EVALUATION OF SELECTED LOCAL SCOUR FORMULAE AROUND PIERS AND GROYNES

By

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A Research Project submitted to the Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, Dhaka in partial fulfilment of the requirements for the degree

of

MASTER OF ENGINEERING IN WATER RESOURCES ENGINEERING .

DECEMBER 1998

CERTIFICATE

This is to certify that this research work entitled, "**EVALUATION OF SELECTED LOCAL SCOUR FORMULAE AROUND PIERS AND GROYNES**" has been done by me under the supervision of Dr. M. Monowar Hossain, Professor, Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, Dhaka. I do hereby declare that this report contains no material which has been accepted for award of any other degree or diploma from any other institution elsewhere.

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DECEMBER 1998

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ABSTRACT

Scour around any obstruction is the lowering of a portion of the riverbed below its natural level. This is caused due to the large scale eddy structures or a system of vortices, which are developed near the obstructions. Though the mechanism of scour is difficult to quantify but its study gives considerable guidance in predicting the magnitude of scour around any hydraulic structure. The determination of scour depth is required for the design of any foundation of hydraulic structures for its safety.

This study was under taken with a view to testing the predicting performance of selected scour depth formulae against the data of scour depth of different piers of East-West interconnector, Hardinge Bridge, laboratory and model tests. For this purpose empirical formulae of Inglis (1949), Blench (1962), Ahmed (1962) and Shen et al (1969) were chosen to test the efficacy against the data of piers of East-West Interconnector and Hardinge Bridge. The study revealed that scour depth prediction by Blench formula indicated better correlation compared to other formulae.

Empirical formulae of Inglis (1949), Ahmed (1962), Blench (1962), Larra (1963), and Shen et. al (1969) were considered for prediction of scour depth using the laboratory test data of different piers. In this case performance of Blench formula appeared to be better compared to other formulae for a particular pier size as well as for different pier size. The study revealed that scour depth increases with the increase in angle of attack and the highest value was found for 30^0 angle of attack. The study further indicated that scour depth increase with the increase in pier size.

Empirical formulae of Ahmed (1953), Liu et. al (1961) and Breuser (1991) were tested against the laboratory data of groynes. Prediction by Breuser formula was found to be closer compared to other formulae in this case. Thus the study reveals that depending on the situation, the prediction capability of a particular formula could different.

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NOTATIONS

Symbols	Meaning	Dimension
b	Diameter of pier	m
D / D_{50}	Diameter of sand particle	mm
d_b	Mean depth	m
d_e	Lacey's regime depth	m
d_s	Scour depth	m
d_{se}	Equilibrium scour depth in uniform sediment	m
d_t	Total scoured depth below water level	m
f	Silt factor	-
f_b	Blench bed factor	-
F_r / F	Froude number	-
f_s	Blench side factor	-
h_o	Water depth before scouring	m
h_s	Scouring depth	m
H_s	Significant wave height	m
h_{sm}	Maximum depth in scouring hole	m
q	Discharge per unit width	m^3/sec/m
Q	Discharge	m^3/sec
U	Average velocity of flow	m/sec
U_c	Critical velocity of flow	m/sec
W_b	Blench regime width of channel at half depth	m
W_e	Width of water surface	m
y	Normal approach flow depth	m

ABBREVIATION

BUET	Bangladesh University of Engineering and Technology
BPDB	Bangladesh Power Development Board
BWDB	Bangladesh Water Development Board
DR	Discrepancy Ratio
FAP	Flood Action Plan
FPS	Foot Pound Second
HRD	Hydraulic Research Directorate
JMBP	Jamuna Multipurpose Bridge Project
MKS	Meter Kilogram Second
RRI	River Research Institute

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CHAPTER ONE

INTRODUCTION

1.1 General

Scour refers to the removal of material by running water. This usually occurs when the velocity of flow in the waterway exceeds the velocity, which will cause the bed material to move. Scour may occur progressively over the life of a structure or it may occur very quickly as a result of some relatively high flow. Many watercourses experience erosion and deposition and consequent changing of bed levels with time as part of their natural life cycle. The presence of a bridge can results in additional lowering of their bed at the bridge site, and it is this reduction in bed level which is generally meant by bridge scour. If the bridge has one or more piers sited within the waterway, or if the water level reaches one or both abutments, the structure then interacts with the river. The first effect will be a reduction in the natural breadth of the river channel at the bridge site. For a given flow, this reduction in breadth will result in a rise in water level upstream of the bridge and an increase in the mean flow velocity through the opening. If this increase in velocity results in movement or additional movement of bed material, the bed level in the vicinity of the bridge will be reduced. This reduction of level is known as scour due to bridge construction. In contrast, general scour may affect the whole breadth of the channel, and may extend both upstream and downstream of a river reach. The second effect of the interaction of the bridge with the water will be local disturbance to the flow. This may result in acceleration of the water past a bridge pier and increased turbulence, which can result in lowered bed levels close to the disturbing structure. This reduction in bed level is known as local scour. The definition sketch of local scour is shown in Fig. 1.1. Scour can also occur in coastal regions as a result of the passage of waves.

Scouring, in the sense of soil erosion, means loss of valuable asset. It should be emphasized that scour is not in itself usually a problem. Scour, or erosion, only becomes a problem when the lowered level of the river bed results in instability or failure of some structure founded in or close to the river. The most common structures, which interact with a river, are a bridge and river training structures. Failure of the foundations of a bridge pier or abutment is the single greatest cause of bridge collapse in the world. Scour is thus a potential problem throughout the life of any crossing founded on erodible material. With a new structure it has to be allowed for in the design; with an existing structure the potential has to be assessed and if necessary, steps taken to reduce the risk. Similar arguments are also valid for river training works.

Scouring of river bed in alluvial rivers endangers river bank and structures along the alluvial rivers. Sometimes important township, industries and infrastructures are threatened due to scouring associated with erosion. The river erosion threatens a number of townships like Sirajgonj, Kazipur, Sariakandi, Chandpur, Bhairab Bazar, Rajshahi, Sylhet, Kurigram, etc..

Involvement of a number of factors influencing the depth of scour makes the problem complex for estimation of scour depth analytically as well as experimentally. The most important factors which influence scour depth around pier and groyne are: shape and size of pier and groyne, angle of attack, flow depth and velocity, size and gradation of bed materials, bed slope, and fluid properties. In this sense, scouring is a complex phenomena. It varies from river to river due to variation of parameters like sinuosity and braiding intensity, bed slope, bank and bed materials and discharge.

The water level and velocity at a crossing, and the resulting possibility of scour, will be affected by the cross section area and by the geometry of the channel upstream and downstream of the crossing. Most formulae require knowledge of the upstream breadth and depth. Depth is difficult to determine, being dependent upon water level and the section shape. Both mean depth and maximum depths are important. Great care should be taken in attempting to estimate depths from measurements of breadth using empirical relationships.

A detailed knowledge of the scour pattern of different types of piers, caissons, abutment, groyne etc. are of great value when dealing with the design of bridges, barrages, training works, flood control works, channel improvement and so on. The safe and economical design of bridge pier, caisson, barrage, dam, retaining wall, flood wall etc. requires accurate prediction of the maximum scour depth of the stream bed around them.

1.2 Importance of the Present Study

Bangladesh is a land of rivers, interlaced by the numerous channels of the three mighty river systems of the world such as the Ganges, the Brahmaputra and the Meghna. Many structures have been constructed across these river beds. The East-West Interconnector, the Bangabandhu Bridge and the Hardinge Bridge are the most important among these constructed structures from the hydraulic point of view.

Scour at bridge piers and abutments has been regarded as a potential cause of bridge failure. Much effort has gone into trying to understand the mechanisms of scour and into developing methods of predicting the depth of scour. Yet even today new bridges still fail as a result of scour attack. The potential for scour is in some situations very obvious. In some situation, this is not so obvious. In such cases, other arrangement should be made beyond the exputed flow depth.

While knowledge of scour should now be sufficient among hydraulic and bridge engineer and methods of estimating scour in sands and gravels subject to a given flow are reasonably well developed, it is still the case that estimation of the likelihood of occurrence of scour in a real river situation is difficult. The effects of varying flow and varying material size, and the high turbulence in a real flood situation introduce areas of relative ignorance. Where scour is likely to occur to an extent, which could endanger the foundations of a bridge, adequate prevention and protection must be considered.

The East-West Interconnector, link between the east and west power grid system of Bangladesh crosses the river Jamuna approximately between Aricha and Nagarbari. It consists of eleven caissons. The diameter of the caisson is 35 ft (10.67 m) and the average depth of foundation of these caissons is 325 ft (99.09 m) from the highest water level, which is considered one of the world's deepest foundation. It transmit 230 kV through the national power grid of the country. Confluence of the Ganges and Jamuna with the position of East-West Interconnector, is shown in Fig. 1.2.

Bangabandhu Bridge is a multipurpose bridge constructed over the Jamuna River which connect the eastern and north-western part of the country. The bridge consists of bridge itself, to guide bunds and two hard points. The bridge having a curved length of 4.8 km and of which width is 18.4 m. It consists of fourty seven compound piers. The pile of this bridge are sunk at average depth of 85 to 100m below water level and the distance between the pile cap and bridge guarder is about 10 m.

The Hardinge Bridge is a railway bridge over the river Ganges. It consists of sixteen piers. Piers of this bridge are sunk at a depth of 160 to 180 ft (48.78 to 54.88 m) below water level. It was constructed between the year of 1909 and 1912.

These three examples show that enormous cost has been involved for large length of piers embedded beyond the scour depth. In future, there is possibility of some more new structures across these river systems. If scour depth is correctly predicted, then considerable amount of money can be saved. Therefore, it is felt that there is need to study scour phenomenon around bridge piers to obtain reliable data for evaluation of scour depth.

At present, different types of groynes are used as river training works. These groynes are normally constructed within the alluvial river. Groyne head is usually subjected to heavy scour. Therefore, provision should be made in the design so that the groyne does not fail due to scour.

Keeping this fact in view, this study was undertaken to predict local scour depth for pier and groyne on alluvial bed material. Field data of East-West Interconnector, Hardinge Bridge and laboratory test data of Kabir, (1984) and RRI (1995 to 1998) for pier and groyne were collected. These were analyzed and compared with the predicted value by few predictors of scour depth.

1.3 Objectives of the Study

The safe and economical design of bridge pier, caisson, groyne etc. requires accurate prediction of the maximum expected scour depth of the stream bed around them. An under-estimation of the scour will result in failure, while its over- estimation will increase the cost of the structures. Hence the knowledge of predicting scour depth under various conditions is essential. With this context the present study has been planned with the following objectives:

1. To estimate the maximum scour depth using selected scour formulae against the available data from piers and groynes in Bangladesh

2. To compare the estimated scour depth with the observed values and determine the predictive performance of the selected formulae.

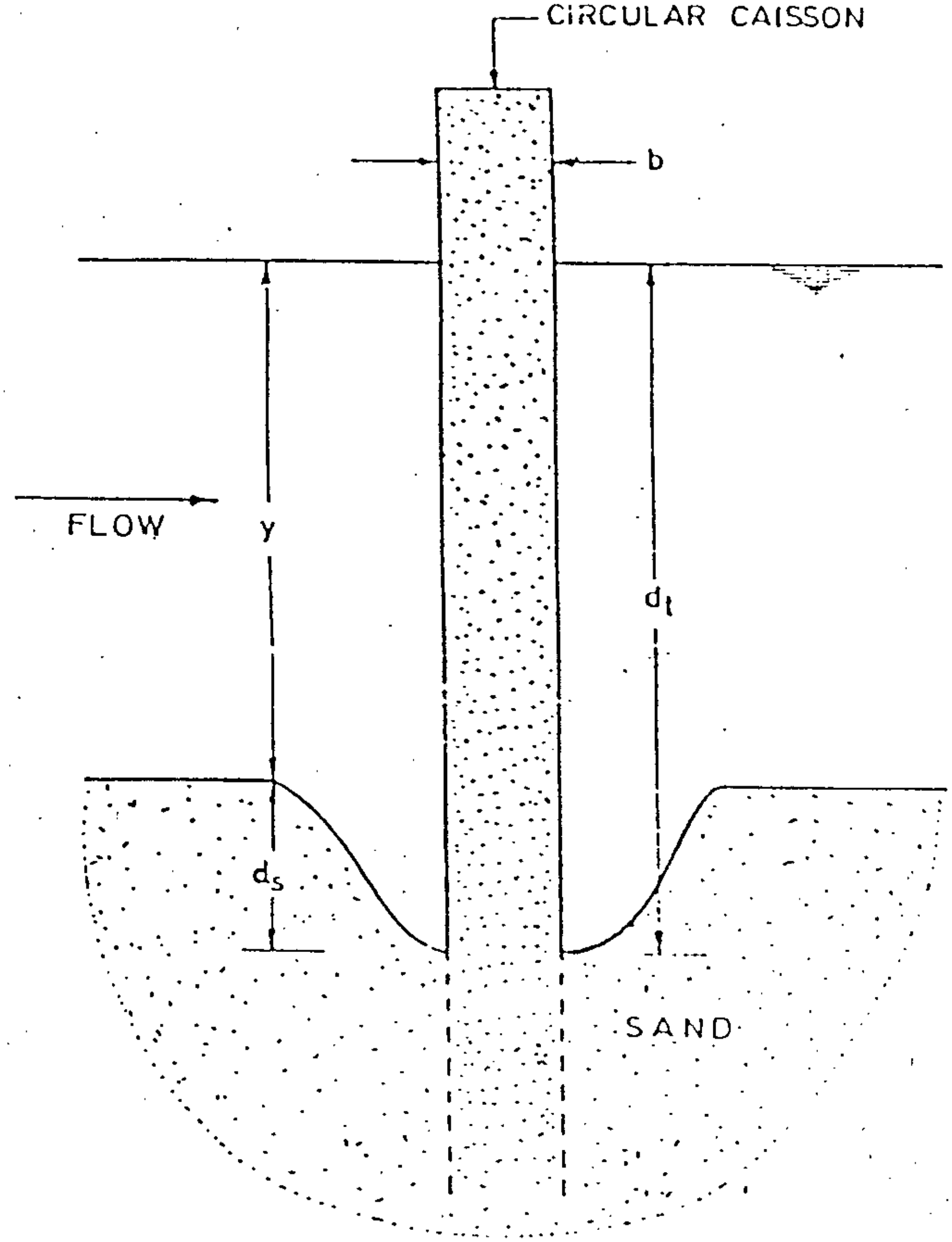

FIG. 1·1 DEFINITION SKETCH OF LOCAL SCOUR

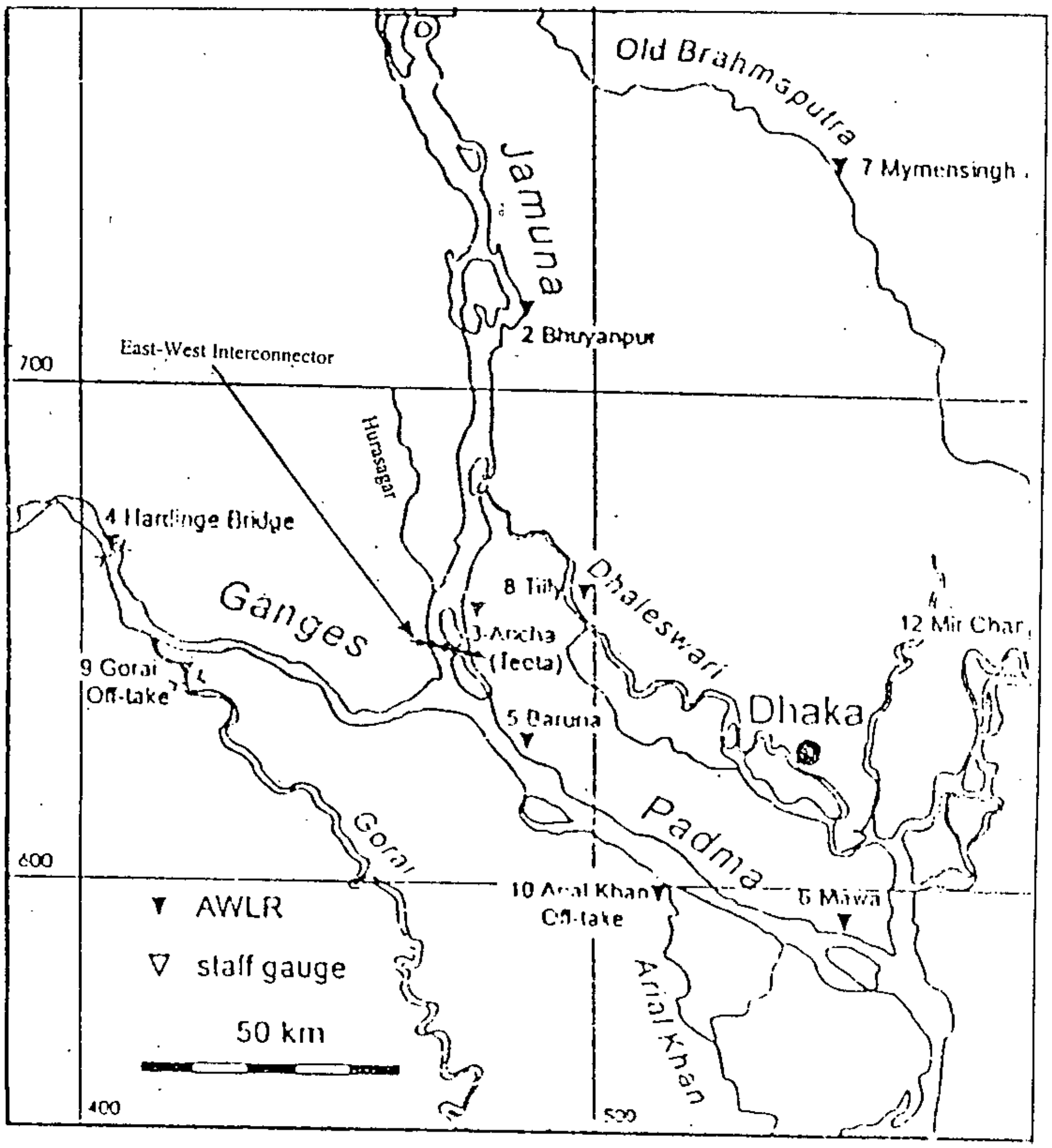

FIG. 1·2 : CONFLUENCE OF GANGES AND JAMUNA
(Showing the position of East-West Interconnector)

CHAPTER TWO

REVIEW OF LITERATURE

2.1 Introduction

The subject of local scour around bridge piers has been extensively studied. A literature survey of the various formulae for estimating scour depth was published by Breusers et. al (1977) and Raudkivi and Sutherland (1961). They found that various formulae gives widely differing estimates. These disparities have been discussed by Melvile (1974), Anderson (1974), and Hopkins et. al (1983). One of the main reasons for this lack of consistency lies in the failure of many investigators to distinguish between different sediment transport conditions on the approach flow bed and local scour beyond natural river bed level caused by an obstruction placed in the stream. Failure of bridges due to scour at their supports is a common occurrence. It may be due to scour at a pier or at an abutment. At times even scour of the river bank leading to the stream changing its course altogether and outflanking the bridge. Major scouring usually occurs during floods, that is, when the flow is unsteady. At low flow scour may take place due to obliquity. Additional problems are caused by floating debris or ice packs.

Chabert and Engledinger (1959) appear to have been first to describe the behavioral of scour phenomena with time and flow velocity. They showed that the clear water scour approaches equilibrium asymtotically, over a period of days, whereas the live bed scour develops rapidly in its depth fluctuations in response to the passage of bed features. Shen et al. (1969) suggested that the mean value of live-bed scour depth was about 10% less than the maximum clear-water scour depth. Raudkivi (1982) suggests that a second peak exists as shown by the dashed line in figure - 2.1.

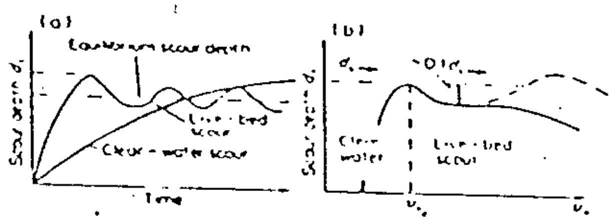

Figure 2.1: Scour Phenomena With Time and Flow Velocity

2.2 Classification of Scour

Scour is a natural phenomenon caused by the erosive action of flowing water on the bed and banks of alluvial channels. The types of scour, which may occur at a bridge site, could be grouped as follows:

1. General scour of the stream which would occur irrespective of whether the bridge was there or not.
2. Localized scour (or constriction scour) which may occur because of the constriction of the waterway and rechannelling of berm flow by the bridge.
3. Local scour which is caused by the local flow field around the piers and abutments.
4. Confluence scour which is caused by the local turbulence at the junction point of two or more rivers.
5. Reef scour occurs due to river bank erosion.

Chabert and Engeldinger identified two types of scour according to the condition of sediment transport in the approach flow:

(a) Clear-water scour, where material is removed from the scour hole, but not replenished by the approach flow.

(b) Live- bed scour, where the scour hole is continually supplied with sediment by the approach flow.

Local and localized scour can occur in one of two ways, such as clear-water scour and live-bed scour. Again local scour can be superimposed on both a general and a localized scour.

2.3 Local Scour Phenomenon and its Causes around Piers

2.3.1 Causes of Local Scour

Lowering of the stream bed in the vicinity of the obstructions can occur from a variety of causes. A useful distinction can be made separating the various causes into general categories (1) those characteristic of the stream itself, and (2) those due to the modification of the flow by the bridge crossing on any other structures.

The erosive power of flowing water on a channel boundary is determined primarily by the local shear stress or drag exerted by the flow on the boundary, and by the associated velocities and turbulent fluctuations of velocity near the boundary. The relationship of local velocities to cross-sectional average velocities is complex and depends on depth of flow, boundary roughness and channel geometry. Macro turbulent flow phenomena such as eddies, helicoidal flow, rollers and surges may also be important factors influencing scour. Average velocities and depths therefore give at best a very rough indication of erosive power, but calculations based on more refined measures are impracticable for the engineer in many cases. The factors, which appear to have more effect on modification of the flow, are size and shape of the obstruction and angle of attack. These factors also have effect on the scour phenomenon.

2.3.2 Flow Pattern at a Cylindrical Pier

The flow pattern past a cylinder, protruding vertically from a horizontal plane boundary, in uniform open channel flow is complex in detail, and the complexity increases with the development of a scour hole at the base of the cylinder. Studies of flow patterns for such a case have been reported by Hjorth (1972, 1975), Melville (1975), and Melville and Raudkivi (1977). The component features of the flow pattern are the downflow in front of the cylinder, cast-off vortices and wake, boundary layer, horseshoe vortex and bow wave, as shown diagrammatically in Fig. 2.2.

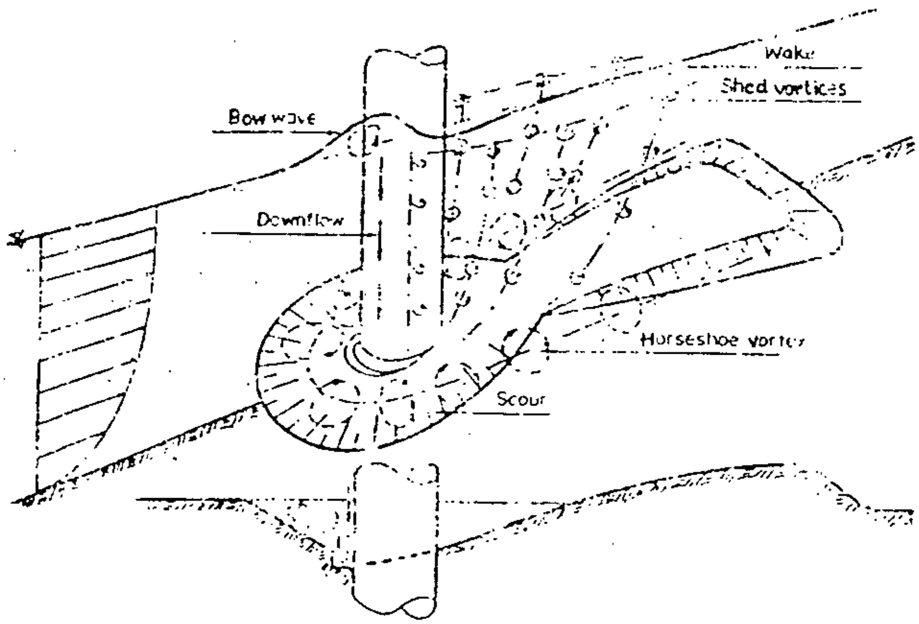

Fig. 2.2 : Diagrammatic flow pattern at cylindrical pier.

(Source: A. J. Raudkivi, 1990)

The so-called horseshoe vortex develops as a result of separation of flow at the upstream rim of the scour hole; it is a lee eddy similar for example, to the eddy or ground roller downstream of a dune crest. The horseshoe vortex is a consequence of scour, not the case of it. The horseshoe vortex extends downstream, past the side of the pier, for a few pier diameters before losing its identity and becoming part of general turbulence. The horseshoe vortex also pushes the maximum downflow velocity within the scour hole closer to the pier.

The separation of flow at the sides of the cylinder creates the wake with the cast-off vortices at the interfaces to the main stream. The cast-off vortices are translated downstream with the flow and interact with the horseshoe vortex at the bed causing it to oscillate laterally and vertically. The cast-off vortices also act as little tornadoes lifting sediment from the bed.

A bow wave develops at the surface with rotation in the opposite sense to that in the horseshoe vortex. The bow wave becomes importanrt in relatively shallow flows where it interferes with the approach flow and causes a reduction in the strength of the down flow.

The trailing vortex system (Fig. 2.3) occurs only when the pier is completely submerged. It is composed of one or more discrete vortices attached to the top of the pier and extending downstream.

