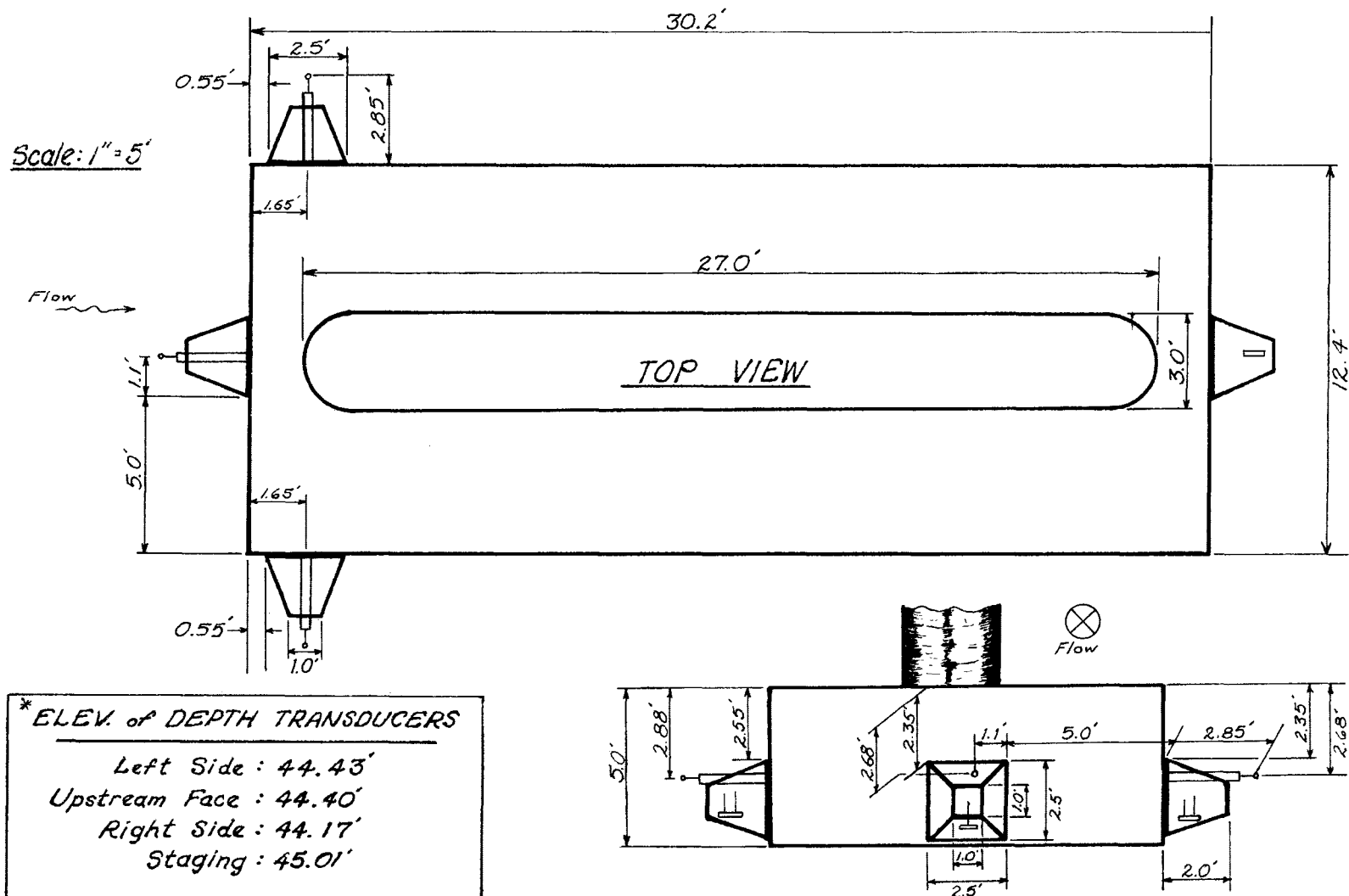


Figure 121. Location of Scour Monitoring Instrumentation Packages on the footing of Pier 3.

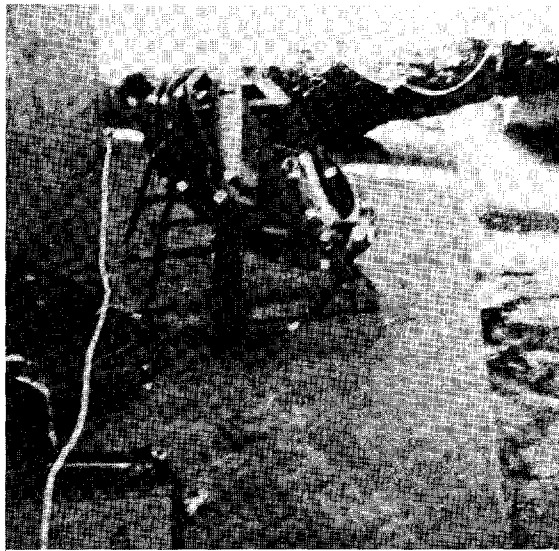


Figure 122. Instrumentation and equipment on pier footing, prior to installation.

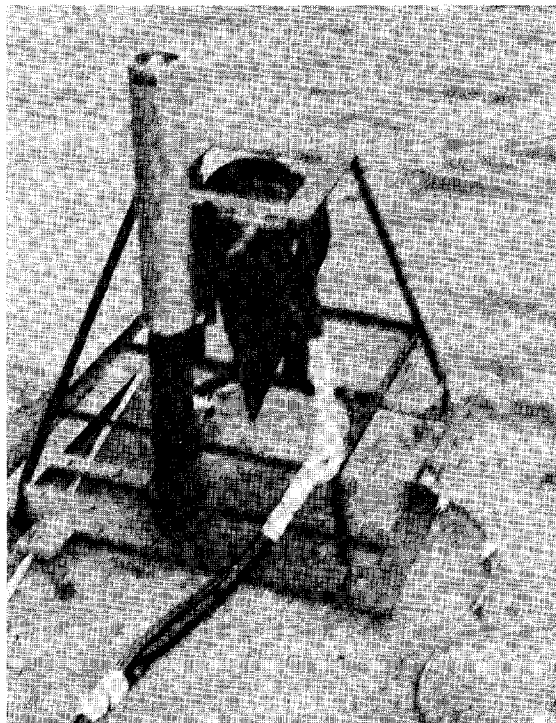


Figure 123. Instrumentation package, incorporating both velocity and depth transducers in one unit.



Figure 124. Lowering of package over the side of the footing.

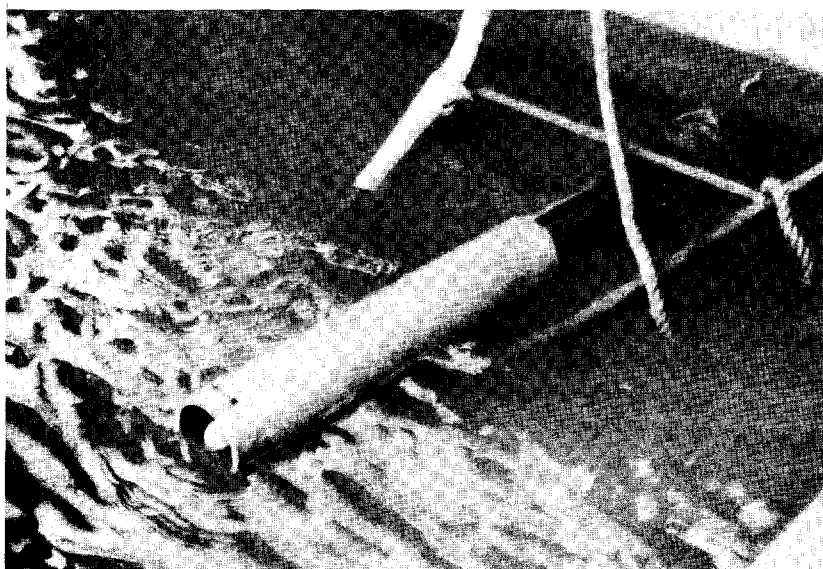


Figure 125. Completed installation of instrumentation package.

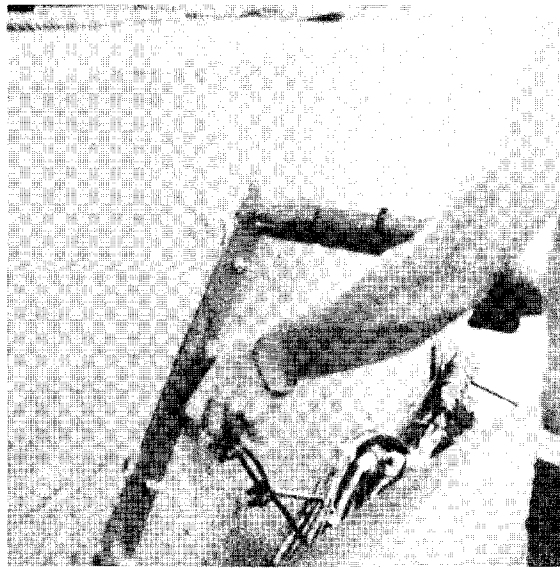


Figure 126. Attachment of safety cable to pier footing.

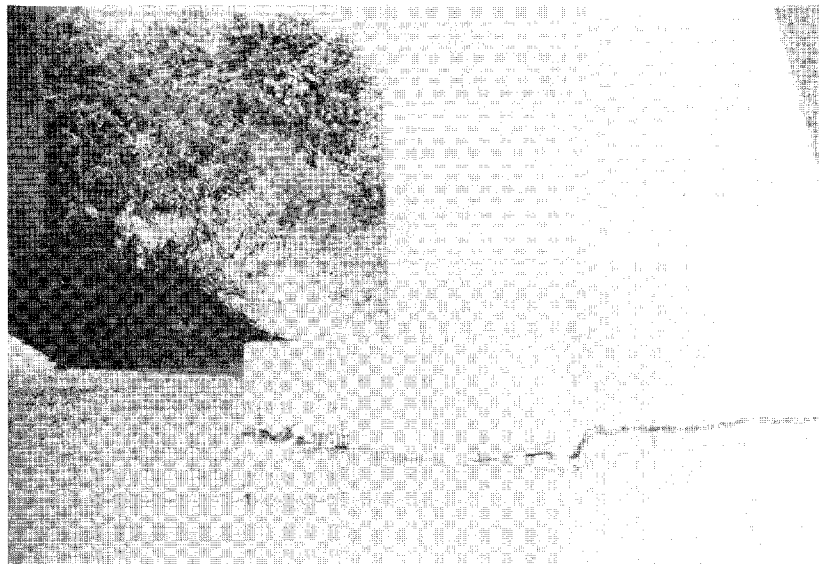


Figure 127. Completed installation of instruments and cables on pier footing.

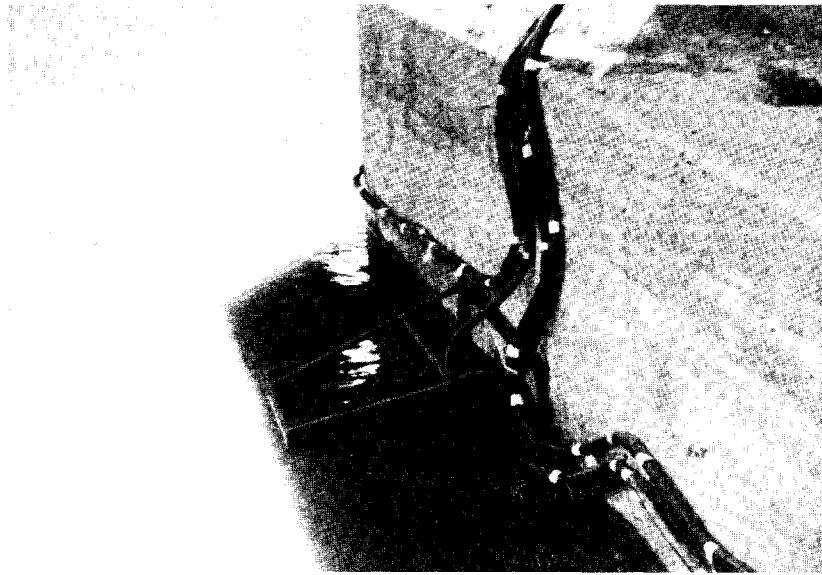


Figure 128. Stage transducer and cables from the depth-velocity packages intersect at rear of pier.



