Session - 2

Bed Erosion
and Scour

Design Flood - An Overview of Indian Practice

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SUMMARY

A Committee of Engineers, appointed by Government of India in 1957, identified rational estimation of Design Flood as critically important for bridge foundation design. The report of the Committee initiated a landmark national Study. Regional synthetic unit hydrograph and design input storm parameters have been defined on the basis of this study, for the entire country divided into 26 hydrometeorologically homogeneous regions. The method applies to catchments of about 5000 km^2 area. For larger projects, flood frequency analysis is generally adopted when adequate gauging data is available. Flood with a return period of 100 years is now unambiguously defined as Design Flood. The evolution of current design practice is discussed with particular reference to road bridges.

1 INTRODUCTION

Indian River System is large and so also is the number of crossings by roads and railways. Rivers unlimited carry an annual flow of about 1700 billion m^3 [1]. They all swell in summer with high monsoon precipitation in their catchments. In Himalayan rivers, snowmelt may add to it. In the alluvial plains, many overflow the banks and flood the land around, often ravaging it.

Flood is the most familiar and frequent natural disaster in India. It afflicts one river basin or the other almost every year, with varying fury. Protection of vital communication links of roads and railways during floods aids disaster mitigation in a big way. This lends added socio-economic significance to designing bridge foundations safe against the severest floods.

Flood is an extreme natural event with many faces. The principal characteristics, usually represented in flood hydrograph, are i) Peak Discharge ii)Water Level iii)Volume iv) Flood Duration. Peak Discharge alone may be the prime concern in bridge hydrology. It will be deemed here as synonymous with flood. The current design practice of estimating design flood is reviewed here with particular reference to road bridges.

2 CODE SPECIFICATION AND PAST PRACTICE

The first Section of the national Bridge Code, IRC-5:1985 (referred as Code), lays down specifications relevant to design flood, currently valid for all road bridges[2]. However, codes tend to be static. Design practice, often, progresses much beyond the bounds of codes. For Indian bridges also, current code specification only defines past practice. Demerits of Code specifications, which eventually made them invalid for current use, are discussed below.

2.1 Historic Flood

The design flood is simply defined in Code as the maximum observed flood or historic flood, for a mandatory minimum period of record of 50 years[3]. This earliest method of selecting design flood has grown rather dated. Its demerits are well known. The probable frequency of the selected flood remained unknown. The design flood at the same site could increase as period of record increased. Insufficiency of flood records, more as a rule than exception, was, of course, its biggest flaw. Accepting it in its stride, Code offers a long list of alternative methods, obviously intended to find an equivalent. Two of them discussed below are of prime interest. These methods used in combination essentially defined past design practice, in conformity with Code.

2.2 Area Velocity Method[2]

This is really an extension of the method of Historic Flood. Instead of records for historic flood, the maximum water level reached in historic flood is sought to be estimated on the evidence of local witnesses. These may include flood marks on banks and structures close to project site or even fading memories of how high the highest flood rose on the ancient tree or building. The variability of bed profile and flood slope from those measured before/after flood is ignored. Computation of stream velocity relies on subjective selection of an empirical coefficient. The return period of the design flood is left to uncertainty.

2.3 Empirical Formula

A large family of empirical formulae for quick and ready estimation of design flood was developed in India. These have spilled from the past century into the present. The first one that made its debut in 1885 is Dickens formula[3]. It also happened to be the one most frequently used in bridge design, until recently. It read as-$Q = C * A^{0.75}$ where Q is design flood (ft^3/s), A catchment area ($mile^2$) and C a constant.

The formula was surely developed for small catchments, with limited data available and for a small region. These obvious limitations have been largely ignored. Its validity has been extended from regional into near national without many qualms. Although meant for small catchments, it has been used for catchments exceeding a few hundred thousand km^2

Invalid extrapolation used the simple expedient of varying constant C as wide as 200 to 2000. The critical choice of C was left to the subjective judgment of designer, who had little clues to go by except personal preference. The uncertainty around the frequency of the design flood resulting from the formula remained as the common malady.

2.4 Multiple Methods in Combination

Code preferred to rely on multiple methods to improve reliability, which could be elusive. It all boiled down to computing values twice over (or more), once by area velocity method and then again by Dickens formula (and/or equivalent). The values were compared and the largest only qualified for selection as the design value. The inherent fallacy should be obvious. If both methods were unreliable, comparison and combination may, in all probability, compound the errors. Overestimation by Dickens formula could easily negate the efforts of a more rational hydrologic analysis. The same fallacy recurred when design discharge adopted for bridges in vicinity was called in for comparison. If the reference values themselves were estimated by unreliable methods, any comparison could have little relevance to a rational estimation.

3 PRESENT PRACTICE AND THE CHANGE IN APPROACH

'Period of Empiricism' no longer rules the scene[4]. As disenchantment with it grew, search for rational methods for practical use began as far back as the fifties. Dickens formula is now invalid even for minor bridges. A simple rational substitute method of regional analysis was put to practice in 1973[5]. Further development through two decades has followed and brought in its wake a complete change in approach.

3.1 Report of Committee of Engineers and Follow-up

The report of a high powered Committee of Engineers on bridges appointed by Government of India (referred as Committee) was published in 1959[6]. It identified design flood and its rational evaluation as critically important. Following its recommendations, 'sustained and systematic collection of hydro-meterological data' was undertaken for the entire country on a short and long term plan[6].

The short term plan was completed in 1973. A regional Synthetic Unitgraph (SUG) method was evolved for estimation of design floods of bridges with catchments of 25 to 5000 km^2[5]. The long term plan has continued since with joint efforts of hydrologists, meteorologists and bridge engineers of roads and railways. The national cooperative study (referred as Study) was a landmark event in flood hydrology of Indian bridges. A brief description of the Study and the method of estimation of design flood evolved follows.

3.2 Basic Approach for the Study

The approach has to be tailored to availability of data and project size. Large investment intensive bridge projects should, of course, go in for detailed hydrologic analysis supported by project specific hydro-meterological investigation, if needed. For many large projects, gauging stations with adequate period of record may be available at

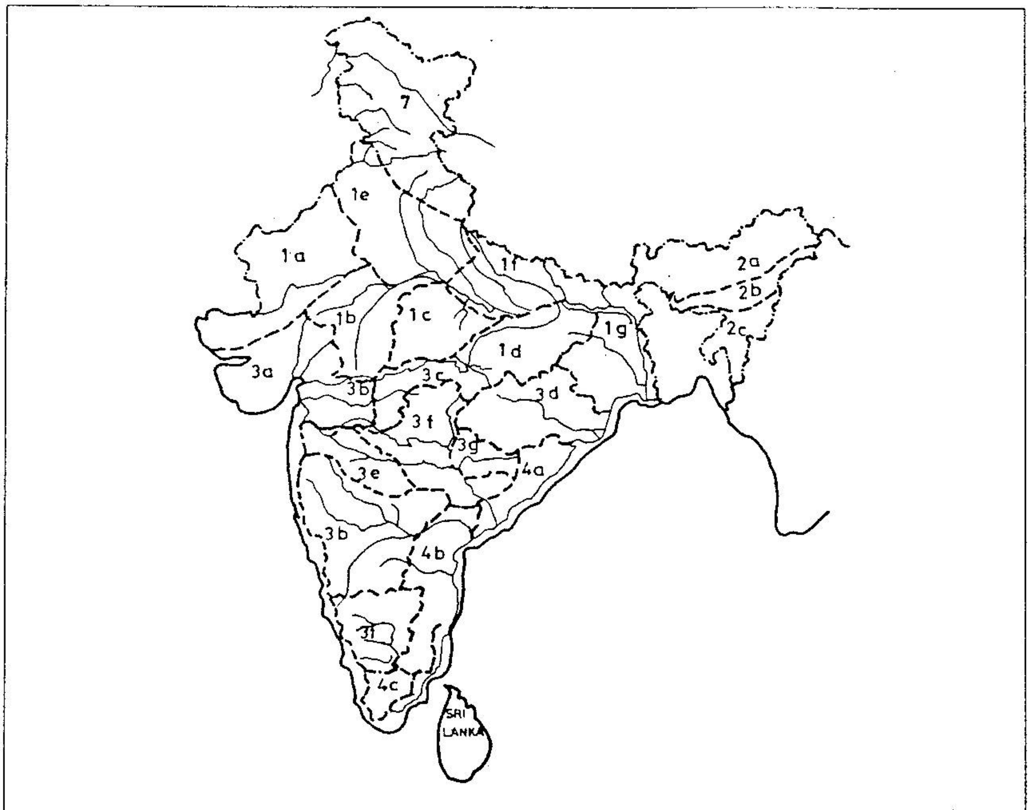


Fig. 1 Hydrometerologically Homogeneous Regions – 26 Subzones and Major Rivers

site or in vicinity. Flood frequency analysis could be feasible and preferred. An example has been discussed later.

The thrust area identified for the Study, therefore, related to bridges with catchments upto 5000 km ^{2}. These claimed the lion's share of total national investment in bridges. Most of their catchments were ungauged. Project specific investigation was not feasible and Regional Analysis was the obvious option open.

Two candidate approaches considered for regional flood estimation were i) flood frequency ii) hydro-meterological. The latter was adopted for better availability of data and in conformity with the recommendations of the Committee.

Regional flood estimation studies were taken up for hydro-meterologically homogeneous regions. For this purpose, the country was divided into 26 such regions (called subzones; principal zones number 7) as shown in Fig 1. The salient features of these subzones vary widely in drainage basin area, topography, rainfall, land use, etc. Results were reported separate for each subzone.

The Study has been jointly undertaken by four apex bodies of Government of India - Central Water Commission (CWC), Research and Standards Organisation (RDSO) of Ministry of Railways, India Meteorological Department (IMD) of Ministry of Science and Technology and Ministry of Surface Transport (MOST).

3.3 Study Methodology [7]

3.3.1 Flood Flow Data (RDSO/MOST)

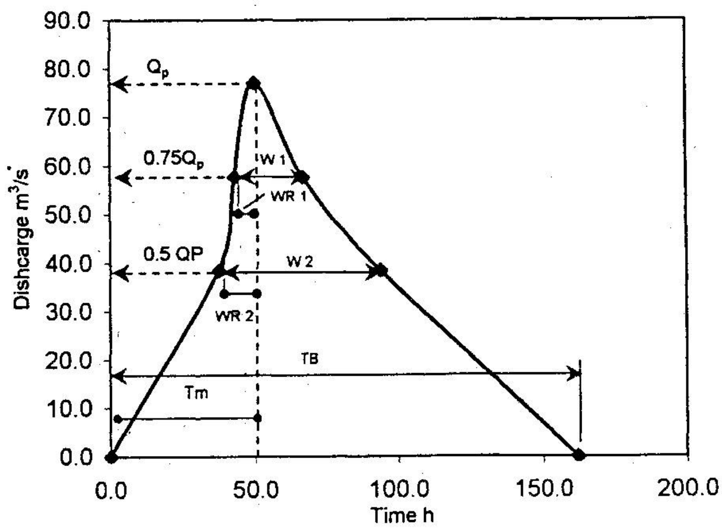
Rainfall and flood flow data were collected at selected representative railway bridge catchments (RDSO) numbering about 10 to 30 for each subzone. Period of observations in phases varied from 5 to 10 years beginning from 1965. These were supplemented by observations at total number of 45 road bridge sites (MOST), beginning 1979.

3.3.2 Storm Analysis (IMD)

Long term rainfall data for object subzone for a large number of raingauge stations, both ordinary and self recording, were collected by IMD from its National Data Centre. These were combined with rainfall data mentioned in 3.3.1. IMD made rainfall depth-duration-frequency analysis of data for each subzone and furnished the following components of design storm- i) Isopluvial maps of 24 h point rainfall of 25,50 and 100 year return period (T) ii) Ratios of Short Duration to 24 h Rainfall iii) Time Distribution Curves of Storms of various duration iv) Ratios of Areal to Point Rainfall. Hourly design storm rainfall increments could be redily estimated with the aid of maps, tables and charts given by IMD.

3.3.3 Hydrologic Analysis (CWC)

CWC collated concurrent rainfall and flood data furnished for gauged catchments in a subzone. After due scrutiny and finalisation of gauge and discharge rating, several storm/flood events were selected for study. One hour unit hydrographs (UG) were derived by usual methods. A few characteristics of UG curve were identified and measured for the several UG curves in view. These were correlated to physiographic characteristics of catchment by regression analysis. A simple relation of the form $y = k*x^n$ where k, n are constants, did suffice to define the SUG for ungauged catchments in a subzone. Fig 2 shows a typical SUG developed including the constants defining it.



River	Karmanasha	
Subzone	Sone 1(d)	
Catchment Area	$A = 2000\ km^2$	
Length of Stream	$L = 180\ km$	
Equivalent Slope	$S = 1.5\ m/km$	
t_p	$0.314(L/S^{0.5})^{1.012}$	$= 50$
q_p	$1.664/(t_p)^{0.965}$	$= 0.0385$
W_2	$2.534/(q_p)^{0.976}$	$= 57$
W_1	$1.478/(q_p)^{0.86}$	$= 24$
W_{2R}	$1.091/(q_p)^{0.75}$	$= 13$
W_{1R}	$0.672/(q_p)^{0.719}$	$= 7$
TB	$5.526(tp)^{0.866}$	$= 162$
Tm	$t_p+t_r/2$	$= 50$
Q_p	$q_p \times A$	$= 77$
Design Storm Duration	$1.1*t_p$	$=54$
Design LossRate		$=0.25$ cm/h
Base Flow		$=0.045*A$

Fig. 2 Synthetic Unit Hydrograph – An example

Average constant infiltration loss(ϕ-index) rate per hour, base flow per km^2 to be used in design estimation and design storm duration were also derived by CWC on a general basis, from analysis of a number of flood events.

3.4 Study Report and Method of Flood Estimation(CWC) [7]

Results of the study for each subzone have been reviewed by a co-ordination committee and published successively by CWC separate for each subzone. The reports lay down a method of estimation of design flood with return period of T = 25,50,100 years by SUG. It is based on the basic assumption that design storm of T years return period causes a flood of T years return period. No significant interception is presumed.

Deign flood with desired return period is computed in three simple steps- i) draw the SUG curve; tabulate its hourly ordinates ii)estimate the hourly rainfall increments (deduct losses) iii) Compute direct runoff; add base flow. Method of estimation of flood of T = 25, 50, 100 years is lucidly set out with tables, charts, maps and worked out examples to aid easy and unfettered use. Hydrographs can also be prepared.

The utility of these reports extends much beyond its prescription of a rational method of flood estimation for minor bridges. Each self complete subzone report contains detailed documentation of data collected, methods of analysis and results along with some general topographic, climatic, meteorological data. 21 separate reports, covering all but 2 subzones and 91% of the country, have been published by CWC to date[8]. A large national hydrometeorologic database has been compiled and deserves to be extended in future.

3.5 Design Flood Defined- Anomalies Abandoned

3.5.1 Anomalies in Definition

Estimation of Design Flood can only as good as Design Flood is defined. Anomalies in definition may undo all the rationale of evaluation. Some did creep into Committee recommendations quoted below [6].

"Committee felt that design discharge should be maximum flood on record for a period of not less than 50 years. Where adequate records are available extending over not much less than 50 years, design flood should be 50 year flood determined from probability curve on the basis of recorded floods during the period."

Committee thus defined Design Flood as Maximum Observed Flood (definition 1)and NOT as Flood with Return Period T = 50 Year or any other T fixed a priori (definition 2). Code definition is identical and the lack of logic has been discussed in 2.1[2].

Option of Committee for definition 1 is unambiguous. Definitions do not alter as a function of period of flood record. It would be highly anomalous to presume that definition 1 could be substituted by definition 2 if period of record just fell short of the threshold value of 50 years. Nor could flood frequency analysis be invalid for 50 years' record. When this context is ignored, anomalies arise. These are best illustrated by Table 1.

Table 1 Maximum Observed Floods and Return Periods
Case Study for Yamuna at Tajewala- Annual Flood Peak Series 1913-78.

Maxm Observed Historic Floods			Floods of T year Return Period (Probability Curve)	
Year	Flood m^3/s	T Year	T Year	Flood m^3/s
1924 Sept	25110	105	50	20320
1947 Sept	18390	35	100	25020
1955 Oct	13234	13	200	30240
1978 Sept	26410	130	1000	47550

Taking the annual peak series>50years (1913-78)into account, a flood with T as high as 130 years should be selected as design flood. Given a hypothetically truncated series of 50 years- (1925-75), a flood with T as low as 35 years would be selected as design flood as defined by Committee. The real dilemma, more commonplace, should arise when a hypothetically truncated series over 1938-78 (40 years) is considered. Maximum observed flood with T=130 years occurs in this series and selecting anything lower as design flood would amount to a gross violation of the basic definition. At best, the intent of Committee could be interpreted (for insufficient flood record) to find a probable flood in 50 years with probability of exceedance left anomalously undefined.

3.5.2 Foundation Design Flood [6], [2]

Committee also recommended two kinds of design floods for bridges-1) foundation design flood 2) deck design flood. The former claiming a higher safety level is obtained by incrementing design flood by a Factor of Safety varying (from 1.1 to 1.3) inversely as the catchment area. The latter with a lower safety demand is assigned factor of safety of 1. The length of deck or waterway should be determined by design flood. A higher value only applies to foundations with the higher safety levels.

Safety factors may not have much relevance in bridge hydrology. Desire for higher safety and higher safety factors can increase non-linearly following a flood event. There are instances of Factor of safety for foundation design flood rising as high as 1.5 for large projects.

Distinction between safety levels for waterway(deck) and foundation is quite impracticable. Scour around foundation is a function of Q/W, where Q is the design discharge, W length of waterway. So a lower waterway enhances the risk of foundation failure.

3.5.3 Rational Definition

Rational methods of estimation of design flood for road bridges in current practice could not but abandon the anomalies discussed above. No distinction is made between foundation design flood and other design flood. Nor is 50 year return period flood incremented by a factor of safety (1.3 or more) considered relevant.

Rational definition of design flood in terms of T year return period fixed a priori is only adopted in present practice. It applies uniformly to all bridges minor, medium and major alike. The probability of exceedance of a design flood with a given return period during design service life is shown in Fig. 3[10]. The design service life of road bridges in India can be notionally defined as 50 years. Fixing a return period of 50 years for design flood would yield a probability of exceedance of 65% which may be deemed too high. Higher return periods of 100 and 1000 year will reduce the risk to 39% and 5% respectively.

The optimal choice of a return period of 100 years defines present practice. The Study report includes estimation of floods with T = 100 years. The same return period applies to inflow design flood of small dams according to IS Guidelines[11]. It is interesting to recall that the very first version of Bridge Code draft dating back to 1946 opted for a 100 year design flood [12].

So the present definition of design flood as one with T=100 year abandons all anomalies of past practice. Choice of

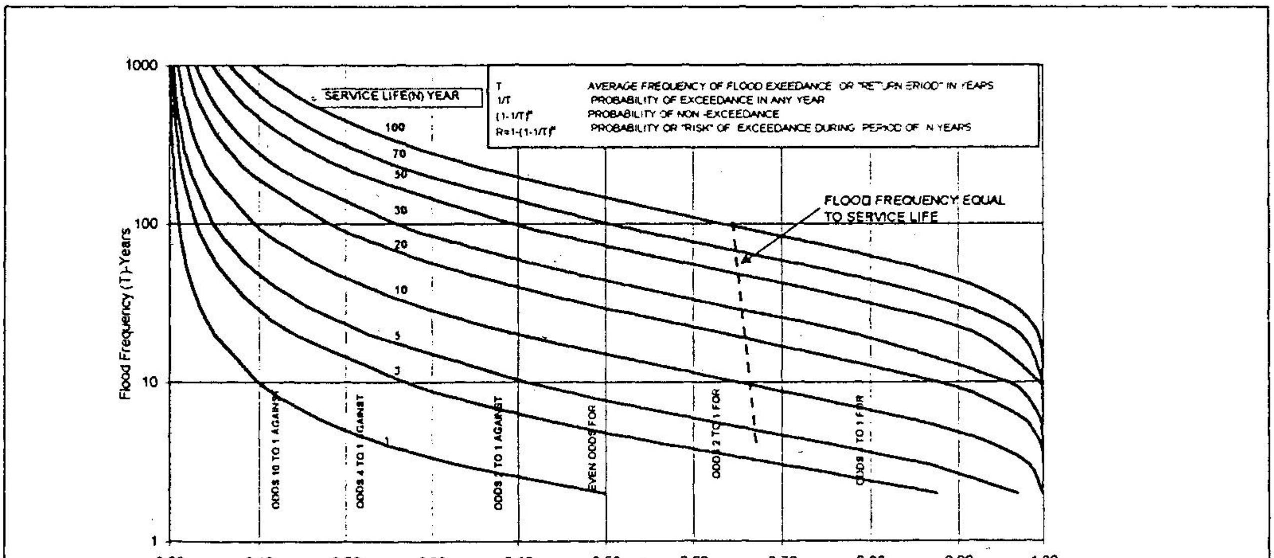


Fig. 3 Flood Frequency and Risk of Exceedance

Design Flood is a risk based decision aimed at socio-economic optimisation utility of a structure. India can hardly afford the luxury of designing bridges for 1000 year flood.

3.6 Regional Flood Frequency Analysis

Regional Flood Frequency Analysis has not been developed in the Study reports. Inadequacy of flood flow data is the obvious reason. However, the Regional Flood Frequency Model was developed with limited data for one Subzone(Sone)[13]. Data of 11 catchments spread over the subzone with areas varying from 30 to 500 km ^{2} were used. Annual flood peak series of 11 to 25 years was available. Gumbel EV-1 distribution was used. Values of floods (Q_T) for various return periods T = 2.33(mean annual flood Q_m),25(Q_{25}), 50(Q_{50}) ,100(Q_{100}) were obtained by fitting a straight line through plotted positions. The following ratios of Q_T/Q_m have been derived - Q_{25}/Q_m=2.83, Q_{50}/Q_m=3.38, Q_{100}/Q_m=3.82. The regional formula for mean annual flood is related to physiograpic characteristics of catchment derived by least squares method read as, $Q_m=2.33*A^{0.795}*S^{0.567}*F^{0.520}$, where Q_m is in m^3/s ,A catchment area in km^2, S equivalent storm slope in m/km, F form factorA/L^2 , L being basin length.

3.6.1 Mean Annual Flood

Results yielded by the method were acceptable and proved the potentials of future use. Incidentally, estimation of Q_m is key step in flood frequency analysis and it is equally so in foundation design. Accidental load combination like earthquake or barge impact often determines foundation design [14]. Code specifications lay down that Mean Annual Flood. and NOT Design Flood should be combined with earthquake. As such estimation of the former gets equal importance in current rational practice. Arbitrary coefficients promoted by cursory code prescriptions are worth ignoring[14]. Flood level corresponding to Q_m is also to be evaluated, as depth of scour is to be measured from it only.

3.7 Flood Frequency Analysis for Large Catchments

Large bridges need project specific investigation and analysis for rational flood estimation. Application of the regional analysis is limited to about 5000 km^2. Choice of methods is left to designer. This, of course, precludes any return to empiricism.

Single unit hydrograph cannot be applied to large catchments. The total drainage area has to be divided into a number of subbasins .Separate flood hydrographs may be derived for each sub-basin from analysis of different storms. These hydrographs are routed down river to site. Appropriate flood routing methods are used[1]. Calibration of flood hydrographs and flood routing parameters is essential.

However, a flood frequency analysis is the preferred method in practice. With the large network of gauge and discharge stations of CWC in major river basins, it is feasible to find one not far from the site. An example of a simple application of rational flood estimation procedure is given below [15].

The catchment area was as large as 368302 km^2. In large drainage basins($A>10000$ km^2) floods in tributary basins occur at different times at random. Combination of these make the flood event in the main river. As the number of tributaries increases, frequency distribution curve should tend to the normal distribution[16]. Flood gauge data were available at CWC station close to site for 1971-95. A rating curve was developed for determining discharges corresponding to the gauze records. The annual peak series was then analysed using normal distribution curve to yield the following flood values in m^3/s- $Q_m=25000$,$Q_{50}=35800$, $Q_{100}=37.200$. The corresponding flood levels(m) were given as 83.750, 87.540, 88.000 m. It was interesting to note that maximum observed flood (1978) was identical to Q_{100}. Dickens formula yielded a discharge 42155 m^3/s which exceeds 1000 year flood.

4. CONCLUSION

The method of estimation of design flood has been weaned away from the past practice of empiricism and irrational definition of design flood. Regional synthetic unitgraph (SUG) method is used instead. Parameters have been derived for the entire country divided into 26 regions. Design Flood is simply defined by a Return Period of 100 years. Maximum observed flood or Factors of safety for foundation design flood are no longer relevant. For large bridges and catchments regional analysis is precluded. Flood frequency analysis is usually adopted using nearest gauging station records.

There is little room for ambiguity in the present practice of flood estimation, given above. Yet, the boundaries between past practice and present may not be as clearly delimited as presumed in the paper. Traditions die hard and so do defunct Code prescriptions. These aberrations are better ignored without much ado.

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ROLE OF HYDRAULIC MODEL STUDIES IN BRIDGE DESIGN

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SUMMARY

Bridge construction requires careful planning and in-depth study as no undue risk should be taken in its design and construction. Hydraulic consideration in the bridge design comprises several aspects such as selection of site, determination of waterway, assessment of scour for design of foundation of piers and abutments, design of guide banks, approach banks, protection works, etc. Undermining of the piers due to excessive scour could become a potential cause for bridge failure. Mechanism of scour around bridge pier, factors affecting scour, estimation of depth and extent of scour, scour protection measures and some case studies conducted in this regard in Central Water and Power Research Station (CWPRS), Pune, have been discussed in this paper.

1. INTRODUCTION

1.1 River Characteristics

Indian rivers in flood plains are shallow and flow in a wide alluvial belt with meandering and braiding characteristics. The river Brahmaputra is intensely braided about 30 km upstream of Guwahati with a width of about 10 km. Thereafter it naturally constricts to 1.5 km at Saraighat bridge and again widens to 18 km at about 30 km downstream of this bridge. In 1980, construction of a 17 span road bridge at Tezpur was started on the river Brahmaputra from the hill located at the right bank. By the time construction progressed the deep channel shifted considerably towards the left and the bridge had to be completed with additional 7 spans on the left side to accommodate the lateral shift in the river regime. Before formulating any hydraulic project, it is therefore essential to understand the behaviour of the river in the vicinity of the project area including upstream and downstream stretch of the river regime.

Bridge construction requires careful planning and in-depth study as no undue risk should be taken in its design and construction. Study made by Smith on the failures of 143 bridges constructed between 1847 and 1975 indicated that majority of the bridges have failed due to scour around the piers and abutments. Other causes were defective design, overloading, adoption of inadequate or unsuitable erection techniques, earthquake forces and use of material or type without taking into account certain salient aspects which are critical or not known to be critical at the time of design and construction.

2.0 HYDRAULIC ASPECTS

Hydraulic aspects of bridge design consists of selection of site, optimum orientation and waterway, location of abutments, design of guide banks, approach embankments and design of bridge piers. As far as possible bridges are to be located on straight reaches and with alignment normal to the flow. Nodal points are ideal for locating bridges. High cost of bank to bank bridges and bank protection required on the upstream and downstream stretches of the river made the engineers to look for constricted bridges with guide banks and approach embankments. Waterway design depends upon the design discharge, type of river, whether aggrading or degrading, and nature of river such as braiding or meandering, etc. The empirical relation evolved by Lacey for stable width in alluvial rivers is widely used to determine the waterway for bridges. Inadequate waterway can result in excessive velocities across the bridge causing deep scour at the piers and the guide banks in addition to an undesirably high afflux on the upstream side. Excess waterway causes slackness in the flow thereby causing aggradation, promoting the formation of shoals resulting in non-uniform flow distribution and oblique approach of the flow to the bridge. Deviation from Lacey waterway becomes imperative in some cases to take care of special site conditions.