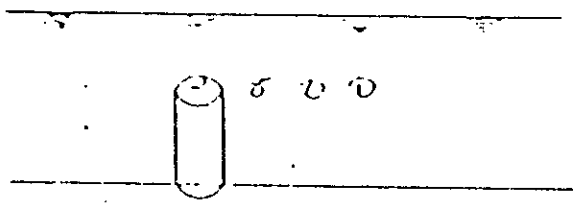

FIG. 2·3 TRAILING VORTEX SYSTEM

2.3.3 Effect of Sediment Grading

Raudkivi and Ettema (1977) showed that sediment grading has a strong influence on the equilibrium depth of clear-water scour. It is seen that the clear-water scour in typical river gravels with σ_g (geometric standard deviation) $\cong 3.5$ is only about 20% of the depth in uniform sediment. The effect of sediment grading on the depth of live-bed scour is considerably more complex. The armouring gradually increases the effective critical velocity of the bed surface and leads to a deeper equilibrium scour depth, up to the limiting critical shear velocity of the armoured surface, u_{*a}, the armour layer fails and the bed surface does not armour anymore. At shear velocities $u_{*c} < u_* < u_{*a}$ the sediment transport could be unsteady, approaching zero as the armour layer stabilizes, or it could be an equilibrium state.

If the flow conditions persists long enough at the critical shear velocity of the armoured surface and the transport rate goes to zero, the local scour depth approaches that for maximum clear-water scour conditions. The local scour depth under an equilibrium state of sediment transport depends on the armour finer sediment in transport and on the grading of the parent sediment, and is less than the clear-water peak value. On passing the critical shear stress, τ_{ca} , the surface layer fails and the sediment transport rate increases rapidly, leading to a reduction of local scour depth, analogous to transition from clear-water scour to live-bed scour.

After initial decrease, the scour depth starts to increase again with increasing applied shear stress, up to the transition flat bed condition. First, the coarser fractions of sediment are still effective in armouring the bed of the local scour hole, but their effectiveness decreases with increasing bed shear stress and vanishes at the transition flat bed conditions altogether, that is, if the largest stones in the sediment are not large relative to the width of the groove excavated by the downflow.

2.3.4 Effect of Flow Depth

There are several references in the literature stating that the scour depth is affected by the flow depth relative to the pier size; for example, the scour depth becomes independent of flow depth greater than three pier diameters. A decreasing depth of flow leads to increasing prominence of the surface roller (bow wave), which increases with the downflow and reduces its strength. No exact dependence has been obtained for the relative flow depth effect but data collected, both at clear-water scour conditions and live-bed scour at transition flat bed conditions, are shown in Fig. 2.4 as relative flow depth y_0/b versus k_y , where k_y is the factor by which the scour depth at the given pier in relatively deep water (unaffected by flow depth) is to multiplied to obtain the scour depth in shallow flow. For $b / d_{50} > 75$ and $y_0 / b < 3$ the k_y factor is approximately given by, $k_y = 0.77 (y /b)^{0.21}$.

2.3.5 Effect of Alignment and Shape

For pier shapes other than cylindrical, the depth of local scour depends on the alignment of the pier with flow. The local scour depth is related to the projected width of the pier, and this width increases rapidly with the angle of attack of flow. With an increasing angle

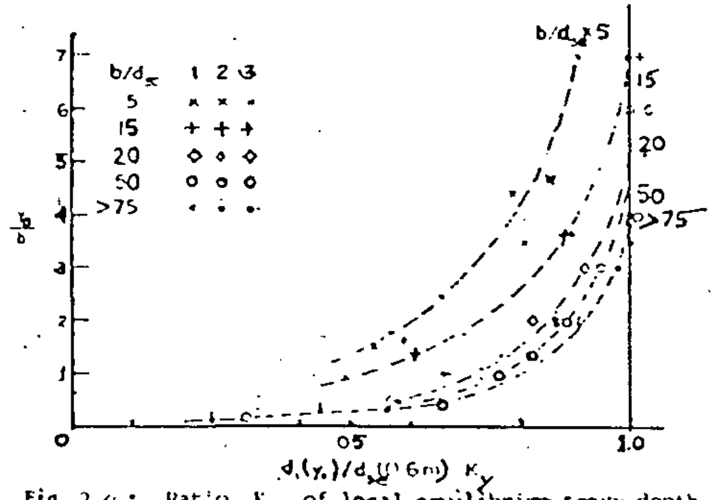

Fig. 2.4 : Ratio, K_{y}, of local equilibrium scour depth in shallow water to that in deep water relative to pier size (independent of depth) is function of y_o/b with relative sediment size as parameter (series 1 are clear-water scour data).

(Source : A. J. Raudkivi, 1990)

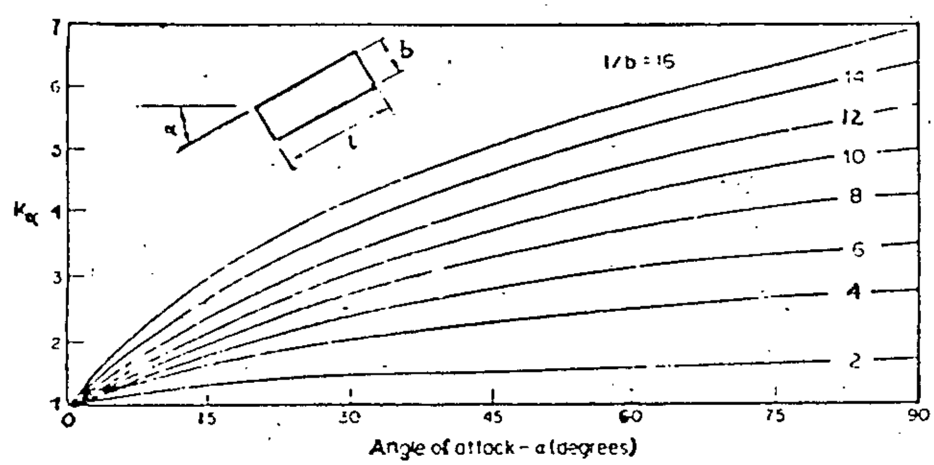

Fig. 2.5 : Alignment factor K_α for piers aligned with flow.

(Source : A. J. Raudkivi, 1990)

of attack, the location of maximum scour depth moves along the exposed side of the pier from the front to the rear end of the pier. The downflow on the exposed side combines with velocity into a strong spiraling current, which is a very powerful scouring agent, and excavates a deep hole near the end of the pier. In general, angles of attack greater than 5 - 10 ° should be avoided. Where this is not possible the use of a row of cylindrical columns would produce a shallower scour ; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder. The graph of multiplying factors for the depth of local scour as a function of angle of attack and the length- to- thickness ratio of the pier by Laursen and Toch (1956) is still commonly in use and indicates right orders of change, Fig.2.5.

There are also multiplying factors for pier shape , ranging from about 0.7to 1.2 ,but the shape effect will be over shadowed by small changes alignment or even a little debris caught on the pier.

Apart from the cross-sectional shape of the pier, the shape of the leading edge affects the scour depth. For example, laboratory results show that a 22.5 ° slope from vertical in the upstream direction increases the scour depth by about 20% whereas the same slope in the downstream direction decreases the scour depth by about 25% because the flow spills sideways (Raudkivi, 1990).

2.4 Scour Protection

The need for scour protection can be minimized by locating hydraulic structures on stable tangential reaches of channels and by placing foundations on non-erodible materials. But, such a solutions is not always practicable, economical, or desirable from an alignment standpoint of a structure. Even, the choice among many protective works and other alternatives, depends on several factors including load bearing requirements, sub-soil conditions actually encountered, economies, feasible constructions methods and schedules, inspection procedures etc.

Responsibility for ensuring the safety of bridges rests with the relevant bridge authority. Attention must also be given to structures which under normal conditions do not span watercourses, but which in flood conditions can span flowing water and as a result be liable to scour. It is of considerable assistance to the bridge engineer to have as much background information as possible about a structure prior to site inspection. As much historical information as can be obtained about previous flooding should be investigated. No matter how often bridges are inspected, it is unlikely that inspection will coincide with incidence of scour. The best that can be achieved is regular inspections, backed up by additional inspections immediately after severe flooding. The conventional and still most reliable method of scour inspection is manual/visual checking, usually by diver. The scour survey should be undertaken from the water surface using measuring rods, weighted tapes or in deeper rivers and estuaries, electronic sounding devices. Two sets of scour measurements should be taken around each structure, one to establish local scour and the second to establish general and overall scour. The general scour soundings have to cover a larger area. The survey report must give the fullest possible details of the structure and the river. Data should be presented as location and contour plans, cross sections and photographs. Previous depths and locations should be indicated and areas of scour shown for comparison.

Of paramount importance in determining the possible effects of scour is knowledge of the foundation depths and the nature of the river bed material extending down to foundation level. The established methods of ascertaining this information are; (i) by reference to 'as built' drawings and previous site surveys, (ii) by excavation adjacent to the pier or abutment, (iii) by coring, both vertically through the river bed and inclined through the pier or abutment.

Various other methods have been tried and/or are being developed. None are yet proven . They include radar, sonar and in-situ probes. In-situ probes, including accoustic, magnetic and nuclear, are claimed to be suitable for determining the depth of piles, integrity of metal structures and foundation depth.

Once the possibility of scour, or the existence of scour, has been identified, and the scour is to a depth, which is likely to result in failure of the structure, various steps can be taken to reduce the risk. These include improving the structural integrity of the foundation, reducing the approach velocity to below the threshold velocity of the bed material, and raising the size of the bed material such that the threshold velocity is above the approach velocity.

2.4.1 Reduction of the Approach Velocity:

The velocity of the approaching flow can be reduced by increasing the cross-section of the channel upstream, by realigning the channel such that flow is parallel to bridge piers. It may also be possible to keep potential scouring velocities clear of abutments or piers by training works such as groynes, while guide banks may be formed on a flood plain to steer the overbank flow into the main or flood relief openings. With any guide wall, the downstream side should not be forgotten. The placement of structures such as groups of piles upstream of a bridge pier has also been shown to be beneficial in reducing approach velocities.

2.4.2 Bed Protection:

The most useful way to prevent scour is to reduce or eliminate the erodibility of the bed material in the area of the pier or abutment. This can be achieved by driving a sheet pile skirt around piers or abutments, or by replacing or covering the natural bed material with rigid or flexible protection. Sheet piling is attractive in that it can give high security against future failure. Rigid protection would include concrete, masonry and brick. This would be laid across the full breadth of the channel to form an invert if general scour is the problem, or as a footing, apron or plinth around a pier for protection against local scour. Flexible bed protection includes loose stone, cement bags, slag, rubble or rock, and similar materials contained within a wire or plastic mattress, Gabon or mash. This form of protection should be carefully designed and material should not just be randomly tipped from the bridge. With loose material, the size and size grading requires to be carefully selected with knowledge of the flow velocity.

2.5 Previous Investigations of Experimentally Derived Scour Depth Formulae for Piers

Since the local scour phenomenon is very complex due to the involvement of numerous variables, it is almost impossible to study the effect of all the variables simultaneously. Many investigators in the past had studied the effect of one or more parameters on the magnitudes of scour around hydraulic structures. Most experimental studies have resulted in formulae intending to predict the dimension of scour depth around bridge piers.

Inglis (1949) has developed a mathematical relationship from the experiments conducted at the Central Water power Irrigation and Navigation Research Station, Poona, India in the period of 1924-1942 with the objective to determine means of protecting the piers of the Hardinge Bridge, located on the river Ganges at Bheramara against scour. Geometrically similar models of Hardinge Bridge piers were set into the center of the parallel sided channel having a bed of Ganges sand of mean diameter 0.29 mm. Model scales of 1:65 and 1:210 were used. From this experiment he obtained the equation of the following form:

$$d_t/b = 2.32\,(q^{2/3}/b)^{0.78} \tag{2.1}$$

Where, d_t = Total scoured depth below high water level in meter.

b = Width of pier in meter.

q = Discharge per unit width of the approach flow in m^3/sec per meter width.

As it was difficult to correlate this with the depth of scour at prototype piers, due to intensity of discharge per meter (q) depending upon the curvature of the river upstream which varies from river to river, it was considered desirable to study cases of actual scour in prototypes and workout a general empirical relationship. Data were, therefore, collected for scour around bridge piers and a general relationship was proposed as follows:

$$d_t = 2[0.473(Q/f_e)^{1/3})] \tag{2.2}$$

$$= 2\, d_e \text{ (Lacey)}$$

Since Lacey's regime depth, $d_e = 0.473(Q/f_e)^{1/3}$ where, Q is the maximum discharge in cusecs, d_e is the maximum depth of scour below highest flood level, f_e is equal to $1.76\sqrt{D_{50}}$.

Blench (1962) reviewed the model results reported by Inglis. The data of severe scour around piers were for discharge ranges from 30,000 cfs to 2,600,000 cfs. Blench has reported the Inglis data along with model data of his own and of workers at the university of Alberta, and stated that the formula presented by Inglis (1949) could be reduced to the form:

$$d_t/y = 2.43\ (b/y)^{0.25} \qquad (2.3)$$

Where y = normal approach flow depth in m. Blench did not elaborate on this formula, and it was not immediately evident how he derived it from equation (2.1). However, it was later revealed that regression equation was developed with the field data.

Shen, Schneider and Karaki (1969) conducted numerous sets of experiments at Colorado State University, USA. The flume used was 60 feet long by 6 feet wide and a cylindrical pier of 0.5 foot diameter was placed on a sand bed with mean bed material size of 0.24 mm. In addition to their experimental data the authors also incorporated in their analysis the data of Chabert and Engeldinger, 1956, for piers of 0.246 foot and 0.328 foot diameters respectively. They obtained the following relationship:

$$d_s/y = 2[F^2(b/y)^3]^{0.215} \qquad (2.4)$$

Ahmed (1962) conducted experiments on models of several bridge sites of Pakistan. The experiments were carried out so that there was good general movement of the bed. The relationship of discharge intensity to total scoured depth, both around the piers and abutments and in bends, was investigated and he proposed a formula of the following type:

$$d_t = 1.486\ K.q^{2/3} \qquad (2.5)$$

Where the coefficient K is a function of boundary geometry, shape of pier nose or abutments, characteristics of bed material and distribution of velocity in the channel section at the piers. Ahmed (1962) recommended that a value for K of 1.7 to 2.0 be adopted for use in predicting the total scoured depth around piers, provided that guide banks at least equal to the length of the river crossing were constructed to provide and maintain straight channel flow characteristics in the vicinity of the piers, thus precluding the development of severe channel bends and the associated extreme scour depths in the critical areas around bridge piers and abutments.

Arunachalam (1965) reviews several of the more widely used methods of calculating local scour and attempts to explain the major differences between them. He claims that the different reference points used by past investigators for measuring the depth of scour is the main cause of confusion surrounding the relative effects of flow velocity and pier geometry on local scour. He suggests that the total scoured depth below water surface must be separated into the upstream flow depth, which is a function of velocity or discharge and the depth of local scour the ambient stream bed which is a function of only the depth of flow and pier geometry. He derives a relationship for calculating the depth of local scour below stream bed based on the Poona experiments on the Hardinge Bridge piers, but rewritten to express the discharge in terms of depth. His expression is

$$ds/b = [y/b*1.95/(y/b)^{1/6}] - 1 \qquad (2.6)$$

Arunachalam recommends to use the 'Lacey mean depth' for calculating the upstream flow depth.

Bata (1960, after Acres 1970) reported results of a series of experiments on local scour around bridge piers. During this experiments the general bed movement existed. However, data on widths of flume, size of land, depths and velocities of flow, and shapes and sizes of piers tested were not quoted. Bata found that the influence of the Reynolds Number on local scour was negligible, as was the diameter of the sand particles. Finally he put forward the following expression for the limiting depth of local scour:

$$\frac{d_s}{y} = 10\left(\frac{v^2}{gy} - \frac{3D}{y}\right) \tag{2.7}$$

Where D = diameter of sand particle ($D = D_{50}$)

Tarapore (Acres 1970) established a theoretical method of calculating local scour based on the potential flow around the obstruction in the channel. He attempted to verify his theoretical findings with a series of experiments conducted in a 12 inch wide flume, using a cylindrical pier of two inches in diameter. Since his theoretical investigation took an advanced rate of bed load transport into account, it is assumed that his experiments were conducted with a fully active bed. He presents the relationship:

$$d_s = 2.7\ (b/2) \tag{2.8}$$

He claims, this equation is valid for large depth of flow. However, a criterion for determining whether the depth of local scour was found to be dependent on the depth of approach flow as well as the radius of the pier. Suspended load was found to have no effect on the depth of local scour.

Larras (Acres 1970) presents a formula for the depth of local scour at bridge piers based on the experimental results of Chabert and Engeldinger (1956). The formula given is:

$$d_s = 1.42\ b^{3/4} \tag{2.9}$$

Multiplying factors to be applied to the basic formula to account for pier nose shape and for angle of attack of the approach flow were given by Larras.

The relationship between scour depth at cylindrical bridge piers founded in cohesionless sediments, and mean approach flow velocity is defined for flows above the threshold of particle motion by Melville. The experiments were conducted in a tilting flume, 14.8 m long, 0.6 m wide and 0.4 m deep, at the school of Engineering, University of Auckland. Experimental data are collected in which pier size, sediment size and mean approach flow velocity are systematically varied. Based upon the results from this study together with Ettema's data, the generalized scour depth versus flow velocity relationship shown in Fig. 2.1 has been prepared. The general shape of this curve was first deduced by Raudkivi. The relationship is multivalued exhibiting scour maxima at both the threshold condition and the transition flat bed condition of bed motion. The occurrence of the first maximum is consistent with the findings of previous investigations. The second maximum, however, has not previously been identified, although it is shown that some data from other researchers are consistent with this functional shape. The effect of particle size is found to depend on whether the particular bed sediment does, or does not, form ripples and consequently separate functions have been identified for each sediment size.

Maximum scour depth for uniform nonripple-forming sediments occurs at the threshold of motion condition, as found by previous investigators. However, for uniform ripple-forming sediments, the maximum scour depth occurs at the transition flat bed condition. The data show that overall maximum scour depth to pier diameter ratios are up to 2.4 to 2.5. The effects of sediment nonuniformity have not been considered in this investigation. However, it is to be expected that local scour depths would be reduced from those for uniform sediment of the same mean size in situations where armouring occurs.

Maximum clear-water equilibrium scour depth, d_{se}, for sediments with a shape factor near one, was found to depend on the grading, σ_g, of the sediment. Results of earlier studies were reported by Raudkivi and Ettema. Their results indicate that value of the equilibrium scour depth, d_{se} (σ_g), for nonuniform alluvial sediments can be estimated in terms of the geometric standard deviation, σ_g, of the sediment form.

$$[\, d_{se}\,(\sigma_g)] \,/\, b = K_\sigma * d_{se}/b \qquad (2.10)$$

In which d_{se} = the equilibrium scour depth in uniform sediment ($\sigma_g \cong 1.0$)

b = pier diameter

K_σ = a function which depends on σ_g

For a cylindrical pier in a uniform grain size sediment, with a scour process uneffected by relative flow depth or relative grain size that the maximum depth of clear-water local scour is

$$\left[\frac{d_{se}(\sigma_g)}{D} \right]_{max} \approx 2.3 \qquad (2.11)$$

Thus for nonuniform sediment, Eq. 2.10 becomes

$$[d_{se}(\sigma_g) / D]_{max} = 2.3\ K_\sigma \qquad (2.12)$$

Where $u_*/u_{*c} \leq 1.0$.

In view of the ability of nonuniform sediments to develop a protective armour layer on the bed surface, as well as in the scour hole, clear-water local scour may continue to occur for flows with shear velocities greater than that required for the entrainment of the sediment were in uniform. That is, the clear-water scour condition could occur for values of shear velocity such that $u_*/u_{*A} < 1$; where u_{*A} is the shear velocity of the flow required to erode the armour layer. As equilibrium scour depth occurs when the flow can no longer entrain particles from the base of the scour hole, the maximum equilibrium depth of local scour for a nonuniform sediment occurs when $u_*/u_{*A} \cong 1$ and the maximum equilibrium depth can be estimated from Eq. 2.12.

Chabert and Engeldinger (Rao 1974) found that depth of scour varies almost linearly with tractive force for clean water and reaches a maximum value of the tractive force necessary for continuous transport. They found that the depth of scour varied in a similar way with mean velocity.

Laursen (1962) summarized that Joglekar, Chaitale, Thomas, Ahmed, Blench and Bradely all in that the depth of scour is proportional to two thirds power of the discharge per unit width which may be expressed in the following way :

$$d_e = \frac{q^{2/3}}{F_b^{\ 1/3}} \qquad (2.13)$$

Where q is the discharge intensity and F_b is the appropriate bed factor. Since the unit discharge is the product of depth and velocity, then for a given depth of flow the depth of scour should vary with velocity.

Kandasamy and Melville (1998) proposed a simple equation that can be used to predict the maximum local scour depth at either piers or abutments aligned perpendicular to the flow is as follows:

$$d_s / K_s = Ky^n L^{1-n} \qquad (2.14)$$

Where, $K = 5, n = 1$ $\quad 0.04 \geq y/ L$

$K = 1, n = 0.5$ $\quad 0.04 < y / L < 1$

$K = 1, n = 0$ $\quad y / L \geq 1$

K = coefficient

K_s = shape factor depending on L/y

L = the maximum lateral dimension of the pier or abutment

2.6 Local Scour Around Groynes

2.6.1 General

Groynes or spurs are stone, gravel, rock, earth or pile structures built at an angle to a river bank to deflect flowing water away from the critical zones, to prevent erosion of the bank, to establish a more desirable channel for flood control, navigation etc. They are used in wide braided rivers to establish a well-defined channel that neither aggrades and degrades nor shifts its location from year to year. In this case, the spurs may have long dikes at their outer end to help define the channel and many miles of a river are controlled by spurs. Spurs are also used on meandering rivers to control flow into or out of a bend or through a crossing (Richardson, 1975). Spur can be considered as blunt abutment. It also constrict a part of river bed.

Groynes may be aligned either perpendicular to the bankline or at an angle pointing upstream or downstream. A groyne pointing upstream has the property of repelling the river flow away from it and is termed a repelling groyne. When the groyne has a short length, usually it changes only the direction of flow without repelling it. This kind of groyne is known as deflecting groyne and gives only local protection. On the contrary, a groyne pointing downstream attracts the river flow towards it, and is called an attracting groyne. Straight and other types of groynes are represented in the following figure (Fig. 2.6).

2.6.2 Flow Pattern Around Groynes

Flow constrictions, whether man made or natural, occur whenever there is a significant reduction in the width of the canal or river bed. The simplest case is the long contraction in alluvial channels. Another typical example is the diversion of a river course by construction of a spur dike or groyne either to protect the banks from erosion or to train the mainstream along a desired alignment. All such structures concentrate river flow and introduce 3-D flow patterns and can therefore cause local scour.

Due to the flow concentration and the local disturbance to the flow in the vicinity of the structures, scouring of the alluvium within and around the constricted part usually occur. In hydraulic engineering, the extent of this erosion and the disturbance caused by these structures to the river flow is important.

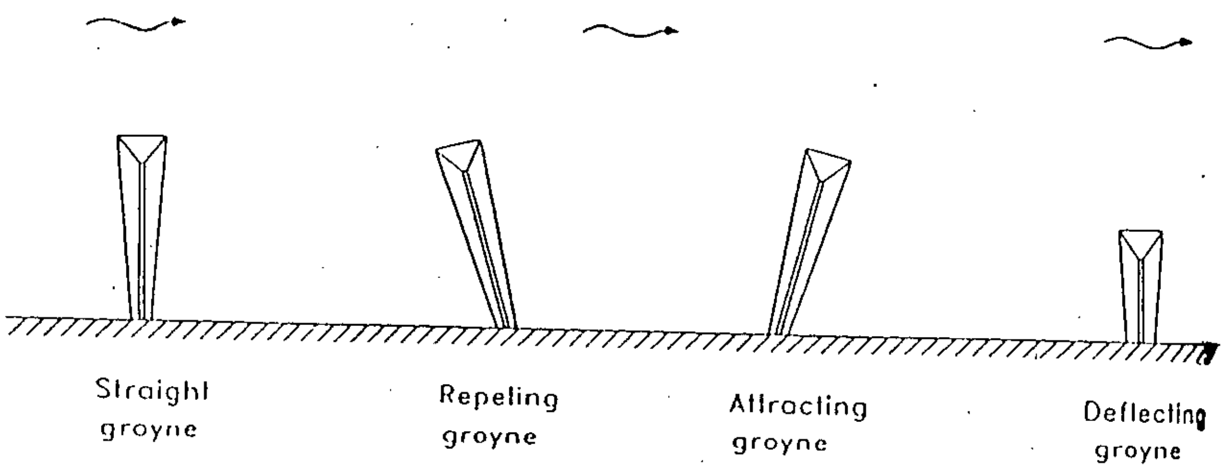

Fig. 2.6-1: Various types of straight groynes

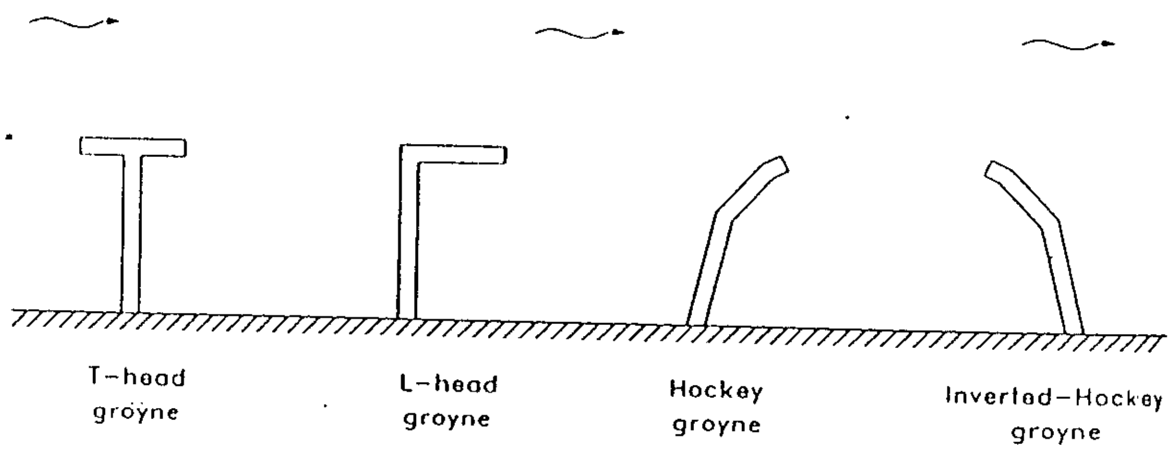

Fig. 2.6-2: Other types of groynes

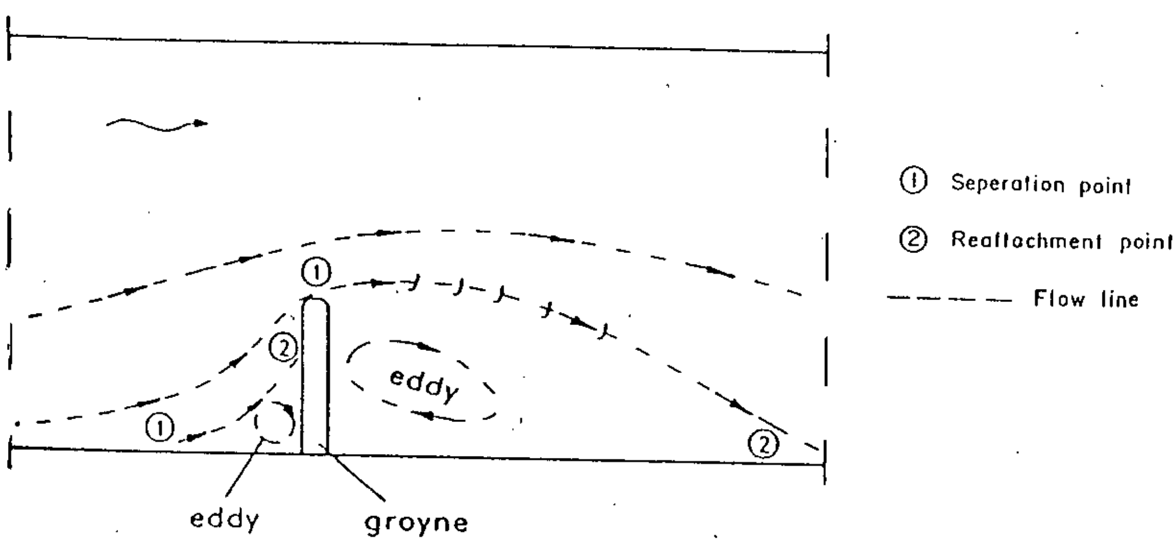

Fig. 2.7 : Flow around a groyne

The flow field around a spur dike is characterised by an acceleration from upstream to the most contracted cross-section somewhere in, or just downstream of the head of the groyne, followed by a deceleration of the flow. A separation point of the flow is located at the head of the spur dike. Downstream of the groyne, the main flow is separated from a large eddy by a vortex street. Downstream of the reattachment point normal flow will be established again. In an extreme situation, Just upstream from the spur dike a separation point and a small eddy can exist. Then, on the upstream side of the groyne there is a reattachment point , (Fig. 2.7); the down flow at the groyne with vertical walls can generate a strong spiral motion near the bed.