Figure 129. Snooper being used to attach cable to rear of pier.
(Snooper furnished by Texas Highway Department)

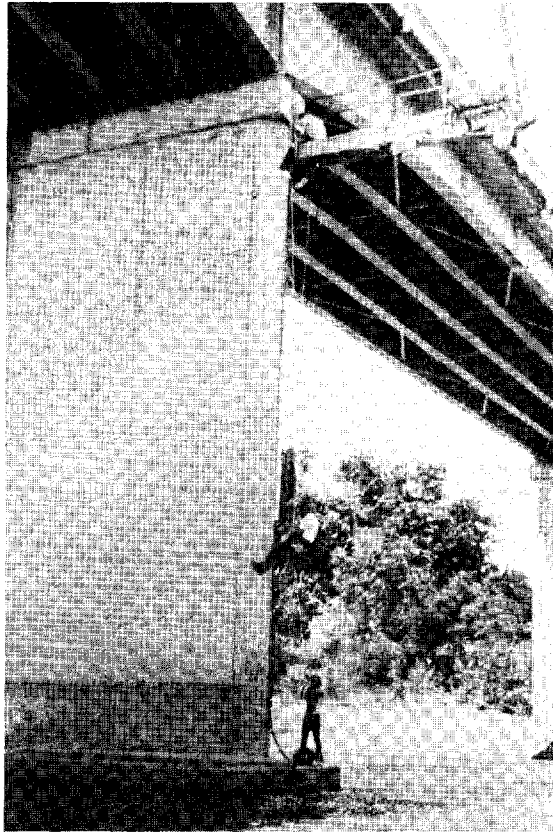


Figure 130. Attachment and covering of cables to pier.

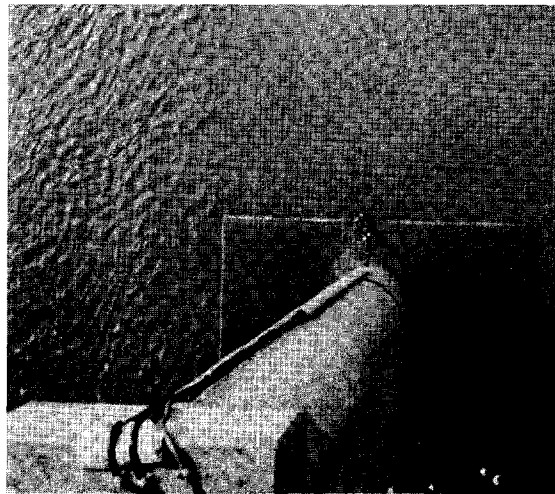


Figure 131. Cable attached to pier.

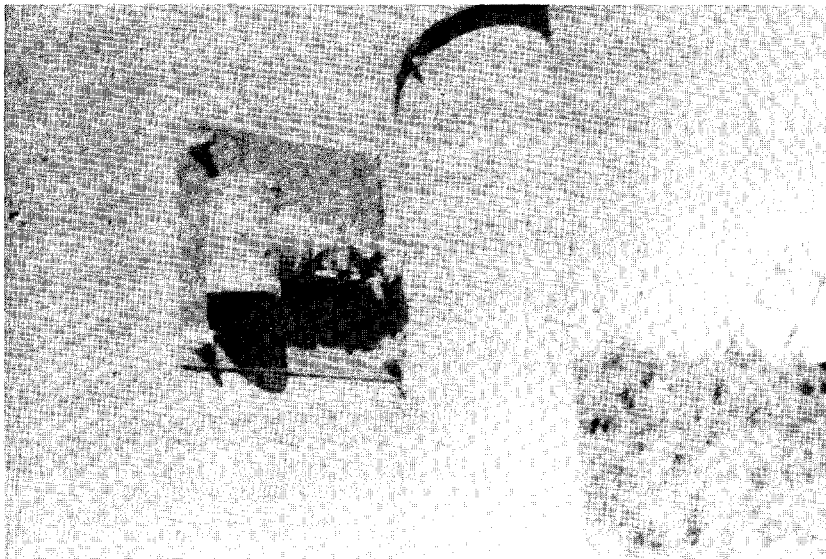


Figure 132. Location of level indicator switch on the right side of pier.

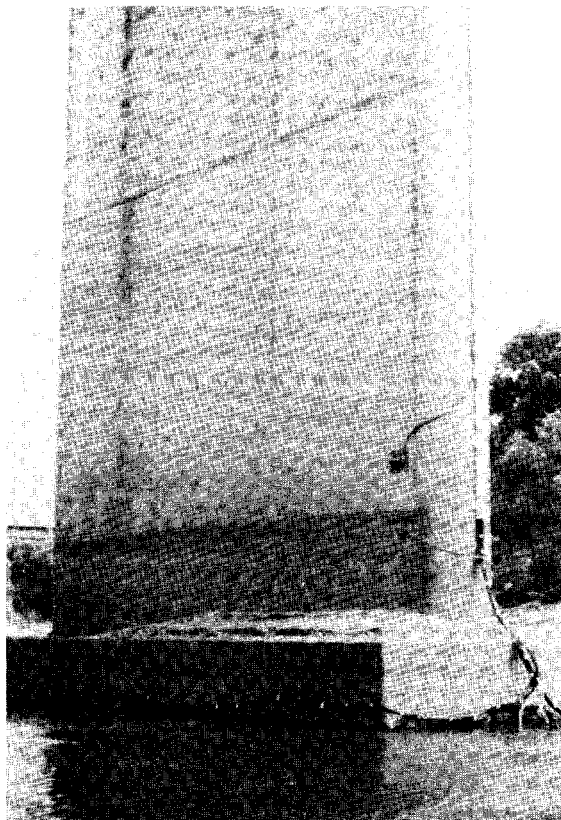


Figure 133. Completed installation of instrumentation system on pier.

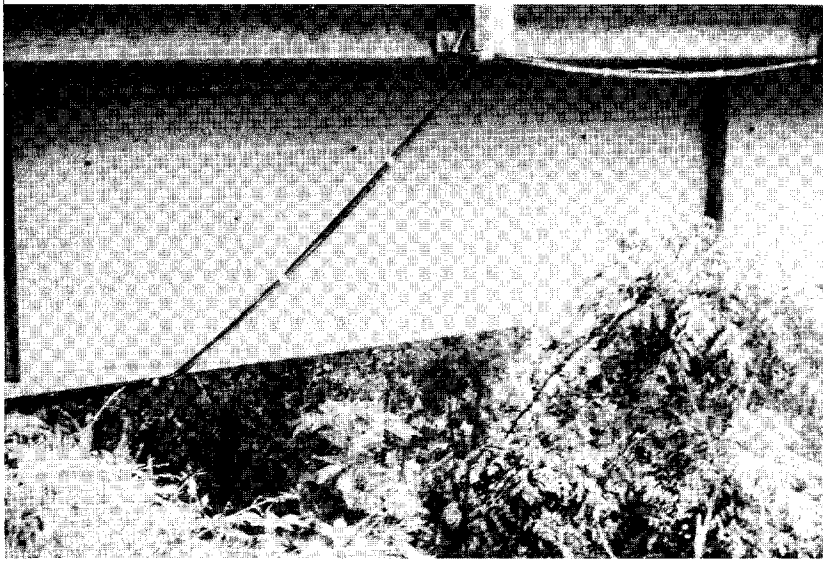


Figure 134. Cable attachment to sidewalk railing across span of bridge.

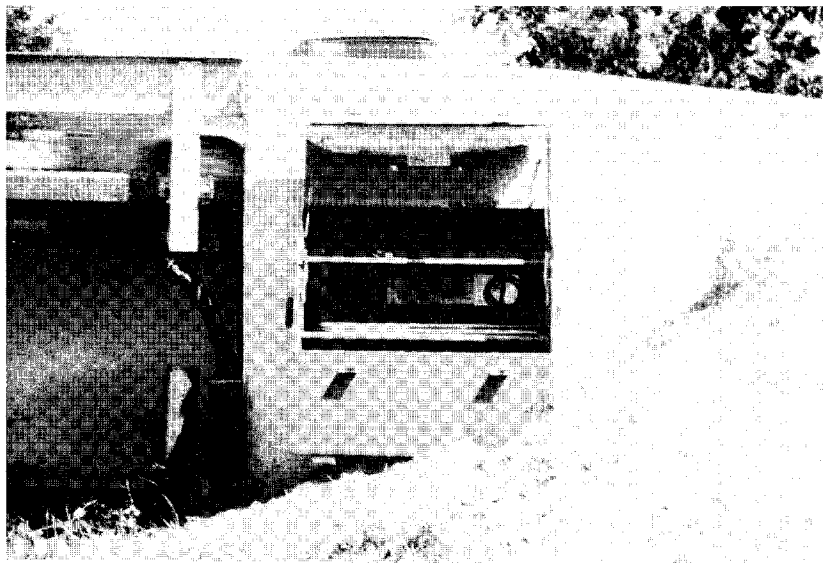


Figure 135. Instrumentation housing located on upstream side of right abutment,



Figure 136. I-275 Bridge over the Ohio River at Lawrenceburg, Indiana.

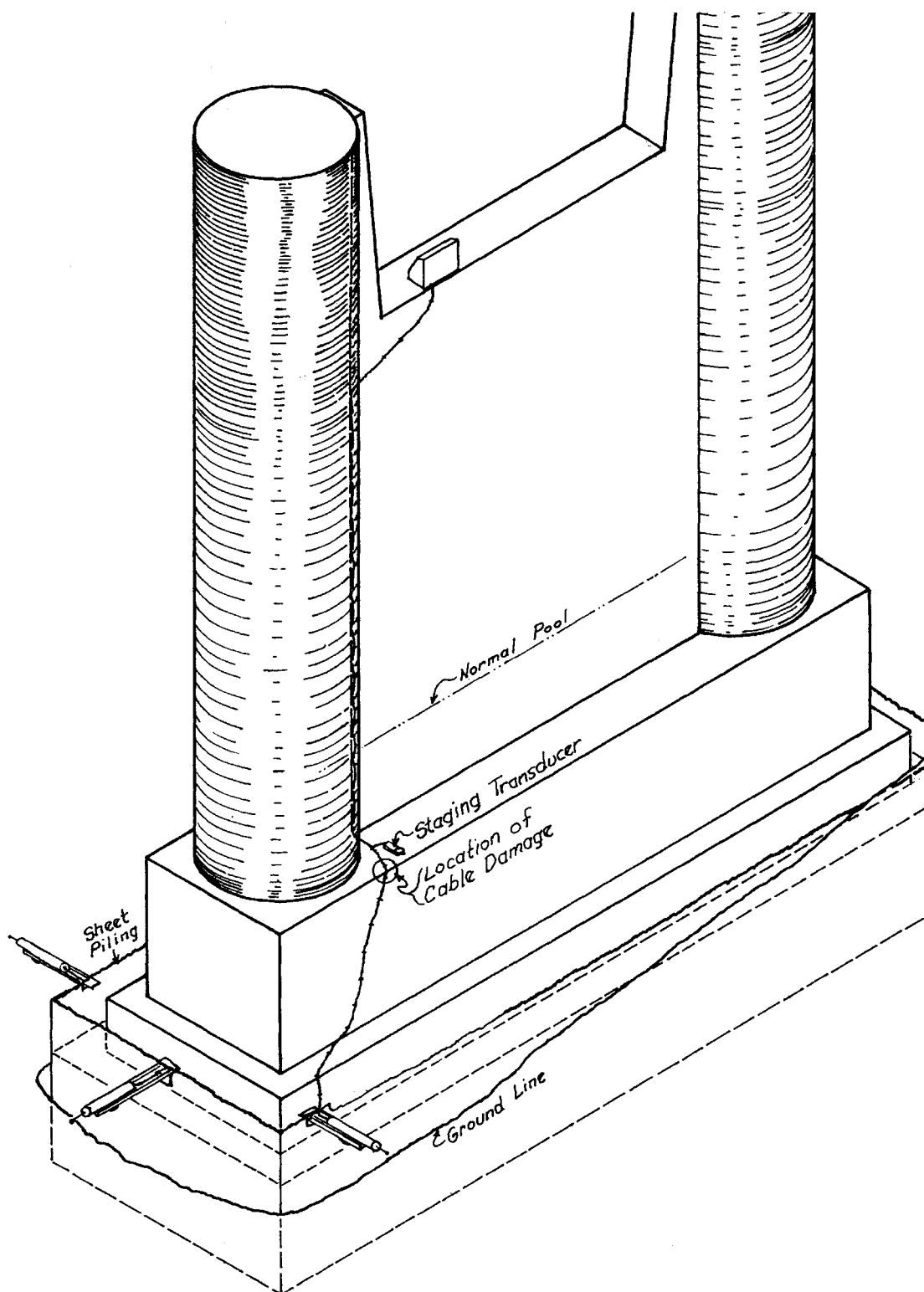


Figure 137. Instrumented Pier B.

