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SCOUR AT BRIDGE STRUCTURES AND CHANNEL AGGRADATION AND DEGRADATION FIELD MEASUREMENTS

State of the Art

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16. Abstract

A study on bridge scour and bridge related channel instability field measurements was conducted for the Arizona Department of Transportation (ADOT). The objectives of the research were to review and evaluate procedures and field techniques for measuring scour at bridge structures and for monitoring channel aggradation, degradation, and lateral migration; and to recommend a program to initiate data collection in Arizona. The ultimate goal of this research is to provide data and procedures to identify bridges that may be susceptible to scour and channel instability problems so that adequate countermeasures can be undertaken to avoid and mitigate bridge scour incidents and to collect data on scour and channel instability processes in Arizona so that appropriate design and construction procedures can be developed and used in Arizona.

Pilot programs for bridge scour have been recommended. These include the use of ground penetrating radar, buried miniature transmitters, post-flood reconstitutions, and mobile teams. A phased monitoring program for channel aggradation, degradation, and lateral migration has been recommended. Level 1 monitoring uses readily available data from the bridge inspection program to identify bridges that may be experiencing these longer term channel changes. Subsequent, more rigorous, levels of monitoring are to be used if the Level 1 monitoring indicates the need. Bridge data files are recommended for the compilation and analysis of data and general information.

A general implementation plan has been recommended that includes ADOT, USGS, engineering consultants, and university research groups. A technical advisory team should be established, and a special consultant should be retained to assist in defining future scopes-of-work, setting priorities, organizing and implementing the programs, and reviewing program progress.

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EXECUTIVE SUMMARY

A study on the field measurment of bridge scour and bridge related channel instability was conducted for the Arizona Department of Transportation (ADOT). The objectives of the research were to review and evaluate procedures and field techniques for measuring scour at bridge structures and for monitoring channel aggradation, degradation, and lateral migration at and near bridge structures; and to recommend a program or programs to initiate data collection in Arizona. The ultimate goal of this and subsequent research is to assist ADOT with the following; 1) to provide data and procedures to identify bridges that may be susceptible to scour and channel instability problems so that adequate countermeasures can be undertaken to avoid and mitigate bridge scour incidents, and 2) to collect data on scour and channel instability processes in Arizona so that appropriate design and construction procedures can be developed and used in Arizona.

Pilot programs have been recommended for bridge scour. The purpose of the pilot programs is to develop methodologies; to test equipment, techniques, and procedures for data collection; to provide opportunities for training and for the preparation of guidelines, instructions, training and techniques manuals; and to identify possible areas for needed research and development.

Four pilot programs for bridge scour have been recommended, and these are: 1) The use of ground penetrating radar (GPR) to measure the depth and extent of scour after flooding has occurred. 2) The use of buried miniature transmitters to measure the depth and extent of scour, both during and after flood events. 3) Post-flood reconstitution of flood and scour events. 4) The use of mobile teams to measure scour during flooding. These are short-term programs that could be used to define long-term (10 years or longer) data collection programs.

A monitoring program for channel instability has been recommended. This is a phased program wherein all bridges can be subjected to an initial evaluation in regard to channel aggradation, degradation, and lateral migration. After the initial phase of monitoring (Level 1), bridges that have been identified as having active channel instability can proceed into a Level 2 and possibly a Level 3 of monitoring. This program provides the appropriate level of monitoring depending upon the need at each bridge site.

A bridge data file has been recommended. This would consist of a document file and a computerized data base file. The document file would be the repository of all background and descriptive information on each bridge. This would include engineering drawings, annotated photographs, aerial photographs, bridge inspection records, hydrologic analyses, reports, descriptions of flood incidents, and river channel data. The computerized data file would contain succinct characteristics of the bridge and waterway and would be the repository of all hydrologic, hydraulic, and bridge scour data. The data files would be used to identify bridges for scour and channel monitoring programs, and to test, evaluate, and develop bridge scour equations and techniques.

A general implementation plan has been suggested that includes a long-term data collection program with the U.S. Geological Survey (USGS). Such a program would need to be negotiated with the USGS. Several other portions of the overall program could be contracted to engineering consulting firms and/or universities. ADOT would need to be actively involved in all the programs. Because of the need for a cooperative effort between ADOT, the USGS, consultants, and university research groups, a special consultant should be retained to serve in a review and advisory capacity. The special consultant would assist ADOT in defining the scope-of-work to be negotiated with the USGS, would assist ADOT in organizing and implementing its bridge scour activities, and would assist in prioritizing projects for which consultant services would be required. The consultant could also be available for other bridge related assignments to ADOT.

The main conclusions of this research are; 1) there has been substantial damage to bridges and transportation systems due to scour and flooding in Arizona, 2) no reliable field data on bridge scour have been collected in Arizona or the Southwest, 3) bridge pier and abutment scour equations cannot be adequately tested for use in Arizona, 4) many field techniques and equipment that have been developed to study scour are not appropriate for use in Arizona, and 5) a long-term (10 years or longer) data collection program should be initiated in Arizona.

The main recommendations are; 1) bridge data files should be developed to select sites for field programs, and to store data, 2) four pilot programs for bridge scour should be implemented, these are to be used to test equipment and develop techniques, 3) implement a phased program to monitor channel aggradation, degradation, and lateral migration at bridges, 4) the various programs can be implemented through a combined effort of ADOT, USGS, engineering consultants, and university research groups, and 5) a team of experts should be assembled as advisors, and a special consultant should be retained to coordinate, review, and advise ADOT on the program.

The benefits of the recommended program are; 1) a systematic means is provided to identify bridges that may be experiencing the cumulative effects of scour and/or channel instability, 2) bridges that are susceptible to scour can be monitored during flood events, 3) priorities can be set for bridge scour countermeasures, 4) improved bridge design procedures can be developed, 5) damage to bridges can be assessed after floods, and 6) bridge safety will be enhanced.

INTRODUCTION

Objective and Goals

This report presents the results of research that was conducted for the Arizona Department of Transportation (ADOT) during the period of July 1988 through November 1989. The objectives of this research were to review and evaluate procedures and field techniques for measuring scour at bridge structures and for monitoring channel aggradation, degradation, and lateral migration at and near bridge structures; and to recommend a program or programs to initiate data collection in Arizona. If existing procedures and techniques were found to be unsatisfactory, a research program was to be developed to establish procedures and techniques that are appropriate to Arizona conditions. The ultimate goal of this and subsequent research is to assist ADOT with the following; 1) to provide data and procedures to identify bridges that may be susceptible to scour and channel instability problems so that adequate countermeasures can be undertaken to avoid and mitigate bridge scour incidents; and 2) to collect data on scour and channel instability processes in Arizona so that appropriate design and construction procedures can be developed and used in Arizona.

Scope and Limitation

This project is to address bridge scour and channel degradation and aggradation field data measurements. Bridge scour is defined to mean either local scour at a bridge pier or abutment, or general scour in the bridge waterway. Degradation is the process by which the bed of the watercourse over a relatively long distance falls (scours), and aggradation is the process by which the bed of the watercourse rises (fills). Aggradation and degradation are reflected in the vertical response of the bed of the channel. However, lateral movement of the watercourse banks or realignment of the flow can have serious consequences to bridges. Lateral migration, flow realignment, meandering, braiding, channel widening, and avulsions on alluvial fans need to be addressed as well as aggradation and degradation. In this report, channel instability is defined to mean aggradation, degradation, lateral migration and other processes as indicated above, or the combination of any of these.

In this report there is a distinction between pilot programs and monitoring programs. Pilot programs are relatively short duration programs to test equipment, develop field practices, and train personnel. Monitoring programs imply a long-term (on the order of 10 years) or continuous commitment to a data collection and analysis program. The purpose of pilot programs is to collect information and gain experience so that effective monitoring programs can be defined and executed.

This project addresses bridges within the state-system of highways in Arizona. However, there could be situations where it is advantageous to measure or monitor scour at bridges that are not within the state-system. This may be particularly true for pilot programs for which the intent is to test equipment, develop field practices, and to train personnel. Certain bridges that are under the jurisdiction of cities and counties may offer opportunities for pilot programs that are attractive because of physical conditions at the bridge, the hydrologic regime, or logistics. Therefore, it is suggested that all bridges over waterways within Arizona be considered for implementation of these recommendations. If bridges with potential scour problems are identified that are particularly attractive to the goals of this research that are not within the state-system, then the jurisdictional agency for that bridge should be approached concerning a cooperative pilot program.

There are about 800 bridges over waterways in Arizona that are within the state-system. There are many other waterway bridges in Arizona that are not within the state-system. It will be difficult to select bridges in Arizona for pilot programs and monitoring programs because of the large number of bridges that must be considered. A program of data collection, storage, and analysis has been defined to facilitate the selection of bridges for pilot programs and monitoring programs. The initial data collection should be limited to bridges within the state-system. In subsequent years, after the data base files are developed for the state-systems, efforts should be made to incorporate all waterway bridges in Arizona into the data files. The incorporation of bridges that are not within the state-system could be accomplished as a cooperative program between ADOT and the various cities and counties. It is envisioned that once the ADOT bridge scour programs and data collection systems are established, data for these other bridges could be supplied to

ADOT by the appropriate city or county for its bridges. This would greatly expand the data base for future analytic purposes and would enhance the statewide bridge inspection programs.

Background

Bridge failures result in significant economic loss and disruption to commerce and travel, and may be the cause of loss of life. When a bridge failure is a large-scale disaster, such as the failure on 5 April 1987 of the Schoharie Creek bridge on the New York State Thruway system claiming 10 lives, it receives nationwide media attention which may result in lack of confidence in the highway systems by the public. Significant advances have been made in bridge design and construction, and bridge inspection is a standard operation and maintenance procedure for all states; however, failure of bridges due to scour and other hydraulic factors continues.

Analytic methods to predict scour are based mostly on theoretical considerations and empirical relations developed from laboratory experiments. Many of these scour equations have been evaluated using actual bridge scour measurements, however, such field data are sorely deficient for arid regions and such data are virtually nonexistent for Arizona. The applicability of bridge scour equations for use in Arizona is uncertain due to the vastly different geologic and hydrologic conditions that exist as compared to the conditions for which most of the equations were developed.

ADOT inspects each bridge under its jurisdiction in the state highway system biennially and this program has been and will continue to be very beneficial in improving the safety, reliability, and cost effectiveness of Arizona highways. However, only limited success can be expected from the bridge inspection program in regard to mitigation of scour incidents at bridges. This is mainly because of the ephemeral nature of the watercourses in Arizona and the often sudden and catastrophic nature of flooding and resultant bridge scour. Bridge waterways that are inspected one day and found to be in excellent condition with no external evidence of present or past scour problems can be subjected to extreme flooding and destroyed by scour within hours after the inspection. Guidelines to identify bridges that are imminently susceptible to scour and channel instability problems are not well defined or tested for Arizona and other semi-arid regions.

In the past, bridge design and construction practices in Arizona and elsewhere have not always adequately addressed the potential for scour at bridges nor the need for channel control works in the vicinity of the bridge. Many bridges exist in Arizona that have been built on spread footings without piles or on piles that are too short. The embedment depth of these foundations may not be adequate to sustain the necessary load bearing capacities during flood events. In addition, many watercourses are undergoing long-term aggradation, and bridges over these streams also are jeopardized due to the loss of hydraulic capacity through the bridge waterway.

More recent designs have used pile foundations for bridge piers. Although generally considered less susceptible to bridge scour, if the piles are not long enough, these also can be damaged or destroyed by bridge scour. Of equal concern to pier scour is abutment scour and embankment scour of the bridge approach roadways. Many bridges are more susceptible to abutment scour and approach embankment scour than to pier scour because of the combined effects of waterway encroachment, overbank flow hydraulics, and channel instabilities such as lateral migration.

This research program is intended to assist ADOT in collecting the necessary data and documentation of bridge scour and related channel instability problems so that better design practices and inspection programs can be conducted. The implementation of this research can result in improved tools for management of the bridges in Arizona and should be of assistance to the ADOT Bridge Scour Team in assessing bridge scour problems and setting priorities for bridge scour countermeasures.

Organization of Report

This report is in four parts: Part 1 summarizes the theory of scour at bridge crossings, discusses the current status of field practices and studies, provides a compilation of selected field data, and discusses bridge damage due to scour in Arizona. A more comprehensive presentation of bridge scour theory and procedures is contained in Appendix A.

Part 2 provides recommendations and cost estimates for implementing bridge scour pilot programs and long-term monitoring programs, and presents a plan to manage the data from those programs.

Part 3 identifies the research elements of some of the recommended pilot programs, and long-term research requirements for bridge scour and bridge related channel instabilities.

Part 4 summarizes the conclusions and recommendations of this study.

PART 1 - THEORY AND DATA

THEORY OF SCOUR AT BRIDGE CROSSINGS

General

Detailed descriptions of the processes involved in scour at bridge crossings and of current practices for predicting scour depth at abutments and bridge piers are given in the Federal Highway Administration (FHWA) report Interim procedures for evaluating scour at bridges (FHWA, 1988), parts of which are abstracted in Appendix A. This section presents a brief summary of the material in Appendix A, with particular emphasis on the variables and quantities that need to be measured in order to use existing equations to predict scour depths or to test field data against the theories.

Components of Bridge Scour

Scour at bridges usually is divided into three components: (1) channel aggradation or degradation which raises or lowers the mean bed elevation over a long reach of a channel, (2) general scour which may occur naturally or may result from a contraction in the approach section and through the bridge opening, and (3) local scour which results from vortices formed in the vicinity of local obstructions such as piers or abutments. These components are generally additive. In addition to the above, low-flow channel incisement, scour in bends, and scour due to dunes and other bed forms also need to be considered. Debris piled against the bridge or ice jams may increase both general and local scour. Lateral migration of the stream channel also may be an important factor by changing the angle of the approach flow or more directly by eroding abutments or approaches to the bridge.

Channel Aggradation or Degradation

Aggradation and degradation are the raising and lowering, respectively, of the streambed over long reaches of the channel, as opposed to scour and fill, which usually are considered to be local phenomena. The factors that lead to aggradation or degradation are described in Appendix A, and they may be either natural or man-induced. Usually, aggradation and degradation occur slowly over long time; they are long-term processes. Sometimes, however, the changes are rapid and spectacular, such as the widespread arroyo development and headcutting in the Southwestern United States during the last half of the nineteenth century. Trends in streambed elevation changes can be detected by repetitive surveys, aerial photos in the case of channels that are dry most of the time, and geological and geomorphological studies. Channel responses to man-induced changes, such as reservoirs, diversions, channelization, gravel mining and land-use modifications sometimes can be predicted using mathematical models as described in Appendix A. Generally, predictions of future conditions are difficult to make and are not very accurate.

General Scour

At any location along an alluvial streambed, scour occurs when the local sediment transport capacity of the flow is greater than the incoming sediment load and fill occurs when the local sediment transport capacity is less than the incoming sediment load. Scour and fill can be computed from sediment discharge relations, knowing the properties of the flow, the fluid, and the sediment. This applies to scour in a bend, to natural cycles of scour and fill associated with annual cycles of water and sediment discharge, or to scour induced by artificial contractions. The time rates of scour can be computed using mathematical models and the equilibrium scour depths in long contractions can be estimated by one of the several methods described in Appendix A. For these latter cases, the data required to carry out the calculations are the properties of the flows through the upstream reach for the design flood, the properties of the bed sediments and the fluid, and the geometry of the contracted reach. Only bed load transport is considered. Because these are equilibrium calculations, the incoming bed load is assumed to be transported at the equilibrium capacity of the upstream flow and the depth of scour through the contraction follows directly from bed continuity considerations. If the bed sediments contain a wide range of sizes, provisions to account for sorting and armoring have to be included in the calculations.

Local Scour

The principles of local scour are the same as the principles for general scour outlined above. At any location, if V_s is the volume of scour or fill, then the change in volume with time is:

$$\frac{dV_o}{dt} = c(Q_{si} - Q_{so}) \tag{1}$$

where Q_{si} is the incoming sediment discharge, Q_{so} is the outgoing sediment discharge, c is a coefficient to convert units of transport to units of inplace volume, and t is time. The total volume of scour or fill in time t is:

$$V_o = c\int_0^t (Q_{si} - Q_{so})dt \tag{2}$$

Pier Scour -- Local scour occurs as a result of increased velocities associated with vorticies that form around piers and local obstructions. The mechanisms are described in Appendix A (Section 5).

The main quantities that determine the local scour depth, y_s, at a circular pier are identified in the definition sketch, Figure 1, and the variables are identified in Equation 3:

$$y_s = f(V,y,d,\sigma,a) \tag{3}$$

where V is approach velocity, y is approach depth, d is median diameter of the bed sediment, σ is a measure of the gradation or size distribution of the bed sediment, and "a" is the pier diameter. Other quantities that usually are assumed constant are density of the fluid, γ, density of the sediment, γ_g, and the acceleration due to gravity, g. The fluid viscosity usually does not enter the calculations, but for certain calculations is may be required, so water temperature, T, should also be recorded.

In the case of piers other than circular, pier shape and length and angle of the approach flow also are important (Appendix A, Section 7). Local scour is classified as clear-water scour when V is less than the critical velocity to initiate bed sediment motion, V_c, (Q_{si} in Equation 1 is zero), and as live-bed scour when V is greater than V_c (Q_{si} in Equation 1 is greater than zero). The change of scour depth with time from Equation 1 is:

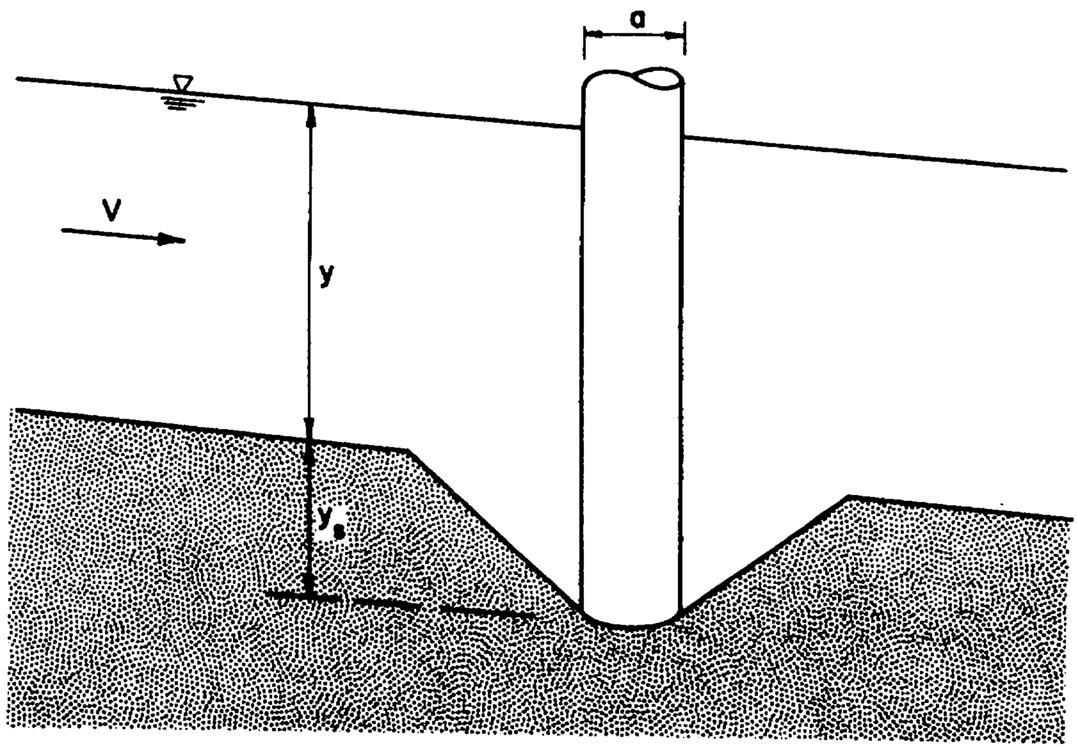

FIGURE 1

Definition sketch of pier scour

$$\frac{dy_s}{dt} = \frac{dV_o}{Kdt} = \frac{c(Q_{si} - Q_{so})}{K} \tag{4}$$

and the scour depth as a function of time is

$$y_s = \frac{c}{K} \int (Q_{si} - Q_{so})dt \tag{5}$$

where K is a coefficient relating scour depth to scour volume. The general trends of y_s as functions of time and velocity for a given pier and sediment size are sketched in Figure 2 (after Raudkivi, 1986; see also Appendix A, Figure 7). The oscillations about the equilibrium scour depth for live-bed scour are caused by migrating sand waves. Maximum scour depth is greater for clear-water scour than for live-bed scour. Raudkivi (1986) suggests that for circular piers the maximum scour depth is $y_s = 2.3(a)$. Melville and Sutherland (1988) propose $y_s = 2.4(a)$ as a design criterion for circular piers, and they identify multipliers to account for pier shape, alignment, and sediment size distribution. The work by Melville and Sutherland is a continuation of the University of Aukland studies cited in Appendix A, and these studies are based on laboratory experiments.

Attempts have been made to derive analytical expressions for local scour at circular piers (Imamoto and Ohtoshi, 1986; Tsujimoto and Nakegawa, 1986), but these models have not been tested to any great extent. The more common approach is to derive equations based on dimensional analysis and experimental data. Breusers and others (1977) review these developments, and several equations currently used are listed in Appendix A. The Colorado State University equation is typical and is used extensively, it is:

$$\frac{y_s}{y} = 2.0k_1k_2\left(\frac{a}{y}\right)^{0.65} F_r^{\ 0.43} \tag{6}$$

where F_r is the Froude number of the approach flow, k_1 is a coefficient to account for pier type, and k_2 is a factor to account for angle of attack of the flow. For cylindrical piers, k_1 and k_2 are unity (see Appendix A, Tables 2 and 3), and for a dune bed, 30 percent is added to the predicted scour depth. The equation is based on flume experiments with a sand bed. Bed

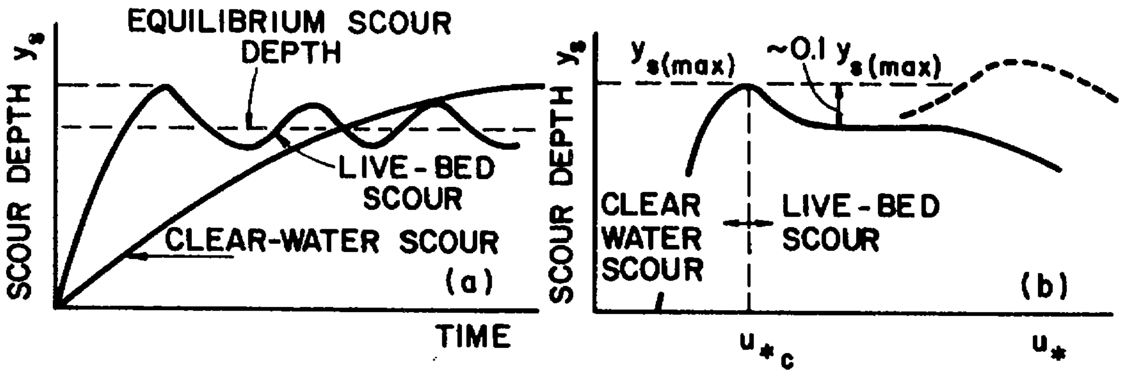

FIGURE 2

Scour depth for given pier and sediment size:
(a) as function of time; (b) as function of
shear velocity or approach velocity
(Raudkivi, 1986)

sediment size and gradation are not included; the predicted scour depths probably are conservative because if the bed contains a large range of sizes, armoring in the scour hole tends to reduce the scour depth relative to the scour depth for a uniform sediment size (Melville and Sutherland, 1988). Some equations that attempt to account for bed sediment size and gradation are listed in Appendix A.

Froehlich's equation (Appendix A, Equation 31) is of special interest because the coefficients and exponents were derived by multiple regression against field data. The data used by Froehlich are described in a later section of this report.

Abutment Scour -- The factors that control scour at abutments are the same as the factors that control scour at piers, except the pier diameter, a, is replaced by length of the abutment projected normal to the flow or some other characteristic length. For example, Liu and others (1961) propose the equation

$$\frac{y_s}{y} = 2.15\left(\frac{a}{y}\right)^{0.40} F_r^{\ 0.33} \tag{7}$$

which is very similar to Equation 6. Equation 7 is based on flume studies with a sand bed, vertical walls, and Froude numbers less than one. Scour at abutments is greatly complicated by geometry of the cross-section and the type of abutment. Various proposed equations for seven cases are given in Appendix A. Two important admonitions of the appendix should be repeated here; (1) there are few field data to test abutment scour equations, and (2) the commentaries on the various equations should be read and understood prior to attempting to use the equations for design purposes.

Summary

At any location along a stream, if the channel capacity to transport sediment is greater than the incoming sediment load, erosion takes place. If the channel capacity to transport sediment is less than the incoming sediment load, deposition takes place. Thus, long-term aggradation and degradation and the processes of scour and fill are governed by continuity considerations, Equation 1, and by sediment transport relations.

Predictions of long-term aggradation or degradation for natural stream conditions usually are difficult to make because future flows and sediment discharges have to be estimated, and in many regions, the long hydrologic records needed to make these estimates do not exist. If a river basin is developed so that future flows and sediment discharges are controlled and can be specified, reservoir releases, for example, then channel aggradation and degradation can be predicted using one-dimensional mathematical models. The data requirements, computational methods, and limitations of these models are considered by Cunge and others (1980).

Naturally occurring scour and fill usually are in response to the natural cycle or variations of water and sediment discharge and to the local geometry. General scour and fill can be predicted from sediment transport relations and bed continuity considerations, but special attention has to be paid to channel geometry. For example, wide and narrow sections of a river respond differently to changes in flow and sediment discharges and scour processes in bends are dominated by secondary flow. The field data required to carry out calculations for complex geometries and two-dimensional flows usually are not available.

Scour in a long contraction can be predicted using the methods described in Appendix A. The geometry of the approach and contracted section and the properties of the flow, the fluid, and the bed sediment are required to carry out the calculations. Some of the equations use bed shear stress or shear velocity as measures of flow strength, rather than mean velocity, so the energy slope and Manning's roughness coefficient have to be determined.

The simpler equations for predicting pier scour or abutment scour require only the approach depth and velocity, pier or abutment geometry and angle of attack of the approach flow. More complicated prediction relations include the size distribution of the channel bed material. Fluid and sediment properties usually are assumed to be constant, but in some cases fluid viscosity may be important. In any data collection program, samples of bed sediments and water temperatures should always be collected, even if they are not of immediate use.

CURRENT STATUS OF FIELD PRACTICES FOR MEASURING SCOUR

General

There are no standard procedures for collecting data on scour processes. In part, this is because scour studies are usually site specific, each bridge crossing or reach of river has its own peculiarities so investigations are tailored to the specific requirements of the site. In addition, the investigators may be constrained by limitations of resources, equipment, time, manpower, and other such factors so the kinds of data that are collected and the quality and reliability of the observations vary from one study to another. Probably, though, the reason no standard procedures have evolved is because in the United States there have been no long-term coordinated programs to collect information on scour at bridge sites.

Although procedures are not standardized, there is general agreement on the data that need to be collected (Culbertson and others, 1967; Neill, 1973; Garrett and Boyle, 1986). For both general and local scour, sequential contour maps of the streambed along the reach of interest are the most important information for determining rates and total amounts of scour. In order to predict local scour or to test the various equations that have been proposed, it is necessary to collect information on the hydraulics of the flow, the properties of the bed sediments, and the geometry of the channel and the structures, as described in Appendix A and in the previous section on theory of scour at bridge crossings.

Equipment and Methods

Earlier Observations

The earliest observations of local scour were made with fairly simple equipment, a lead weight on a sounding line to measure depth, current meters or floats to determine velocities, and a drag sampler to obtain samples of bed sediments. In spite of the relatively crude equipment, it was possible to collect valuable information on rivers where the flood durations were long, debris was not a problem, and the sites were accessible by boats or from bridges. The results reported by Inglis (1949) of scour studies at the Harding Bridge across the Ganges River is a good example of these earlier studies. Scour depths were predicted using a regime theory approach, a method that is still used extensively in some parts of the world (see Neill, 1964; 1973), so the approach depths and velocities were not determined. Also, Inglis does not include in his summary any information about the pier shapes and sizes or the abutment geometries, so his data are not very useful for testing current theories.

The classic study of local scour in the United States was carried out at the University of Iowa by Lausen and Toch (1956). Prototype measurements and scaled laboratory experiments were carried out together. In the field water surface elevations were recorded continuously with a stage recorder and scour depths upstream of the bridge pier were recorded continuously using a specially designed conductance meter.

The development of sonar and various echo and sonic sounders was probably the most important single advance made during this century in studies of local scour. These devices, when properly used, can provide reliable and rapid measures of scour depth, the shape and volume of the scour holes, and upstream and downstream bed topography. Today, they are standard equipment in most scour studies.

Current Field Practices

Present practices vary widely. Bedingfield and Murphey (1987) describe the applications of side scan sonar and fathometers for emergency detection of scour problems immediately after floods, and suggest that these devices will be useful in long-term studies where the additional flow, geometry and sediment data are collected. This study was carried out for the Massachusetts Department of Public Works, which also used divers to inspect bridge piers for scour problems after floods. Skinner (1986) describes the design criteria for a remotely operated boat that could be used to maneuver around bridge piers and abutments for soundings and describes improved designs for conductance meters that can be mounted on bridge piers to monitor both the bed and water surface elevations during floods. Mohan and San Gupta (1970) reported on dynamic cone penetration tests close to the bridge pier that can be used to determine the maximum depth of scour.

One of the more comprehensive reports of current field practices is by Jarrett and Boyle (1986), who present the results of a pilot study the purpose of which was to develop and test guidelines for collecting scour data on site during high flows. The methods use equipment and procedures commonly used in the U.S. Geological Survey stream gaging program. Pier scour data were collected at four sites. The work was done from the bridge decks and no permanent equipment was installed. No high floods were experienced and the working conditions were nearly ideal. Typically, it took a two-person crew 7 to 10 hours to collect the data at one bridge site. The report contains basic data collected during 1984 and includes recommendations on preliminary planning of scour studies, site selection, approaches, and equipment and methods. It also contains suggestions for methods of post-failure investigations.

Canadian experience and results of several scour studies are given in Proceedings of the Workshop on Bridge Hydraulics convened at Banff by the Alberta Research Council (Harrington and Gerard, Eds., 1983). The paper by Harrignton and McLean contains many examples of bed contour maps determined by sounding from a boat, which show clearly the local scour patterns. This and other reports in the proceedings deal with both local and pier scour, but not much information is given on equipment and techniques.

Programs to Measure Scour Processes

Systematic observations of bridge scour were initiated in the People's Republic of China in the 1950's and are continuing today (Kan Yi and others, 1986). No information is given on methods used to collect the field data, and so far as we know, the data have not been published and are not now available. Apparently, similar systematic studies were undertaken in the Soviet Union; Zhuravljov (1978) in his report on bridge scour equations includes a compilation of 174 sets of field data, but again, no information is given on methodology. Presumably, this program also is continuing. In Canada, the Alberta Research Council has maintained a cooperative research program with the Departments of Alberta Environment and Alberta Transportation to investigate scour processes; more than 20 years of observations have been carried out. The best sets of data from sites on four sand-bed rivers and four gravel-bed rivers have been selected for analyses and publication (David Andres, personal communication, 1988). The data are reported to be fairly complete and include some bend-scour studies. Some observations were carried out under extreme flood conditions, with a 50-year flood on the Swan River, one of the sand-bed streams, and an 80-year flood on the North Saskatchewan River, one of the gravel-bed rivers. Reports are in preparation, and the results should be available in early 1990.