As the water flows around the spur dike, the flow pattern is changed due to the reduction of width of channel, and as a result of this, the shear stress distribution around the groyne is changed. This leads to scouring action until equilibrium is established between the various forces influencing the scouring action. In other words, local scour around spur dike results from acceleration of flow and vortex systems induced by obstruction to the flow. As the scour depth increases, vortex strength reduces, transport rate reduces and an equilibrium is established and finally scouring ceases. Although the vortex system is known to be the cause of local scour, it has not been possible as yet to calculate the strength of vortex and relate the velocity field with subsequent scour. In general, the stability of the not-submerged spur dike is endangered by a scour hole due to the strong vortices just downstream of the separation point at the head of the spur dike. A too steep scour hole can induce a sliding of a part of the head of groyne into the scour hole or can undermine this head.

2.6.3 Local Scour Formulae for Groynes and Spurs

Many researchers have worked on the various aspects of the hydraulic characteristics of groynes, especially about the estimation of the maximum local scour depth at the head , or just downstream of the groyne. **Breuser(1991)** recommends a scour depth as follows:

$$h_o + h_{sm} = 2.2\left(\frac{Q}{B-b}\right)^{\frac{2}{3}} \qquad (2.15)$$

Where, h_o = Water depth before scouring

For spurs with any angle with the flow multiplying factors are shown below to be applied. When the head of the spur is not vertical but has a slope of 45^0, the depth of scour is 15% lower. With very gentle slope a reduction of 50% may be made. Spurs constructed at the concave bank have higher scour depth of the order of 10 to 50% more, depending on the bend radius.

Table- 2.1 : Multiplying factors for different $\alpha(^o)$

$\alpha(^o)$	Factor
30	0.8
45	0.9
60	0.95
90	1.0
120	1.05
150	1.10

Garde et. al (1961) conducted an extensive study on plate-form spur dikes in a 2 ft width flume and studied the effect of flow, spur dike and sediment characteristics. It was found that the Froude number of the normal channel, the opening ratio, the angle of inclination of the groyne, and the averaged drag coefficient of the sediment particle adequately represent the influence of flow, spur dike and sediment characteristics on the maximum scour depth. They found the equation as follows:

$$\left[\frac{h+\Delta h_{t}}{h}\right] = k\frac{1}{\alpha}F_{r}^{n} \tag{2.16}$$

In which K and n are functions of C_D (Fig. 2.8)

$$C_{D} = \frac{4\Delta_{s}D}{3W^{2}\rho}$$

In which

Δs = difference of specific weight between sediment and water

W = settling velocities of sediment

α = opening ratio = [(B-b)/B]

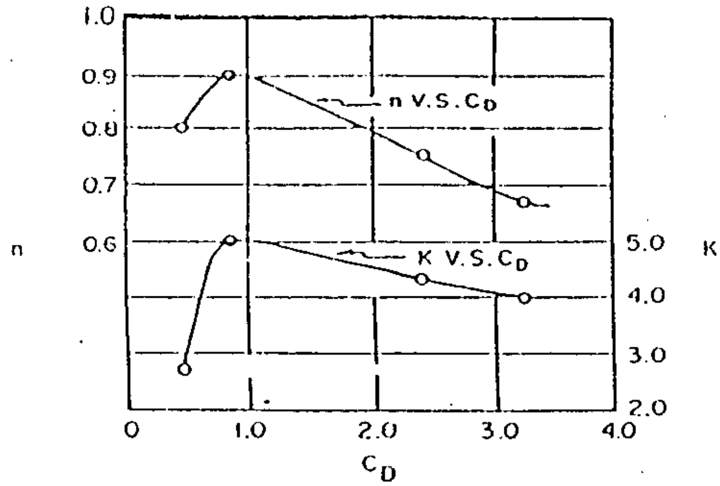

Fig. 2.8 : Variation of n and K with C_D (Source: FAP 21/22, Jan. 1993)

Liu et al (1961) proposed the following formula on the basis of dimensional analysis and laboratory test for abutment and spur.

$$\frac{h_{se}}{h_o} = 2.15\left[\frac{B_s}{h_o}\right]^{0.4} F_r^{\,0.33} \tag{2.17}$$

where, B_s = width of a scour hole

They recommend 30% increase of h_{se}/h_o for maximum scour depth.

The regime methods of scour depth prediction at the head of a spur or an abutment depends on correlation with the field observations. **Inglis (1949)** related the total mean depth $H_s = h_s + h_o$ to the regime depth of Lacey, relying heavily on Indian data

$$H_s = 0.47\left[\frac{Q}{f}\right]^{\frac{1}{3}} \tag{2.18}$$

Where, $f = 1.75(d_{mm})^{0.5}$. To estimate the maximum scour depth, a multiplying factor must be applied, provided by Indian Code as follows :

Table-2.2 : Multiplying factors as per by Indian Code

Nature of location	Factor
Straight reach of channel	1.25
Moderate bend	1.50
Severe bend	1.75
Right angle abrupt turn	2.0
Nose of piers	2.0
Along walls, abutment	2.25
Noses of guide banks	2.75

Using Lacey's formula, the scour depth can be related to the discharge intensity by the following relation:

$h_o + \Delta h_1 = 0.90\ [q^{2/3}/f^{1/3}]$

Where, q = the discharge per foot width of main channel

f = silt factor

Gill (1972), conducted numerous experiments in a 2.5 feet wide flume ,to study the effect of grain size on bed material, flow depth and also the effect of involving bed load movement of flow on the depth of equilibrium scour around spur dikes. He concluded that the depth of equilibrium scour is affected by the size of bed material and also the depth of uniform flow upstream of the groyne location. The combined effect of both of these have been empirically formulated as:

$$\alpha = 8.375\left[\frac{D}{h_1}\right]^{0.25}$$

Where, h_1 = depth of flow at the upper uncontracted channel

Also he found that the depth of maximum scour occurs when the sand bed upstream of the spur is at threshold of movement and for the purpose of modelling and basic research, it is necessary the distinction between clear water scour while the scour caused by bed load transporting flow should be considered. To determine the maximum depth of scour, Gill introduce the following formula:

$$\frac{h_{s1}}{h_1} = \alpha\left[\frac{B_1}{B_2}\right]^{\frac{6}{7}}\left[\frac{1}{\left(\frac{B_1}{B_2}\right)^{\frac{1}{6}}\left(1-\frac{\pi_c}{\pi_1}\right)+\frac{\pi_c}{\pi_1}}\right]^{\frac{3}{7}} \qquad (2.19)$$

$1 - \pi_c/\pi_1 = 0 \quad$ for $\quad \pi_c/\pi_1 \geq 1$

Da Cunha (1971) performed a great number of tests with various vertical-side headed groynes and water depth in the range of 0.04 to 0.14 m with velocities between 1.0

and $1.4U_c$. He found the maximum scour depths around the head of the groyne from the following formula:

$$\frac{h_s}{h_o} = 1.65\left[\frac{b}{h_o}\right]^{0.3}\left[\frac{h}{h_o}\right]^{0.95}\left[\frac{w}{U_*}\right]^{0.15} \tag{2.20}$$

Where, U_* = shear velocity

h_o = depth of uniform flow

Ahmed (1953) proposed a formula to predict scour depth at the nose of spur in terms of regime depth of Lacey as follows:

$$H_s = 1.34\left(\frac{q^2}{f}\right)^{\frac{1}{3}} \tag{2.21}$$

where q is the discharge per unit width in the contracted section. The associated co-efficient $K = \frac{H_s}{q^{\frac{2}{3}}}$ values in terms of angle of spur measured from downstream flow direction were given as follows :

Table-2.3 : Values of *Ks* for different α(°)

α(°)	30	45	60	90	120	150
K	1.8	2.0	2.15	2.25	2.4	2.45

2.7 Empirical Approach of Analytical Design :

In this approach to predict the depth of local scour, a stable or regime depth of flow is estimated for the unobstructed river channel using an empirically derived formula and a multiplying factor then applied to this estimated depth to account for obstructions such as piers and abutments. The following description is based on the approach by Lacey and Blench. Their formulae are based on the analysis of the numerous volume of data obtained from the operation of irrigation canals in India and Pakistan.

2.7.1 Lacey's Regime Depth Formula :

Lacey's (Acres 1970) has given an expression for regime depth in rivers flowing in alluvium in 1929. The equation of regime width is,

$$W_e = 2.67Q^{1/2} \tag{2.22}$$

and that for depth

$$d_e = 0.473\ (Q/f_e)^{1/3} \tag{2.23}$$

Where, W_e = Width of water surface

Q = Flood discharge

d_e = Lacey mean depth

f_e = Lacey's silt factor, defined by $1.76\sqrt{D_{50}}$.

Lacey proposed multiplying factors to be applied to the " Lacey mean depth" in order to calculate the maximum total scoured depth, which could occur in stable reaches with no obstructions. Factors are shown in the following table,

Table-2.4 : Multiplying factors proposed by Lacey

Condition	Factor
Maximum depth in a straight, stable reach	1.27
Moderate bend in the river	1.50
Severe bend in the river	1.75
Right angled bend	2.00

For reaches where the width of the river was artificially constricted, such as by guide banks. The "Lacey mean depth" becomes :

$$d_e = 0.9\frac{q^{\frac{2}{3}}}{f_e^{\frac{1}{3}}} \tag{2.24}$$

Same multiplying factors as used on an unconstricted reach are to be applied to obtain the maximum total scoured depths. These equations have apparently been checked against actual conditions at stable bridge sites in large sand bed rivers with good agreement.

2.7.2 Blench Regime Formula:

Blench (1966) has replaced the silt factor in Lacey's formula and introduces the 'bed factor' and a 'side factor'. Blench has differentiated the nature of channel bed and banks. His equations for a stable channel width and depth are :

$$W_b = \left[\frac{f_b}{f_s} \right]^{1/2} * Q^{1/2} \tag{2.25}$$

$$d_b = (f_s/f_b^2)^{1/3} * Q^{1/3} \tag{2.26}$$

Where, W_b = Blench regime width of channel at half depth

f_b = Blench bed factor = fbo (1+0.12c)

f_s = Blench side factor

fbo = $1.9\sqrt{D_{50}}$

Q = discharge

c = bed load charge, measured as dry weight per second of bed load divided by weight of water flow per second and deduced to parts per hundred thousand

d_b = Mean depth, equal to the cross sectional area divided by W_b

Table-2.5 : Values of f_s suggested by Blench

Degree of cohesiveness	f_s
Slight	0.1
Moderate	0.2
High	0.3

Where the river width is imposed by an artificial constriction, the depth equation becomes :

$$d_b = \frac{q^{2/3}}{fb^{1/3}} \tag{2.27}$$

Where, q = intensity of discharge.

Table- 2.6 : Multiplying factors to be applied to the equation proposed by Blench

Condition	Factor
Maximum depth in a straight reach	1.5
Maximum depth in a bend of normal meander, curvature	1.7

2.7.3 Breuser's Empirical Relation

An empirical relation for scour depth around bridge piers in sand beds below general bed level was given by **Breusers** et al (1977)as follows :

$$\frac{d_s}{b} = \left[2\tanh\left\langle\frac{y}{b}\right\rangle\right]\left[f_1\left\langle\frac{\overline{U}}{U_c}\right\rangle\right][f_2 f_3] \qquad (2.28)$$

Where

$$f_1 = 0 \; for \; 0.0 < \left\langle\frac{\overline{U}}{U_c}\right\rangle < 0.5$$

$$f_1 = 2\left\langle\frac{\overline{U}}{U_c}\right\rangle - 1 \; for \; 0.5 < \left\langle\frac{\overline{U}}{U_c}\right\rangle < 1.0 \text{ (clear water scour)}$$

$$f_1 = 1.0 \; for \; 1.0 < \left\langle\frac{\overline{U}}{U_c}\right\rangle \text{ (scour with continuous sediment motion)}$$

$\overline{U}$ is the average velocity and U_c is the critical velocity of flow

2.8 Recent Studies in Bangladesh

A number of morphological studies were done in Jamuna, Ganges and Meghna River in recent time under Flood Action Plan (FAP). In Jamuna river Jamuna Multipurpose Bridge Project (1989) conducted morphological studies and scale model study for the protection of above bridge. Jamuna Multipurpose Bridge Project (JMBP) analyzed BWDB cross section to derive "regime equations", and under took special hydrographic survey aiming at improved understanding of :

- scour in channel bends
- confluence scour
- bed forms
- local scour

The river Jamuna is a braided river and as such characteristics of individual channels of the braided Jamuna river was compiled by JMBP with the regime equation. This was done with dominant discharge of 44000 m^3/s and individual channels were plotted against discharge. The plotted data was described with following equation in S1 unit :

$$h = 0.23\ Q^{0.32} \tag{2.29}$$

$$B = 16.1\ Q^{0.53} \tag{2.30}$$

Where 'h' is the average depth and 'B' is the bankfull width.

JMBP has studied the bend, confluence and local scour. They concluded confluence scour dominant over the bend scour. This study only deals with general scour however it is necessary to discuss on other from of scour particularly bend and confluence scour. The study observed that local scour depth is more for single groyne and that is less for a well designed system of groynes.

Ashmore & Parker (1983) analyzed a large number of field data and proposed the following relation for confluence scour :

$$h_s/h = f\,(F_o, i_w, \theta, E) \tag{2.31}$$

Where, h_s = maximum (confluence) scour depth (m)

h = average depth of flow of the anabranches

F_o = densimetric Froude number, defined via

$$F_o = u_o / (A_g D_g)^{0.5}$$

u_o = average anabranches velocity (m/s)

D_g = average mean grain size of the u/s anabranch

Δ = relative density, defined via

$\Delta = (\rho_s - \rho) / \rho$

ρ_s = density of sediment (kg/m^3)

ρ = density of water (kg/m^3)

i_w = average anabranches d/s water surface slope

θ = Angle of incidence of anabranches of confluence

$E = (Q_L - Q_R) / 0.5Q_T$

JMBP study found the following relationship between h_s/h and θ as follows :

$$h_s/h = 2.235 + 0.0308\,\theta \qquad (2.32)$$

Where, $20^0<\theta<30^0 = 0.17$

$30^0<\theta<40^0 = 0.17$

$40^0<\theta<50^0 = 0.22$

$50^0<\theta<60^0 = 0.11$

$60^0<\theta<70^0 = 0.28$

$70^0<\theta<80^0 = 0.06$

Bhuiyan (1991) conducted "an experimental study on confluence scour" found that relative scour h_s / h (h = confluence scour, and h_s is the average depth at u/s of tributaries) depends on the confluence angle. Its observed that if confluence angle is less than 30^0 there is less turbulence and scour is also less. Scour depth was found to be increase rapidly for angle of incidence 30^0 to 75^0.

Kabir (1984) conducted "an investigation of local scour around bridge piers" and found that relative scour depth , d_t / b increases with the increase of Froude number, F. It is observed that the relative scour depth increases with the increasing relative approach depth. According to the study by Kabir (1984) it was concluded that the scour formula by Blench (1962) was the best predictor among Inglis, blench and Shen et. al for round-nose piers.

Alam (1996) conducted "a study of general scour of alluvial rivers" and found that the predicted depth of Surma river is 9.18 m, Upper Meghna river is 12.35 m and Brahmaputra river is 17.32 m, by using Lacey's equation which was found to be close to observed average depth. But in case of Teesta river, predicted depth by using Blench equation is 4.15 m, which is close to, observed average depth. It appears that while one regime equation is suitable for one river it may not be suitable for another river.

CHAPTER THREE

DATA AND METHODOLOGY

3.1 Introduction

Phenomena involving scour around bridge piers have been studied in the Hydraulic Research laboratory of RRI and data collected from experiments have been plotted and analyzed on the basis of the theoretical approach.

Field data of the East-West Interconnector and Hardinge Bridge and some groynes have also been collected and plotted on the basis of empirical equations forwarded by Inglis(1949), Blench(1962), Ahmed(1962), Shen et. al(1966).

Local scour at bridge piers or near the embankment is the result of vortex systems developed at the piers or large wake vortices (or eddies) set up downstream of embankments, the river bank and the stream bed. The scouring is achieved due to the resulting development of high velocity because of partial blockage of the flow by the pier as seen commonly as eddies. The shape and size of the pier also significantly affects scour depth because it reflects a strength of the horseshoe vortex at the base of the pier as observed by Shen, Karaki et. al (1969). A blunt-nose pier will cause the greatest scour depth while stream lining the front end of the pier will reduce the scour. Moreover, streamlining the downstream end of the piers will reduce the strength of the wake vortices.

Mathematical theory and experimental data have developed practical solutions to many hydraulic problems. Important hydraulic structures are now designed and built only after extensive model studies have been made. Application of dimensional analysis and hydraulic similitude enable the engineer to organize and the simplify the experiments and to analyze the results therefrom.

3.2 Field Data Collection

Data sheet of the year of 1992 to 1997 has been collected. In the data sheet scouring has been measured from the top of the caissons. Also the water level inside and outside of the caissons have been measured from the top of the caissons. The diameter of the caissons is of 35 feet. Average of the water level inside and outside of the caissons has been taken. The scour depth from water level has been found by deducting average water level from the scouring level. Discharge data have been collected from BWDB for the discharge station Bahadurabad. The discharge data were than similitude for Aricha –Nagarbari station. Scour and related data for E-W Interconnector are shown in Table - 1.1 through 1.9 for caisson no. 2 through 10 respectively.

The Hardinge Bridge crosses the river Ganges approximately in between Paksey and Bheramara. It consists of sixteen composite piers. Scour and related data of this bridge were collected from Kabir (1984) and are shown in Table-2.

3.2.1 Hydraulic Characteristics of Jamuna River

Jamuna/Brahmaputra River originates in Tibet on the north slope of the Himalayas and drains an area of about 550,000 km^2, extending over China, Bhutan, India, and Bangladesh. Its traverse path is about 2,740 km before meeting with the Ganges River at Aricha. The mean annual discharge is 20,000 m^3/s; the maximum discharge recorded till now is 100,000 m^3/s (in 1988), and the bankfull discharge is about is 48,000 m^3/s. The slope of the river within Bangladesh decreases in the downstream direction and is $8.5*10^{-5}$ at the upstream end, and $6.5*10^{-5}$ near the confluence with the Ganges. The bed material sizes also decreases from the upstream towards downstream part and ranges from 0.22 mm to 0.16 mm.

The Chezy roughness values of the river Jamuna at Sirajgonj vary between 40 $m^{1/2}/s$ for low flow and 100 $m^{1/2}/s$ for flood condition (Klaassen et al, 1988). In the Jamuna Bridge Project study, it was found that the Chezy number also varies between 40 $m^{1/2}/s$ for low flow and 100 $m^{1/2}/s$ for flood conditions based on BWDB discharge measurements

(RPT/NEDECO/BCL, August 1989). In that study it was seen that the Chezy's roughness value varies with water depth. On the basis of these studies, the Chezy roughness value of 70 to 74 $m^{1/2}/s$ was used for discharge ranging between 65,000 to 75,000 m^3/s for main channel and 64 $m^{1/2}/s$ was used for minor channels. In the present study the same value is used for calculating velocity and other required parameters.

3.3 Laboratory Experiment and Model Test Data

Some experimental data of local scour for circular and round-nose pier were collected from Kabir (1984) and are shown in Table-3.1 & 3.2.

Data observed from the seven laboratory test runs by placing three different sizes of piers on same bed material and varying discharges were collected. The mean diameter (i.e. D_{50}) of the bed material chosen for the study were 0.23 mm. Different discharges for which test runs were conducted ranged between 35 lps and 141 lps. The flow conditions for the various runs and the results obtained in these tests are summarized in Table –4.

RRI conducted physical model test for finalizing the proposed structures to be constructed within the river for the protection of vulnerable area. Specially local scour is observed in these model tests. But all of these model studies are not with groynes. Some scour data for Ganges and Jamuna River are collected from different physical model test report of which are tested with groynes. Scour and related data for groynes are shown in Table - 5.

Data were observed and collected from five laboratory test runs of Paksey Bridge Pile Configuration Flume study by placing two different size of pier (with pile) group on same bed material . The mean diameter (i.e. D_{50}) of the bed material chosen for the study were 0.08 mm. The discharge, velocity and flow depth in the model of this study was 384 lps, 0.53 m/s and 34 cm respectively. The scale of the model was 1:90. The flow conditions for the various runs and the results obtained in these tests are summarized in Table –6.1 through 6.4.

3.4 Design and Construction of Experimental Channel

The experimental channel was designed for pre-selected discharges and water depths based on critical velocity criteria. The critical velocity is defined as the maximum mean velocity of a channel that will not cause erosion of the channel boundary (banks and bed). Several formulae for critical velocity of noncohesive bed material have been proposed. For example, Mavis and laushey (1949), Carstens (1966) and Neill (1967).

In this experiment the critical velocity in m/s was calculated using the following equation presented by Van rijn (1985)

$$v_{cr} = 0.19(D_{50})^{0.1} \log\frac{12D}{3D_{90}}, \text{ for } 100 \le D_{50} \le 500\ \mu m \tag{3.1}$$

$$V_{cr} = 8.5(D_{50})^{0.6} \log\frac{12D}{3D_{90}}, \text{ for } 500 \le D_{50} \le 2000\ \mu m \tag{3.2}$$

For satisfying the research objective and also for designing rigid boundary rectangular channel whose bed are subject to scour, the average velocities was taken as 1.25 times the critical velocity.

Finally, the dimensions of the channel was taken as follows:

Width of the channel = 3.52 m

Depth of the channel = 0.25 m

Length of the channel (movable bed) = 19.15 m (including 1 m fixed in the up stream)

Length of u/s fixed bed = 9.95 m

Length of d/s fixed bed = 4.1 m

Bed slope = 28 cm per km

The present study program was carried out in the laboratory of Hydraulic Research Directorate. (HRD) , River Research Institute (RRI), Faridpur. About three feet deep sand layer over a concealed foundation is suitable for constructing a movable bed channel. The laboratory is well facilitated for supplying measuring water. This study was performed from August, 1996 to March, 1997 with continuous observation, test, data analysis and evaluation. The daily average temperature ranges from 12 to 32°c and humidity ranges from 60 to 95 percent. Plan of the experimental channel is shown in Fig. 3.1.

3.5 Methodology

Scour and related data for the period of 1984 to 1997 have been collected from Bangladesh Power Development Board (BPDB). Necessary additional data on scour around piers and groynes were collected from laboratory experiments and model studies in RRI. These were then be processed and compiled to usable form. Later the data was utilized to estimate the scour depths using some of the well-known scour formulae. The presently selected formulae are those of Inglis (1949), Ahmed (1962), Blench (1962), Larra (1963), Shen et al. (1969) and Lacey (1970) for piers and Ahmed (1953), Liu et al (1961) and Breuser (1991) for groynes. The basis of selection of these formulae are due to their regional validity. Few formulae were selected on the basis of recommendation from some previous evaluations (Kabir, 1984; Hossain et al, 1989). The predicted scour depths were then compared with the observed scour value interms of discrepancy ratios. Statistical analysis have been conducted to finalize the order of suitability of these equations against the observed data. Some merit table have also been prepared based on the data coverage between two selected discrepancy bands and on the basis of statistical tests. Some correlations have been developed for scour depth against discharge and pier size.