In constricted bridges, the abutments are provided with guide bunds (also called guide banks) and approach embankments. The guide bunds which ensure smooth passage of the river flow through the bridge, are so designed that, a safe marginal distance is available between extreme swing of deep channel with possible worst loop.

Bridge piers are founded on wells or in some cases on piles. When rocky strata is not available at a considerable depth and river bed is highly erodible, well foundations are suitable. When rocky strata is available at 6-20 m below bed level, pile foundation is preferred. In Karnataka and Goa, most of the bridges of Konkan Railway are located in the reaches of rivers affected by tidal variations (estuarine conditions) and strata comprises of marine silt or clay followed by dense sand, sandy clay, soft rock, etc. Pile foundations were considered suitable for these bridges. Speed of construction, economical and accurate construction and elimination of problems of tilting, shifting, etc. are the advantages of pile foundations over the well foundations.

- Undermining of the piers by scour is a potential cause of failure of bridge foundations. Local scour that is scour which occurs due to the presence of an obstruction to the flow causes a decrease in the bed elevation only in the area surrounding the obstruction. The dominant feature of the flow around a bridge pier essentially comprises the system of vortices. The most important of these are the horse-shoe vortex and wake vortex system (Fig.1). As the flow approaches the pier a stagnation plane is formed. Because of the vertical velocity profile a pressure gradient is formed along the stagnation plane on the pier. This gradient produces a downflow in front of the pier, which acts like a vertical jet in eroding the bed material. The indentations and downflow combine to excavate a hole at the leading edge of the pier. The incoming flow separates at the edge of the scour hole, creating a circulation or roller within the scour hole. The downflow divides at the bottom of the scour hole and spirals downstream past the pier. This together with the ground roller forms a horse-shoe vortex. It is very efficient in transporting dislodged sediment particles away from the pier. Wake vortices form at the downstream side of piers and are the result of flow separation at the sides of the pier. The wake vortices dissipate as they move downstream. The frequency of periodical vortex shedding downstream is directly proportional to the approach velocity and inversely proportional to the pier diameter.

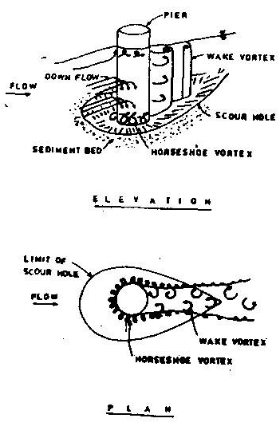

FIG.1 FLOW PATTERN AT CIRCULAR PIER

Depth of scour depends on a number of variables such as depth, velocity and angle of attack of flow, width of obstruction, soil strata and sediment size. In the case of non cohesive materials the characteristics of bed material which affect scour include sediment density, median size and standard deviation. Since lighter sediment will move at lower mean velocity or shear, greater scour can be expected. When the channel is not transporting sediment, the bed around the pier will continue to lower until the shear in the scour hole is critical.

- Clays are transported by water and after flocculation get deposited in main river channel on flood plains and in lakes and estuaries. When sandy material is mixed with silts and clays in different percentages, the material, exhibits a certain amount of cohesion. Adequate information is not available to determine scour depth around bridge piers in cohesive soils. Kand suggested that Lacey's silt factor be increased in the case of cohesive soils by using the relation. $f_C = F\ (1+C^{0.5})$ where f_C is Lacey silt factor for cohesive soils, C is cohesion in Kg/cm^2 and F is a coefficient based on angle of internal friction $\varnothing$. $F = 1.5$ for $\varnothing = 11^\circ$ to 15° and $F = 2.0$ for $\varnothing = 5^\circ$ or less. If $\varnothing$ is greater than 16° and $C \geq 0.2$ kg/cm^2, it is sandy soil with clay binding, and can be treated as sandy.

- Very limited data are available on scour around bridge piers in gravel bed rivers. The bed material of these rivers is usually characterised by relatively large mean size. It is during relatively large flood that all the particles in the bed material move, as the discharge reduces the coarser particles which cannot be moved, accumulate on the bed surface and form a layer of non-movable particles on the bed. This is known as protective armour layer or paving. When a bridge pier is constructed in a gravel - bed river as the scour progresses during the flood, coarser particles will accumulate in the scour hole and armouring effect will be increased. As a result, the scour depth will be much smaller than that in an alluvial river with relatively finer and uniform materials.

- Estimation of maximum scour can be grouped under three components viz., (a) general scour due to design flood, (b) scour due to constriction and (c) local scour due to pier obstruction. Laboratory studies are useful in predicting more accurately the third part i.e., local scour due to pier obstruction. Lacey-Inglis method of estimating scour around bridge piers is commonly used in India for piers placed in alluvial rivers and is recommended by the Indian Road Congress and Indian Railways. Inglis advocated maximum scour depth D_s below HFL, around a bridge pier as $D_s = 2\ D_L$ where D_L is the general scour depth below HFL suggested by Lacey as $1.34\ (q^2/f)^{1/3}$ where q = maximum discharge intensity in cum/s/m and f is silt factor = $1.76\ m^{0.5}$, where 'm' is the mean diameter of the bed material in mm. This method is meant for sandy rivers of meandering type.

- In an estuary or a tidal river where flow is subjected to periodical change in direction, the scour of the river bed occurs mostly during ebb tide (seaward flow). During flood the scour of the tidal river bed is supposed to be nominal because the increase in discharge is being accommodated mostly by rise in water level rather than by lowering of the bed levels by scouring. The phenomena of the scour depend considerably upon the order of velocity which persists for a prolonged period in the tidal cycle which occurs generally at the mean tide level (MTL). Therefore, for computations of regime depth D in the tidal river, the normal depth of water should be measured from MTL and not from high flood level (HFL). Also the computations of discharge intensities and mean velocity should be undertaken at the mean tide level. The regime depth is thus obtained using Lacey's empirical regime formula which is applicable to alluvial river. In tidal river of this kind, the maximum natural depth of scour is obtained by using a multiplying factor of 1.25 to 2.7 to the regime depth.

- The criteria for scour protection and the level of foundation are different for the deep and shallow types. For deep foundations, usually no scour protection is provided. But there are many cases where a shallow type pier foundation has to be selected. This type of foundation is greatly subjected to scour risk and therefore adequate design allowance and scour protection are required.

  Stones for scour protection (Fig.2) are laid over filters which help in arresting the leaching of finer base material or river sediment through the rip-rap primarily due to upward hydraulic gradient and turbulence within the rip-rap layer. The filter should be fine enough to prevent the base material from entering

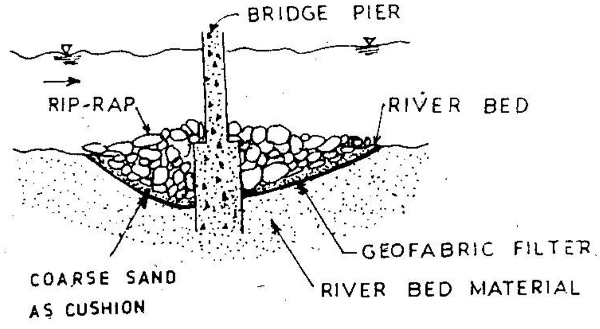

FIG.2. SCOUR PROTECTION FOR A BRIDGE PIER

and it should be much more permeable to water than the base material. Using geofabric filter is a relatively new and modern development and is advantageous both in terms of economy and ease of construction as compared to the graded filter. In order to prevent damage to the geofabric filter while placing the stones, a 15 cm thick layer of coarse sand should be provided over the filter as a cushion.

3. NEED FOR MODEL STUDIES

In spite of availability of many empirical formulae associated with analysis of certain river parameters in the vicinity of the bridge, it has been found that model studies either physical or mathematical would be valuable in optimising the design parameters to suit the specific site conditions, thereby reducing the risk of bridge failures. Morphological studies of the river upstream and downstream would help to understand the river behaviour, changes in the river channel alignment, formation and development of shoals, bars, islands, bank erosion, etc. Information analysed under pre-bridge conditions would help to estimate the likely morphological changes in the river under post-bridge conditions. CWPRS has conducted physical, mathematical and morphological studies for various bridges to derive optimum design parameters or to solve certain problems faced by the engineers.

4. CASE STUDIES

4.1 The Toka Bridge

The Toka bridge is situated across the Godavari river on Pune-Aurangabad sector of State Highway No.27. The construction of the bridge was completed in the year 1961. Safety of the bridge was required to be ascertained in view of the construction of a dam downstream of the bridge at Paithan. On the basis of the analysis of data for 9 years, the permissible scour level to achieve the required grip length was worked out which was more or less equal to the existing average river bed level. It was, therefore, necessary to provide proper protection at the existing river bed level for preventing local scour thereby maintaining the design grip length. It was therefore suggested to provide 0.6 m X 0.6 m X 0.45 m cement concrete blocks over a granular filter in 6.40 m width around the piers with top of the protection flush with the river bed level (Fig.3). Since laying of the granular filter under flowing water was difficult, project engineers laid cement concrete blocks in two layers with staggered joints to minimise loss of bed sand through the gaps. After the construction of protection works, heavy flood occurred and the performance was reported to be satisfactory except that a few blocks at the edges of the protection were disturbed.

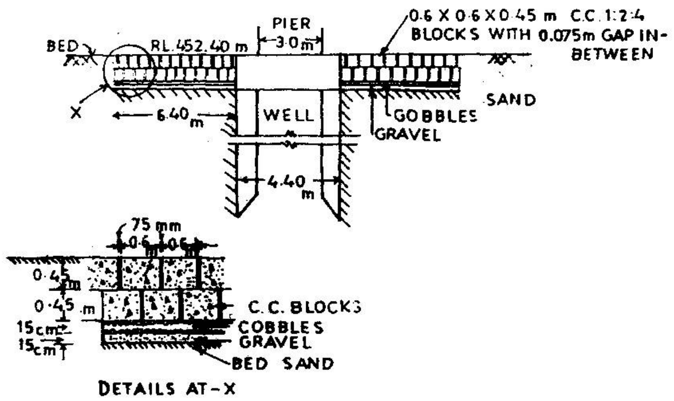

FIG.3.TYPICAL PIER PROTECTION AT TOKA BRIDGE

4.2 The Alamuru Bridge

The Alamuru bridge is situated across the Godavari river downstream of the Dowlaiswaram anicut on the stretch of National Highway between Vijaywada and Visakhapatnam. The width of the river at this location is 2441 m. On the basis of model studies conducted at CWPRS for the design discharge of 56,600 cum/s, a bridge with waterway of 1454 m with suitable guide bunds was suggested to achieve uniform distribution of flow in various spans. However, in the year 1967, a bridge with total waterway of 2341 m was constructed.

During the year 1978, deep scour developed at the second pier from the left abutment resulting in its tilting. At that time the discharge was only 13,000 cum/s which was much less than the design discharge of 56,600 cum/s. However, excessive waterway had resulted in the formation of islands which led to the concentration of flow with an oblique approach. Based on some measurements at site, the discharge intensity at the affected pier was estimated to be about 71 cum/s/m with an obliquity of 25° to 30° and the observed scour level was (-) 16.15 m. The bottom level of wells was at RL (-) 25.0 m.

The general scour level corresponding to the discharge intensity of 71 cum/s/m worked out to be RL (-) 0.6 m. However, analysis of data after the floods indicated that the deepest bed level had shifted from pier No.2 to 4 and was at RL (-)2.0 m. It was difficult to do the excavation of 4 m in the water and therefore protection at RL (-) 2.0 m over a width of 11.5 m around the pier was suggested (Fig. 4). At other locations, the bed level was higher where the protection was not needed immediately.

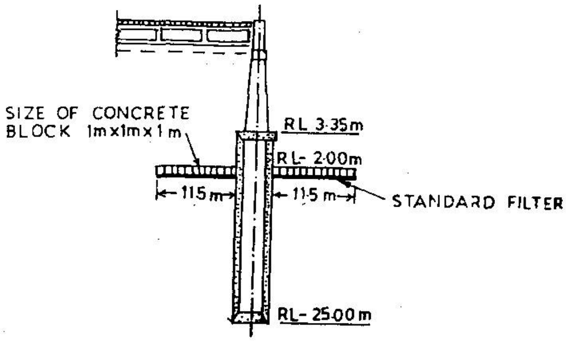

FIG. 4. DETAILS OF PROTECTION AT ALAMURU BRIDGE PIER NO.4

4.3 The Delhi-NOIDA Bridge

50 km of river Yamuna traverses through the National Capital Territory of Delhi. In the urban area of Delhi, i.e., within 25 km three barrages and four bridges exist, most of which have been constructed based on the model studies and recommendations of the CWPRS. During the last 10 years, proposals for four more bridges were studied in the CWPRS. NOIDA is on the left bank of the river. In order to connect NOIDA with Delhi, a road bridge (Fig. 5) was proposed in between Nizamuddin road bridge and Okhla weir.

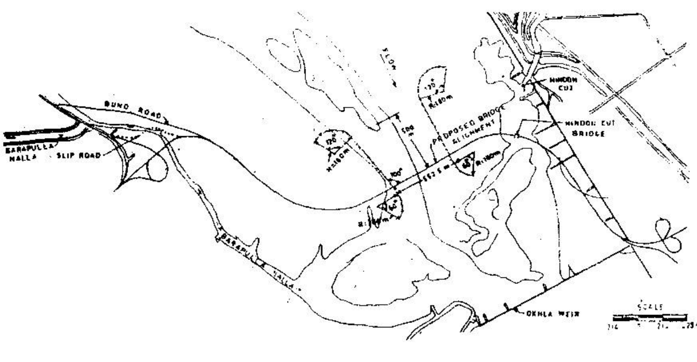

FIG. 5 DELHI NOIDA BRIDGE SITE PLAN

Model studies were carried out to study the scouring pattern and to compare the scour around the group of piles with that around the well foundation of the bridge pier for similar flow condition (Fig. 6). Studies were carried out in a flume and the scour development in both the cases were studied and compared. It was found that depth of scour was more for the well foundation as compared to that for the foundation with group of piles. Studies conducted with normal flow and obliquity of flow indicated more depth of scour with oblique flow compared to that with normal flow.

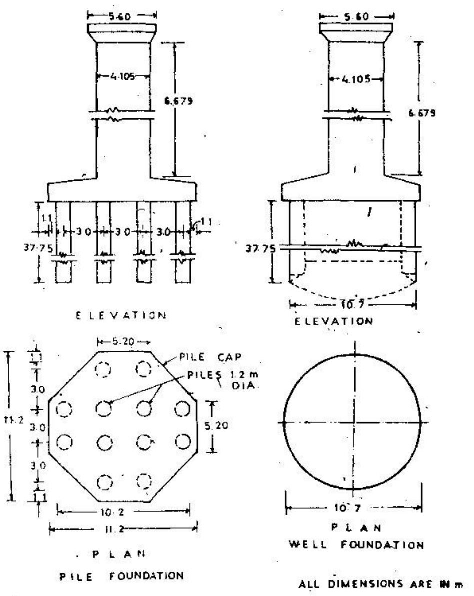

FIG. 6, DETAILS OF PIER FOUNDATION

4.4 <u>The Gouranga Bridge Near Nabadwip</u>

The Gouranga bridge is situated on the river Hooghly about 160 km upstream of Calcutta (Fig.7). The left bank of the river upstream of the bridge had been eroding for the last few years and there seemed to be a possibility of the bridge being outflanked. Due to concentration of flow the second pier from the left abutment was experiencing deep scour. In order to avoid further deterioration of the situation and limited time available before the onset of the next monsoon, short term measures were suggested on the basis of site inspection and available data. To avoid the danger of the bridge being outflanked, protection was recommended in the upstream embayment for a length of 160 m. In addition, porcupines were suggested at the toe in the further upstream reach of about 100 m. For control of scour at the endangered pier, provision of two layers of 50 kg stones in a 60 m width all around the pier at the existing river bed over geo-jute/nylon bags was suggested. For evolving long term measures, studies were subsequently conducted on a physical mobile bed model constructed to a horizontal scale of 1:300 and vertical scale of 1:50 reproducing a river reach from 3.50 km upstream to 2 km downstream of the bridge. These studies revealed the necessity of providing continuous protection from 100 m downstream of the bridge to 100 m upstream of the embayment along the left bank with stones weighing 40 kg or more in two layers over a synthetic filter. For the protection of the toe of the left bank, 15 m wide apron consisting of two layers of 40 kg stones over a synthetic filter was recommended.

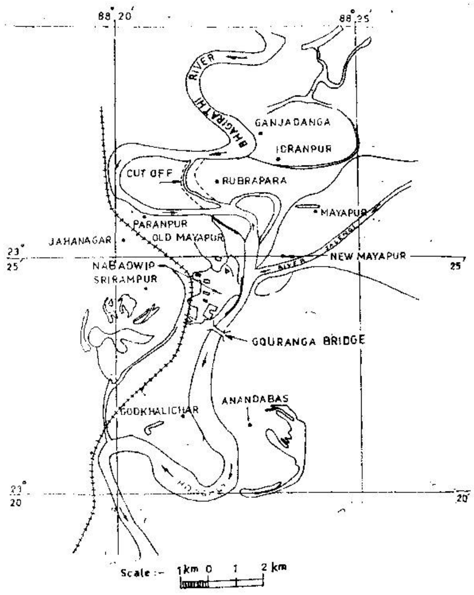

FIG. 7 LOCATION OF GOURANGA BRIDGE AND THE CUT-OFF DEVELOPED

5. CONCLUSIONS AND FUTURE STUDIES

- When the river is degrading, bridge scour should be monitored for scour and when necessary, protection by way of garlanding should be provided.

- River plan form changes can be monitored with the help of Remote Sensing data. This analysis would help to estimate change in discharge intensities across a bridge which in turn help in estimating the maximum scour.

- Adoption of pile foundation for bridge pier can reduce local scour and this may therefore be adopted when feasible.

- Understanding of soil-structure interaction, scour and fill process in a river with a boulder-bed are some of the grey areas where considerable research including field monitoring, need to be taken up. In India scour observations at hydraulic structures during floods are very rarely taken due to non-availability of suitable instruments. Sometimes observations are taken when the flood recedes and scour pockets get filled up by bed material. The scour observations are taken by sounding from bridge decking which are affected by drifting due to high velocities. For scour observations during floods, it is very much essential to install automatic recording type instrument. Such instrument should be compatible for installing at the bridge pier. It should give a clear indication of the depth of scour under all flow conditions. The system should record the onset of scour, maximum depth of scour and filling of scour holes following high flow events.

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Design and Evaluation of Bridges for Scour in the United States of North America

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Summary

The catastrophic failures of several bridges in the United States of North America has focused national attention towards the need to develop technology for designing new bridges and for evaluating existing bridges over waterways for the effect of total scour around the bridge foundations. The FHWA, an agency of the United States Department of Transportation, has taken the lead in developing and disseminating technology and guidance on stream stability, scour, and scour countermeasures for highway bridges over waterways. The FHWA has disseminated state-of-the-art technology through its Hydraulics Engineering Circular (HEC) -18, "Evaluating Scour at Bridges," HEC-20, "Stream Stability at Highway Structures," and HEC-23, "Scour and Stream Instability Countermeasures." This paper will discuss how the technology presented in these HEC's is used in the United States of North America for designing new bridge foundations and for evaluating the stability of the foundation of existing bridges over waterways for the safety of the public users.

1. Introduction

The FHWA has been proactive in disseminating state-of-the-art technology and guidance for the design of new bridges and the evaluation of bridges susceptible to scour since 10 people lost their lives during the failure of the New York Thruway bridge over the Schoharie Creek in New York in 1987. Other failures include: the I-29 crossing of the Big Sioux River in South Dakota in 1962; the I-80 crossing of the John Day River in Oregon in 1964; 73 bridges destroyed by flooding in Pennsylvania, Virginia and West Virginia in 1985; 17 bridges in New York and New England states in the spring of 1987; the US 51 bridge over the Hatchie River in Tennessee in 1989 (eight people were killed); the I-5 bridges over Arroyo Pasajero in California in 1995 (seven people were killed); and the bridge over the Wantagh Parkway in New York in 1998.

The scour evaluation of bridges over waterways were established by the FHWA in 1991. State departments of transportation (DOTs) have been reporting progress towards completing their scour evaluations in a biannual basis. The current status is presented later on in this paper. In addition, the FHWA recommends that new bridges be designed for scour from floods equal to or less than the 100-year flood. The current editions of HEC-18 and HEC-20, third and second edition, respectively, contain updated technology for calculating total scour and for assessing stream instability of channels. In addition, FHWA has published HEC-23 to provide DOTs with state-of-the-art guidance for the selection and design of bridge scour and stream instability countermeasures.

2. The National Approach

The Federal Highway Administration (FHWA) maintains an inventory of bridges through its National Bridge Inventory. The bridge inventory contains a database of over 575,000 bridges as reported by DOTs. About 84% of these bridges, or 484,060, are over waterways. The Technical Advisory (TA) 5140.20, "Scour at Bridges," released by FHWA in 1988, contained guidance for designing new bridges for scour and for conducting scour evaluations on existing bridges over waterways. Techniques for estimating scour were presented in an attachment to the TA, the FHWA's Interim Procedures for Estimating Scour at Bridges. The guidance contained in this TA and its attachment has been followed by DOTs for designing new bridges and for evaluating the condition of existing bridges from scour. TA 5140.20 was superseded in 1991 by TA 5140.23, "Evaluating Scour at Bridges," which introduced the FHWA's Hydraulic Engineering Circular No. 18 (HEC-18), which superseded the FHWA Interim Procedures. The TA 5140.23 is very comprehensive and focuses on the development and implementation of a scour evaluation program for designing new bridges to resist damage from scour, evaluating existing bridges for vulnerability to scour, using scour countermeasures, and improving the state-of-practice for estimating scour at bridges.

2.1 Recommended Five Step Process

The guidance from the TA gives a five step procedure to follow:

1. An interdisciplinary team of hydraulic, geotechnical and structural engineers should conduct scour evaluations.
2. New bridges should be designed to be scour safe for a superflood on the order of magnitude of a 500-year flood.
3. All existing bridges over waterways with scourable beds should be evaluated for the risk of scour failures for such a superflood.
4. A plan of action should be develop for all bridges that are determined to be scour critical.
5. All bridges should be inspected for scour during the regular two year bridge inspection cycle.

FHWA also has three technical publications that provide technical guidance: HEC-18--provides guidance for developing a scour evaluation program and analyzing bridges for scour; HEC-20--provides guidance for analyzing the effect of stream instability on bridges; and HEC-23--provides guidance for the selection of suitable countermeasures to mitigate potential damage to bridges and highways at stream crossings.

2.1.1 Interdisciplinary Team

In designing new and evaluating the existing condition of bridges for scour, a careful evaluation of the hydraulic, geotechnical and structural aspects of the bridge foundations is required. An interdisciplinary team of experienced engineers is needed to make engineering judgements resulting from the complex nature of streams, flow patterns, oil and structure design. In addition, the team should establish priorities for scour evaluations, determine if the bridge is scour critical and recommend countermeasures and monitoring schedules to mitigate the potential effect of scour on the stability of bridge foundations.

2.1.2 Guidance for Designing New Bridges for Scour

New bridges over waterways on scourable streambeds should be designed for scour from floods equal to or less than the 100-year flood and checked for the potential scour resulting from the magnitude of a superflood (i.e., a 500-year event or 1.7 times the magnitude of the 100-year event). The geotechnical analysis should assume that the streambed material within the scour prism (total scour) is not available for bearing or lateral support. For the superflood condition, the geotechnical analysis for the superflood should incorporate a factor of safety of 1.0.

Prior to estimating total scour it is necessary to identify any potential for streambed aggradation or degradation as well as any potential for lateral streambed migration. With this information available and knowing the streambed characteristics then one can estimate total scour for a new bridge following these steps, as recommended in HEC-18:

Step 1. The designer should select a flood event or events that are expected to produce the worst scour condition.

Step 2. Water surface profiles for the flood flows should be developed. Hydraulics variables such as velocity and depth of water should be calculated.

Step 3. Estimate total scour. Check for geotechnical safety factors commonly accepted by the department of transportations.

Step 4. Plot total scour depths.

Step 5. Evaluate the results and apply engineering judgement.

Step 6. Evaluate bridge type, size and location based on results.

Step 7. Perform a foundation analysis on the basis that all streambed material in the total scour prism has been removed and is not available for bearing or lateral support of the bridge foundation.

Step 8. Repeat Steps 2 through 7 for a superflood condition. Check that the foundation have a minimum factor of safety of 1.0 (ultimate load) under this condition.

2.1.3 Guidance for Evaluating Existing Bridges for Scour

Existing bridges over riverine or tidal waterways should be evaluated to assess their vulnerability to floods and to determine if they are scour critical (foundations are unstable) or low risk to scour. The FHWA recommended that these evaluations should be conducted by the interdisciplinary team. In addition, the FHWA recommends that the evaluations should be made for the magnitude of a superflood condition (i.e., 500-year flood). Steps 1 through 7 presented for designing new bridges could be followed for the scour evaluation of a bridge for the superflood condition.

If a bridge is found to be scour critical, the bridge owner should have an action plan with specific procedures to follow to make the bridge less vulnerable to scour for the safety of the public users. The procedures may include among others specific instructions to close the bridge during floods and the timely installation of scour countermeasures. The equations presented in HEC-18 are recommended by the FHWA for evaluating scour at bridges.

2.1.4 Evaluation Procedure

Step 1. Bridges over waterways should be screened by an interdisciplinary team into five categories: 1) low-risk; 2) scour susceptible; 3) unknown foundations; 4) tidal waterways; 5) scour critical

Step 2. Bridges identified as scour susceptible bridges, unknown foundations and over tidal waterways should be prioritized for evaluation by conducting a preliminary office and field review using factors identified by the interdisciplinary team.

Step 3. Conduct office and field scour evaluations of the bridges which were prioritized under step 2. Steps 1 through 7 presented under "Guidance for Designing New Bridges for Scour" should be followed. The 500-year flood condition should be used during the evaluation.

Step 4. Bridges identified as scour critical should have a plan of action for correcting the scour problem.

Step 5. Remaining bridges (low-risk) should be evaluated giving priority status to the functional classification of the highway and bridges that are vital links in the transportation network of a city or region.

2.1.5 Plan of Action

A plan of action for each scour critical bridge should be developed by the interdisciplinary team. The plan of action should include:

- instructions for the type and frequency of inspections to be made at the bridge site;
- monitoring the bridge scour performance with contingency to closure;
- and/or scheduling timely design and construction of scour countermeasures.

3. Hydraulics Engineering Circular No. 18

HEC-18 contains the stat-of-the-art methodology for designing new bridges over waterways to resist the effect of scour around its foundations and for estimating scour at existing bridges over waterways. The third edition of HEC-18 presents the latest advances in technology including: conversion to the metric system of units; the addition of a gradation correction factor for the pier scour equation; and equation for estimating the correction factor for the flow angle of attack with respect to a pier; an interim procedure for estimating pier scour considering the effect of debris; and updated information on scour detection equipment. In addition, clarification has bed added for: estimating pier scour for exposed footings; pile caps located at different elevations in the flow; the effect of multiple columns skewed to the flow; preliminary information on scour resulting from pressure flow; and criteria for designing the foundation depth of a bridge abutment. Furthermore, HEC-18 presents basic concepts and definitions of scour; guidelines for designing bridges to resist scour, guidelines for estimating scour at existing bridges; guidelines for inspecting bridges for scour; and guidelines for establishing a plan of action for installing scour countermeasures.