Bridge scour studies in Alaska were carried out for a number of years (Norman, 1975). The results of these studies are among the data compiled by Froehlich, which are given in Appendix B. In the Alaska study and in the investigations by Jarrett and Boyle (1986), the installation of permanent equipment was not considered as a practical method for collecting scour data because water-bourne debris might destroy the sensors, the equipment is difficult to maintain in the harsh environment, and floods at the installation may not occur during the period of study. Observations from boats also were not considered practical because of the hazards of operating in the high velocities and because of turbulence around the bridge piers.

A number of field studies are underway in various states. Generally, these studies are cooperative efforts between the U.S. Geological Survey (USGS), Federal Highway Administration and the various state agencies. Usually, the U.S. Geological Survey is involved in the field data collection. Studies are planned or in progress in Virginia, Delaware, Maryland, Ohio, New York, Arkansas, Utah, Washington, Oregon, and Connecticut. In addition, the USGS has a national scour study, the emphasis of which is to involve the USGS District offices with the various state agencies in cooperative studies. Results of these studies are not yet published. So far as we can determine, no studies to collect field data on general and local scour at bridge crossings have been undertaken in the semi-arid regions of the United States, and none are currently in progress.

CURRENT STUDIES OF CHANNEL MORPHOLOGY AS RELATED TO BRIDGES

General

Channel morphology studies generally deal with channel planforms and cross-section characteristics, aggradation and degradation, and rates of lateral migration. However, many of the channel morphology studies of the Southwestern United States have dealt with the problems of arroyo (also called wash) formation which occurred over wide areas of the region through the period from about 1850 to 1920, especially from 1870 to 1890. Arroyos usually are defined as deeply incised gullies characterized by vertical or steeply sloping walls in cohesive fine sediments with flat and generally sandy beds (Cooke and Reeves, 1976). The onset of arroyo formation is often associated with man-made factors, such as overgrazing, however, even small climatic changes have triggered this process.

From the point of bridge design, the most important aspects of channel morphology are channel migration, aggradation and degradation.

Channel Migration

Most alluvial streams tend to migrate back and forth across their flood plains. The process is usually slow, with bank erosion occurring on the outside of bends and point bars forming on the inside. However, if the stream is aggrading, the shift may occur as an avulsion when the channel bed becomes high relative to the adjacent flood plains. On alluvial fans, avulsions are common.

Erosion and channel migration upstream of a bridge has the undesirable effect of changing the alignment of the flow through the bridge opening, which usually increases the local scour, and if the lateral erosion is severe enough, the piers, abutments, or approach sections to the bridge may be damaged or destroyed. Lateral erosion by stream action is a fairly common problem; in a survey of 283 bridges with scour problems, Brice and others (1978) reported lateral erosion as the major problem in 102 cases.

Problems associated with channel migration can be minimized by selecting the most stable sections of the river for bridge crossings, but in many cases this is not possible, and river training works have to be used to hold the channel and align the flow through the bridge opening.

There is no theoretical basis to predict channel migration rates; usually the rates of lateral erosion and channel migration have to be determined from repetitive surveys of monumented cross-sections or from aerial photography. Evidence of channel migration should be reported as part of any bridge inspection program.

Aggradation and Degradation

Streams in the semi-arid Southwest have gone through numerous cycles and epicycles of erosion and sedimentation (Love, 1979). Superposed on these natural cycles and epicycles during the past 100 years or so are the influences of land-use changes, dams, diversions, channelization, and other disturbances brought about by the development of the water resources of the region. The end result is that in some parts of the region, streams are actively aggrading, while in other parts degradation is occurring. For example, the channel under a bridge near Caliente, Nevada has aggraded enough to reduce the clearance under the bridge from about 15 feet to 6 feet, while a headcut on Las Vegas Wash, only a short distance away, initiated severe erosion in the valley bottom and resulted in a discontinuous arroyo that now extends some 10 or 12 miles upstream from Lake Meade and degradation of as much as 40 feet since 1968.

Both processes can lead to serious problems at bridge crossings. Aggradation can reduce the opening through the bridge, leading to overtopping by floods. Degradation can undermine piers, footings, and abutments, and in some cases is a more serious problem than local or general scour.

Predicting Morphological Changes

It is usually not possible to predict long-term morphological changes of a river in its natural state. However, the response of a stream channel to changes in its incoming flow or sediment loads or to changes in base level can be estimated qualitatively from the principles of fluvial geomorphology and quantitatively using mathematical models. For example, Graf (1983) reviewed 112 years of change in the channel of the Salt River, and Li and Fullerton (1987) describe a mathematical model developed and tested to route flows and sediment discharges for the 100-year flood along a short reach of the same river. The general approach and some additional references are given in Appendix A. Details of mathematical modeling are given by Cunge and others (1980).

Documenting Morphological Changes

The simplest way to document changes in channel morphology is to lay in a series of cross-sections along the channel and survey them periodically. The cross-sections should be marked by temporary bench marks tied to a common datum and located by coordinates in case they are lost or destroyed. If the stream has gaging stations along it, trends in mean bed elevation from the discharge measurement notes and specific stage records can be used to identify aggradation or degradation. Information from maps, survey notes, and aerial photos often can be used to determine changes in channel patterns and rates of channel migration.

COMPILATION OF SELECTED FIELD DATA

Although many field studies have been undertaken to collect data on local scour at bridge crossings, it is difficult to find compilations of field data that are complete and consistent. The reports are scattered through the literature, the methods of making observations are not standardized to any degree, and different investigators may measure and report different variables. Many of the earlier reports do not contain complete data. For example, the studies of the Ganges River reported by Inglis (1949) contain only the water discharge, the mean size of bed sediments, and the total scoured depth, $y + y_s$, measured from the water surface to the bed at the location of the local scour. Inglis used the regime theory approach to estimate the scour, which specifies the total scoured depth from the water surface as some coefficient times the regime depth. The regime depth depends only on the dominant or bankfull discharge and the mean particle size of bed material. The coefficient depends on pier or abutment shape, approach angle, degree of curvature of the channel, and other quantities that might affect the local scour.

Nonetheless, at least two important compilations exist. Froehlich (1988) has compiled 83 sets of field data for pier scour from published reports, and Zhuravljov (1978) lists data from 133 natural measurements and 41 large-scale tests in the Soviet Union. Both compilations are listed in Appendix B. Froehlich screened his data carefully, and his data sets are considered to be complete and consistent. No information about the data collected in the Soviet Union is given by Zhuravljov, but from the footnote of this table, it appears that the raw data were all reduced to equivalent scour depths that would have occurred if the piers had been round, and the scour depths listed are these equivalent depths.

For this report, no efforts have been made to analyze the field data, but the data have been plotted in order to test the scour depth design criterion suggested by Melville and Sutherland (1988). Ratios of scour depth to pier diameter, y_s/a, are plotted against ratios of approach velocity to critical velocity, V/V_c in Figure 3. All data points listed in Appendix B were plotted. Only a few values represent clear-water scour, $V/V_c = 1$, and none of

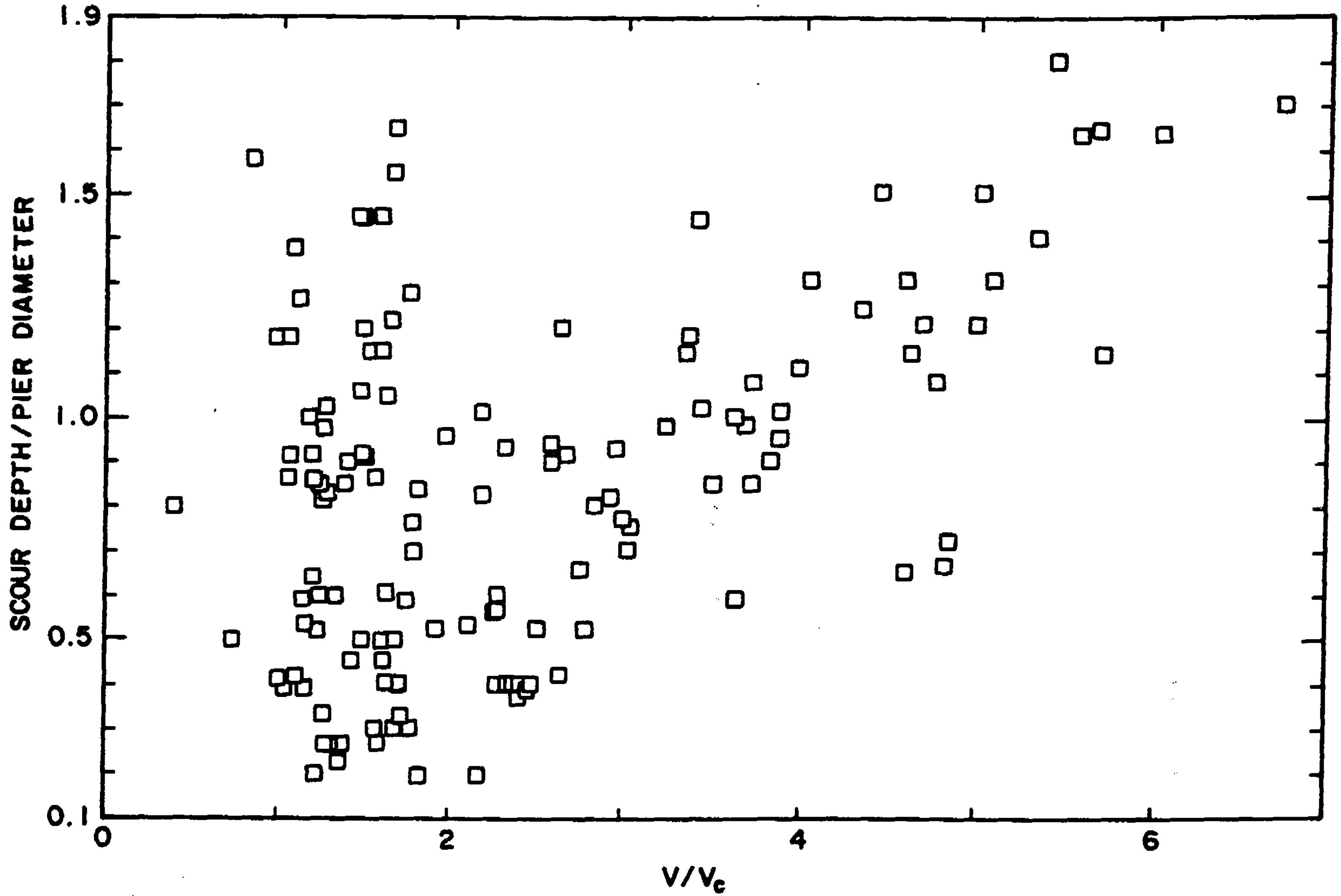

FIGURE 3

Ratios of scour depth (y_s) to pier diameter (a) plotted against ratios of approach velocity (v) to critical velocity (V_c) for data from Froehlich (1988) and Zhuravlyov (1978). Data shown in Appendix B.

the data points plot above the ratio $y_s/a = 1.9$. The design criterion recommended by Melville and Sutherland (1988), $y_s/a = 2.4$, appears to be conservative.

BRIDGE DAMAGE DUE TO SCOUR IN ARIZONA

Summary of Bridge Damages

Data on the frequency and cost of damages to bridges and transportation systems in Arizona provides a framework for the magnitude of bridge scour in Arizona. These data are also valuable in identifying stream systems that have experienced the most frequent and costly bridge repairs. A comprehensive data base of bridge damages and cost of repairs or replacements is not available; however, data are available to adequately assess the cost of damages and to identify the streams that have a history of severe flood damage to transportation systems.

A joint review by the Arizona Department of Transportation and the Federal Highway Administration (1979) reported that flood damages to roadways located on the Federal-Aid System in Arizona exceeded $30 million during a less than 2 year period. These damages were the result of four major storms that occurred in October 1977, March 1978, December 1978, and January 1979. These floods occurred after a relatively long period of very infrequent major storm activity in Arizona. Therefore many of the bridges that were damaged or lost to service during that time period had not experienced major flood events prior to that time.

A summary of Emergency Repair Projects and costs have been reported (Simons, Li and Associates, Inc., 1988). These are summarized in Table 1. It is noted that these projects are located on only 9 of the river systems in Arizona, and that bridges on other river systems may be subject to similar damage if infrequent and isolated severe storm events were to occur in these river systems.

Damage to transportation systems in Arizona have been reported by the Corps of Engineers in various Flood Damage Reports, and the cost of these flood damages have been compiled (Simons, Li and Associates, Inc. 1988). Table 2 illustrates the random nature of bridge flood damage in Arizona in both time and space.

TABLE 1

Summary of Emergency Repair Projects for bridge damages due to flooding in Arizona

River System	Number of Repair Projects	Cost of Repairs in $1000
(1)	(2)	(3)
Gila River	20	7,668
Hassayampa River	2	634
Agua Fria River	7	5,813
New River	4	18
Salt River	13	26,263
Santa Cruz River	19	6,941
Rillito and Pantana Rivers	12	684
San Pedro River	2	113
Verde River	1	290
	Total Cost	$48,424

TABLE 2

Summary of Flood Damages to Transportation Systems in Arizona
(from Table 3.3, Simons, Li and Associates, Inc., 1988)

All values in $1,000

River	Dec 1965 Jan 1966	Oct 1972	Oct 1977	Flood Feb-Mar 1978	Dec 1978	Feb 1980	Oct 1983	Total for River
Salt River, Granite Reef Dam to Gila River	1,686							1,686
Gila River, to Gillespie Dam	91							91
Gila River, Safford Valley, Graham County		227						227
Gila River, in Duncan and York Valleys, Greenlee Cnty		1						1
San Francisco River at Clifton		184						184
Nogales Wash, Santa Cruz County			69					69
Santa Cruz River, Santa Cruz County			682					682
Santa Cruz River, Pima County			784					784
Santa Cruz River, Pinal County			54					54
Salt River, from Granite Reef Dam to 115th Avenue				11,809				11,809
Gila River , Maricopa County				340				340
Salt River, Metro Phoenix					17,985	16,339		34,324
Gila River, Metro Phoenix					1,526	1,360		2,886
Agua Fria River, Metro Phoenix					1,999	4,242		6,241
All Rivers within Pima County							28,000	28,000
All rivers within Greenlee County							4,320	4,320
All rivers within Santa Cruz County							3,880	3,880
All rivers within Graham County							1,660	11,660
Total for Flood	1,777	412	1,589	12,149	21,510	21,941	37,860	97,238

These statistics are presented to illustrate the magnitude of bridge damages that have occurred from floods in Arizona, and the erratic and unpredictability of bridge damage due to floods of infrequent occurrence and limited aerial extent in Arizona. Therefore, there are numerous waterway bridges in Arizona that have not experienced scour or flood related damage; however, this successful record of performance of those bridges may be more related to the absence of major flood events rather than an ability to withstand such damage during a flood discharge.

A scour project list was obtained from the ADOT Structures Section. This list shows the bridge scour countermeasure projects that have been undertaken by ADOT since 1979. The cost of countermeasures to protect existing bridges against scour are shown in Table 3. Since the initiation of the bridge scour projects, over $7 million has been expended to construct countermeasures and 120 or more bridges have been investigated for the need to initiate countermeasures.

TABLE 3

Cost of bridge scour countermeasures in Arizona

Year	Construction Cost in $1,000	Accumulated Cost in $1,000
(1)	(2)	(3)
1979	219	219
1980	2,659	2,878
1981	1,290	4,168
1982	694	4,862
1983	286	5,148
1984	465	5,613
1985	451	6,064
1986	1,079	7,143
1987	0	7,143

Selected Case Histories

General

Incidents of bridge scour and reports of channel degradation and aggradation at bridge crossings have been investigated. The source of this information and data, unless otherwise referenced, has been the periodic bridge inspection reports and miscellaneous office memorandums that are contained in the files of the ADOT Structures Section, Bridge Operations Services, Maintenance Branch. It is not possible to provide a comprehensive review of all bridge scour and channel stability problems in Arizona because the necessary records are not available. For the incidents that are reported herein it is not always possible to clearly and confidently relate the hydrologic and hydraulic factors that led up to and resulted in the damage or failure of the bridge. Post-flood analyses and adequate documentation of hydrologic and hydraulic conditions at bridges are seldom performed. ADOT recognizes the need for and the value of such post-flood analyses and documentation of hydrologic and hydraulic conditions at bridges, however, ADOT does not have the personnel or resources for performing these activities.

The following are selected incidents of bridge scour and bridge related channel instabilities. These incidents are presented so as to assist in better defining the needs of bridge scour monitoring programs in Arizona.

I-10 Salt River Bridge (Structure No. 882)

This bridge was designed in 1960-1961 and constructed in 1965. The bridge is 1,569 feet long and is supported on 19 piers with spread footings. A 250-foot wide low-flow channel was constructed in 1965, and this channel was riprap lined to a thickness of 4 feet. The bottom elevation of the low-flow channel was Elevation 1078.0. Piers 6 through 9 were in the low-flow channel. The bottom elevations of the pier footings as-built are approximately:

Piers 1 through 5 - about Elevation 1072,
Piers 6 through 9 - Elevation 1066.70 to Elevation 1066.97,
Pier 10 - Elevation 1073.49, and
Piers 11 through 19 - Elevation 1077.77 to Elevation 1077.11

No scour analysis was made, but footings were placed at an elevation about 22 feet minimum below the river bed (except for piers 6 through 9 in the low-flow channel), and this embedment depth included an allowance of 3.5 feet for scour as noted in the pier design calculations. The embedment depth for piers 6 through 9 was about 9 feet in the low-flow channel (Sverdrup & Parcel and Associates, Inc., 1979).

The bridge has been subjected to a series of flood events as shown:

March 1978 - 120,000 to 130,000 cfs,
December 1978 - 120,000 to 130,000 cfs,
January 1979 - about 80,000 cfs,
March 1979 - about 50,000 cfs, and
February 1980 - about 170,000 cfs.

The bridge was designed for a 50-year flood event with a peak discharge of 170,000.

Pier 11 settled about 9 inches during the March 1979 flood. The reported cause of the pier settlement was the result of lateral movement of the channel thalweg during the March 1978 flood. The riprap lined, low-flow channel was not effective in controlling the main channel of the river at flood conditions. The pier undermining was probably the result of general scour due to lateral migration of the thalweg and local scour at the pier.

River bed scour data were obtained using boat mounted sonar during 27 February through 5 March 1980 during the recession of the February 1980 flood. These data were obtained for ADOT by Dames and Moore, Inc. to monitor the scour and placement of riprap during the flood. The maximum scour depth measured was upstream of pier 9 at an Elevation of 1068.5 (footing Elevation 1066.97). The discharge during the monitoring was less than 14,200 cfs which is considerably less than the estimated peak discharge of about 170,000 cfs that occurred about 12 days prior to the sonar measurement program. The maximum scour depth at peak discharge is unknown.

This bridge was replaced in 1986 (Structure No. 2003), and the older bridge was removed.

SR 74 Agua Fria River Bridge (Structure No. 1620)

This bridge was designed in 1968, redesigned in 1971, and constructed in 1972. The bridge is 405 feet long and is supported on four piers with spread footings. Pier No. 1 has steel piling and a pile cap with the piling driven to bedrock. The river bed elevation at the time of construction was about Elevation 1390 and the deepest footing is for pier 2 with Elevation 1374.61 (less than 15-ft embedment depth). The abutments and roadway embankments were lined with RB-2 Type A bank protection. The embankments were faced with rock rubble above the railbank. The bridge was designed for a 50-year flood with a peak discharge of 15,000 and the bank protection was designed for 25,000 cfs. The bridge is located about 2 miles downstream of Lake Pleasant, a reservoir formed by the construction of Waddell Dam in 1927.

In March 1978, 16,000 cfs was released from Lake Pleasant because of high runoff. No damage to the bridge was reported. In December 1978, about 60,000 cfs were released to prevent overtopping of the dam. Inspection of the bridge after the December 1978 flood indicated that pier 1 had settled about 5 to 6 inches, the 7-ft diameter pier column had cracked, and the superstructure was shifted laterally at the west abutment.

The reported cause of the bridge damages was the result of the December 1978 flood flows overtopping the railbank protection by 2 to 3 feet and eroding the west abutment, erosion of the abutment riprap, and undercutting of the toe (Sverdrup & Associates, Inc., 1979). This resulted in the flow being reoriented toward pier 1 which led to streambed scour and pier settlement.

The west abutment and pier 1 were constructed in the streambed containing coarse gravel, cobbles, and boulders of undetermined depth. The remaining three piers were founded on bedrock. Pier 1 was founded on erodible material at an elevation 1.80 feet higher than the pier 2 foundation on bedrock. The riprap abutment protection may have been adequate for the design flow of 25,000 cfs; however, the riprap toedown depth was not deep enough nor the bank protection high enough for the 60,000 cfs flow event.

SR 89 Santa Cruz River (Sahuarita) Bridge (Structure No. 405)

This bridge, built in 1953, is 332 feet long and is founded on 10-inch steel piles (10BP42). During the flood of 1 October 1983 the bridge suffered scour damage due to a combination of high flows and debris load on the piers. Pier 4 rotated vertically with the upstream end settling about 0.61 foot. The bridge was repaired by jacking the superstructure and placing shims under the upstream girders of pier 4. The repaired pier was satisfactorily tested with a load of 106 tons. The bridge was subjected to another severe flood without incident on 28 December 1984 that completely submerged pier 4.

In June 1985, while working on a contract for channel rehabilitation to prevent further scour problems, excavation around the upstream end of pier 4 revealed extensive damage to the steel piles. The first four piles were bent back completely beneath the pile wall and approximately one-third of the pier was resting on logs, branches, debris, and loose alluvium. Evidently, this had happened during the October 1983 flood and the damaged piles and submerged debris had been buried during the recession of the flood. The damaged piles and highly scour susceptible pier would not have been discovered had it not been for the rehabilitation work.

<u>I-19 Santa Cruz River Bridges (Structure Nos. 1243, 1244, and 1547)</u>

This location consists of three bridges; the northbound I-19 bridge (Structure No. 1243), the southbound I-19 bridge (Structure No. 1244, and the Mission Road ramp bridge (Structure No. 1547). The I-19 bridges were constructed in 1967, and the design discharge was 22,000 cfs. Heavy flood runoff during 9-10 October 1977 caused damage to the south abutments of the northbound and southbound bridges and to the west abutment of the Mission Road ramp bridge. The flood had a duration of about 18 to 20 hours with a peak discharge of about 22,000 cfs. The depth of flow was about 10 feet. Damage was similar for the three bridges. The sheet piling protection around the abutments was eroded from behind and the embankment fill material was piped through the piling. The piling was bent over towards the piers. The streambed had degraded about 5 feet since the previous inspection.

The bridges were repaired and put back into service. On 19 December 1978 the Santa Cruz River flooded again. The flood had a duration 7 to 8 hours and a peak discharge of about 12,000 cfs. This time, the wire-tied bank protection above the sheet piling on the north abutment of the northbound bridge failed. During the flood of October 1977 the thalweg was adjacent to the south abutment of the bridges. Repairs to the bridges resulted in re-alignment of the low-flow channel to prevent further damage to the sheet piling at the south abutment. Perhaps because of this re-alignment the north abutment was more directly attacked. The flow conceivably undermined the fill materials behind the bank protection which resulted in some damage to the embankments. The railbank protection along the roadway embankment upstream of the north abutment was undercut by the streambed degradation.

The sheet piling protection along the west abutment of the Mission Road ramp bridge along with the railbank protection and the wire-tied riprap were severely damaged. The front row of pilings along the west abutment were totally exposed.

After the 1978 flood the entire bank protection was extensively rebuilt by removing the sheet piling and replacing it with loose riprap. This repair was monitored after each runoff event and some settlement and movement of the riprap was observed. A flood on 4-6 February 1983 with a peak discharge of about 5,000 cfs resulted in the loss of about 150 feet of the dumped riprap on the north abutment of the northbound bridge. It was also reported that there was other minor damage including some scour of the mainline bridge piers. This is the first account of pier scour being evident at this bridge. Recommendations were made for additional bank protection including a spur dike tied into the bedrock bank about 300 feet upstream of the bridge.

On 3 October 1983 a major flood on the Santa Cruz with a peak discharge of about 45,000 cfs occurred which resulted in the erosion of the north bank protection and loss of the north abutment of the northbound bridge. With the loss of the abutment a span of the bridge deck collapsed.

<u>I-19 Agua Fria Canyon Bridges (Structure Nos. 353 and 906)</u>

The northbound bridge (Structure No. 353) was built in 1951 and the southbound bridge (Structure No. 906) was built in 1967. In 1968 C-23 Type A bank protection was added to the upstream south approach roadway, and later C-17.01 Type 2 railbank protection upstream and between the two bridges was added. In 1982 a concrete floor and spur dike were added to the bridges for scour protection and flow alignment.

Flooding of Agua Fria Canyon occurred on 9 October 1977 with a peak discharge of about 11,000 to 12,000 cfs and a peak duration of about 2 hours. Stream flows can be affected by storage in Pena Balance Lake which is located about 7 miles upstream. The Frontage Road bridge which is on old Highway 89, and is located about 700 feet upstream of I-19 was destroyed. The bridge apparently failed due to scour of the spread footings supporting the piers because the abutments and upstream railbank protection remained intact. The flow crested at a stage about equal to the bridge deck elevation. A 100-foot overflow section adjacent to the south abutment did not provide adequate conveyance to prevent loss of the bridge. This bridge was replaced in 1979 by Structure No. 1703.

At the southbound bridge (Structure No. 906) flow from the October 1977 flood overtopped the north abutment by about 1 foot over the roadway. This overtopping eroded a section of roadway embankment and undermined the approach slab to the bridge. The north channel bank upstream of the bridge moved about 20 to 30 feet laterally exposing the north abutment. The northbound bridge (Structure No. 353) was not seriously damaged. Flow did not overtop the bridge or approaches. Relatively minor scour occurred behind the downstream wing walls of both the northbound and southbound bridges. Erosion also occurred behind the upstream north wingwall of the northbound bridge as flows that overtopped the southbound bridge re-entered the watercourse in the median and flowed under the northbound bridge.

Stream bank protection was repaired and extended as a result of the 1977 flood. On 9 October 1983 the bridges were subjected to a flow of about 7,000 cfs. About 125 feet of the north upstream railbank protection and about 30 feet of dumped riprap on the nose of the south spur dike were damaged.

SR 160 Hamblin Wash Bridge (Structure No. 531)

This bridge was damaged during the flooding of October 1983. The dumped riprap at the east abutment was almost completely eroded for a length of about 40 feet. The channel degraded about 2 feet under span number 1 and about 1 foot under span numbers 2 and 3, and the channel aggraded as much as 2 to 3 feet under span numbers 6 and 7.

Piers 1 and 2 settled about 2.4 inches. The foundation records indicate that the H piles were driven through alluvium to or near the top surface of hard red clay. These are apparently relatively short piles which may have contributed to the loss of foundation capacity.

I-10 Rillito Creek Bridges (Structure Nos. 391 and 854)

These bridges have been subjected to channel degradation. During 1981 to 1988 the channel cross-sections in the bridge inspection report indicate a degradion of up to 8 feet. As of the 1985 bridge inspection,`the piles were exposed about 2.4 feet beneath the pile caps for abutment number 1 and piers 1 through 4 of the westbound bridge, and the piles were exposed about 2.5 feet beneath the pile caps for piers 1 through 7 of the eastbound bridge.

During the October 1983 flood the north Frontage Road and embankment were overtopped and about a 50-foot wide section was washed out behind the abutment, and much of the bank protection and riprap were eroded.

I-10 Gila River Bridge (Structure Nos. 1085 and 1086)

The south approach embankments to both of these bridges were washed out during the 5 October 1983 flood. The bridges cross a wide, flat floodplain and have a structure length of about 1,000 feet. The low-flow channel is located along the south abutment. A gravel pit is located about 1,000 feet downstream of the bridges and is protected from flooding by a low, "soft" dike. The 50-year design discharge is 33,000 cfs at a stage of 1192. The peak discharge during the flood has been estimated at 60,000 cfs and stage of 1197.

During the flood the floodplain was 4,000 to 6,000 feet wide, and the south overbank flow was estimated at about 50 percent of the total flow. No damage occurred prior to the failure of the downstream dike around the gravel pit. Backwater may have inhibited the flow contraction into the bridge waterway thus avoiding a local scour. Upon failure of the dike, the roadway approach embankment began to scour immediately and progressed rapidly.

The cause of the failure was reported to be the result of local scour caused by a combination of extremely large flows in excess of the design discharge, large overbank flow being constricted at the bridge opening, inadequate bank protection for this discharge (the bank protection was overtopped by about 3 feet), and a backwater that was rapidly drawn down after overtopping of the downstream dike. The velocity of the ponded upstream water probably accelerated greatly when the dike was breached and flood flows entered the gravel pit and this may have precipitated the failure.

About 400 to 800 feet of the south approach roadway was lost. Soundings taken several days after the failure indicated a scour hole adjacent to the south abutment of about 14 feet deep. The maximum scour depth at peak discharge is unknown.

Post-flood measurements showed that the new low-flow channel is about 2 feet higher than that existing when the bridge was built. Channel aggradation may be occurring for a variety of reasons including the constriction of the flood plain to about 25 percent of its normal width.

U.S. 89 Moenkopi Wash Bridge (Structure No. 612)

This bridge has experienced periodic channel aggradation problems. This may be caused by the heavy growth of tamarisk and by the meandering downstream channel that have resulted in reduced flow velocities and reduced sediment transport capacities.

SR 264 Polacca Wash Bridge (Structure No. 1014)

This bridge has been experiencing a consistent trend in channel aggradation since its construction in 1969. The streambed has undergone about 7 feet of aggradation during the period 1955 through 1983. Measurements of the cross-section at the bridge and calculations of the discharge capacity are shown:

Year	Cross-Section Area square feet	Discharge Capacity cfs
1955	3,766	57,000
1976	1,892	18,900
1980	1,591	12,900
1983	1,431	10,600

Field inspections have been conducted to determine the cause of the continuing aggradation but no geomorphic factors have been positively identified. It has been observed that springs occur locally within the drainage basin. Tamarisk with stands of cottonwood trees exist in the vicinity of the bridge for several miles both upstream and downstream. Also there are small fields of cultivated corn scattered along the bottom of the wash for several miles downstream of the bridge. About 5 miles downstream of the bridge the riparian vegetation reverts to that characteristic of high desert. This bridge was replaced in 1986 by Structure No. 2010.

PART 2 - IMPLEMENTATION OF PROGRAMS IN ARIZONA

GENERAL

This research has resulted in recommendations for three general types of activities that should be undertaken by ADOT:

1. The development of bridge data files.
2. The implementation of certain bridge scour pilot programs.
3. The implementation of a phased program for monitoring aggradation, degradation, and lateral migration.

Ultimately, the results of these three activities must serve the needs of ADOT in regard to the design of new bridges, the Bridge Scour Team, the design of bridge scour countermeasures, and the bridge inspection program, and therefore, ADOT must assume responsibility for these activities. However, it is recognized that ADOT has constraints in regard to staff time and capabilities. Therefore, ADOT may have to contract some or most of the activities that have been recommended. Long-term field data collection programs should be negotiated directly with the USGS. ADOT is presently communicating with the USGS, Arizona District, in this regard and an agreed scope-of-work should be negotiated and implemented. Some of the recommendations, particularly for the bridge scour pilot programs, require special qualifications that are normally beyond the capabilities and staff limitations of ADOT. The USGS may want to include some of these within its scope-of-work, if so, these should be negotiated with the USGS. However, ADOT may desire to implement some of the recommendations that neither ADOT nor the USGS can undertake, and in that case, a request for proposals should be prepared and the work carried out under contract. Finally, because of the need for a cooperative effort between ADOT, the USGS, university research groups, and possibly one or more consultants, a special consultant should be retained to serve in a review and advisory capacity. The special consultant would assist ADOT in defining the scope-of-work to be negotiated with the USGS, would assist ADOT in organizing and implementing its bridge scour activities, and would assist in prioritizing projects for which consultant services would be required. The special consultant could also be retained to be available for other assignments, such as the immediate response to bridge scour incidents, for review of the design of new bridges and scour countermeasures, and for assistance in plans to install scour measuring equipment at new bridges.