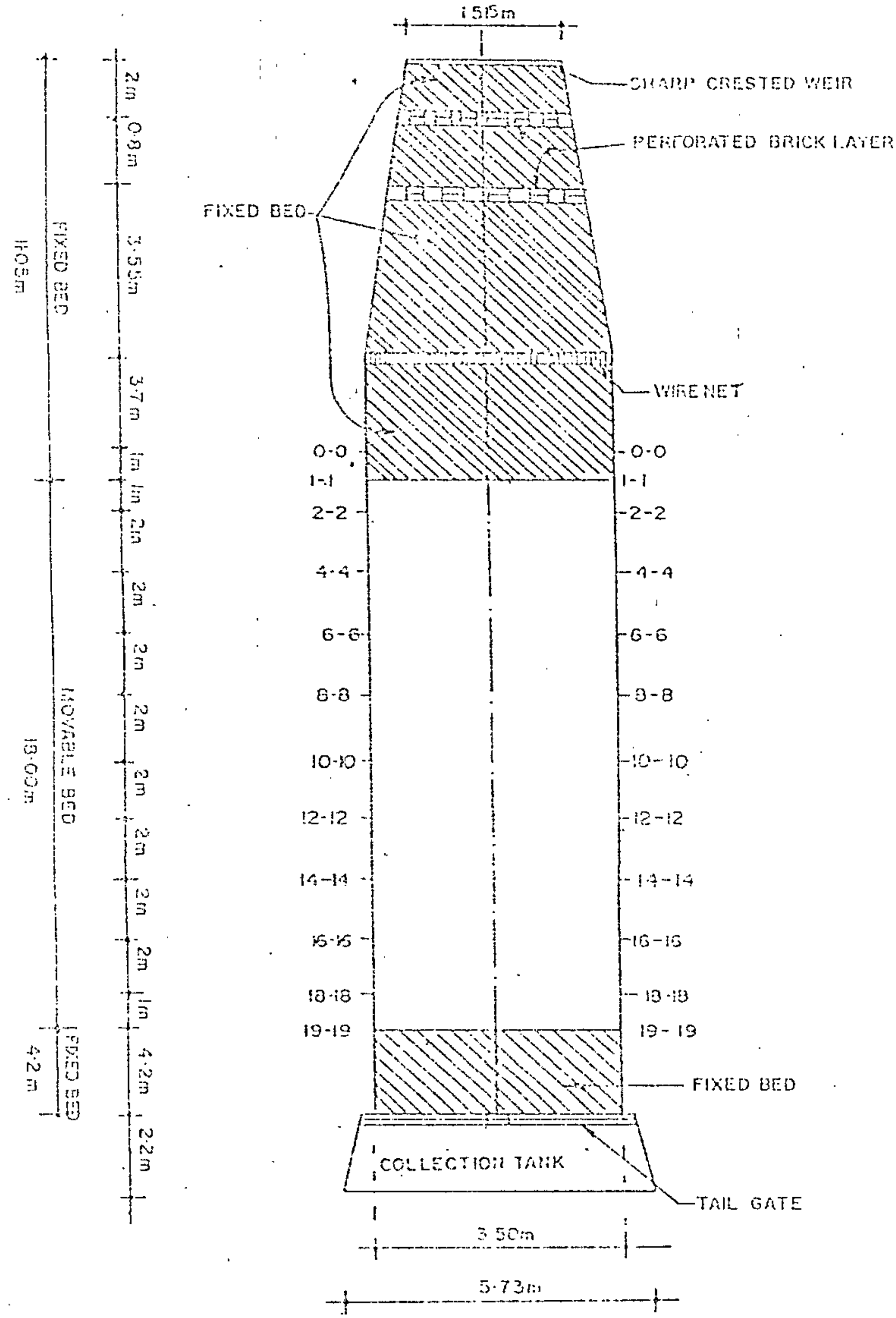

FIG. 3·1 : PLAN OF EXPERIMENTAL CHANNEL

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Comparison of Field Data

East-West Interconnector

The hydraulic data relevant for East-West Interconnector were utilized to calculate the scour depth around caisson no. 2 to 10 using equations derived by Inglis (1949), Ahmed (1962), Blench (1962) and Shen et. al (1969). The computed values of scour depth were compared with the measured value. The scatter diagrams were plotted with observed data against predicted data to observe the deviation from the line of perfect agreement (see fig. 4.1 to 4.9). It can be seen that Ahmed's formula highly over predicted for caisson no. 2 & 3 (fig. 4.1 & 4.2) of the above mentioned caissons and under predicted for the caisson no. 6 to 10 (fig. 4.5 to 4.9). On the other hand some data were over predicted and under predicted for the caisson no. 4 & 5 (fig. 4.3 & 4.4). So, it appears that Ahmed's formula is not very suitable for the prediction of scour for East-West Interconnector.

It may be observed that Inglis formula over predicted for all caissons except caissons no. 4 & 5 (fig. 4.1, 4.2 and 4.5 to 4.9) and under predicted for the caissons no. 4 & 5 (fig. 4.3 & 4.4).

It can be observed that Blench and Shen et. al formula over predicted and under predicted for all caissons (see fig. 4.1 to 4.9) respectively.

Correlations were developed between D_t/b and $q^{2/3}/b$; D_t/y and b/y and d_s/y and $F^2(b/y)^3$ for data of East-West Interconnector. Regression equations were developed for each pair of parameters that are shown in fig. 4.10 to 4.12. Correlation is shown in Table- 4.1.

Table- 4.1 : . Correlation between various parameters for East-West Interconnector

Name of the Investigator	Equations (FPS system)	Correlation
Inglis (1949)	$D_t/b = 1.7\ (q^{2/3}/b)^{0.78}$	$D_t/b = 0.8807\ (q^{2/3}/b)^{0.2818}$ (4.1) $R^2 = 0.2873$
Blench (1962)	$D_t/y = 1.8\ (b/y)^{0.25}$	$D_t/y = 1.4596\ (b/y)^{0.7168}$ (4.2) $R^2 = 0.7219$
Shen et. al (1969)	$d_s/y = 2\ [F^2(b/y)^3]^{0.215}$	$d_s/y = 3.8754\ [F^2(b/y)^3]^{0.3387}$ (4.3) $R^2 = 0.7168$

Equations 4.1 to 4.3 are for East-West Interconnector and shown similarity to the respective equations stated in chapter two. Only differences are in constants and in the value of the index power. Comparing equation 4.3 to Shen et. al formula, it is found that the variations are 93% and 57% in the values of the coefficient and index power respectively.

The degree of compliance of the scour formulae representing the interrelationship between the observed and predicted scour depths was determined by calculating a term known as discrepancy ratio. The discrepancy ratio (D.R.) is defined as the ratio of scour depth calculated by using any of the selected equation to the measured scour depth. These ratios for each individual data using all the selected formulae were computed. These discrepancy ratio were then analyzed to evaluate the degree of performance of various equations in predicting the scour depth. This is actually done by determining the percentage of data coverage between a selected band of discrepancy ratio. A discrepancy ratio of unity means a perfect agreement of the predicted value with the measured value. The selected band of discrepancy ratio is one-half to two. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in the tabular form.. The equations selected were arranged according to their performance based on data coverage. However, it may be mentioned that a different order of prediction could perhaps be obtained if adjustment were made to the mean discrepancy ratio. However, no such attempts were made here.

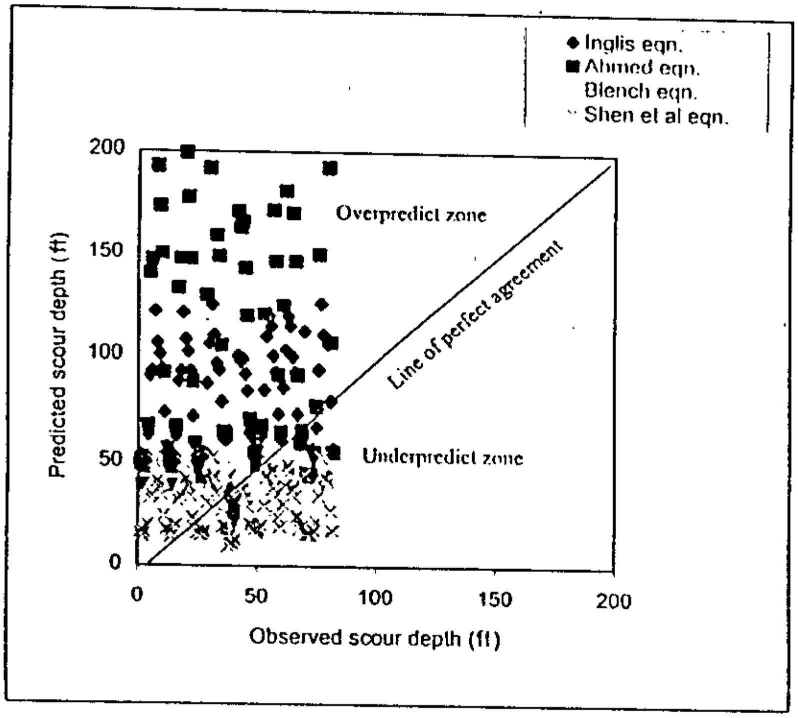

Fig. 4·1 : Comparison of predicted and observed value of scour depth for caisson no. -2 of East-West Interconnector

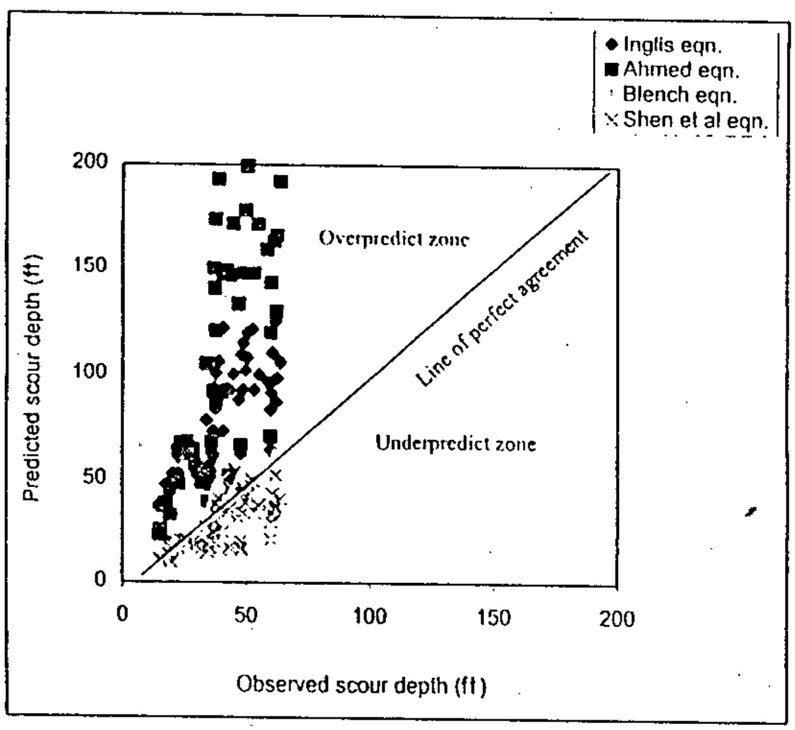

Fig. 4·2 Comparison of predicted and observed value of scour depth for caisson no. -3 of East-West Interconnector

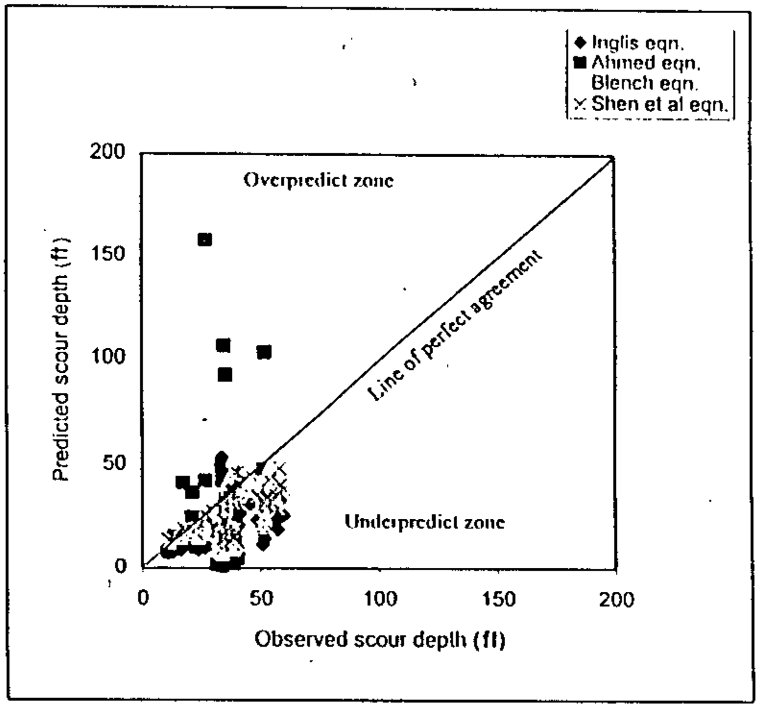

Fig. 4.3: Comparison of predicted and observed value of scour depth for caisson no.-4 of East-West Interconnector

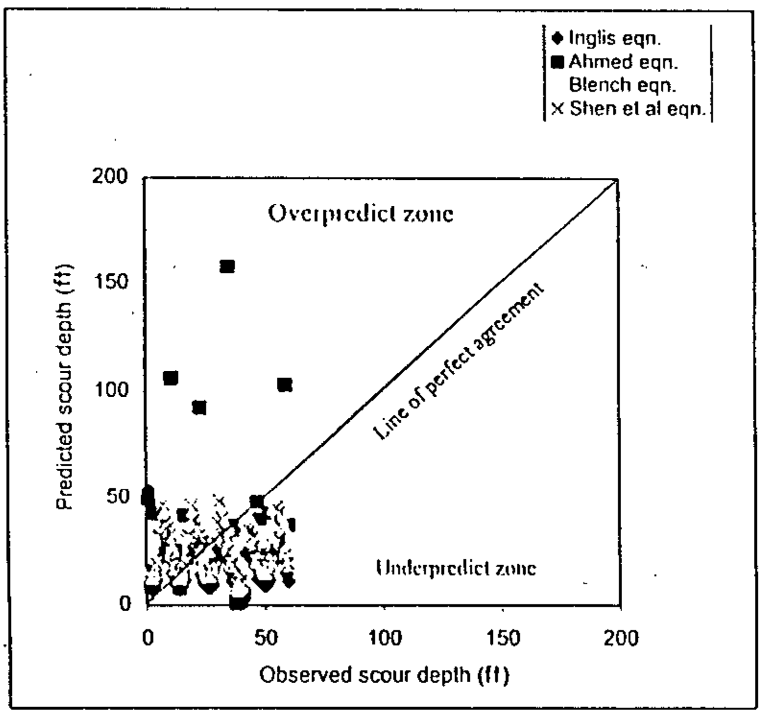

Fig. 4.4: Comparison of predicted and observed value of scour depth for caisson no. -5 of East-West Interconnector

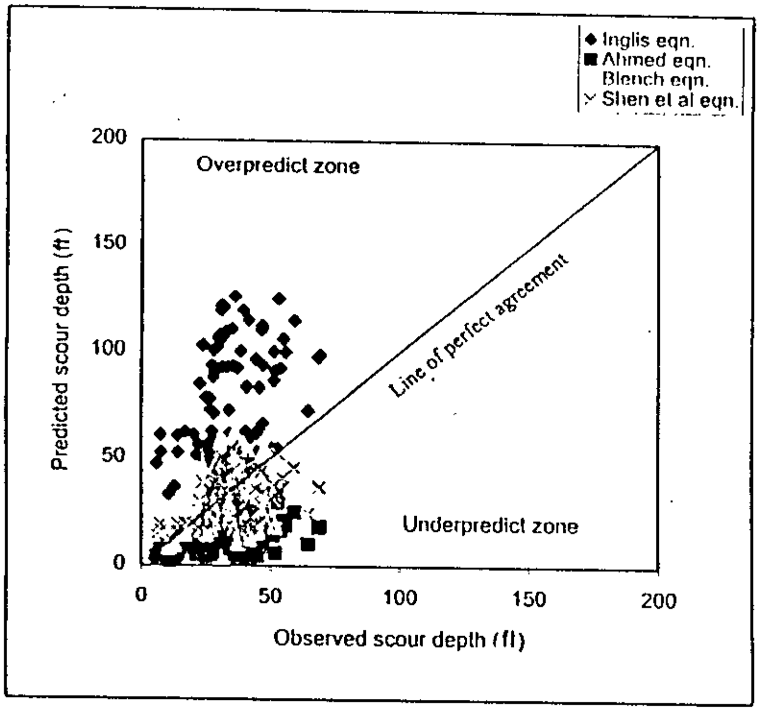

Fig. 4·5: Comparison of predicted and observed value of scour depth for caisson no.-6 of East-West Interconnector

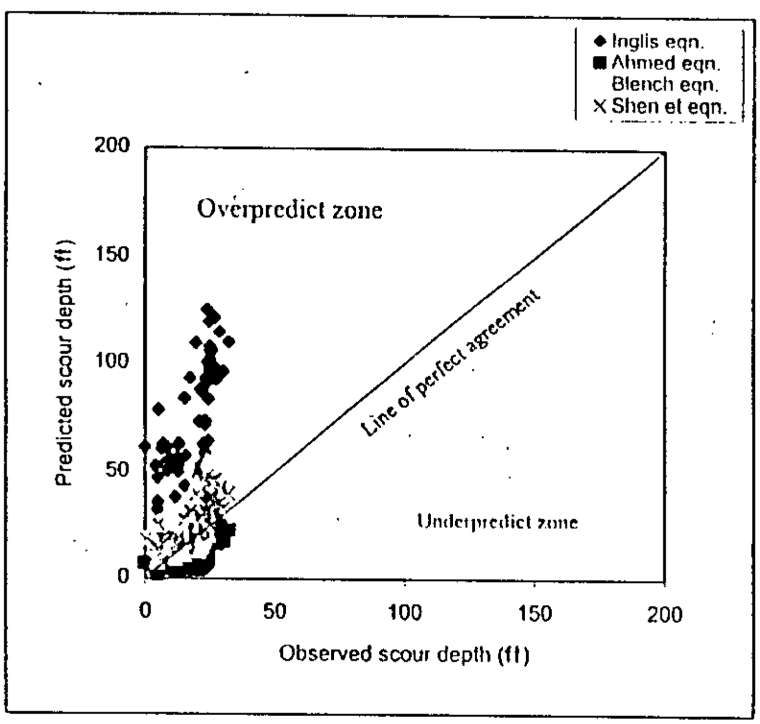

Fig. 4·6: Comparison of predicted and observed value of scour depth for caisson no.-7 of East-West Interconnector

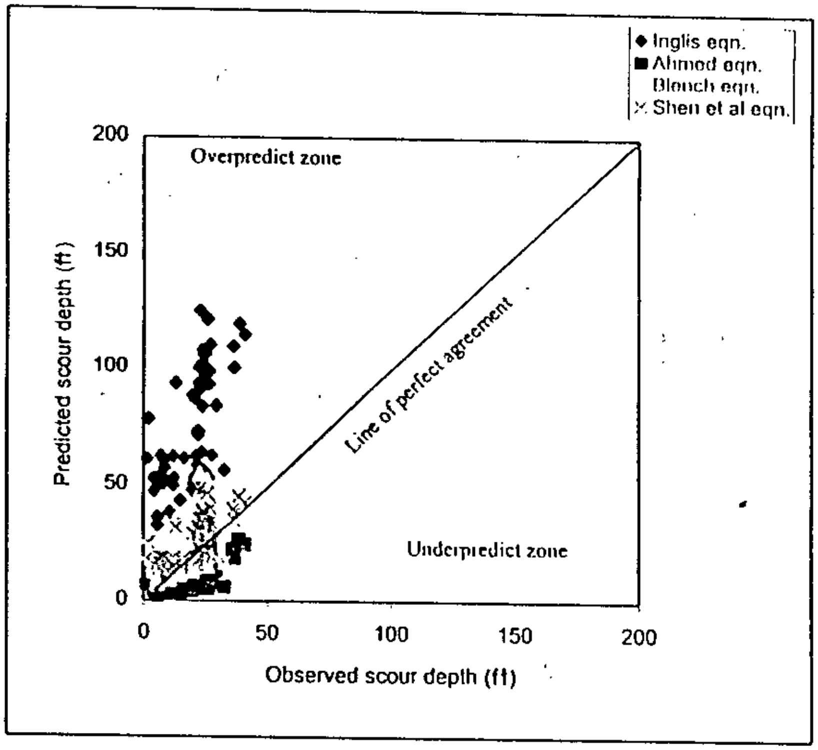

Fig. 4·7: Comparison of predicted and observed value of scour depth for caisson no.-8 of East-West Interconnector

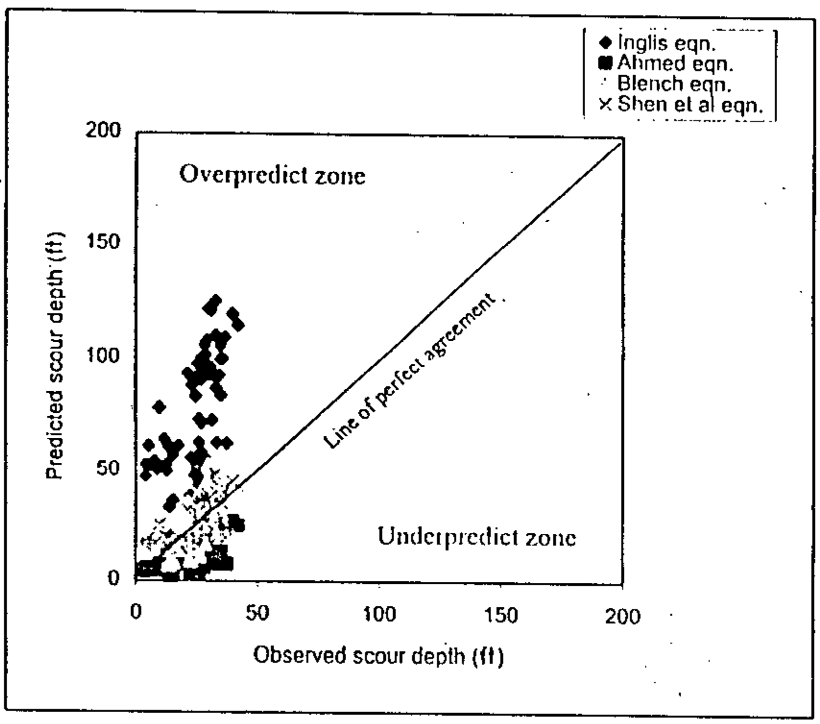

Fig. 4·8: Comparison of predicted and observed value of scour depth for caisson no.-9 of East-West Interconnector

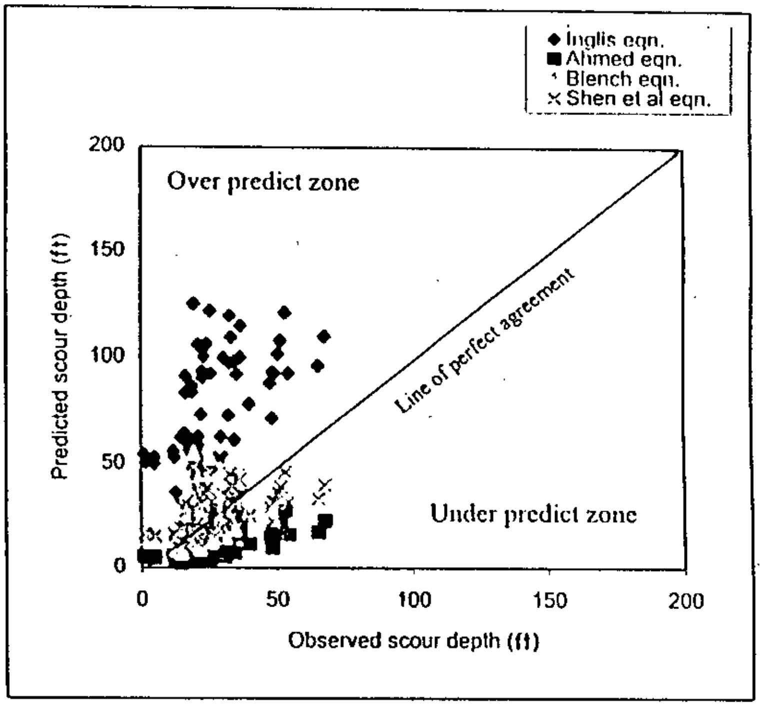

Fig. 4·9: Comparison of predicted and observed value of scour depth for caisson no.-10 of East-West Interconnector