Four depth transducers, three velocity transducers and one staging transducer were attached to the pier as shown in figure 137. The instrumentation package located on the nose of the pier had two depth transducers, with both a 3 degree and 15 degree beam width. Due to the fact that sheet piling had been left on the pier foundation after construction, a special clamp had to be designed to attach the instruments to the pier.

The installation is shown pictorially in figures 138 through 147. The instruments and clamps are shown in figure 140 and figure 141, respectively. The instrumentation housing was hoisted to the pier cap, figure 146, and attached there with a safety railing, figure 147.

B.5.1 Flood Damage to Instrumentation Cables

After a January, 1974 rise in stage of the Ohio River it was learned that the transducer cables had been destroyed by debris. The cables had been protected from debris, a distance of at least ten feet below the water surface. However, they were severed and torn from the pier at the attachment point on the top edge of the distribution block, figure 137.

The re-installation of the scour monitoring system on the Ohio River site is scheduled for late August or early September, 1974. This installation will consist of only one instrumentation package, located on the nose of the pier and a staging transducer mounted to the rear of the pier.

B.6.0 Installation Problems

The problems encountered in river diving cannot be over-emphasized. Likewise, not enough can be said about the forces and damage resulting from debris during river flooding. It is only hoped that some of W. Va. University's experiences involving field work may be of benefit to other researchers.

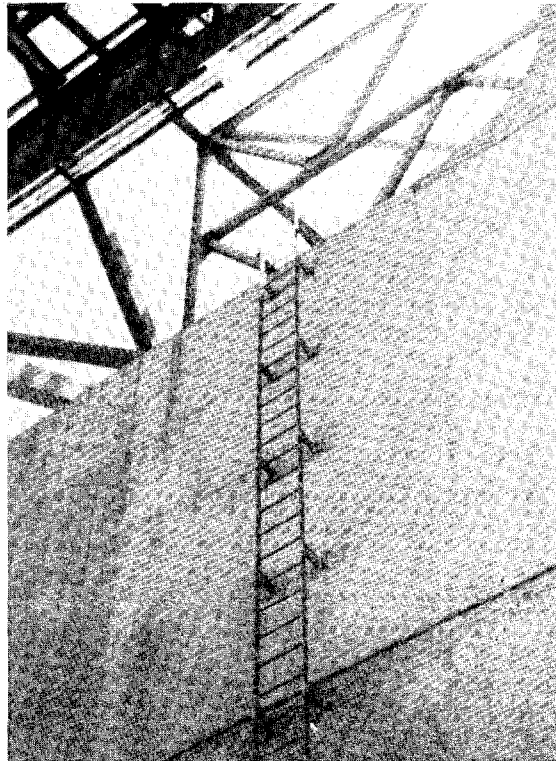


Figure 138. Access ladder to pier cap, Pier B.

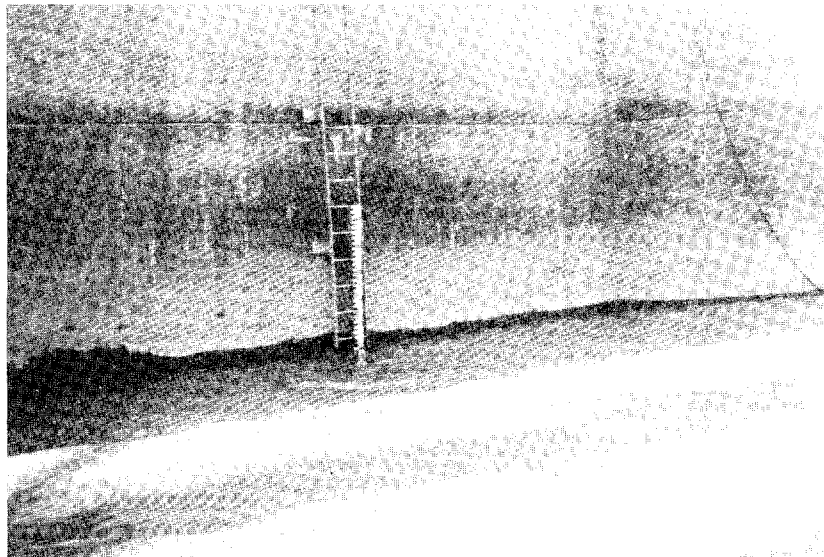


Figure 139. Elevation rod attached to access ladder, Pier B.

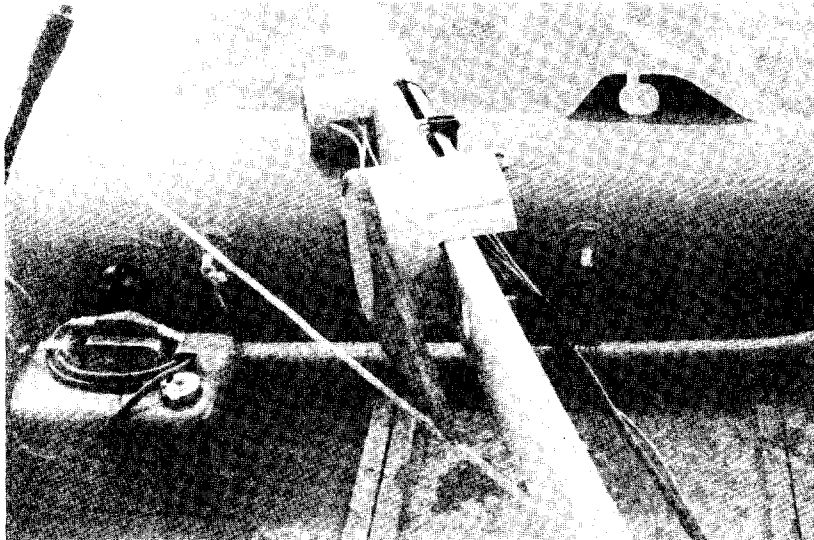


Figure 140. Combined depth-velocity instrumentation package.

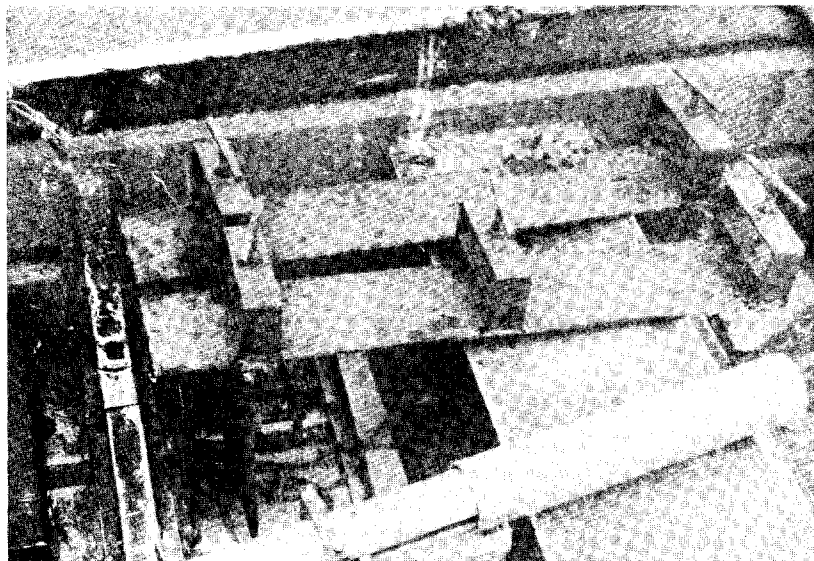


Figure 141. Clamps used to attach instrument package to pier.

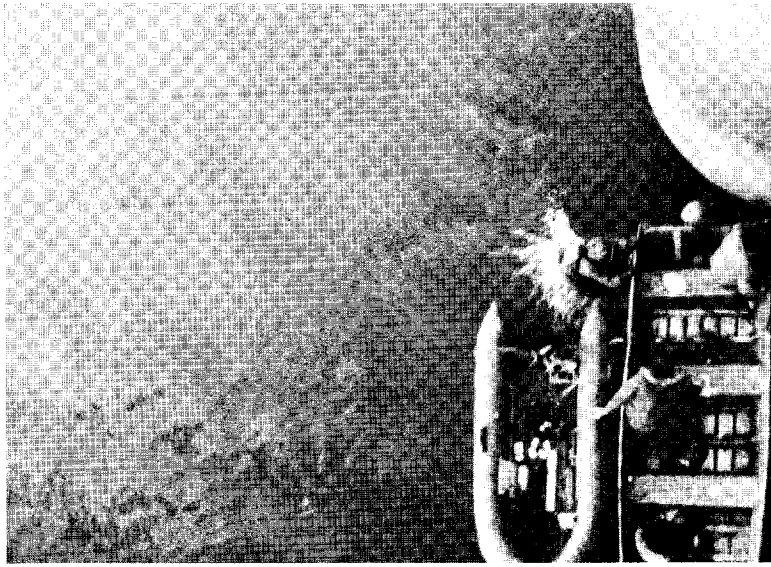


Figure 142. Diving operation.



Figure 143. Diving operation.



Figure 144. Location of cables at corner of pier column and web wall.

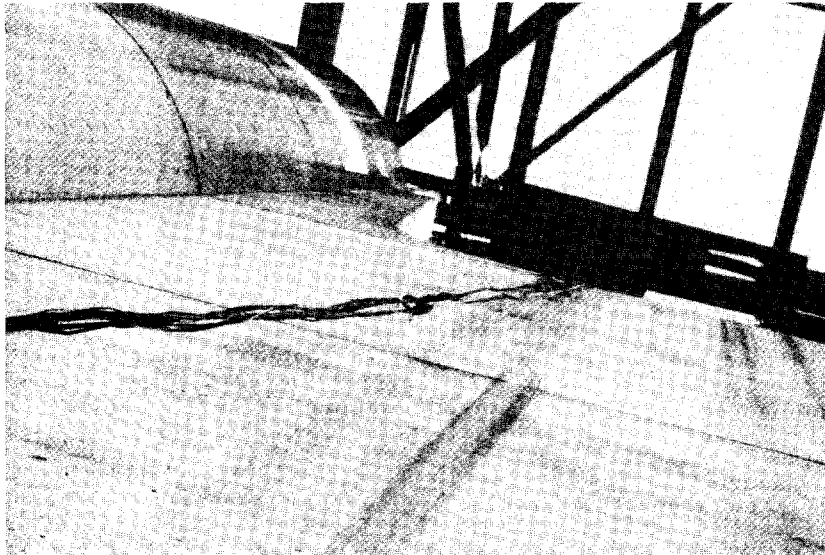


Figure 145. Location of cables at corner of pier column and web wall.