BRIDGE DATA FILE

General

A bridge data file is a compilation of information concerning the design and construction, scour countermeasures (if any), bridge scour and flood related incidents, bridge inspection data, flood records, scour data, and information relating to the hydraulics, hydrology, and geomorphology of the watercourse. The intent of establishing and maintaining bridge data files is to have a record of each bridge so that potential scour and channel instability can be periodically and systematically monitored, and so that statistics of bridges can be compiled and analyzed in regard to assessing bridge design procedures and success of scour countermeasures. Specifically, the bridge data files will provide information to:

1. Assess the susceptibility of each bridge to scour and channel instability problems,
2. Develop a statistical base for the assessment of the relative success of various design and construction methods to protect bridges against scour,
3. Identify bridges that should be monitored for scour and channel instability, and
4. Identify bridges that are in need of bridge scour countermeasures.

Two types of data files should be developed for each bridge. One would be a document file containing engineering drawings, photographs, reports, memorandums, and narrative descriptions of flood events and scour incidents. Presently, ADOT maintains a file that contains the inspection records for each bridge. This file should be expanded into the recommended document file with the bridge inspection record being a subset of that file. The other would be a computerized data base file for each bridge containing statistics, data, and key information on the bridge. A comprehensive computerized data base file should be produced that would be a summary of key statistics for each bridge in regard to design and scour.

ADOT presently has a comprehensive computerized data base file and it may be possible to expand this existing file, as needed. This existing file has been produced by using commercially available software. It is understood that this file is not at capacity and that more columns of data could be added.

This software has numerous data management options including importing and exporting files, sorting data, rearranging the data columns for output files, and other format functions. This software is maintained and executed on personal computers with hard disks that are used by the ADOT Bridge Section. Such a data file can be used to identify bridges that may be susceptible to scour. Informative tables could be produced from this data base or an expanded data base. It is recommended that this system be evaluated for use in producing the individual bridge data base files described below.

Eventually, the bridge data files should serve the needs of management, design, operation and maintenance, and research. Logically, the files should merge into or be compatible with some kind of data information system, such as a GIS (geographical information system), with scanning capabilities for data entry so that plans, figures, photographs, and entire documents can be entered into computer files. Most commercial spread sheets are compatible with existing information systems.

The bridge data files are a central part of the long-term bridge scour monitoring program, and this program should not be considered fully implemented until these bridge data files are produced. However, some of the pilot programs and monitoring programs could proceed concurrently with the generation of the data files. The interaction of the pilot and monitoring programs to the bridge data files is shown in Figure 4.

The following is a description of the data that should be compiled and entered into these files. Initially, there would have to be a significant effort to compile the necessary data and to enter the data into the files. Subsequently, both the document files and the computerized data base files would be updated after the biennial inspection of each bridge, after major repair projects and bridge scour countermeasure projects, and after major floods and/or bridge scour incidents. Because of the effort that will be required, the initial files should be generated with the assistance of an engineering consultant; the files should be updated periodically and maintained by ADOT.

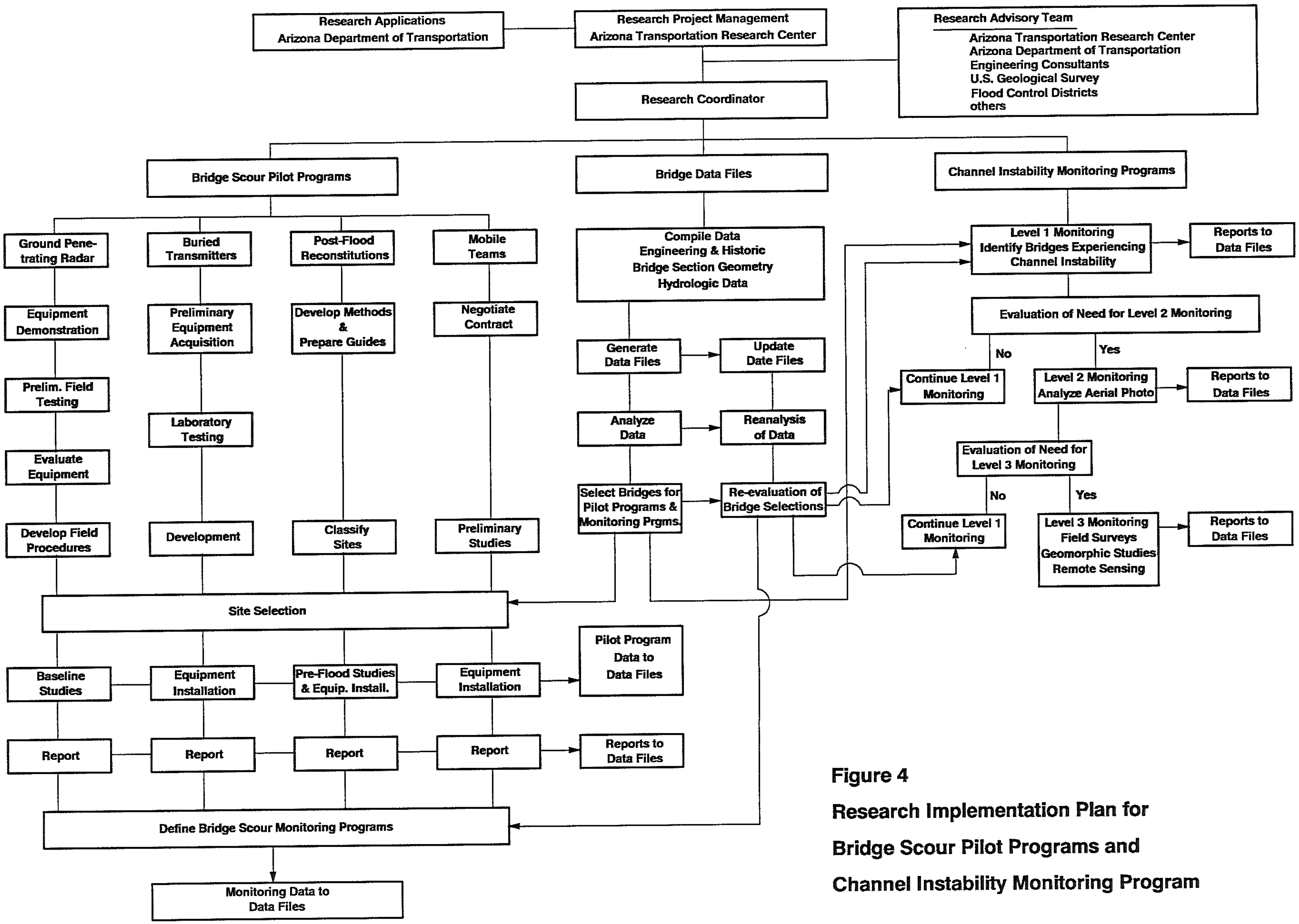

Figure 4
Research Implementation Plan for
Bridge Scour Pilot Programs and
Channel Instability Monitoring Program

Engineering and Historic Data

It is recommended that a reasonable effort be expended to compile engineering and historic records for each bridge into the document file. This work element should be conducted as a cooperative effort of ADOT and an engineering consultant (EC). ADOT staff would be responsible for compiling the basic data and the consultant would be responsible for preparing the narrative descriptions of bridge incidents, and performing the aerial photograph search. The following are suggested:

Activity	Responsibility
1. Engineering drawings of piers, abutments, and flow alignment works.	ADOT
2. Dates of: a. Original construction, b. Repairs and rehabilitations affecting the bridge hydraulics, and c. Bridge scour countermeasures, if any.	ADOT
3. Latitude and longitude of bridge (needed to locate aerial photographs and remote sensing data).	EC
4. Records and descriptions of hydraulic bridge incidents, if any.	ADOT/EC
5. All available photographs of the bridge, with annotations.	ADOT
6. List of aerial photographs (with dates, scale, and source of photography) that are available from the National Cartographic Information Center (USGS, Denver) and other sources.	EC
7. Copies of all reports and memorandums concerning bridge hydraulics and floods.	ADOT

Bridge Section Geometry

A systematic approach for evaluating the vulnerability of existing bridges to scour is outlined in FHWA (1988). Bridges vulnerable to scour damage are called "scour critical" bridges. The recommended procedures are designed to identify bridges that may need to be repaired or replaced or that need scour countermeasures or bridges that require more intense bridge scour monitoring programs. The bridge data files proposed here should assist in the evaluation. As part of the evaluation, data available in the bridge inspection files on the geometry of the waterway collected during each biennial inspection need to be compiled and analyzed so that monitoring programs can be defined and bridges needing scour countermeasures identified.

The bridge waterway geometry data should be digitized, and these data analyzed for trends and sudden changes. All the data from the bridge inspection files should be digitized and all new data should be digitized as it becomes available. The data would be entered into a data file and analyses of the data conducted.

The bridge geometry data from the bridge inspection program are obtained during the biennial inspection of each bridge. The field measurements are made by dropping a tape from the bridge deck and measuring the distance to the channel bed. This measurement is then plotted in the field on a sketch of the bridge waterway. Presently, the measurements are not recorded. This procedure could result in some errors because of mistakes in plotting the data in the field, and digitizing from these field sketches would not be as accurate as using the recorded measurements. It is recommended that this field procedure be reviewed by ADOT in light of the need for recording and analyzing these data and that consideration be given to recording the measurements in the field in addition to making the field sketches.

The data analysis would consist of calculation of bridge waterway area, thalweg width, mean channel bed elevation, and minimum thalweg channel bed elevation. Graphs would be prepared showing these variables plotted against time. These graphs would be used to identify trends or sudden changes in channel geometry. These channel geometry changes may be related to flood events or activities at the bridge that would be documented in the historic and hydrologic data files.

It is recommended that the initial data compilation be undertaken by a consulting engineer. The consultant would select the data analysis software (possibly the software in present use by ADOT) and would design the data base system. The consultant would use these data to perform the first data analysis. The software and files would be delivered to ADOT, ADOT staff would be trained by the consultant, and all subsequent analyses of data would be performed by ADOT.

This work element would be conducted as follows:

Activity	Responsibility
1. All existing bridge waterway geometry data would be digitized.	EC
2. A data system and analysis program would be selected and designed.	EC
3. The data would be entered into the data file, analyzed, and a summary of analyses results prepared.	EC
4. The software and files would be delivered to ADOT and ADOT staff trained in its use.	EC
5. Files would be maintained and updated analyses performed each subsequent year.	ADOT

Hydrologic Data

Hydrologic data of interest to a bridge scour program consist of frequency and magnitude of flood peaks, frequency of mean daily discharges, and discharge-stage relations. These data are available for streamgaging stations maintained by the USGS. Only a limited number of bridges will have streamgages in the near vicinity for which such data will be available. However, it is likely that stream gage data will be available for many of the major watercourses and these data should be analyzed where available. These data, although available only for a limited number of bridges, may provide the necessary documentation of the hydrologic aspects of bridge scour and channel instability to allow extrapolation of relations to ungaged sites.

The hydrologic analyses that are recommended are:

Activity	Responsibility
1. Flood frequency analyses - The USGS is presently performing a flood frequency analysis for Arizona and the Southwest and the results of this study should be available during the next two years. These results should be incorporated into the appropriate bridge data files when they become available.	USGS
2. Flow-duration analyses - Flow-duration curves should be prepared for each streamgage station for which flood frequency is being performed. This will provide information on the frequency of discharges in watercourses, and will be of value in selecting sites for pilot and monitoring programs because they will indicate which bridges are more likely to have discharge events.	USGS
3. Bed elevation trends and stage-trend analyses - A stage-trend analysis is a time series analysis of the stage associated with a specified discharge at a gaging station. Stage-trend analyses will indicate watercourses that have undergone some change in hydrologic regime; for example, aggradation, degradation, channelization, and floodplain encroachment.	USGS

Hydrologic, Geomorphic, and Other Data

Other file data such as channel plan and profile, bed and bank material, geomorphic data, and any information on hydrologic characteristics will need to be compiled from ADOT and USGS files or will need to be collected. If extensive field work is involved this should probably be contracted to the USGS, universities, or consultants.

Summary of Data Files

The document file on each bridge would consist of the following:

1. Engineering drawings
2. Chronology of construction, repairs, rehabilitations, and flood incidents
3. Descriptions of flood incidents
4. Photographs
5. List of available aerial photographs
6. Copies of all pertinent reports and memorandums
7. Flood frequency data and analyses
8. Flow-duration table and graph
9. Stage-trend analysis
10. Bridge section geometry from the bridge inspection reports
11. Results and graphs of bridge section analyses
12. Biennial bridge inspection records
13. Information on channel bed and bank material
14. Any cross section and channel plan and profile data

A computerized data base file would be developed for each bridge that would contain as a minimum the following entries:

1. State route and mile post number
2. ADOT structure number
3. Latitude and longitude
4. Waterway name
5. Year of construction/Year(s) of repairs and rehabilitations
6. Year of scour countermeasure project
7. Design discharge, cfs
8. Streamgage available (Y/N)/Name and station number
9. Flood frequency table; 100-yr, 50-yr, 25-yr, and 10-yr discharge (if streamgage data is available)
10. Flow-duration table (if streamgage data is available)
11. Stage-trend analysis (if streamgage data is available)
12. Bed and bank material information
13. Channel plan, profile, and cross section data

14. Bridge Section Geometry table (for each year of inspection)

Year

Waterway area

Thalweg width

Mean channel/bed elevation

Minimum thalweg channel bed elevation

Use of Bridge Data

The bridge data files should be used to identify bridges that are susceptible to scour and channel instability problems and to select bridges for monitoring programs especially monitoring programs for general scour, aggradation, and degradation. The files would be very beneficial in post-flood reconstitutions. The data files would also be the repository for all scour data from the pilot and monitoring programs. Eventually, as scour data becomes available, these data files would be used to test, modify, and develop bridge scour predictive equations and procedures, such as those recommended by FHWA (1988), and to store data for boundary conditions and input for mathematical models.

As previously indicated, the Bridge Section of ADOT presently has a computerized data file with valuable data on all of its bridges, and a bridge inspection file is also maintained. However, the existing files probably cannot be used efficiently to set priorities for scour countermeasures or monitoring; for example, the frequency and magnitude of flood flows, channel bed and bank materials, and history of channel aggradation or degradation, and other factors will influence potential bridge scour. Therefore, it is recommended that these files be expanded as described.

It will be possible to quickly and economically extract information from the suggested bridge data files. For example, it would be possible to list bridges for which the mean channel bed elevation has been decreasing, or to list bridges for which the waterway area has been increasing, either of which would indicate active general scour or channel degradation; or to list bridges for which the waterway area is decreasing indicating channel aggradation. The data files could also be used for a variety of administrative and management functions; for example bridges could be listed according to date of construction, state route, frequency of maintenance, etc.

The immediate use of the bridge data files would be to perform the necessary data searches to identify bridges that are experiencing channel aggradation, degradation, or scour. Upon completion of this computerized search it would be possible to select and prioritize bridges for the monitoring program for channel instability. The data search, sort, and output files should be developed by a consultant, and the consultant should implement the files for the purpose of selecting and prioritizing bridges for monitoring programs.

BRIDGE SCOUR PILOT PROGRAMS

General

Before long-term data collection programs are undertaken in Arizona on scour at bridge crossings it will be useful to implement some pilot programs of short duration and limited extent. The purposes of the pilot programs are to develop methodologies; to test equipment, techniques, and procedures for data collection; to provide opportunities for training and for the preparation of guidelines, instructions, training and techniques manuals; and to identify possible areas for needed research and development. Some useful data on scour and related processes can also be collected in connection with the pilot projects, but it is likely that only a long-term program can provide the information over the full range of hydrologic, geographic, and sedimentary conditions that are of interest to ADOT.

This report outlines several pilot projects and recommendations for their implementation. These are discussed as separate activities, but there is no reason why they could not be combined into several or a single project or incorporated into a longer-term data collection program if ADOT desired to do so.

Types of Data Collection Programs

Data collection programs can be implemented at various levels of costs and complexities. Information about scour at bridge crossings and related processes can be obtained by the following types of programs: (1) compilation and analyses of data on past bridge failures, (2) post-flood reconstitution, (3) direct observations by mobile teams, (4) direct observations at sites with fixed installations, and (5) physical model studies. Each of these approaches has advantages and limitations that are discussed in the following subsections, and probably, an effective program for ADOT should incorporate appropriate elements from each.

Analyses of Past Bridge Failures

As previously discussed, permanent records of bridge failures and floods should be maintained, and all relevant data should be entered into computer data files to facilitate access and analyses. Statistical summaries should be updated periodically to provide information about the types of failures and their distribution in time and space, flood return periods, remedial measures, costs, etc. Information of this kind should be used in planning programs and budgets, in identifying design deficiencies, if they exist, in setting priorities for research and development, and other related managerial activities. However, it is unlikely that much quantitative information on hydraulic conditions and scour depths can be extracted from historical records of failures, because records are often incomplete.

Post-Flood Reconstitutions

In this approach, flow parameters are estimated by indirect discharge measurements (slope-area or width-contraction methods) and scour depths are estimated by direct observation, probing, excavation, penetration tests, ground penetrating radar, or other types of post-flood examinations. This might be the only way to collect data on many Arizona streams, but the data are not likely to be very accurate, and usually it is impossible to determine when the maximum depth of scour occurred or to identify the time-history of the scour development. The method becomes more effective if some advance work is done at the site so that scour depths can be estimated by tracers, buried chains, motion-activated radio transmitters, ground penetrating radar, etc. Successful application requires experienced personnel and quite a bit of field work, but fairly simple equipment.

Mobile Teams

This approach is described and evaluated by Jarrett and Boyle (1986) and Norman (1975). It is a very effective method for collecting data provided the sites are accessible, the floods are of long enough duration to make the observations, and work can be conducted from a boat or from the bridge deck. Probably, there are a limited number of sites in Arizona where these conditions apply. The equipment needed for this is standard streamgaging, sampling, and sounding gear. Successful application requires fairly detailed advance work for site selection and an experienced team to collect the data.

Fixed Installations

Fixed installations provide the greatest opportunities for collecting detailed information about scour, sediment transport, and channel processes. They are most effective on perennial streams where there is a wide range of stage between low flow and floods. Usually, the installations are expensive to install and maintain and they may be plagued by vandalism, but where they can be used, they are effective.

Model Studies

Model studies are most effective where they are used in connection with a fixed installation so that the correspondence between model and prototype can be established. This approach was pioneered by Laursen and Toch (1956), but it has not been used very extensively. Confidence in model studies would be increased substantially if more studies of this kind were carried out for which prototype data were available.

Equipment, Methods, and Procedures

A variety of techniques for collecting data on perennial streams have been tried and proven, but many of these techniques are not applicable to flows of ephemeral streams of arid regions because of high velocities, rapid changes of stage, the short duration of the floods, debris, and other hazards. Methods that might be applicable apparently have not been tested. So far as can be determined, no reliable complete data on local scour at bridge piers for streams in arid regions have been published, and apparently very little, if any, has been collected.

In order to evaluate equipment, methods, and procedures, an evaluation matrix was prepared listing 12 methods or items of equipment and rating each qualitatively as to cost, accuracy, reliability, and other criteria. The evaluation matrix, Table 4, shows that most of the items are untested so far as their utility in Arizona streams is concerned, and the table identifies

TABLE 4

ADOT Bridge Scour Study

Evaluation Criteria Matrix

for equipment, methods, and procedures
for measuring and monitoring bridge scour.

Method or Item	Cost	Reliability	Accuracy	Complexity and/or Cost of Operation	Complexity and/or Cost of Maintenance	Level of Training Required for Operation	Level of Training Required for Interpretation	Amount of Initial Field Work	Amount of Continuing Field Work	Utility in Arizona Streams	Remarks
1. Ground penetrating radar	H	H	H	H	H	H	H	M	L	U,T	Used in perennial streams, needs to be tested for use in Arizona.
2. Ground penetrating sonar	H	H	H	H	H	H	H	M	L	U,T	Needs to be tested for use in Arizona streams.
3. Side scan sonar	H	L	M	H	H	H	H	L	L	U	Used in large perennial streams.
4. Sonic sounders	L	H	H	L	M	L	L	L	L	U	Cannot be used in debris laden streams.
5. Conductance meter on pier	H	L-M	M	H	H	M	M	H	L	U	Vandalism and maintenance may be a problem.
6. Fully instrumented experimental pier	H	H	H	H	H	H	H	H	L	U	Vandalism and maintenance may be a problem, need controlled flows.
7. Laboratory model study	M	M	L	M	M	H	NA	H	NA	M	Useful in conjunction with No. 6.
8. Mobile team w/ standard equipment	M	M	M	M	M	H	H	H	H	U	USGS uses this effectively, high intensity activity during floods, permanent crew required.
9. Post-flood reconstruction	M	L	L-M	M	NA	H	H	H	M	T	Very useful anywhere Should be developed.
a. Indirect flow measurements	L	M	M	L	M	H	H	M	L	M	Standard.
b. Excavation for sedimentary structures	M	U	U	M-H	NA	H	H	H	NA	U	Needs to be tested for for use in Arizona streams.
10. Tracer particles	L	L	L	L	L	L	H	H	L	T	Probably has limited utility.
11. Buried transmitters	M	M	M	L	L	L	H	H	L	T	Have potential and should be tested.
12. Buried chains, tapes, pier collars, etc.	L	L	L	L	L	L	H	H	M	H	Standard techniques in arid-zone fluvial morphology.

H = High
M = Medium
L = Low
U = Unkown or untested
T = Test during pilot project
NA = Not applicable

a number of items that we believe should be tested as part of a pilot program. This matrix served partially as a basis for the pilot programs discussed below.

Recommendations for Pilot Programs

Five pilot programs have been considered. These are:

1. Ground penetrating radar.
2. Buried transmitters.
3. Post-flood reconstitution.
4. Mobile teams.
5. Fixed installations.

Some of the pilot programs listed above could be combined if suitable sites are available, or could be incorporated into a long-term data collection program if the resources to do this are available. In particular, certain of the equipment could be field tested together if a site could be located where controlled flows could be obtained, or where there is high probability of flood discharges occurring each year.

Ground Penetrating Radar

Ground penetrating radar (GPR) is widely used today in certain geophysical applications, and it is presently being tested for applications in scour studies by the USGS and FHWA (Gorin and Haeni, 1989). It has the potential to map the stratification in a stream bed or around a pier or abutment. If armoring occurs during a scour event and the scour hole is subsequently filled, the GPR should be able to identify the depth of the armoring and the interface between the old streambed sediment and the less dense recent fill material. If this can be demonstrated, then the device could be used in a variety of scour studies.

A pilot program to determine the applicability of GPR for Arizona should involve the following, as shown in Figure 4:

Equipment Demonstration - The purposes of the demonstration are to acquaint ADOT staff with the general principles and operation of ground penetrating radar and to provide examples of records that identify cross stratification in dry or partially-saturated streambeds. The demonstration should be conducted by individuals who are experienced in the applications of GPR. This could be handled in several ways, 1) ADOT might be able to arrange for USGS and FHWA to come in with their equipment for a demonstration, 2) a firm that specializes in geophysical exploration probably could be retained to do this.

Field Testing and Evaluation of Equipment - This involves acquisition of the necessary equipment and carrying out a series of tests to determine the range of conditions under which GPR can be effectively used and its limitations. The results of field tests will need to be calibrated against ground truth.

GPR has applications in the bridge inspection program and in many other highway activities, so ADOT should consider purchasing the equipment and making it available for the bridge scour pilot program or should include its purchase in the project costs. A GPR unit costs about $16,500; with one additional antenna, carrying cases and extra cable, the price of equipment is around $25,000.

To establish ground truth, it will probably be necessary to rent a backhoe. Possibly, some work could be done at gravel mining or construction sites. For example, the Salt River Project is constructing an inverted siphon across the Salt River, and trenching has established that a dense layer of cobble exists about 10 feet below the surface. The siphon will be below this cobble layer. The GPR should be able to detect the cobble layer, the buried siphon, and the interface between the fill and the pre-existing stream bed.

GPR measures velocity through a medium, and through partially saturated alluvium, the velocity should depend in part on the moisture content. The velocity can be determined directly using a buried target (Ulriksen, 1982).

Development of Field Procedures - The development of field procedures includes preparing instructions for use of the GPR, establishing guidelines for its application, designing a checklist and instructions for collecting ancillary data, such as flow data, bridge and channel geometry and bed sediment samples, and, if necessary, developing specifications for buried targets and instructions for their placement. Most of these items would be developed as part of the field testing and evaluation. In addition to the above, it would be necessary to establish a data acquisition and filing system to insure that all relevant data become part of ADOT permanent files.

Pre-Flood Baseline Studies - After applications and limitations of the equipment are defined and field procedures are established, the GPR should be used to map existing conditions at bridge piers and abutments for as many sites in Arizona as practicable. Priority should be given to those locations that are identified as high risk sites and to locations on gaged streams where the flood flows and the hydraulic parameters at the bridge site can be determined directly.

In addition to the GPR records, it will be necessary to collect and compile data on the channel and approach geometry, bridge geometry, bed sediment size distributions, and other pertinent information. These data will be entered into the data files. It may be necessary to identify or install buried targets for calibration.

If any floods occur at the surveyed sites during the duration of the project, the sites should be resurveyed to try to detect depths of scour, and these data along with information on flow and sediment data should be included in the final report.

Reports - Probably, three reports should be prepared in connection with this program, one on the field testing and evaluation, one on procedures, and one on the pre-flood baseline studies.

Buried Transmitters

Miniature transmitters have been used for a long time to track game and fish. Recently, they were installed in gravel and cobbles to study initiation of motion, particle step length, and other characteristics of bed load transport (W. Emmett, USGS, personal communication). Transmitters of different frequencies could be installed in floats and buried at different depths at a bridge pier. These are motion activated, and they could be detected by a hand-held receiver or from a fixed installation activated by radio, phone, or by the rising water. This would give the time rate of scour during a flood event. For most effective use, the data needs to be combined with a stage record, so this should be used near or downstream of a gaging station. The transmitters can be detected through a certain thickness of sediment, so even if the ones that float out are not detected, it should be fairly simple to go in after the flood and determine which of the transmitters are still in position. This would give an indication of the depth of scour, but it would not provide information about the time history of the scour development. An alternative approach would be to install the transmitters in rocks that will armor the scour hole, these could then be recovered after the flood.

This pilot program would be conducted as follows, as shown in Figure 4:

Preliminary Equipment Acquisition - The first step of this program is to meet with manufacturers, develop specifications for the equipment, and acquire the equipment. Usually, the equipment has to be fabricated, and it may be necessary to allow 30 to 60 days for delivery.

Laboratory Testing - It will be necessary to carry out tests in the laboratory or under controlled field conditions to determine the depth of water and depth of sediments through which the transmitters can be detected. Testing in water could be carried out in a laboratory with a deep sump or in a deep lake or reservoir. To determine the depth of sediment through which signals can be detected, the transmitters will need to be buried at various depths in both dry and wet sand. The signal attention is then determined as a function of the depth of sediment.

Development - The most important item to be developed is some kind of floating container for the transmitters that will not be crushed when it is buried in the streambed. If the streambed is composed of coarse material, excavation and burial will probably have to be done with a backhoe, so considerable stress will be placed on the containers. If the bed is sand, the containers can probably be plastic bottles that are placed in holes by augering, coring, or jetting into the bed.

In either case, the transmitters are expensive and should be recovered and reused if possible. Under some conditions it should be possible to tether the containers to the pier or abutment. Tethering arrangements should be developed and tested.

If the transmitters are designed to accumulate in the scour hole, they can be sealed with epoxy in a drilled rock or mounted in some kind of weighted container.

If floating transmitters are to be used to determine both the time history of the scour and the volume of the scour hole, the maximum depth of scour should be estimated and the optimum three-dimensional grid for spacing of the transmitters needs to be designed. This should first be done for the simplest case, a round pier, where the spacing will be mostly a function of the pier diameter.

<u>Site Selection</u> - During site selection, consideration should be given to at least one automated installation and to several sites, as practicable, using observers with portable receivers during floods or for post-flood reconstitution of the scour event. Monitoring from a fixed installation will require mounting a receiver, antenna, scanner, recorder, and power supply under the bridge deck or in a suitable shelter downstream. Special provisions will have to be made to prevent theft or vandalism. Probably, this should be considered only for sites near a gaging station, because the time history of the scour is not of much value without the time history of the flow. Priority in site selection should be given high risk locations and locations that have a high probability of flows.

<u>Installation of Equipment</u> - Installation in streams with gravel and cobble beds probably will involve contracting the excavation and backfill. Bed sediment sizes, channel and bridge geometry, and other auxiliary data should be collected at the same time.

<u>Report</u> - A report on this work should include the specifications of the equipment, the results of testing and development, the location and frequency of each buried transmitter, and any observations made during floods that occur through the life of the project.

A list of suppliers of the types of miniature radio transmitter (tracking) equipment that is envisioned for this program is shown in Table 5.

TABLE 5

Suppliers of Radio Transmitter Equipment

Supplier	Location
AVM Instrument Company	Champaign, Illinois
Advanced Telemetry Systems, Inc.	Isanti, Minnesota
Dav-Tron, Inc.	Minneapolis, Minnesota
Smith-Root, Inc.	Vancouver, Washington
Telonics, Inc.	Mesa, Arizona

Post-Flood Reconstitutions

Post-flood reconstitution involves flood routing on gaged streams and indirect streamflow measurements on ungaged streams to estimate flow parameters and visual inspection, probing, penetration tests, excavation, and other techniques to determine depths of scour. The advantages of these methods are that they afford wide geographic coverage, require little in the way of equipment, and involve limited manpower. They are more effective if some advance work has been done at the site and can be expanded using scour chains, tracers, GPR, radio transmitters, etc.

A pilot project in this area should involve work at a few selected sites to develop methods and techniques and to provide opportunities for training and the preparation of guidelines and techniques manuals.

This pilot program would be conducted as follows, as shown in Figure 4:

Develop Methods and Prepare Guides - Compile a bibliography, design a checklist of quantities to be observed, prepare a list of equipment and expendable supplies, prepare guides and procedures for non-standard methods (procedures for indirect discharge measurements are given in the USGS techniques manuals).

Classify Sites - It has been recommended that ADOT bridge records and USGS streamflow be compiled and analyzed. From these data it will be possible to classify sites: gaged or ungaged watersheds, ephemeral or perennial flows, rock-controlled or alluvial channels, type of bed material, if alluvial, accessibility, priority, etc. This site classification can be used in site selection for the pilot study, for selecting sites for later long-term studies if they are taken on, and for setting priorities for field surveys if there are major floods involving a number of bridge sites.

Site Selection - Field reconnaissance and the data files will be used for site selection.

Pre-Flood Studies and Equipment Installation - Limit to five or ten sites, select some for advance work, carry out the necessary surveys and install tracers, chains, transmitters, GPR (if these methods are to be studied or have proven to be useful). Establish a file system and data base that will be compatible with existing ADOT records. Undertake the post-flood field work as required and report on the activities.