4. Hydraulics Engineering Circular No. 20

HEC-20 contains guidelines for identifying stream instability problems that may control the location of a bridge. Factors which affect stream stability are classified as geomorphic, hydraulic, location and design factors. A qualitative assessment process leading to a quantitative analysis is given. A three-level approach is suggested in analyzing stream stability. In addition, HEC-20 presents guidelines for the selection of countermeasures for stream instability.

5. Hydraulics Engineering Circular No. 23

HEC-23 provides guidelines for the selection and design of stream stability and scour countermeasures which have been successfully used by DOTs. A matrix of the different countermeasures giving appropriate use of each is presented in HEC-23. This matrix presents the countermeasures by groups: hydraulics, structural and monitoring. It provides a fast way of identifying which countermeasure is appropriate foı specific condition. In addition, it rates each countermeasure on its functional application, suitable river environment, degree of maintenance needed, and installation experience.

6. Status of the Scour Evaluation Program in the United States

The FHWA initiated semiannual status reports on bridge scour on February 5, 1990. Several years have passed since the FHWA initiated the requirement that DOTs submit a biannual status report. The current status, as of April 15, 1998, reported by DOTs is presented in the following table:

Table 1

EVALUATIONS CATEGORIES	NBI Item 113	EVALUATIONS TOTALS	
		NUMBER	PERCENT
Evaluations Completed			
• Low Risk	4,5,7-9	312,294	80.3%
• Scour Critical	0-3	18,090	4.7%
Evaluations Needed			
• Scour Susceptible	6	58,027	14.9%
• Not Screened	6	315	0.1%
TOTAL EVALUATIONS		388,726	100%
• Evaluation Deferred	6	95,334	

The FHWA has continued its proactive approach towards completing the scour evaluations by encouraging its field offices to continue to work in partnership with DOTs management officials to encourage them to develop an action plan that is responsive towards completion of their scour evaluations. FHWA has also provided DOTs that have not made substantial progress towards completing their scour evaluations with example action plans to assist them in developing a revised action plan for the completion of their scour evaluations.

Since technology for the evaluation of bridges with unknown foundations and bridges subject to the influence of tides was not available at the time of initiating the bridge scour evaluations, FHWA exempted these bridges. To date, 95,334 bridges under these categories are pending an evaluation. These categories are represented in Table 1 as "Evaluations Deferred." Since technology needed to evaluate these bridges is now being phased into practice, FHWA has requested that DOTs begin their evaluations of these bridges, as applicable. Guidelines for evaluating bridges over tidal waterways is currently available thanks to the 12 State Pooled-Fund project, led by South Carolina DOT, which produced a users manual titled "Tidal Hydraulic Modeling for Bridges" dated December 1997. Non-destructive tests for identifying unknown foundations have been evaluated and field tested under the National Cooperative Highway Research Project 21-5 titled "Nondestructive Testing for Unknown Subsurface Bridge Foundations."

7. Conclusion

The FHWA will continue being proactive towards the scour evaluations and design of bridges over waterways. FHWA is currently working on updates to its three major publications, HEC's -18, -20, and -23 to continue to provide DOTs with the state-of the-art technology on scour, stream stability and countermeasures. Furthermore, FHWA, in partnership with AASHTO and NCHRP will participate on a scanning tour with the purpose of visiting other countries to investigate their technology on scour countermeasures for potential application in the United States.

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Mathematical Modelling for Scour Around Bridge Piers

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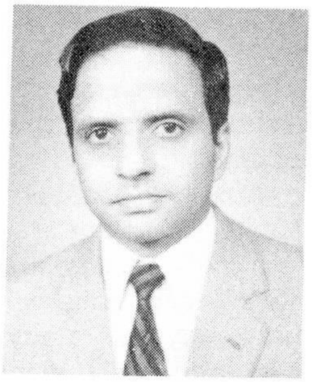

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ABSTRACT

Many design relationships for scour are available as a result of studies carried out so far by several researchers. Lacey- Inglis method is mostly used for estimation of design scour depth in Indian Railways. However, the scour depth as estimated by this or many of the other methods is expected to occur after a very long time of scour activity. However, the flood discharge which is used for the estimation of design scour depth does not last that long. Thus, realistic estimation of scour depth does need the scour modelling for estimation of temporal variation of scour depth and the equilibrium scour depth. In the present paper a mathematical model is discussed for the estimation of temporal variation of scour depth and the equilibrium scour depth. Limitations of the Lacey-Inglis method for scour estimation in alluvial and boulder bed rivers are identified. Also the results available on effect of bed material cohesion on scour are reported.

INTRODUCTION

Local scour around bridge piers and abutments is a problem of continuing interest. Huber (1991) has reported that since that year 1950, over 500 bridges have failed in the USA and a majority of them were the result of hydraulic conditions and primarily the scour of foundation material. Such data are not available for bridges of other countries including India. However, this has remained a matter of concern for all and the realistic estimation of scour depth around bridge piers is thus very important.

It is now well known that for given flow conditions scour at bridge abutments is smaller than that at bridge piers of same dimensions. Several design relationships for estimation of scour are available as a result of studies carried our so far by numerous investigators. These methods are useful in determination of the design scour depth at a bridge pier for steady flow conditions. However, the flow in a river during a flood is unsteady and discharge changes in it are quite rapid. Therefore, scour modelling in unsteady or flood flows is required for realistic estimation of the scour depth.

In the present paper, first a mathematical model for the estimation of temporal variation of scour depth in steady and unsteady flows is discussed. Use of this model for scour estimation under flood flow situations is demonstrated. Next, some available methods for estimation of design scour depth are reviewed. The processes of scour in cohesive and boulder bed rivers are also discussed.

TEMPORAL VARIATION OF SCOUR DEPTH

The time required by the design discharge to scour to its full potential is generally much larger than the time for which it occurs. Therefore, computations on temporal variation of scour depth are important for design purposes. The horse-shoe vortex and associated downflow are considered to be the main agents causing scour at bridge piers. Based on the characteristics of horse-shoe vortex (see Fig. 1) a scheme is proposed by Kothyari et al. (1992,a & b) for computation of temporal variation of scour depth. Various steps involved in computations through this scheme for uniform size bed material are depicted in Fig.2 . The results produced through this scheme have been verified using experimental data of various investigators. However, river bed material invariably consists of sediments of varying size fractions i.e. it is non-uniform . The scheme depicted in Fig. 2 can also be used for computation of temporal variation in non-uniform sediments. For this the effective size d_e is defined as the size of uniform sediment that gets scoured at the same rate as the non-uniform material under the given flow and pier conditions. Kothyari et at. (1992, a) gave the following equation for d_e;

$$\frac{d_e}{d_{50}} = 0.925\,\sigma_g^{0.67} \qquad (1)$$

Here d_{50} is the sediment size of non-uniform material such that 50 percent material is finer than this by weight and σ_g is the geometric standard deviation of the material. Use of d_e as given by Eq. (1) in place of d in Fig .2 will give the temporal variation of scour depth in non-uniform sediments.

For computation of temporal variation of scour due to unsteady flows, the hydrograph causing unsteadiness can be discretised into steady segments as shown in Fig. 3. The scour depth is computed as per Fig.2 in each segment of flow. Scour depth at the end of preceding segment becomes equal to the scour depth in the beginning of the next segment. Comparison of results obtained with the experimentally observed data is also given in Fig .3.

EQUATIONS FOR PREDICTION OF EQUILIBRIUM SCOUR-DEPTH

The equilibrium scour depth can be obtained from the temporal variation as the scour at a large time . It is, however, desirable from practical considerations to have a simple relationship for equilibrium scour depth. Available equations for equilibrium scour depth are expected to provide conservative estimates of the scour depth in unsteady flows. A brief description of important equations is given below :

Lacey -Inglis Equation

Lacey -Inglis approach for scour estimation is used by Indian Railways and other government organizations. During the early part of the present century, Lacey (1929) analyzed the data of stable irrigation canals flowing through loose noncohesive sandy materials in Indo-Gangetic plains and obtained the following equation for flow depth (or hydraulic radius) D_{LQ}

$$D_{LQ} = 0.47\,(Q/f)^{0.33} \qquad (2)$$

Here Q is the discharge in m^3/s, D_{LQ} is the depth in m and f is Lacey's silt factor which is related to median size of the bed material d as below.

$$f = 1.76\sqrt{d} \qquad (3)$$

Here d is in mm. On the basis of analysis of scour data on 17 bridges in alluvial rivers in North India, Inglis (1949) found that the maximum scour below the water level D_{se} is related to computed value of D_{LQ} as

$$D_{se} = K\,D_{LQ} \qquad (4)$$

where K varies from 1.76 to 2.59 with an average value of about 2.0 . When bridge pier foundations are to be designed, this equation will be used for a flood discharge of return period 50 to 100 years, even through Eq. (4) is at best valid for bankful discharge. Also value of coefficient K is Eq. (4) should depend upon pier shape and size, sediment gradation, obliquity of flow etc. Since these factors are not explicitly taken into account, Lacey-Inglis method should not be used outside the range of data on which it is based.

Laursen-Toch Equation

The equation proposed by Laursen and Toch (1956) for prediction of equilibrium scour depth below river bed level d_{se} is give as below.

$$d_{se}/D = 1.35\,(b/D)^{0.70} \qquad (5)$$

Here D is flow depth and b is pier diameter.

Melville and Sutherland Equation

Melville and sutherland (1988) assumed that the largest possible scour depth around bridge piers is given as below :

$$dse = 2.4\ b \qquad (6)$$

This scour depth is reduced by multiplying factors which depend upon whether clear-water or live-bed conditions exist, flow depth is shallow and sediment is graded. The multiplying factors are determined from the analysis of experimental data covering a wide range of pertinent variables.

Kothyari -Garde - Ranga Raju's Method

Based on the analysis of extensive laboratory data collected using uniform, non-uniform and stratified sediments and steady and unsteady flows, Kothyari et at. (1992, a & b) have proposed the following equations for scour estimation

Clear- water condition :

$$\frac{d_{se}}{b} = 0.66 \left(\frac{b}{d}\right)^{-0.25} \left(\frac{D}{d}\right)^{0.16} \left(\frac{U^2 - U_c^2}{\frac{\Delta\gamma_s}{\rho} d}\right)^{0.4} \alpha^{-0.30} \qquad (7)$$

where the average critical velocity U_c is given by

$$\frac{U_c^2}{\frac{\Delta\gamma_s}{\rho} d} = 1.2 \left(\frac{b}{d}\right)^{-0.11} \left(\frac{D}{d}\right)^{0.16} \qquad (8)$$

and opening ratio α is given as $\alpha = (B-b)/B$

Here U is flow velocity, ρ is mass density of water, $\Delta\gamma_s = \gamma_s - \gamma_f$, γ_s and γ_f are specific weights of sediment and water respectively, α is opening ratio and B is center to center spacing of piers.

Live-bed Condition :

$$\frac{d_{se}}{b} = 0.88 \left(\frac{b}{d}\right)^{-0.33} \left(\frac{D}{d}\right)^{0.4} \alpha^{-0.3} \qquad (9)$$

It may be seen that in sediment transporting flows, the scour depth is not dependent on velocity. When the sediment is non-uniform, effective sediment size d_e can be used in Eq. (7) and (9) instead of d, the former being given by Eq. (1).

Grade and Kothyari (1997) have tested the above methods for scour estimation by using field data from 17 bridges in India, 55 bridges in USA, 6 bridges in New Zealand and 5 bridges in Canada. Results obtained through these are summarized in Table - 1.

TABLE 1: Comparison of Accuracy of Prediction of Scour Depth by Different Methods

Methods	% of Data points falling within given error band		
	± 30	± 50	± 90
Lacey-Inglis	59	85	100
Laursen- Toch	38	65	98
Melville-Sutherland	79	95	100
Kothyari - Garde- Ranga Raju	86	96	100

It can thus be seen that among the methods tested, the methods by Melville-Sutherland and Kothyari et al. give results of better accuracy. These methods indeed take into accurant the effects of flow depth, velocity, pier shape and size and the size distribution of river bed material.

SCOUR IN COHESIVE SOILS

When the river bed consists of clayey material, forces also act between soil particles imparting cohesion into it which resists the dislodgment of particles by the flow. Therefore, scour in cohesive materials is more complex and less understood than the scour in noncohesive sandy material. The rate and amount at which clayey material gets eroded due to flow depends upon type and percentage of clay, antecedent moisture conditions in clayey beds, quality of water etc. Some investigators have tried to relate the scour in cohesive soils to plasticity index, vane shear strength and other such properties, but these attempts are not very successful. Some basic work on scour in cohesive soils has been undertaken by the writers. Preliminary results reveal that for the given pier, scour in cohesive soils can even be more than that in cohesionless soils depending upon moisture state of the soil prior to the start of scouring, See Fig. 4 (Sarfaraz,1998).

SCOUR IN GRAVEL-BED RIVERS

Gravel-bed rivers are characterised by relatively large median size and large standard deviation. When a bridge pier is constructed in such strata, the coarser particles would accumulate in the scour hole thus forming an armor layer and partly inhibiting further development of scour. Hence scour depth obtained would be smaller than that in uniform material having the same d_{50} size.

The IRC-78 (1979) code recommends that scour depth in gravel-bed rivers be taken as a multiple of flow depth estimated by using Lacey-Inglis approach involving discharge intensity q (i.e. river discharge per unit width), as below:

$$D_{Lq} = 1.33 \left(\frac{q^2}{f}\right)^{1/3} \tag{12}$$

and a silt factor of 24. In this connection, it may be stated that basically, Inglis-Lacey relation for flow depth was derived for sandy bed rivers, and gravel bed rivers are not expected to follow the same. Published data of gravel bed rivers indicate the depth of flow relationship as below (Hey and Heritage, (1993)

$$D = a_o \; Q^{a1} \; d_{50}^{a2} \tag{11}$$

Here, the coefficient a_1 is found to vary between 0.33 and 0.49 and a_2 between -0.03 and -0.12. This is thus different from Lacey's relation i.e. Eq. (11). Also bed material size of gravel bed rivers varies over a wide range and an armor layer may be formed during scour. Hence use of a constant silt factor of 24 in Eq. (11) is thus questionable. Also Lacey's method does not take into account the effect of pier shape and size on scour.

The methods of Kothyari -Garde- Ranga Raju and Melville-Sutherland, however take into account the effect of pier size, shape, sediment non-uniformity and hence armoring effects. Nevertheless, there is a need to collect scour data from gravel-bed rivers for studying the relative accuracy of available methods.

CONCLUSIONS

Available methods for computation of design scour depth around bridge piers are reviewed. It is seen that the computation of temporal variation of scour depth is required for realistic estimation of design scour depth. Enough information is not found to exist on scour around bridge piers in clayey and boulder bed rivers. It is concluded that Lacey-Inglis method should be used for sand bed rivers only precisely within the range for it was developed. This should be used with caution, if at all, in rivers with clayey or gravel beds.

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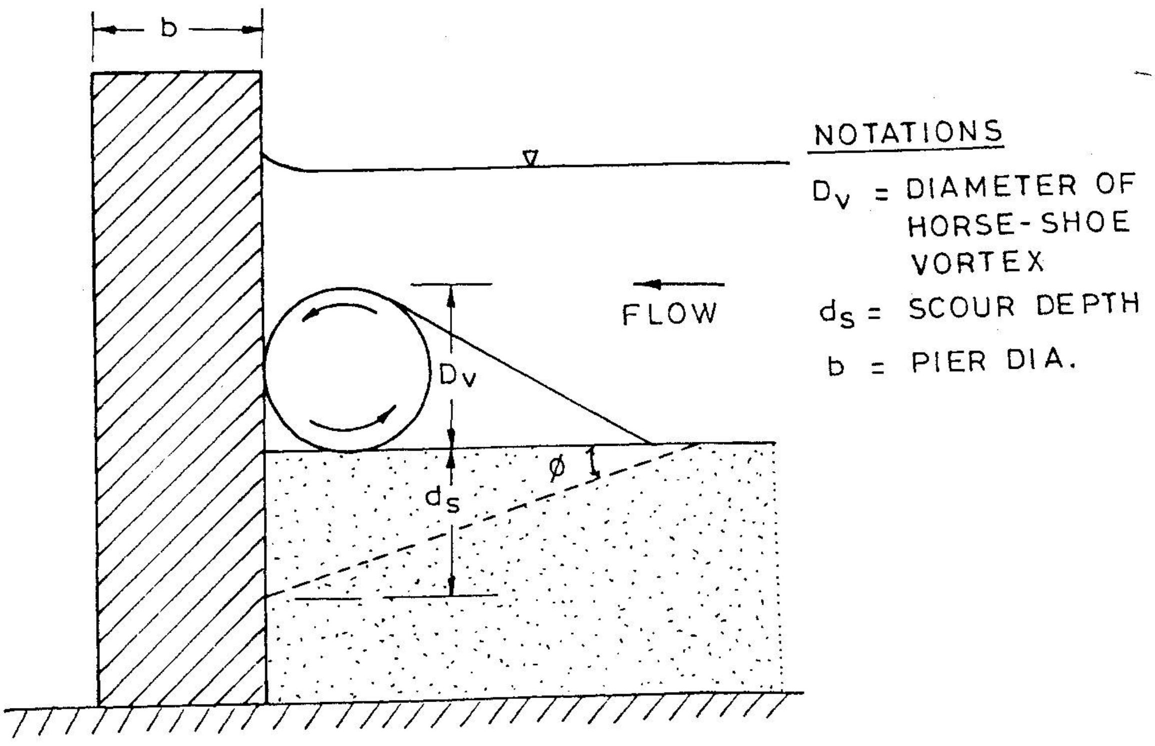

Fig.1 Definition diagram of horse-shoe vortex

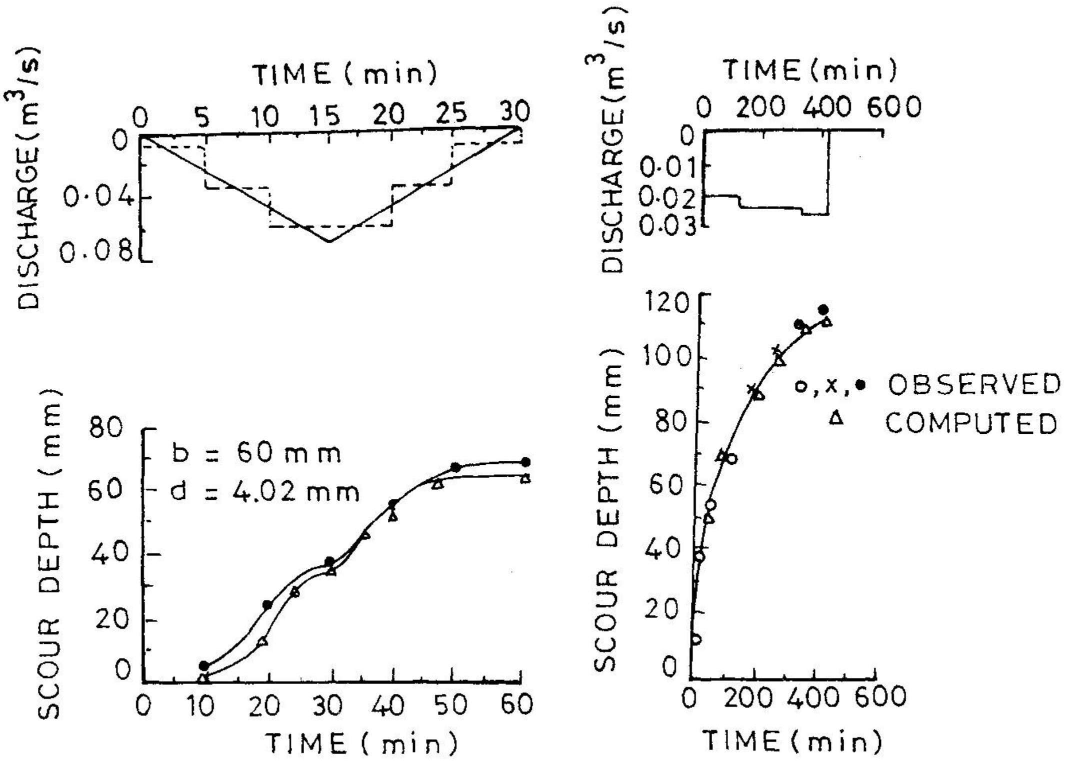

Fig. 3 Temporal variation of scour depth in unsteady flows

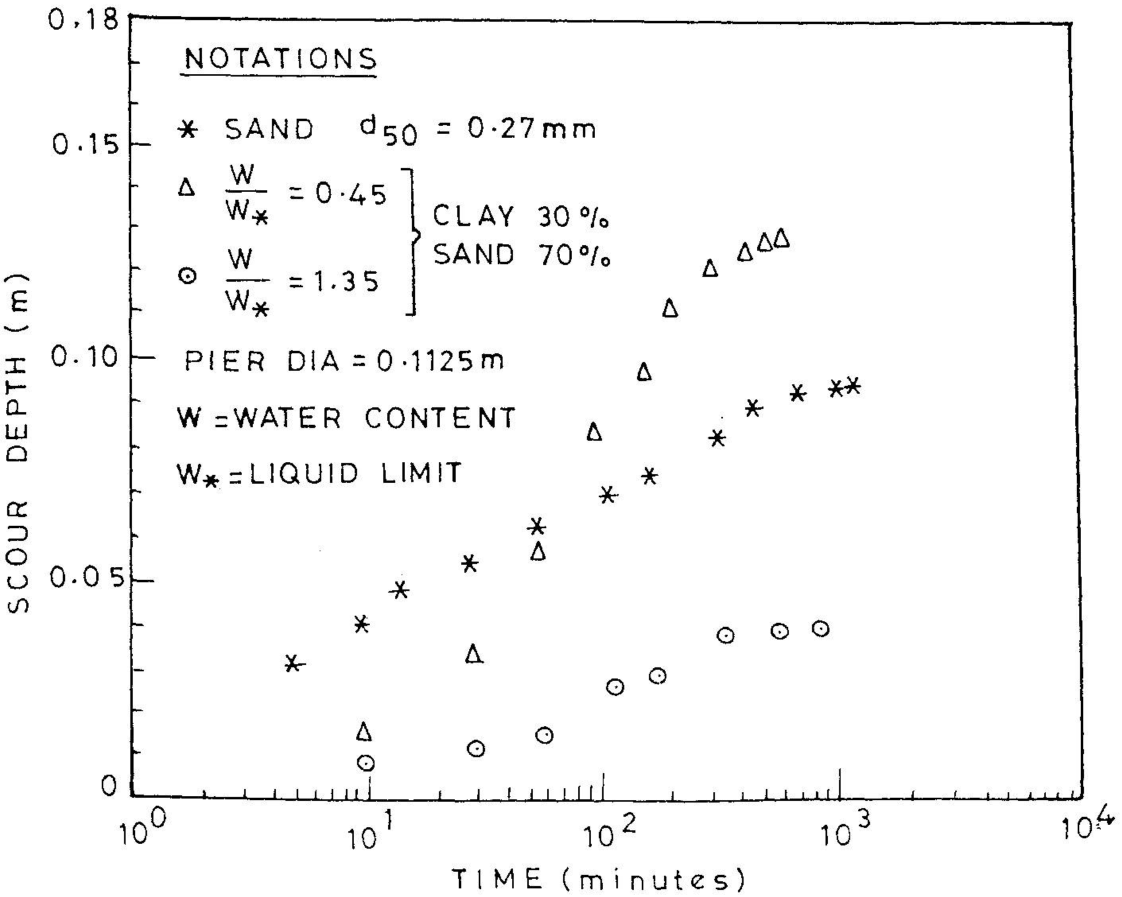

Fig. 4 Temporal variation of scour depth in cohesive soils

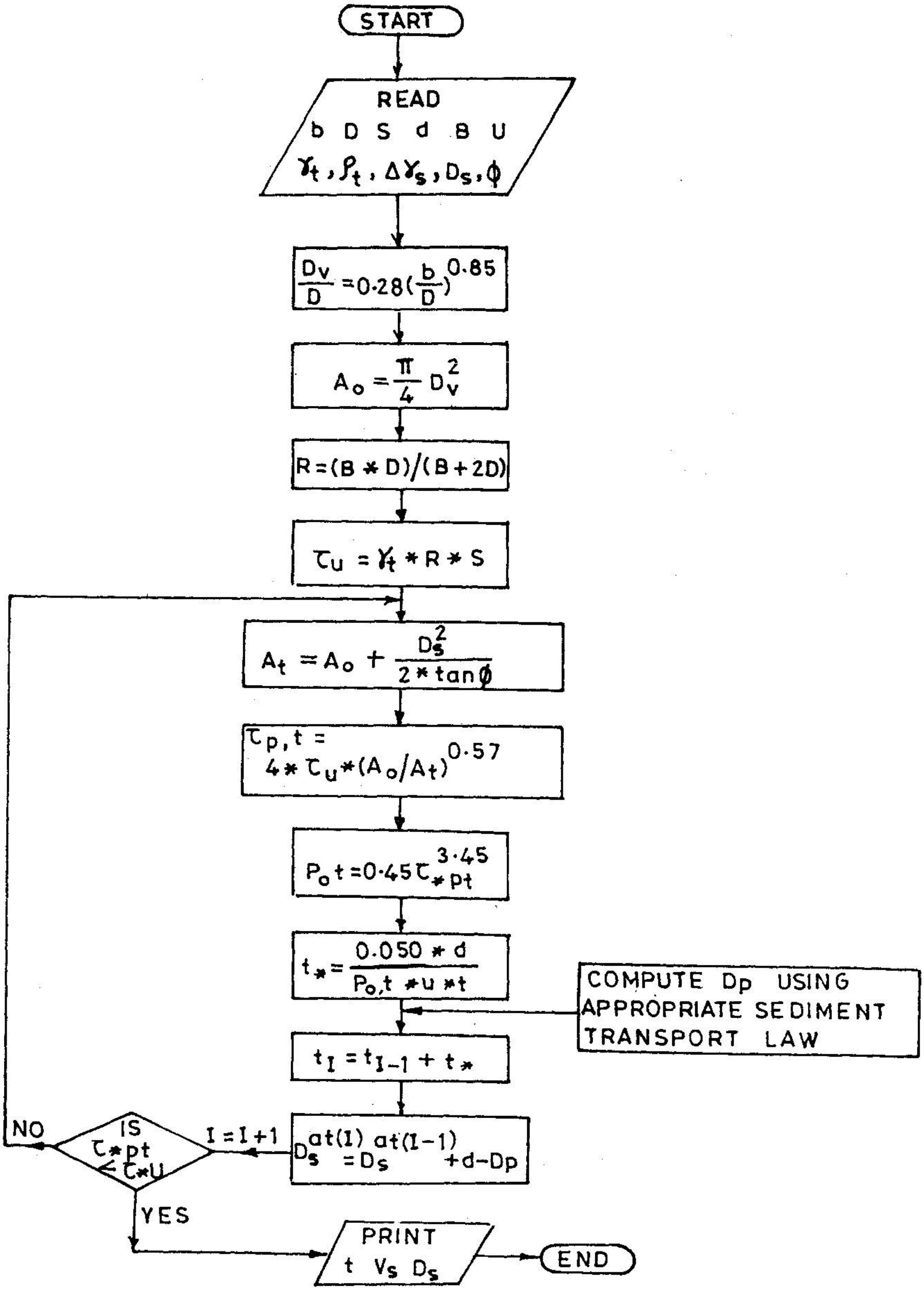

Fig.2 Algorithm for calculation of temporal variation of scour depth

Application of the Erodibility Index Method to Estimate Scour at Bridge Piers

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Dr. Annandale graduated with degrees in civil engineering from the University of Pretoria and the University of the Witwatersrand, South Africa. He specializes in river engineering, having authored and co-authored books and papers in the fields of sedimentation and scour. He developed the Erodibility Index Method that is used by consultants and government agencies to analyze scour at structures including bridge piers, dam foundations and pipeline crossings.