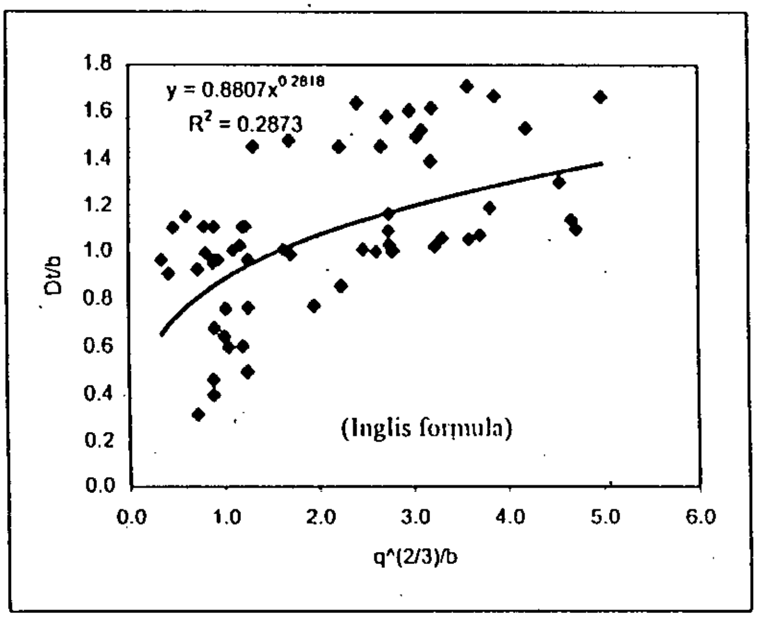

Fig. 4·10: Relation between Dt/b and q^(2/3)/b for East-West Interconnector

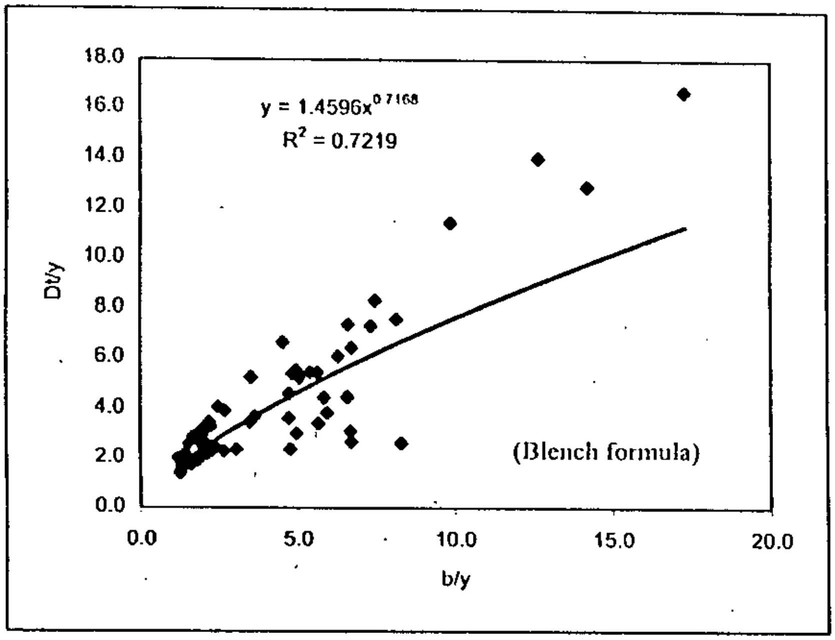

Fig. 4·11 : Relation between Dt/y and b/y for East-West Interconnector

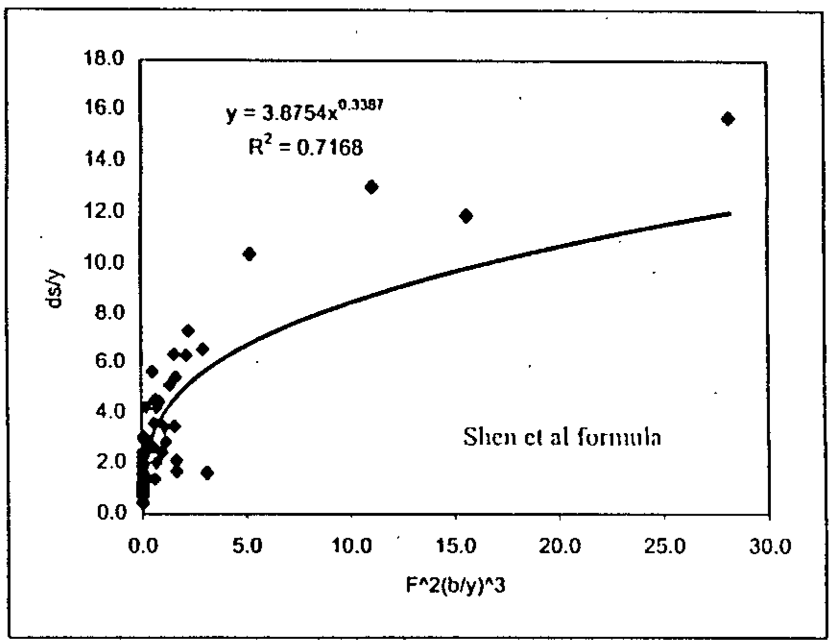

Fig. 4·12: Relation between ds/y and F^2(b/y)^3 for East-West Interconnector

Table- 4.2 : Comparison of Data coverage using East-West Interconnector data

Name of Equations	% of data coverage for a range of discrepancy ratio			
	0.5-2.0	0.25-4.0	0.125-8.0	rest
Blench equation	81	99	100	-
Shen et al equation	77	98	100	-
Inglis equation	40	89	99	1
Ahmed equation	32	56	73	27

From the criteria of data coverage between the discrepancy ratio of one-half to two as shown in the above table, Blench formula is found to the top of the ranking. This is followed by Shen et al equation, Inglis equation and Ahmed equation.

Hardinge Bridge (Paksey Rail-way Bridge)

The data for Hardinge Bridge were considered for testing the efficacy of the Inglis (1949), Ahmed (1962), Blench (1962) and Shen et. al (1969) formulas. The predicted values of scour depths by these formulas were compared with the observed values. Figure-4.13 shows the plotting of predicted values by these equations against the observed scour depths. It may be observed from this figure that Ahmed's formula predicts in a very inconsistent form. However, it should be noted that range of observed scour value is small. It appears that the prediction performance of Ahmed's formula is not suitable against the data of Hardinge Bridge.

Critical examination of the figure reveals that Inglis, Blench and Shen et. al formula were over predicted and under predicted and is very close to the line of perfect agreement compared to Ahmed formula. It is also observe that some values lies on the line of perfect agreement for these equations. To select the best predictor for assessing the scour depth at Hardinge Bridge, another analysis was done and is discuss below.

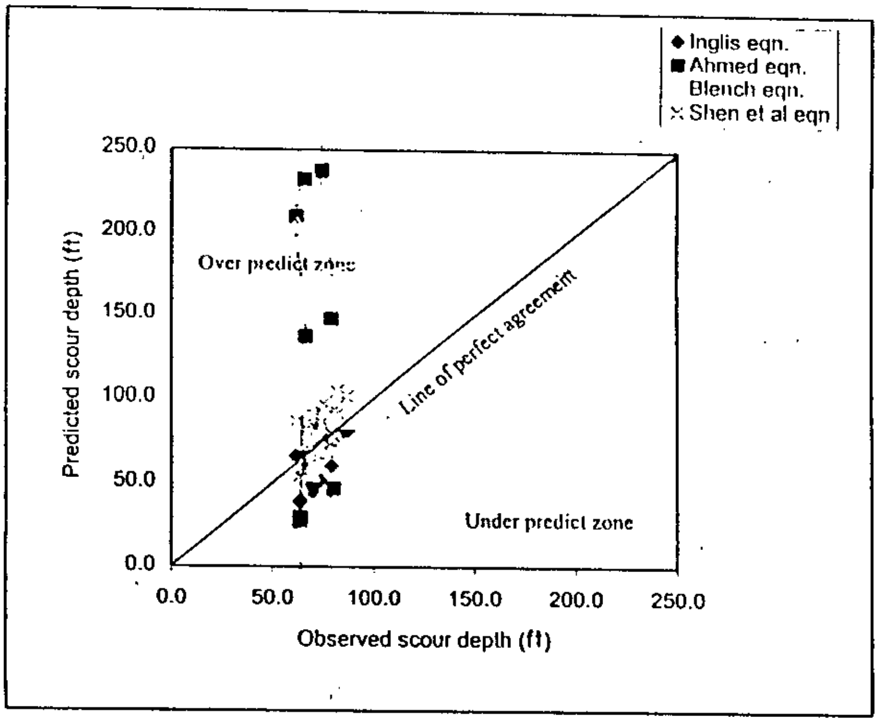

Fig. 4.13 : Comparison of predicted and observed value of scour depth for Hardinge Bridge

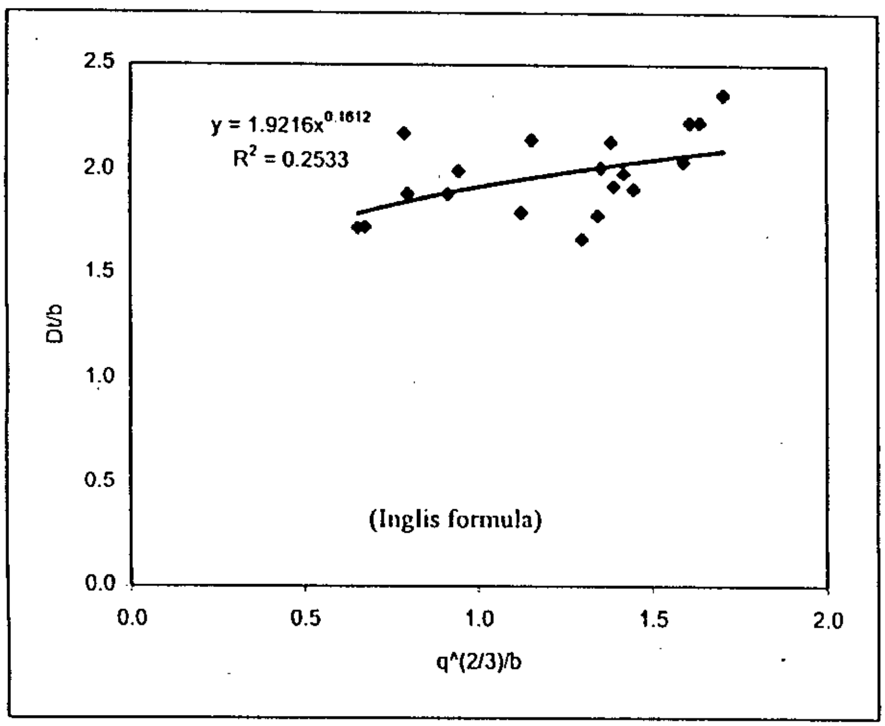

Fig. 4.14: Relation between Dt/b and q^(2/3)/b for Hardinge Bridge

An attempt was made to develop regression equation of power type. Figs. 4.14 to 4.16 were plotted for this purpose and the statistical equations obtained are given below in the tabular form (Table – 4.3)

Table- 4.3 : Correlation between various parameters for Hardinge Bridge

Name of the Investigator	Equations (FPS system)	Correlation
Inglis (1949)	$D_t/b = 1.7\ (q^{2/3}/b)^{0.78}$	$D_t/b = 1.9216\ (q^{2/3}/b)^{0.1612}$ (4.4) $R^2 = 0.2533$
Blench (1962)	$D_t/y = 1.8\ (b/y)^{0.25}$	$D_t/y = 1.8288\ (b/y)^{0.7576}$ (4.5) $R^2 = 0.8144$
Shen et. al (1969)	$d_s/y = 2\ [F^2(b/y)^3]^{0.215}$	$d_s/y = 8.7846\ [F^2(b/y)^3]^{0.6422}$ (4.6) $R^2 = 0.4546$

Equations 4.4 to 4.6 are for Hardinge Bridge and shown similarity to the respective equations stated in chapter two. Only differences are in constants and in the value of the index power. Comparing equation 4.2 to Blench formula, it is found that the variations are 2% and 200% in the values of the coefficient and index power respectively. For other formulas the variations of the coefficient and index power are very high.

Percentage data coverage between the specified discrepancy ratio bands was also examined for the scour data of Hardinge Bridge. In this case, the selected band of discrepancy ratio is 0.8-1.24. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in the tabular form (Table-4.4). The equations selected were arranged according to their performance based on data coverage.

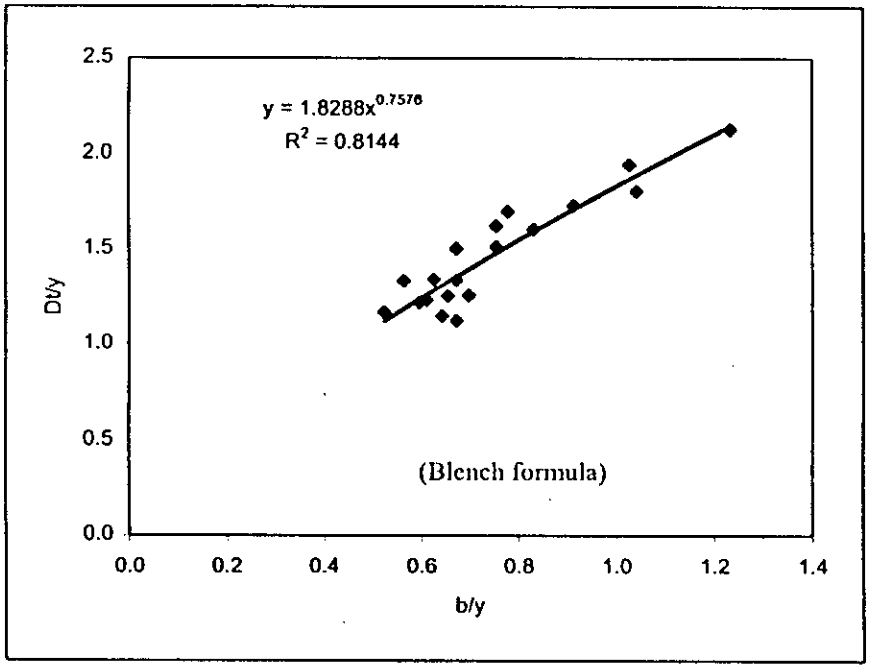

Fig. 4·15: Relation between Dt/y and b/y for Hardinge Bridge

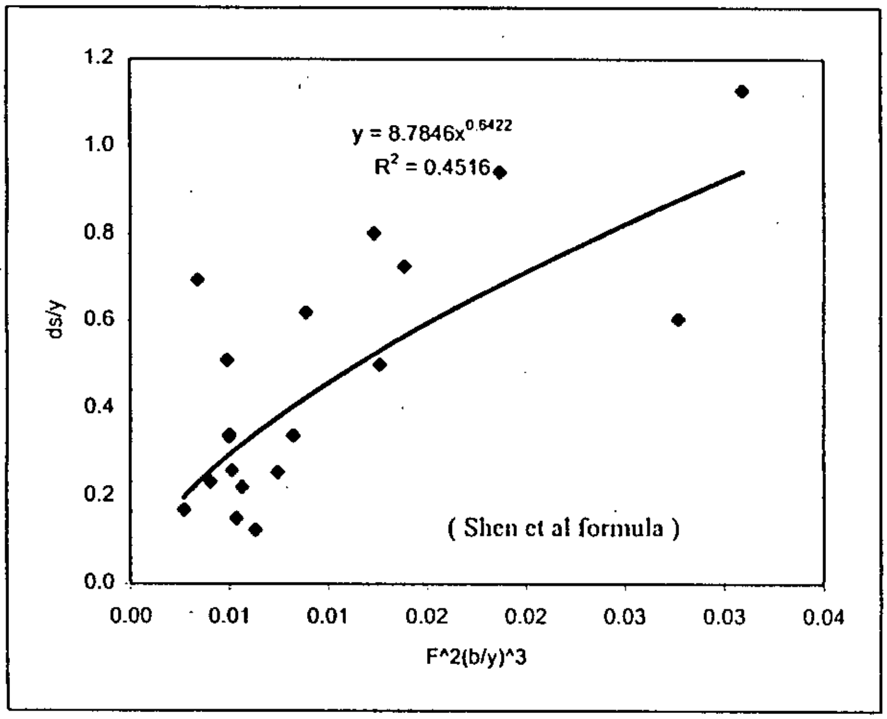

Fig. 4·16: Relation between ds/y and F^2*(b/y)^3 for Hardinge Bridge

Table- 4.4 : Comparison of Data coverage using Hardinge Bridge data

Name of Equations	% of data coverage for a range of discrepancy ratio				
	0.8-1.25	0.5-2.0	0.25-4.0	0.125-8.0	Rest
Blench equation	90	100	-	-	-
Shen et al equation	74	100	-	-	-
Inglis equation	63	100	-	-	-
Ahmed equation	11	26	74	100	-

From the criteria of data coverage between the discrepancy ratio 0.8-1.25 as shown in the above table, Blench formula is found to the top of the ranking. This is followed by Shen et al equation, Inglis equation and Ahmed equation in the order.

Scour Data of Piers and Groynes (RRI)

Some scour data for circular pier and model test data for groyne have also been collected from RRI and compiled to usable form. The scour data of pier were analyzed on the basis of the equations derived by Inglis (1949), Ahmed (1962), Blench (1962) and Shen et. al (1969). The analysis were done with laboratory test data and compared with the predicted data by the aforesaid equations. The scatter diagrams were plotted with observed data against predicted data to observe the deviation from the line of perfect agreement (see fig. 4.17). It can be observed that Inglis formula highly over predicted. So, it can conclude that Inglis formula is not suitable for the prediction of scour for this case.

It can be observed that Blench formula were over predicted for some case but in most cases it gives accurate prediction. Ahmed and Shen et. al formula gives accurate prediction and is very close to the line of perfect agreement compared to Inglis and Blench formula. It is also observe that the predicted values lies on the line of perfect agreement for these equations (fig. 4.17). To select the best predictor for this case, another analysis was done and is discuss below.

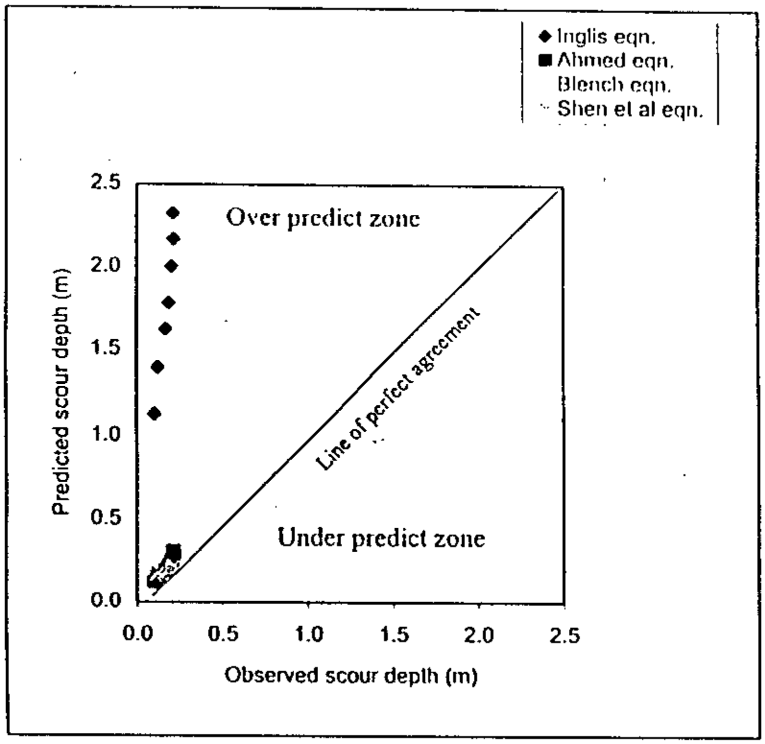

Fig. 4·17 : Comparison of predicted and observed value of scour depth for Laboratory test data (RRI)

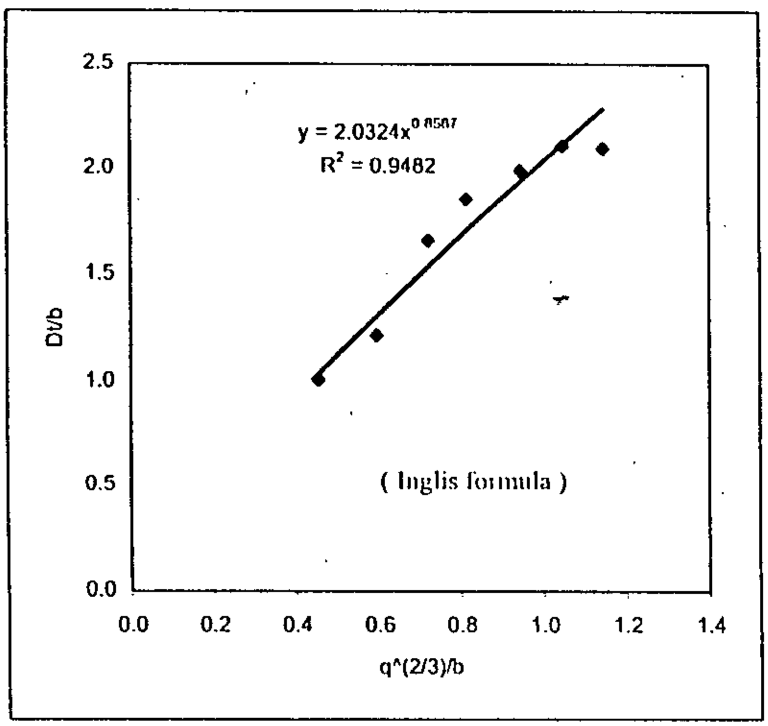

Fig. 4·18 : Relation between Dt/b and q^(2/3)/b for laboratory test data (RRI)

An attempt was made to compare the laboratory test data with its respective similar equations. But no correlation was found for Blench formula (fig. 4.19). Figs. 4.18 to 4.20 were plotted for this purpose and the statistical equations obtained are given below in the tabular form (Table- 4.5).

Table- 4.5 : . Correlation between various parameters for scour data of Piers (RRI)

Name of the Investigator	Equations (SI unit)	Correlation
Inglis (1949)	$D_t/b = 2.32\ (q^{2/3}/b)^{0.78}$	$D_t/b = 2.0324\ (q^{2/3}/b)^{0.8587}$ (4.7) $R^2 = 0.9482$
Shen et. al (1969)	$d_s/y = 2\ [F^2(b/y)^3]^{0.215}$	$d_s/y = 18.072\ [F^2(b/y)^3]^{1.2526}$ (4.8) $R^2 = 0.8439$

Equations 4.7 & 4.8 show similarity to the respective equations stated in chapter two. Only differences are in constants and in the value of the index power. Comparing equation 4.7 to Inglis formula, it is found that the variations are 12% and 10% in the values of the coefficient and index power respectively. This means that Inglis equation is very much suitable for this case.

Similar analysis was done for laboratory test data of circular pier (RRI) as in the case of East-West Interconnector and Hardinge Bridge. In this case, the selected band of discrepancy ratio is 0.8-1.24. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in the tabular form (Table- 4.6). The equations selected were arranged according to their performance based on data coverage.

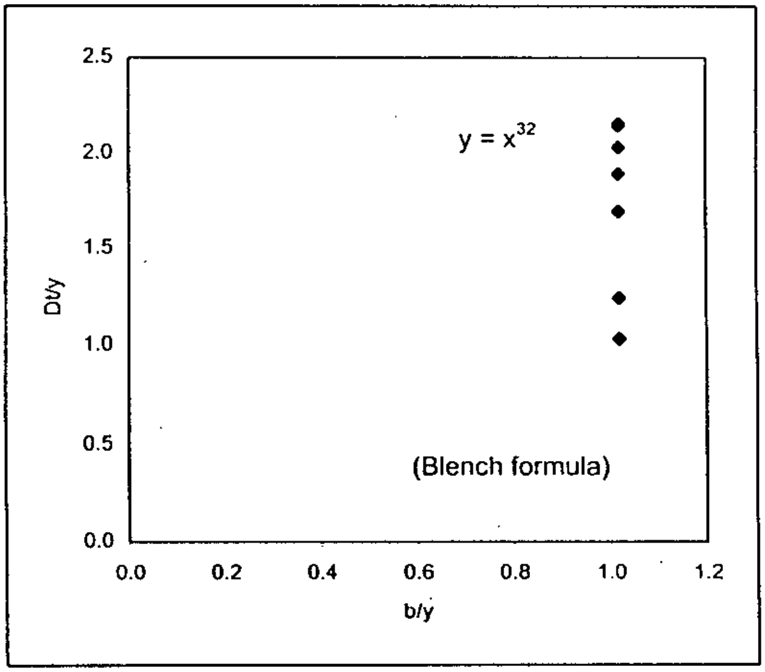

Fig. 4·19 : Relation between Dt/y and b/y for laboratory test data (RRI)

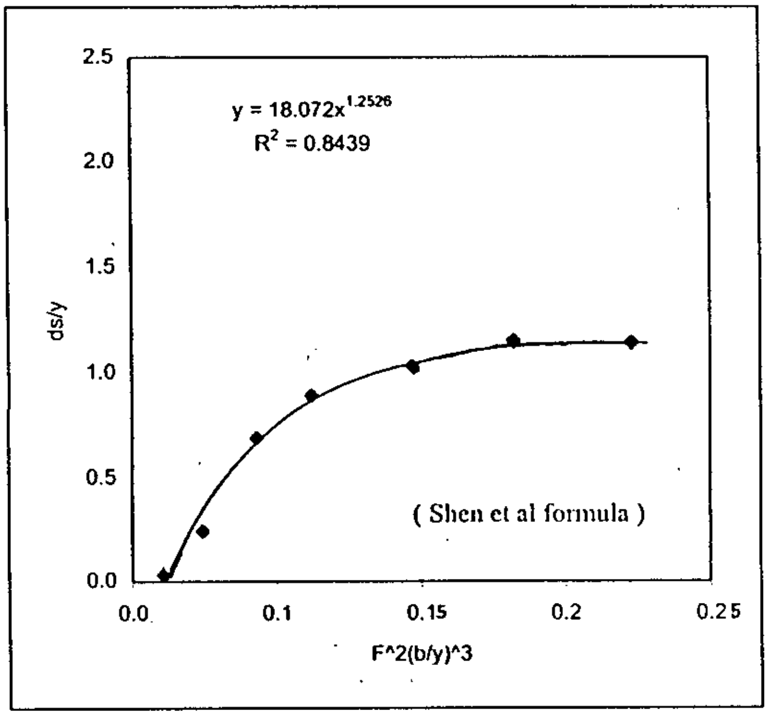

Fig. 4·20: Relation between ds/y and F^2(b/y)^3 for laboratory test data (RRI)

Table- 4.6 : Comparison of Data coverage using RRI data of Circular Pier

Name of Equations	% of data coverage for a range of discrepancy ratio				
	0.8-1.25	0.5-2.0	0.25-4.0	0.125-8.0	Rest
Shen et. al equation	71	100	-	-	-
Ahmed equation	43	100	-	-	-
Blench equation	-	-	20	70	30
Inglis equation	-	-	-	40	60

From the criteria of data coverage between the discrepancy ratio 0.8-1.25 as shown in the above table, Shen et. al equation is found to the top of the ranking. This is followed by Ahmed, Blench and Inglis equation.