Mobile Teams

Jarrett and Boyle (1986) describe a pilot study carried out in Colorado. The project involved about 9 man-months of effort and cost about $50,000 in 1984 (R.D. Jarrett, personal communication, 1989). There is no reason why a similar study could not be designed and carried out in Arizona. However, the ideal conditions of the Colorado study are not likely to be found in Arizona, and the simple equipment and techniques used by Jarrett and Boyle (1988) cannot be used in debris-laden stream. A pilot study for Arizona would require a more complicated and innovative approach and probably would cost considerably more. Advance work would need to be done at the sites, perhaps involving some low-cost fixed installation described in the next section. Certain criteria need to be met for the mobile-team approach to be effective; for example, (1) a flood warning system, (2) ability to get to the site during the flood, and (3) restrict studies to floods of 10-year or greater return periods. Geographic coverage is important. USGS has offices in Phoenix, Tucson, Yuma, and Flagstaff, so they could provide a fairly broad coverage. A pilot program should be negotiated with USGS.

Fixed Installations

A fully automated fixed installation is one that can collect specified data unattended. Ideally, if a fixed installation is designed as a research facility to study scour processes, it should have the capabilities to record as a function of time the approach velocity and depth, the direction of the approaching flow relative to the piers and abutments, the water temperature, sediment discharge, and shape and change in volume of the scour hole. For a single pier, this would probably involve the installation of an upstream gaging station with a cable crossing for observations, the development of the stage-discharge relation, a pumping sediment sampler, an electromagnetic flow meter or some other directional velocity sensor near the pier, and some method to map the scour hole, such as a sonic sounder or series of transducers that can scan a specified area of the bed. The data should be recorded, preferably at an off-site location, in the event the bridge fails during a flood. If power is not available at the site, it would be necessary to install solar cells, a generator, or some other reliable source to power the equipment. The installations would need to be weather proof and vandal proof and protected from debris or ice damage.

Clearly, such an installation would be expensive to install and would require a fairly high level of technical training for those who operate and maintain the facilities. However, the greatest drawback to fixed installations is that once they are installed and operating, many years may pass before a significant flood occurs, so the benefit-to-cost ratio may be very low. Several investigators have concluded that using a mobile team is more effective than using highly instrumented fixed installations in studying scour processes (Norman, 197; Jarrett and Boyle, 1986; Alberta Research Council, Dave Andres, personal communication, 1989). We believe that highly instrumented and fully automated fixed installations would not be cost-effective on Arizona streams.

At a more practical level, the bridge designer may be interested only in the maximum depth of scour associated with a particular flood. This is a much simpler program, and for these cases, fixed installations may be fairly simple and cost-effective.

There are many techniques and devices that can be used to measure maximum scour depths at piers and abutments. These devices and techniques fall into three general categories, as follows: (1) direct bed elevation soundings, (2) indirect bed elevation determinations, and (3) tracers and other miscellaneous techniques.

The most reliable method for determining scour depth is through direct bed elevation soundings. The simplest fixed installation for this is a vertically supported sounding rod mounted to the bridge pier that settles to the bottom of the scour hole. An alternative is to mount one or more transducers for a sonic sounder on the pier and run the power cables to the bank so that a single sounder could be used with many transducers. This is relatively inexpensive; a reliable sonic sounder can be purchased for about $600.00, the transducers are about $50.00, and the power cable runs about 50 cents to a dollar per foot. The transducers would need to be mounted in a stilling well to protect against debris and to avoid air entrainment under the transducer, which causes loss of signal. The transducers commonly used for bed elevation soundings "see" through a cone of about 8 degrees, so they would have to be mounted a short distance upstream from the pier with due attention to the geometry of the situation.

Indirect bed elevation determinations are made by devices mounted to the bridge pier and extending into the bed that can sense the water-sediment interface. The more common of these are conductivity, heat dissipation, or light scattering devices. Laursen and Toch (1958) were among the first to use one of these devices, which measured conductance. Skinner (1986) reported on the feasibility of using casting resins for heat dissipation gages, but a prototype has not been fabricated or tested. These devices are not used much because they are not commercially available; they have to be specially designed and constructed.

Tracers and other miscellaneous techniques involve devices that have to be recovered after the flood. Some of these can be used effectively and some are recommended in our proposed pilot programs.

Fixed installations have not been used much in the United States or Canada to determine scour at bridge piers and abutments, and we do not recommend a separate pilot project for these devices. However, some development and testing of these devices should be incorporated in both the mobile team and post-flood evaluation pilot projects. Recently, The National Cooperative Highway Research Board (NCHRB) contracted for a 27-month research project to develop, evaluate, and test devices for measuring scour at bridge piers, so any work undertaken by ADOT should be coordinated with NCHRB to avoid duplication of efforts.

In summary, we believe that fully instrumented and automated fixed installations may not be cost-effective for Arizona streams. Provisions to test low-cost non-automated fixed installations are included in our proposed pilot studies, and more elaborate systems could easily be included if ADOT so desires. However, any major efforts on fixed installations should be coordinated with the NCHRB efforts.

AGGRADATION, DEGRADATION, LATERAL MIGRATION MONITORING PROGRAMS

General

The purposes of the pilot projects are to test equipment, develop field techniques, and to demonstrate the utility of certain bridge scour measurement programs. Pilot programs must be carried out in a relatively short time period, such as 1 to 2 years. It is not practical to define pilot programs for long-term processes such as aggradation, degradation, and lateral instability, collectively called channel instability. Therefore, only monitoring programs are suggested for these long-term processes.

Because of the complex nature and areal extent of channel instability it is not possible to define simple, at-a-station type monitoring programs. A phased monitoring program is recommended. The phased approach has been selected so that monitoring and data analysis for a bridge can proceed from fairly simple techniques to more advanced and costly methods depending upon the need at each site, independently of the level of effort selected for other sites. This approach will provide effective overview of channel instability potential at many bridges followed by increasingly complex analyses at bridges for which the simpler methods do not provide the necessary results.

Recommended Monitoring Program

The recommended monitoring program for long-term channel processes affecting bridges in Arizona is as follows, as shown in Figure 4:

1. The bridge data files will be generated as previously described.
2. The bridge data will be analyzed to identify bridges that may be experiencing channel instability (Level 1 monitoring). The types of analyses that are required have been discussed in a previous section. The results of the data analyses will be interpreted in an effort to identify the causes of the data trends that have been recorded. Decisions will be made as to which bridges warrant additional studies.

3. Those bridges for which additional studies have been indicated will enter into Level 2 analysis. This will consist of a compilation and analysis of aerial photographs. The intent of this analysis will be to identify causes for trends as identified in the Level 1 analysis. For example, the aerial photographs may show changes in upstream land use such as urbanization or agriculture, or they may show changes in the floodplain such as gravel mining or channelization, or the photos may not indicate any discernable reason for the changes. At Level 2 it may be possible to define the necessary administrative or engineering actions to either counter the channel instabilities or the decision may be made that additional monitoring or analyses are needed. At this level of analysis and monitoring one of two or both of the following could be recommended.
4. Field surveys (Level 3) could be performed at selected rangelines across the watercourse and a longitudinal profile of the channel obtained. These same rangelines could be resurveyed during the biennial inspections or at some other time interval, as deemed appropriate. Field surveys in conjunction with aerial photographs and possibly satellite imagery, may provide a comprehensive level of monitoring.
5. As an alternative to or in addition to the field survey, a geomorphic study could be undertaken to attempt to identify the cause of the change. For example, the San Pedro River in Cochise County experienced significant channel degradation near the turn of the century. The cause of the degradation has been identified as the result of a severe earthquake and ensuing range fires that occurred in 1887. The application of remote sensing and satellite imagery may assist in the documentation of more recent geomorphic changes.

It is not possible at this time to provide a cost estimate to perform these studies because the number of bridges and the level of analyses that will be required cannot be determined. It is recommended that the Level 1 monitoring be performed so as to identify those bridges for which channel instability may be occurring. At the completion of this level of study it should be possible to define the scope-of-work and cost estimate for the next level of analysis.

IMPLEMENTATION AND PRIORITIES

The projects and programs suggested here can be implemented in various ways, depending mostly on available financing and personnel. ADOT could do all of the work in-house, they could negotiate with USGS to do the work, or they could initiate requests for proposals to involve universities and consultants. It is unlikely that ADOT can do the work in-house because of personnel limitations, so some combination of the above is probably the optimum approach. Certainly, the data-collection programs should be negotiated with USGS, preferably as a part of the USGS cooperative program whereby up to 50% of the costs are covered by Federal funds. Some of the pilot projects might be suitable studies for MS level theses, so possible participation by universities should be considered. Developing the bridge files is fairly specialized, and some part of this should be contracted to a consultant or firm that specializes in designing computer information systems, with oversight and review by either ADOT or an outside consultant. USGS is a scientific organization, they normally are not concerned about operation, maintenance or design, so aspects of the programs involving these items should be handled by ADOT professional staff or by a consultant. In the previous sections, we have recommended that an engineering consultant be retained to provide overall coordination and review and to assist ADOT in other activities.

Highest priority should be given to compiling the hard copy files and developing the computer data files. Maximum use should be made of any existing files, and any computer files that are initiated should be designed so that they can merge into an overall information system that eventually will serve the needs of managements, design, inspection, operation and maintenance, and research. Most commercial spread sheets can meet these requirements.

Among the pilot programs, first priority should be given to post-flood reconstitution and mobile teams because these together afford wide geographic coverage and could be implemented quickly. Such pilot programs would need to be defined and the cost negotiated with the USGS for mobile teams. Post-flood reconstitutions could be performed by the USGS or qualified consultants. The GPR demonstration probably should be carried out before a full commitment is made to that project; perhaps this could be arranged as part of the FHWA-USGS national bridge scour program at no cost to ADOT. If the demonstration shows favorable results, GPR should be given fairly high priority.

The applications of buried transmitters have the potential to monitor the time rate of change of both scour volume and maximum sour depth. This pilot project should have high priority as a basic research project. However, from a practical or applied point of view, only the maximum scour depth needs to be determined, and there are simpler and less expensive methods to monitor that.

The monitoring programs to identify channel instabilities and document long-term aggradation and degradation processes could be implemented at any time on a limited basis and at fairly low costs. If the ADOT bridge inspection program has already identified potential problem sites, they should be given high priority, otherwise we recommend the phased approach suggested above.

TIME AND COST ESTIMATE

Schedule

It is estimated that it would take about 2 years to undertake the programs that have been recommended in this report. The first year would mainly involve the development of the data files and the preliminary equipment testing and evaluations for the pilot programs. The end of the first year would result in the selection of sites for the pilot programs. The second year would be the implementation of the scour pilot programs and the initiation of the Level 1 channel instability monitoring program. A general time schedule is shown in Figure 5.

Time Estimate

The level of effort by an engineering consultant for each task has been estimated and is shown in Table 6. This estimate is based on selecting only about 5 to 10 bridge sites for each of the bridge pilot programs, but includes a Level 1 monitoring program for channel instability for all waterway bridges in Arizona (about 800).

The cost of the consultant staff time is shown in Table 6. These costs are based on assumed unit rates of $500 per day for an engineer and $150 per day for a technician.

Two tasks, the hydrologic data analyses and the Mobile Team bridge scour pilot program should be negotiated directly with the USGS. It is also noted that the ADOT Bridge Section staff would be required to contribute to the compilation of information for the engineering and historic data file and the bridge section geometry data file. Neither the negotiated cost of the USGS programs nor the internal cost to ADOT have been estimated.

FIGURE 5

Time Schedule for Scour Pilot Programs and Channel Instability Monitoring Program

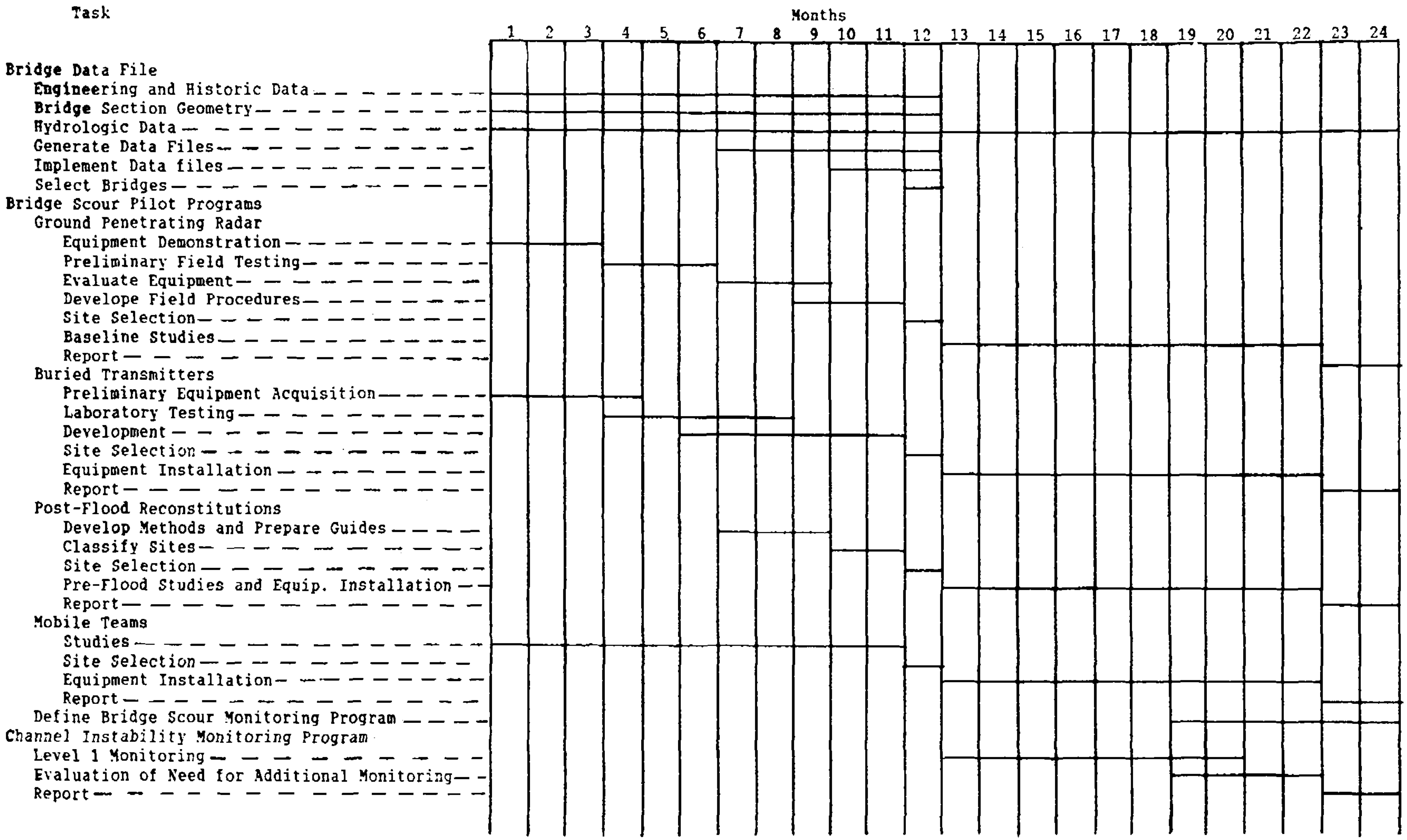

TABLE 6

Time Estimate for Consultant, in man-days

Task		Engineer	Technician
I. Bridge Data File			
Engineering and Historic Data		50	0
Bridge Section Geometry		25	60
Hydrologic Data		Negotiate with USGS	
Generate Computer Data Files		10	40
Implement Data Files and Select Site		10	0
	Sub-Totals	95	100
	Cost	$47,500	$15,000
II. Bridge Scour Pilot Programs			
A. Ground Penetrating Radar			
Equipment Demonstration		5	5
Preliminary Field Testing		10	10
Evaluate Equipment		5	5
Develop Field Procedures		15	15
Baseline Studies		40	40
Report		20	0
	Sub-Totals	95	75
	Cost	$47,500	$11,250
B. Buried Transmitters			
Preliminary Equipment Acquisition		5	0
Laboratory Testing		15	10
Development		15	10
Equipment Installation		40	40
Report		20	0
	Sub-Totals	95	60
	Cost	$47,500	$ 9,000
C. Post-Flood Reconstitutions			
Develop Methods and Prepare Guides		10	0
Classify sites		10	0
Pre-Flood Studies and Equip. Installation		20	20
Report		10	0
	Sub-Totals	50	20
	Cost	$25,000	$ 3,000
D. Mobile Teams		Negotiate with USGS	
III. Channel Instability Monitoring Program			
Level 1 Monitoring of all bridges		80	40
Evaluation of Need for Additional Monitoring		20	0
Report		20	0
	Sub-Totals	120	40
	Cost	$60,000	$ 6,000

Direct Cost Estimate

The direct costs to conduct these studies have been estimated and are shown in Table 7. These costs are based on estimates of the major items that will be needed for this project plus the usual direct costs that would be associated with a project of this complexity for a 2-year period. Actual costs would vary depending on the site selections and equipment specifications.

TABLE 7
Direct Cost Estimate

Major Task	Item	Cost (dollars)
I. Bridge Data File	Computer supplies & charges	1,000
II. Bridge Scour Pilot Programs	GPR equipment	25,000
	Transmitter equipment	15,000
	Laboratory and testing supplies	2,000
	Field equipment and rental	7,500
	In-state travel (40 days)	5,000
III. Channel Instability Monitoring Programs	Computer supplies and charges	1,000
	In-state travel (10 days)	1,250
Miscellaneous		
Typing and secretarial		8,000
Telephone		500
Postage and delivery		250
Copy charges		750
Report printing		500
Contingencies		5,000
	Total Direct Costs	$72,750

Benefits/Costs

Costs

The bridge scour programs that have been recommended will require a commitment of resources by the State of Arizona and by ADOT. It is anticipated that some of the financial commitment can be met by the FHWA and cooperative agreements with the USGS. However, ADOT will be the user of the research results and therefore ADOT must assume responsibility for the implementation of the programs to assure that the research will be responsive to its needs. This will require a financial commitment and a staff commitment by ADOT, and this is a management decision. To make such a decision requires understanding of the costs, benefits, and liabilities of its actions. It is not possible to fully estimate the costs of all programs because of factors such as the extent of FHWA and USGS cooperation which cannot be defined. Likewise, it is not possible to anticipate how ADOT would choose to distribute and manage the activities that have been suggested for it. The costs and time commitments that can be anticipated have been estimated and these are summarized below:

	Labor Cost	Direct Cost	Total Cost
I. Bridge Data Files	$62,500	$ 4,000	$66,500
II. Bridge Scour Pilot Programs			
A. Ground Penetrating Radar	$58,750	$32,000	$90,750
B. Buried Transmitters	$56,500	$25,500	$82,000
C. Post-Flood Reconstitutions	$28,000	$ 6,000	$34,000
III. Channel Instability Monitoring Programs			
Level 1 Monitoring	$66,000	$ 5,250	$71,250
	$271,750	$72,750	$344,500

The cost of implementing this research must be considered in terms of offsetting the design and operational costs that are associated with a continuation of present procedures. It is not possible to accurately define these costs, however data is available on the cost of repairs and damages that have occurred as the result of flooding and related bridge scour. These costs have been identified previously in Tables 1, 2, and 3 and are summarized below:

	Total Cost
Emergency Repair Projects due to flooding	$ 48,424,000
Flood damages to transportation systems since 1965	97,238,000
Cost of Bridge Scour Countermeasures since 1979	7,143,000
Total	$152,805,000

Benefits

The benefits that can be identified with the recommended research are:

1. A systematic means is provided to identify bridges that may be experiencing the cumulative effects of scour and/or channel instability.
2. Bridges that are susceptible to scour can be instrumented so that the extent of scour during flood events can be monitored and remedial repairs can be initiated, if possible.
3. Priorities can be set for bridge scour countermeasure projects.
4. Improved bridge design procedures can be developed that will reduce the uncertainty in design and construction.
5. Potential damage to bridges can be assessed after major flood events.
6. Bridge safety will be enhanced.

PART 3 - RESEARCH NEEDS

GENERAL

During the course of the project, a number of research needs were identified. Some of these relate to long-standing problems associated with local scour processes, some are peculiar to Arizona streams particularly and to arid regions in general, and some are components of the pilot projects that were recommended for implementation in the Interim Report. The most important of these are listed in the following sections.

SCOUR AND TRANSPORT PROCESSES

Transport processes in rivers are inherently complex, and local scour processes are among the most difficult of all to investigate empirically or experimentally or to describe analytically. Despite the fact that these processes have been observed and studied for over a century, all of the existing equations for predicting local scour are empirical relations based on laboratory or field observations and have little if any theoretical basis beyond the usual dimensional reasoning. One of the major research needs today is to develop the mathematical expressions that describe the local scour processes.

Scour Equations

Local scour is described by a simple continuity equation:

$$dV/dt = Q_{si} - Q_{so} \tag{8}$$

Where V is the volume of the scour hole, t is time, Q_{si} is the transport rate of sediment into the scour hole, and Q_{so} is the transport rate of sediment out of the scour hole. The equation says simply that the rate of change in the volume of a scour hole is equal to the difference between the inflow and outflow of bed sediment. The transport rates are governed in someway by the properties of the fluid, the sediments, and the flow. The characteristics of the flow depend on the discharge and the geometry of the bridge site. The scour depth, (d_s), is not specified in the above relation, but usually it is assumed to relate to the volume of the scour hole through some function of the approach and pier or abutment geometry so that the volume can be expressed as a coefficient, k, times the maximum scour depth,

$$V = k(d_s) \tag{9}$$

The equilibrium scour depth occurs when the inflow and outflow of sediment are equal. It also is a function of the properties of the fluid, the sediments, and the flow, and in experimental studies, it is usually measured directly and related to dimensionless parameters to define empirical predictive relations.

For clear-water scour, the second term in Equation 8 is zero. For live-bed scour, the first term is usually measured, the second term is measured or computed, and the third term is determined as the difference between the two.

In principle, at least, to investigate scour processes, one should measure each quantity that goes into the terms of Equation 8. In practice, this is rarely done. The sediment transport out of the scour hole is almost never measured, it is controlled by the complex vortex patterns that develop around the pier or abutment, and even the flow patterns are difficult to measure or to describe mathematically. As a consequence, the mathematical models that have been proposed to describe the scour processes (for example, those of Imamoto and Ohtoshi, 1986, and Tsujimoto and Nakagawa, 1986) are not very satisfactory. It is clear that there is a need for fundamental research to measure the flow patterns around the complex geometries of piers and abutments and to develop the mathematical expressions to describe these flows. If the flows can be adequately described, then it should be possible to derive expressions to relate the sediment transport capacity to the properties of the flows and to investigate more fully the transport out of the scour hole and the sorting and armoring that takes place at the base of the scour hole. There is substantial need for both experimental and theoretical developments.

Grain Size Distribution - Scaling Scour Processes

To now, the effects of grain-size distribution on the scour processes have been investigated in the laboratory for only a limited range of conditions. Whether or not these laboratory results can be scaled to field conditions remains to be determined. Scale models for rigid-boundary flows are fairly straight-forward, but modeling transport processes is much more difficult because exact similitude is rarely possible. Model results for clear-water scour can be transferred to field conditions with more confidence than model results for live-bed scour because in the latter case there are large uncertainties involved in trying to scale transport processes. There is a clear need for experimental work with a larger range of grain sizes and for investigations on scaling the results to prototype conditions.

Field Verification

A number of other problems persist and deserve attention. For example, most experimental and theoretical work is carried out for circular piers, and the effects of complex geometries are accounted for by applying an empirical coefficient to the circular pier results. The approach is summarized in a recent paper by Melville and Sutherland (1988). But the coefficients determined by different investigators do not agree. In addition, the field measurements of comparable geometries do not exist, so there is no basis for scaling the laboratory results to field conditions with any degree of confidence.

In all of the concerns described above, a common thread is the lack of field data to test and verify the experimental work. For many years, the need for reliable field data has been recognized as probably the most pressing research area in local scour. Nonetheless, only a few concerted efforts have been undertaken, and only a few reliable data are available. There is a clear need for some nationwide coordination of efforts, a commitment to long-term data collection programs, and continuing research on the development of equipment and techniques.

RESEARCH FOR ARID REGIONS

Major research is needed in the hydrology of arid regions, in unsteady flow and transport processes, and in instrumentation and field data collection.

Hydrology

In many cases, the streams of arid regions are ungaged. If streamflow records are not available, the design floods for bridge crossings usually are estimated from empirical equations, rainfall-runoff models, or regional flood-frequency relations. However, the hydrology of arid regions is not very well understood, and flood flows estimated by these methods are likely to involve large uncertainties. There is substantial need for research in almost all aspects of the hydrology of arid regions.

Unsteady Flow and Sediment Transport

Experimental studies of pier and abutment scour are carried out with steady, uniform approach flows. Floods in arid regions, however, are characterized by a high degree of unsteadiness. It would seem to be fairly important to determine whether or not the relations developed for steady uniform flows apply to such cases.

Field Verification

Almost all field data that we have been able to locate have been for perennial streams in temperate or humid regions. The general lack of reliable field data was discussed above, but this shortage is especially critical for arid regions. The development of long-term data collection programs on bridge scour in arid-region streams should have high priority. Implementation of the pilot projects and monitoring program that have been recommended is the first step to the development of such programs.

RESEARCH ELEMENTS OF THE PILOT PROGRAMS

The pilot programs proposed in the Interim Report involve mostly the application of existing technology, but there are elements of research in some of them. For example:

Ground Penetrating Radar

The application of Ground penetrating radar in a partially-saturated alluvium involves a number of problems on calibration and interpretation. There seems not to be much information on this topic.

Miniature Radio Transmitters

The application of miniature radio transmitters to scour studies has not yet been investigated. Thus far, the use of transmitters has been limited to the same applications in sediment transport studies as conventional sediment tracers; that is, painted, fluorescent, radioactive, and magnetic particles. In addition, the design of optimum spacing of the transmitters to determine both the depth of scour, total scour volume, and time of occurrence presents some interesting statistical problems.

PART 4 - CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The main conclusions of this research are the following:

1. A review of selected case histories shows that there has been substantial bridge damage in Arizona due to channel instabilities (aggradation, degration, and lateral migration) and to local scour.
2. No complete reliable field data on bridge scour have been collected in Arizona, or, so far as we can determine, on any arid-region streams, and no programs to collect data are planned or underway in any of the arid-region states by either State or Federal agencies.
3. Because of this lack of data, ADOT cannot verify proposed pier and abutment scour equations or test the adequacy of past design practices.
4. Many of the techniques and some of the equipment used to study scour in perennial streams cannot be used in Arizona.
5. A long-term data collection program in Arizona on scour and channel processes should be initiated. By "long-term", we mean on the order of ten (10) years.

RECOMMENDATIONS

The main recommendations of this research are the following:

1. An effective program for Arizona should include (a) establishing bridge data files, (b) pilot projects to develop and test techniques and equipment, and (c) a phased monitoring program for channel instabilities.
2. Bridge data files should be developed to classify and select sites and to store and use data. Existing files should be evaluated for these purposes and modified or replaced as needed.
3. Four pilot projects are recommended. Two of these, ground penetrating radar and miniature transmitters, are directed to testing equipment. The other two are designed to develop techniques for observing local scour directly by mobile teams and indirectly by post-flood investigations.
4. A phased program is recommended for monitoring channel instabilities. If unstable channels are identified through bridge inspections or other sources, these channels should be monitored. Otherwise, sites for monitoring can be selected using the bridge files and field inspections.

5. The program can be implemented in various ways, depending on ADOT staff and financial constraints. It could be carried out by ADOT, under contract to universities and consultants, through direct negotiation with USGS, or through some combination of these groups. We believe that data collection and hydrologic analyses should be carried out by USGS, preferably on a cost-sharing basis.
6. If the entire program is implemented, we recommend that ADOT impanel a group of experts to serve as advisors, as shown in Figure 4, and that a consultant or consultants be retained to coordinate, review and advise ADOT on the program.

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APPENDIX A

Bridge Scour - Theory and Equations

BRIDGE SCOUR

1. General.

All material in a stream bed will erode. It is just a matter of time. However, some material such as granite may take hundreds of years to erode. Whereas, sandbed streams will erode to the maximum depth of scour in hours. Sandstone, shales,and other sedimentary bedrock materials, although they will not erode in hours or even days will, over time, if subjected to the erosive forces of water, erode to the extent that a bridge will be in danger unless the substructures are founded deep enough. Cohesive bed and bank material such as clays, silty clays, silts and silty sands or even coarser bed material such as glacial tills, which are cemented by chemical action or compression, will erode if subjected to the forces of flowing water. The erosion of cohesive and other cemented material is slower than sand bed material but their ultimate scour will be as deep if not deeper than the scour depth in a non-cohesive sandbed stream. It might take the erosive action of several major floods but ultimately the scour hole will be equal to or greater in depth than with a sand bed material.

This does not mean that every bridge foundation must be buried below the calculated scour depth determined for non bed rock streams. But it does mean that so called bed rock streams must be carefully evaluated.

2. Total Scour.

Total Scour at a highway crossing is composed of three components. In general the components are additive. These components are:

2.1 Long Term Bed Elevation Change..

The change in river bed elevation (aggradation or degradation) over long lengths and time due to chages in controls, suchas dams, chages in sediment discharge and changes in river geomorphology, such as changing from a meandering to a braided stream. May be natural or man induced.

2.2 General Scour.

The scour that results from the acccelaration of the flow due to either a natural or bridge contraction or both (contraction scour). General scour may also result from the location of the bridge on the stream. For example, its location with respect to a stream bend or its location upstream from the confluence with another stream. In this latter case, the elevation of the downstream water surface will affect the backwater on the bridge, hence, the velocity and scour. General scour may happen during the passage of a flood and the river may fill in on the falling stage.

2.3 Local Scour.

The scour that occurs at a pier or abutment as the result of the pier or abutment obstructing the flow. These obstructions to the flow accelerate it and create vortexes that remove the material around them.

Generally, scour depths from local scour are much larger than the other two, often by a factor of ten. But if there are major changes in stream conditions, such as a large dam built upstream or downstream of the bridge or severe straightening of the stream, long term bed elevation changes can be the larger element in the total scour.

2.4 Lateral Shifting of the Stream.

In addition to the above, lateral shifting of the stream may also erode the approach roadway to the bridge and by changing the angle of the flow in the waterway at the bridge crossing change the total scour.

3. Long Term Bed Elevation Changes.

Long term bed elevation changes (aggradation or degradation) may be the natural trend of the stream or may be the result of some modification to the stream or watershed condition.

The stream bed may be aggrading, degrading or not changing (equilibrium) in the bridge crossing reach. When the bed of the stream is neither aggrading or degrading it is in equilibrium with the sediment discharge supplied to the bridge reach and the elevation of the bed does not change. In this section we are considering long term trends, not the cutting and filling of the bed of the stream that might occur during a runoff event (general scour). A stream may cut and fill during a runoff event and also have a long term trend of an increase or decrease in bed elevation. The problem for the engineer is to determine what the long term bed elevation changes will be during the life time of the structure. What is the current rate of change in the stream bed elevation? Is the stream bed elevation in equilibrium? Is the stream bed degrading? Is it aggrading? What is the future trend in the stream bed elevation?

During the life of the bridge the present trend may change. These long term changes are the result of modifications of the state of the stream or watershed. Such changes may be the result of natural processes or the result of man's activities. The engineer must assess the present state of the stream and watershed and determine future changes in the river system and from this determine the long term stream bed elevation.

Factors that affect long term bed elevation changes are: dams and reservoirs (upstream or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoff of a meander bend (natural or man made), changes in the downstream base level (control) of the bridge reach, gravel mining from the stream bed, diversion of water into or out of the stream, natural lowering of the total system, movement of a bend, bridge location in reference to stream planform and stream movement in relation to the crossing. Examples of long term bed elevation changes are given in Chapter VII of Highways in the River Environment (HIRE) (Richardson et al 1975).