SUMMARY

The Erodibility Index Method (EIM) [2] is a new method that can be used to estimate the erosion threshold of a wide variety of earth materials, including cohesionless granular material, cohesive soils and rock. The EIM defines the erosion threshold for earth materials by relating the erosive power of water and a geo-mechanical index. The geo-mechanical index quantifies the relative ability of earth materials to resist erosion. It is a function of mass strength, block or particle size, inter-particle shear strength, dip and strike of rock, and its relative shape. The erosive power of water is expressed in terms of rate of energy dissipation. Existing methods to predict scour at bridge piers assume that the piers are founded on cohesionless granular material. These methods do not fully account for the resistance to scour offered by more complex earth materials, such as clay and rock, and can lead to over-prediction of scour. Prediction of bridge pier scour by using the EIM allows engineers to take account of the resistance to scour that is offered by materials as diverse as cohesionless granular material, cohesive soils and rock. This paper outlines the approach for using the EIM to calculate scour around bridge piers.

1 INTRODUCTION

Conventional bridge pier and abutment scour equations were developed in laboratory flumes using cohesionless granular soil (see e.g. [3] and [4]). Such equations do not account for resistance to scour offered by more complex earth materials, such as rock or clay. A generalized erosion threshold that is defined by Annandale's Erodibility Index Method [2] can be used to quantify the relative ability of any earth material (ranging from silt, through sand, gravel, clay and rock) to resist erosion. This paper outlines the application of this method to calculate ultimate scour depth at bridge piers.

2 CONCEPTUAL APPROACH

A comparison between scour depths calculated with conventional pier scour equations (e.g. [3]) and with the method proposed in this paper is conceptually shown in Figure 1. The scour depth calculated with a conventional pier scour equation is considered to be the maximum possible scour depth. Such estimates do not take account of resistance to scour offered by foundation material. Conceptually, the ultimate scour depth estimate that takes account of foundation material properties will be equal to or less than the maximum possible value.

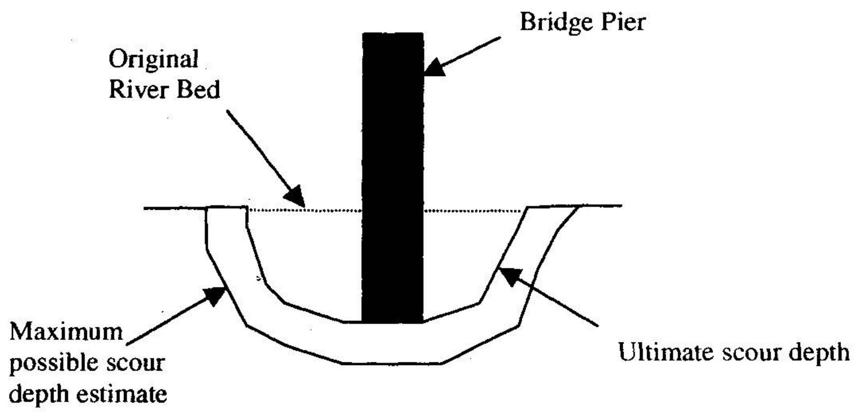

Figure 1. Relationship between the maximum possible scour depth estimated with conventional pier scour equations and the ultimate scour depth estimated with the method proposed in this paper.

The basis of the approach that is used to estimate ultimate scour depth by taking account of material properties, is explained in Figure 2. This figure shows a relationship between the erosive power of water and elevation below the original river bed. The two curves on the figure represent the **available** erosive power at the base of the pier as the scour hole increases in depth, and the erosive power that is **required** to cause scour at different elevations below the original river bed. The **available** erosive power at the base of a pier is a function of scour hole depth. The erosive power that is **required** to cause scour of the earth material is determined from the Erodibility Index Method. Normally the strength of earth material, especially rock, increases as a function of elevation below the original river bed. In such cases the erosive power that is **required** to scour the earth material also increases as a function of elevation below the original river bed. Research (see e.g. [6]) has shown that the erosive power of water at the base of a bridge pier decreases as the scour hole increases in depth . The maximum scour depth will occur at the cross-over elevation where the available erosive power is equal to the power that is required to cause scour. This approach has been successfully confirmed with prototype scale tests ([1] and [7]).

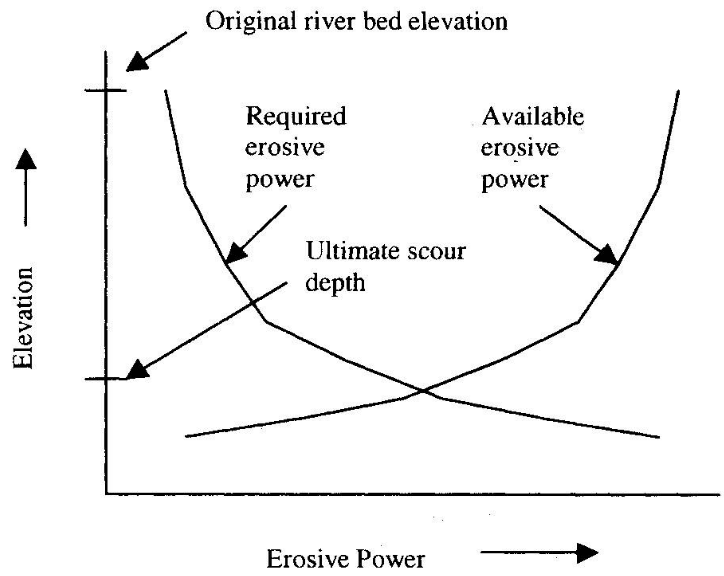

Figure 2. Calculation of ultimate scour depth by comparing the available erosive power at the base of a bridge pier and the erosive power that is required to scour earth material

3 RESISTANCE TO SCOUR

The erosion threshold shown in Figure 3 relates a geo-mechanical index (known as the Erodibility Index) and the erosive power of water [2]. This relationship holds for a wide variety of flow conditions and earth materials. The relative ability of earth material to resist erosion can be quantified by making use of the erosion threshold line in Figure 3.

The stream power required to scour earth material is determined by first indexing the earth material by means of the Erodibility Index. The Erodibility Index is a function of mass strength, block / particle size, shear strength, relative orientation and shape. Tables that can be used to quantify the Erodibility Index are presented in [2]. Once the earth material has been indexed a line is drawn vertically from the abscissa at the associated value on Figure 3 to meet the dotted erosion threshold line. From this location it is then drawn parallel to the abscissa to determine the required stream power value on the ordinate. This procedure is repeated for various elevations below the river bed. These pairs of values (elevation and **required** stream power) are plotted on a graph similar to Figure 2.

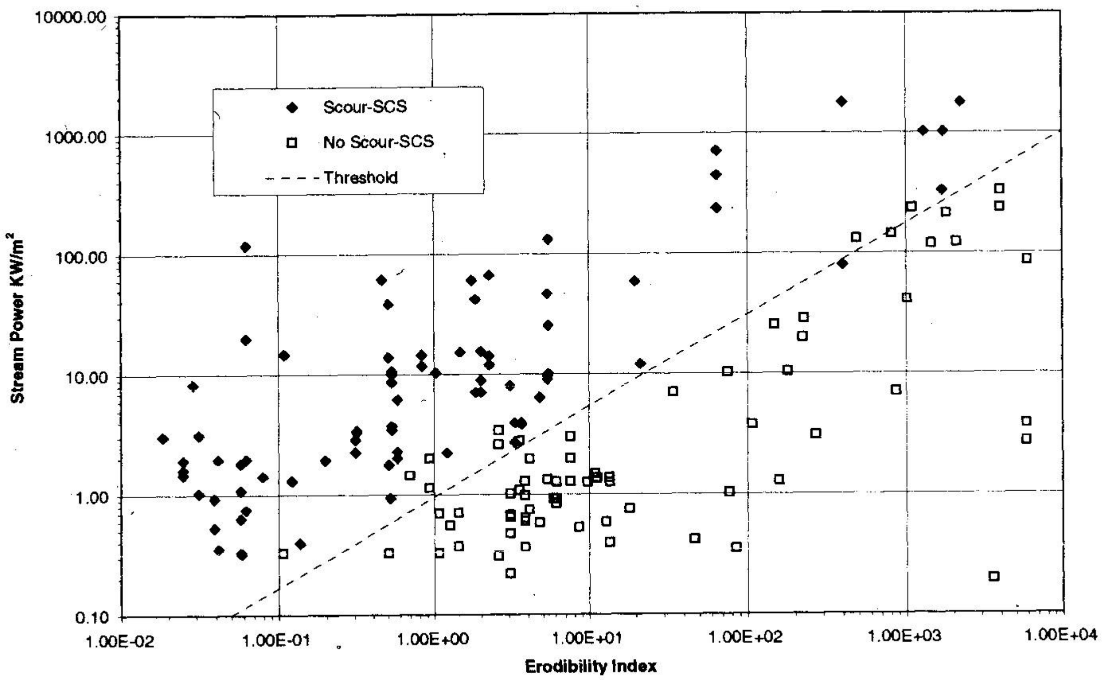

Figure 3. Erosion Threshold for a Variety of Earth Materials

4 EROSIVE POWER OF WATER

The change in the erosive power of water around a bridge pier as a function of elevation is determined by making use of graphs that were developed for this purpose (see e.g. [6]). These graphs relate dimensionless stream power at the base of a pier and dimensionless scour depth, following the general shape of the curve in Figure 4. The variable on the ordinate of this graph represents the ratio between the magnitude of the stream power at the base of the scour hole (P_p) and the stream power in the river section upstream of the bridge (P_r). The variable on the abscissa represents the ratio between variable scour depth (Y_s) and maximum scour depth (Y_{max}). When the latter variable is zero, it indicates the elevation of the original river bed before commencement of scour. When the same has a value of one, it represents the maximum scour depth. Values in between zero and one represent potential ultimate depths of scour.

The stream power is quantified by multiplying the various values of the ratio on the ordinate in Figure 4 with the stream power in the river upstream of the pier. The latter is quantified as the product of the unit weight of water (γ - kN/m^3), unit discharge (q – m^3/s/m) and energy slope (s), i.e.

$$\text{Power in river upstream of pier} = \gamma \cdot q \cdot s \qquad \text{(Equation 1)}$$

The scour depth on the abscissa is quantified by multiplying the various values of the ratio on the abscissa with the maximum scour depth estimate calculated with conventional pier scour equations (e.g. the equations in [3]). Once both the stream power and the potential scour depths in Figure 4 are quantified, the information is transferred to Figure 2. The latter curve represents the **available** stream power.

5 ULTIMATE SCOUR DEPTH

The ultimate scour depth is determined by plotting the required and available stream power as a function of elevation (Figure 2). The ultimate depth is located at the elevation where the **available** stream power is equal to the **required** stream power.

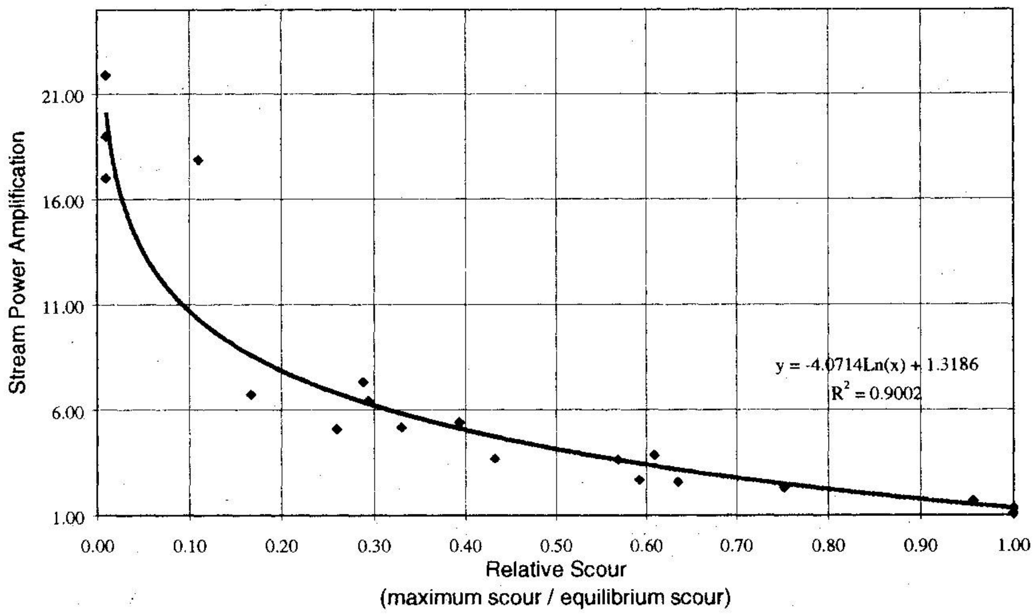

Figure 4. Stream power amplification at square bridge piers as a function of relative scour depth [6].

6 SUMMARY

A method that can be used to calculate ultimate scour depth by taking account of bed material properties is outlined in the paper. The method is based on comparison between the available erosive power and the erosive power that is required to scour a particular earth material. The relative magnitude of the erosive power of water around a bridge pier is determined by making use of dimensionless relationships that were developed for this purpose. The power that is required to scour the earth material is estimated at various elevations below the original river bed by making use of the Erodibility Index Method. The power that is available to cause scour is then compared to the power that is required to scour the earth material under consideration. This comparison is done at various assumed scour depths. The scour depth where the available erosive power is less than the power that is required to scour the earth material is considered to be the ultimate scour depth.

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MAXIMUM SCOUR DEPTHS AROUND A BRIDGE PIER IN SAND AND IN CLAY ARE EQUAL?

by

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SUMMARY

The maximum scour depth around bridge piers in sand is calculated using well established formulas based on experimental model calibrations. There are no such formulas for the maximum scour depth around bridge piers in clay. In practice and by conservatism the maximum scour depth in clay is taken to be equal to the maximum scour depth in sand. However no such evidence exists and common sense tells us that clays scour much more slowly than sand.

This paper presents flume test results of pier scour in clay. The piers are cylinders with diameters varying from 25 mm to 220 mm. The soils were a low plasticity porcelain clay, a medium plasticity armstone clay, a high plasticity bentonite clay and two uniform sands. The results show that the maximum depth of scour is the same in the sands and in the clays. However the rate of scour is drastically different. This shows that in clays a site specific scour rate analysis is necessary while it is not necessary in sands.

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INTRODUCTION

In US practice the maximum scour depth around a bridge pier in sand is calculated by using the "HEC 18" formula (Richardson and Davis, 1995)

$$z_{max} = 2z_o K_1 K_2 K_3 K_4 \left(\frac{D}{z_o}\right)^{0.65} F_o^{0.43} \quad (1)$$

where z_{max} is the maximum scour depth around the bridge pier z_o the depth of flow, K_1, K_2, K_3, K_3 are coefficients to take into account the shape of the pier, the angle between the direction of the flow and the direction of the pier, the stream bed topography, and the armoring effect, D is the pier diameter, and F_o is the froude number defined as $v/(gz_o)^{0.5}$ where v is the mean flow velocity and g the acceleration due to gravity.

For clays there is no such formula and by conservation equation 1 is used for clays. Yet it is well recognized that clays scour much more slowly than sands. In order to investigate if z_{max} is the same in sand and in clay, a series of flume experiments were conducted at Texas A&M University.

THE FLUMES AND THE SOILS

Two flumes were used. The first flume was 457mm wide and the second 1525 mm wide. The diameter of the cylindrical piers varied from 25 mm to 76 mm for the smaller flume and from 76 mm to 229 mm for the larger flume. A false bottom was constructed to allow space for placing the soil and then push the hollow pier in the soil. The water depth in the flumes varied from 0.16 m to 0.4 m and the water velocity from 0.2 m/s to 0.83 m/s.

The soils used were three clays and two sands. The first clay was a low plasticity clay used to make porcelain craftware. The second clay was a medium plasticity clay called armstone also used for pottery. The third clay was a high plasticity clay with a 30% content of bentonite. The first sand was a medium uniform silica sand with a particle diameter D_{50} equal to 0.6mm and 5% passing sieve no. 200 (0.076mm). The second sand was a fine uniform silica sand with a particle diameter equal to 0.14 mm and 0% passing sieve no 200 (0.076 mm) The properties of the soils tested are summarized in Table 1 and the grain size curves are in Figure 1.

THE FLUME TESTS

A total of 43 tests were performed. 6 in the larger flume with porcelain clay and 37 in the smaller flume. Of those 37, 4 were performed with the medium sand, 3 with the fine sand, 2 with the bentonite clay, 4 with the armstone clay, and 24 with the porcelain clay. The clay was prepared in blocks 0.3m x 0.15m x 0.15m in size. The clay blocks were placed side by side, compacted with a metal plate to remove air voids and smoothed out with a hand trowel to obtain a smooth surface. The sand was dumped in a loose state into the soil area around the pier.

The water flow was initiated and measurements were made to record the velocity and the depth of scour. The velocity profile was recorded with an acoustic doppler velocimeter (ADV) and the depth of scour with a point gage mounted on an instrument carriage.

RESULTS OF THE TESTS

The detailed results are described in Gudavalli (1997). The first observation is that the scour hole originated on the front side of the piers at a 45 degree angle and that the scour hole developed on the side and mostly behind the pier with very little scour if any in front of the pier. Therefore, in clays, it may not be wise to place monitoring devices in front of the pier.

The result of a test consists of the scour depth vs. time curve for a given velocity, water depth, pier size and soil type (Figures 2, 3, and 4). As can be seen on Figure 4, even after 200 hours (8.33 days) of flow the scour depth was still increasing. In order to obtain the maximum depth of scour the experimental data was fitted with a hyperbola:

$$z = \frac{t}{\dfrac{1}{\dot{z}_i} + \dfrac{t}{z_{max}}} \tag{2}$$

where $\dot{z}_i$ is the initial rate of scour and z_{max} is the maximum depth of scour. In the case of Figure 4 (experiment #41) $\dot{z}_i$ was 1.67mm/hour and z_{max} was 208 mm. Note that the hyperbola fits the data very well. For all the experiments z_{max} was calculated in such a way; the z_{max} values are shown in Table 2.

Figure 5 shows the comparison between a fine sand (experiment #32) and a low plasticity clay (experiment #22) for very similar conditions of pier diameter, water depth and water velocity. For the fine sand z_{max} is 41 mm compared to 48.7 mm for the clay; however the initial rate of scour $\dot{z}_i$ is 840 mm/hr for the sand compared to only 0.95 mm/hr for the clay. This shows that while the maximum depth of scour may be the same for sand and clay the rate of scour in clay may be 1000 times less than in sand.

Figure 6 is a plot of z_{max} vs the pier Reynold's number R_e defined as $R_e = \frac{VD}{\upsilon}$ where υ is the kinematic viscosity of the water $(10^{-6} m^2/s)$. On Figure 6 some of the early experiments where problems occurred are omitted (experiments # 5, 10, and 14). The figure indicates that the maximum depth of scour is the same for clay and for sand and the regression line gives:

$$z_{max}(mm) = 0.18 R_e^{0.635} \tag{3}$$

Note that the HEC-18 equation also fits the data quiet well (Table 2).

CONCLUSIONS

The 43 flume tests performed in this study tend to show that the maximum depth of scour in clay occurs behind the pier, not in front of it, and that the maximum depth of scour is the same in sand and in clay. However the rate of scour is drastically different. Therefore in clay it is necessary to have a method which gives the progression of the scour depth as a function of time because, at a very slow scour rate, the maximum depth of scour may not be reached during the design-life of the bridge. Such a method has been developed at Texas A&M University for a given hydrograph.

ACKNOWLEDGEMENTS

This project was sponsored by the Texas Department of Transportation where the following people were very helpful: Ms. Kim Culp, Ms.Melinda Luna, Mr. Tony Schnieder, Mr. Peter Smith, Mr. Jay Vose.

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Table 1 - Soil Properties

No.	Property	Porcelain	Armstone	Bentonite	Medium Sand	Fine Sand
1	Liquid Limit, %	34.40	44.20	67.00	-	-
2	Plastic Limit, %	20.25	18.39	27.22	-	-
3	Plasticity Index, %	14.15	25.81	39.78	-	-
4	Specific Gravity	2.61	2.59	2.55	-	-
5	Water Content, %	28.51	26.18	39.28	-	-
6	Mean Diameter D_{50}, mm	0.0062	0.0032	0.0006?	0.60	0.14
7	Sand Content, %	0.00	25.00	0.00	95.00	100.00
8	Silt Content, %	75.00	30.00	35.00	5.00	0
9	Clay Content, %	25.00	45.00	65.00	0	0
10	Shear Strength, kPa (lab. vane)	12.51	16.57	39.56	-	-
11	CEC, (meq/100 g)	8.30	10.00	16.10	-	-
12	SAR	5.00	2.00	21.00	-	-
13	PH	6.00	5.20	8.50	-	-
14	Electrical Conductivity, (mmhos/cm)	1.20	1.10	1.10	-	-
15	Unit Weight, (kN/m^3)	18.0	17.89	17.45	-	-
16	Relative Density				loose	loose

Table 2 - Flume Test Results

Expt No.	Flume Size*	Soil Type**	Z_o (m)	D (mm)	V (m/s)	Re	Fr	Z_o/D	t (hrs)	Z_{max} (mm)	Z_{max} (mm)	
											HYPER	HEC-18
1	S	1	0.4	25	0.47	11750	0.24	16	95	75	98	71.1
2	S	1	0.4	25	0.40	10000	0.20	16	72.3	50	63.5	66.3
3	S	1	0.4	25	0.608	15200	0.31	16	46.9	77	122	79.4
4	S	1	0.4	25	0.317	7925	0.16	16	57.4	40	55	60
5	S	1	0.4	25	0.204	5100	0.1	16	37	8	11	49.6
6	S	1	0.4	25	0.4	10000	0.2	16	92.25	53	65.5	66.3
7	S	1	0.4	25	0.83	20750	0.42	16	16.17	44.8	109	90.8
8	S	1	0.4	75	0.608	45600	0.31	5.33	68.92	104	170	162.2
9	S	1	0.4	75	0.319	23925	0.16	5.33	43.4	58	76.9	122.9
10	S	1	0.4	75	0.204	15300	0.1	5.33	37	22	31.5	101.4
11	S	1	0.4	75	0.4	30000	0.2	5.33	60.25	78	142.8	135.4
12	S	1	0.4	75	0.48	36000	0.24	5.33	63	99	147	146.5
13	S	1	0.4	75	0.39	29250	0.2	5.33	131	95	161.3	134
14	S	1	0.4	75	0.318	23850	0.16	5.33	73	39	49.1	122.7
15	S	1	0.4	75	0.48	36000	0.24	5.33	142.5	116	178.6	146.5
16	S	1	0.4	75	0.83	62250	0.42	5.33	16.17	58	180	185.4
17	S	1	0.16	25	0.266	6650	0.21	6.4	99	27	51.5	46.2
18	S	1	0.16	75	0.266	19950	0.21	2.13	99	44	79.7	94.3
19	S	1	0.16	25	0.348	8700	0.28	6.4	152	53	67.3	54.3
20	S	1	0.16	75	0.348	26100	0.28	2.13	152	74	103	110.9
21	S	1	0.4	25	0.47	11750	0.24	16	54.8	60	-	-
22	S	1	0.4	25	0.315	7875	0.16	16	62.24	26	48.7	59.8
23	S	1	0.4	25	0.41	10250	0.21	16	93.25	48.06	81.8	67.0
24	S	1	0.4	25	0.41	10250	0.21	16	114	48.2	107	67.0

*S = Small Flume (0.46 m wide)
L = Large Flume (1.52 m wide)
**1 = Low Plasticity Porcelain Clay
2 = Medium Plasticity Armstone Clay
3 = High Plasticity Bentonite Clay
4 = Medium Uniform Sand
5 = Fine Uniform Sand

Table 2 - Flume Test Results (Continued)

Expt No.	Flume Size*	Soil Type**	Z_o (m)	D (mm)	V (m/s)	Re	Fr	Z_o/D	t (hrs)	Z_{max} (mm)	Z_{max} (mm)	
											HYPER	HEC-18
25	S	3	0.4	25	0.32	8000	0.16	16	75	55	64.5	60
26	S	3	0.4	25	0.39	9750	0.2	16	37.66	50	59.3	65.5
27	S	4	0.17	50	0.243	12150	0.19	3.4	-	67	-	74.8
28	S	4	0.17	50	0.245	12250	0.19	3.4	-	48	-	75.1
29	S	4	0.32	50	0.348	17400	0.2	6.4	-	85	-	95.1
30	S	4	0.33	50	0.448	22400	0.25	6.6	-	115	-	106.4
31	S	5	0.4	25	0.242	6050	0.12	16	9.23	35	35.8	53.4
32	S	5	0.4	25	0.282	7050	0.14	16	4.87	41	-	57.1
33	S	5	0.4	75	0.212	15900	0.11	5.33	6	70	-	103.1
34	S	1	0.4	25	0.3	7500	0.15	16	114.42	20	28	59.4
35	S	1	0.4	75	0.3	22250	0.15	5.33	114.42	54	106	121.4
36	S	1	0.4	25	0.4	10000	0.2	16	117.4	57	73.5	66.3
37	S	1	0.4	75	0.4	30000	0.2	5.33	117.4	95	133	135.4
38	L	1	0.4	75	0.37	27750	0.19	5.33	154.25	84	156	130.9
39	L	1	0.3	150	0.3	45000	0.17	2	182.5	128	250	202.2
40	L	1	0.25	150	0.39	58500	0.25	1.67	175.5	75	190	176.3
41	L	1	0.3	210	0.316	66360	0.18	1.43	210.66	130	208	230.5
42	L	1	0.3	210	0.404	84850	0.24	1.43	104.33	96	225	255.5
43	L	1	0.3	210	0.317	66570	0.18	1.43	146.67	111	187.5	229.6

*S = Small Flume (0.46 m wide)
L = Large Flume (1.52 m wide)
**1 = Low Plasticity Porcelain Clay
2 = Medium Plasticity Armstone Clay
3 = High Plasticity Bentonite Clay
4 = Medium Uniform Sand
5 = Fine Uniform Sand