The scour data of groyne were analyzed on the basis of the equations derived by Breuser (1991), Liu et. al (1961) and Ahmed (1953). The analysis were done with laboratory test data and compared with the predicted data by the aforesaid equations. The scatter diagram was also plotted for groyne with observed data against predicted data to observe the deviation from the line of perfect agreement (see fig. 4.21). It can be observed that all equations give under prediction but is close to the line of perfect agreement To select the best predictor for this case, another analysis were done and is discuss below.

An attempt was made to compare the laboratory test data for groyne with its respective similar equations. Figs. 4.22 to 4.24 were plotted for this purpose and the statistical equations obtained are given below in the tabular form (Table- 4.7)

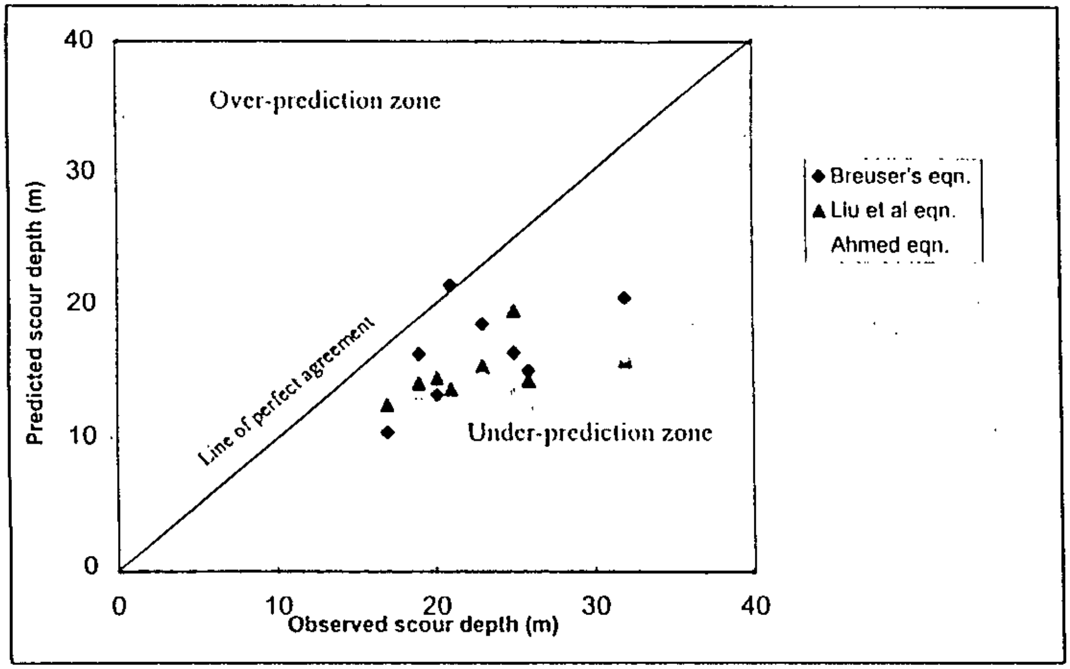

Fig. 4·21 : Comparison of predicted and observed value of scour depth for groyne

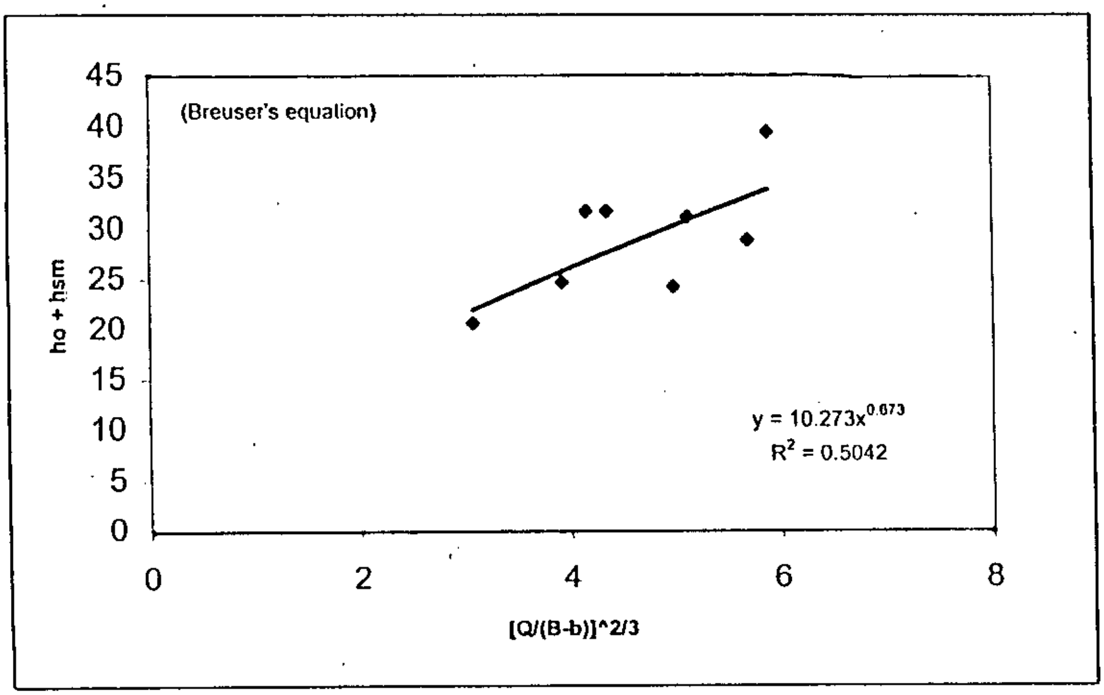

Fig. 4·22: Relation between (ho + hsm) and [Q/(B-b)]^2/3 for groyne

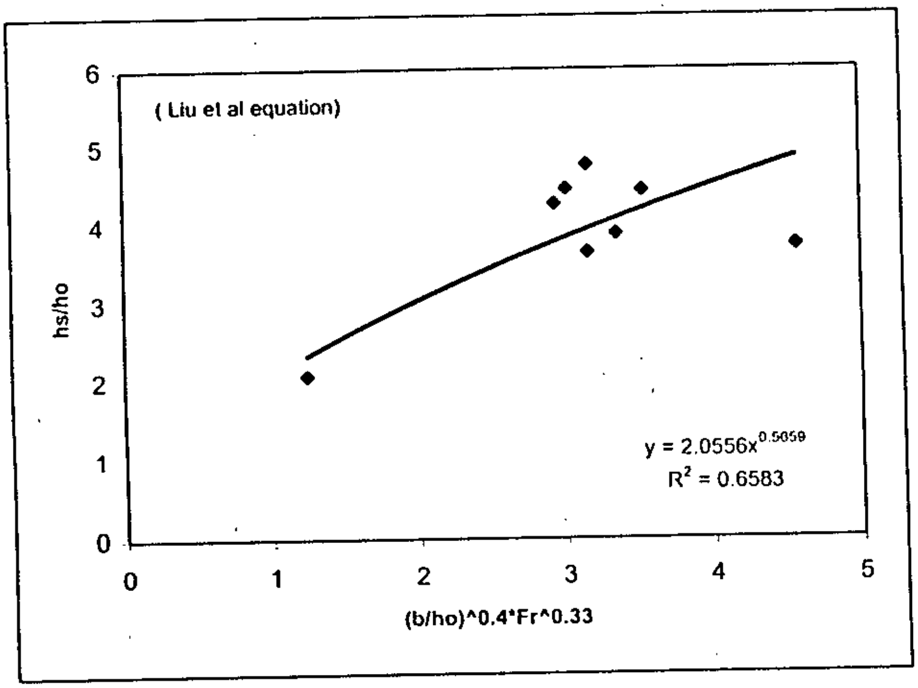

Fig. 4·23: Relation between (hs / ho) and (b/ ho)^0.4* Fr^0.33 for groyne

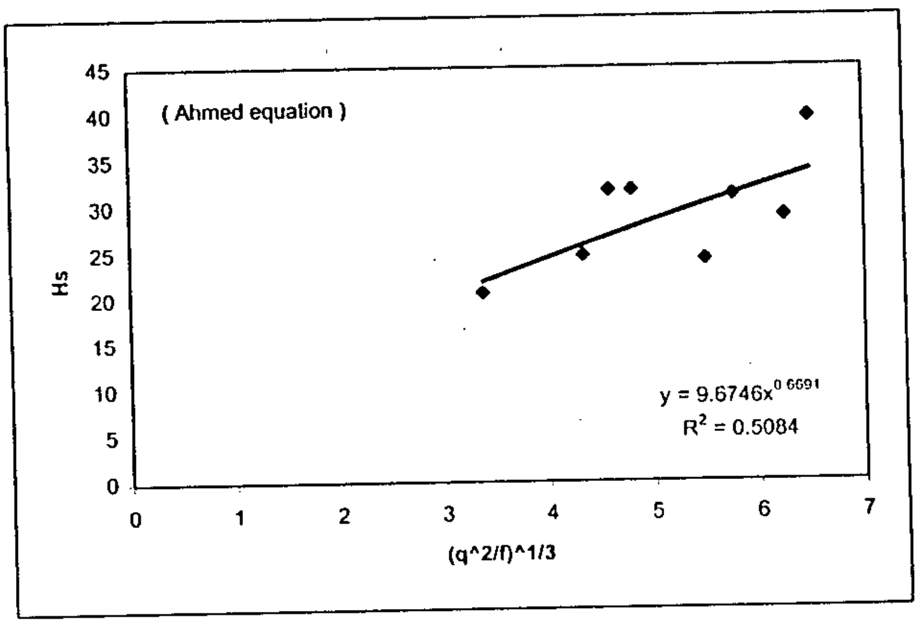

Fig. 4·24: Relation between Hs and (q^2/f)^1/3 for groyne

Table- 4.7 : . Correlation between various parameters for scour data of Groyne (RRI)

Name of the Investigator	Equations (SI unit)	Correlation
Breuser (1991)	$h_0+h_{sm}= 2.2[Q/(B-b)]^{2/3}$	$h_0+h_{sm} = 10.273\ [\{Q/(B-b)\}^{2/3}]^{0.673}$ (4.9) $R^2 = 0.5042$
Liu et al (1961)	$h_s/h_0= 2.15\ (b/h_0)^{0.4}F_r^{0.33}$	$h_s/h_0 = 2.0556\ [(b/h_0)^{0.4}F_r^{0.33}]^{0.5659}$ (4.10) R2 = 0.6583
Ahmed (1953)	$H_s = 1.34\ (q^2/f)^{1/3}$	$H_s = 9.6746\ [q^2/f)^{1/3}]^{0.6691}$ (4.11) $R^2 = 0.5084$

Equations 4.9 to 4.11 show similarity to the respective equations stated in chapter two. Only differences are in constants and in the value of the index power. Comparing equation 4.10 to Liu et. al formula, it is found that the variations are 4.7% and 43% in the values of the coefficient and index power respectively. This means that Liu et. al formula is suitable for this case.

Similar analysis was also been done for laboratory test data of groyne (RRI) as in the case of East-West Interconnector and Hardinge Bridge. In this case, the selected band of discrepancy ratio is 0.8-1.24. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in the tabular form (Table- 4.8).

Table- 4.8 : Comparison of Data coverage using RRI data of Groyne.

Name of Equations	% of data coverage for a range of discrepancy ratio				
	0.8-1.25	0.5-2.0	0.25-4.0	0.125-8.0	Rest
Breuser equation	38	100	-	-	-
Ahmed equation	13	75	100	-	-
Liu et al equation	-	100	-	-	-

From the criteria of data coverage between the discrepancy ratio 0.8-1.25 as shown in the above table, Breuser formula is found to the top of the ranking. This is followed by Ahmed, and Liu et al formula.

The deviation of scour depth obtained from the empirical equations and that of the field data may be due to the following reasons:

(a) Development of the empirical formulae from the small-scale laboratory investigation where many parameters were kept constant, while the conditions differ in the actual field.

(b) Conditions and nature of bed material may vary considerably at different points and depths, data for that were not available.

(c) The scour also depends on the skewness of the flow and on the state i.e. weather the flood is rising or falling. Precise observations regarding direction of current and state of flood were not available.

4.2 Effect of Size of Pier and Angle of Attack on Scour Depth

Some laboratory and field data for different pier size were collected to investigate the effect of size on scour depth using different well-known formula. Observed and predicted scour depth by Inglis, Larra and Blench formulae for this data are shown in the following table (Table- 4.9).

Table- 4.9: Scour data of different pier size

Data source	Size of Pier b, (ft.)	Approach flow depth y, (ft.)	q, (cfs/ft)	Depth of scour from WL (obs.) Dt, (ft.)	Dt/b (obs.)	Dt/b predicted by		
						Inglis	Blench	Larra
RRI (1996)	0.333	0.328	0.1076	0.705	2.117	1.257	1.780	2.854
RRI (1996)	0.167	0.328	0.1076	0.571	3.419	2.154	2.986	4.185
RRI (1996)	0.083	0.328	0.1076	0.469	4.651	3.716	4.045	6.597
Kabir (1984)	0.164	0.082	0.0648	0.159	0.970	1.678	1.070	2.731
RRI (1998)	0.109	1.092	1.8827	1.894	17.376	13.309	10.136	12.490
RRI (1998)	0.082	1.092	1.8827	1.712	20.878	16.617	12.548	14.971
RRI (1998)	0.109	0.364	0.3765	0.911	8.358	4.763	4.447	4.811
RRI (1998)	0.082	0.364	0.3765	0.765	9.329	7.196	4.505	7.093

The predicted and observed scour depth were plotted and are shown in the following figure (fig. - 4.25). It can be observed from the figure that the scour depth predicted by Blench equation is close to the observed value and gives better correlation ($R^2 = 0.9741$).

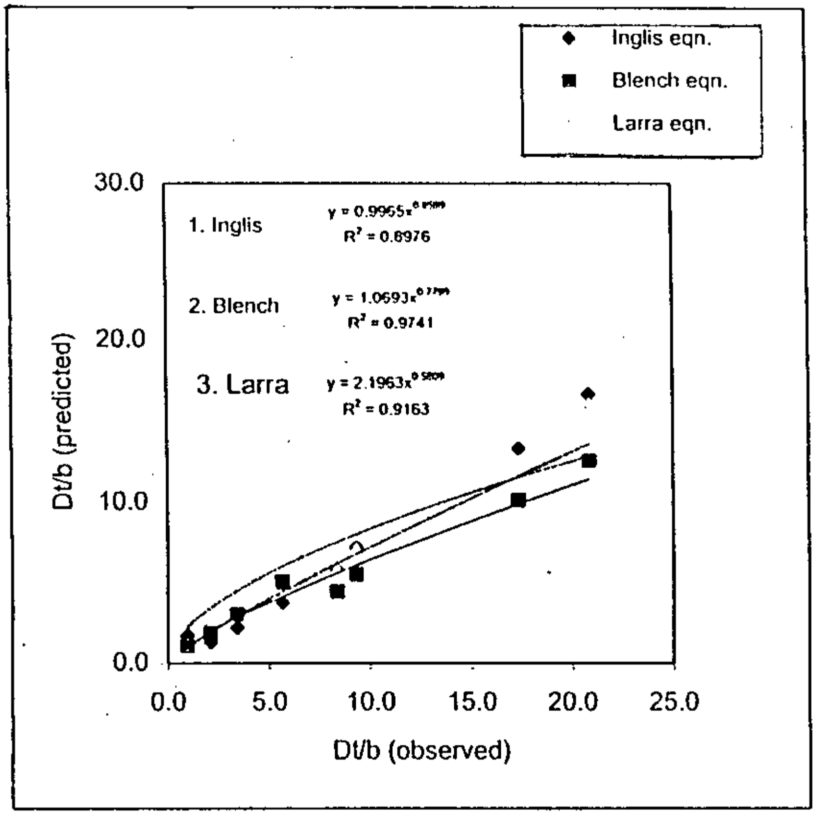

Fig. – 4.25 : Performance efficacy of different equations.

For pier shapes other than cylindrical, the depth of local scour depends on the alignment of the pier with flow. The local scour depth is related to the projected width of the pier, and this width increases rapidly with the angle of attack of flow. With an increasing angle of attack, the location of maximum scour depth moves along the exposed side of the pier from the front to the rear end of the pier. To investigate the effect of angle of attack on scour depth, equilibrium scour (mPWD) versus distance from left bank (m) graphs were plotted for upstream and downstream transect of different angle of attack (see fig. 4.26 & 4.27). It is observed from the figure that the depth of scour increases with the increase in angle of attack. Similar investigation were done for individual pile for Pier group-A (size = 0.082 ft.) and Pier group-B (size = 0.109 ft) and are shown in Fig.

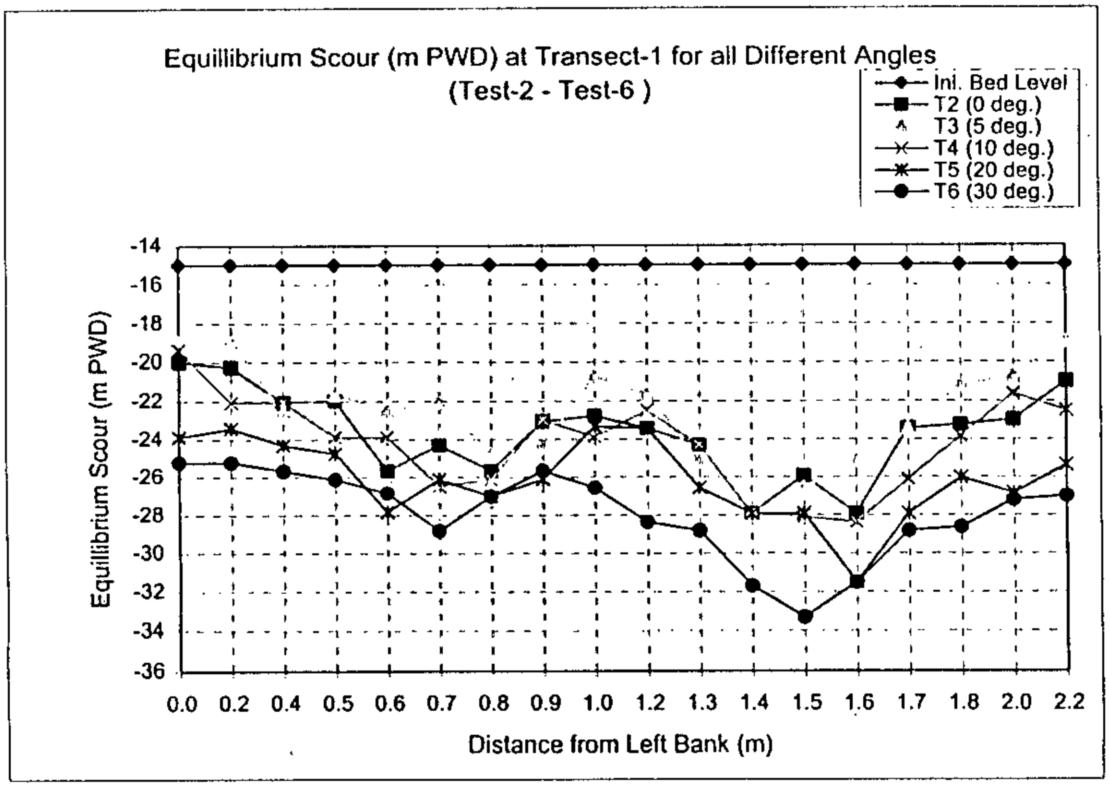

Fig. 4·26: Equillibrium Scour of Transect -1 (T-2 - T-6)

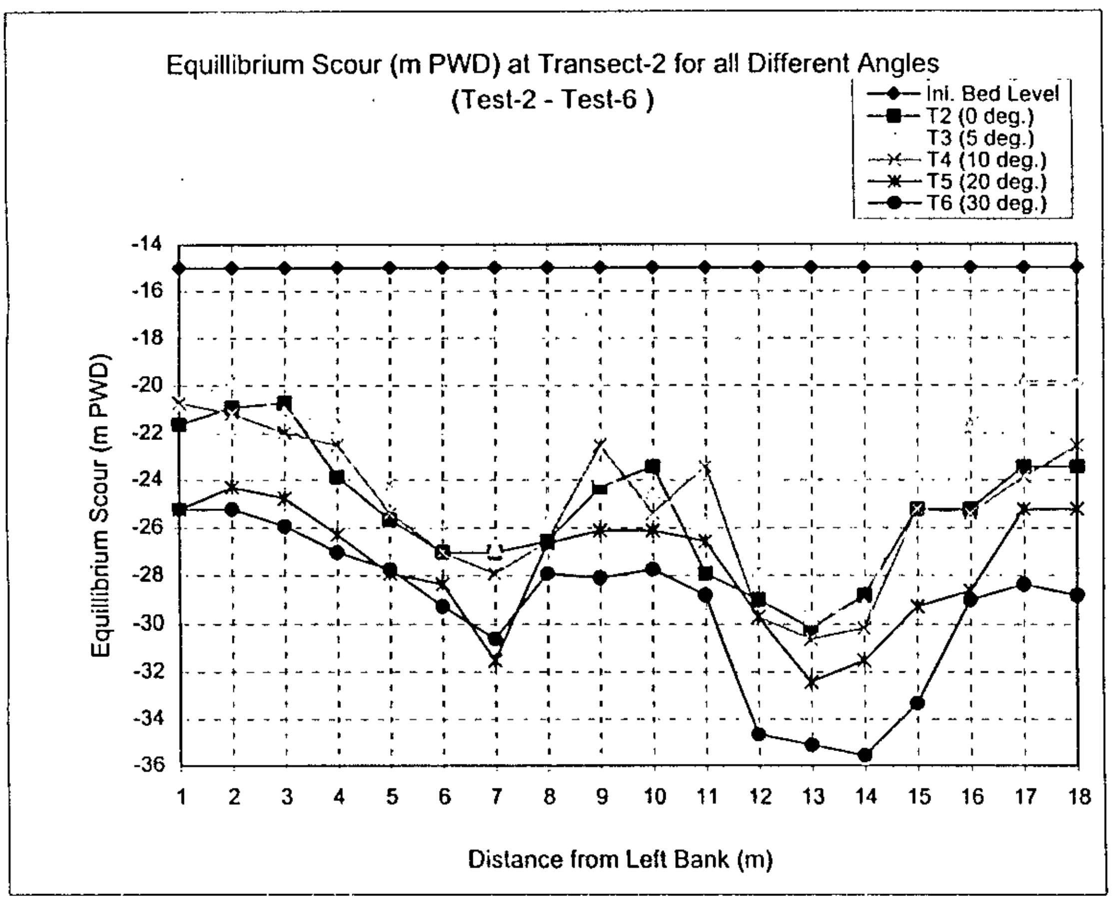

Fig. 4·27: Equillibrium Scour of Transect -2 (T-2 - T-6)

4.28 & Fig. 4.29 respectively. It is also observed from the figure that the depth of scour increases with the increase in angle of attack and gives highest value for 30^0 angle of attack (T-6).

4.3 Comparison of Field and Laboratory data

Observed scour depth in field and laboratory were compared with the scour depth predicted by various formulae, such as ; Inglis (1949), Ahmed (1962), Blench (1962) and Shen et al (1969). Figs. 4.30 to 4.33 and 4.34 to 4.37 are the plots of relative observed scour depths and predicted scour depths for circular pier (East-West Interconnector) and round-nose pier (Hardinge Bridge) respectively.

Circular Pier :

Figs. 4.30 through 4.33 shows the comparison between the observed scour depths and the scour depths predicted by various formulae for circular pier. Inglis relation (fig. 4.30) over predicted for both field and experimental data. Ahmed relation (fig. 4.31) also over predicted but some values are close and lie on the line of perfect agreement compared to Inglis relation. Blench relation (fig. 4.32) under predicted most of the experimental data and both under predicted and over predicted the field data. Some field and experimental data lies on the line of perfect agreement. Shen et al relation (fig. 4.33) under predicted most of the field data and both under predicted and over predicted the experimental data.

By comparing these figures, it may be concluded that the scour formula established by Blench (1962) is the best predictor among those compared in this study for circular pier.