Analysis of long term stream bed elevation changes must be made using the principals of river mechanics in the context of a fluvial system analysis. Such analysis of a fluvial system require the consideration of all influences upon the bridge crossing ie. runoff from the watershed to the channel (hydrology), the sediment delivery to the channel (erosion), the sediment transport capacity of the channel (hydraulics) and the response of the channel to these factors (geomorphology and river mechanics). Many of the largest impacts are from man's activities, either in the past, the present or the future. This analysis requires a study of the past history of the river and man's activities on it; a study of present water and land use and stream control activities and finally contacting all agencies involved with the river to determine future changes in the river.

A method to organize such an analysis is to use a three level fluvial system approach. This method provides three level of detail in an analysis, they are 1) a qualitative determination based on general geomorphic and river mechanics relationships, 2) engineering geomorphic analysis using established qualitative and quantitative relationships to establish the probable behavior of the stream system to various scenarios of future conditions and 3) quantify the changes in bed elevation to be expected as the result of the changes in the stream and watershed, using available physical process mathematical models such as HEC-6, straight line extrapolation of present trends and engineering judgement. Methods to be used in stage 1 and 2 are given in recent FHWA reports such as Stream Channel Degradation and Aggradation: Analysis of Impacts to Highway Crossings (Brown et al, 1981).

4. General Scour.

General scour at a bridge can be caused by a decrease in channel width, either naturally or by the bridge, which decreases flow area and increases velocity. This is contraction scour. General scour can also be caused by short term (daily, weekly, yearly or seasonally) changes in the downstream water surface elevation that controls the backwater and hence the velocity through the bridge opening. Because this scour is reversible it is included in general scour rather than in long term scour. General scour can result from the location of the bridge with regard to a bend. If the bridge is located on or close to a bend the concentration of the flow on the outer part of the channel can erode the bed.

General scour can be cyclic. That is, during a runoff event the bed scours during the rise in stage (increasing discharge) and fills on the falling stage (deposition).

General scour from a contraction occurs when the flow area of a stream is decreased from the normal either by a natural constriction or by a bridge. With the decrease in flow area there is an increase in average velocity and bed shear stress. Hence, there is an increase in stream power at the contraction and more bed material is transported through the contracted reach than is transported into the reach. The increase in transport of bed material lowers the bed elevation. As the bed elevation is lowered, the flow area increases and the velocity and shear stress decreases until equilibrium is obtained so that the bed material transported into the reach is equal to that which is transported out of the reach.

The contraction of the flow by the bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel and/or the piers taking up a large portion of the flow area. Also, the contraction can be caused by the approaches to the bridge cutting off the overland flow that normally goes across the flood plain during high flow. This latter case causes clear-water scour at the bridge section because the overland flow normally does not transport any bed material sediments. This clear water picks up additional sediment from the bed when it returns to the bridge crossing. In addition, if it returns to the stream channel at an abutment it increases the local scour there. A guide bank at that abutment decreases the risk from scour of that abutment from this returning overbank flow. Also, relief bridges in the approaches, by decreasing the amount of flow returning to the natural channel decrease the scour problem at the bridge cross section.

Factors that can cause contraction scour are:

1) a natural stream constriction,
2) long approaches over the flood plain to the bridge,
3) ice formation or jams,
4) berm forming along the banks by sediment deposits,
5) island or bar formations upstream or downstream of the bridge opening,
6) debris, and
7) the growth of vegetation in the channel or flood plain.

To determine the magnitude of general scour from a varialble backwater requires a study of the stream system to (1) determine if there will be variable backwater and if this condition exists to (2) determine the magnitude of general scour for this condition. Of particular value in determining if backwater affects exist and the magnitude of the affects on the velocity and depth is the WSPRO (Shearman, 1987) computer model. The difference in depth between the highest expected elevation and the lowest ecpected elevation for the design discharge is the value of the general scour.

General scour of the bridge opening may be concentrated in one area. If the bridge is located on or close to a bend the scour will be concentrated on the outer part of the bend. In fact there may be deposition on the inner portion of the bend, further concentrating the flow, which increases the scour at the outer part of the bend. Also, at bends the thalweg (the part of the stream where the flow or velocity is largest) will shift toward the center of the stream as the flow increases. This can increase scour and the non uniform distribution of the scour in the bridge opening.

Often the magnitude of general scour can not be predicted and inspection is the solution for general scour problems. Also, a physical model study can be used to determine general scour.

4.1 Predicting Contraction Scour.

There are several methods and equations for estimating the magnitude of contraction scour. These will be given in this section. Unfortunatly the equations are based on laboratory studies with very little field data.

Contraction scour can be caused by different bridge site conditions. There are 4 main conditions (cases) which are as follows:

Case 1. Overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge. The bridge and/or the channel width is narrowerer than the normal stream width.

Case 2. The normal river channel width becoming narrower either because of the bridge itself or by the bridge site being on a narrower reach of the river.

Case 3. A relief bridge in the overbank area. With little or no bed material transport in the overbank area.

Case 4. A relief bridge over a secondary stream in the overbank area.

These 4 cases are illustrated in Fig. 1

4.1.1 6 Case 1

Overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge. The bridge and/or the channel width is narrowerer than the normal stream width. Laursen 's (1960) equation given below is used to predict the depth of scour in the contracted section y_2.

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{A} \left(\frac{n_2}{n_1}\right)^{B} \qquad 1$$

$y_s = y_2 - y_1$ Average scour depth

Where
y_1 = average depth in the main channel
y_2 = average depth in the contracted section
W_1 = Width of the main channel
W_2 = Width of the contracted section
Q_t = flow in the contracted section
Q_c = flow in main channel
n_2 = Manning n for contracted section
n_1 = Manning n for main channel

A & B are transport coefficients from the following:

V_{*c}/w	e	A	B	Mode of Bed Material Transport
<0.5	0.25	0.59	0.066	mostly contact load
1.0	1.0	.64	.21	some suspended bed material
>2.0	2.25	.69	.37	mostly suspended bed material

V_{*c} = $(gy_1S_1)^{0.5}$, shear velocity
w = fall velocity of D_{50} of bed material
g = gravity constant
S_1 = slope, energy grade line main channel

$$A = \frac{6\ (2+e)}{7\ (3+e)}$$

$$B = \frac{6e}{7\ (3+e)}$$

e = transport factor

Note 1. The Manning n ratio can be significant in you have a dune bed in the main channel and plain bed, washed out dunes or antidunes in the contracted channel, see Chapter III of HIRE.

Note 2. The average width of the bridge opening (W_2) is normally taken as the top width with the width of the piers subtracted.

Note 3. Laurson's equation for a long contraction will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or if the contraction is the result of the bridge abutments and piers. But at this time it is the best equation available.

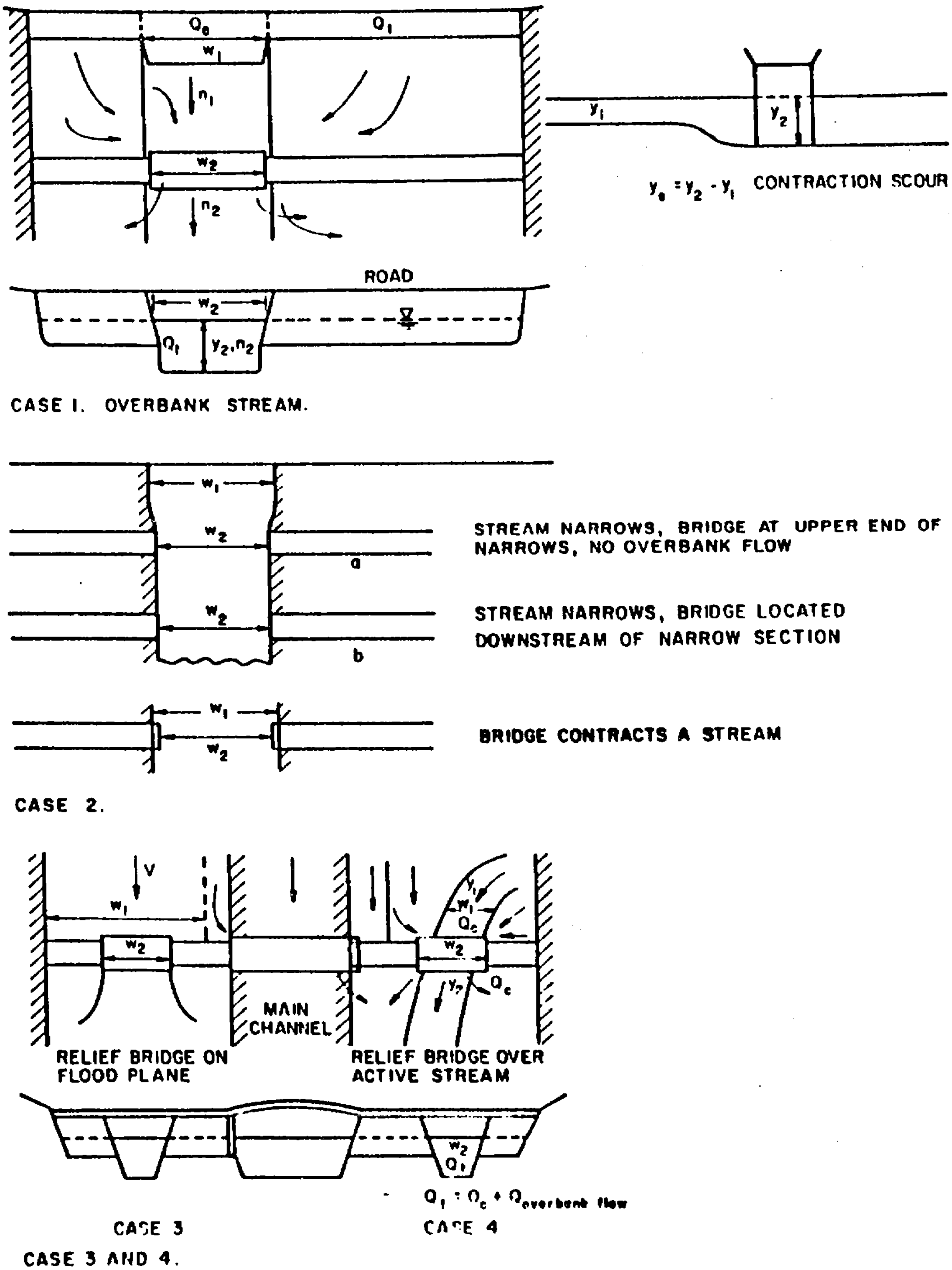


Fig. 1 The four main cases of contraction scour.

4.1.2 Case 2.

No overbank flow but the stream channel narrows either naturally or by the bridge abutments encroaching on the channel. That is flows confined to the channel.

If the contraction of the channel is less than 10 percent the contraction scour should be negligible.

Three method will be given for estimating contraction scour for this case. These will be:

1) The use of Equation 1 given above,
2) a method developed by Nordin (1971), and
3) an equation developed by Straub (1940).

1. To estimate contraction scour using equation 1 set Q_t equal to Q_c

2. Nordin's method is as follows:

The approach flow depth, y_1, and average approach flow velocity, V_1, results in the sediment transport rate q_{s1}. The total transport rate to the contraction is $W_1 q_{s1}$ in which W_1 is the width of the approach. If the water flow rate, $Q_1 = W_1 q_1$, in the upstream channel is equal to the flow rate at the contracted section, then by continuity

$$q_2 = \frac{W^1}{W_2} q_2 \qquad 2$$

Here $q_1 = y_1 V_1$ and $q_2 = y_2 V_2$ and subscript 2 refers to conditions in the contracted section. The sediment transport rate at the contracted section after equilibrium is estimated must be

$$q_{s2} = \frac{W_1}{W_2} q_{s1} \qquad 3$$

The relationships of y and V at sections 1 and 2 are shown in Fig. 2 for constant q_1 and q_2.

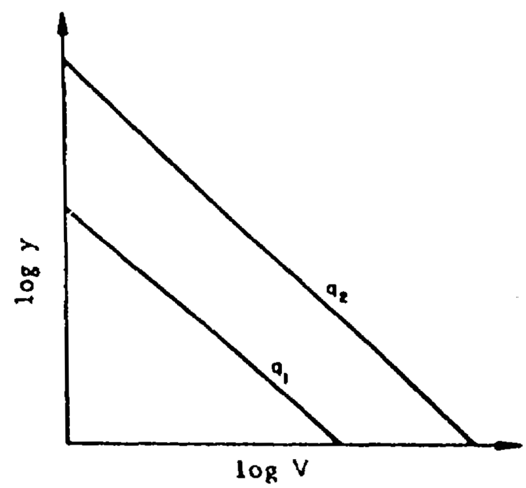


FIGURE 2 Unit discharge as a function of depth and velocity

Using the various sediment transport equations it is possible to construct curves for transport rates of sediment of given median size as functions of flow depth and velocities. An illustration of such dependence is shown in Fig. 3 using the method of Colby (Richardson et al, 1987). Now overlap Fig. 2 with Fig. 3. The result is shown in Fig. 4.

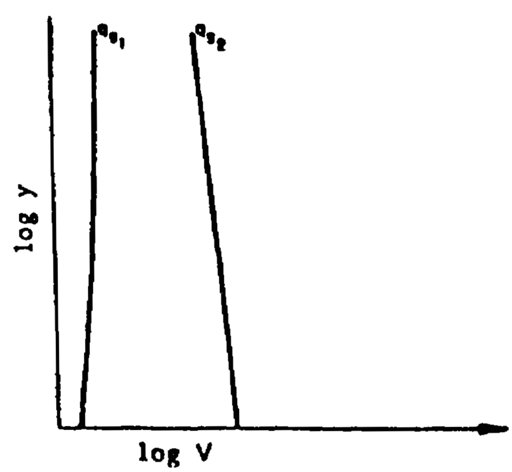


FIGURE 3. Sediment transport rate as a function of depth and velocity

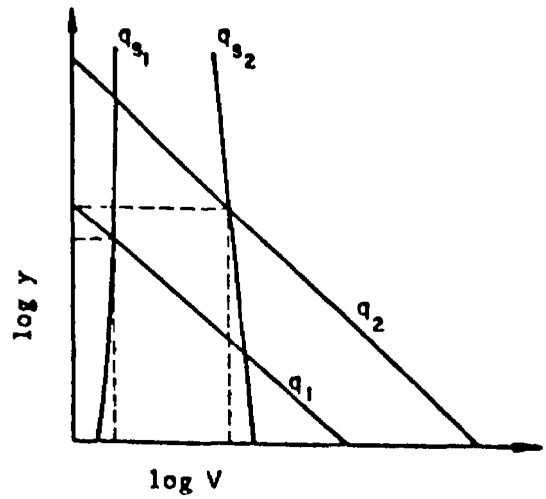


FIGURE 4. Determination of scour depth

The depth of scour due to the contraction is then

$$y_s = y_2 - y_1 \qquad 4$$

Here y_1, V_1 and W_1 are the depth, velocity and width of the approach flow, and y_2 is the general scour flow depth at the bridge. The term D_{50} is the median diameter of the bed materials at the bridge.

3. Straub's Equation - Consider the long contraction shown in Fig. 5. In the wide approach reach, at the cross section designated Section 1, the average velocity if V_1, the average depth of flow is y_1 and the width if W_1. The flowrate across Section 1 is

$$Q = V_1 \; y_1 \; W_1 \qquad 5$$

In the contracted reach at the cross section designated Section 2, the average velocity is V_2, the flow depth is y_2 and the width is W_2. The flowrate across Section 2 is

$$Q = V_2 \; y_2 \; W_2 \qquad 6$$

For a given flowrate, Q and a given contraction ratio W_2/W_1 we would like to know the depth ratio y_2/y_1 for the clear-water scour case.

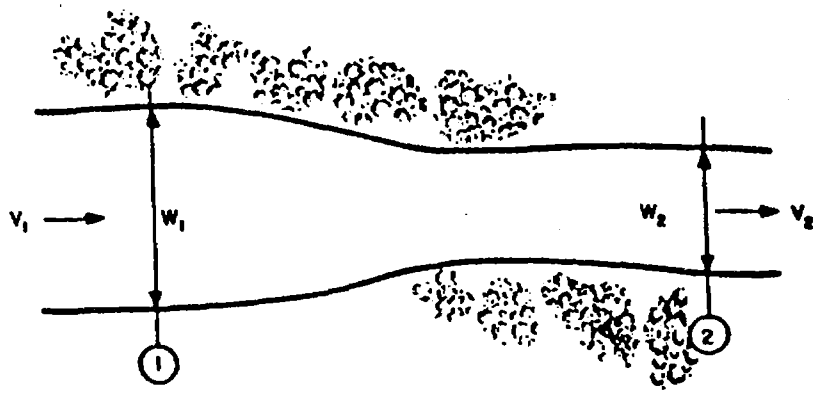


FIGURE 5 Plan view of the long contraction

In the clear-water scour case, there is no transport in the wide upstream section. The shear stress here is less than the critical shear stress (the shear stress causing initial movement of the bed particles). That is,

$$\tau_1 = \gamma y_1 S_{f1} < \tau_c \qquad 7$$

Here S_{f1} is the slope of the energy grade line at Section 1.

Assume for the time being that scour occurs in the long contraction. Scour will continue until the bed shear stress in the long contraction has been reduced to the critical shear stress. Then, at Section 2

$$\tau_2 = \tau_c = \gamma y_2 S_{f2} \qquad 8$$

When this condition is reached there is no longer a sediment transport at Section 2. As well, there is no sediment transport at Section 1 ($\tau_1 < \tau_c$). Hence the term "clear-water scour" is employed.

By employing Eqs. 7 and 8, the depth ratio is

$$\frac{y_2}{y_1} = \frac{S_{f1}}{S_{f2}} \frac{\tau_c}{\tau_1} \qquad 9$$

Manning's equation can be employed to determine the friction slope ratio. Accordingly,

$$\frac{S_{f1}}{S_{f2}} = \left(\frac{n_1}{n_2}\right)^2 \left(\frac{V_1}{V_2}\right)^2 \left(\frac{y_2}{y_1}\right)^{4/3} \qquad 10$$

so

$$\frac{y_2}{y_1} = \left(\frac{n_1}{n_2}\right)^2 \left(\frac{V_1}{V_2}\right)^2 \left(\frac{y_2}{y_1}\right)^{4/3} \frac{\tau_c}{\tau_1} \qquad 11$$

The velocity ratio V_1/V_2 is obtained by equating Eqs.5 and 6 (constant discharge) or

$$\frac{V_1}{V_2} = \frac{y_2}{y_1} \frac{W_2}{W_1} \qquad 12$$

By putting this ratio into Eq. 11, the expression

$$\frac{y_2}{y_1} = \left(\frac{n_2}{n_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{\tau_1}{\tau_c}\right)^{3/7} \qquad 13$$

is obtained for clear-water scour. If it is assumed that

$$n_1 = n_2$$

then Eq. 12 reduces to

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{\tau_1}{\tau_c}\right)^{3/7} \qquad 14$$

which is the form of clear water scour equation first developed by Straub (1940).

4.1.2 Case 3.

<u>Relief bridge where there is no bed material transport on the upstream flood plane use Laursen (1980) equation given below:</u>

$$Y_2/Y_1 = (W_1/W_2)^{6/7} [V_1^2/(120\ Y_1^{1/2}\ D_{50}^{2/3})]^{3/7} \qquad 15$$

Where

Subscript 1 refers to the upstream conditions and 2 to the width and depth in the relief bridge.

W_1 = width upstream of the relief bridge. It is estimated by assuming a point of stagnation between the main bridge and the relief bridge.

V_1 = average velocity one bridge length upstream

D_{50} = Median diameter of bed material at relief bridge.

4.1.4 Case 4.

<u>Relief bridge with bed material transport.</u> For this case use the equation given for case 1 with appropriate adjustments of the variables. This case can occure when a relief bridge is over a secondary channel on the flood plain.

5. <u>Local Scour.</u>

The basic mechanism causing local scour at a pier or abutment is the formation of a vortex at their base. The formation of these vortexes results from the pileup of water on the upstream face and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex is to remove bed materials away from the base region. If the transport rate of sediment away from the local region is greater the transport rate into the region, a scour hole develops. As the depth of scour is increased, the strength of the vortex or vortexes is reduced, The transport rate is reduced and an equilibrium is reestablished and scouring ceases.

The vortex is illistrated in Fig. 6 with their formation around a pier. With a pier, in addition to the vortex around the base, the horseshoe vortex, there is a vertical vortex downstream of the pier, the wake vortex. Both vortexes remove material from around the pier. However, immediately downstream of a long pier there is often deposition of material.

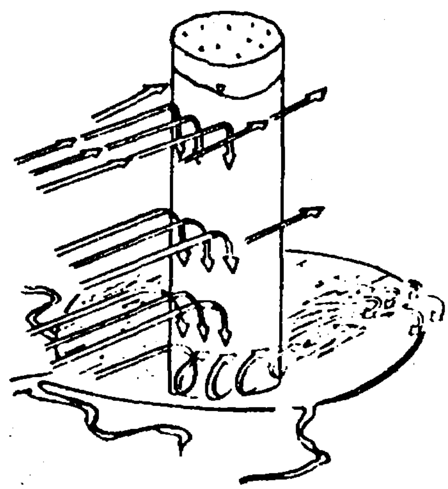


Fig. 6 Schematic representation of scour at a cylindrical pier.

5.1 <u>Factors Effecting Local Scour.</u>

The factors that effect local scour are as follows:

1) width of the pier (a),
2) projection length (a) of the abutment into the flow,
3) length of the pier (L),
4) depth of flow (y_1),
5) velocity of the approach flow (V_2),
6) size of the bed material (D),
7) angle of the approach flow to the pier or abutment (angle of attack),
8) shape of the pier or abutment,
9) bed configuration,
10) ice formation or jams and
11) debris.

1. Width of pier has a direct affect on the depth of scour. With an increase in pier width there is an increase in scour depth.

2. Projected length of an abutment into the stream affects the depth of scour. With an increase in the projected length of an abutment into the flow there is an increase in scour. However, there is a limit on the increase in scour depth with an increase in length. This limit is reached when the ratio of projected length into the stream (a) to the depth of the approaching flow (y_1) is 25.

3. Length of a pier has no appreciable affect on scour depth as long as the pier is lined up with the flow. If the pier is at an angle to the flow the length has a very large affect. At the same angle of attack doubling the length of the pier increases scour depth 33 percent. Some equations take the length factor into account by using the ratio of pier length to depth of flow or pier width and the angle of attack of the flow to the pier. Others uses the projected area of the pier to the flow in their equations.

4. Flow depth has a direct affect on scour depth. An increase in flow depth can increase scour depth by a factor of 2 or larger for piers. With abutments the increase is from 1.1 to 2.15 depending on the shape of the abutment.

5. Velocity of the approach flow increases scour depth. The

larger the velocity the deeper the scour depth. There is also a high probability that whether the flow is tranquil or rapid (subcritical of supercritical) will affect the scour depth. Most research and data is for flows with Froude Numbers much less than one (Fr. < 1).

6. Size of the bed material in the sand size range has no affect on scour depth. Larger size bed material if it will be moved by the approaching flow or by the vortexes and turbulence created by the pier or abutment will not affect the ultimate or maximum scour but only the time it takes to reach it. Very large particles in the bed material, cobbles or boulders, may armor plate the scour hole. But in the case of the Schoharie Cr. bridge collapse large riprap was in time removed from around the piers by a series of large flows (Richardson, et al, 1987). The size of the bed material also determines whether the scour at a pier or abutment is clear-water of live-bed scour. This topic is discussed later in this section.

Fine bed material (silts and clays) will have scour depths as deep or deeper than sandbed streams. This is true even if bonded together by cohesion. The affect of cohesion is to determine the time it takes to reach the maximum scour. With sand bed material the maximum depth of scour is measured in hours. With cohesive bed materials it may take days, months or even years to reach the maximum scour depth.

7. Angle of Attack of the flow to the pier or abutment has a large affect on local scour as was pointed out in the discussion of the affect of pier length above. The affect on piers will not be repeated here. With abutments the depth of scour is reduced for embankments angled downstream and is increased if the embankments are angled upstream. According to the work of Ahmad (Richardson et al, 1975 and 87) the maximum depth of scour at an embankment inclined 45 degrees downstream is reduced by 20 percent, whereas, the scour at an embankment inclined 45 degrees upstream is increased about 10 percent.

8. Shape of pier or abutment has a significant affect on

scour. With a pier, streamlining the front end reduces the strength of the horseshoe vortex reducing scour depth. Streamlining the downstream end of piers reduces the strength of the wake vertices. A square-nose pier will have maximum scour depths about 20 percent larger than a sharp-nose pier and 10 percent larger than a cylinder or round-nose pier. Abutments with vertical walls on the streamside and upstream side will have scour depths about double that of spill slope abutments.

9. Bed configuration effects the magnitude of local scour. In streams with sand bed material the shape of the bed (bed configuration) as determined by Simons and Richardson (1963) and discussed in Chapter III of HIRE, may be ripples, dunes, plane bed and antidunes. The bed configuration depends on the size distribution of the sand bed material, flow conditions and fluid viscosity. The bed configuration may change from dunes to plane bed or antidunes during an increase in flow. It may change back with a decrease in flow. The bed configuration may also change with a change in water temperature or change in concentration of silts and clays.

10.Ice and debris by increasing the width of the piers, changing the shape of piers and abutments, increasing the projected length of an abutment or causing the flow to plunge downward against the bed can increase both the local and general (contraction) scour. The magnitude of the increase is still largly undetermined. But debris can be taken into account in the scour equations by estimating how much the debris will increase the width of the pier or length of the abutment. Debris and ice affects on general (contraction) scour can also be accounted for by estimating the amount of flow blockage (decrease in width of the bridge opening) in the equations for contraction scour. Field measurements of scour at ice jams indicate the scour can be in the 10's of feet.

5.2 Clear-water and live-bed scour.

There are two conditions of local scour. These are 1) clear-water scour and 2) live-bed scour.

Clear-water scour.

Clear-water scour occurs when there is no movement of the bed material of the stream upstream of the crossing but the acceleration of the flow and vortices created by the piers or abutments causes the material at their base to move.

Live-bed scour.

Live-bed scour occurs when the bed material upstream of the crossing is also moving.

Bridges over coarse bed material streams often have clear-water scour at the lower part of a hydrograph, live-bed scour at the higher discharges and then clear-water scour on the falling stages.

Clear-water scour reaches its maximum over a longer period of time than live-bed scour, Fig. 7. This is because clear-water scour occurs mainly on coarse bed material streams. In fact clear-water scour may not reach its maximum until after several floods. Also, maximum clear-water scour is about 10 percent greater than the maximum live-bed scour.

Live-bed scour in sand bed streams with a dune bed configuration fluctuates about an equilibrium scour depth, Fig. 7. The reason for this is the fluctuating nature of the sediment transport of the bed material in the approaching flow when the bed configuration of the stream is dunes. In this case (dune bed configuration in the channel upstream of the bridge) maximum depth of scour is about 30 percent larger than equilibrium depth of scour.

The maximum depth of scour is the same as the equilibrium depth of scour for live-bed scour with a plain bed configuration. With antidunes occurring upstream and in the bridge crossing the maximum depth of scour from the limited research of Jain and Fisher (1979) is about 20 percent greater than the equilibrium depth of scour.

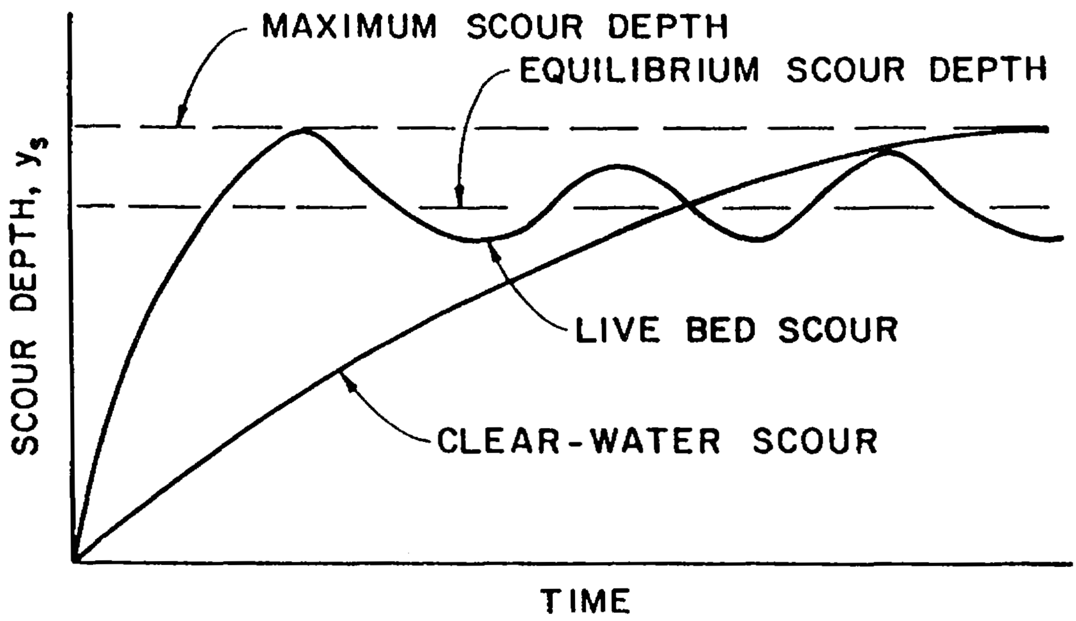


Figure 7 Scour Depth as a Function of Time.

5.3 Armoring.

Armoring occurs on a stream or in a scour hole when the forces of the water during a particular flood are unable to move the larger sizes of the bed material. This protects the underlying material from movement. Scour around an abutment or pier may initially occur but as the scour hole deepens the courser mbed material moves down in the hole and protectes the bed so that the full scour potential is not reached.

When armoring occurs, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor in the scour holes and thus limit futher scour for a particular discharge. This armoring affect can decrease scou hole depghs which were predicted to occur based on formulae developed for sand or other fine material channels for particular flow conditions. When larger flow conditions occur the armor layer can be broken and the scour hole deepened either untill an new armor layer is developed or the maximum scour as given by the sand bed equations is reached. Unfortunatly knowlege of how to predict the decrease in scour hole depth when there are large particals in the bed material is lacking. Research in New Zewland by Raudkivi (1986) and in Washington State (Copp and Johnson, 1987) gives a bases for calculating the decrease in scour depth by armoring but their equations need field verification. The results of this research for pier scour will be given later.

5.4 Estimating local scour depths.

Equations for estimating local scour are based on three methods of analysis. These methods are;

1) Dimensional analysis of the basic variables causing local scour.
2) The use of transport relations in the approching flow and in the scour hole.
3) Regresion analysis of the available data.

Equations for estimating local scour at abutments or piers developed by the three methods are given in the next sections. In HEC 18 (FHWA, 1989) and in the June 1988 "Interim Procedures for Evaluating Scour at Bridges" (FHWA, 1988) only one method or equation is recommended. The additional equations are given in this report for basis of comparison, to be used in additional study of a particular bridge site and for use in reasearch. It should be noted that these equations were developed from laboratory experiments with limited field data.

In analysis for scour the engineer should evaluate his problem and select the equation or method that in his judgement best suits the case at hand. Sometimes it may be nessissary to use more than one equation or method and then use Engineering judgment in selecting the local scour depth. For example, if the stream contains large qualities of course bed material. Then use both the sand bed equation and the armoring equation. Based on knowlege of the stream, the bed material, the flows and type of highway select the value of the scour.