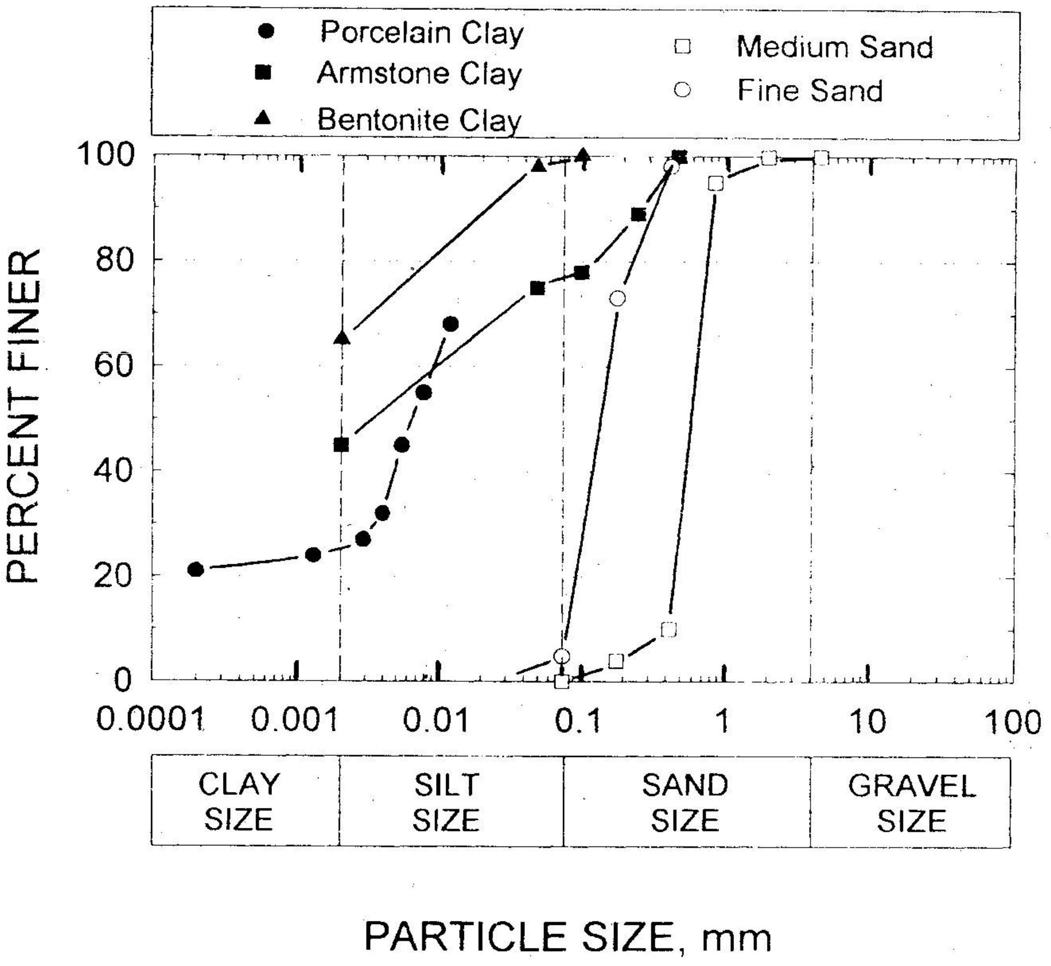

Fig. 1 Particle Size Distributions

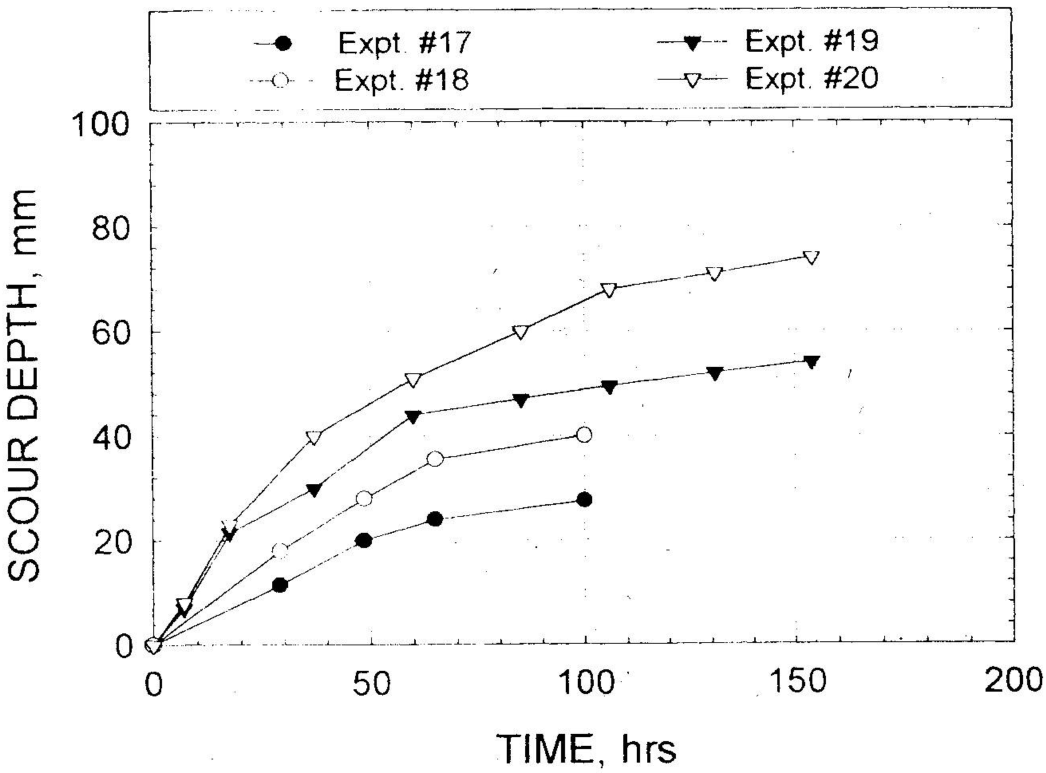

Fig. 2 Scour Depth vs. Time in a Clay

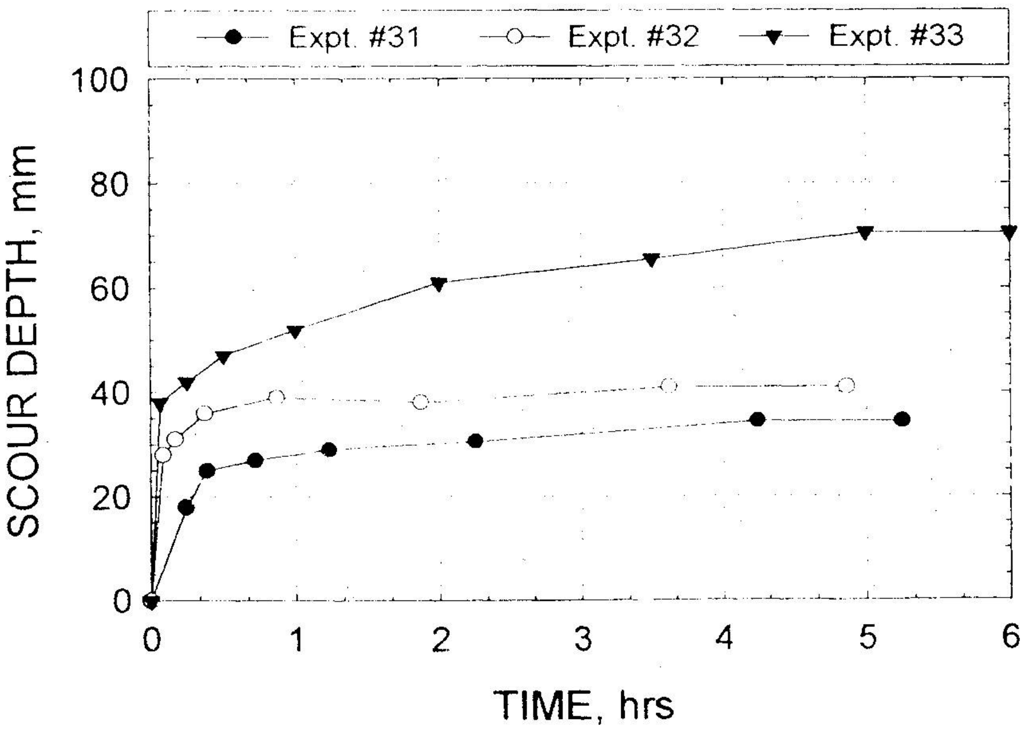

Fig. 3 Sour Depth vs Time in a Sand

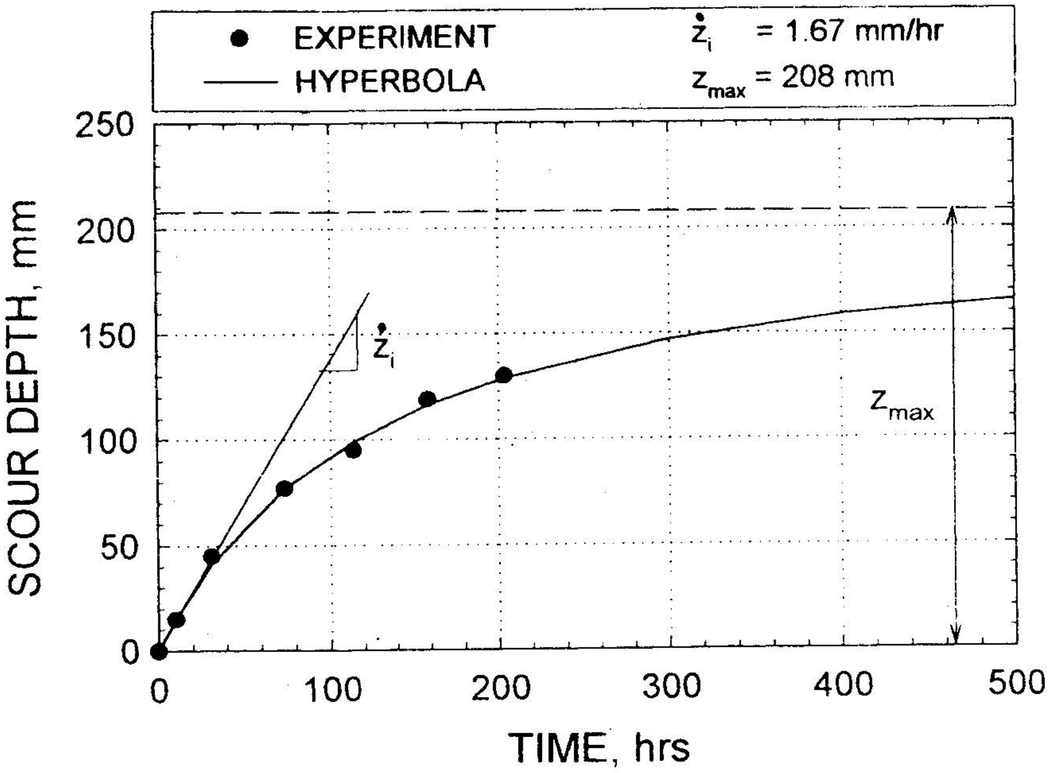

Fig. 4 Hyperbolic Extrapolation for Experiment #41

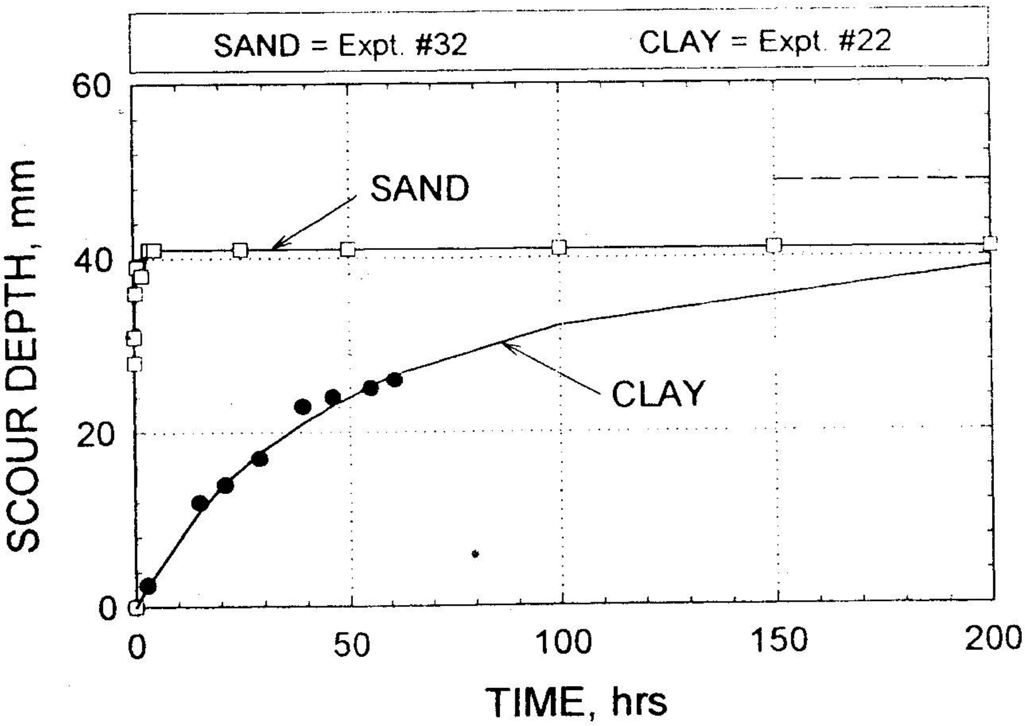

Fig. 5 Comparison Between Exp #32 in Sand and Exp # 22 in Clay

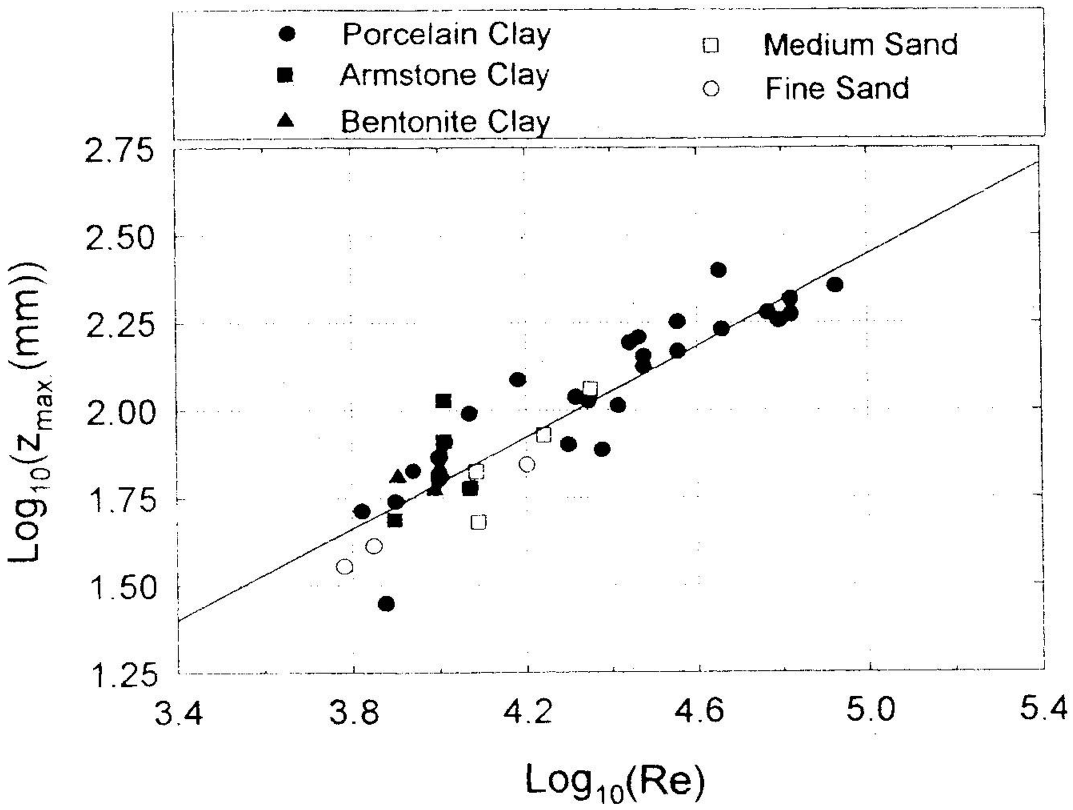

Fig 6 Maximum Scour Depth vs Reynolds Number for two Sands and three Clays

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MONITORING OF LATERAL EARTH PRESSURES ON WELL FOUNDATIONS THROUGH INSTRUMENTATION

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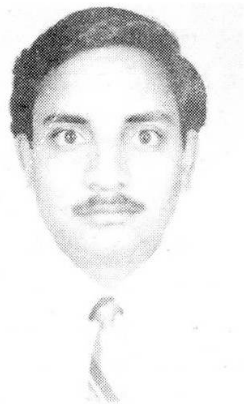

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Summary

A major project involving extensive instrumentation of a large well foundation has been carried out at the recently completed Ganga Bridge at Varanasi. The instrumentation is intended for long-term monitoring of the design parameters for such large foundation wells. One of the wells has been instrumented with earth pressure cells, inclinometer and vibrating wire rebar load gauges. The earth pressure data obtained over a period of five years since the well was sunk to its founding level, along with the inferences drawn therefrom are presented in the paper.

1.0 INTRODUCTION

In the context of the currently acute need for scientific monitoring of the health of major bridges and for creating a reliable data base for their efficient management, the construction of the Ganga Bridge at Varanasi presented a unique opportunity to comprehensively instrument both its superstructure and substructure and monitor its performance from inception. This opportunity was utilised to plan and execute a major project aimed at long-term performance monitoring of the bridge through instrumentation.

The project involved, interalia, extensive instrumentation of the major components of the bridge viz. the superstructure, the pier, the pier head and the well foundation. A large number of structural parameters such as strains, deflections, slopes, tilts, thermal gradients, earth pressures etc. were continuously monitored during the construction of the bridge and would continue to be monitored for a few years during its service life. The data so obtained is expected to shed light on its short-term and long-term behaviour.

Of particular interest in this project was the instrumentation of one of the foundation wells of the bridge, since field data relating to the design parameters of such large well foundations is solely lacking. The instrumentation scheme for foundation wells including the parameters to be monitored and the corresponding instrumentation techniques used were described in earlier papers [1,2,3]. The details of installation of the sensors, the devices used for protecting the sensors and their cables during concreting and sinking operations and the data obtained during certain intermediate stages of construction were also described therein. The present paper, while touching briefly upon some of these aspects, presents an analysis of the data relating to the earth pressures on the well recorded since the well was sunk to its founding level.

2.0 SCOPE OF THE PROJECT

The general arrangement of the bridge is shown in Fig.1. The bridge deck is a twin-cell box girder with a deck slab supporting a 19.6m wide, 4-lane carriageway. The box girder cantilevers to 65.75m on either side of the pier in the main spans. The foundations for the piers of the main spans are 65m deep reinforced concrete wells with inside and outside diameters of 8m and 13m respectively. The region of the bridge marked for instrumentation at Pier P7 is also highlighted in Fig.1.

3.0 INSTRUMENTATION OF THE FOUNDATION WELL P7

The most important parameters which are critical to the structural design and stability of foundation wells and which are amenable to direct measurements are the lateral earth pressures at the soil-well interface, the tilt and shift of the well and the actual strains within the body of the well. Current design procedures for well foundations are predicated upon a number of assumptions relating to these parameters, particularly for large and deep wells as in the present case. A knowledge of the actual values of these parameters would throw considerable light on the validity of the design assumptions and on the true structural behaviour of the well and the level of safety inherent in the current design procedures.

The design and performance parameters of foundation wells which thus call for in-situ measurement and monitoring, together with their corresponding instrumentation techniques are summarised in Table 1. These techniques imply that a well would have to be instrumented at several levels throughout its height. These ideas formed the basis of the instrumentation of well P7.

TABLE 1. Parameters Monitored and Corresponding Instrumentation Techniques used in the Well P7

Parameter	Technique
- Soil Pressure on the well	- Vibrating Wire [VW] Earth Pressure Cell
- Strain in concrete/reinforcement	- VW Rebar Load Gauge [RLG]
- Inclination of the well	- Inclinometer System

Fig.2 shows the three levels at which the instruments and sensors were installed in the well while Fig.3 shows schematically the typical layout of the instruments at a level. A summary of the final status of instrumentation of the well P7 is provided in Table 2. The instrument readings were recorded at several stages during the construction and sinking of the well. Evidently, at each stage a different set of conditions obtained with respect to the height of the well constructed, the extent of sinking, the position of the instruments vis-a-vis the water and bed levels. These data for some of the intermediate positions of the well were obtained while the well was still under construction and were presented earlier [3]. The data obtained after the well reached its founding level are presented and analysed in this paper.

TABLE 2. Final Positions of Instruments in Well P7

Instrument Level [IL]	RL [m]	Height above cutting edge [m]	Depth below bed level [m]	Type and Number of instruments installed
1. IL-1	19.15	24.15	30.15	- Inclinometer Casings [2 Nos.] - Earth Pressure Cells [6 Nos.] - 12 mm Rebar Load Gauges [6 Nos.]
2. IL-2	29.15	34.15	20.15	Same as IL-1
3. IL-3	39.15	44.15	10.15	Same as IL-1

4.0 MEASUREMENT OF EARTH PRESSURE ON THE WELL

From Fig.3, it is evident that the lateral earth pressure on the well is being measured with the help of VW pressure cells installed along the external face of the well as shown in the figure. At each of the three instrumented levels in the well, six pressure cells were installed symmetrically, at intervals of 60 degrees, starting from the longitudinal bridge axis. These positions from 0 to 300 degrees are termed IP 1 to IP 6, respectively. The earth pressures were recorded on different dates, starting from the day on which the well was finally sunk to its founding level, viz. from 7 January 1994. Fig.4 shows the distribution of lateral eath pressure around the well at the lowermost instrument level viz. IL 1. Fig. 5 shows the lateral earth pressure history at IL 1. Fig.6 shows the progressive variation in lateral earth pressure distribution along the height of the well at three instrument positions viz. IP 1, IP3 and IP5 for the same dates as in Fig. 4. Figs. 4, 5 and 6 are typical for all the three levels of instrumentation and instrument positions.

5.0 INFERENCES DERIVED FROM FIELD DATA

A massive amount of data has been obtained from the instruments installed in the well P7. This data is currently being analysed and would be eventually compared with the corresponding analytical results. However, the broad inferences that can be derived from the field data with reference to Figs. 4,5 and 6 are given below :

(i) In general, at all the three instrument levels, the pressures around the well obtained shortly after the well reached its founding level have shown a continuous decrease over a six month period, the maximum decrease of pressure at any one instrument position during the six month period being about 8 to 10 %

(ii) The pressure distribution around the well is quite uniform at the lowermost instrument level IL 1. (Fig.4). The pressure distributions at levels IL 2 and IL 3 however, show a marked deviation from uniformity, with a sharp increase of pressure values at one or two instrument positions. The deviation is particularly sharp at instrument position IP 1 and is much greater at level IL 2 than at level IL 3 . The increased pressures at IP 1 could possibly be attributed to the surcharge pressure exerted by the land mass which rises sharply a few metres away from the the edge of the water, along the river bank adjoining the well P7.

(iii) Fig. 5 indicates the variation in earth pressure right from the time of installation of the pressure cells, through a three year period after the well was constructed. The pressure history of Fig. 5 is typical of all the levels and indicates that the earth pressure on the well from the time it reached its founding level has been almost uniform and has stabilised over the years.

(iv) The earth pressure distribution along the height at all the six instrument positions, remained virutally unchanged throughout the period of construction of the pier.

(v) The pressure distribution along the height of the well at each of the six instrument positions viz. IP-1 to IP-6, on the same dates as in Fig.4 shows an almost linear variation of pressure along the height, except at IP-1 and IP-6. (Fig.6.)

6.0 CONCLUSIONS

For the first time, a large foundation well of a major bridge has been instrumented and its structural parameters monitored from the construction stage onwards. The work involved was indeed voluminous, with a very large number of activities to be performed to a strict time schedule e.g. planning of the scheme, procurement of equipment, installation of instruments at site etc. In spite of the arduous and hazardous site work involved it was gratifying to be able to implement the programme of instrumenting one of the largest well foundation for a bridge. The site data together with the collateral analytical work are expected to provide a basis for a more realistic assesment of the design parameters for such foundation wells.

7.0 ACKNOWLEDGEMENTS

The project would not have seen the light of the day without the sponsorship and financial assisstance of the Roads Wing of the Ministry of Surface Transport [MOST], Govt. of India, New Delhi. The valuable input and guidance in planning and implementation of the scheme received from MOST is also gratefully acknowledged.

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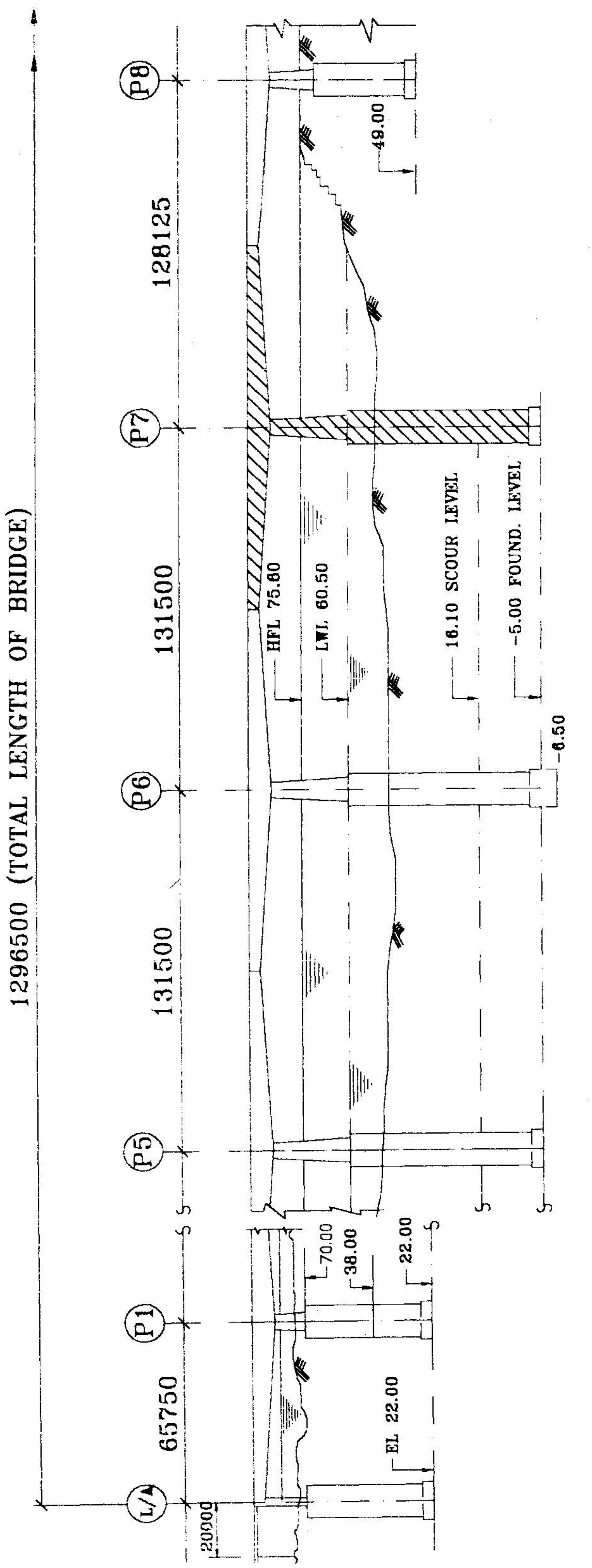