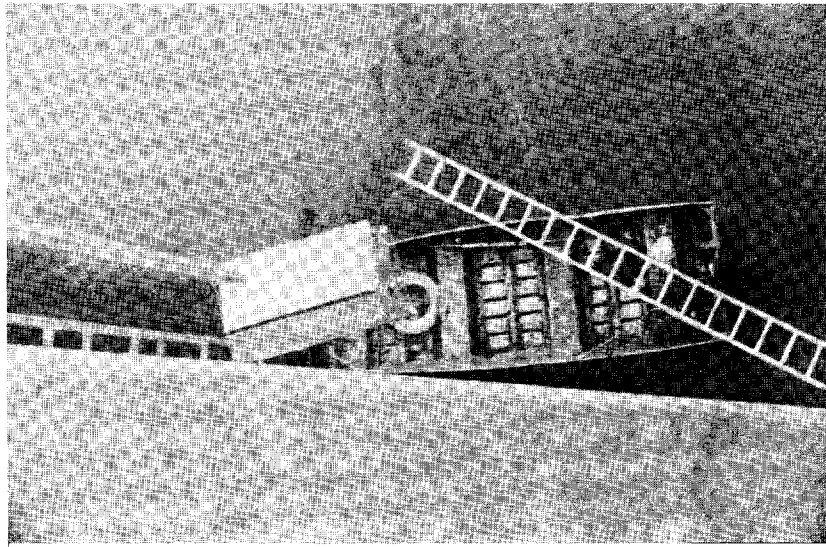


Figure 146. Hoisting instrumentation housing to top of web wall.

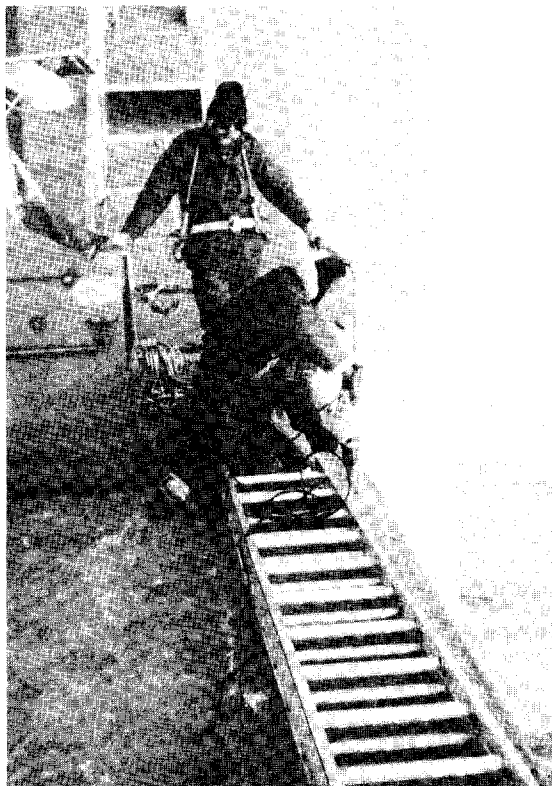


Figure 147. Attachment of instrumentation housing and safety railing to top of web wall,

C. INSTRUMENTATION, OPERATION AND MAINTENANCE MANUAL

C.1.0 Introduction

This manual has been designed to assure the proper operation and maintenance of the "Scour Around Bridge Piers" instrumentation system. It should be read through completely to familiarize the operator with each instrument and its controls. For problems encountered that may not be covered in this manual, contact:

Dr. Gordon R. Hopkins
Project Principal Investigator

or

Mr. Richard W. Vance
Project Field Engineer

Telephone - 304-293-3380 - commercial
304-296-3419 - F.T.S.

Write - B-55 Engineering Sciences Bldg.
West Virginia University
Morgantown, W. Va. 26506

C.2.0 Description and Specifications

The basic scour monitoring system includes both depth and velocity measuring instrumentation. Transducers which are affixed to the pier are monitored by instrumentation located in a 16 gage steel housing. This housing is located either on the pier cap or on the nearest abutment and is equipped with a rain canopy.

Cables from the depth and stage transducers terminate at the instrumentation housing with 3-pin quick-connect plugs. These plugs connect to the Automatic Control Unit which automatically turns the entire depth system on and sequences the readings of the transducers. Output from the control unit is then simultaneously recorded on a Raytheon DE-719 Fathometer recorder and a back-up Rustrak 288 recorder.

Cables from the velocity transducers terminate at the housing with 5-pin plugs. These are two plugs per transducer which connect to two bridge amplifiers. Output from the amplifiers is then recorded on a two-channel recorder. Operation of this system is completely manual and since the cost of automating it would be so great, the amplifiers and recorder are carried to the site when data is to be collected.

C.2.1 Power Supply

A) Rechargeable Batteries - The primary power supply for the depth system consists of three rechargeable batteries. The automatic control

unit requires both 12 and 24 volts for its operation. The two larger 90 amp hr, 12 volt batteries are connected in series to supply this power. In addition to powering the A.C.U. the same 12 volt source supplies power to both the DE-719 and Rustrak recorders.

The smaller 30 amp hr, 12 volt battery is used to supply power to the DE-719 analog circuit board. This additional battery is needed to prevent ground loops between the basic DE-719 recorder and the circuit board.

B) Battery Charger - Located on the back wall, inside the instrumentation housing, the 110 volt A.C. operated battery charger is constantly supplying a .9 amp charge to the two larger 12 volt batteries. The smaller battery, supplying power to the circuit board cannot be charged from the same source, again because of ground loops. However, since the circuit board requires little power for its operation, the smaller 30 amp hr. battery will last for several months under normal 1/36 hr. operation. The battery charger and batteries are protected against over charging by a 5 amp fuse.

C) Service Outlet - A two receptical outlet located beside the charger has been provided for convenience. The outlet, protected by a 15 amp fuse may be used for soldering irons and testing equipment such as oscilloscopes.

C.2.2 Depth System

A) Automatic Control Unit - The control unit, designed and developed at West Virginia University is a completely automated timer and transducer sequencing device. Through a series of relays and stepping switches the unit automatically turns on at preset intervals and supplies power to the remaining depth system instrumentation. After a 30 second warm-up period the unit begins sequencing each of the transducers, letting each transducer record for 30 seconds. The unit has a capability of sequencing four transducers and then automatically turning off all power.

The long life and low power consumption of the clock, relays, and stepping switches make the control unit ideal for use with a battery power supply. All of the component parts along with the operating controls are located inside the unit and are accessible through a hinged door.

Plug receptacles for output from the depth and stage transducers, along with transducer input to the Raytheon recorder are located on the right side of the unit. Power-in and power-out receptacles are located on the left side of the unit along with a receptacle for a level indicator switch.

The level indicator switch affixed to the bridge pier at an elevation equal to flood stage is closed during floods. The closing of the level indicator switch causes an over-riding of the preset turn on

interval and the control unit automatically turns on every hour for more frequent depth recordings.

Specifications for the control unit are:

Dimensions: 15 7/8" long x 13 5/8" wide x 7 3/4" high

Weight: 17 lbs.

Turn-On Intervals: 1/hr., 1/6 hrs., 1/12., or 1/36 hrs.
for 3 minutes

Sequencing Capability: four transducers @ 30 seconds each

Power Requirements: 12 VDC and 24 VDC

B) Raytheon DE-719 Fathometer Depth Recorder - The DE-719 is a complete echo depth sounder designed to provide a detailed permanent recording of underwater topography in water depths between 2 and 410 feet.

The DE-719 is advance design equipment using completely solid-state circuitry, magnetic keying and electronically controlled chart speed. The equipment is housed in a splash-proof, aluminum case to assure maximum protection when operated under adverse conditions. The viewing window in the front cover is hinged to permit access to the chart paper for annotations and for adjustments to the Tide and Draft control, the Speed of Sound control and the X2 control (Doubles all Range Scales). A safety guard covers the stylus and belt assembly to prevent accidental contact with the rotating stylus arm while entering notes and data on the chart paper.

The DE-719 will provide high resolution recordings due to the narrow transducer beam width, high sounding rate, fast chart speed and high signal frequency. The versatility of the DE-719 is further enhanced by a Tide and Draft control, a Speed of Sound control and a Range doubling control. Any changes in the setting of the preceding controls is permanently recorded on the chart paper for future reference.

The operating controls such as the Power ON-OFF, Range Selector, Sensitivity, Chart Advance Speed and the Fix Mark control are located on the front panel. The auxiliary controls such as the Tide and Draft control, the Speed of Sound control and the Range X2 control are located inside and are accessible through the hinged front window.

The transducers are two-way energy conversion devices which function similar to the combination microphone/speaker in an intercom unit. During transmission, it converts pulses of electrical energy into pulses of supersonic energy which travel through the water toward the bottom. During reception it receives the echoes of supersonic energy which are reflected from the bottom and converts them to electrical signals.

Specifications for the DE-719 and transducers are:

<u>Dimensions</u>	<u>Recorder</u>	<u>Transducer</u>	
		<u>2445 AD (15°)</u>	<u>7245 (3°)</u>
Height	18 inches	5 5/8 inches	5 3/16 inches
Width	15 3/8 inches	2 3/4 inches	7 5/16 inches
Depth	9 1/16 inches	4 1/4 inches	7 5/16 inches

Net Weight

Recorder-with transducer and rigging	47 pounds
Recorder-only	38 pounds
Transducer-only	3 pounds

Depth Range: 0 to 205 feet in four phases

Range 1: 0-55 feet	(0-16.5 meters)
Range 2: 50-105 feet	(15-31.5 meters)
Range 3: 100-155 feet	(30-41.5 meters)
Range 4: 150-205 feet	(45-61.5 meters)

The X2 range control doubles all range scales

Accuracy:	0.5% $\pm$ in. of indicated depth
Soundings Per Minute:	534 soundings per minute
Chart Paper Speed:	1,2,3 and 4 inches per minute
Chart Paper Size:	7 inches wide by 60 feet long
Operating Frequency:	200 kHz
Transducer:	Barium Titanate-Type 2445AD (Type 7245 optional)
Transducer Beam Width:	Type 2445AD 8° at the half power points Type 7245 2.5° at the -3db points 3.5° at the -6db points 4° at the -10db points

Power Requirements: 2.5 amps @ 12 volts DC-30 watts

C) Rustrak 288 Recorder - The Rustrak recorder prints through the impinging action of its stylus, driven by the chopper bar against pressure-sensitive chart paper. Its presentation is a series of data appearing as a continuous line.

Recording on the chord of the stylus arc by the edge of the chopper bar is possible with a stylus able to write along its length rather than its point. This results in chart paper printed with straight lines and rectilinear recordings.

Writing speed varies with motor speed. Chart speed and trace density depend on the ratio of the interchangeable gear box which couples the paper drive to the motor.

The necessary controls are located on the front of the recorder. A mechanical zero adjust is located under the nameplate and chart paper may be advanced with a thumb-wheel. A sliding front window may be opened for annotations on the chart paper.

Specifications for the Rustrak are:

Dimensions: 5 5/8" high x 3 5/8" wide x 4 5/16" deep

Range: 0-5 V full scale

Writing Speed: 1 strike per second

Chart Paper Speed: 60 inches per hour

Chart Paper Size: 2 9/16" wide x 60' long

Power Requirements: amps @ 12 volts DC

C.2.3 Velocity System

A) Air Control - The air control system is used to extend and retract the velocity meter protecting cover. Two air hoses, per velocity meter, terminate at the instrumentation housing with quick-connect fittings. Both air hoses from any velocity meter are plugged into the air control unit.

The air control unit consists of a system of three-way valves, pressure reduction valves, and manifolds. Air at 1600 p.s.i. from an air cylinder (vol. = 26 cu. ft.) is reduced to 60 p.s.i. in the air control unit and directed to either the extend or retract air hose.

Specifications for the air control system are:

Poly. - Flo (Polyethylene) Tubing:

Tube O.D.: 1/4"

Burst Pressure: 400 p.s.i.

Kwik-Connect Couplings & Fittings

Recommend man. working pressure: 100 p.s.i.

Plug-Type Shut-Off Valves (3-way)

Working pressure: 100 p.s.i.

First Pressure Reduction Valve

Max. Working Pressure: 3000 p.s.i.

Max. Pressure @ Outlet: 120 p.s.i.

Second Pressure Reduction Valve

Max. Working Pressure @ Inlet: 160 p.s.i.

Pressure @ Outlet: variable 0-120 p.s.i.

B) BA-4 Bridge Amplifiers - The Model BA-4 Bridge Amplifier made by Vishay Instruments, Inc. is used for resistance type strain gages and transducers. It is battery operated and completely contained in an individual case. The BA-4 includes bridge power, balance, calibration, dc amplifier and drives all galvos and oscilloscopes.