6. Local Scour at Abutments.

6.1 General.

Equations for predicting scour depths are based almost entirely on either laboratory data or inductive reasoning from sediment continuity equations. There are little field data to compare abutment scour equations. Equations for estimating abutment scour are derived by three methods.

6.1.1 Types of Equations.

1) From dimensional anlaysis of the variables and developing relationships among the major dimensionless paramenters such as, the ratio of scour depth y_s to flow depth y_1; ratio of abutment length a to flow depth y_1; the Froude Number Fr, etc. Liu, et al's 1961 equation given here is an example.

2) From the use of transport relations and the change in trasport because of the aceleration of the flow caused by the abutment. Laursen's (1980) equations given here are an example.

3) From regresion analysis of available data. Froehlich's 1988 equation given here is an example.

6.1.2 Position of Abutments.

Abutments can be set back from the natural stream bank or can project out into the flow; they can have various shapes (vertical walls, spill through slopes) and they can be set at an angle to the flow. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow that is intercepted by the approaches to the bridge and returned to the stream at the abutment.

Scour at abutments can be caused by the abutment porjecting into the flow, it can be caused by the approches to the bridge intercepting overland flow and forcing it back into the channel at the abutment, or it can be a combination of conditions. The various conditions (cases) are given in Table 1 and illustrated in Fig. 8. In Table 1 equations are given for each case. No single equation is recommended for a given situation when more than one equation is applicable, because with the lack of field data for verification, it is not know which equation is best. It is recommended that the designer determine what case fits the design situation and then use all equations that apply to the case. IT IS MOST IMPORTANT THAT THE COMMENTARY ON EACH OF THE EQUATIONS BE READ AND UNDERSTOOD PRIOR TO ATTEMPTING TO USE THE EQUATIONS FOR DESIGN PURPOSES. Engineering judgment must be used to select the depth of foundations. The designer should take into consideration the potential cost of repairs to an abutment and danger to the travelling public in selecting scour depths. Finally design measures such as spur dikes and riprap should be used to increase the safety of the bridge.

o Comments on Table 1 and Fig. 8.

1) Equations for these cases (except for Case 6) are based on laboratory studies with little or no field data.

2) The factor a/y_1 = 25 as a limit for Cases 1-5 is rather arbitrary but it is not practical to assume that scour depth, y_s, would continue to increase with an increase in abutment length "a".

3) There are two general shapes for abutments. These are vertical wall abutments with wing walls and spill-through abutments, Fig. 9. Depth of scour is about double for vertical wall abutments as compared with spill through abutments.

6.1.3 Maximum Depth of Scour

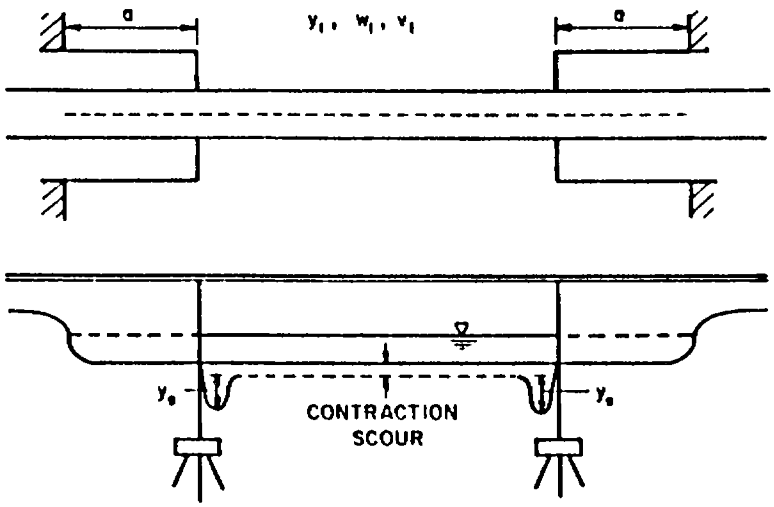
1) For live-bed scour with a dune bed configuration, the maximum depth of scour is about 30 percent greater than equilibrium scour depth given by Liu, et al's (1961) equations (Eqs. 16 and 17). Therefore the values of scour that are calculated for these equations should be increased by 30 percent when the bed form is dunes upstream of the bridge. The reason for this is that the research that was used for determining scour depth for the live-bed scour case was run with a dune bed and equilibrium scour was measured.

2) For clear-water scour (Eq. 20), the maximum depth of scour is about 10 percent greater than live-bed scour; however, there is no need to increase the scour depths because the equations predict the maximum scour.

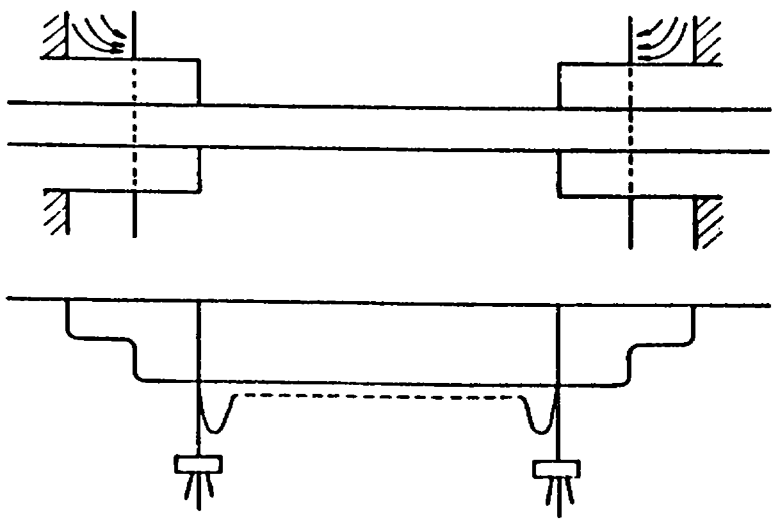
TABLE 1 SUMMARY OF ABUTMENT SCOUR EQUATIONS

CASE	ABUTMENT LOCATION	OVERBANK FLOW	VALUE OF a/y_1	BED LOAD CONDITION	ABUTMENT TYPE	EQUATION NUMBER
1	Projects into channel	No	$a/y_1 < 25$	Live bed	Vertical wall	17,18,22
					Spill through	16,18,22
				Clear water	Vertical Wall	20,21
					Spill through	20,21
2	Projects into channel	Yes	$a/y_1 < 25$	Live bed	Not Designated	18,24
				Clear water	Not Designated	20,24
3	Set back from main channel	Yes	$a/y_1 < 25$	Clear water	Not Designated	20
4	Relief bridge on floodplain	Yes	$a/y_1 < 25$	Clear water	Not Designated	20
5	Set at edge of main channel	Yes	$a/y_1 < 25$	Live bed	Not Designated	24
6	Not designated	Yes	$a/y_1 > 25$	Not designated	Not designated	25
7	Skewed to stream	-	-	-	-	*

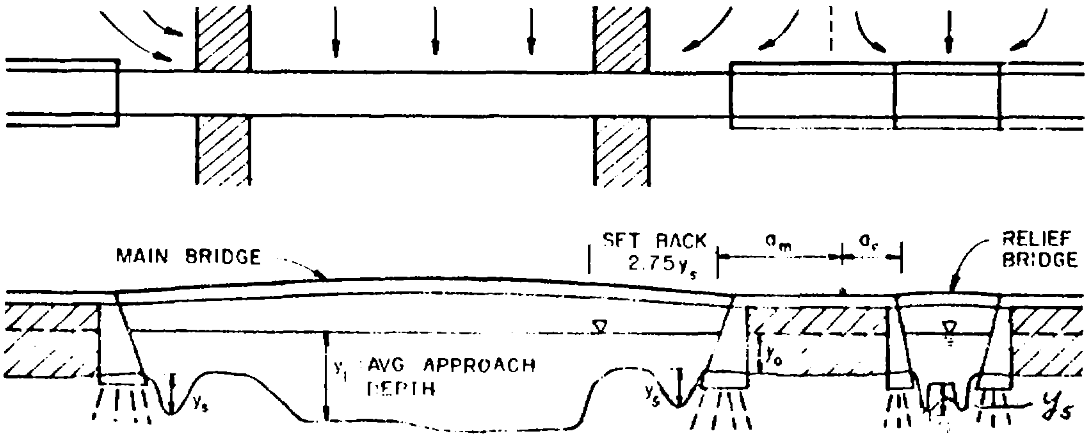
* Adjust scour estimate for equations 16 - 25 using Fig. 17.



CASE 1 ABUTMENTS PROJECT INTO CHANNEL, NO OVERBANK FLOW

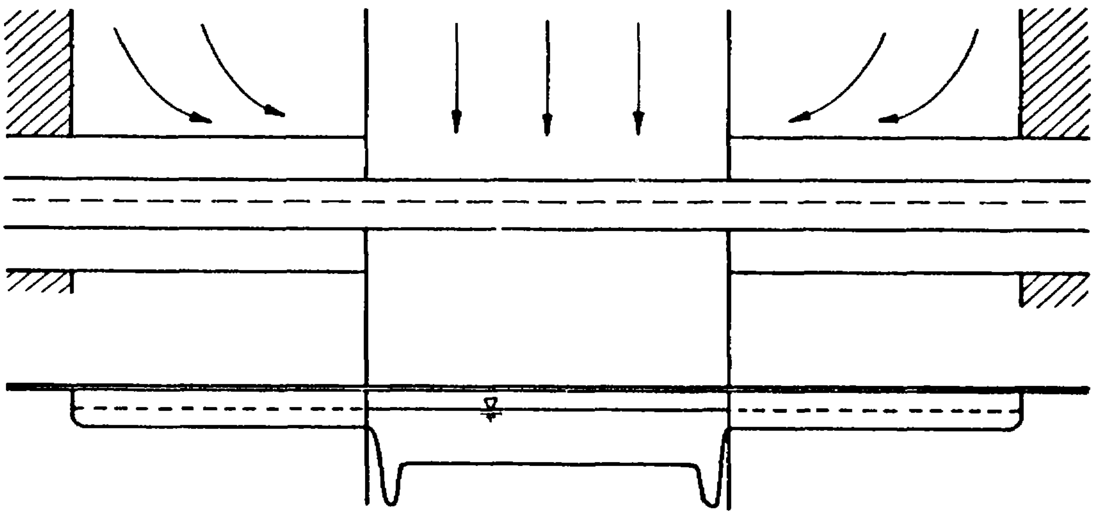


CASE 2 ABUTMENTS PROJECT INTO CHANNEL, OVERBANK FLOW

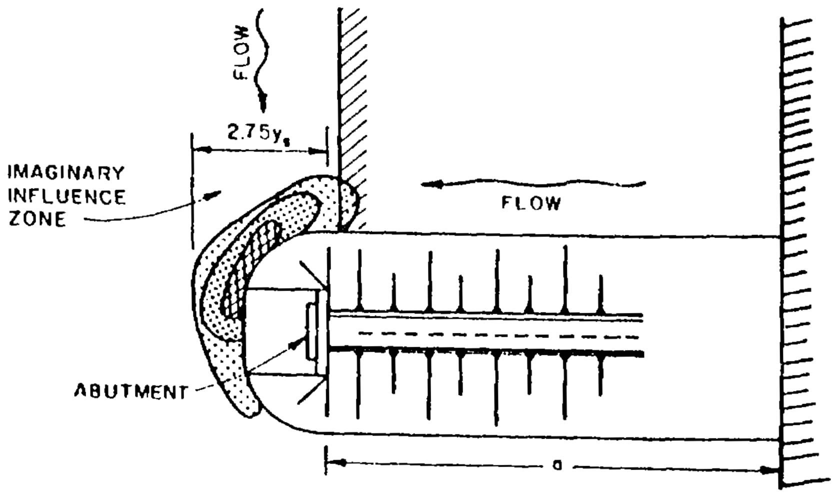


CASE 3 ABUTMENT SETBACK FROM THE CHANNEL MORE THAN 2.75 y_s
CASE 4 RELIEF BRIDGE

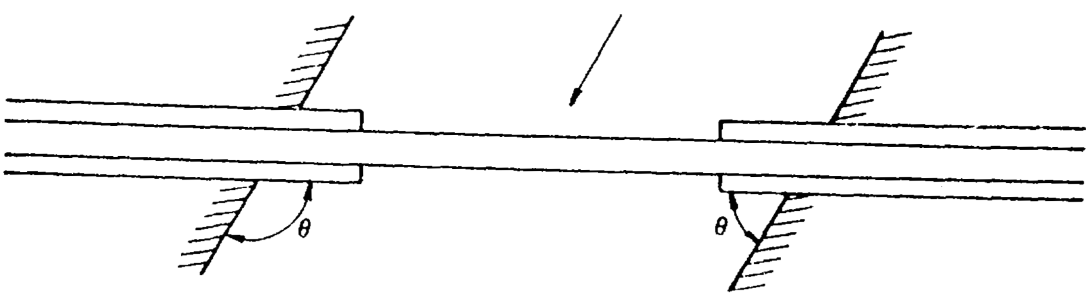
Fig. 8 Abutment Scour,Cases 1, 2, 3 AND



CASE 5 ABUTMENT SET AT EDGE OF CHANNEL, OVERBANK FLOW

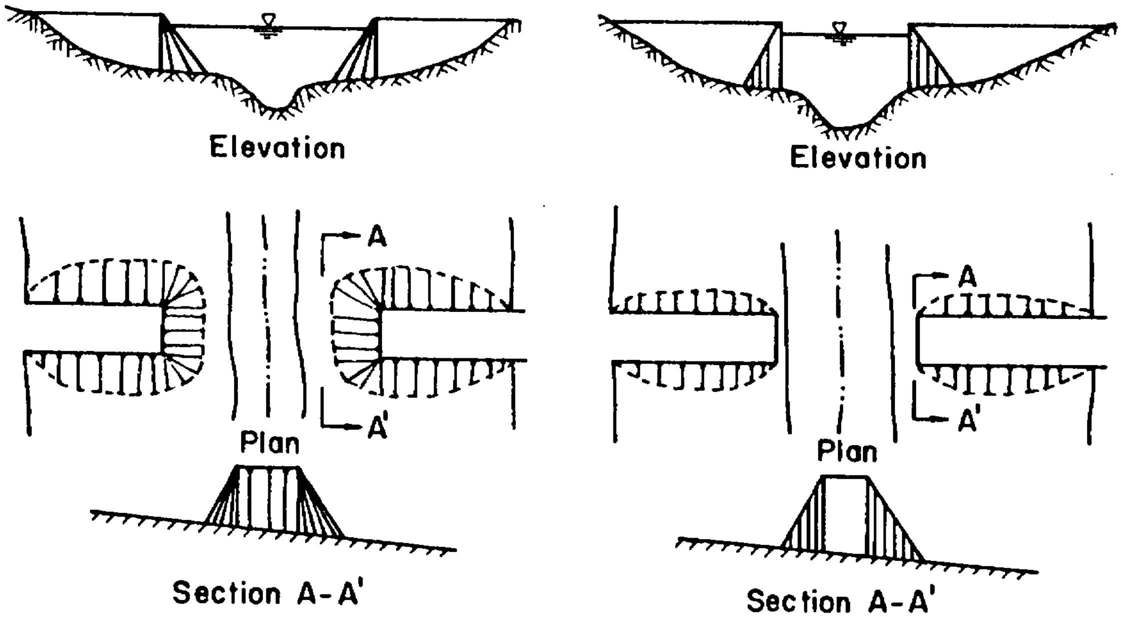


CASE 6 ABUTMENT LENGTH, a, TO FLOW DEPTH, y_1, RATIO > 25



CASE 7 ABUTMENT SET AT AN ANGLE, θ, TO THE FLOW

Fig. 8 Cont. Abutment Scour, Cases 5, 6 AND 7



(A) SPILL THROUGH (B) VERTICAL WALL

Fig. 9 **Abutment shape**

6.2 Calculating Abutment Scour for the Different Cases.

CASE 1 ABUTMENTS PROJECT INTO CHANNEL, NO OVERBANK FLOW, Fig. 10.

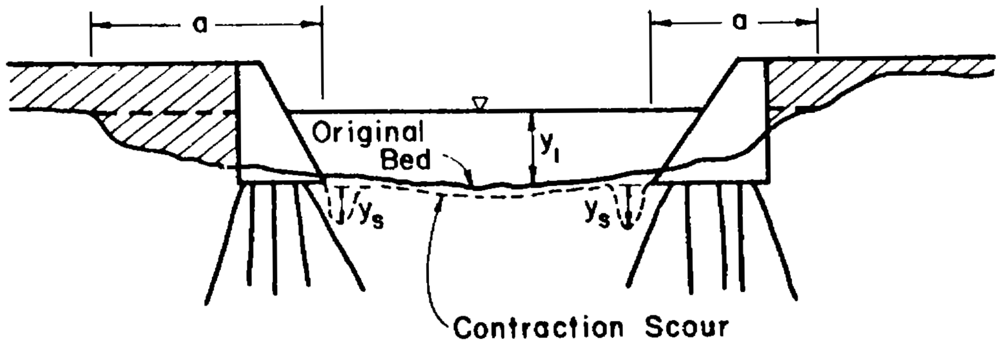


FIGURE 10 DEFINITION SKETCH FOR CASE 1 ABUTMENT SCOUR

Seven equations are given for this case (Eqs. 16, 17,18, 19, 20, 21 and 22). These are equations based on dimensional analysis, transport relations and regression analysis. These equations are limited to cases where $a/y_1 < 25$. For $a/y_1 > 25$ go to Case 6.

o Liu, et al's Equations.

Live-bed scour at a spill through abutment, Eq. 16.

According to the studies of Liu, et al., (1961) the equilibrium scour depth for local live-bed scour in sand at a stable spill slope when the flow is subcritical is determined by Equation 16.

$$\frac{y_s}{y_1} = 1.1 \left(\frac{a}{y_1}\right)^{0.40} Fr_1^{0.33} \qquad 16$$

y_s = equilibrium depth of scour (measured from the mean bed level to the bottom of the scour hole)

y_1 = average upstream flow depth in the main channel

a = abutment and embankment length (measured at the top of the water surface and normal to the side of the channel from where the top of the design flood hits the bank to the outer edge of the abutment)

Fr_1 = upstream Froude number

$$Fr_1 = \frac{V_1}{(gy_1)^{0.5}}$$

Live bed scour at a vertical wall abutment, Eq. 17.

If the abutment terminates at a vertical wall and the wall on the upstream side is also vertical, then the scour hole in sand calculated by equation 16 nearly doubles (Liu, et al, 1961 and Gill, 1972).

$$\frac{y_s}{y_1} = 2.15 \left(\frac{a}{y_1}\right)^{0.40} Fr_1^{0.33} \qquad 17$$

o Laursen's Equations.

Live-bed scour at vertical wall abutment, Eq. 18.

Laursen (1980) suggested two relationships for scour at vertical wall abutments for Case 1. One for live-bed scour and another for clear-water scour depending on the relative magnitude of the bed shear stresses to the critical shear stress for the bed material of the stream. For live-bed scour ($\tau_1 > \tau_c$), use Eqs. 18 or 19.

$$\frac{a}{y_1} = 2.75 \left(\frac{y_s}{y_1}\right) \left[\left(\frac{1}{11.5} \frac{y_s}{y_1} + 1\right)^{1.7} - 1 \right] \qquad 18$$

Simplified form: $$\frac{y_s}{y_1} = 1.5 \left(\frac{a}{y_1}\right)^{0.48} \qquad 19$$

Clear-water scour ($\tau_1 < \tau_c$) at vertical wall abutment, Eq. 20.

$$\frac{a}{y_1} = 2.75 \left(\frac{y_s}{y_1}\right) \left[\frac{\left(\frac{1}{11.5}\frac{y_s}{y_1} + 1\right)^{7/6}}{\left(\frac{\tau_1}{\tau_c}\right)^{1/2}} - 1 \right] \qquad 20$$

τ_1 = shear stress on the bed upstream
τ_c = critical shear stress of the D_{50} of the upstream bed material. The value of τ_c can be obtained from Fig. 11

Scour at other abutment shapes.

Scour values given by Laursen's equations are for vertical wall abutments. He suggests the following multiplying factors for other abutment types for small encroachment lengths:

Abutment Type	Multiplying Factor
45 degree Wing Wall	0.90
Spill-Through	0.80

Laursen's equations are based on sediment transport relations. They give maximum scour and include contraction scour. FOR THESE EQUATIONS, DO NOT ADD CONTRACTION SCOUR TO OBTAIN TOTAL SCOUR AT THE ABUTMENT.

Laursen's equations require trial and error solution. Curves developed by Chang (1987) are given in Fig. 12. Note that the equations have been truncated at a value of y_s/y equal to 4.

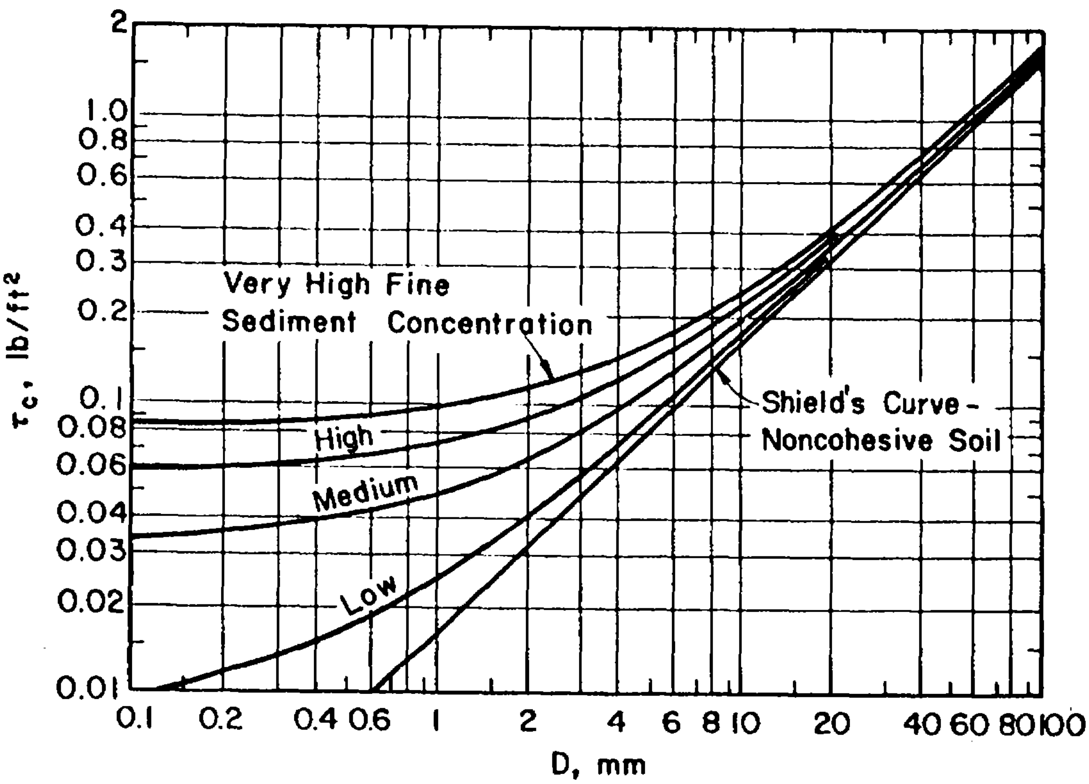
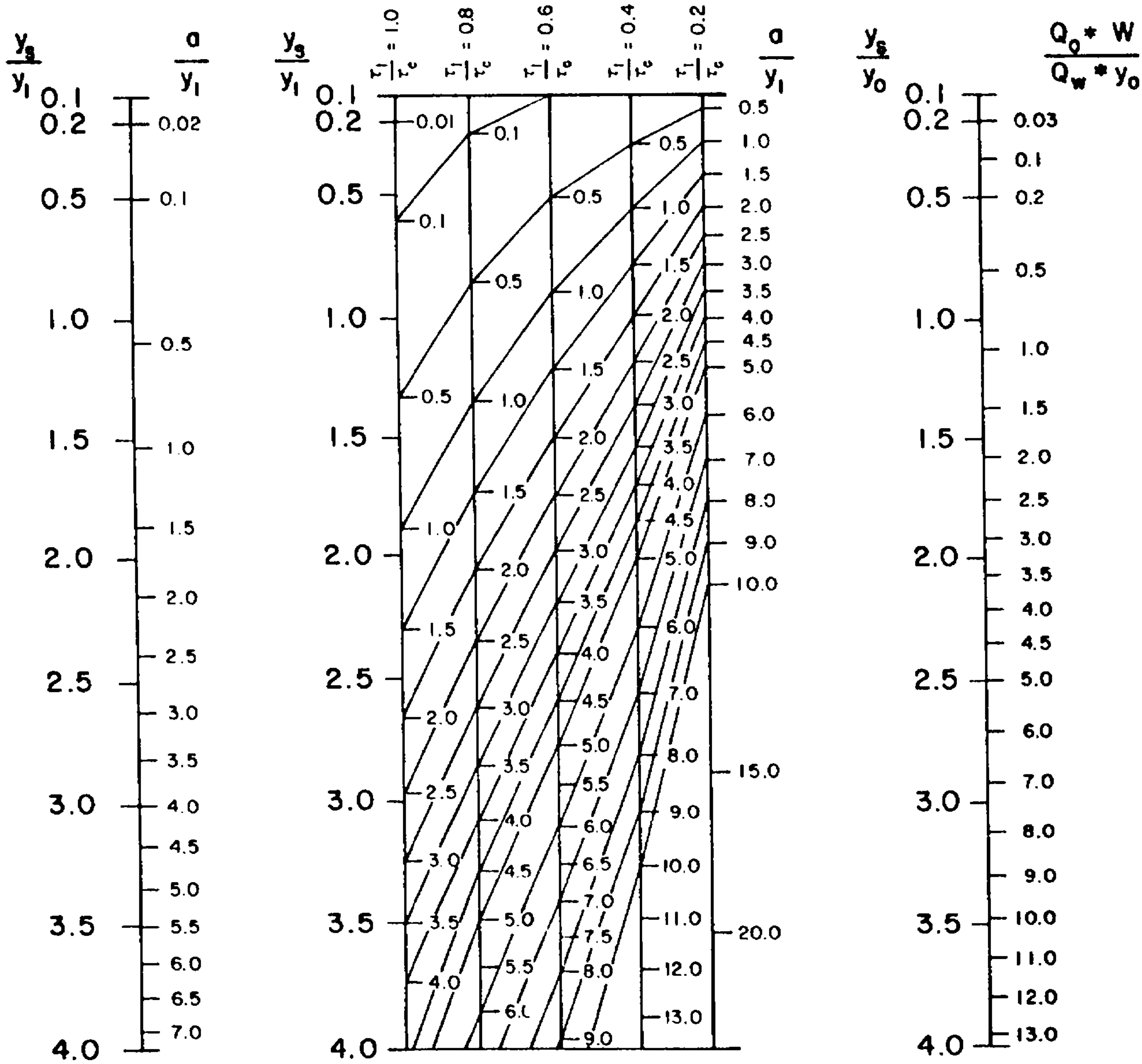


Fig. 11 Critical shear stress as a function of bed material size and suspended fine sediment.



Eq. 18

Eq. 20

Eq.24

FIG. 12 Nomograph for Abutment Scour

o Froehich's Equations.

Clear-water scour at an abutment, Eq. 21.

Froehich (1987) using dimensional analysis and multiple regression analysis of 164 clear-water scour measurements in a laboratory flumes developed the following equation:

$$\frac{y_s}{y_1} = 0.77\ K_1\ K_2\ \left(\frac{a'}{y_1}\right)^{.63} Fr^{1.19} \left(\frac{y_1}{D_{50}}\right)^{.44} G^{-1.88} + 1.0 \qquad 21$$

Live-bed scour at an abutment, Eq. 22.

He analysed 170 live-bed scour measurements to obtain the following equation:

$$\frac{y_s}{y_1} = 2.29\ K_1\ K_2\ \left(\frac{a'}{y_1}\right)^{.42} Fr^{.61} + 1.0 \qquad 22$$

Where

K_1 coefficient for abutment shape.
K_2 coefficient for angle of embankment to flow
a' length of abutment projected normal to flow
Fr Froude number of flow upstream of abutment
G geometric standard deviation of bed material
y_1 depth of flow at abutment
y_s scour depth

Values of K_1	K_1
Vertical abutment	1.0
Vertical abutment with wingwalls	.82
Spillthrough abutment	.55

Values of K_2 $\quad K_2 = \left(\frac{\theta}{90}\right)^{.13}$

θ angle of embankment to flow in degrees

Values of a' $\quad a' = A_e/y_1$

A_e = flow area of approach cross section obstructed by embankment

$G = (D_{84}/D_{16})^{.5}$

D_{50}, D_{84}, D_{16} are size of bed material. The subscript indicates the per cent finer than.

$Fr = V_e/(gy_1)^{.5}$
V_e = velocity of flow approching abutment

The + 1 in Froehlich's equation is a safety factor that makes the equation predict a scour depth larger than any of the measured scour depths in the experiments.

6.2.1 CASE 2 ABUTMENT PROJECTS INTO THE CHANNEL, OVERBANK FLOW

No bed material is transported in the overbank area and $a/y_1 < 25$. This case is illustrated in Fig. 13.

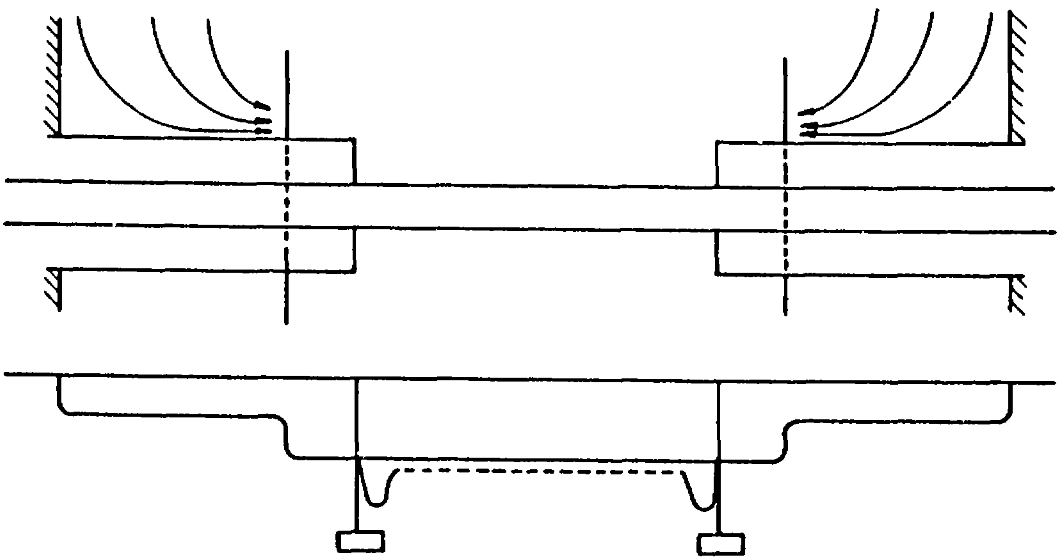


Fig. 13 Bridge Abutment in Main Channel and Overbank Flow.

Equation 18 or 20 should be used to calculate the scour depth with abutment length "a" determined by equation 23. Engineering judgment must be used in selecting the scour depth for design purposes. Equation 24 can also be used for this case with the appropriate selection of variables.

Live bed scour ($\tau_1 > \tau_c$) use equations 18 and 24.

Clear water scour ($\tau_1 < \tau_c$) use equations 20 and 24.

$$a = \frac{Q_o}{V_1 y_1} \qquad 23$$

τ_1 = The shear stress in the main channel.
τ_c = The critical shear stress for the bed material in the main channel. The value can be determined from Fig. 11
Q_o = Flow obstructed by abutment and bridge approach.
y_1 = Average upstream flow depth in the main channel.
V_1 = Average velocity in the main channel.