FIG.1 GANGA BRIDGE AT VARANASI: GENERAL ARRANGEMENT

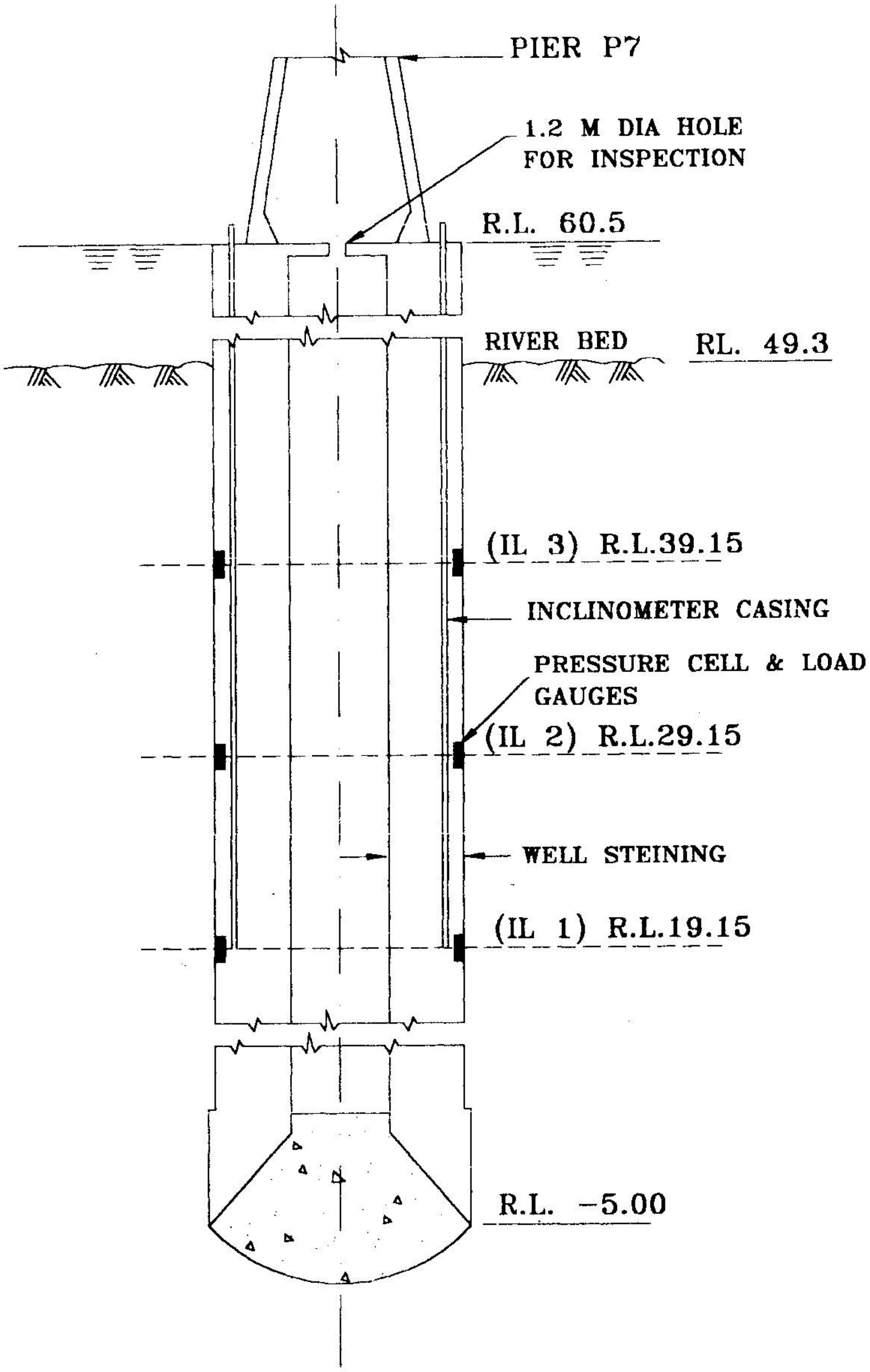


FIG.2 INSTRUMENTATION LEVELS (IL) OF WELL P7 (SCHEMATIC)

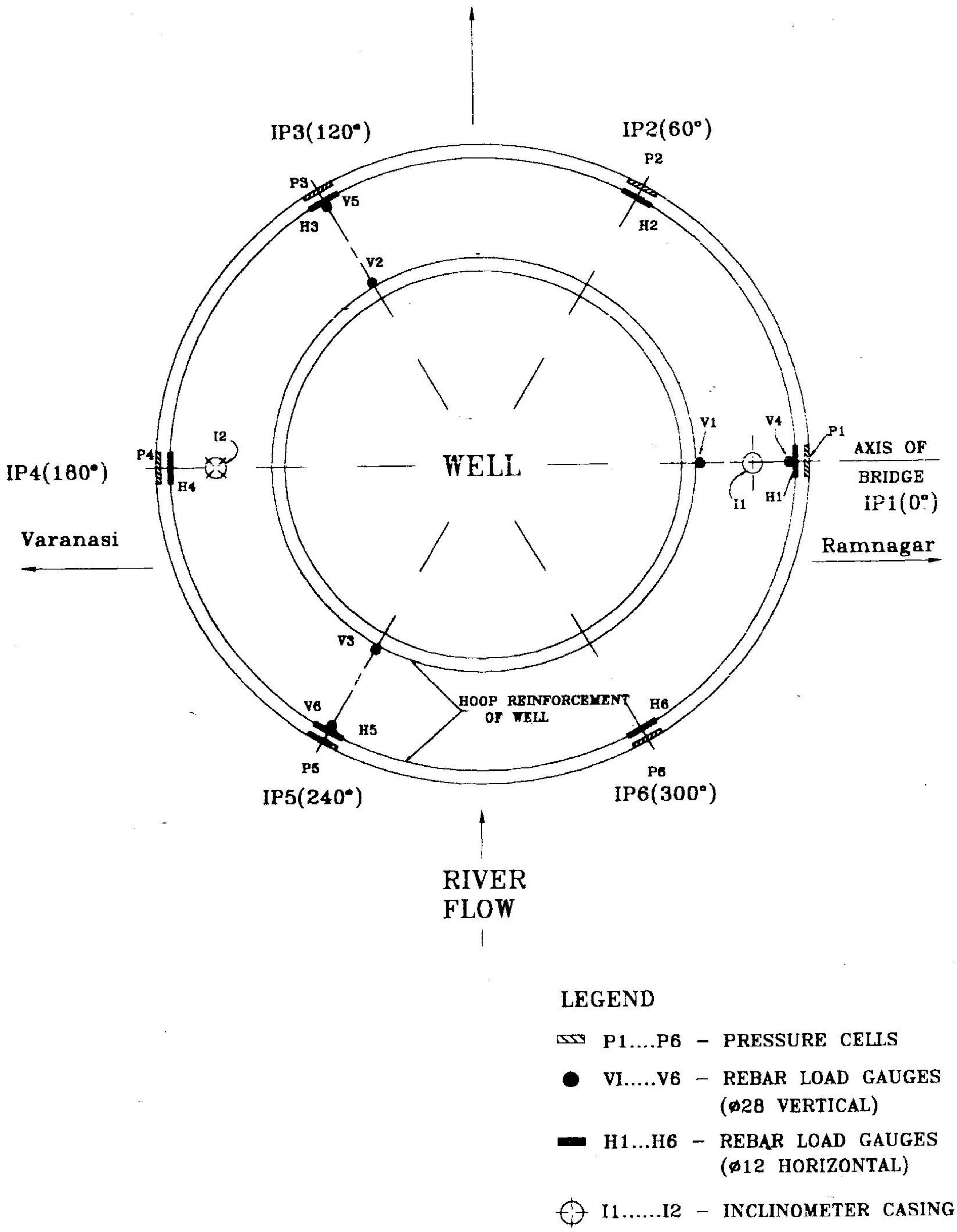


FIG.3 TYPICAL INSTRUMENTATION SCHEME AT A LEVEL IN WELL P7

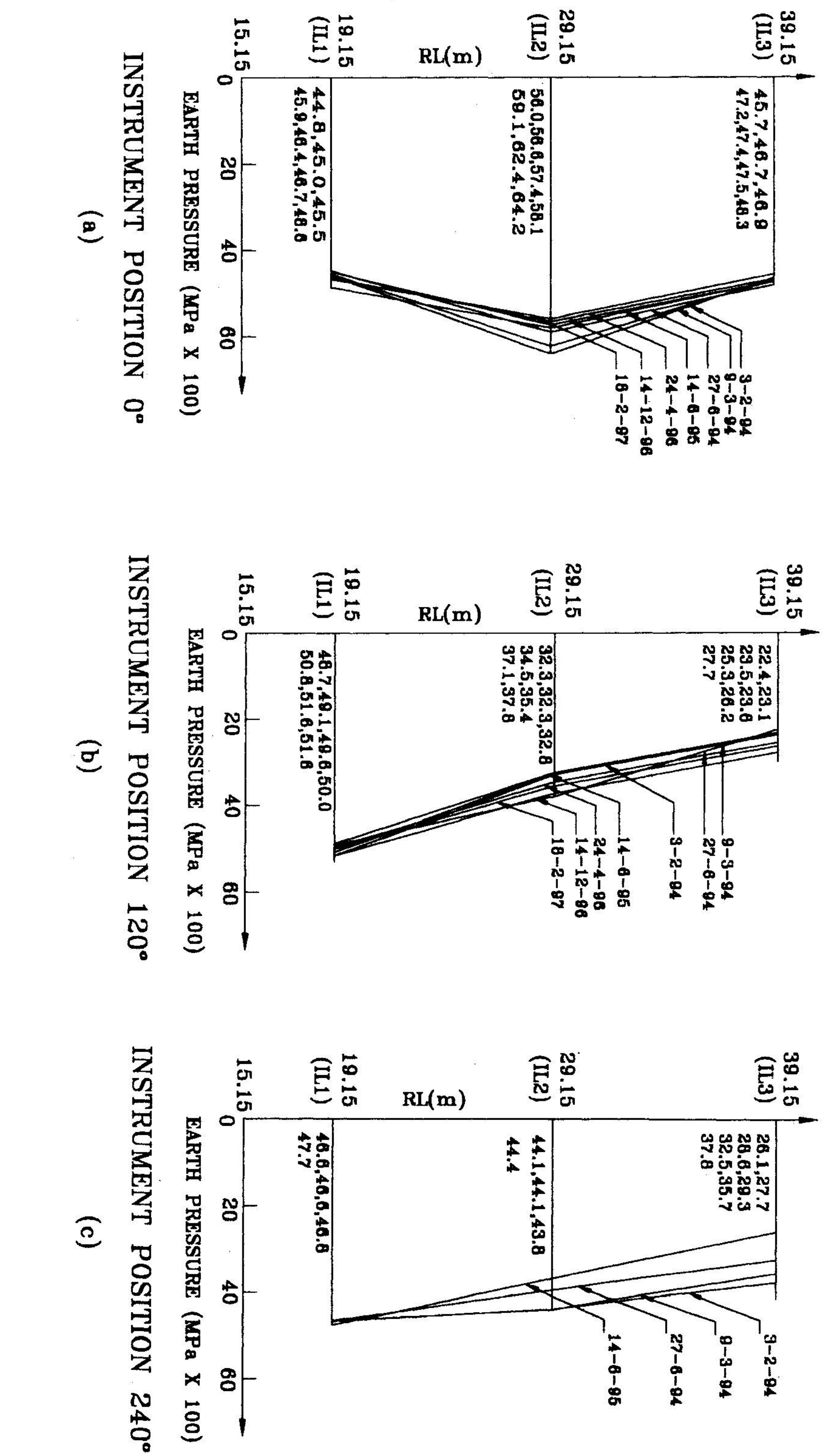
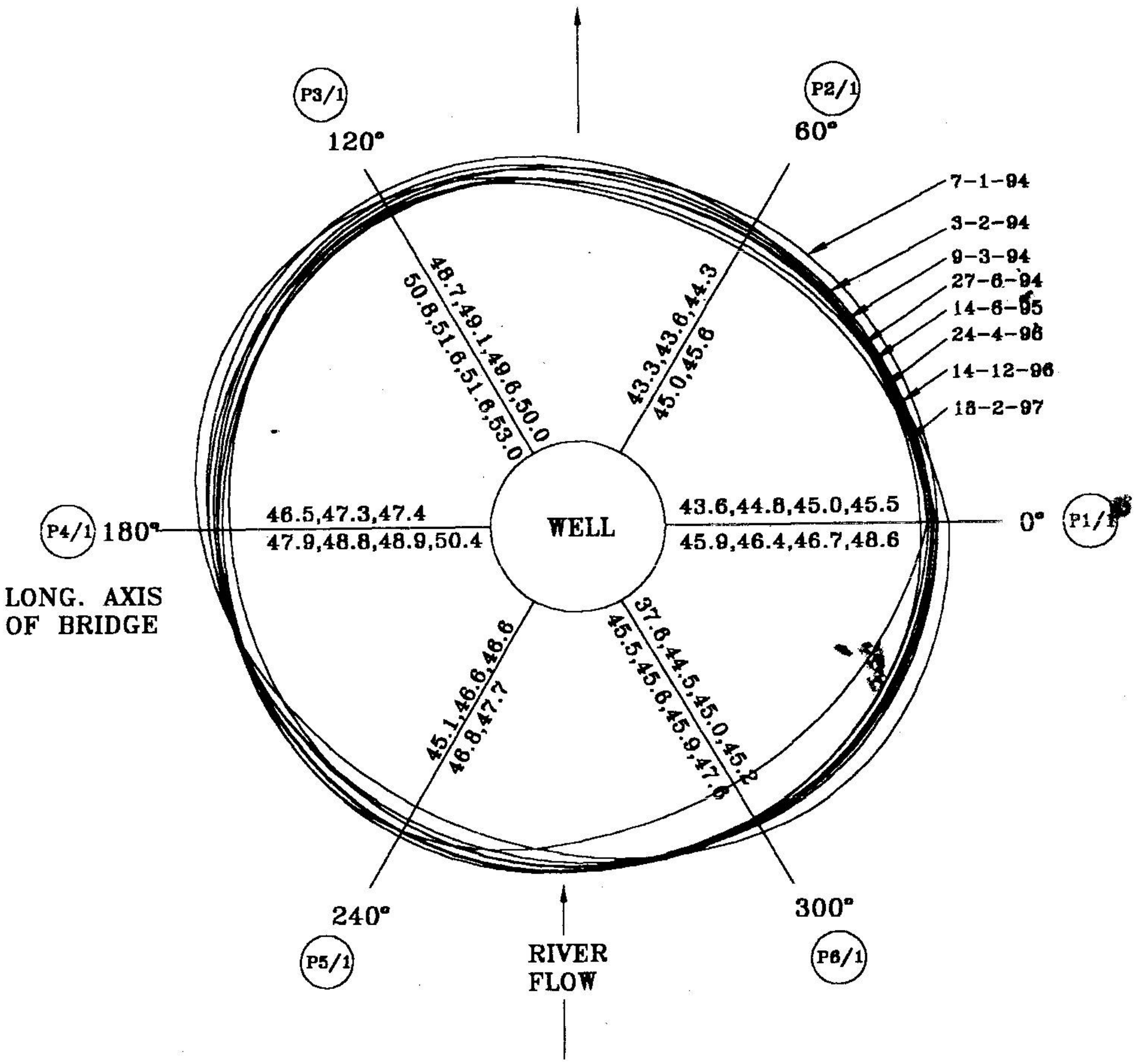


FIG.6 PROGRESSIVE VARIATION IN LATERAL EARTH PRESSURE DISTRIBUTION ALONG THE HEIGHT OF WELL P7



NOTES: 1.EARTH PRESSURE VALUE ARE IN MPaX100

2. P6/1.... PRESSURE CELL NO. 6 AT IL-1

FIG.4 PROGRESSIVE CHANGE IN LATERAL EARTH PRESSURE DISTRIBUTION AROUND WELL P7 AT IL-1 AFTER REACHING FOUNDING LEVEL(TYP.)

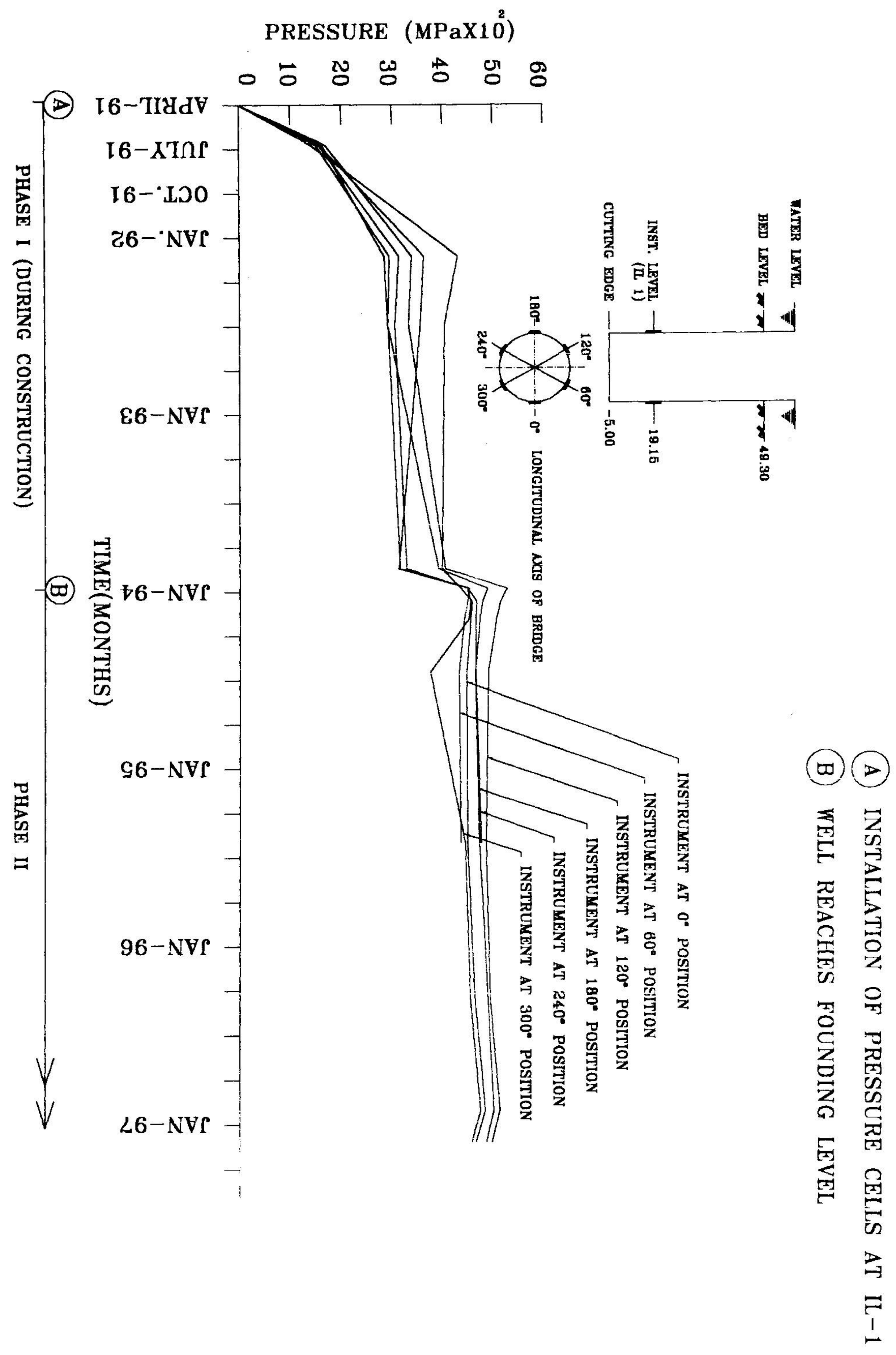


FIG.5 LATERAL EARTH PRESSURE HISTORY AT IL-1 (TYP.)

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DEVELOPMENT AND TESTING OF INSTRUMENTATION FOR MONITORING SCOUR AT BRIDGES

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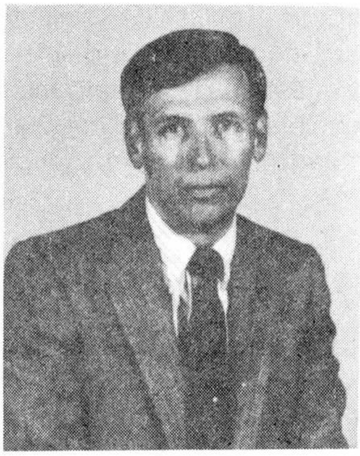

Peter Lagasse obtained his engineering degrees from U.S. Military Academy (BS), University of California, Berkeley (MS), and Colorado State University (PhD). After 20 years with the U.S. Army Corps of Engineers, he has been involved for the last 18 years in River Engineering Consulting, including stream stability and scour at bridges.

SUMMARY

This paper summarizes the results of research sponsored by the U.S. National Research Council, Transportation Research Board, to develop, test, and evaluate fixed instrumentation that would be both technically and economically feasible for use in monitoring maximum scour depth at bridge piers and abutments. A variety of scour measuring and monitoring methods were tested in the laboratory and in the field, including sounding rods, driven rod devices, sonic depth finders (fathometers), and buried devices. Two fixed instrument systems, a low-cost fathometer and a magnetic sliding collar device using a driven rod approach are described in detail. Cooperative efforts with state highway agencies proved that both systems can be installed with equipment and technical skills normally available to District level highway agency maintenance and inspection personnel. Installation, operation, and fabrication manuals for the low-cost sonic instrument system and magnetic sliding collar devices are referenced.

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1. INTRODUCTION

There are many scour susceptible bridges on spread footings or shallow piles in the United States and a large number of bridges with unknown foundation conditions [1]. With limited funds available, these bridges cannot all be replaced or repaired. Therefore, they must be monitored and inspected following high flows. During a flood, scour is generally not visible and during the falling stage of a flood, scour holes generally fill in. Visual monitoring during a flood and inspection after a flood cannot fully determine that a bridge is safe. A reliable device to measure or monitor maximum scour would resolve this uncertainty.

Recognizing this need, the Transportation Research Board (TRB) under the National Cooperative Highway Research Program (NCHRP) initiated NCHRP Project 21-3 "Instrumentation for Measuring Scour at Bridge Piers and Abutments" in 1989. The basic objective of this research was to develop, test, and evaluate fixed instrumentation that would be both technically and economically feasible for use in measuring or monitoring maximum scour depth at bridge piers and abutments [2]. The scour measuring or monitoring device(s) must meet the following mandatory criteria:

Mandatory Criteria

- Capability for installation on or near a bridge pier or abutment
- Ability to measure maximum scour depth within an accuracy of ± 0.3 m
- Ability to obtain scour depth readings from above the water or from a remote site
- Operable during storm and flood conditions

Since the mandatory criteria required that the instruments be capable of installation on or near a bridge pier or abutment, the research was limited to fixed instruments only. This paper summarizes the results of this research.

An initial literature search on scour instrumentation in 1990 revealed, and a resurvey of technology in 1994 confirmed, that fixed scour-measuring and -monitoring instruments can be grouped into four broad categories:

- Sounding rods - manual or mechanical device (rod) to probe streambed
- Buried or driven rods - device with sensors on a vertical support, placed or driven into streambed
- Fathometers - commercially available sonic depth finder
- Other Buried Devices - active or inert buried sensor (e.g., buried transmitter)

As a result of the literature review a laboratory testing program was designed to test at least one device from each category and to select devices for field testing that would have the greatest potential for meeting mandatory and desirable criteria.

2. FIELD TESTING OF INSTRUMENTS

The primary objectives of field testing of scour instrumentation were to test the adaptability of promising instruments to a wide range of bridge pier and abutment geometries and subject the instruments to a variety of geomorphic and environmental conditions. An additional significant objective was to gain experience in working with local State Highway Agency personnel who would ultimately be responsible for installation, maintenance, and collection of data from scour-monitoring devices.

2.1. Magnetic Sliding Collar Devices

Both simple (manually read) and automated readout magnetic sliding collar devices were installed and tested in a variety of locations in the field. Testing included pier installations of simple sliding collar instruments and pier and sloping abutment installations of automated magnetic sliding collar devices at riverine and tidal bridges.

Laboratory testing of a driven rod with an open architecture sliding collar with attached 152 mm magnets (see Figure 1) indicated that the sliding collar accurately tracked the progression of scour. Using this concept, a field prototype of a magnetic sliding collar was designed and fabricated. This instrument consisted of a 51-mm diameter stainless steel support pipe in 1.5-m sections. A magnetic collar, similar in design to the original collar used for laboratory testing, was fabricated to slide on the support pipe; however, the externally mounted magnetic switches tested in the laboratory were replaced by a much simpler approach to measuring scour. To determine the position of the collar, a sensor (probe) consisting of a magnetic switch attached to a battery and buzzer on a long graduated cable was fabricated. In operation, the probe is lowered through the annulus of the support pipe and the buzzer activates when the sensor reaches the magnetic collar. Collar position is determined by using the graduated cable to determine the distance from an established datum near the top of the support pipe to the magnetic collar.

Following field testing of manual readout magnetic sliding collar devices at the Colorado and New Mexico test sites, it was apparent that the support pipe or extension conduit, which is normally fastened to the upstream face of a bridge pier, can be vulnerable to ice or debris impact. Development of an automated readout magnetic sliding collar device could reduce this vulnerability to debris and ice impact if only the head of the device protrudes from the streambed in front of a pier or adjacent to an abutment (Figure 1). A flexible conduit with the wiring for the automated readout could carry the signal by a less vulnerable route, such as along a pile cap or pier footer and up the downstream face of a pier to a datalogger.

In order to automate the operation of the magnetic sliding collar, a laboratory prototype electronic insert (probe) was developed. The insert consists of string of magnetically actuated reed switches located at 152-mm intervals along the length of a stainless steel support structure. Magnets on the sliding collar actuate the reed switch at a given position as it comes in proximity. A datalogger provides excitation voltage for a brief sampling period. The probe is encased with waterproof flexible tubing, and is then inserted into the stainless steel pipe section(s) that comprise the support rod for the instrument. Sensors at different levels are activated as the magnet on the sliding collar slides down the stainless steel pipe as scour develops.

2.2. Low-Cost Fathometer Instrument Systems

Field testing of sonic depth finders (fathometers) included pier installations at riverine and tidal bridges. A low-cost fathometer was also configured and installed on a sloping abutment.

Standard practice for installation of fathometers to monitor bridge scour has been to mount the sonic transducers into a small durable steel encasement which was then bolted to the pier of the bridge below water level. The NCHRP project developed an alternative which permits mounting the transducer so that it can be serviced from the bridge deck or above water. Either steel or PVC conduit is bracketed to the bridge substructure to "aim" the sonic transducer at the most likely location for scour. The transducer was encased in a PVC "probe," which was pushed down through a larger diameter steel or PVC conduit (Figure 2). The probe snapped into position so that it protruded through a fitting located below water at the bottom of the conduit. With this arrangement the transducer is serviceable from above water.

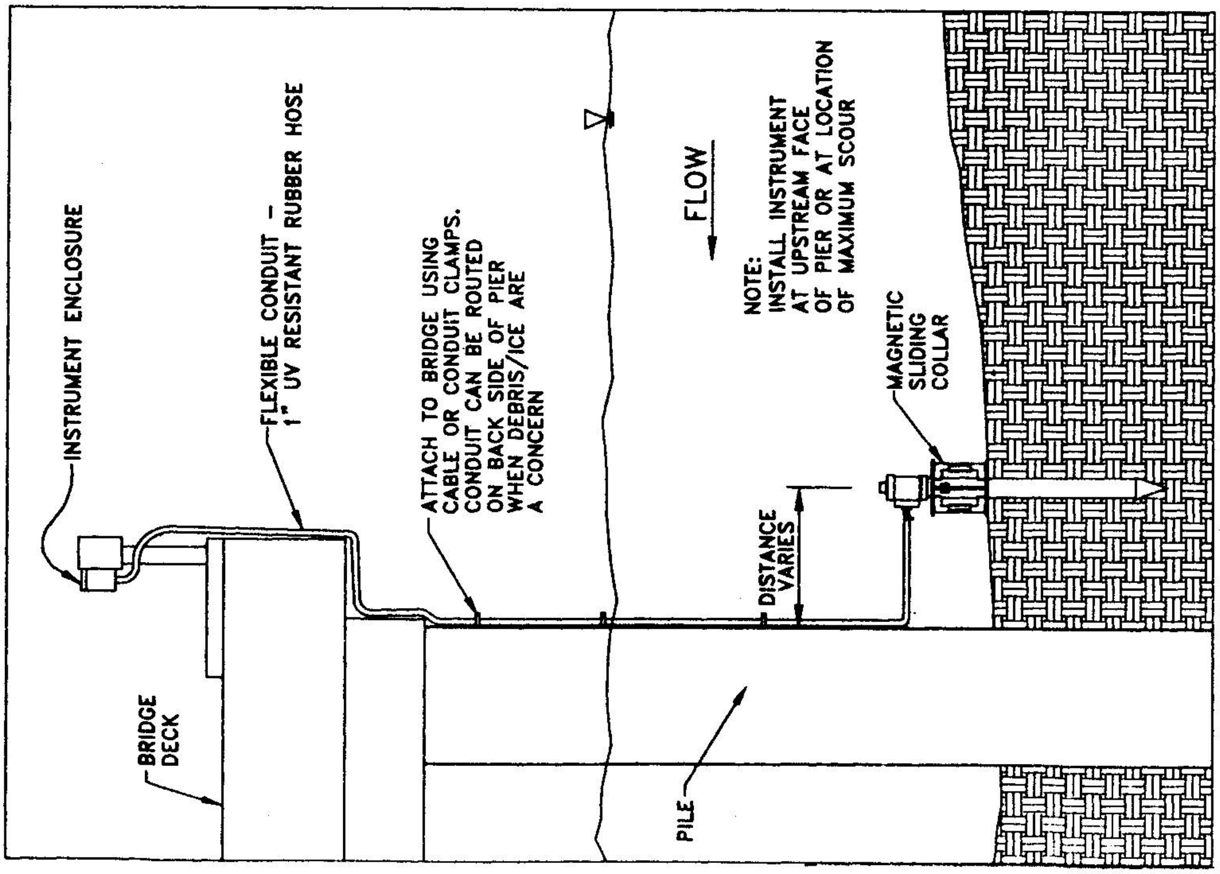

Figure 1. Automated read-out magnetic sliding collar device [3].