Specifications for the BA-4 are:

Input: 2 or 4 arm bridges.
50 to 2,000 ohm arms.
5 microvolts min. discernible signal.
+ 15 millivolts max. signal with GAIN at min.
 $\pm 1/2$ millivolt max. signal with GAIN at max.
max. sensitivity - 200 microin. strain on single active gage gives full output.
min. sensitivity - 20,000 microin. strain on single active gage for full output. Easily reduced further by external resistance in gage circuit.
Electrical noise pickup reduced by differential amplifier.
Use shielded cables; max. length 100 ft.
Input and output must not be cross-connected.
Float gage circuit free of ground.
Ground shields and center tap of output voltage divider.
Amplifier input impedance 30,000 ohms.

Output: Differential
+ 5 volts into 1,000 ohms or more.
 ± 50 milliamps into 50 ohms.
Drives galvos and other ungrounded devices.
Differential output impedance less than 1 ohm.
- Single sides from center tapped 2,000 ohm voltage divider across differential output.
 $\pm 2 1/2$ volts into 100,000 ohms or more.
When full differential load is simultaneously applied allowable single-sided output drops to ± 1 volt.
Drives oscilloscopes, tape recorders and other grounded high impedance devices.
Single-sided output impedance 500 ohms.

Bridge Power: 8 volts dc.
Regulated within 2% for life of batteries.
Bridge current adjustable by potentiometer, connected in series with bridge.
Current in arms indicated on meter.

Calibration: By resistor connected in parallel with either of two adjacent bridge arms.
 7 one percent resistors 10,000 to 1,000,000 ohms.
 3 ten percent resistors 2,000,000 to 10,000,000 ohms.

Frequency: 0-10,000 cps within 5%.

Temperature: 30-100°F.

Linearity: Within 2% at max. signal, max. load. Normally within 1/2%.

Drift: Within 10 microvolts after 1/2 hour operation at steady temperature. (Unit 5 hrs. at steady temperature.)
 Within 1 microvolt /°F temperature variation.

Noise: 5 microvolts peak to peak equiv. input - 500 ohms input.

Gain: 200, 400, 800, 1,600, 3,200, 6,400 voltage gain.
 Half these at single-sided output.
 Accuracy within 2%.

Meter: Checks batteries.
 Indicates balance.
 Indicates current in gage arms.

Batteries: Two 7 1/2 volt Eveready 707 or equiv. in series for bridge.
 Two of same in series for amplifier.
 Life approximately 100 hours.

Case: 11 1/4 x 7 x 2 3/4.

Weight: 9 lbs.

C) Brush 222 Recorder - The solid state Brush 222 is an internally powered, accurate, rugged, lightweight, two channel recorder for use anywhere. Traces are crisp, clear and smudge-free produced by a pressurized ink system. A disposable ink cartridge holds up to a one year supply of ink and can be cleanly replaced in minutes. Whenever the power is off or low, or the chart paper runs out, the writing system stops and ink is withdrawn from the pen tip.

This completely self-contained portable recorder has two 40mm analog channels and two event markers. Other features include built-in preamplifiers, electronic limiters, a measurement range from one millivolt/division to 500 volts full scale, linearity of 99.5% enforced by the exclusive Brush frictionless pen position servo system, frequency response

above 30 Hz full scale, and six chart speeds. An external d-c chart drive mode permits variable chart speeds over the full range from 1mm/sec to 50mm/sec.

Both inputs are isolated from each other, from chassis and from the output, the 222 can be used with all types of signal sources--grounded, floating, or driven off-ground--without affecting accuracy or creating system noise.

The internal battery supply, consisting of two sealed batteries, has a total operating life of up to 6,000 hours when recycled by the recorder's built-in charger. The charger is activated simply by plugging its power input module into an external source of a-c or d-c (depending upon the model selected). The batteries allow continuous operation for up to 12 hours and can be completely recharged in 16 hours. To avoid recording of inaccurate data an automatic ink shut-off is actuated if the battery voltage falls below a predetermined level.

"Line assisted" operation allows unrestricted recording time. This is accomplished by powering the charger from an external source at all times, so as to keep a full charge on the batteries.

Specifications for the Brush 222 are:

Number of Channels:	2 Analog, 2 Event.
Channel Width:	40mm, 50 divisions per channel.
Trace Presentation:	Rectilinear.
Trace Width:	0.01 inch nominal.
Writing Method:	Pressurized fluid (U. S. Patent 3,054,109).
Writing Fluid Capacity:	1 oz.; sufficient for one year of normal recording.
Chart Description:	4.3 inches wide--two 40mm grids graduated in 50 divisions across span. Graduated 1mm increments on time axis. Right and left event marker channels.
Chart Capacity:	275 feet - high contrast. 400 feet - reproducible.
Chart Speeds:	1,2,5,10,25,50 millimeters per second and STOP.
Chart Speed Accuracy:	1%

Operating Temperature: 0°C to +37°C. Recorder will remain within specification over range of $\pm 10^\circ\text{C}$ from temperature at calibration.

Storage Temperature: -40°C to +37°C.

Principle Dimensions: 9.12" wide x 13.06" high x 9.37" deep.

Weight: 26 pounds.

Sensitivity: 1 mv/div to 10 volts/div.

Measurement Range: 1 millivolt per chart division to 500 volts d-c full scale (50 chart div.)

Input Circuit: Differential, balanced-to-guard.

Input Impedance: 10 megohms balanced, 5 megohms each terminal to guard.

Signal Input:

In-Phase Rejection: 80 dB at 60 Hz with maximum source unbalance of ten kilohm.

Maximum Signal Input: 500 volts d-c, or peak-to-peak (either input to case ground)

Maximum Common Mode Voltage: 5,000 times the VOLTS/DIV sensitivity up to a maximum of 500 volts.

ATTENUATOR SETTING	MAX. COMMON-MODE VOLTAGE
1 mV/div	5V d-c
2 mV/div	10V d-c
5 mV/div	25V d-c
10 mV/div	50V d-c
20 mV/div	100V d-c
all others	500V d-c

Non-Linearity: 0.5% full scale

Frequency Response: 50 div $\pm$ div d-c to 34 Hz
 1- div $\pm$ div d-c to 10 Hz
 3 dB down at 100 Hz

Stability after 10 second warm-up and excluding paperdrift)

(Zero with respect to)

Time: ± 0.1 div for 8 hours
Temperature: ± 0.1 div per $^{\circ}\text{C}$, $25^{\circ}\text{C} \pm 10^{\circ}\text{C}$

(Gain with respect to)

Time: $\pm 0.1\%$ per 8 hours
Temperature: $\pm 0.05\%$ per $^{\circ}\text{C}$, $25^{\circ}\text{C} \pm 10^{\circ}\text{C}$
Power Input (a-c): 115 volts $\pm 10\%$, 50-400 Hz, 41 VA

C.3.0 Operation and Maintenance

Specific instructions for the proper operation and maintenance of each instrument will be given in the following sections. Once these have been read and understood the step-by-step check list at the end of the manual should be all that is needed to properly service the entire system.

C.3.1 Power Supply

Important: Before any work is done on the power supply, remove the fuses from the fuse box located on the back wall inside the instrumentation housing. The 110 volts a.c. operating the battery charger and the 90 amp hour 24 volts d.c. operating the depth system can, of course, be dangerous if handled improperly.

The power supply is either on or off and has only one, but very important, maintenance requirement. That is the battery water level must be maintained.

C.3.2 Depth System

A) Automatic Control Unit

Operator Controls and Function

- a) ON/OFF: Applies power to clock, relays and stepping switches. NOTE: If power is being delivered to the A.C.U. after a long off period, it must be done gradually to prevent overwinding of the clock. In other words, if the system has been without power long enough (in excess of one week) the control unit clock will completely unwind. After a new power supply is connected to the A.C.U., gradual winding of the clock may be accomplished by turning the ON/OFF switch on and off in one second intervals.

- b) ONCE PER HR/INTERVAL PER HR; Allow the control unit to deliver power to the depth system every hour or at preset intervals using the INTERVAL switch.
- c) INTERVAL: Selects the depth recording intervals; 1/4 hrs, 1/6 hrs, 1/12 hrs or 1/36 hrs.
- d) STEP SWITCH ADVANCE: Allows the interval stepping switch to be advanced in order to deliver power to the depth recording system.
- e) CLOCK DIAL: Manually rotating the dial in a clockwise fashion advances the interval stepping switch once per complete revolution of the dial.

Operation

- a) Make sure ON/OFF switch is in the OFF position.
- b) Connect 12 volt and 24 volt power supply to power in receptacle on left side of control unit.
- c) Repeatedly turn the ON/OFF switch to the ON and OFF positions to allow a gradual winding of the clock.
- d) Turn the ONCE PER HR./INTERVAL PER HR. switch to the ONCE PER HR. position.
- e) Manually cycle clock (always turn clockwise) until magnet on dial closes the magnetic relay. Indicator light will come on if system is operating properly.
- f) Turn clock until indicator light goes out.
- g) Turn the ONCE PER HR/INTERVAL PER HR. switch to the INTERVAL PER HR. position.
- h) INTERVAL switch may be set at any of the four settings 1/4 hrs., 1/6 hrs., 1/12 hrs., 1/36 hrs.
- i) Slowly and firmly press the STEP SWITCH ADVANCE button to advance the interval stepping switch. When the contacts on the stepping switch are in contact with a brown lead wire the system will operate on the 1/4 hrs. INTERVAL switch setting. Similarly the orange lead wires are for the 1/6 hrs. setting and the blue lead wire is the 1/12 hrs. setting. When the protrusion on the wheel of the interval stepping switch makes contact with the black lead wire the system will operate on the 1/36 hrs. system. The stepping switch should be advanced to one position

behind the contact position that is desired for a particular INTERVAL setting. Example: Set INTERVAL switch to 1/6 hrs. Advance the interval stepping switch to one position before the contact position with an orange lead wire. Cycle the clock dial until the magnetic switch is closed. This will advance the stepping switch to make contact with the orange lead wire. The indicator light will come on if the system is operating properly.

Maintenance - No maintenance necessary.

B) Raytheon DE-719 Fathometer Depth Recorder

Operator Controls and Function

- a) OFF/SENSITIVITY: Applies power to equipment and adjusts the receiver sensitivity (gain).
- b) FIX MARK SWITCH: A spring loaded switch-causes the stylus to inscribe a reference mark the full height of the chart paper. A remote fix-mark switch is available as an optional accessory.
- c) TIDE and DRAFT: Adjustment to advance or delay the start of the transmitter pulse to compensate for tide and/or draft difference.
- d) SPEED OF SOUND: Enables operator to vary stylus drive motor speed to compensate for salinity content and temperature of water.
- e) RANGE X1, X2: Changes speed of stylus to multiply scale by 2 i.e. from 50 feet per phase to 100 feet per phase.
- f) CHART SPEED: Selects the rate of chart paper feed; 1,2,3 or 4 inches per minute.
- g) CAL ZERO: Enables the operator to adjust the first marker line to 0 (zero) on the chart paper. This is to compensate for stylus wear, etc.
- h) PHASE SWITCH: Selects the desired operating depth range.

Operation

- a) Set the Phase Switch to the 0-55 foot range. Set the RANGE X1, X2 control to X1 (control is located inside hinged front window).
- b) Open the front cover of the Recorder and rotate the stylus belt counterclockwise through one complete revolution by

rotating the upper pulley. Check that the stylus rides in the track at the left side of the assembly and the pulleys turn smoothly. Close the front cover and latch securely.

- c) Turn the OFF/SENSITIVITY control slightly clockwise until the switch clicks. This applies power to the Recorder circuitry, stylus drive motor and the chart feed motor.
- d) Adjust the front panel CAL ZERO control until the "calibrate zero line" falls on the zero calibration on the chart paper (NOTE: For this adjustment the direct transmission mark should be moved off the zero calibration by the TIDE and DRAFT control). After the CAL ZERO adjustment is completed readjust the TIDE and DRAFT control until the direct transmission mark falls on the chart paper zero line, or as required.
- e) Adjust the SPEED OF SOUND control (located inside the hinged window) until the calibrate mark falls on the CALIBRATE line near the bottom of the chart paper. This adjusts the speed of sound to 4800 feet per second.
- f) Set the chart paper speed to the desired feed rate 1,2,3 or 4 inches per minute.
- g) Turn the phase switch to the range which brackets the approximate depth reading. NOTE: The zero transmission mark appears only on the 0-55 foot range.
- h) Advance the SENSITIVITY control slowly clockwise until the bottom echo appears. Continue advancing the control until there is no change in the depth indication. Read the upper edge of the recording for the proper depth reading.