6.2.3 **CASE 3 ABUTMENT IS SET BACK FROM MAIN CHANNEL MORE THAN 2.75 y_s**

There is overbank flow with no bed material transport (clear-water scour). Fig. 14 illustrates this case.

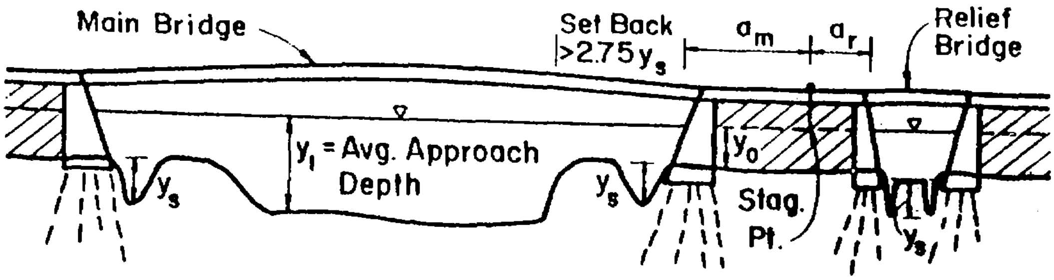


Fig. 14 Bridge Abutment Set back from Main Channel Bank and Relief Bridge .

With no bed material transport in overbank flow, scour at a bridge abutment, set back more than 2.75 times the scour depth from the main channel bank line, can be calculated using Eq.20 from Laursen (1980) with:

τ_o = Shear stress on the overbank area upstream of the abutment.
τ_c = Critical shear stress of material inoverbank area. Can be determined from Fig. 11 using the D_{50} of the bed material of the cross-section under consideration.

When there are relief bridge in the overbank areas "a" in Eq. 20 is taken as a_m.

The lateral extent of the scour hole is nearly always determinable from the depth of scour and the natural angle of repose of the bed material. Laursen suggested that the width of the scour hole is $2.75y_s$.

6.4 CASE 4 ABUTMENT SCOUR AT RELIEF BRIDGE

Scour depth for a relief bridge on the overbank flow area having no bed material transport is calculated using Eq. 20 where y_1 is average flow depth on the flood plain. Use a_r for a in the equation. Draw stream lines or use field observations to delineate where the separation point is for the flow going to the main channel and to the relief bridge. (See Fig. 14)

6.5 CASE 5 ABUTMENT SET AT EDGE OF CHANNEL

The case of scour around an abutment set right at the edge of the main channel, as sketched in Fig. 15 can be calculated with Eq. 24 proposed by Laursen (1980) when $\tau_o < \tau_c$ on the flood plain.

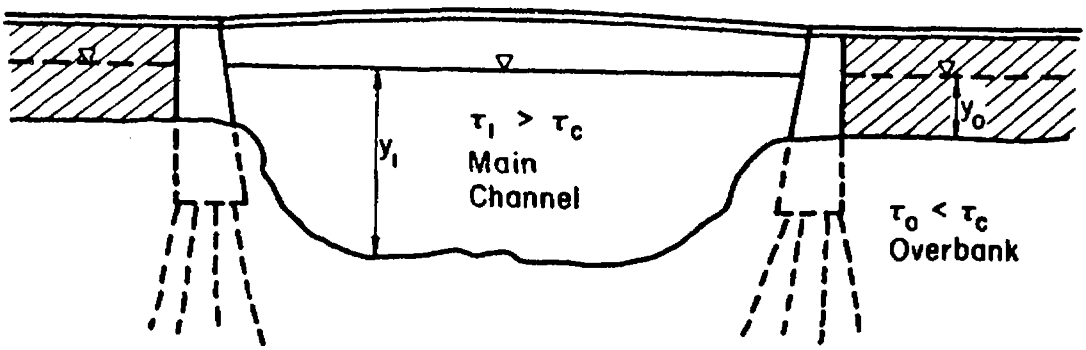


Fig. 15 Abutment set at Edge of Main Channel.

$$\frac{Q_o}{q_{mc} y_o} = 2.75 \left(\frac{y_s}{y_o}\right) \left[\left(\frac{1}{4.1} \frac{y_s}{y_o} + 1\right)^{7/6} - 1 \right] \quad 24$$

q_{mc} = the unit discharge in the main channel, Q/W
Q = Discharge in main channel
W = width of the main channel
Q_o = overbank flow discharge
y_o = overbank flow depth

Note that if there is no overbank flow for this case then there is no appreciable scour.

6.5.1 Values of calculated scour depth for the five different cases are given in Fig. 16.

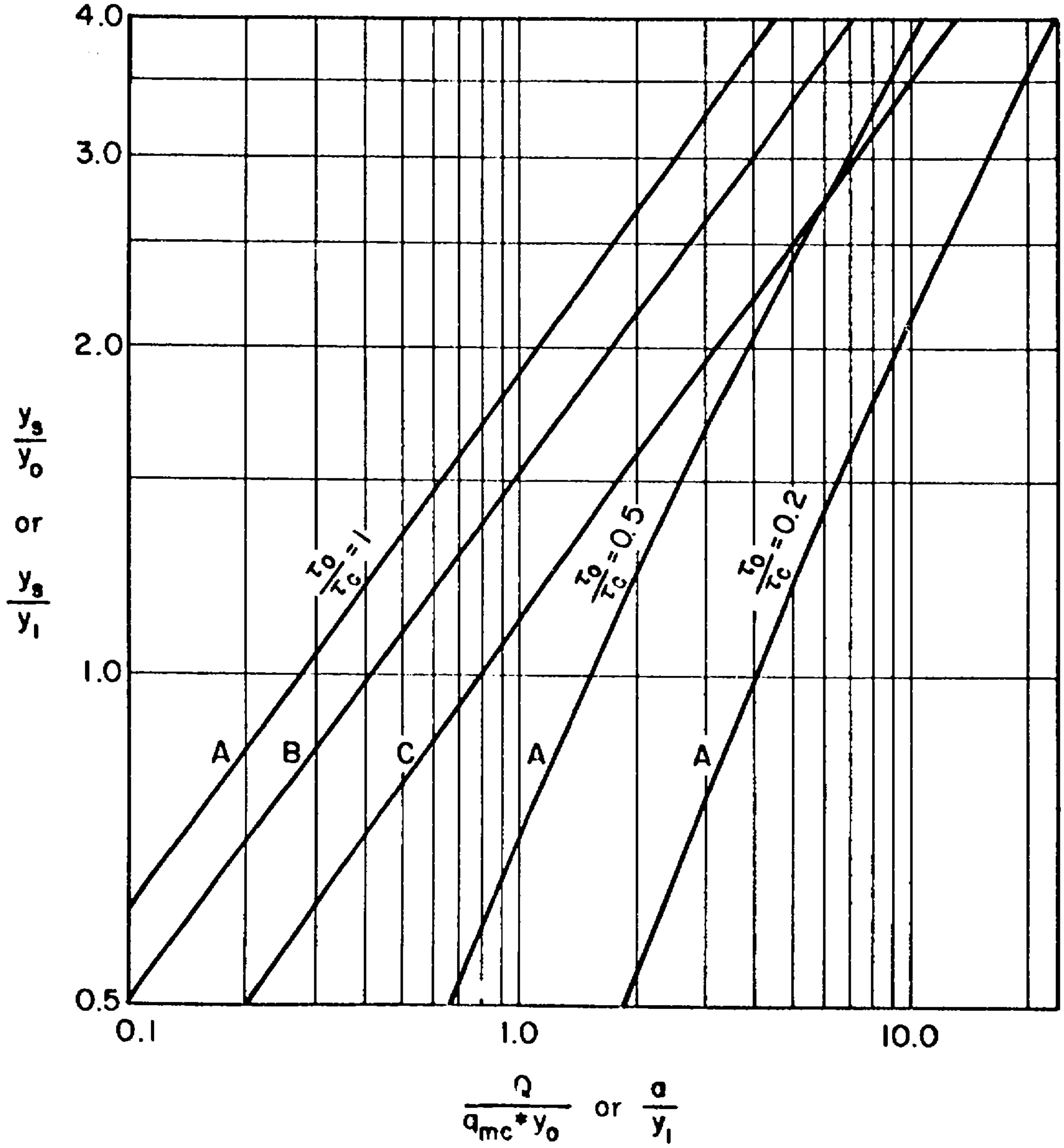


Fig. 16 Calculated Scour Depth from Equations 18, 20 and 24 (A is Eq. 20, B is Eq. 18 and C is Eq. 24)

6.6 CASE 6 SCOUR AT ABUTMENTS WHEN $a/y_1 > 25$

Field data for scour at abutments for various size streams are scarce, but data collected at rock dikes on the Mississippi indicate the equilibrium scour depth for large a/y_1 values can be estimated by equation 25:

$$\frac{y_s}{y_1} = 4\ Fr_1^{0.33} \qquad 25$$

The data are scattered, primarily because equilibrium depths were not measured. Dunes as large as 20 to 30 feet high move down the Mississippi and associated time for dune movement is very large in comparison to time required to form live-bed local scour holes. Nevertheless, it is believed that these data represent the limit in scale for scour depths as compared to laboratory data and enables useful extrapolation of laboratory studies to field installations.

Accordingly, it is recommended that Eqs. 18 through 24 be applied for abutments with $0 < a/y_1 < 25$ and Eq. 25 be used for $a/y_1 > 25$.

6.7 CASE 7 ABUTMENTS SKEWED TO THE STREAM

With skewed crossings, the approach embankment that is angled downstream has the depth of scour reduced because of the streamlining effect. Conversely, the approach embankment which is angled upstream will have a deeper scour hole. The calculated scour depth should be adjusted in accordance with the curve of Fig. 17 which is patterned after Ahmad (1953).

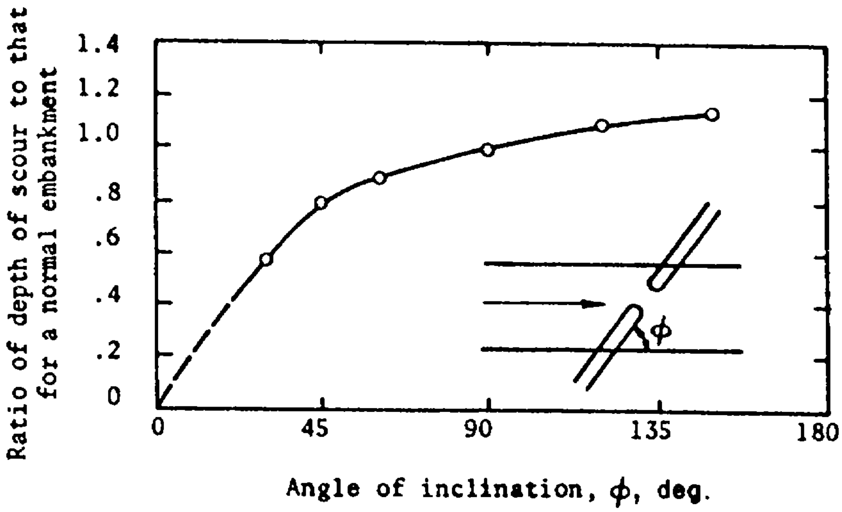


Fig. 17 Scour estimate adjustment for skew.

7. Local Scour at Piers.

7.1 Introduction.

Local scour at piers is a function of bed material size, flow characteristics, fluid properties and the geometry of the pier. The subject has been studied extensively in the laboratory but there is very little field data. As a result of the many studies there are many equations. In general, the equations are for live-bed scour in cohesionless sand bed streams, and they give similar results. In this section we willgive several equations. These equations will be as follows:

1. Colorado State University 's (CSU) equation.
2. Jain and Fisher's equation.
3. Graded and/or armored streambeds equations.
4. Froelich's equations.

As will be explained in the following paragraphs the CSU equation is recomended but the other equations are given for compaison, research and for special cases such as streams with a large quantity of lardge size particals. It is believed that the CSU equation will give the ultimate scour. Engineering judgment will be needed in the case of the other equations.

Sterling Jones (1983) compared many of the more common equations. His comparison of these equations is given in Fig. 18 Some of the equations have velocity as a variable (normally in the form of a Froude number). However some equations, such as Laursen do not include velocity. A Froude number of 0.3 was used (Fr = 0.3) in Fig. 18 for purposes of comparing commonly used scour equations. In Fig. 19 the equations are compared with some field data measurements. As can be seen from Fig. 18 the CSU equation encloses all the points but gives lower values of scour than Jain, Laursen and Niel's equations. The CSU equation includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. Chang (1988) points out that Laursen's (1980) equation is essentially a special case of the CSU equation with the Fr = 0.5.

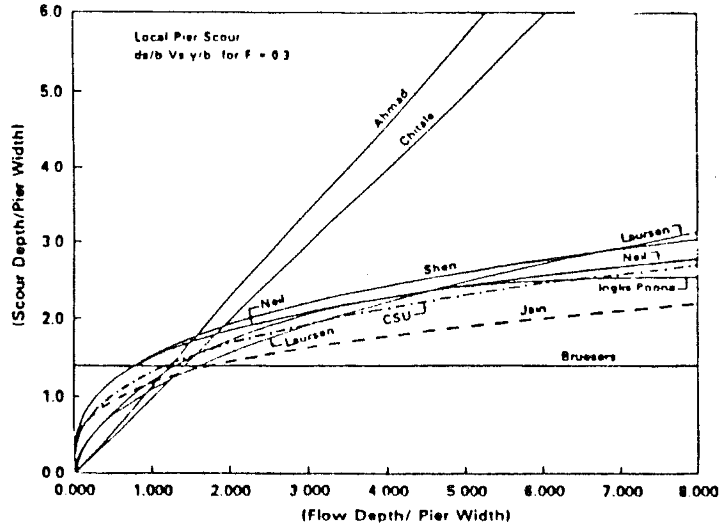


FIG. 18 COMPARISON OF SCOUR FORMULAS FOR VARIABLE DEPTH RATIOS (y/a) (JONES 1983)

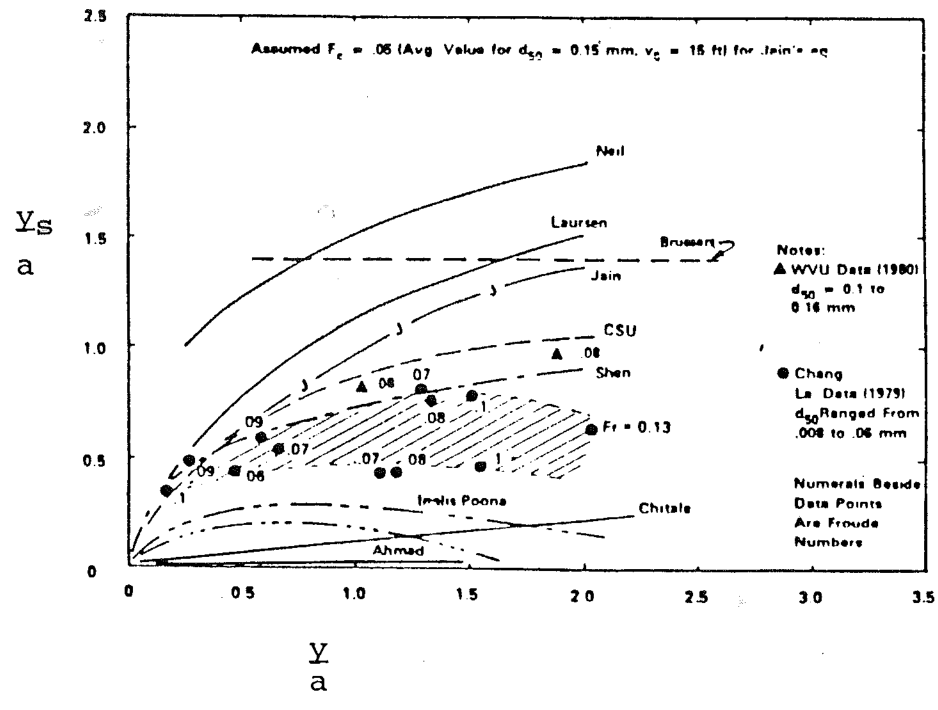


FIG. 19 COMPARISON OF SCOUR FORMULAS WITH FIELD SCOUR MEASUREMENTS (JONES, 1983)

The equations illustrated in Figs. 18 and 19 do not take into account the possibility that larger sizes in the bed material could armor the scour hole. That is, the large sizes in the bed material will at some depth of scour limit the scour depth. Raudkivi (Raudkivi and Sutherland, 1981, and Raudkivi and Ettema, 1983, Raudkivi, 1986) studied pier scour in streams with large particles in the bed. Washington State Deptarment of Transportation (Copp and Johnson, 1987, and Copp, Johnson, and McIntosh, 1988) developed an equation based on Raudkivi's researh for streams with a large range of partical sizes which would tend to armor the scour hole. The significance of this factor of armoring of the scour hole over a long time frame and over many floods is not known. Therefore, their equation is not recommended for use at this time. However it is given in this manual.

For the determination of pier scour, the Colorado State University's equation is recommended for both live-bed and clear water scour. With a dune bed configuration the equation predicts equilibrium scour depths and maximum scour will be 30% greater. For flow with plane bed configuration or antidunes, CSU's equation gives the maximum scour.

The extent to which a pier footing or pile cap affects local scour at a pier is not clearly determined. Under some circumstances the footing may serve as a scour arrester, impeding the horseshoe vortex and reducing the depth of scour hole. In other cases where the footing extends above the stream bed into the flow, it may serve to increase the effective width of the pier, thereby increasing the local pier scour. As an interim guide, if the top of the pier footing is slightly above or below the stream bed elevation (taking into account the effect of contraction scour), use the width of the pier shaft for the value of "a" in the pier scour equation. If the pier footing projects well above the stream bed to the extent that it significantly obstructs the flow, use the width of the pier footing for the value of "a". Interpolate between these two values depending upon the extent to which the footing may be expected to affect the local scour patterns.

7.2 Pier Scour Equations.

7.2.1 CSU's Equation

The Colorado State University's equation (Richardson et al, 1975) is as follows:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ \left(\frac{a}{y_1}\right)^{0.65}\ Fr_1^{0.43} \qquad 26$$

Where:
y_s = scour depth
y_1 = flow depth just upstream of the pier
K_1 = correction for pier shape from Table 2 and Fig. 20.
K_2 = correction for angle of attack of flow from Table 3
a = pier width
Fr_1 = Froude number = $y_1/(gy_1)^{0.5}$

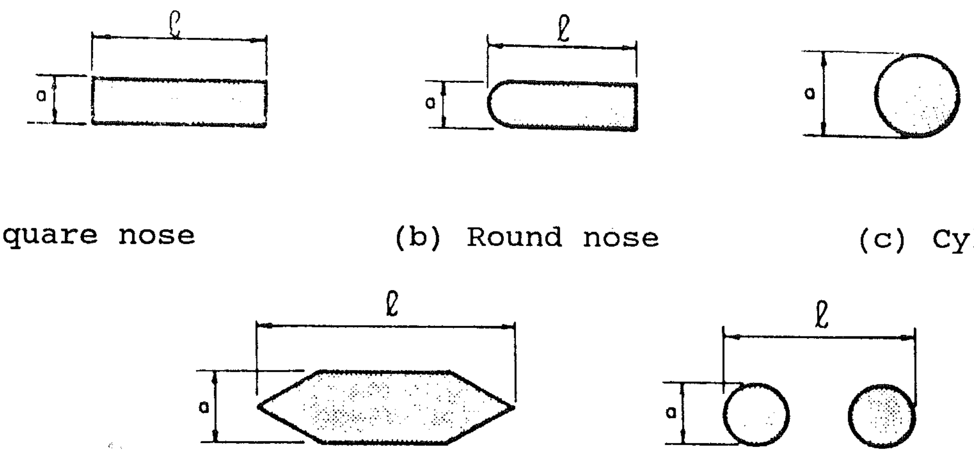
TABLE 2 Correction Factor, K_1 for Pier Type

	Type of Pier	K_1
(a)	Square nose	1.1
(b)	Round nose	1.0
(c)	Circular cylindar	1.0
(d)	Sharp nose	0.9
(e)	Group of cylindars	1.0

TABLE 3 Correction factor, K_2 for angle of attack of the flow

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length of pier



(a) Square nose (b) Round nose (c) Cylindar

(d) Sharp nose (e) Group of cylindars

FIG. 20 COMMON SHAPES

Cylindrical piers have been widely investigated in the laboratory The exponents in Eq. 26 were determined from laboratory data shown in Fig. 27. In this figure, the abscissa is labeled $(a/y_1)^3 Fr_1{}^2$ to spread the data.

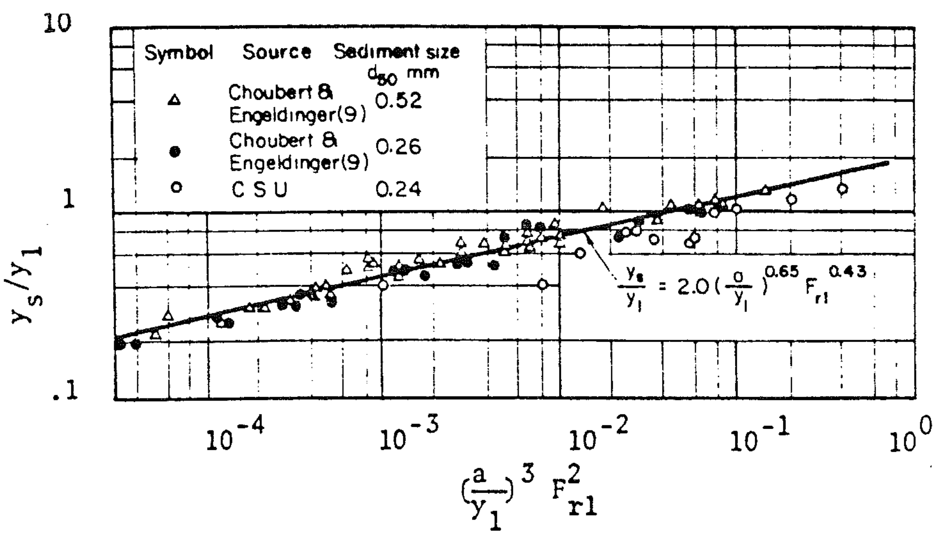


Fig. 27 Results of laboratory experiments for scour at circular piers.

7.2.2 Jains and Fishers Equation.

Jain and Fisher (1979) studied in the laboratory local pier scour at large Froude numbers. They found that scour at a circular pier in sediment transport regime ($Fr > Fr_c$) first slightly decreases and then increases with th increase in the Froude number. Scour depth at high Froude numbers is larger than the maximum clear-water scour. The contribution of bed-form scour to the total scour depth in the upper flow regiem becomes significant with higher flow velocities. They developed the folowing two equations:

For live-bed scour $(Fr - Fr_c) > 0.2$

$$y_s/a = 2.0\ (Fr - Fr_c)^{0.25}\ (y_1/a)^{0.5} \qquad 27$$

For maximum clear-water scour

$$y_s/a = 1.84\ (Fr_c)^{0.25}\ (y_1/a)^{0.3} \qquad 28$$

These equations are functions of the critical Froude number Fr_c corresponding to pending sediment transport, the procedure for computing Fr_c is as follows:

1. Estimate the median diameter, D_{50}, for the bed material;
2. Determine τ_c from Fig. 11;
3. Compute $U_{*c} = (\tau_c/\rho)^{0.5}$;
4. Compute $\delta = 11.6\nu/U_{*c}$ (assume $\nu = 1.08 \times 10^{-5}\ ft^2/s$);
5. Compute D_{50}/δ ;
6. Select Einstien's X from Fig. 28;
7. Compute $V_c = U_{*c}(2.5\ ln_s(11\ y\ X/D_{50}))$ and
8 Computer $Fr_c = V_c/(gy_1)^{0.5}$

It is also recommended that the scour depth for $0 < (Fr - Fr_c) < 0.2$ can be assumed equal to the larger of the two values of scour obtained from Eqs. 27 and 28.

For shapes differnt than circular piers and pier alignment other than parallel with the flow direction multiply the results given by Jain and Fishers equation by the coefficients given in Tables 2 and 3.

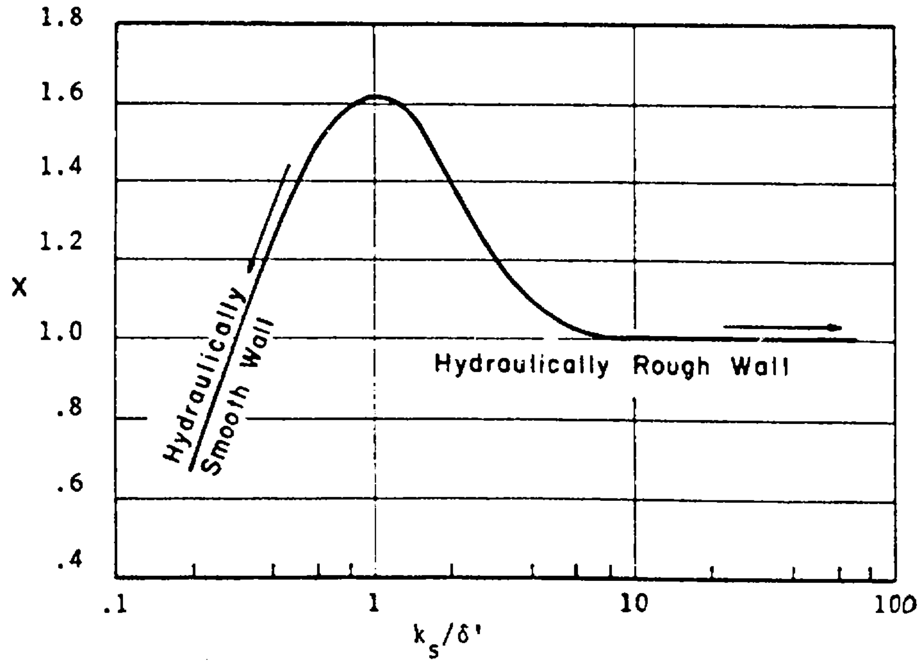


Fig. 28 Einstein's Factor X in the Logarithmic Velocity Equations (Einstein, 1950)

7.2.3 <u>Graded and/or Armored Stream Bed Equations.</u>

There are very little field data for determining the decrease in scour depth as the result of coarse particles in the bed material of a stream. However there are good indications (laboratory studies and some field data) that larger size particles in the bed material armor the scour hole and decrease scour depths.

Although field data are limited, equations are given here for thi case. Until additional field data is available, they should be used with care and use of good engineering judgement.

The equations for circular piers (adapted from equations develope by Washington State Deptarment of Transportation (Copp and Johnson, 1987, and Copp, Johnson, and McIntosh, 1988) for streams with a large range of partical sizes which would tend to armor the scour hole) are as follows:

University of Aukland (UAK) Equations

For $(a/D_{50} > 18)$

$$y_s/a = 2.1\ K_1\ K_2\ K_3 \tag{29}$$

For $(a/D_{50} < 18)$

$$y_s/a = 0.45\ K_1\ K_2\ K_3\ (a/D_{50})^{0.53} \tag{30}$$

Where

y_s = Depth of local scour
a = Pier width
K_1 = Coefficient for pier type, Table 2
K_2 = Coefficient for angle of attack of the flow, Table 3
K_3 = Coefficient for effect of sediment grading, Fig. 29
K_g = Gradation coefficient = $(D_{84}/D_{16})^{0.5}$

Copp and Johnson (1987) recommend that values obtained from the above equations be multiplyed by a factor of safety K_{fs} because there is little actual field data on scour depths in graded streambed material. They state the following:

> "A purely heuristic approach is to select Kfs equal to $1/K_3$ whenever K_g is less than about 2.0. If K3 is greater than 2.0, select K_{fs} = 1.5. This nullifies scour depth reductions for material gradations when K_3 < 2.0 but allows for the full depth of scour when K_3 > 2.0."

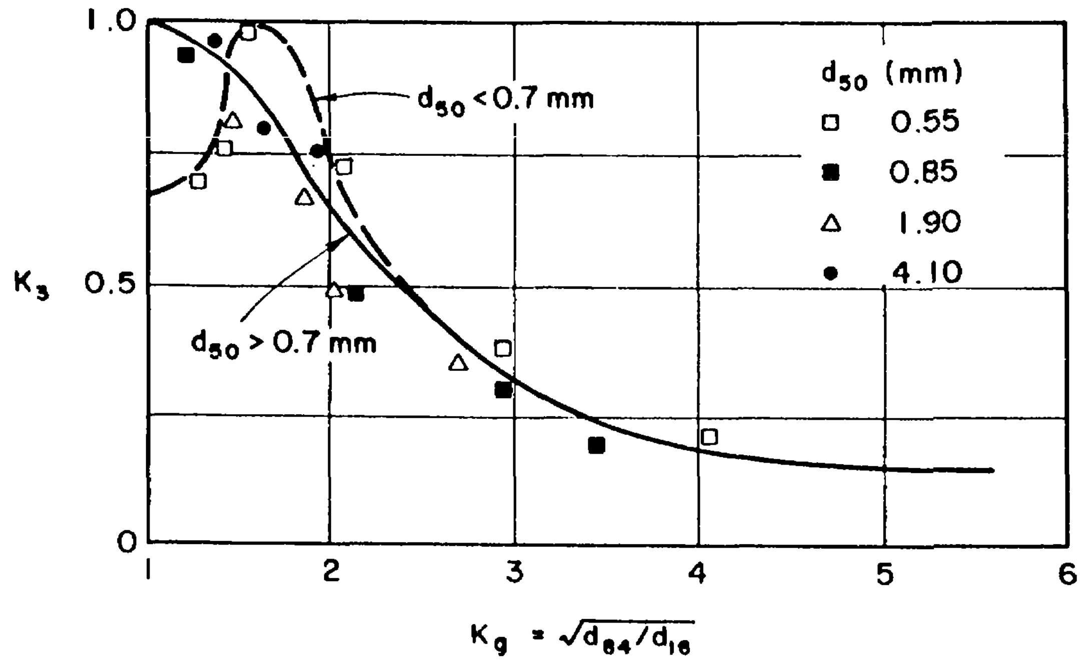


Fig. 29 Partical Size Coefficient, K_3, Vs. Geometric Deviation, K_g. (Ettema, 1980)

7.2.4 <u>Froehlich's Equation.</u>

Live-bed Scour.

Using linear regression analysis of 83 field measurements of pier scour Froehlich (1988) developed the following equation:

$$y_s/a = 0.32\ K_1\ (a'/a)^{0.62}\ (y_1/a)^{0.46}\ Fr^{0.20}\ (a/D_{50})^{.08} + 1.0 \qquad 31$$

Where

K_1 = Coeffiecent for pier type. Froehlich obtained K_1 = 1.3 fo a square-nosed pier, 1.0 for round and round-nose piers, and 0.7 for a sharp-nosed pier from his regression analysis. This is close agreement with values given in Table 2.

a' = Pier width projected normal to the approach flow.

a' = a' = a cos α + L sin α . Where α is the angle of attack and L is pier length.

The other symbols are as defined before.

The + 1.0 in the equation is to give a factor of safety for design purposes. The regression analysis gives the expected value of the scour depths. Fifty percent of the scour holes could be deeper and fifty percent shallower. All the measured values of scour were less than the expected value + 1.0 as given in Eq. 31.

Clear-water scour.

Froehlich (1988) only used live-bed scour data to develope Eq. 31. He classified the data as to either clear-water or live-bed scour data on the basis of Neill's (1968) equation given below:

$$V_c = 1.58\ ((S_s-1)gD_{50})^{0.5}\ (y_1/D_{50})^{0.167} \qquad 32$$

Where

S_s = 2.65 was assumed for all measurements.
V_c = critical mean velocity.