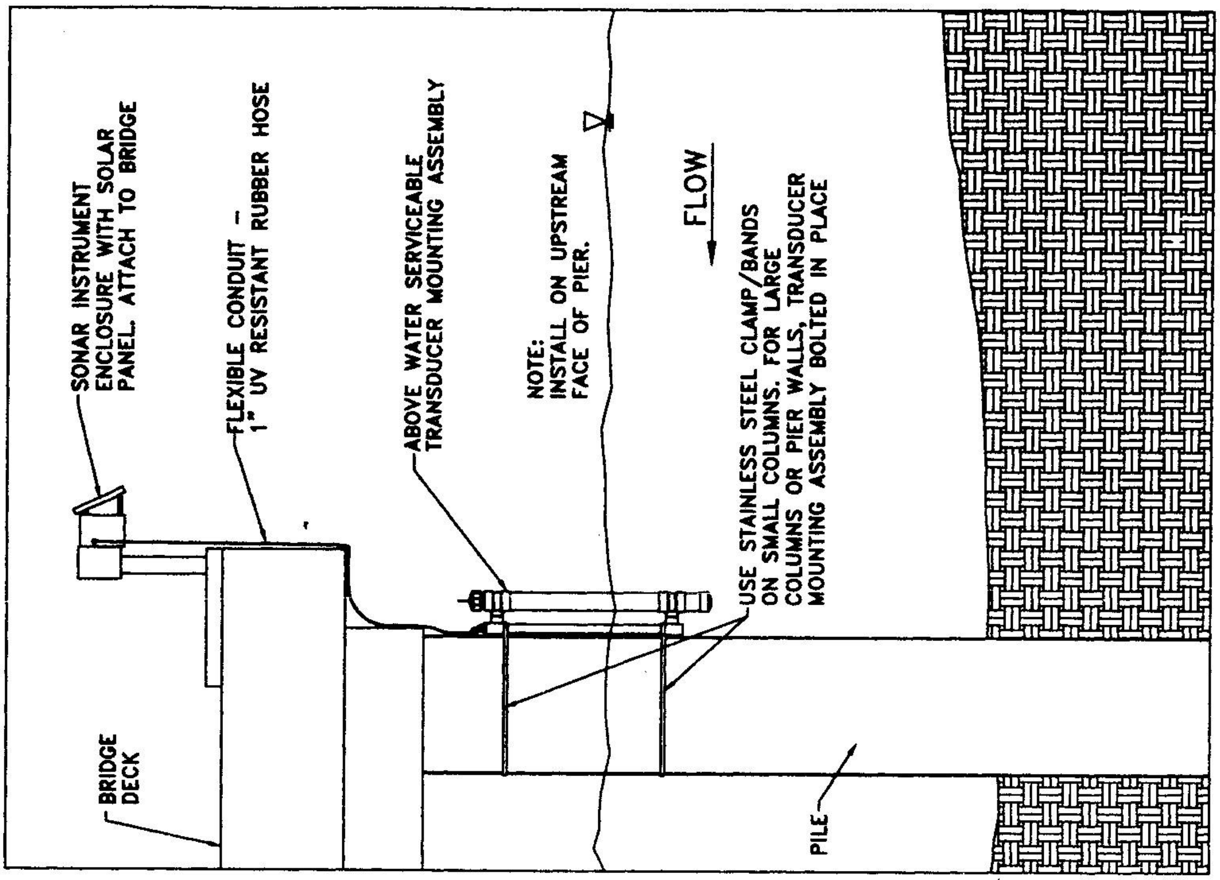

Figure 2. Above-water serviceable low-cost fathometer system [4].

2.3. Other Buried Devices

In late 1997 following completion of the NCHRP project, a buried transmitter "float out" device was developed for application on bridge piers over ephemeral stream systems. This device consists of a radio transmitter buried in the channel bed at a pre-determined depth. When the scour reaches that depth, the float-out device rises to the surface and begins transmitting a radio signal that is detected by a receiver in an instrument shelter on the bridge. Installation requires using a conventional drill rig with a hollow stem auger. After the auger reaches the desired depth, the float out transmitter is dropped down the center of the auger. Substrate material refills the hole as the auger is withdrawn.

The float out device can be monitored by the same type of instrument shelter/data logger currently being used to telemeter low-cost fathometer or automated sliding collar data. The instrument shelter contains the data logger, cell-phone telemetry, and a solar panel/gell-cell battery for power. The data logger monitors the sliding collar and sonar scour instruments, taking readings every hour and transmitting the data once per day to a computer at a central location (e.g., DOT District). A threshold elevation is defined that, when reached, initiates a phone call to a pager network. The bridge number is transmitted as a numeric page, allowing identification of the bridge where scour has occurred. The float out devices are monitored continuously, and if one of these devices floats to the surface, a similar call is automatically made to the pager network.

2.4. Instrument Costs

The "low-cost" sonic system as tested under NCHRP Project 21-3 will cost approximately $4,000 (U.S.). The cost of a magnetic sliding collar device will range from $2,500 for a simple manual-readout device to $4,000 for an automated system. Instrument system costs include the basic instrument and mounting hardware, as well as power supply, data logger, and instrument shelter/enclosure, where applicable. A cell-phone telemetry link will add approximately $3,000 to the system cost. A float-out buried transmitter can be fabricated for approximately $500, and monitored by the same data logger/cell-phone system installed for either a sonic system or automated sliding collar.

The installation costs for sliding collar and sonic devices can vary dramatically depending on the complexity of the installation. For large rivers where the installation must be conducted from the bridge deck, the level of effort required for installation of an instrument system can be 4-6 person days, plus the necessary equipment for installation.

3. RESEARCH FINDINGS

The two instruments developed under NCHRP Project 21-3, a low-cost sonic system and either a manual-readout or automated magnetic sliding collar device, have been tested extensively and are fully field-deployable. **Both instrument systems met all of the mandatory criteria and most of the desirable criteria established for this project**. Use of these instruments as scour monitoring countermeasures will provide State Highway Agencies with an essential element of their plans of action for many scour-critical, scour-susceptible, or unknown foundation bridges.

No single methodology or instrument can be utilized to solve the scour monitoring problems for all situations encountered in the field. Considering the wide range of operating conditions necessary, environmental hazards such as debris and ice, and the variety of stream types and bridge geometry's encountered in the field, it is obvious that several instrument systems using different approaches to detecting scour will be required.

The Installation, Operation, Fabrication Manuals for the low-cost sonic system and magnetic sliding collar devices [3] and [4] provide complete instrument documentation, including specifications and assembly drawings. That information, together with the findings, appraisal, and applications information of the final report [2], provide a potential user of a scour monitoring device complete guidance on selection, installation, operation, maintenance, and if desired, fabrication of two effective systems, one of which could meet the need for a fixed scour instrument at most sites in the field. In addition, a third instrument system consisting of float-out buried transmitters has been installed at several bridge sites on ephemeral streams, and at one site detected scour at 3.6 m below the streambed.

Of the devices tested extensively in the field, the low-cost sonic system and the manual-readout sliding collar device are both vulnerable to ice and debris; however, both proved to be surprisingly resistant to damage from debris or ice impact at field test sites. The sonic system can be rendered inoperative by the accumulation of debris, and presumably ice, between the transducer face and streambed. The manual-readout sliding collar requires an extension conduit, generally up the front face of a pier, which can be susceptible to debris or ice impact damage unless the extension can be firmly anchored to a substructure element. From this perspective, the automated sliding collar device has the distinct advantage of having a configuration which places most of the device below the streambed, and therefore, less vulnerable to ice or debris. The connecting cable from the device to a datalogger on the bridge deck can be routed through a buried conduit and up the downstream face of a bridge pier or abutment where it is much less vulnerable to damage.

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BRIDGE SCOUR AND STREAM INSTABILITY COUNTERMEASURES - CURRENT PRACTICE IN THE UNITED STATES

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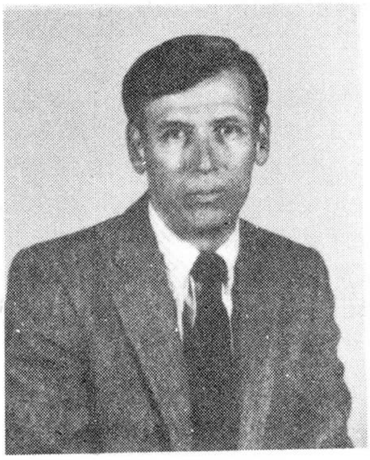

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SUMMARY

This paper provides an overview of the U.S. Federal Highway Administration (FHWA) publication, Hydraulic Engineering Circular Number 23 (HEC-23), "Bridge Scour and Stream Instability Countermeasures" published in July 1997. The HEC-23 manual provides experience, selection, and design guidelines in the form of a countermeasure matrix as an aid to identifying types of countermeasures which have been used by State Highway Agencies for bridge scour and stream instability problems. The matrix supports the selection of appropriate countermeasures considering such characteristics as the functional application, suitable river environment, and estimated allocation of maintenance resources. References are included for each type of countermeasure. Design guidelines for eight countermeasures are also provided in HEC-23.

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1. INTRODUCTION AND MAGNITUDE OF THE PROBLEM

On March 10, 1995, at about 9:00 p.m., the southbound and northbound bridges on Interstate 5 over Arroyo Pasajero in California collapsed during a large flood. Four vehicles plunged into the creek, resulting in seven deaths. The two bridges were built in 1967. Each bridge was approximately 32 meters (m) long and consisted of four concrete-slab spans supported by 3 bents with 6 drilled shafts (0.41 m in diameter). After a period of degradation, the piles were reinforced with a 3.66 m high web wall. Long-term degradation, contraction scour, and local scour from the March 10 flood exposed the piles approximately 7.6 m below the original streambed. This scour depth was 2.4 m below the pile steel reinforcement and they collapsed due to the force of water and debris on the piles and web wall.

The Arroyo Pasajero tragedy is only the latest in a series of bridge failures in the U.S. that have highlighted the national problem of bridge scour. The catastrophic failure of the Schoharie Creek bridge on the New York Thruway in April 1987, which cost ten lives, focused attention in the U.S. on the bridge scour problem; and the subsequent failure of the U.S. 51 bridge over the Hatchie River in April 1989, which cost eight lives, broadened the concern to stream stability problems, as well. The damages and economic costs of the Mississippi River floods in 1993 and floods in Georgia in 1994 underscored the vulnerability of the nation's transportation system to bridge scour and stream instability.

There are more then 575,000 bridges in the U.S. National Bridge Inventory. Approximately 84 percent of these bridges are over water. Highway bridge failures cost millions of dollars each year as a result of both direct costs necessary to replace and restore bridges, and indirect costs related to disruption of transportation facilities. In the U.S., stream instability, long-term streambed aggradation or degradation, contraction scour, local scour, and lateral scour or erosion cause 60 percent of these failures.

Following the failure of the Schoharie Creek bridge in April 1987, the Federal Highway Administration (FHWA) issued a Technical Advisory (TA) that established a national scour evaluation program as an integral part of the National Bridge Inspection Program. To support the implementation of this program, the FHWA contracted for development of a training course on Stream Stability and Scour at Highway Bridges. This course is based on FHWA's Hydraulic Engineering Circular (HEC) No. 18, entitled, "Evaluating Scour at Bridges" [1] and HEC-20, "Stream Stability at Highway Structures" [2]. These two documents, prepared by the authors of this paper, establish the current state-of-the-art for the analysis of bridge scour and stream stability problems in the U.S. The training course, based on these documents, is the principal vehicle for technology transfer to state highway and transportation departments for initial scour screening, follow-on scour evaluation, and design of foundations for new and replacement bridges.

2. SCOUR PROCESSES

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams. Scour depths of up to 36 m have been measured at bridge piers, while depths of 5 to 12 m are common.

Total scour at a highway crossing consists of three components: (1) long-term aggradation or degradation, (2) contraction scour, and (3) local scour. Generally, total scour is the algebraic sum of the components. HEC-18 [1] presents procedures, equations, and methods to analyze these scour components in both riverine and coastal areas. The equations for estimating contraction and local scour are based on laboratory experiments with limited field verification, and those recommended in HEC-18 are considered to be the best available for estimating scour depths.

Aggradation and degradation are long-term streambed elevation changes due to natural or man-induced causes which can affect long reaches of a river. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge; whereas, degradation involves the lowering or scouring of the bed of a stream due to a deficit in sediment supply from upstream.

Contraction scour in a river involves the removal of material from the bed across all or most of the channel width in the bridge reach as the result of increased velocities and shear stress on the bed. Contraction scour often occurs when the bridge approach embankments encroach onto the floodplain or into the main channel.

Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by the flow obstructions. Determining the magnitude of both contraction scour and local scour is complicated by the cyclic nature of scour. Both types of scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes refill with sediment. This fact contributed to the Schoharie Creek bridge failure.

In addition to the types of scour mentioned above, naturally occurring lateral migration of the main channel of a stream within a floodplain may increase pier scour, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers. As described in HEC-20 [2], factors that affect lateral stream movement are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials. Lateral instability was the primary cause of the Hatchie River bridge failure.

3. THE NATIONAL RESPONSE

Following the catastrophic failure of the Schoharie Creek bridge, the FHWA established a national scour evaluation program. The 1988 revision of the National Bridge Inspection Standards (NBIS) requires an inspection program that includes procedures for underwater inspection. Specifically, each of the more than 575,000 bridges in the U.S. are to be inspected at regular intervals not to exceed two years (longer intervals can be used when justified and approved). Bridges with underwater members that cannot be evaluated visually for scour and structural integrity must be inspected by divers at least every five years.

Results of each bridge inspection are documented according to the guidelines provided in the "Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges" [3], more commonly referred to as the "Coding Guide." The Coding Guide requires coding more than 100 separate items at each inspection. Relevant to stream stability and bridge scour are items 60 (Substructure), 61 (Channel and Channel Stability), 71 (Waterway Adequacy), 92 and 93 (Underwater Critical Feature Inspection), and 113 (Scour-Critical Bridges). The two-year cycle bridge inspections are the basis for coding items 60, 61, 71, 92, and 93. Item 113 coding is based on scour evaluations in accordance with the FHWA T 5140.23.

T 5140.23 [4], provides guidance on the development and implementation of procedures for evaluating bridge scour. The TA indicates that every bridge over a waterway, whether existing or under design, should be evaluated for scour in order to determine prudent measures to be taken for its protection. The evaluations are to be conducted by an interdisciplinary team of hydraulic, geotechnical, and structural engineers.

The TA specifies that new bridges must be designed assuming that all streambed material in the computed scour prism has been removed and is not available for bearing or lateral support. Existing bridges found to be scour-critical, either from field observations or from results of the analytical scour evaluation, require development of a Plan of Action. The Plan of Action should include instructions regarding the type and frequency of inspections, particularly as it may relate to the need to close a bridge, if necessary, and a schedule for the timely design and construction of scour countermeasures. Initial scour susceptibility screening was completed for the most part by October 1992. FHWA established January 1997 as the target date for completing scour evaluations of all bridges identified as scour-susceptible. The results of this national bridge scour screening program, as of January 1998, are shown in Table 1.

The number of bridges with "unknown" foundations points to a significant shortcoming of record-keeping in the U.S. in relation to bridge construction programs. An unknown foundation rating means that after office and field reviews, it was uncertain what the structural foundation condition was or what pile lengths were for pile-supported foundations. Thus, for 20 percent of the bridges over water in the U.S., an in-depth scour evaluation cannot be completed. Except for Interstate bridges, unknown foundation bridges are to be monitored until such time as technology becomes available to determine foundation conditions in-situ.

Table 1. National Bridge Scour Screening Program Results.

Categories	Number of Bridges	Percentage
EVALUATION COMPLETE		
Low risk bridges	301,658	62.2
Scour critical	17,030	3.5
EVALUATION NEEDED		
Scour susceptible	66,523	13.7
Not screened	2,580	0.5
EVALUATION DEFERRED		
Unknown foundations	97,599	20.1

4. TECHNOLOGY TRANSFER TO SUPPORT THE NBIS

To support the implementation of bridge scour evaluations for the NBIS, the FHWA, through the National Highway Institute (NHI) contracted for development of a training course on Stream Stability and Scour at Highway Bridges. The FHWA scour evaluation program specifically requires analytical evaluation of scour and appropriate training of inspectors. The procedures described in HEC-18 [1] and HEC-20 [2] are not typically taught in undergraduate engineering programs, and for the most part were not historically incorporated in the bridge design process. Thus, much of this technology is new to engineers and designers charged with completing scour evaluations and/or designing or approving new bridges. Therefore, a training course was needed to facilitate technology transfer from HEC-18 and HEC-20 to bridge design professionals. In addition, bridge inspectors, who are well versed in pavement and steel bridge inspection procedures, need an understanding of scour and stream instability and specific instruction in the factors important to scour-critical bridges in order to provide follow-on scour inspections.

Given this background, the training course, "Stream Stability and Scour at Highway Bridges" was developed during 1988-1990 by the authors of this paper. Course objectives included:

- Identify stream stability and scour problems at bridges
- Understand problems caused by stream instability and scour
- Estimate magnitude of scour at bridge piers and abutments and in the bridge reach
- Propose potential countermeasures for stream instability and scour problems

The course was designed to provide comprehensive training in the understanding and prevention of hydraulic-related failures of highway bridges. The effects of stream instability, scour, and stream aggradation and degradation are covered. Countermeasures to these problems are also provided. HEC-20 provides a multi-level step-wise approach to the problem, including reconnaissance-level geomorphic analyses and basic engineering analysis techniques such as the application of the standard computer models to develop hydraulic variables for scour evaluation. HEC-18 provides specific computational procedures for the various scour components under riverine and tidal flow conditions. A revised metric version of the course (and supporting documents) as well as an abbreviated version of the course designed to meet the specific needs of bridge inspectors were introduced in January 1996. To date, these courses have been presented more than 100 times to State Highway Agencies, federal agency personnel, and consultants.

In July of 1997, the FHWA National Highway Institute issued Hydraulic Engineering (HEC) No. 23, "Bridge Scour and Stream Instability Countermeasures." [5] This document provides experience and selection for a wide range of countermeasures and specific design guidelines for several countermeasures frequently used by State Highway Agencies.

5. SUMMARY

Recent catastrophic bridge failures in the United States and a nation-wide screening of bridges over water for scour vulnerability have focused national attention on the bridge scour problem. In the last ten years, the U.S. has made a substantial investment in field data gathering, research, and development of analytical techniques to determine the scope of the problem, plan remedial actions for existing bridges, and design new bridges to be safe from the effects of scour and stream instability. Training courses on scour and stream stability problems at bridges are available from the Federal Highway Administration National Highway Institute, and Hydraulic Engineering Circulars 18, 20, and 23 issued by the Federal Highway Administration provide technical guidelines for analyzing and evaluating the bridge scour problem in the United States

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SCOUR AND STREAM STABILITY PROBLEMS AT HIGHWAY BRIDGES IN THE UNITED STATES

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SUMMARY

This paper provides an overview of the magnitude of the bridge scour problem in the United States. Procedures and results from the ongoing national program to evaluate all bridges over water for scour vulnerability are highlighted. Current practices for analyzing bridge scour are reviewed and sources of technology transfer are referenced and highlighted, including training courses on bridge scour and stream stability offered by the U.S. Federal Highway Administration, National Highway Institute.

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1. INTRODUCTION

As of February 1998, results of a national screening of bridges over water by State Highway Agencies indicate that approximately 66,000 bridges are scour susceptible and another 97,000 have unknown foundations. Of the scour susceptible bridges that have been evaluated, about 17,000 have been identified as scour critical. These bridges will require monitoring, repair, or scour protection through the installation of bridge scour and stream instability countermeasures.

Countermeasures for bridge scour and stream instability problems are defined as measures incorporated into a highway-stream crossing system to monitor, control, inhibit, change, delay, or minimize stream instability and bridge scour problems. An action plan for monitoring structures during and/or after flood events can also be considered a countermeasure. Countermeasures also include river stabilizing works over a reach of the river up- and downstream of the crossing. Countermeasures may be installed at the time of highway construction or be retrofitted to resolve stability problems as they develop at existing crossings.

While considerable research has been dedicated to design of countermeasures for scour and stream instability, many countermeasures have evolved through a trial and error process. In addition, some countermeasures have been applied successfully in one locale, state or region, but have failed when installations were attempted under different geomorphic or hydraulic conditions. In many cases, a countermeasure that has been used with success in one state or region is virtually unknown to highway design and maintenance personnel in another state or region. Thus, there is a significant need for information transfer regarding bridge scour and stream instability countermeasure design, installation, and maintenance.

This need resulted in the publication [1] of Hydraulic Engineering Circular Number 23 (HEC-23) in July 1997. HEC-23 "Bridge Scour and Stream Instability Countermeasures - Experience, Selection, and Design Guidance," represents an initial step toward sharing countermeasure experience, selection, and design guidelines among Federal, State, and local highway agency personnel. This information is intended to facilitate the selection and design of countermeasures as State Highway Agencies develop Plans of Action for bridges identified as scour critical.

2. THE COUNTERMEASURES MATRIX

A wide variety of countermeasures have been used to control scour and stream instability at highway bridges. The countermeasure matrix presented in HEC-23 is organized to highlight the various groups of countermeasures and to identify their individual characteristics. The matrix identifies most countermeasures used by State Highway Agencies and lists information on their functional applicability to a particular problem, their suitability to specific river environments, the general level of maintenance resources required, and which states have experience with specific countermeasures. Finally, a reference source for design guidance is noted, where available.

While page limitations and format restrictions preclude presenting the HEC-23 countermeasures matrix in this paper, Table 1 shows the Functional Applications section of the matrix. In Table 1 countermeasures were organized into groups based on their functionality with respect to scour and stream instability. The three main groups of countermeasures are: hydraulic countermeasures, structural counter-measures and monitoring.

Table 1. Bridge Scour and Stream Instability Countermeasures Matrix - Functional Applications

Countermeasure Group	FUNCTIONAL APPLICATIONS				
	Local scour		Contraction Scour	Stream Instability	
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral
GROUP 1. HYDRAULIC COUNTERMEASURES					
GROUP 1.A. RIVER TRAINING STRUCTURES					
TRANSVERSE STRUCTURES					
Impermeable spurs (jetties, groins, wing dams)	◗	◗	○	○	●
Permeable spurs (fences, netting)	◗	◗	○	○	●
Transverse dikes	○	○	○	○	●
Bendway weirs/Stream barbs	◗	◗	○	○	●
Hardpoints	○	○	○	○	●
Drop structures (check dams, grade control)	◗	◗	◗	●	○
Embankment Spurs	◗	○	◗	○	○
LONGITUDINAL STRUCTURES					
Longitudinal dikes (crib/rock toe/embankments)	◗	○	○	○	●
Retards	◗	○	○	○	●
Bulkheads	●	○	○	○	●
Guide banks	●	◗	◗	○	◗
AREAL STRUCTURES/TREATMENTS					
Jacks/tetrahedron jetty fields	○	○	○	○	●
Vanes	○	◗	○	○	●
Channelization	◗	◗	○	○	●
Flow relief (overflow, relief bridge)	◗	◗	●	○	○
Sediment detention basin	○	○	○	●	○
GROUP 1.B. ARMORING COUNTERMEASURES					
REVETMENTS AND BED ARMOR					
Rigid					
Soil cement	●	◗	◗	◗	●
Concrete pavement	●	◗	●	◗	●
Rigid grout filled mattress/concrete fabric mat	●	◗	◗	◗	●
Grouted riprap	◗	○	○	○	◗
Flexible/articulating					
Riprap	●	◗	◗	◗	●
Self launching riprap (windrow)	○	○	○	○	◗
Riprap fill-trench	◗	○	○	○	●
Gabions/gabion mattress	●	◗	◗	◗	●
Wire enclosed riprap mattress (rail bank/sausage)	●	○	○	○	●
Articulated blocks (interlocking and/or cable tied)	●	◗	◗	◗	●
Articulating concrete/grout mattress (fabric-formed)	●	◗	◗	◗	●
LOCAL SCOUR ARMORING					
Riprap (fill/apron)	●	◗	N/A	N/A	N/A
Grouted riprap	◗	○	N/A	N/A	N/A
Concrete armor units (Toskanes, tetrapods, etc.)	◗	◗	N/A	N/A	N/A
Grout filled bags/sand cement bags	●	◗	N/A	N/A	N/A
Gabions	●	◗	N/A	N/A	N/A
Articulated blocks (interlocking and/or cable tied)	●	◗	N/A	N/A	N/A
Sheet pile/cofferdam	◗	◗	N/A	N/A	N/A

Table 1. (Cont'd) Bridge Scour and Stream Instability Countermeasures Matrix - Functional Applications

Countermeasure Group	FUNCTIONAL APPLICATIONS				
	Local scour		Contraction Scour	Stream Instability	
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral
GROUP 2. STRUCTURAL COUNTERMEASURES					
FOUNDATION STRENGTHENING					
Crutch bents/Underpinning	○	●	●	●	◗
Cross bracing	○	●	●	●	○
Continuous spans	○	●	●	●	○
Pumped concrete/grout under footing	●	●	◗	◗	◗
Lower foundation	●	●	●	●	●
PIER GEOMETRY MODIFICATION					
Extended footings	N/A	●	N/A	N/A	N/A
Pier shape modifications	N/A	●	N/A	N/A	N/A
Debris deflectors	N/A	●	N/A	N/A	N/A
Sacrificial piles/dolphins	N/A	●	N/A	N/A	N/A
GROUP 3. MONITORING					
FIXED INSTRUMENTATION					
Sonar scour monitor	◗	●	●	●	◗
Magnetic sliding collar	●	●	●	●	◗
Sounding rods	◗	●	●	●	◗
PORTABLE INSTRUMENTATION					
Physical probes	●	●	●	●	●
Sonar probes	●	●	●	●	●
VISUAL MONITORING					
Periodic Inspection	●	●	●	●	●
Flood watch	●	●	●	●	●

● **well suited/primary use** - the countermeasure is well suited for the application; the countermeasure has a good record of success for the application; the countermeasure was implemented primarily for this application.