Maintenance

- a) Chart Paper Replacement: Release the hasps securing the front cover of the recorder, lower the cover. Rotate the stylus belt counterclockwise until the stylus is off the paper. Release the twist-lock at the top of the platen and swing the platen assembly to the right. Lift up on the knurled knob and shaft. Restore knurled knob to original position in the slot. Feed the end of the chart paper around the roller at the right side of the platen, across the face of the platen, around the rubber roller and onto the take-up roller. Remove the slack from the paper by turning the knob on the rubber roller clockwise.
- b) Stylus Belt and Stylus Replacement: Release the hasps securing the front cover of the recorder, lower the cover.

Rotate the stylus belt assembly until the stylus is off the chart paper, slip the old belt off the pulleys. Install the new belt by reversing the procedure. NOTE: The belt tension may be relieved by raising up on the spring end of the upper pulley support arm. Install the new belt with the open ends of the stylus holder facing down when the holder is on the right side of the stylus track.

To replace the stylus only, rotate the belt counterclockwise until the stylus holder is accessible. Install the loop in the center of the holder. (Be sure the curled end of the stylus is uppermost). Slip the free ends of the stylus into the slots of the holder. Rotate the assembly a few times by hand to be sure that the stylus and belt will track properly before applying power.

- c) Cleaning: The recorder interior should be cleaned with a soft brush and rag to remove the carbon dust which collects as a result of the recording process.
- d) Lubrication: The DE-719 is constructed with oilite and ball type bearings which are designed to provide many hours of trouble-free operation. The bearings are pre-lubricated at the factory and will not require lubrication for the life of the bearing. The drive gears are nylon, therefore, do not require lubrication. All hinges and hasps should be periodically lubricated with a light oil to prevent seizure especially when the equipment is used around salt water environment.

C) Rustrak 288 Recorder

Operator Controls

- a) Chart Advance Wheel: When depressed and turned advances chart paper.

Operation

- a) Recorder has been wired and calibrated such that it can operate unattended until chart paper supply is depleted.

Maintenance

- a) Chart Paper Replacement: Open recorder. Unlatch retaining clips. Open chassis latch. Remove supply and take-up rollers. Slide supply roller into full roll of chart paper, roller shoulder nearest paper perforations. Unroll about a foot and slide paper (back side up) between side plate and latch. Engage paper perforations in. Slide cardboard sleeve onto take-up roller. Butt paper against disc and

tape end of chart paper to sleeve. Turn one revolution for proper paper alignment. Roll paper tightly and straight on take-up roller and drop into deeper notch, engaging gear. Close retaining clips. Snap up chassis latch. Close recorder. Advance paper by depressing and turning chart advance wheel on front panel.

- b) Cleaning and Lubrication: Not necessary. Keep access window closed to prevent dust from accumulating inside recorder case.

C.3.3 Velocity System

A) Air Control

Operation

- a) Plug kwick-connect fitting from air cylinder into the AIR IN RECEPTACLE of the air control unit. Make sure fitting locks into place!
- b) Open air cylinder valve slowly making sure the pressure from the final reduction valve does not exceed 60 p.s.i.
- c) Turn both EXTEND/RETRACT valves to desired operation.

Maintenance

No maintenance necessary.

B) BA-4 Bridge Amplifiers

Operator Controls and Function

- a) POWER switch controls batteries and meter.
 - OFF Disconnects batteries.
 - A Checks amplifier batteries under load.
 - New batteries swing meter full scale.
 - Replace batteries when below 3/4 full scale.
 - B Checks bridge batteries under load.
 - New batteries swing meter full scale.
 - Replace batteries when below 3/4 full scale.
 - BAL Powers unit.
 - Connects meter across output.
 - Balance point is center of scale.
 - CUR Powers unit.
 - Connects meter in series with bridge supply.
 - Indicates current in each gage at full scale of 25 ma or total bridge current at full scale of 50 ma.

- b) GAIN switch changes feedback in amplifier.
 - 2 - 200 voltage gain at differential output.
 - 4 - 400 voltage gain at differential output.
 - 8 - 800 voltage gain at differential output.
 - 16 - 1600 voltage gain at differential output.
 - 32 - 3200 voltage gain at differential output.
 - 64 - 6400 voltage gain at differential output.
 - One half gain values at single-sided output.
- c) BDG PWR switch in series with bridge power.
 - up position connects bridge power.
 - down position disconnects bridge power.
- d) CALIBRATION switch selects calibration resistor.

.1 - 10,000,000 ohms	5 - 200,000 ohms
.2 - 5,000,000	10 - 100,000
.5 - 2,000,000	20 - 50,000
1 - 1,000,000	50 - 20,000
2 - 500,000	100 - 10,000
- e) + push button switch connects cal. resistor across bridge arm AC.
- f) - push button switch connects cal. resistor across bridge arm AD.
- g) AMP BAL is single turn pot. balancing dc amplifier.
- h) BDG BAL is ten turn pot. across arms AC and AD balancing bridge.
- i) BDG CUR is single turn rheostat in series with bridge power.
- j) BDG ARMS switch on rear panel controls internal bridge arms.
 - 2 connects 400 ohm arms to AC and AD.
 - 4 disconnects internal bridge arms.

Operation

- a) BDG ARMS switch at 2
- b) GAIN switch at 64
- c) CAL switch at 2
- d) BDG PWR switch to BAL
- e) POWER switch to A - If below 3/4 full scale replace A batteries

- f) POWER switch to B - If below 3/4 full scale replace B batteries
- g) BDG PWR switch on (up)
- h) POWER switch to CUR - adjust BDG CUR to 10 ma.
- i) BDG PWR switch off (down)
- j) POWER switch to BAL - zero meter with AMP BAL.
- k) BDG PWR switch on (up) - zero meter with BDG BAL.

Maintenance

Renew batteries when meter indicates below 3/4 full scale on A or B. Remove chassis from cabinet by unscrewing rubber feet and sliding out of the case through the rear. Make sure battery clips are well placed and tight before reassembling. Always replace both batteries of the pair at the same time. There are two in series for A and two in series for B. Use Eveready 707 or equivalent.

To tune the internal amplifier adjustment pots:

Remove case.
 Disconnect INPUT and OUTPUT plugs.
 Center AMP BAL pot. on front panel.
 GAIN at 64.
 POWER switch at BAL.
 BDG ARMS at 2 - zero meter with BAL. TRIM. pot.
 BDG ARMS at 4 - zero meter with INPUT STAB. pot.
 Repeat the two adjustments in order several times.

C) Brush 222 Recorder

Operator Controls and Functions

- a) Battery - Meter indicates battery condition. When indicator falls into red area, batteries should be recharged.
- b) Chart Speed - Seven position switch permits selection of six chart speeds and OFF. When in OFF position, all power is removed from recorder. Depressing any chart speed button activates recorder, and chart will drive at desired speed. Chart speeds may be changed while chart is running.
- c) Mark Event - Two momentary switches permit manual marking of an event. When left MARK EVENT button is depressed the left event marker pen will deflect to the left. When the right MARK EVENT button is depressed the right event marker pen will deflect to the right.

NOTE: The event marker pens will remain deflected until the MARK EVENT button is released.

- d) Sensitivity - Provides 13-step attenuation of input signal from one millivolt per division to ten volts per division, and OFF.
- e) Sensitivity X1 - Provides intermediate sensitivities between the fixed settings of the stepped sensitivity control. When the sensitivity is fixed as indicated on the stepped sensitivity control.

When rotated counterclockwise, the sensitivity is less than indicated on the stepped sensitivity control to a maximum ratio of 2.5 to 1. This control is fitted with a locking nut to prevent inadvertent turning.

NOTE: This is a variable control, but not calibrated.

- f) Position - Permits pen positioning anywhere within the channel and allows either edge of channel, or any point between to be used as zero signal position. This control is fitted with a locking nut to prevent inadvertent turning.
- g) Signal Input Terminals - Three input terminals are provided for each channel. Ground strap is removable to provide balanced or floating input. Input is single-ended with ground strap connected.
- h) Writing Table - Complete assembly consisting of table, writing bar and paper supply shaft. To gain access to the paper supply, pull out on pull bar and swing table up. To close, swing table down into position and push on pull bar until table snaps in place.

NOTE: Writing table must be completely closed or recorder will not operate. Recorder is equipped with an interlock circuit which prevents operation when writing table is not properly closed or when paper supply is exhausted.

Operation

- a) With signal disconnected and OFF button depressed, open writing table by pulling on pull bar and be sure adequate paper for operation is on supply roll, and that it is properly threaded.
- b) Connect unit to proper voltage and frequency as indicated on Input Power Adapter Module.

- c) Momentarily depress any chart speed pushbutton to check battery condition on battery indicator. Indicator should read in green area. If indicator reads red area, do not operate unit unless "line assisted" operation is used.

CAUTION: If battery conditions reads in red area, it is recommended that the unit be connected as in "line assisted" operation, but that recorder not be used. This will charge the battery. Do not attempt to operate this recorder without a battery installed.

- d) Depress 5 MM/SEC chart speed button and operate recorder to insure good ink flow on all channels. If no writing is present after first one foot of paper, check ink cartridge for adequate ink.
- e) Set SENSITIVITY control to OFF position.
- f) Turn SENSITIVITY X1 control to full clockwise position (detent) and secure with locking knob.
- g) Depress 5 MM/SEC chart speed button, and connect input signal.
- h) Position pen at exact chart center or any other position desired as signal--zero position with the POSITION control. Lock knob in this position.
- i) Switch SENSITIVITY control to a range that will permit maximum pen deflection without going beyond chart edge.
- j) Chart deflection will be proportional to the SENSITIVITY control setting for the voltage being recorded.
- k) Select a chart speed that will best display the signal with respect to the frequency of the signal.

NOTE: Power is removed from recorder by a paper interlock switch when paper supply is exhausted. Because of the necessity of keeping paper under pens at all times, some unused paper will remain on the roll.

Maintenance

- a) Chart Paper Removal - Press chart speed OFF button. Grasp pull bar on bottom of writing table and pull out and up to place writing table in open position. Hold paper supply core and shaft pull down on paper supply release lever and move paper supply core and shaft. Unscrew paper core from supply roll shaft and discard paper core.

- b) Chart Paper Installation - Slide new roll of paper onto supply roll shaft. NOTE: Check to be sure flange of shaft is at right side adjacent to channel 2 on chart paper and paper unwinds from top of roll. Screw supply roll shaft into paper core so that flange is snug against paper core. Insert flanged end of paper supply roll shaft into shaft mounting holes in writing table assembly and push up on release lever to lock shaft into position. CAUTION: Check to be sure left end of supply roll shaft is seated in writing table assembly and supply roll shaft rotates freely. Feed paper over writing bar, through slot in front of writing table, behind pressure roll and out front of writing table assembly. NOTE: Refer to paper threading diagram on side of writing table assembly. CAUTION: Use care during paper threading not to damage pens. Slowly close swinging table assembly while gently pulling on paper. When friction resists rotation of paper roll, pull paper taut, center on writing table, and close writing table. NOTE: Check to be sure writing table is properly engaged by pressing on both sides of writing table. Turn recorder on and operate at 50 millimeters per second to run about two feet of chart paper to align chart paper.

C.4.0 Checklist

This checklist provides the operator with a condensed form of the subject matter presented in the Operation and Maintenance Manual for the Scour Around Bridge Piers Instrumentation System. It should not be used without first reading the entire manual. Each step for the proper operation and maintenance of this system should be followed in order. For problems not encountered in the manual contact:

Dr. Gordon R. Hopkins
Project Principal Investigator

or

Mr. Richard W. Vance
Project Field Engineer

B-55 Engineering Sciences Building
West Virginia University
Morgantown, West Virginia 26506

Phone: (304) 293-3380 - commercial number
or (304) 296-3419 - F.T.S. number

C.4.1 Depth System and Power Supply

STEP 1: Check Battery Water Level. If battery appears to be completely dry, it may be internally shorted causing a short in the charger. DO NOT fill batteries with water and TAKE OUT FUSES.