If V_c was larger than the mean velocity then the scour was clear-water scour.

However, all the clear-water scour depths in his data were less than those given by Eq. 31. So Eq. 31 could be used for clear-water scour.

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APPENDIX B

Selected Bridge Scour Data

From Froehlich (1988)

TABLE 1.--Onsite Measurements of Local Scour at Bridge Piers

Reference(s) and location of measurement	Measurement number	Date of measurement	Depth of scour in meters	Pier Data: Type code[a]	Pier Data: Width in meters	Pier Data: Length in meters	Approach Flow Data: Depth in meters	Approach Flow Data: Velocity in meters per second	Approach Flow Data: Angle in degrees	Bed Material Data: Median diameter in millimeters	Bed Material Data: Geometric standard deviation	Comments
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Bata (1960): Danube River Bridge at	1	--[b]/--/26	4.3	2	4.5	14.0	18.8	1.84	0	0.25	2.2	
Novi Sad, Yugoslavia	2	--/--/58	3.0	2	4.5	14.0	17.4	2.28	0	0.25	2.2	
Neill (1965): Beaver River La Corey Bridge, Alberta, Canada	3	06/19/62	1.74	2	1.92	17.37	5.39	1.8	5	0.5	--	center pier
Breusers (1970), and Breusers and others (1977): Niger River Onitsha Bridge	4	--/--/--	7.8	2	8.5	8.5	9.0	0.65	12	0.67	--	
Melville (1975): Waikato River Tuakau Bridge, New Zealand	5	08/15/58	2.75	1	2.40	8.85	3.45	0.96	10	0.78	2.3	
Norman (1975): Susitna River	6	07/02/71	0.76	3	1.52	6.10	5.8	1.98	0	70	--	pier 1
Anchorage-Fairbanks	7	"	0.76	3	1.52	6.10	4.1	2.59	0	70	--	pier 2
Hwy. bridge near	8	"	0.61	3	1.52	6.10	3.4	2.13	0	70	--	pier 3
Sunshine, Alaska	9	08/11/71	0.61	3	1.52	6.10	5.3	3.05	0	70	--	pier 1
	10	"	0.61	3	1.52	6.10	6.6	2.90	0	70	--	pier 2
	11	"	0.61	3	1.52	6.10	5.2	3.51	0	70	--	pier 3
Knik River Bridge near Palmer, Alaska	12	07/11/65	0.82	3	1.80	9.60	5.5	3.67	0	1.5	--	pier 5
Knik River Bridge	13	06/17/66	0.30	2	1.52	11.58	1.2	0.49	0	0.5	--	pier 3
near Eklutna, Alaska	14	"	0.30	2	1.52	11.58	1.5	0.76	0	0.5	--	pier 4
	15	"	0.30	2	1.52	11.58	1.2	0.88	0	0.5	--	pier 5
	16	"	0.76	2	1.52	11.58	0.5	0.27	0	0.5	--	pier 6
	17	"	1.22	2	1.52	11.58	0.6	0.15	0	0.5	--	pier 7
	18	06/24/66	0.61	2	1.52	11.58	2.1	1.52	0	1.6	--	pier 1
	19	"	0.61	2	1.52	11.58	2.0	1.55	0	1.6	--	pier 2
	20	"	0.91	2	1.52	11.58	3.0	1.58	0	1.6	--	pier 3
	21	"	1.22	2	1.52	11.58	3.2	1.98	0	1.6	--	pier 4, exposed foundation
	22	"	1.37	2	1.52	11.58	3.0	1.80	0	1.6	--	pier 5
	23	"	1.07	2	1.52	11.58	2.6	2.07	0	1.6	--	pier 6

TABLE 1.--Onsite Measurements of Local Scour at Bridge Piers--Continued

Reference(s) and location of measurement	Measurement number	Date of measurement	Depth of scour in meters	Pier Data: Type code[a]	Pier Data: Width in meters	Pier Data: Length in meters	Approach Flow Data: Depth in meters	Approach Flow Data: Velocity in meters per second	Approach Flow Data: Angle in degrees	Bed Material Data: Median diameter in millimeters	Bed Material Data: Geometric standard deviation	Comments
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	24	"	1.83	2	1.52	11.58	3.0	1.83	0	1.6	--	pier 7
	25	06/28/66	0.46	2	1.52	11.58	0.9	0.94	0	1.6	--	pier 1
	26	"	0.61	2	1.52	11.58	0.9	0.98	0	1.6	--	pier 2
	27	"	0.46	2	1.52	11.58	1.8	1.10	0	1.6	--	pier 3
	28	"	0.61	2	1.52	11.58	2.4	1.16	0	1.6	--	pier 4, exposed foundation
	29	"	0.76	2	1.52	11.58	2.3	1.13	0	1.6	--	pier 5
	30	"	0.46	2	1.52	11.58	1.5	1.13	0	1.6	--	pier 6
	31	"	0.76	2	1.52	11.58	2.0	0.98	0	1.6	--	pier 7
Tazlina River Richardson Hwy. bridge near Glennallen, Alaska	32	04/22/69	0.6	3	4.60	--	0.6	--	0	90	--	
	33	09/02/71	1.5	3	4.60	--	3.7	2.90	0	90	--	
	34	09/04/71	1.7	3	4.60	--	4.6	3.51	0	90	--	
	35	10/01/71	0.9	3	4.60	--	1.5	0.61	0	90	--	
Tanana River Richardson Hwy. bridge at Big Delta, Alaska	36	07/16/71	1.8	2	1.52	10.36	3.7	2.16	35	14	--	pier 1
	37	"	2.1	2	1.52	10.36	3.7	2.22	35	14	--	pier 2
	38	"	1.8	2	1.52	10.36	4.6	2.07	35	14	--	pier 3
	39	"	2.4	2	1.52	13.56	4.3	1.74	35	14	--	pier 4
Tanana River Anchorage-Fairbanks Hwy. bridge at Nenana, Alaska	40	07/30/67	1.8	2	3.05	17.60	6.7	2.59	0	15	--	
Snow River Seward Hwy. bridge near Seward, Alaska	41	09/22/70	0.9	2	0.98	0.98	1.7	1.61	0	8	--	
Chang (1980): Red River Hwy. 6 bridge near Grand Encore, La.	42	12/27/77	4.0	2	4.9	12.8	1.8	--	0	0.053	8.8	
	43	06/16/78	4.6	2	4.9	12.8	4.6	--	0	0.053	8.8	
Red River Hwy. 8 bridge near Boyce, La.	44	06/28/77	3.7	2	8.2	8.2	4.9	0.46	0	0.060	11.5	
	45	06/06/78	4.3	2	8.2	8.2	4.3	0.61	0	0.060	11.5	
Red River Hwy. 3026 bridge near Alexandria, La.	46	06/06/77	7.3	2	13.0	38.0	4.1	0.55	5	0.027	8.3	
	47	11/21/77	6.8	2	13.0	38.0	3.4	0.66	15	0.027	8.3	
	48	06/19/78	8.5	2	13.0	13.0	5.4	1.16	20	0.027	8.3	

TABLE 1.--Onsite Measurements of Local Scour at Bridge Piers--Continued

Reference(s) and location of measurement	Measurement number	Date of measurement	Depth of scour in meters	Pier Data: Type code[a]	Pier Data: Width in meters	Pier Data: Length in meters	Approach Flow Data: Depth in meters	Approach Flow Data: Velocity in meters per second	Approach Flow Data: Angle in degrees	Bed Material Data: Median diameter in millimeters	Bed Material Data: Geometric standard deviation	Comments
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Atchafalaya River	49	07/14/77	4.3	3	9.8	12.5	11.0	0.73	5	0.008	18.7	
Hwy. 190 bridge near	50	"	8.2	3	9.8	12.5	12.8	0.81	30	0.008	18.7	
Krotz Springs, La.	51	02/29/77	4.6	3	9.8	12.5	13.6	1.08	15	0.008	18.7	
	52	"	7.9	3	9.8	12.5	16.3	1.22	25	0.008	18.7	
	53	07/12/78	4.0	3	9.8	12.5	11.6	0.82	15	0.008	18.7	
	54	"	7.6	3	9.8	12.5	13.4	0.91	25	0.008	18.7	
Mississippi River Hwy. 65 bridge at Natchez, Miss.	55	09/07/77	6.1	1	9.4	19.5	19.5	1.80	0	0.036	6.1	
Mississippi River Hwy. 190 bridge at Baton Rouge, La.1	56	06/14/77	10.4	2	19.5	38.0	11.3	0.66	15	0.036	6.3	
Hopkins and others (1980): Red River Texas Street bridge at Shreveport La.	57	12/18/72	2.8	2	3.66	17.30	3.60	0.64	0	0.1	--	
Davoren (1985): Oahu River near Twizel,	58	09/20/82	1.3	2	1.5	1.5	3.1	2.38	0	20	5.3	
New Zealand	59	09/22/82	1.3	2	1.5	1.5	3.0	2.69	0	20	5.3	
	60	06/10/82	0.8	2	1.5	1.5	2.5	2.54	0	20	5.3	
	61	01/19/82	0.9	2	1.5	1.5	1.4	2.65	0	20	5.3	
	62	01/20/82	0.9	2	1.5	1.5	1.3	2.43	0	20	5.3	
	63	08/09/82	0.4	2	1.5	1.5	1.3	2.68	0	20	5.3	
	64	07/04/82	0.4	2	1.5	1.5	1.0	2.39	0	20	5.3	
	65	07/05/82	0.5	2	1.5	1.5	0.9	2.33	0	20	5.3	
	66	08/11/82	0.4	2	1.5	1.5	0.9	2.56	0	20	5.3	
	67	07/15/82	0.4	2	1.5	1.5	0.7	2.24	0	20	5.3	
	68	04/28/82	0.4	2	1.5	1.5	0.6	--	0	20	5.3	
Jarrett and Boyle (1986); and R. D. Jarrett, U.S. Geological Survey (written commun. 1986): South Platte River	69	10/02/84	0.61	1	0.29	3.66	0.76	1.04	15	1.5	--	pier 5
County Road 87 bridge	70	"	0.61	1	0.29	3.66	0.61	1.36	15	1.5	--	pier 6
at Masters, Colo.	71	"	0.52	1	0.29	3.66	0.73	1.17	15	1.5	--	pier 7
	72	06/25/84	0.58	1	0.29	3.66	0.43	1.13	10	2.3	--	pier 5
	73	"	0.46	1	0.29	3.66	0.58	1.02	10	2.3	--	pier 6

TABLE 1.--Onsite Measurements of Local Scour at Bridge Piers--Continued

Reference(s) and location of measurement	Measurement number	Date of measurement	Depth of scour in meters	Pier Data			Approach Flow Data			Bed Material Data		Comments
				Type code[a]	Width in meters	Length in meters	Depth in meters	Velocity in meters per second	Angle in degrees	Median diameter in millimeters	Geometric standard deviation	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	74	"	0.49	1	0.29	3.66	0.70	1.12	10	2.3	--	pier 7
	75	05/18/84	0.66	1	0.29	3.66	1.81	1.22	15	2.3	--	pier 5
Arkansas River County Road 613 bridge near Nepasta, Colo.	76	05/23/84	0.64	2	1.22	6.40	2.13	1.17	0	0.6	--	pier 1
	77	"	0.40	2	1.22	6.40	0.55	0.69	0	0.6	--	pier 3
	78	06/05/84	1.22	2	1.22	6.40	2.32	1.70	0	0.6	--	pier 1
	79	09/27/84	0.61	2	1.22	6.40	0.70	0.66	0	0.6	--	pier 1
Rio Grande Hwy. 285 bridge at Monte Vista, Colo.	80	05/22/84	0.37	3	0.94	27.43	1.40	1.54	0	7.9	--	
	81	06/01/84	0.15	3	0.94	27.43	1.22	1.35	0	4.3	--	
South Platte River Hwy. 37 bridge near Kersey, Colo.	82	05/21/84	0.98	3	0.52	8.29	3.21	1.68	10	1.2	--	pier 1
	83	06/26/84	0.65	3	0.52	8.29	2.14	1.17	10	1.8	--	pier 1

[a]Pier type code defined as: 1 = square-nosed; 2 = round-nosed; 3 = sharp-nosed.

[b]Information not available.

From Zhuravlyov (1978)

Register of local scour depth, measured in the nature and in large scale tests.

Table 1

# of Measurement	River	h_M, M	H, M	V, m/s	b, m	Shape of pier	d, MM	α°	$h\ell$,m, estimated BCH	$h\ell$,m, estimated Sojuzdoruii
1	Volga	5,0	15,42	1,61	4,90	кр	0,25		6,77	5,06
2	"	3,60	12,27	1,33	4,66	"	0,20		6,10	5,16
3	"	2,55	5,0	0,63	4,20	"	0,16		2,89	2,39
4	Agul	3,65	4,39	2,30	4,59	ов	35		4,08	3,06
5	Danube	5,03	18,80	1,84	4,70	"	0,40		7,34	6,59
6xx	"	3,51	17,40	2,28	4,70	"	0,40		–	–
7	Power Line	0,32	0,82	0,88	0,40	пр	1,9		0,44	0,42
8	Chirchik	1,31	2,0	2,10	2,0	ов	110	25	1,44	1,46
9	"	1,29	1,1	1,62	2,0	"	100	15	1,09	1,02
10xx	"	1,52	5,9	2,82	2,5	"	150		–	–
11	"	1,38	2,0	2,64	2,0	"	110	29	1,95	1,70
12	"	1,31	1,8	2,58	2,0	"	110	26	1,90	1,61
13	"	1,26	1,0	2,02	2,0	"	100	17	1,66	1,14
14x	"	0,93	2,4	2,44	2,2	"	110	29	–	–
15	"	1,24	1,9	2,30	2,0	"	110	24	1,76	1,51
16	"	1,32	0,8	2,06	2,0	"	100	15	1,58	1,06
17	"	2,57	4,7	2,93	2,5	"	140		2,19	2,62
18	"	2,12	1,7	2,50	2,0	"	120	26	1,89	1,54
19	Chirchik	1,55	1,6	2,49	2,0	ов	120	25	1,86	1,51
20	"	1,28	1,1	2,02	2,0	"	100	14	1,67	1,18
21	"	3,51	4,0	3,33	2,36	"	150		2,60	2,62
22	"	1,77	1,9	2,09	2,0	"	120	28	1,38	1,42
23	"	1,39	1,8	1,84	2,0	"	110	26	1,09	1,28
24	"	1,08	1,1	1,48	2,0	"	100	14	0,87	0,87
25	"	1,40	2,8	2,55	2,34	"	120		2,12	1,87
26	"	2,11	3,8	2,61	2,41	"	130		1,86	2,22
27x)	"	0,66	1,6	1,31	2,0	"	100	31	0,27	1,00
28	"	1,16	1,6	1,38	2,0	"	100	27	0,43	1,04
29	"	0,83	1,0	1,02	2,0	"	80	14	0,05	0,73
30	"	1,64	1,3	2,38	2,0	"	100		1,77	1,39
31	"	2,11	3,3	2,60	2,4	"	130		2,04	2,12
32	"	1,23	2,0	1,23	2,0	"	110	30	-0,59	1,02
33	"	1,33	1,6	1,18	2,0	"	110	28	-0,19	0,92
34	"	1,12	1,0	1,13	2,0	"	110	12	0,12	0,77
35	"	1,05	2,1	2,33	2,0	"	110		1,73	1,59
36	"	2,57	2,6	2,82	2,3	"	120		2,33	2,03
37	"	1,64	1,4	2,71	2,0	"	100		1,77	1,51
38	"	2,81	2,2	2,63	2,15	"	120		2,14	1,79
39	"	1,40	1,2	2,48	2,0	"	100		1,74	1,33
40	"	2,93	2,1	2,84	2,11	"	120		2,06	1,86
41xx	"	0,47	3,0	1,66	2,2	"	110		–	–
42	Labu	2,80	1,60	2,54	3,20	св	29	20	2,35	1,09
43x	"	1,09	1,60	2,12	3,20	"	29	20	2,32	1,66

# of a Measurement	River	h_M, M	H, M	V M/s	b m	Shape of Pier	d, MM	α°	hℓ,m, estimated BCH	Sojuzdoruii
44	Galizga	0,85	0,35	1,40	1,50	ов	34	20	0,94	0,59
45x	"	0,76	0,40	0,95	1,50	"	34	20	0,65	0,49
46xx	Moscow	0,48	1,10	2,00	1,50	"	30	40	–	–
47	Sozh	2,22	5,0	0,50	4,3	"	0,05	5	3,79	2,54
48	"	2,57	7,0	0,81	4,3	"	0,05	4	3,97	3,24
49	Dnieper	3,83	8,7	1,35	5,5	"	0,30	22	5,78	4,71
50	"	2,63	8,0	1,27	5,0	"	0,30	23	4,97	4,23
51	"	3,71	10,1	1,45	3,5	"	0,30	7	4,51	4,29
52	"	3,51	10,2	1,45	3,5	"	0,30	7	4,40	4,72
53xx	"	2,53	5,54	1,87	6,0	"	0,25	30	–	–
54	"	3,74	4,99	1,87	6,0	"	0,25	39	6,93	5,12
55xx	"	2,11	5,69	2,12	6,0	"	0,25	40	–	–
56	"	4,20	5,18	1,22	6,0	"	0,25	16	5,05	3,68
57	"	4,45	5,46	1,34	6,0	"	0,25	24	5,45	3,69
58	"	4,45	10,1	1,56	6,0	"	0,25	20	7,18	4,84
59	"	3,16	4,65	0,90	6,0	"	0,25		3,89	3,04
60	"	3,51	6,0	0,82	6,0	"	0,25	5	4,12	3,13
61	"	2,46	5,60	0,82	6,02	"	0,25	5	3,95	3,05
62	"	3,16	5,1	0,89	6,0	"	0,25	5	4,03	3,16
63	Dou	1,52	11,20	1,68	3,80	"	связ	7	–	–
64	Elek	2,34	11,45	2,40	5,0	ов		–	–	–
65	Deek	2,11	3,0	1,11	5,0			–	–	–
66x	Anu-Darya	1,8	2,7	1,42	3,05	кр	0,20	16	3,47	2,79
67	Anu-Darya	4,4	2,4	1,32	3,05	"	0,20	6	3,15	2,53
68	"	2,6	2,5	1,44	3,05	"	0,20	19	3,42	2,73
69	"	3,1	1,8	1,46	3,05	"	0,20	20	3,19	2,58
70	"	5,03	8,0	2,42	3,05	"	0,20	4	6,27	5,80
71	"	3,0	1,4	1,35	3,05	"	0,20	5	2,78	2,30
72	"	2,2	2,0	1,84	3,05	"	0,20	16	4,01	3,32
73	"	4,6	3,2	1,76	3,05	"	0,20	4	4,30	3,48
74	"	4,3	4,2	2,16	3,05	"	0,20		5,28	4,55
75	"	5,0	5,0	2,48	3,05	"	0,20	6	6,02	5,36
76	"	2,8	2,2	1,02	3,05	"	0,20	8	2,50	1,90
77	"	3,3	1,8	1,40	3,05	"	0,20	9	3,19	2,43
78	"	3,5	2,5	1,30	3,05	"	0,20	16	3,15	2,46
79	"	3,5	2,3	1,78	3,05	"	0,20	3	4,0	3,33
80	"	4,0	3,8	2,04	3,05	"	0,20	3	4,97	4,25
81	"	5,2	4,0	2,72	3,05	"	0,20	6	6,33	5,75
82	"	3,4	1,4	1,46	3,05	"	0,20	8	2,99	2,47
83	"	4,0	1,8	1,52	3,05	"	0,20	9	3,31	2,69
84	"	3,8	3,2	1,72	3,05	"	0,20	3	4,18	3,43
85	"	5,5	4,2	2,20	3,05	"	0,20	6	5,36	4,66
86	"	2,5	0,90	1,03	3,05	"	0,20	16	1,80	1,57
87	"	3,5	1,9	2,15	3,05	"	0,20	3	4,57	3,92
88	"	3,7	3,1	1,85	3,05	"	0,20		4,41	3,71

# of a Measurement	River	h_M, M	H, M	V m/s	b m	Shape of Pier	d, MM	α°	hℓ,m, estimated BCH	Sojuzdoruii
89	Anu Darya	4,6	4,3	2,04	3,05	кр	0,20	6	5,07	4,37
90	"	3,7	6,6	2,10	3,05	"	0,20	3	5,50	4,88
91	"	2,3	1,2	1,10	3,05	"	0,20	16	2,15	1,84
92	"	2,6	1,3	1,28	3,05	"	0,20	3	2,55	2,08
93	"	4,0	2,8	1,80	3,05	"	0,20		4,22	3,51
94	"	5,0	3,2	2,20	3,05	"	0,20	6	5,11	4,36
95	"	3,0	3,7	1,30	3,05	"	0,20	6	3,50	2,67
96x	"	3,3	10,8	2,09	3,05	"	0,20	3	–	–
97	Ishru	5,50	12,2	2,9	3,1	ов	связ	8	–	–
98	"	3,86	12,7	2,9	3,1	"	связ	2	–	–
99x	Skoouk	2,49	3,45	0,53	3,3	"	0,58		1,53	1,35
100	"	1,93	2,61	0,53	3,3	"	0,58		1,42	1,21
101	"	1,93	3,40	0,53	3,3	"	0,58		1,52	1,34
102	Seym	4,0	5,70	1,90	5,28	"	0,35	20	5,78	4,59
103	Ravy at Lakha	6,19	3,56	2,06	3,0	пр	0,20		4,71	4,21
104	Pakustau	4,73	2,95	2,27	3,0	"	0,20		4,97	4,46
105	"	6,18	4,77	2,76	3,0	"	0,20		6,28	6,00
106	"	4,97	3,04	2,12	3,0	"	0,20		4,69	4,25
107	"	5,2	3,31	1,93	3,0	"	0,20		4,41	3,98
108	Ravy	6,08	3,88	1,98	3,0	пр	0,20		4,65	4,18
109	"	5,25	4,71	2,50	3,0	"	0,20		5,76	5,60
110	"	4,01	5,68	1,71	3,0	"	0,20		4,44	3,87
111	"	4,7	3,64	2,04	3,0	"	0,20		4,72	4,25
112	"	5,35	4,86	2,70	3,0	"	0,20		6,20	5,96
113	"	3,36	4,53	2,47	3,0	"	0,20		5,70	5,44
114	Oka	0,53	0,66	0,54	0,50	кр	0,34		0,44	0,40
115	"	0,23	0,78	0,59	0,20	"	0,32		0,25	0,31
116	"	0,61	0,79	0,61	0,50	"	0,32		0,42	0,47
117	"	0,45	0,98	0,54	0,50	"	0,36		0,48	0,44
118	"	0,33	1,08	0,66	0,20	"	0,38		0,27	0,38
119	"	0,31	1,15	0,66	0,20	"	0,41		0,26	0,38
120	"	0,29	1,55	0,57	0,20	"	0,30		0,28	0,39
121	Protva	0,46	1,40	0,62	0,40	"	0,40		0,46	0,50
122	"	0,54	1,05	0,62	0,625	"	0,44		0,58	0,53
123	"	0,52	0,78	0,46	1,0	"	0,35		0,62	0,59
124	"	0,48	1,20	0,60	0,40	"	0,50		0,44	0,44
125	"	0,57	1,02	0,57	0,625	"	0,82		0,56	0,53
126	"	0,40	0,53	0,43	0,40	"	0,38		0,33	0,29
127	"	0,31	0,40	0,39	0,245	"	0,36		0,21	0,21
128	"	0,41	0,53	0,47	0,40	"	0,40		0,33	0,31
129	"	0,51	0,40	0,43	0,625	"	0,32		0,36	0,33
130	"	0,39	0,30	0,35	1,00	"	0,34		0,11	0,30
131x	"	0,52	0,30	0,44	0,625	"	0,40		0,31	0,30

Table 1 Cont'd

# of a Measurement	River	h_M, M	H, M	V M/s	b m	Shape of Pier	d, MM	α°	hℓ,m, estimated BCH	Sojuzdoruii
132x	Protva	0,61	0,33	0,44	0,625	кр	0,40		0,33	0,28
133	"	0,39	0,30	0,40	1,00	"	0,49		0,11	0,32
134	Ravy	9,23	8,0	2,7	3,0	пр	0,20		6,58	6,57
135	Dzhelum	3,97	7,0	1,6	3,0	"	0,20		4,37	3,82
136	Dey	5,83	6,5	2,8	1,8	"	0,20		4,50	6,60
137	Rocky	6,16	4,3	3,2	1,9	"	0,20		4,81	5,71
138	Socham	5,75	5,7	2,8	2,7	"	0,20		5,94	6,10
139x	Indus	10,61	7,6	3,3	2,1	"	0,20		5,66	6,84
140	Oka	0,17	0,85	0,48	0,20	кр	0,42		0,25	0,27
141x	"	0,21	1,03	0,62	0,20	"	0,32		0,25	0,37
142	"	0,29	0,73	0,54	0,20	"	0,32		0,24	0,29
143	"	0,41	1,18	0,70	0,32	"	0,41		0,78	0,46
144	"	0,29	1,05	0,62	0,20	"	0,37		0,26	0,35
145	"	0,26	1,09	0,48	0,20	ов	0.30		0,24	0,32
146	"	0,62	1,07	0,61	0,50	"	0.38		0,51	0,50
147	"	0,49	0,80	0,66	0,20	пр	"		0,25	0,35
148	"	0,47	0,84	0,62	0,20	ов	0.35		0,26	0,33
149	"	0,41	0,90	0,56	0,20	пр	"		0,25	0,32
150	"	0,73	0,88	0,58	0,50	ов	"		0,48	0,46
151	"	0,59	0,79	0,55	0,50	пр	"		0,46	0,43
152	"	0,35	0,75	0,55	0,32	ов	"		0,34	0,34
153	Oka	0,64	1,20	0,64	0,50	ов	0.36		0,54	0,54
154	"	0,28	1,10	0,64	0,20	пр	0.41		0,27	0,35
155	"	0,34	1,12	0,67	0,20	ов	0.31		0,27	0,39
156	"	0,45	1,10	0,62	0,32	"	0.42		0,37	0,42
157	"	0,32	1,07	0,64	0,20	пр	0.31		0,27	0,38
158	"	0,52	0,75	0,51	0,50	"	0.41		0,45	0,39
159	"	0,63	0,95	0,54	0,50	ов	0.31		0,48	0,47
160	"	0,43	1,15	0,56	0,20	пр	0.32		0,27	0,35
161xx	Seleuga	1,17	4,1	1,65	3,4	ов	1.6	20	–	–
162	"	1,40	4,3	1,33	3,4	"	"	20	1,88	1,96
163	Ob	4,69	11,4	0,84	12,0x	пр	0,30	25	6,96	5,07
164	"	3,51	7,6	0,64	13,0x	"	0,30	25	4,36	3,87
165	"	2,04	6,1	0,62	2,4	кр	0,30		1,93	1,81
166	"	2,3	5,8	0,60	10,2	"	0,30		3,44	2,98
167	Djohn	6,0	11,7	3,0	3,0	"	связ		–	–
168x	Ob	1,0	5,6	0,59	2,98	шп	0,30	45	–	–
169	"	1,6	5,6	0,59	3,97	"	0,30	45	2,43	2,06
170	Tobol	4,15	6,0	2,63	4,18	ов	1,0		4,61	4,50
171	"	3,98	5,6	0,95	4,21	"	1,0		3,40	2,35
172	Volga main channel	4,50	14,5	0,69	4,9	кр	0,25		3,76	3,48
173	"	3,0	11,9	0,53	4,66	"	0,20		2,74	2,79
174	"	1,75	4,9	0,43	4,2	"	0,16		1,91	1,82
175x	"	1,20	6,96	0,82	4,50	"	0,60		3,25	2,55
176	"	5,72	17,10	1,59	4,83	"	0,25		6,74	6,35
177	"	4,80	16,55	1,57	5,00	"	1,53		5,66	4.03

Table 1 Cont'd

# of a Measurement	River	h_M, M	H, M	V M/S	b m	Shape of Pier	d, MM	α°	hℓ,m, estimated BCH	Sojuzdoruii
178		4,70	15,40	1,47	5,03	кр	0,35		6,18	5,60
179		4,20	14,70	1,43	5,00	"	1,53		5,50	3,57
180		3,00	13,90	1,36	4,55	"	0,35		5,42	4,88
181 xx		1,87	12,79	1,29	4,45	"	0,34		5,12	4,65
182 xx		1,62	11,19	1,16	4,40	"	0,34		4,75	4,07
183		3,00	10,59	1,12	4,30	"	0,88		4,30	3,55
184 x		1,72	9,89	1,07	4,26	"	0,27		4,41	3,79
185		1,74	6,29	0,76	3,86	"	0,40	13	2,84	2,34
186		2,41	9,34	0,93	4,10	"	0,60	20	3,72	2,93
187		1,71	8,89	0,90	4,16	"	1,84	26	2,56	2,07
188		4,36	9,99	0,95	4,30	"	0,20	19	4,05	3,78
189		4,08	11,19	1,07	4,33	"	0,16	23	4,55	4,40
190		3,65	11,39	1,09	4,40	"	0,44	18	4,50	3,82
191		4,16	11,53	1,10	4,45	"	0,31	13	4,80	4,10
192		2,55	12,18	1,14	4,50	"	0,42	15	4,73	4,00
193 xx		1,74	13,08	1,20	4,50	кр	0,34	16	5,27	4,45
194 xx		1,80	13,73	1,26	4,50	"	0,49	21	5.30	4,34

Notes: 1) All natural scour depth are given brought to round piers and to normal stream attach.

2) The shape of a pier is by: kp - round, OB - oval, rp - rightanular CB - pile foundation,

APPENDIX C

SI Conversion Factors

SI Conversion Factors

To Convert	To	Multiply by
1. Length (L)		
inches	millimeters	25.4
feet	meters	0.305
yards	meters	0.914
miles	kilometers	1.61
2. area (L^2)		
square inches	millimeters squared	645.2
square feet	meters squared	0.093
square yards	meters squared	0.836
acres	hectares	0.405
square miles	kilometers squared	2.59
3. volume (L^3)		
fluid ounces	milliliters	29.57
gallons	liters	3.785
cubic feet	meters cubed	0.028
cubic yards	meters cubed	0.765
4. mass (M)		
ounces	grams	28.35
pounds	kilograms	0.454
short tons (2,000 lb)	megagrams	0.907
5. temperature		
Fahrenheit (F)	Celsius (C)	5(F-32)/9

ARIZONA DEPARTMENT OF TRANSPORTATION
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January 27, 1992

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REPLACEMENT ITEMS FOR:

REPORT NUMBER: FHWA-AZ90-814

REPORT DATE: MAY, 1990

REPORT TITLE: SCOUR AT BRIDGE STRUCTURES AND CHANNEL AGGRADATION AND DEGRADATION FIELD MEASUREMENTS

ADD

Page 101, Appendix A, Reference to Straub (1940):

Straub, L. G. 1940 Approaches to the Study of the Mechanics of Bed Movement, Proceedings of 1st Hydraulics Conference, Engineering Bulletin Number 20, University of Iowa Studies in Engineering, pp.178-192, March, 1940.

REPLACE

Page 102, Equation 15, :
The exponent of y_1 on the right hand side of Eq. (15) should be 1/3 not 1/2.
The correct equation is given below:

$$y_2/y_1 = (W_1/W_2)^{6/7}\,[V_1^{2}/(120 y_1^{1/3}\,D_{50}^{2/3})]^{3/7}$$

SHEET 1 OF 1
END