◗ **possible application/secondary use** - the countermeasure can be used for the application; the countermeasure has been used with limited success for the application; the countermeasure was implemented primarily for another application but also can be designed to function for this application.

In addition, this symbol can identify an application for which the countermeasure has performed successfully and was implemented primarily for that application, but there is only a limited amount of data on its performance and therefore the application cannot be rated as well suited.

○ **unsuitable/rarely used** - the countermeasure is not well suited for the application; the countermeasure has a poor record of success for the application; the countermeasure was not intended for this application.

N/A **not applicable** - the countermeasure is not applicable to this functional application.

Hydraulic Countermeasures are those which are primarily designed either to modify the flow (river training) or resist erosive forces caused by the flow (armoring). Structural Countermeasures involve modification of the bridge structure (foundation) to prevent failure from scour. Monitoring describes activities used to facilitate early identification of potential scour problems. Monitoring allows for action to be taken before the safety of the public is threatened by the potential failure of a bridge. Monitoring can be accomplished with fixed or portable instrumentation or visual inspection.

3. COUNTERMEASURE CHARACTERISTICS

The countermeasure matrix was developed to identify distinctive characteristics for each type of countermeasure. Five categories of countermeasure characteristics were defined to aid in the selection and implementation of countermeasures:

- Functional Applications
- Suitable River Environment
- Maintenance
- Installation/Experience by State
- Design Guidelines Reference

These categories were used to answer the following questions: For what type of problem is the countermeasure applicable? For what type of river environment is the countermeasure best suited or, are there river environments where the countermeasure will not perform well? What level of resources will need to be allocated for maintenance of the countermeasure? What states or regions in the United States have experience with this countermeasure? Where do I obtain design guidance reference material? Only one category (Functional Applications) is shown in Table 1 to illustrate the organization of the matrix.

4. DESIGN GUIDELINES

Following the countermeasures matrix, design guidelines are provided for several countermeasures which have been applied successfully on a state or regional basis, but for which only limited design references are available in published handbooks, manuals, or reports. No attempt has been made to include in HEC-23 design guidelines for all the countermeasures listed in the matrix. There are, however, references in the matrix to publications that contain at least a sketch or photograph of a particular countermeasure, and in many cases contain more detailed design guidelines.

FHWA currently has four publications dealing with stream instability and bridge scour countermeasures:

- HEC-18 "Evaluating Scour at Bridges [2]
- HEC-20 "Stream Stability at Highway Structures [3]
- HIRE "Highways in the River Environment" [4]
- HEC-11 "Design of Riprap Revetment [5]

These documents contain detailed design procedures for many standard countermeasures such as impermeable and permeable spurs, guidebanks, and riprap for abutments, piers, and revetment.

A number of highway agencies provided specifications, procedures, or design guidelines for bridge scour and stream instability countermeasures that have been used successfully locally, but for which only limited design guidance is available outside the agency. Several of these are presented in HEC-23 following the matrix for the consideration of and possible adaptation to the needs of other highway agencies. Design guidelines for the following seven countermeasures are provided based on information obtained from State Highway Agencies: bendway weirs/ stream barbs, soil cement, wire enclosed riprap, articulated concrete block systems, articulating grout filled mattresses, Toskanes (artificial riprap), and grout filled bags. Design Guideline 8 presents guidance for pier and abutment riprap protection from HEC-18 [2].

5. CONCLUSIONS

The countermeasures matrix and design guidelines presented in HEC-23 provide a wealth of information on experience, selection, and design for bridge scour and stream instability countermeasures. This information is not readily available in any other single source document, and should prove useful to State Highway Agencies as they prepare and implement Plans of Action for scour critical bridges.

The first edition of HEC-23 represents an initial step toward sharing countermeasure experience, selection, and design guidelines among Federal, State, and local highway agencies. It is expected that revisions and additions to the Circular will be made as additional technology and techniques become available and are tested in the field.

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CASE STUDIES AND LESSONS LEARNED FROM RECENT SCOUR-RELATED BRIDGE FAILURES IN THE UNITED STATES

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SUMMARY

Bridge failures from scour in the United States in the past 10 years have cost 25 lives, millions of dollars in replacement costs, and many more millions of dollars in the indirect cost of detours, lost commerce, and litigation. As a result of the investigation of the bridge failure over Schoharie Creek, the Federal Highway Administration recommended that all bridges over water in the United States be evaluated as to their vulnerability to failure from scour. In addition, the failures prompted an increase in bridge scour research. Three bridge failures: Schoharie Creek (I-90), the Hatchie River (U.S. 51), and Arroyo Pasajero (I-5) are described, and some of the lessons learned from these failures and the ongoing evaluation program in the United States are highlighted.

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1. INTRODUCTION

There are 575,000 bridges in the Unites States National Bridge Inventory. Eighty four percent of the bridges are over water. Stream instability and scour cause 60 percent of the bridge failures in the United States. Nationally, the annual cost for scour related bridge failures is about $30 million and flood damage repair costs for Federal-aid highways are about $50 million. Three bridge failures from scour in (1) Upstate New York, (2) Western Tennessee and (3) Central Valley California in the past 10 years with the cost of 25 lives, millions of dollars in replacement costs and many more millions of dollars in the indirect cost of detours, lost commerce and litigation illustrate the societal and financial costs of bridge failures. As the result of the investigation into the Schoharie Creek bridge failure all States are required to evaluate the scour susceptibility of all their bridges over water. This paper will briefly describe these three failures and some of the lessons learned from the failures and the States' evaluation program in the past 10 years (1988 to 1998).

2. SCHOHARIE CREEK, NEW YORK (1987) BRIDGE FAILURE

At approximately 10:45 a.m. April 5, 1987, the center span and east center span of the 540-foot-long bridge on the New York State Thruway over Schoharie Creek in Montgomery County, New York, collapsed during a near-record flood (about 1,756 m^3/s). About an hour and a half later, the west center span fell into the water (see Figure 1). One tractor semi-trailer and four automobiles fell nearly 25 m into the river after the first span collapsed, resulting in 10 fatalities.

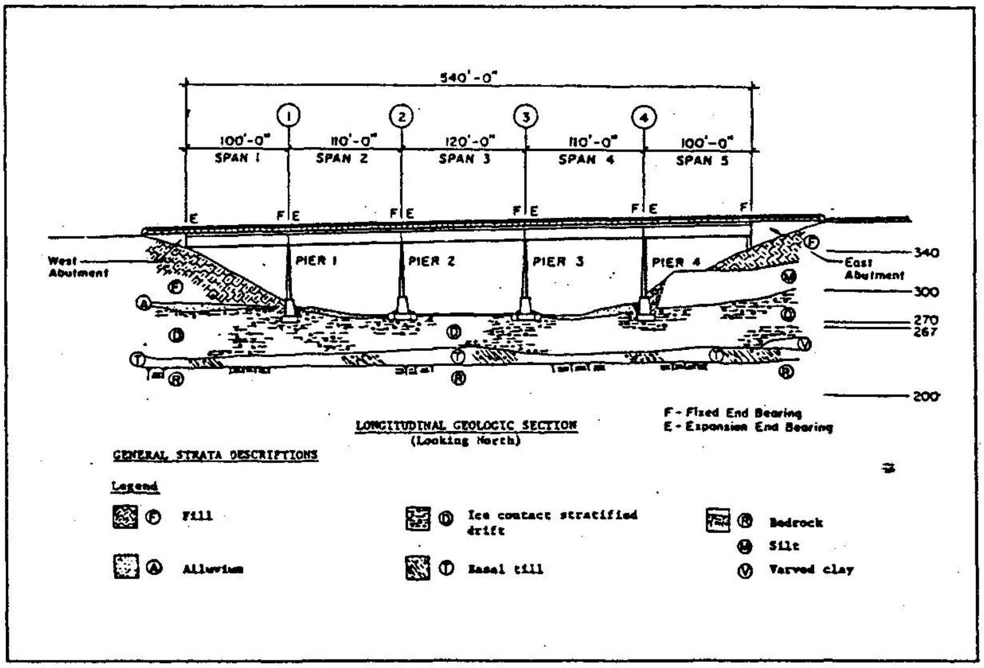

Figure 1. South elevation of Schoharie Creek bridge showing key structural features and schematic geological section.

The substructure consisted of four piers and two abutments. Each pier was a rigid frame (columns and tie beam) supported on a lightly reinforced concrete plinth (pedestal) and spread footing bearing on glacial till just below the streambed. The abutments were founded on piles driven through the embankment fill into the underlying glacial till. The piers were founded on spread footings 1.5 m deep by 5.5 m wide by 25 m long with no piles. The bridge designers assumed that the glacial till substrate was "nonerodible."

After an extensive investigation and detailed analyses, which included hydraulic computer and physical modeling [1], the U.S. National Transportation Safety Board (NTSB) [2] determined that the probable cause of the collapse of the Schoharie Creek bridge was the failure of the Thruway Authority to maintain adequate riprap around the bridge piers, which led to severe erosion (scour) in the soil beneath the spread footings. It was concluded that the 1987 flood alone probably did not cause failure of the Thruway bridge. Rather, the cumulative effect of local scour around pier 3, particularly in the last 10 years, was the most significant hydraulic factor contributing to the failure.

Using the Schoharie Creek bridge and others damaged during the 1987 flooding in New York as examples, an economic study [3] estimated that the indirect costs suffered by the general public, business, and industry because of long detours and lost production time as a result of a bridge failure exceed the direct cost of bridge repair by a factor of five.

3. HATCHIE RIVER, TENNESSEE (1989) BRIDGE FAILURE

On April 1, 1989, at about 8:15 p.m., a section (Bents 70-71) of the 1,280m-long bridge on U.S. Route 51 over the Hatchie River in Tennessee collapsed during a moderate flood (about 224 m^3/s). The accident report revealed that the collapse occurred slowly over a period of about one hour. Four passenger cars and one tractor semi-trailer plunged into the river, resulting in 8 deaths.

The bridge substructure consisted of main channel piers and floodplain bents supported on piles about 6.1 m long (Figure 2). There was about a 4-meter difference in elevation between the pile cap for the main channel piers and the pile cap of the shallower floodplain bents. A post-failure investigation revealed the following rates of channel migration into the north bank of the river at the bridge: 1931 to 1975 - 0.24 m/yr; 1975 to 1989 - 1.37 m/yr; and 1981 to 1989 - 0.58 m/yr [4].

The NTSB determined that the probable cause of the collapse of the northbound U.S. Route 51 bridge spans was the northward migration of the main river channel, which the Tennessee Department of Transportation failed to evaluate and correct. As with the Schoharie Creek failure, the lack of structural redundancy in the design of the bridge spans contributed to the severity of the accident [4].

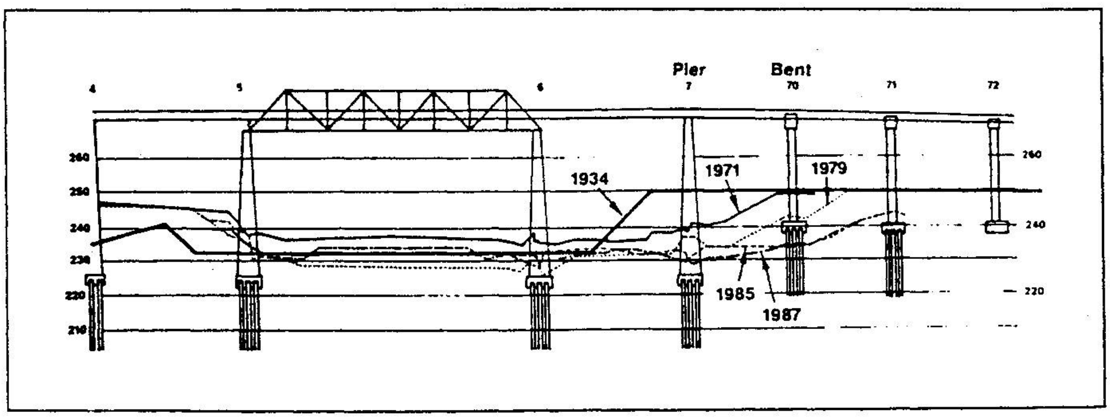

Figure 2. Channel cross section change at the Hatchie River bridge, Tennessee.

4. ARROYO PASAJERO, CALIFORNIA (1995) BRIDGE FAILURE

On March 10, 1995 the I-5 bridges over Arroyo Pasajero near Coalinga, California failed with the loss of 7 lives. The flow was 773 cms with about a 75-year return period. The bridge was constructed in 1967. The foundation of the bridge was 3 bents, each consisting of six 406 mm cast-in-place columns spaced approximately 2.3 m on centers. The abutments were vertical wall with wing walls. The columns were embedded 12.5 m; but the columns only had steel reinforcement for 5.2 m below the original ground. The bents were at an angle to the flow, that in 1995 was estimated to be from 15 to 26 degrees.

A flood in 1969 lowered the stream bed 1.83 m and damaged one column. In repairing the damage a web wall 2.44 or 3.66 m high, 11.6 m long and 0.6 m wide was constructed around the columns to reinforce them. The elevation of the bottom of the web wall was not established. The angle of attack of 15 to 26 degrees was not a factor in local pier scour when the bents were composed of columns but the web wall changed that.

An investigation [5] determined that long-term degradation was 3 m and contraction scour was calculated as 2.6m. Local pier scour, as determined from a model study, ranged from 2 to 2.7 m. The 2.0 m of scour occurred in the model study when the web wall was above the bed and 2.7 m of scour occurred with the web wall at the bed. A minimum potential total scour depth of 7.6 m would result in the column bents having 4.9 m of remaining embedment, but would have exposed 2.4 m of the columns without steel reinforcement. The force of water and debris on the exposed column sections without steel reinforcement caused them to fail.

5. LESSONS LEARNED

These bridge failures, as well as the scour evaluation program and research projects that were initiated after the Schoharie Creek bridge failure resulted in the following lessons learned in the past ten years (1988 to 1998):

- Bridge failures are expensive. In most cases the indirect costs are many times larger than the direct costs of bridge replacement.
- It is dangerous to consider stream bed material as "non erodible." Sedimentary rock may be erodible in high velocity turbulent flow. Even bed rock may be eroded over time.
- Stream instability is an important consideration in bridge evaluation and design, and in many cases stream instability can significantly increase scour potential at a bridge.
- The evaluation of the vulnerability of bridges to scour, design of scour countermeasures and the design of new bridges should be conducted by an interdisciplinary team of hydraulic, geotechnical, structural and bridge engineers.
- Bridges should be evaluated and designed to be safe from the 100-year flood or a smaller overtopping flood if it puts more stress on the bridge. The appropriate geotechnical safety factor should be used in the design for this flood event. The foundation design should be checked for safety from a super flood with a geotechnical safety factor of 1. The magnitude of the 500 year flood is suggested for the super flood.
- Inspection is an important factor in bridge safety and inspectors must be adequately trained to recognize potential stream instability and scour problems.
- Communication between bridge inspectors and decision makers in Highway Agencies is a critical aspect of bridge safety. As noted by the NTSB, "Unfortunately, in the bridge inspection program, itself, there is a lot of paper work being filled out but not, in many cases, adequate follow through to correct the problems being identified."

- The HEC-18 [6] equation for determining local scour at bridges is the best available. However it appears to give excessive scour depths for wide piers.
- Pressure flow scour at bridge piers can increase scour depths by a factor of two to three. Pressure flow occurs when the lower bridge chord and deck become submerged. Preliminary methods for estimating pressure flow scour are given in HEC-18 [6].
- Flume studies and field experience show that the scour on an abutment caused by the upstream horseshoe vortex is twice as deep for vertical wall abutments than for spill through abutments.
- Although some of the flow conditions are different, scour at bridges over tidal waterways can be analyzed using the same equations and methods for non-tidal (riverine) bridges.
- Riprap is not a permanent countermeasure for pier scour. It can be used to protect existing bridge foundations from scour in conjunction with a scour monitoring or inspection program. New or replacement bridges must be constructed with foundations that are stable considering the total scour prism without the use of riprap.
- Instruments were developed for the real time measurement and monitoring of scour depths at piers and abutments by NCHRP Project No. 21-3 [9]. Monitoring of scour depths can be used to determine when scour at a bridge foundation becomes critical enough to close the bridge.

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HYDRAULIC MODELING FOR BRIDGE SCOUR ANALYSES IN TIDAL WATERWAYS

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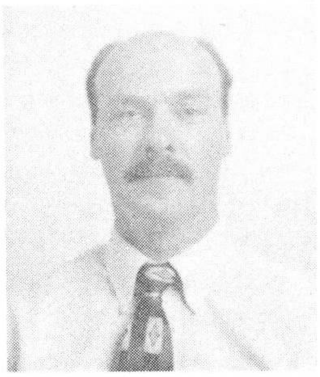

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SUMMARY

Tidal waters are subjected to dynamic flow conditions caused by daily (astronomical) tides, ocean currents, storm surges, and upland runoff. Accurate hydraulic information is necessary for calculating scour at bridge crossings, assessing channel stability, and designing bridge foundations and countermeasures. This paper presents guidance on simulating bridge hydraulics in tidal waterways. Selection criteria for 1- and 2-dimensional hydraulic models for tidal waterways are presented, and guidance is provided for developing appropriate boundary conditions.

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1. INTRODUCTION

Tidal waters are subjected to dynamic flow conditions caused by daily (astronomical) tides, ocean currents, storm surges, and upland runoff. Highway encroachments are subjected to stream instability and foundation scour resulting from these dynamic flow conditions. Although simplified methods for determining tidal hydraulic conditions often provide useful and reasonable results, complex hydraulic conditions may require unsteady flow computer modeling. Computer modeling is the most accurate method for determining the hydraulic conditions for extreme hurricanes that cause scour at many tidally affected bridge crossings.

In 1993, 12 east coast State Highway Agencies in the United States initiated a study to develop computer models to analyze coastal waterway hydraulic conditions at highway structures [1]. Phase I focused on three tasks: (1) compile a database of literature on tidal processes and computer models, (2) evaluate sources and methodologies for determining ocean tide and storm surge hydrographs, and (3) evaluate which computer models are best suited for use by bridge engineers for tidal hydrodynamic and scour investigations. Task 2 included determining the storm tide hydrograph, which consists of the storm surge height, the duration of the rise and fall, and superimposing the storm surge hydrograph on daily tides. Task 3 included accurate representation of bridge, culvert, and embankment overtopping hydraulics.

Phase II of this study [2] focused on three tasks: (1) developing storm surge hydrographs for the east and gulf coasts of the U.S., (2) developing case studies and testing selected models, and (3) developing a users manual and providing training. This paper summarizes model selection criteria and boundary condition generation methodologies developed during this study, and provides references to resources available for bridge scour analyses in tidal waterways.

2. MODEL SELECTION

The modeling approach should be selected based on the geomorphic and hydraulic characteristics of the tidal waterway [3]. Depending on the application, a simple tidal prism or orifice approach could be used. These approaches are presented in HEC-18 [4]. At times, a steady-state hydraulic model, based on the worst-case conditions determined from a simplified procedure, can be used to obtain conservative hydraulic parameters for scour analysis.

When the use of more sophisticated approaches is necessary, the model and approach will also vary depending on the site geomorphic conditions and hydraulic complexity. In Phase I of this study, 21 models were reviewed to determine their applicability to tidal bridge hydraulic and scour studies. It was anticipated that several models would be needed to efficiently model the range of conditions which are encountered in tidal waterways. One-, two- and three-dimensional models were evaluated.

Of the 21 original models, four were subjected to detailed evaluation. These included two 1-dimensional and two 2-dimensional models. The 1-dimensional models were DYNLET1 [5] and UNET [6]. The 2-dimensional models were FESWMS [7] and RMA-2V [8]. Each of the four models performed well for tidal hydraulic modeling. The models replicated observed tide gage readings well, generally within 0.12 m. The 1-dimensional models were easier to set up and ran much faster than the 2-dimensional models. Calibrated Manning n values for the inlet and bay areas were similar for all the tested models. The 1-dimensional models produced similar results to the 2-dimensional models, although it was anticipated that many complex hydraulic situations would require 2-dimensional modeling.

Because analyzing the hydraulics and scour potential at highway structures was the focus of the study, tests were performed of flow through culverts and bridges and over embankments. RMA-2V contained limited structure hydraulic analysis capabilities which consist of specifying various types of rating curves at structure locations. Since the specific geometric characteristics of a structure are not included directly as input, RMA-2V was not included in the structure hydraulic tests. The other models use various methods for computing structure hydraulics, and their performance varied significantly. UNET provided the best structure hydraulic computations. FESWMS performed well for embankment overtopping flows and some culvert conditions, but did not give reasonable results for bridge pressure flow. Of the three models tested for structure hydraulics, DYNLET1 gave the least acceptable structure hydraulic analysis.

Based on the results of the hydraulic tests, UNET (1-D) and FESWMS (2-D) were recommended for use in tidal hydraulic modeling of bridges. UNET was selected because it accurately simulates tidal and structure hydraulics. In comparison to the other models, UNET is most capable of modeling very long river reaches, including branched and looped channel networks. DYNLET1 performed well on tidal hydraulics, but was not as powerful as UNET, did not simulate structure hydraulics as well, and ran much slower than UNET. FESWMS was selected because it accurately simulates tidal hydraulics, adequately simulates many structure hydraulic conditions, and is well suited for simulating complex flow conditions. FESWMS has enhanced pre- and post-processing software [9]. RMA-2V is also well suited for tidal hydraulic modeling, and also has advanced pre- and post-processing systems. RMA-2V is currently being enhanced to include structure hydraulics. Once these enhancements are complete, FESWMS and RMA-2V will have comparable capabilities, and model selection will depend on site specific conditions of the waterway to be analyzed.

For tidal hydraulic modeling, the selection of the model and approach should be directed toward obtaining accurate results for the specific site conditions. Simplified methods have provided reasonable results for many locations with relatively little effort. More complex methods should be used when the limitations of the simplified approaches produce overly conservative, and often costly, results. UNET, DYNLET, FESWMS and RMA-2V have all been successfully applied to many complex tidal applications.

3. BOUNDARY CONDITION DEVELOPMENT

Tidal hydraulic studies require estimates of tide and storm surge stage hydrographs as boundary conditions. Upstream flood hydrographs may also need to be included, as well as wind stresses for some applications.

The Federal Emergency Management Agency (FEMA) and National Oceanographic and Atmospheric Administration (NOAA) publish peak storm surge elevations related to the frequency of occurrence or hurricane severity. Because FEMA's focus is on flooding potential, maximum surge elevations are reported, but the storm tide hydrographs are not available. NOAA reports peak surge elevations for each class of hurricane for use by emergency managers. Although the NOAA data provide an alternative to the elevations reported by FEMA, storm tide hydrographs are also not available from NOAA.

To address the fact that NOAA and FEMA provide peak surge height only and not the full hydrograph, Cialone et al. [10] reported a procedure for developing surge hydrographs from available information. The storm tide (storm surge combined with the daily tide) is computed as

$$S_{tot}(t) = S_p\left(1 - e^{-\left|\frac{D}{T-t}\right|}\right) + H_t(t) \tag{1}$$

where S_p is the peak surge height, D is the storm duration (defined as the radius of maximum winds divided by the storm forward speed), T is the time of the peak surge, t is time, and $H_t(t)$ is the daily tide component. Excluding daily tides results in a storm surge hydrograph symmetrical about time T. Depending on when the surge is assumed to occur during the daily tide, Sp is adjusted to produce a selected extreme condition, $S_{tot}(t)$, from NOAA or FEMA data.

Equation 1 was tested to see if it adequately predicted the shape of storm surge hydrographs. The ADCIRC [11] 3-dimensional model has been used to simulate numerous hurricanes along the east and gulf coasts. In the ADCIRC model, the surge is a result, not an input, so comparing the ADCIRC results with equation 1 is a reasonable test of the equation. Figure 1 shows the twelve largest storm surges predicted by ADCIRC for a 104 year historic record at Sapelo Sound on the Georgia Coast. Also shown is the 100-year surge predicted using equation 1. The daily tide is excluded from all of the hydrographs. The equation appears adequate for use in developing surge boundary conditions. The primary drawback of equation 1 is that negative surge elevations, due to offshore wind, are not predicted.

Judgment and experience are needed to determine whether extreme upland runoff should be included in a storm surge simulation. Where the timing of upland flooding is independent of the timing of the hurricane storm surge, average daily flow should be used as an upstream inflow condition. Where extreme upland runoff is generated by the hurricane conditions <u>and</u> the runoff can reach the tidal waterway during the surge, a more extreme upland flow could be included.

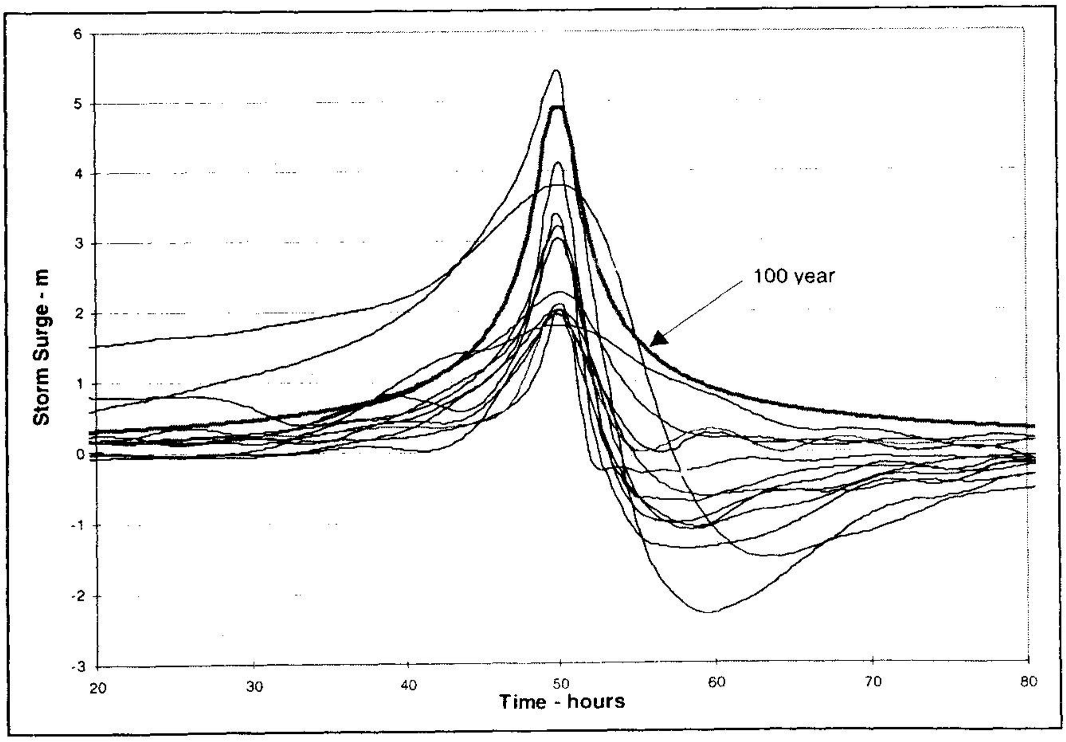

Figure 1. Comparison of design hydrograph with computed historic hydrographs.

4. RESOURCES AVAILABLE IN THE UNITED STATES

The primary product of the east and gulf coast study was a Users Manual for tidal hydraulic modeling of bridges [12]. The manual includes guidance on model selection, model development, data on hurricane characteristics, and case studies illustrating boundary condition development and the use of UNET and FESWMS. Also developed as part of this study is a CD-ROM which contains the selected models, electronic versions of the manuals, the case study input files, data and utility programs for model development and scour calculations.

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