STEP 2: Unplug Level Indicator Switch.

STEP 3: Check Recorders, Paper Supply and Writing Stylus.

- a) If not depleted, record date and time on chart paper.
Do not replace paper until a sample reading is recorded.
- b) If depleted, replace and record time and date on both the replacement chart and chart to be sent to W.V.U.
- c) Replace stylus if necessary.

STEP 4: Advance Interval Stepping Switch. Slowly and firmly press the 'Stepping Switch Advance' button to advance the interval stepping switch (the protrusion on the wheel of the interval stepping switch must be advanced to one position short of closing the 1/36 hrs. circuit) NOTE: Interval switch should be set at 1/36 hrs.

- a) If interval stepping switch advances, go to step 5.
- b) If stepping switch does not advance make sure A.C.U. clock is not in the closed circuit position and try the Stepping Switch Advance button again.
- c) If interval stepping switch still does not advance go to step 5.

STEP 5: Manually Cycle A.C.U. Clock. (Always turn clockwise)

- a) If stepping switch advances and turns on recorders go to step 6.
- b) If stepping switch advances and does not turn on recorders, see Troubleshooting Hint 1.
- c) If stepping switch does not advance, see Troubleshooting Hint 2.

STEP 6: Calibrate Raytheon DE-719 Recorder.

- a) Turn the OFF/SENSITIVITY control to get a clear trace on chart.
- b) Adjust the CAL ZERO control until the calibrate zero line falls on the zero calibration at the top of the chart paper.
NOTE: Tide and Draft may be moved out of the way.
- c) Adjust the SPEED OF SOUND control until the calibrate mark falls on the CALIBRATE line near the bottom of the chart paper.

d) Adjust the TIDE AND DRAFT control until the top of the direct transmission mark is on the zero line.

e) The following control settings remain the same.

Range	X1
Phase	0-55
Chart Speed	1

f) Advance the SENSITIVITY control clockwise as far as necessary to get clear readings from all the transducers.

STEP 7: Take Sample Depth Readings. After all adjustments have been made and the DE-719 and the Rustrak Recorder appear to be working properly, allow the A.C.U. to complete a cycle, thus obtaining readings from all the depth and stage transducers.

STEP 8: Set Clock To Operate On The Hour.

STEP 9: Record Time and Date of Sample Readings On Chart Paper. The time and date should be recorded on the charts of both recordings.

STEP 10: Change Chart Paper. If chart paper was not replaced in Step 4, change it now, and include the Sample Readings at the end of the chart to be sent to W.V.U.

STEP 11: Fill Batteries With Water. If the depth system has operated properly make sure the batteries have sufficient water.

D. BIBLIOGRAPHY

D.1.0 Bibliography

- AHMAD, MUSHTANG, "Discussion of Scour at Bridge Crossings," ASCE, Journal of the Hydraulic Division, Vol. 86, No. HY9, Part I (1960).
- AHMAD, MUSHTANG, Discussion on "Scour at Bridge Crossings" by E. M. Laursen. ASCE, Trans., Vol. 127, Part I, pp. 198-206 (1962).
- AHMAD, MUSHTANG, "Effect of Scale Distortion, Size of Model Bed Material and Time Scale on the Geometrical Similarity of Localized Scour."
- AHMAD, MUSHTANG, "Experiments on Design and Behavior of Spur Dikes." Minnesota International Hydraulics Convention, University of Minnesota, Proc. (1953).
- ANDERSON, ALVIN G., "The Hydraulic Design of Bridges for River Crossings - A Case History." Highway Research Board, Hwy. Res. Record No. 123, pp. 1-16 (1966).
- ANDRU, PETER, "Study of Scour at Obstructions in Non-cohesive Bed." M. S. Thesis, University of Alberta, Canada (1956).
- ARUNACHALAM, K., "Scour Around Bridge Piers." Journal of the Indian Roads Congress, Vol. 29, No. 2, pp. 189-210 (Aug. 1965). Highway Research Board, Hwy. Res. Abstracts, Vol. 36, No. 4, pp. 1-2 (Apr. 1966).
- AUTOMATION INDUSTRIES, INC., "Design and Development of Ultrasonic Scour Meter." FHWA Final Report for Contract CPR-11-2851.
- BAGNOLD, R. A., "An Approach to the Sediment Transport Problem From General Physics," U. S. Geology Survey, Professional Paper, p. 422-J.
- BATA, G. "Erozija Oko Novosadskog Mostovskog Stuba (Serbian) (Scour Around Bridge Piers). Institiut Za Vodoprivreder, Jaroslav Cerai Beozrad Yugoslavia (1960).
- BENSON, M. A. and MATALAS, N. C., "Effect of Floods and Obsolescence on Bridge Life." Journal Highway Sw., ASCE, Vol. 91, No. HW1, Proc. Paper 4208, pp. 55-65 (Jan. 1965).
- BLENCH, T., "Discussion of Scour at Bridge Crossings." ASCE, Journal of the Hydraulic Division, Vol. 86, No. HY5 (1960).
- BLENCH, T., "Regime Behavior of Canals and Rivers." Butterworths Scientific Pub., London (1957).
- BORHEK, R. "Scouring of Foundations, As a Case of Bridge Failure" Roads and Bridges, P. 33, August (1943).
- BRADLEY, J. N., "Discussion of Scour at Bridge Crossings." ASCE, Journal of the Hydraulic Division, Vol. 86, No. HY8 (1960).
- BRADLEY, J. N., "Hydraulics of Bridge Waterways." Bureau of Public Roads, Div. of Hydraulic Research, Hydraulic Design Series No. 1, 53 pp. (1960).
- BREUSERS, H. N. C. "Confirmity and Time Scale in Two Dimensional Local Scour." Research Eng., Delft Hydraulics Lab., Netherland.
- BROOKS, N. H., "Mechanics of Streams with Movable Beds of Fine Sand." ASCE, Journal of Hydraulic Division, Vol. 81 (Aug. 1955).
- CHABERT, J. and ENGELDINGER, P. "Etude des Affouillements Autour des Piles de Ponts" (study of Scour Around Bridge Piers). Laboratoire National D'Hydraulique 6, Quai Watier, Chatou, France (1956).

- CHANG, FRED F. M., "Statistical Summary of the Cause and Cost of Bridge Failures." FHWA Final Report for contract DOT-FH-11-8093, Sept. (1973).
- CHANG, FRED F. M., and YEVDJEVICH, V. M., "Analytical Study of Local Scour." Civil Eng. Section, Colorado State University, Fort Collins, Colorado, CER62FMC26, April (1962).
- CHANG, FRED F. M., and KARIM, MANSOUR, "An Experimental Study of Reducing Scour Around Bridge Piers Using Piles" South Dakota Dept. of Highways in Cooperation with U. S. Dept. of Transportation, Federal Highway Administration, Jan. (1972).
- CHITALE, S. V., "Discussion of Scour at Bridge Crossings" Trans. ASCE, Vol. 127, Part 1, p. 191 (1962).
- CHITALE, S. V., "Discussion of An Analysis of Relief Bridge Scour," ASCE, Hyd. Div., Vol. 90, Part 1, Hy 1 pp. 287 (1964).
- COLBY, B. R., "Scour and Fill in Sand-Bed Streams" Geological Survey Professional Paper 462-D (1964).
- CORRY, MURRAY L., and SAGER, WILLIAM, "Ultrasonic Instrument for Determining Local Scour at Bridge Piers." Public Roads, Vol. 35, No. 4, pp. 91-96 and 108 (Oct. 1968).
- CULBERTSON, D. M., "Scour and Fill in Alluvial Channels with Particular Reference to Bridge Sites." U. S. Department of Interior, Geological Survey, 58 pp. (1967).
- DE SOUSA PINTO, N. L., SYBERT, J. H. and POSEY, C. J., "Model Tests of Riprap Scour Protection," Rocky Mountain Hydraulic Laboratory, Allenspark, Colorado Dec. (1959).
- EINSTEIN, H. A., "The Bed-Load Function for Sediment Transportation in Open Channel Flow." Technical Bulletin No. 1026, U. S. Department of Agriculture, (Sept. 1950).
- EINSTEIN, H. A., "Formulas for the Transportation of Bed Load." ASCE Transactions, No. 2140 (1954).
- ERICKSON, L. L., "Some Measurements of Velocities and Scour at a Mississippi River Bridge Pier." Highway Research Board, Proc., Vol. 26, pp. 124-128 (1946).
- FEDERAL INTER-AGENCY SEDIMENTATION PROJECT, "Field Practice and Equipment used in sampling suspended Sediment." Report No. 1 (1940).
 "Methods of Analyzing Sediment Samples." Report No. 4 (1941).
 "Improved Types of Suspended Sediment Samplers." Report No. 6 (1952)
 "Electronic Sensing of Sediment." Report R (1964). "Instruments and Reports for Fluvial Sediment INvestigations." Catalog (1971).
- GARDE, R. Y., JABRAMANYAN, K., and NAMBU DRIPAD, K. D., "Study of Scour Around Spur Dikes." ASCE, Proc., Journal of the Hydraulics Division, Vol. 87, No. HY6 (1961).
- GRAF, WALTER H., "Hydraulics of Sediment Transport" McGraw-Hill, N. Y. (1971).
- HELLEY, E. J. and SMITH, W., "Development and Calibration of a Pressure-Difference Bedload Sampler." U. S. Department of the Interior, Geological Survey - Water Resources Division, Open File Report, Menlo Park, California (1971).
- HERBICH, H. P., and BRENNAN, L. M., "Prediction of Scour at Bridges." Ontario Dept. of Highways, D.H.O. Dept. No. RR 115, 135 pp. (Apr. 1967). Highway Research Board, HWY. Res. Abstracts, Vol. 37, No. 11, p. 2 (Nov. 1967).

- HICKENLOOPER, IRBY J., GUILLOU, JOHN C., and CHOW, VEN TE, "Hydraulic Studies of a Highway Bridge." University of Illinois, Dept. of Civil Engineering, 76 pp. (1957).
- HUBBARD, P. G., "Field Measurement of Bridge-Pier Scour." Highway Research Board, Proc., Vol. 34, pp. 184-188 (1955).
- INGLIS, C. C., THOMAS, A. R., and JOGLEKAR, D. V., "The Protection of Bridge Central Irrigation and Hydrodynamics Research Station, Poona, India, Research Publication No. 2 (1939).
- ISHIHARA, TOZIRO, "Experimental Study on Scour at Bridge Piers" Trans. of Japanese Society of Civil Engineers, Vol. 24, No. 1, pp. 28-55 (1938). Vol. 28, No. 9, pp. 787-821 (1942). Vol. 28, No. 11, pp. 974-1007 (1942).
- JOGLEKAR, D. V., "Discussion of Scour at Bridge Crossings." ASCE, Journal of the Hydraulic Division, Vol. 86, No. HY9, Part 1 (1960).
- KAIN, D. H., "Scour Effects at the Chirna Bridge." Public Works Department, Zomba, Nyasaland (1961).
- KENTY, B. A., "A Suggested Hypothesis for the Determination of Scour Depth in River Beds." Trans. S. A. Inst. Civil Engineering, Vol. 6, No. 8 (Aug. 1956).
- KARAKI, S. S., "Hydraulic Model Study of Spur Dikes for Highway Bridge Openings." Colorado State University, Fort Collins, Report CER 59 SSK 36 (1959).
- KARAKI, S. S., "Laboratory Study of Spur Dikes for Highway Bridge Protection." Highway Research Board, Bulletin No. 268, pp. 1-12, (1961).
- KASRAIE, BEHZAD, "The Effects of a Boundary Depression on Motion of a Particle Transported by Laminar Flow," M. S. Thesis, West Virginia University (1973).
- KEUTNER, CHR., "The Flow Around Piers of Different Shapes and its Effect on the River Bed." Die Bautechnik, Vol. 10, p. 161 (1932).
- KLINGEMAN, P. C., "Prediction of Pier Scour in Western Oregon." Sponsored by the Federal Highway Administration, U. S. Department of Transportation, Region 8, In Cooperation with the State Highway Division (1971).
- KNEZEVIC, BOGIC, "Contribution to Research Work of Erosion Around Bridge Piers." Translation is Filed at Eng. Research Center, Colorado State University, Fort Collins, Colorado (No. I.D. 67-11) (1960).
- KUHN, S. H., and WILLIAMS, A. A. B., "Scour Depth and Soil Profile Determination in River Beds." Proc. 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1961.
- LANE, E. W. and BORLAND, W. M., "River-Bed Scour During Floods." ASCE Transactions Paper No. 2710 (1955).
- LARRAS, JEAN, "Maximum Depth of Erosion in Shifting Beds Around River Piles." Annales des Ponts et Chaussées, Vol. 133, No. 4.
- LARRAS, JEAN, "Discussion on Local Scour Around Bridge Piers." ASCE, HY6, p. 1368, June (1970).
- LAURSEN, E. M., "An Analysis of Relief Bridge Scour." ASCE, Proc., Vol. 89, No. HY3, pp. 93-118 (1963).
- LAURSEN, E. M., "Bridge Design Considering Scour and Risk." ASCE, Transportation Engineering Journal, Vol. 96, No. TE2, pp. 149-164 (May 1970).
- LAURSEN, E. M., "Model-Prototype Comparison of Bridge-Pier Scour." Highway Research Board, Proc., Vol. 34, pp. 188-193 (1955).