Round-nose Pier :

The observed scour depths and the scour depths predicted by various formulae for round-nose pier are shown in figs.-4.34 through 4.37. It is observed that Inglis relation (fig. 4.34) under predicted the field data and over predicted the experimental data. Ahmed relation (fig. 4.35) also under predicted the field data and over predicted the experimental data and the values are more

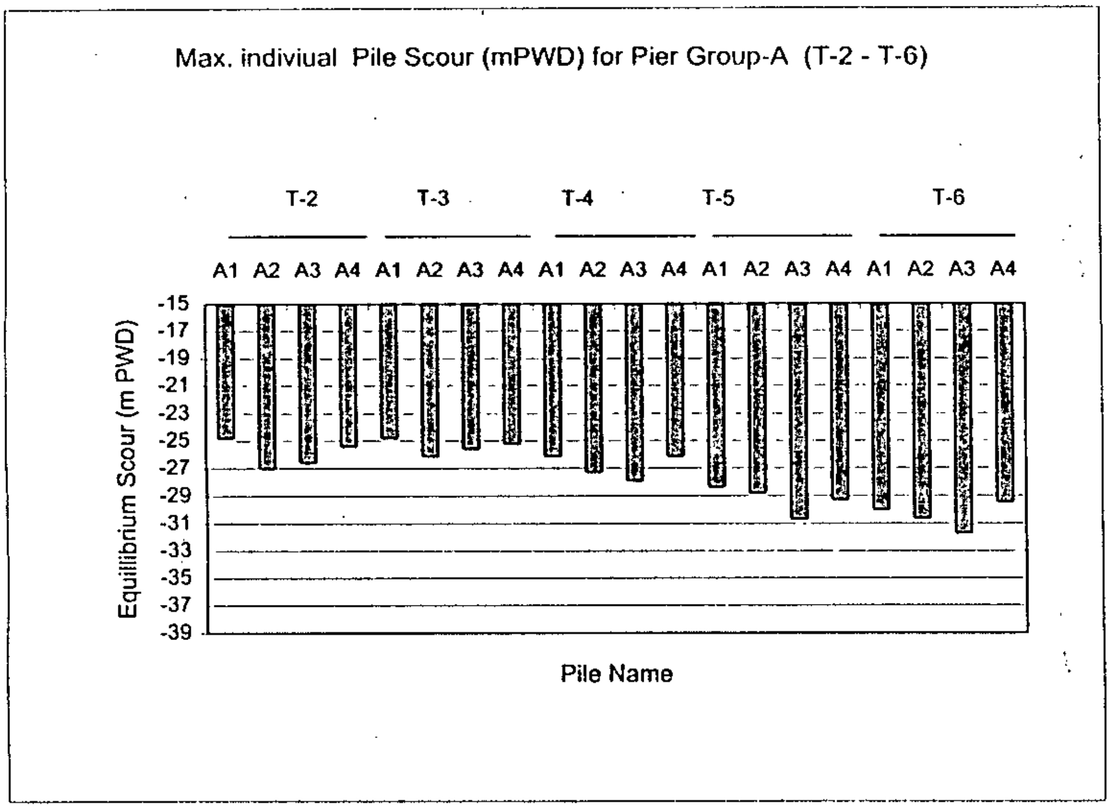

Fig. 4·28: Maximum individual pile scour for Pier Group-A (T-2 - T-6)

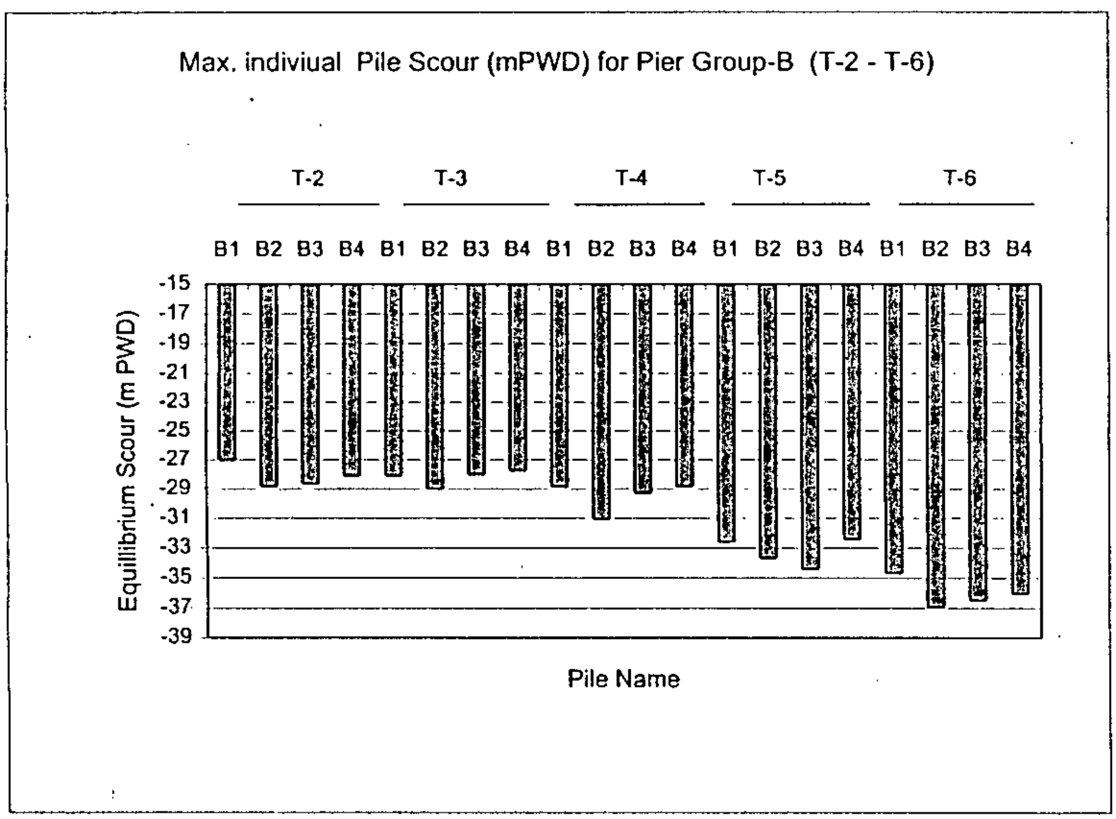

Fig. 4·29: Maximum individual pile scour for Pier Group-B (T-2 - T-6)

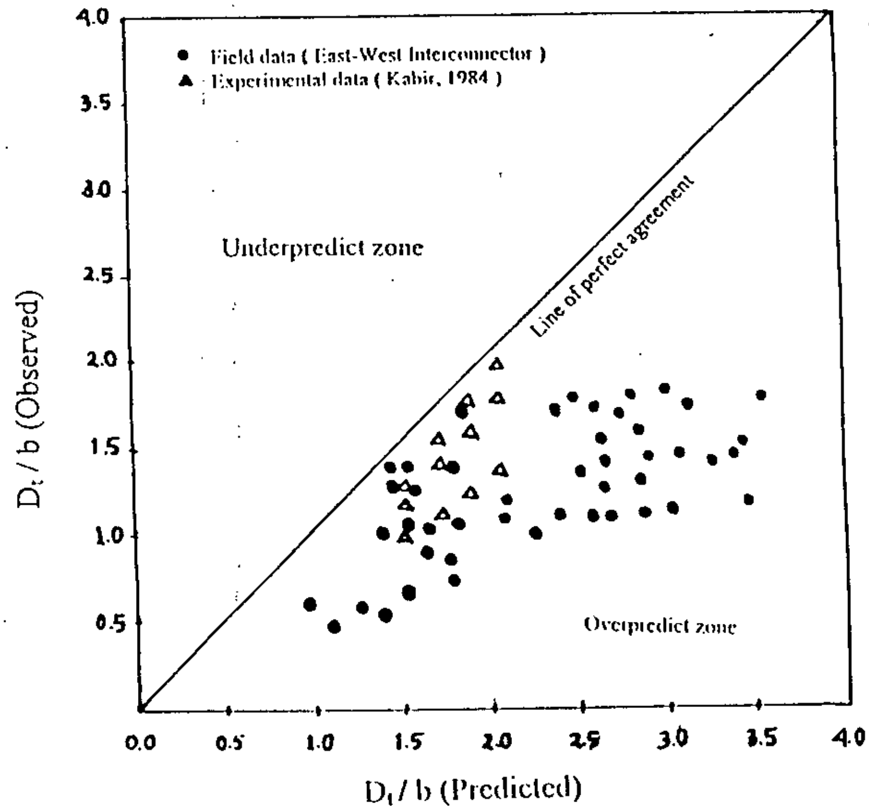

Fig. 4·30 : Comparison of Inglis Formula with Experimental and Field Data

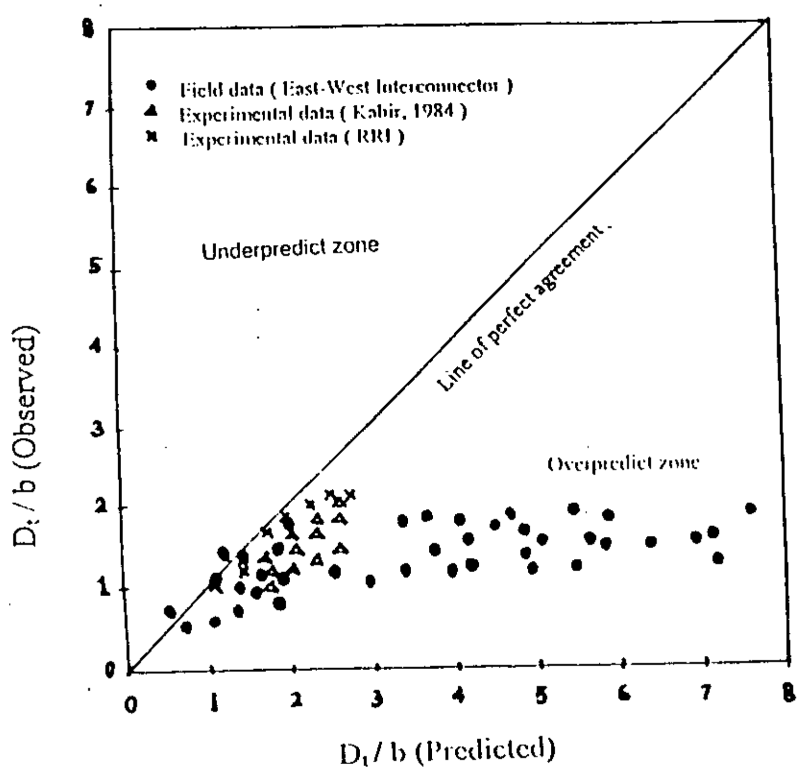

Fig. 4·31 : Comparison of Ahmed Formula with Experimental and Field Data

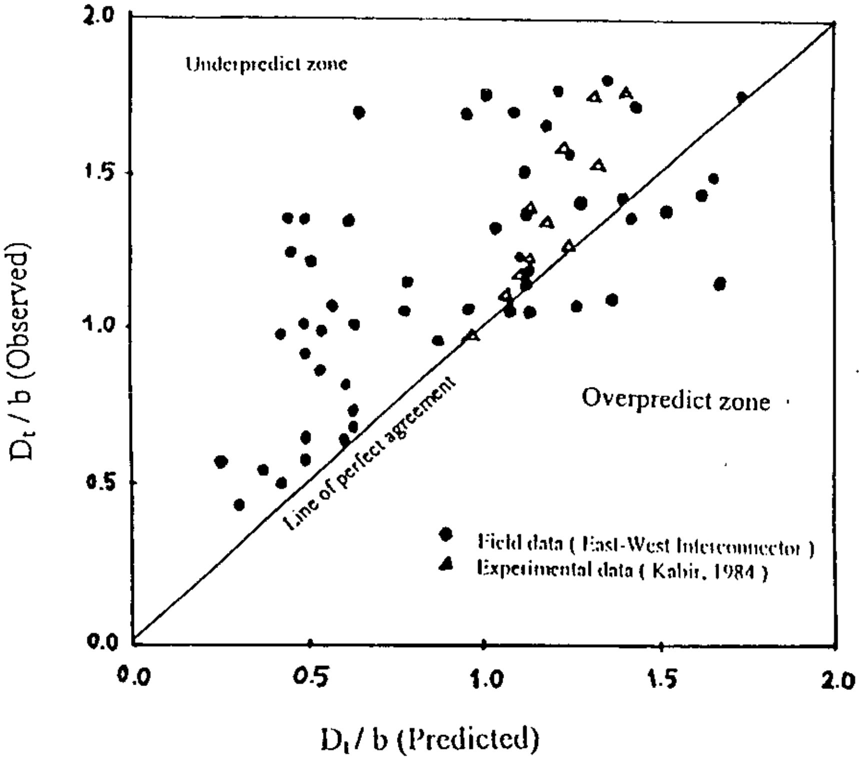

Fig. 4·32 : Comparison of Blench Formula with Experimental and Field Data

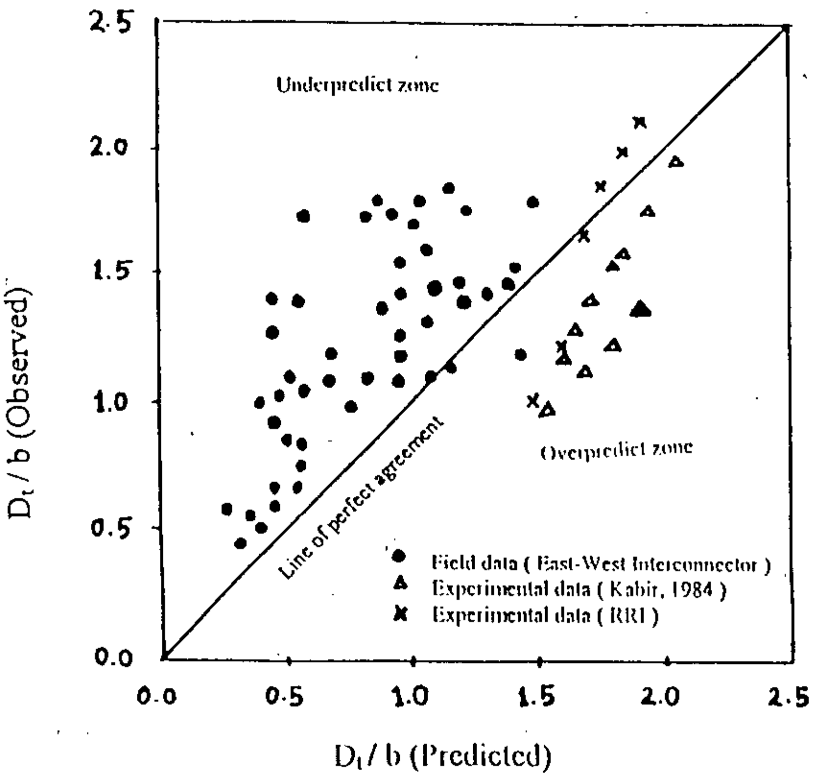

Fig. 4·33 : Comparison of Shen et. al Formula with Experimental and Field Data scatter than that of Inglis relation. Blench relation (fig. 4.36) shows good correlation though the data cluster round a small zone and most of the points lie along the line of perfect agreement. Shen et al relation (fig. 4.37) over predicted the experimental data and both under predicted, and over predicted the field data.

Blench (1962) relation also found to be the best predictor among those compared in this study for round-nose pier.

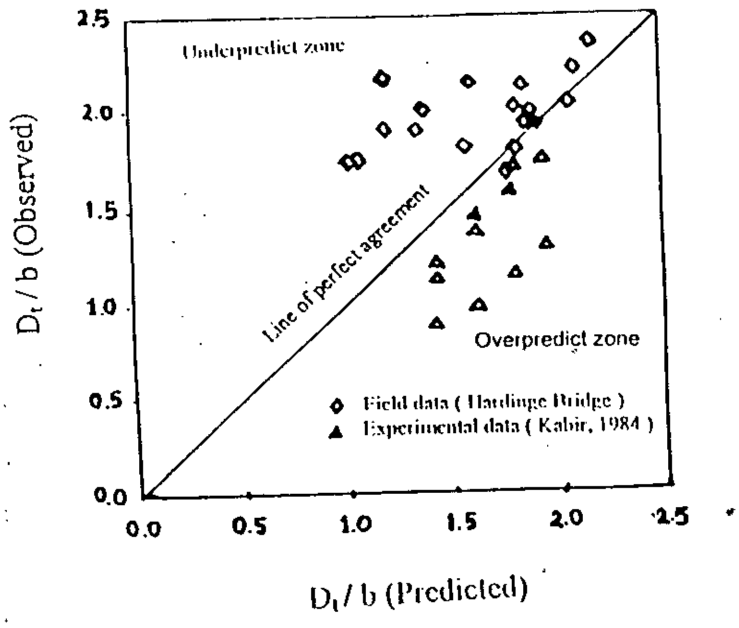

Fig. 4·34 : Comparison of Inglis Formula with Experimental and Field Data

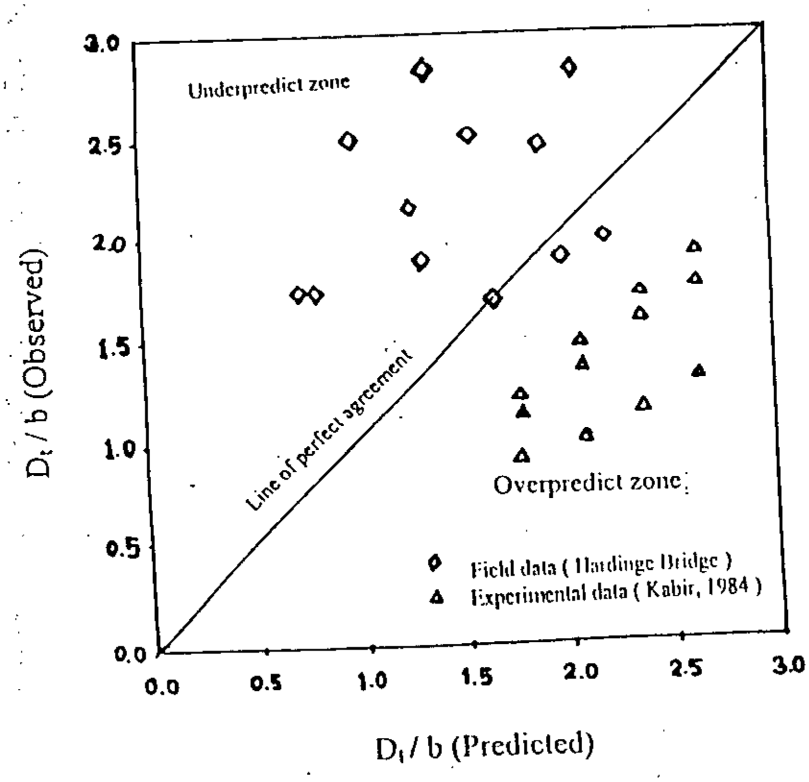

Fig. 4·35 : Comparison of Ahmed Formula with Experimental and Field Data

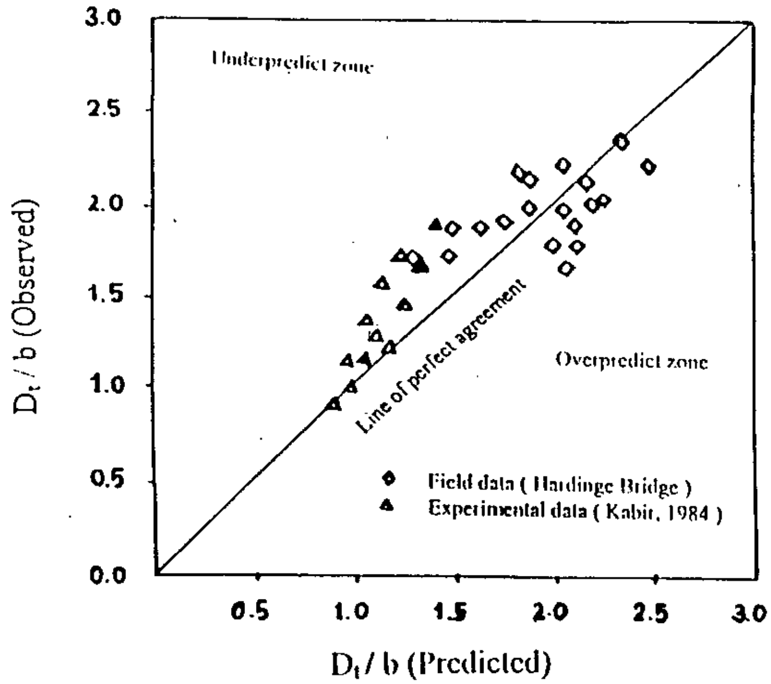

Fig. 4·36 : Comparison of Blench Formula with Experimental and Field Data

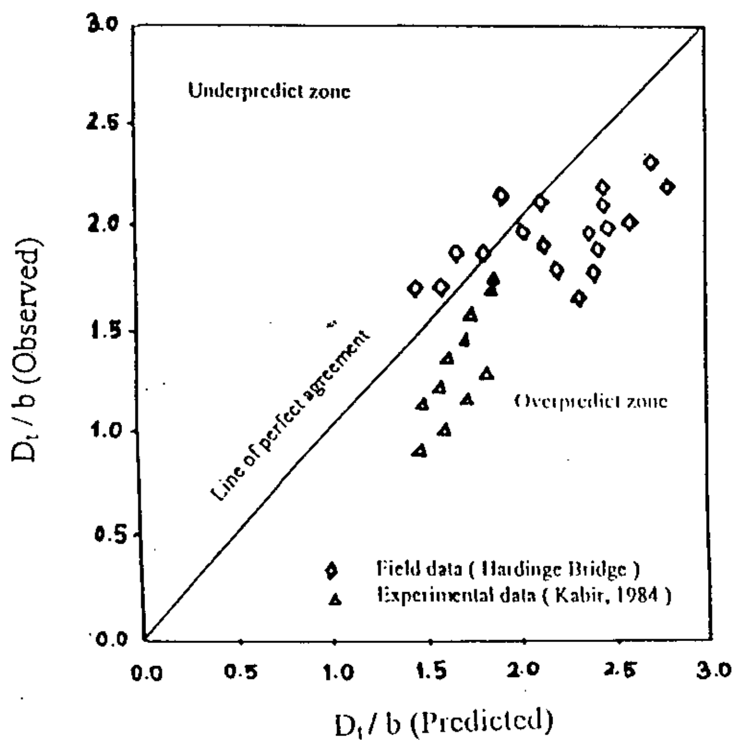

Fig. 4·37 : Comparison of Shen et. al Formula with Experimental and Field Data

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

After analyzing the data on the basis of scatter diagram, statistical analysis and discrepancy ratio, the following conclusion may be drawn for the present study:

(1) It appears that Blench (1962) formula could be a choice for prediction of scour depth in case of East-West Interconnector and Hardinge Bridge. It also appears that Inglis (1949) and Breuser (1991) formula could be used for prediction of scour depth against laboratory test data of pier and groyne respectively.

(2) Blench (1962) relation has been found to be a suitable predictor among those compared in this study on the basis of scatter plot and also on the basis of data coverage between the selected discrepancy bands. This formula was followed by Shen et al, Inglis and Ahmed formulae.

(3) It is found that scour depth increases with the increase in angle of attack and it gives highest value for 30^0 angle of attack. Scóur depth increase with the increase in pier size.

5.2 Recommendation

For understanding the mechanism of scour in an alluvial channel the following parameters are important :

a. Scour depth

b. Caisson, pier or groyne geometry

c. Flow velocity

d. River state

e. Discharge

f. River geometry

g. Properties of bed material and suspended load

h. Direction of flow

i. Water temperature

This study has utilized a small amount of data. To validate any formulae, regional and global data should be used. Therefore, it is recommended that future study should be undertaken on the basis of more data and established equations.

Special attention may be taken for the study of scour phenomenon around bridge pier and groyne by incorporating the following:

(i) In the present study, the scour phenomenons around circular piers were observed and the bed slope and bed materials were kept constant. Therefore, the effect of bed slope and bed materials on scour depth may be studied.

(ii) Small-scale studies for wider range of discharge in a broader tilting flume may be conducted to compare the field data of scour under similar conditions.

(iii) The effect of the flow direction with respect to the groyne or pier shape and position may be studied.

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APPENDIX – A

DATA TABLES

Table 1.1: Scour and related data for the E-W Interconnector (Caisson no. 2)

Diameter of the caisson, b = 35 ft.

Lacey's silt factor, f = 0.757

Width of the river along E-W Interconnector = 11.0 km

Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft.3/sec/ft.)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	66.21	5299	187001.71	5.18	6.21	0.83	1.89
Feb-97	64.88	3795	133925.55	3.71	4.97	0.75	1.85
mar-97	65.88	5081	179308.49	4.97	6.04	0.82	1.88
Apr-97	66.30	8567	302329.43	8.38	8.56	0.98	1.89
may-97	67.63	25650	905188.50	25.09	17.78	1.41	1.93
Jun-97	71.71	27600	974004.00	27.00	18.67	1.45	2.05
Jul-97	71.67	62150	2193273.50	60.79	32.07	1.90	2.05
Aug-97	69.84	41336	1458747.44	40.43	24.44	1.65	2.00
Sep-97	68.71	35282	1245101.78	34.51	21.99	1.57	1.96
oct-97	67.96	28345	1000295.05	27.72	19.00	1.46	1.94
Nov-97	67.21	13593	479696.97	13.30	11.64	1.14	1.92
Dec-97	65.10	6545	230973.05	6.40	7.15	0.90	1.86
Jan-96	44.96	5106	180190.74	4.99	6.06	0.82	1.28
Feb-96	35.80	3699	130537.71	3.62	4.89	0.74	1.02
mar-96	44.92	5081	179308.49	4.97	6.04	0.82	1.28
Apr-96	48.50	8467	298800.43	8.28	8.49	0.98	1.39
may-96	67.71	23650	834608.50	23.13	16.84	1.37	1.93
Jun-96	73.96	27648	975697.92	27.04	18.69	1.45	2.11
Jul-96	72.96	61154	2158124.66	59.81	31.73	1.89	2.08
Aug-96	70.87	43356	1530033.24	42.41	25.23	1.68	2.02
Sep-96	70.12	36584	1291049.36	35.78	22.53	1.59	2.00
oct-96	69.04	27648	975697.92	27.04	18.69	1.45	1.97
Nov-96	68.34	12694	447971.26	12.42	11.12	1.12	1.95
Dec-96	68.08	7040	248441.60	6.89	7.51	0.92	1.95
Jan-95	65.88	5186	183013.94	5.07	6.12	0.83	1.88
Feb-95	65.84	4321	152488.09	4.23	5.42	0.78	1.88
mar-95	61.96	4447	156934.63	4.35	5.53	0.79	1.77
Apr-95	60.96	5599	197588.71	5.48	6.45	0.85	1.74
may-95	85.96	22730	802141.70	22.23	16.40	1.36	2.46
Jun-95	86.75	41009	1447207.61	40.11	24.31	1.65	2.48
Jul-95	84.75	67344	2376569.76	65.87	33.84	1.95	2.42
Aug-95	82.75	45994	1623128.26	44.99	26.24	1.71	2.36
Sep-95	80.59	30971	1092966.59	30.29	20.16	1.50	2.30
oct-95	67.00	28026	989037.54	27.41	18.86	1.45	1.91
Nov-95	56.59	16589	585425.81	16.23	13.30	1.22	1.62
Dec-95	50.59	7966	281120.14	7.79	8.15	0.96	1.45
Jan-94	63.21	7750	273497.50	7.58	8.01	0.95	1.81
Feb-94	60.17	1230	43406.70	1.20	2.35	0.51	1.72
mar-94	59.92	1650	58228.50	1.61	2.85	0.57	1.71
Apr-94	66.25	1960	69168.40	1.92	3.20	0.60	1.89

Table 1.1 : (continued)

may-94	68.96	2850	100576.50	2.79	4.11	0.68	1.97
Jun-94	77.42	34522	1218281.38	33.77	21.67	1.56	2.21
Jul-94	81.42	32147	1134467.63	31.44	20.67	1.52	2.33
Aug-94	80.67	32980	1163864.20	32.26	21.02	1.53	2.30
Sep-94	78.34	26379	930914.91	25.80	18.11	1.42	2.24
oct-94	77.97	20121	710070.09	19.68	15.12	1.30	2.23
Nov-94	77.97	9145	322727.05	8.94	8.94	1.00	2.23
Dec-94	66.04	8320	293612.80	8.14	8.39	0.97	1.89
Jan-93	62.34	6261	220950.69	6.12	6.94	0.88	1.78
Feb-93	53.33	5223	184319.67	5.11	6.15	0.83	1.52
mar-93	54.96	6101	215304.29	5.97	6.83	0.87	1.57
Apr-93	60.63	8588	303070.52	8.40	8.57	0.98	1.73
may-93	62.84	20357	718398.53	19.91	15.24	1.31	1.80
Jun-93	72.75	45200	1595108.00	44.21	25.94	1.70	2.08
Jul-93	77.42	58719	2072193.51	57.43	30.88	1.86	2.21
Aug-93	79.84	51989	1834691.81	50.85	28.47	1.79	2.28
Sep-93	79.67	34664	1223292.56	33.91	21.73	1.56	2.28
oct-93	79.00	27331	964510.99	26.73	18.55	1.44	2.26
Nov-93	76.71	13400	472886.00	13.11	11.53	1.14	2.19
Dec-93	66.00	7996	282178.84	7.82	8.17	0.96	1.89
may-92	71.09	21357	753688.53	20.89	15.74	1.33	2.03
Jun-92	79.46	37565	1325668.85	36.74	22.93	1.60	2.27
Jul-92	83.96	58650	2069758.50	57.37	30.86	1.86	2.40
Aug-92	86.59	51950	1833315.50	50.81	28.46	1.79	2.47
Sep-92	80.59	34300	1210447.00	33.55	21.58	1.55	2.30
oct-92	77.04	27331	964510.99	26.73	18.55	1.44	2.20
Nov-92	70.59	13400	472886.00	13.11	11.53	1.14	2.02
Dec-92	67.37	7050	248794.50	6.90	7.52	0.92	1.92
Nov-90	14.00	8151	287648.79	7.97	8.28	0.96	0.40
Aug-89	29.34	48573	1714141.17	47.51	27.21	1.75	0.84
Jan-88	17.50	4810	169744.90	4.70	5.82	0.81	0.50
Feb-88	14.88	4440	156687.60	4.34	5.52	0.79	0.43
mar-88	15.42	4570	161275.30	4.47	5.63	0.79	0.44
Apr-88	16.76	6360	224444.40	6.22	7.02	0.89	0.48
may-88	25.60	10335	364722.15	10.11	9.70	1.04	0.73
Jun-88	30.20	28300	998707.00	27.68	18.98	1.46	0.86
Jul-88	36.13	68730	2425481.70	67.23	34.30	1.96	1.03
Aug-88	37.17	46440	1638867.60	45.42	26.41	1.72	1.06
Sep-88	32.50	45100	1591579.00	44.11	25.90	1.70	0.93
oct-88	30.34	41250	1455712.50	40.35	24.40	1.65	0.87
Nov-88	23.60	17000	599930.00	16.63	13.52	1.23	0.67
Dec-88	21.35	6300	222327.00	6.16	6.97	0.88	0.61

Table 1.2: Scour and related data for the E-W Interconnector (Caisson no. 3)

Diameter of the caisson, b = 35 ft.