- LAURSEN, E. M., "Observations on the Nature of Scour." State University of Iowa, 5th Hydraulics Conference, Proc., (1953).
- LAURSEN, E. M., "Progress Report on Model Studies of Scour Around Bridge Piers and Abutments." Highway Research Board, Research Report 13-B, pp. 1-11 (1951).
- LAURSEN, E. M., "Scour at Bridge Crossings." ASCE, Trans., Vol. 127, Part 1, Paper 3294, pp. 166-209 (1962).
- LAURSEN, E. M., "Scour at Bridge Crossings." Iowa HRB, Bull. No. 8, 53 pp. (Aug. 1958).
- LAURSEN, E. M., "Some Aspects of the Problem of Scour at Bridge Crossings." Federal-Inter-Agency Sedimentation Conference, Jackson, Mississippi (1963).
- LAURSEN, E. M., "The Total Sediment Load of Stream." ASCE, Journal of the Hydraulic Division, Vol. 84 (1958).
- LAURSEN, E. M., and TOCH, A., "A Generalized Model Study of Scour Around Bridge Piers and Abutments." Minnesota Int'l. Hydraulics Convention, University of Minnesota, Proc. (1953).
- LAURSEN, E. M. and TOCH, A., "Model Studies of Scour Around Bridge Piers and Abutments - Second Progress Report," Highway Research Board, Proc., Vol. 31, pp. 82-87 (1952).
- LEVEEN, L. S., "Investigation of Scour at Selected Bridge Crossings in Alaska." U. S. Interior Department, Geological Survey, Water Resources Division, Progress Reports, 30 pp. (Nov. 1966) and 22 pp. (Apr. 1967).
- LIU, H. K., CHANG, F. M., and SKINNER, M. M., "Effect of Bridge Constriction on Scour and Backwater." Civil Eng. Section CER 60 HKL22, Colorado State University, Fort Collins, Colorado, Feb. (1961).
- LIU, H. K., BRADLEY, J. N., and PLATE, E. O., "Backwater effects on Bridge Piers and Abutments." Civil Eng. Section CER 57 HKL2, Colorado State University, Fort Collins, Colorado (1957).
- LIU, H. K., and SKINNER, M. M., "Laboratory Observations of Scour at Bridge Abutments." Highway Research Board, Bull. 242, pp. 69-77 (1960). "Mechanics of Local Scour," Parts I and II, State of Knowledge and Bibliography, Colorado State University, CER63SSK46, November 1963. "Mechanics of Local Scour," Colorado State University, CER66HWS10, June 1966.
- MARIN, J. N. M., "An Investigation of Local Scour Around Bridge Piers." M. Tech. Thesis, University of South Wales, Water Resources Lab., Manleyvale, N.S.W. Australia (1962).
- MAZA ALVARER, J. A. and SANCHES BRIBIESCA, J. L., "Contribucion al Estudio de la Socavacion local en Pilas de Puente." Universidad Nacional Autonoma de Mexico, Facultad de Ingenieria Publicacion Numero 84 (1964).
- MEYER-PETER, E. and MULLER, R., "Formulae for Bedload Transport," Procedures 2 Meeting IAHR, Stockholm, 1948, pp. 39-64.
- MOULTON, L. K., BELCHER, C., and BUTLER, B. E., "Report on an Investigation of Scour at Bridges Caused by Floods." Civil Engineering and Public Works Review, Vol. 53, No. 624, June 1958.
- MOULTON, L. K., BELCHER, C., and BUTLER, B. E., "Report on Investigation of Scour at Bridges Caused by Floods of 1955." Highway Research Board, Hwy. Res. Abstracts, Vol. 27, No. 8, pp. 14-31 (Sept. 1957).

- NEILL, C. R., "Discussion of Scour Around Spur Dikes," ASCE, Proc. Journal of the Hydraulics Division (1962).
- NEILL, C. R., "Introduction to the Problem of Scour at Bridges." Unpublished informal paper presented to the Subcommittee on Bridge Scour of the Committee on Bridges and Structures, Canadian Good Roads Association, Winnipeg, Manitoba (1963).
- NEILL, C. R., "Measurements of Bridge Scour and Bed Changes in a Flooding Sand-Bed River." Institution of Civil Engineers, Paper 6775, pp. 415-436, (Feb. 1965). Highway Research Board, Hwy. Res. Abstracts, Vol. 35, No. 6, p. 10 (June 1965).
- NEILL, C. R., "River-Bed Scour. A Review for Bridge Engineers." Canadian Good Roads Assoc., Tech. Pub. No. 23, 37 pp. (Dec. 1964). Hwy. Research Board, Hwy. Res. Abstracts, Vol. 35, No. 5, p. 9 (May 1965).
- NEILL, C. R., "Discussion on Local Scour Around Bridge Piers." ASCE, HY5 May (1970).
- PATEL, M. S., "Scour Control at Skew Bridge Abutments by the use of Spur Dikes." Fritz Lab. Report No. 280-M-30, Lehigh University Institute of Research Bethlehem, Penna. (1961).
- POSEY, C. J., "Why Bridges Fail in Floods." Civil Engineering, Vol. 19, No. 2, P. 94 (Feb. 1949).
- POSEY, C. J., APPEL, D. W., and CHAMNESS, E., JR., "Investigation of Flexible Mats to Reduce Scour Around Bridge Piers." Highway Research Board, Research Report 13-B, pp. 12-22 (1951). "River Bed Scour," Technical Publication No. 23, Canadian Good Roads Assoc., Ottawa, Canada, December 1964.
- POSEY, F. "Scour at Bridge Piers, 1 Protection of Threatened Piers," Civil Engineering Vol. 33, pp. 48, May (1963).
- RAUDKIVI, A. J., "Loose Boundary Hydraulics" Pergamon Press Ltd., London, England (1967).
- SABURO KOMURA, "Equilibrium Depth of Scour in Long Constriction." ASCE, Journal of Hydraulic Division, HY5, pp. 17, Sept. (1966).
- SANDEN, E. J., "Scour at Bridge Piers and Erosion of River Banks." Western Assoc. of Canadian Highway Officials, 13th Annual Conf., Proc. (1960).
- SANDEN, E. J., and NEILL, C. R., "Measuring Scour Around Bridge Foundations in Floods." Public Works in Canada, Vol. 11, No. 7, pp. 14-17 (1963).
- SCHNEIDER, V. R., "Mechanics of Local Scour." Ph.D. Dessertation, Colorado State University, Fort Collins, Colorado (1968).
- SCHNEIBLE, DOUGLAS E., "An Investigation of the Effect of Bridge Pier Shape on the Relative Depth of Scour." M. S. Thesis, University of Iowa, 92 pp. (1951). "Scour Around Bridges." Highway Research Board, Research Report 13-B, 22 pp. (1951).
- SCHNEIBLE, D. E., "Some Field Examples of Scour at Bridge Piers and Abutments." Better Roads, (1954).
- SEARCY, J. K., "Use of Riprap for Bank Protection." Bureau of Public Roads, Hydraulic Engineering Circular No. 11 (June 1967).
- SHEN, HSIEH W., and SCHNEIDE, VERNE R., "Effect of Bridge Pier Shape on Local Scour." ASCE National Meeting on Transportation Engineering, Boston, Massachusetts, July 13-17, 1970.

- SHEN, H. W., SCHNEIDER, V. R., and KARAKI, S. S., "Local Scour Around Bridge Piers." ASCE, Journal of Hydraulics Division, Paper No. 6891, (Nov. 1969).
- SHEN, H. W., SCHNEIDER, V. R., and KARAKI, S. S., "Mechanics of Local Scour." Colorado State University, Engineering Res. Center, CER66HWS22, 57 pp. (June 1966). "Short Piles at Big Sioux." Engineering News-Record, Vol. 168, No. 18, p. 17 (May 3, 1962).
- SHEN, H. W., "Field Trip to Palmer, Alaska." Civil Engineering Dept. (CER66HWS35). Research Center, Colorado State University, Fort Collins, Colorado (1965).
- STABILINI, L., "Scour at Bridge Piers, Cause and Effect." Civil Engineering (N. Y.), Vol. 33, No. 5, pp. 46-7 (May 1963).
- STIEFEL, R. C., "Scour at Relief Bridges." M. S. Thesis, State University of Iowa, Iowa City, Iowa (1959).
- STRAUB, L. G., "Dredge Fill Closure of Missouri River at Fort Randall." Minnesota International Hydraulic Convention, Minneapolis (1953).
- SUGIMOTO, S. and INADA, Y., "On Separation of Flow From Bridge-Pier Surface." Jap. Civil Engineering Vol. 36, No. 7, pp. 21-25 (1951).
- SUGIMOTO, S. and INADA, Y., "Study on Hydraulics Around Piers." Jap. Civil Engineering, Vol. 36, No. 10, pp. 9-12 (1950).
- TARAPORE, Z. S., "A Theoretical and Experimental Determination of the Erosion Pattern Around Obstructions Placed in an Alluvial Channel with Particular Reference to Vertical Circular Cylinders and Piers." Ph.D. Dissertation, University of Minnesota (1962).
- THOMAS, A. R., "Discussion of Scour at Bridge Crossings." ASCE, Journal of the Hydraulic Division, Vol. 86, No. HY9, Part I (1960).
- THOMAS, A. R. and LAURSEN, E. M., "Discussions on Local Scour Around Bridge Pier." ASCE, HY9, Sept. (1970).
- TISON, L. J., "Discussion of Scour at Bridge Crossings." ASCE, Journal of the Hydraulic Division, Vol. 86, No. HY9, Part I (1960).
- TISON, L. J., "Local Scour in Rivers." J. Geophys. Res. (Wash.), Vol. 66, No. 12, pp. 4227-4233 (1961).
- TISON, L. J., "L'Erosion et le Transport les Materiaux solides dans les d'Eau." (Erosion and Transportation of Sediment in Water Courses). IVWBV. IAHSR (Int'l. Assoc. for Hyd. str. Res.) AIRTH, Liege, Vol. II, Paper No. 3, pp. 71-90 (1939).
- TISON, L. J., "Studies of Critical Tractive Force for the Entrainment of Bed Materials." Minnesota International Hydraulic Convention, Minneapolis (1953).
- U. S. GEOLOGICAL SURVEY, WATER RESOURCES DIVISION. "Investigation of Scour at Bridges in Alaska." Quarterly Progress Report F-91902-883, For the period of Oct. 1-Dec. 31. (1971).
- VARZELIOTIS, A. N., "Model Studies of Scour Around Bridge Piers and Stone Aprons." Thesis University of Alberta, Canada (1960).
- VEIGA DA CUNHA, L., SHANKARACHAR, D. and CHANDRASE KHARA, T., "Discussion on Local Scour Around Bridge Piers." ASCE, HY8, August (1970).
- YALIN, M. S., "Mechanics of Sediment Transport" Pergamon Press Ltd., London, England (1972).
- ANDERSON, Alvin G., "A Report on Scour at Bridge Waterways-A Review." Federal Highway Administration, Oct. 1974.