Lacey's silt factor, f = 0.757

Width of the river along E-W Interconnector = 11.0 km

Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	35.80	5299	187001.71	5.18	6.21	0.83	1.02
Feb-97	34.50	3795	133925.55	3.71	4.97	0.75	0.99
mar-97	35.54	5081	179308.49	4.97	6.04	0.82	1.02
Apr-97	26.05	8567	302329.43	8.38	8.56	0.98	0.74
may-97	37.38	25650	905188.50	25.09	17.78	1.41	1.07
Jun-97	40.71	27600	974004.00	27.00	18.67	1.45	1.16
Jul-97	40.55	62150	2193273.50	60.79	32.07	1.90	1.16
Aug-97	38.87	41336	1458747.44	40.43	24.44	1.65	1.11
Sep-97	37.83	35282	1245101.78	34.51	21.99	1.57	1.08
oct-97	37.08	28345	1000295.05	27.72	19.00	1.46	1.06
Nov-97	36.79	13593	479696.97	13.30	11.64	1.14	1.05
Dec-97	29.37	6545	230973.05	6.40	7.15	0.90	0.84
Jan-96	22.79	5106	180190.74	4.99	6.06	0.82	0.65
Feb-96	17.54	3699	130537.71	3.62	4.89	0.74	0.50
mar-96	20.46	5081	179308.49	4.97	6.04	0.82	0.58
Apr-96	23.88	8467	298800.43	8.28	8.49	0.98	0.68
may-96	46.92	23650	834608.50	23.13	16.84	1.37	1.34
Jun-96	53.34	27648	975697.92	27.04	18.69	1.45	1.52
Jul-96	52.51	61154	2158124.66	59.81	31.73	1.89	1.50
Aug-96	50.42	43356	1530033.24	42.41	25.23	1.68	1.44
Sep-96	49.75	36584	1291049.36	35.78	22.53	1.59	1.42
oct-96	48.71	27648	975697.92	27.04	18.69	1.45	1.39
Nov-96	37.88	12694	447971.26	12.42	11.12	1.12	1.08
Dec-96	37.71	7040	248441.60	6.89	7.51	0.92	1.08
Jan-95	47.90	5186	183013.94	5.07	6.12	0.83	1.37
Feb-95	47.96	4321	152488.09	4.23	5.42	0.78	1.37
mar-95	44.00	4447	156934.63	4.35	5.53	0.79	1.26
Apr-95	43.00	5599	197588.71	5.48	6.45	0.85	1.23
may-95	62.13	22730	802141.70	22.23	16.40	1.36	1.78
Jun-95	63.80	41009	1447207.61	40.11	24.31	1.65	1.82
Jul-95	61.88	67344	2376569.76	65.87	33.84	1.95	1.77
Aug-95	60.63	45994	1623128.26	44.99	26.24	1.71	1.73
Sep-95	58.63	30971	1092966.59	30.29	20.16	1.50	1.68
oct-95	42.30	28026	989037.54	27.41	18.86	1.45	1.21
Nov-95	33.92	16589	585425.81	16.23	13.30	1.22	0.97
Dec-95	28.67	7966	281120.14	7.79	8.15	0.96	0.82
Jan-94	22.75	7750	273497.50	7.58	8.01	0.95	0.65
Feb-94	20.00	1230	43406.70	1.20	2.35	0.51	0.57

Table 1.2 : (continued)

mar-94	14.88	1650	58228.50	1.61	2.85	0.57	0.43
Apr-94	15.38	1960	69168.40	1.92	3.20	0.60	0.44
may-94	19.21	2850	100576.50	2.79	4.11	0.68	0.55
Jun-94	55.13	34522	1218281.38	33.77	21.67	1.56	1.58
Jul-94	61.58	32147	1134467.63	31.44	20.67	1.52	1.76
Aug-94	62.55	32980	1163864.20	32.26	21.02	1.53	1.79
Sep-94	60.05	26379	930914.91	25.80	18.11	1.42	1.72
oct-94	60.00	20121	710070.09	19.68	15.12	1.30	1.71
Nov-94	59.92	9145	322727.05	8.94	8.94	1.00	1.71
Dec-94	47.88	8320	293612.80	8.14	8.39	0.97	1.37
Jan-93	34.84	6261	220950.69	6.12	6.94	0.88	1.00
Feb-93	32.04	5223	184319.67	5.11	6.15	0.83	0.92
mar-93	30.50	6101	215304.29	5.97	6.83	0.87	0.87
Apr-93	35.88	8588	303070.52	8.40	8.57	0.98	1.03
may-93	37.67	20357	718398.53	19.91	15.24	1.31	1.08
Jun-93	48.04	45200	1595108.00	44.21	25.94	1.70	1.37
Jul-93	50.76	58719	2072193.51	57.43	30.88	1.86	1.45
Aug-93	48.84	51989	1834691.81	50.85	28.47	1.79	1.40
Sep-93	44.84	34664	1223292.56	33.91	21.73	1.56	1.28
oct-93	43.46	27331	964510.99	26.73	18.55	1.44	1.24
Nov-93	40.71	13400	472886.00	13.11	11.53	1.14	1.16
Dec-93	25.34	7996	282178.84	7.82	8.17	0.96	0.72

Table 1.3: Scour and related data for the E-W Interconnector (Caisson no. 4)

Diameter of the caisson, b = 35 ft.

Lacey's silt factor, f = 0.757

Width of the river along E-W Interconnector = 11.0 km

Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	33.67	5299	187001.71	5.18	6.21	0.83	0.96
Feb-97	32.34	3795	133925.55	3.71	4.29	0.87	0.92
mar-97	33.30	5081	179308.49	4.97	5.21	0.95	0.95
Apr-97	33.79	8567	302329.43	8.38	7.38	1.14	0.97
may-97	35.04	25650	905188.50	25.09	15.32	1.64	1.00
Jun-97	38.17	27600	974004.00	27.00	16.09	1.68	1.09
Jul-97	38.46	62150	2193273.50	60.79	27.64	2.20	1.10
Aug-97	36.88	41336	1458747.44	40.43	21.06	1.92	1.05
Sep-97	35.79	35282	1245101.78	34.51	18.95	1.82	1.02
oct-97	35.17	28345	1000295.05	27.72	16.38	1.69	1.00
Nov-97	34.58	13593	479696.97	13.30	10.03	1.33	0.99
Dec-97	21.00	6545	230973.05	6.40	6.16	1.04	0.60
Jan-96	16.17	5106	180190.74	4.99	5.22	0.96	0.46
Feb-96	11.01	3699	130537.71	3.62	4.21	0.86	0.31
mar-96	13.92	5081	179308.49	4.97	5.21	0.95	0.40
Apr-96	17.34	8467	298800.43	8.28	7.32	1.13	0.50
may-96	35.29	23650	834608.50	23.13	14.51	1.59	1.01
Jun-96	40.83	27648	975697.92	27.04	16.11	1.68	1.17
Jul-96	39.96	61154	2158124.66	59.81	27.34	2.19	1.14
Aug-96	37.55	43356	1530033.24	42.41	21.74	1.95	1.07
Sep-96	37.05	36584	1291049.36	35.78	19.41	1.84	1.06
oct-96	36.09	27648	975697.92	27.04	16.11	1.68	1.03
Nov-96	35.25	12694	447971.26	12.42	9.59	1.30	1.01
Dec-96	35.17	7040	248441.60	6.89	6.47	1.06	1.00
Jan-95	38.73	5186	183013.94	5.07	5.28	0.96	1.11
Feb-95	38.79	4321	152488.09	4.23	4.67	0.90	1.11
mar-95	34.71	4447	156934.63	4.35	4.76	0.91	0.99
Apr-95	33.71	5599	197588.71	5.48	5.55	0.99	0.96
may-95	57.29	22730	802141.70	22.23	14.14	1.57	1.64
Jun-95	59.83	41009	1447207.61	40.11	20.95	1.91	1.71
Jul-95	58.25	67344	2376569.76	65.87	29.16	2.26	1.66
Aug-95	58.38	45994	1623128.26	44.99	22.61	1.99	1.67
Sep-95	56.21	30971	1092966.59	30.29	17.37	1.74	1.61
oct-95	35.00	28026	989037.54	27.41	16.25	1.69	1.00
Nov-95	27.00	16589	585425.81	16.23	11.46	1.42	0.77
Dec-95	21.09	7966	281120.14	7.79	7.03	1.11	0.60
Jan-94	35.88	7750	273497.50	7.58	6.90	1.10	1.03
Feb-94	33.71	1230	43406.70	1.20	2.02	0.59	0.96

Table 1.3 (continued)

mar-94	31.67	1650	58228.50	1.61	2.46	0.66	0.90
Apr-94	38.63	1960	69168.40	1.92	2.76	0.69	1.10
may-94	40.38	2850	100576.50	2.79	3.54	0.79	1.15
Jun-94	48.75	34522	1218281.38	33.77	18.68	1.81	1.39
Jul-94	52.42	32147	1134467.63	31.44	17.81	1.77	1.50
Aug-94	53.34	32980	1163864.20	32.26	18.12	1.78	1.52
Sep-94	51.00	26379	930914.91	25.80	15.61	1.65	1.46
oct-94	50.91	20121	710070.09	19.68	13.03	1.51	1.45
Nov-94	50.96	9145	322727.05	8.94	7.70	1.16	1.46
Dec-94	38.79	8320	293612.80	8.14	7.23	1.13	1.11
Jan-93	26.55	6261	220950.69	6.12	5.98	1.02	0.76
Feb-93	23.67	5223	184319.67	5.11	5.30	0.96	0.68
mar-93	22.54	6101	215304.29	5.97	5.88	1.01	0.64
Apr-93	26.71	8588	303070.52	8.40	7.39	1.14	0.76
may-93	29.92	20357	718398.53	19.91	13.13	1.52	0.85
Jun-93	41.75	45200	1595108.00	44.21	22.35	1.98	1.19
Jul-93	45.67	58719	2072193.51	57.43	26.61	2.16	1.30
Aug-93	53.63	51989	1834691.81	50.85	24.54	2.07	1.53
Sep-93	56.59	34664	1223292.56	33.91	18.73	1.81	1.62
oct-93	55.29	27331	964510.99	26.73	15.98	1.67	1.58
Nov-93	51.88	13400	472886.00	13.11	9.94	1.32	1.48
Dec-93	38.75	7996	282178.84	7.82	7.04	1.11	1.11

Table 1.4: Scour and related data for the E-W Interconnector (Caisson no. 5)
Diameter of the caisson, b = 35 ft.
Lacey's silt factor, f = 0.757
Width of the river along E-W Interconnector = 11.0 km
Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	24.04	5299	187001.71	5.18	6.21	0.83	0.69
Feb-97	23.17	3795	133925.55	3.71	4.29	0.87	0.66
mar-97	24.04	5081	179308.49	4.97	5.21	0.95	0.69
Apr-97	24.50	8567	302329.43	8.38	7.38	1.14	0.70
may-97	25.80	25650	905188.50	25.09	15.32	1.64	0.74
Jun-97	28.71	27600	974004.00	27.00	16.09	1.68	0.82
Jul-97	28.50	62150	2193273.50	60.79	27.64	2.20	0.81
Aug-97	27.00	41336	1458747.44	40.43	21.06	1.92	0.77
Sep-97	25.96	35282	1245101.78	34.51	18.95	1.82	0.74
oct-97	25.25	28345	1000295.05	27.72	16.38	1.69	0.72
Nov-97	24.79	13593	479696.97	13.30	10.03	1.33	0.71
Dec-97	15.60	6545	230973.05	6.40	6.16	1.04	0.45
Jan-96	5.75	5106	180190.74	4.99	5.22	0.96	0.16
Feb-96	3.67	3699	130537.71	3.62	4.21	0.86	0.10
mar-96	7.04	5081	179308.49	4.97	5.21	0.95	0.20
Apr-96	10.21	8467	298800.43	8.28	7.32	1.13	0.29
may-96	24.75	23650	834608.50	23.13	14.51	1.59	0.71
Jun-96	29.92	27648	975697.92	27.04	16.11	1.68	0.85
Jul-96	29.34	61154	2158124.66	59.81	27.34	2.19	0.84
Aug-96	27.42	43356	1530033.24	42.41	21.74	1.95	0.78
Sep-96	27.13	36584	1291049.36	35.78	19.41	1.84	0.78
oct-96	25.91	27648	975697.92	27.04	16.11	1.68	0.74
Nov-96	25.13	12694	447971.26	12.42	9.59	1.30	0.72
Dec-96	25.09	7040	248441.60	6.89	6.47	1.06	0.72
Jan-95	34.12	5186	183013.94	5.07	5.28	0.96	0.97
Feb-95	34.25	4321	152488.09	4.23	4.67	0.90	0.98
mar-95	30.17	4447	156934.63	4.35	4.76	0.91	0.86
Apr-95	29.21	5599	197588.71	5.48	5.55	0.99	0.83
may-95	51.25	22730	802141.70	22.23	14.14	1.57	1.46
Jun-95	55.17	41009	1447207.61	40.11	20.95	1.91	1.58
Jul-95	52.26	67344	2376569.76	65.87	29.16	2.26	1.49
Aug-95	52.13	45994	1623128.26	44.99	22.61	1.99	1.49
Sep-95	50.21	30971	1092966.59	30.29	17.37	1.74	1.43
oct-95	35.20	28026	989037.54	27.41	16.25	1.69	1.01
Nov-95	24.79	16589	585425.81	16.23	11.46	1.42	0.71
Dec-95	14.04	7966	281120.14	7.79	7.03	1.11	0.40
Jan-94	31.84	7750	273497.50	7.58	6.90	1.10	0.91
Feb-94	29.21	1230	43406.70	1.20	2.02	0.59	0.83

Table 1.4 (continued)

mar-94	26.63	1650	58228.50	1.61	2.46	0.66	0.76
Apr-94	31.96	1960	69168.40	1.92	2.76	0.69	0.91
may-94	35.55	2850	100576.50	2.79	3.54	0.79	1.02
Jun-94	34.21	34522	1218281.38	33.77	18.68	1.81	0.98
Jul-94	53.46	32147	1134467.63	31.44	17.81	1.77	1.53
Aug-94	54.46	32980	1163864.20	32.26	18.12	1.78	1.56
Sep-94	52.05	26379	930914.91	25.80	15.61	1.65	1.49
oct-94	52.00	20121	710070.09	19.68	13.03	1.51	1.49
Nov-94	33.09	9145	322727.05	8.94	7.70	1.16	0.95
Dec-94	34.13	8320	293612.80	8.14	7.23	1.13	0.98
Jan-93	32.80	6261	220950.69	6.12	5.98	1.02	0.94
Feb-93	30.33	5223	184319.67	5.11	5.30	0.96	0.87
mar-93	32.17	6101	215304.29	5.97	5.88	1.01	0.92
Apr-93	37.75	8588	303070.52	8.40	7.39	1.14	1.08
may-93	39.25	20357	718398.53	19.91	13.13	1.52	1.12
Jun-93	49.46	45200	1595108.00	44.21	22.35	1.98	1.41
Jul-93	53.00	58719	2072193.51	57.43	26.61	2.16	1.51
Aug-93	56.26	51989	1834691.81	50.85	24.54	2.07	1.61
Sep-93	55.75	34664	1223292.56	33.91	18.73	1.81	1.59
oct-93	54.50	27331	964510.99	26.73	15.98	1.67	1.56
Nov-93	51.75	13400	472886.00	13.11	9.94	1.32	1.48
Dec-93	36.79	7996	282178.84	7.82	7.04	1.11	1.05

Table 1.5: Scour and related data for the E-W Interconnector (Caisson no.6)
Diameter of the caisson, b = 35 ft.
Lacey's silt factor, f = 0.757
Width of the river along E-W Interconnector = 11.0 km
Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	26.29	5299	187001.71	5.18	6.21	0.83	0.75
Feb-97	25.30	3795	133925.55	3.71	4.97	0.75	0.72
mar-97	26.92	5081	179308.49	4.97	6.04	0.82	0.77
Apr-97	27.38	8567	302329.43	8.38	8.56	0.98	0.78
may-97	28.47	25650	905188.50	25.09	17.78	1.41	0.81
Jun-97	31.29	27600	974004.00	27.00	18.67	1.45	0.89
Jul-97	31.17	62150	2193273.50	60.79	32.07	1.90	0.89
Aug-97	29.67	41336	1458747.44	40.43	24.44	1.65	0.85
Sep-97	28.04	35282	1245101.78	34.51	21.99	1.57	0.80
oct-97	27.29	28345	1000295.05	27.72	19.00	1.46	0.78
Nov-97	26.75	13593	479696.97	13.30	11.64	1.14	0.76
Dec-97	25.95	6545	230973.05	6.40	7.15	0.90	0.74
Jan-96	7.71	5106	180190.74	4.99	6.06	0.82	0.22
Feb-96	6.01	3699	130537.71	3.62	4.89	0.74	0.17
mar-96	14.21	5081	179308.49	4.97	6.04	0.82	0.41
Apr-96	17.04	8467	298800.43	8.28	8.49	0.98	0.49
may-96	27.79	23650	834608.50	23.13	16.84	1.37	0.79
Jun-96	33.38	27648	975697.92	27.04	18.69	1.45	0.95
Jul-96	32.05	61154	2158124.66	59.81	31.73	1.89	0.92
Aug-96	30.63	43356	1530033.24	42.41	25.23	1.68	0.88
Sep-96	29.55	36584	1291049.36	35.78	22.53	1.59	0.84
oct-96	28.30	27648	975697.92	27.04	18.69	1.45	0.81
Nov-96	28.17	12694	447971.26	12.42	11.12	1.12	0.80
Dec-96	27.96	7040	248441.60	6.89	7.51	0.92	0.80
Jan-95	45.05	5186	183013.94	5.07	6.12	0.83	1.29
Feb-95	45.09	4321	152488.09	4.23	5.42	0.78	1.29
mar-95	41.09	4447	156934.63	4.35	5.53	0.79	1.17
Apr-95	39.76	5599	197588.71	5.48	6.45	0.85	1.14
may-95	51.46	22730	802141.70	22.23	16.40	1.36	1.47
Jun-95	55.04	41009	1447207.61	40.11	24.31	1.65	1.57
Jul-95	53.09	67344	2376569.76	65.87	33.84	1.95	1.52
Aug-95	46.67	45994	1623128.26	44.99	26.24	1.71	1.33
Sep-95	44.42	30971	1092966.59	30.29	20.16	1.50	1.27
oct-95	35.70	28026	989037.54	27.41	18.86	1.45	1.02
Nov-95	26.55	16589	585425.81	16.23	13.30	1.22	0.76
Dec-95	7.51	7966	281120.14	7.79	8.15	0.96	0.21
Jan-94	14.04	7750	273497.50	7.58	8.01	0.95	0.40
Feb-94	10.71	1230	43406.70	1.20	2.35	0.51	0.31

Table 1.5 (continued)

mar-94	12.87	1650	58228.50	1.61	2.85	0.57	0.37
Apr-94	37.58	1960	69168.40	1.92	3.20	0.60	1.07
may-94	42.00	2850	100576.50	2.79	4.11	0.68	1.20
Jun-94	51.59	34522	1218281.38	33.77	21.67	1.56	1.47
Jul-94	68.71	32147	1134467.63	31.44	20.67	1.52	1.96
Aug-94	69.34	32980	1163864.20	32.26	21.02	1.53	1.98
Sep-94	51.75	26379	930914.91	25.80	18.11	1.42	1.48
oct-94	45.83	20121	710070.09	19.68	15.12	1.30	1.31
Nov-94	45.80	9145	322727.05	8.94	8.94	1.00	1.31
Dec-94	44.96	8320	293612.80	8.14	8.39	0.97	1.28
Jan-93	39.05	6261	220950.69	6.12	6.94	0.88	1.12
Feb-93	36.29	5223	184319.67	5.11	6.15	0.83	1.04
mar-93	51.92	6101	215304.29	5.97	6.83	0.87	1.48
Apr-93	40.54	8588	303070.52	8.40	8.57	0.98	1.16
may-93	40.96	20357	718398.53	19.91	15.24	1.31	1.17
Jun-93	32.84	45200	1595108.00	44.21	25.94	1.70	0.94
Jul-93	39.46	58719	2072193.51	57.43	30.88	1.86	1.13
Aug-93	41.50	51989	1834691.81	50.85	28.47	1.79	1.19
Sep-93	38.59	34664	1223292.56	33.91	21.73	1.56	1.10
oct-93	37.25	27331	964510.99	26.73	18.55	1.44	1.06
Nov-93	33.88	13400	472886.00	13.11	11.53	1.14	0.97
Dec-93	20.34	7996	282178.84	7.82	8.17	0.96	0.58
may-92	22.75	21357	753688.53	20.89	15.74	1.33	0.65
Jun-92	23.79	37565	1325668.85	36.74	22.93	1.60	0.68
Jul-92	30.92	58650	2069758.50	57.37	30.86	1.86	0.88
Aug-92	59.42	51950	1833315.50	50.81	28.46	1.79	1.70
Sep-92	55.84	34300	1210447.00	33.55	21.58	1.55	1.60
oct-92	54.00	27331	964510.99	26.73	18.55	1.44	1.54
Nov-92	64.75	13400	472886.00	13.11	11.53	1.14	1.85
Dec-92	41.37	7050	248794.50	6.90	7.52	0.92	1.18
Nov-90	34.25	8151	287648.79	7.97	8.28	0.96	0.98
Aug-89	46.80	48438	1709377.02	47.38	27.16	1.74	1.34
Jan-88	21.50	4810	169744.90	4.70	5.82	0.81	0.61
Feb-88	24.63	4440	156687.60	4.34	5.52	0.79	0.70
mar-88	24.87	4570	161275.30	4.47	5.63	0.79	0.71
Apr-88	25.96	6360	224444.40	6.22	7.02	0.89	0.74
may-88	47.21	10335	364722.15	10.11	9.70	1.04	1.35
Jun-88	47.30	28300	998707.00	27.68	18.98	1.46	1.35
Jul-88	36.13	68730	2425481.70	67.23	34.30	1.96	1.03
Aug-88	35.00	46440	1638867.60	45.42	26.41	1.72	1.00
Sep-88	33.50	45100	1591579.00	44.11	25.90	1.70	0.96
oct-88	31.20	41250	1455712.50	40.35	24.40	1.65	0.89
Nov-88	25.10	17000	599930.00	16.63	13.52	1.23	0.72
Dec-88	22.30	6300	222327.00	6.16	6.97	0.88	0.64

Table 1.6: Scour and related data for the E-W Interconnector (Caisson no. 7)
Diameter of the caisson, b = 35 ft.
Lacey's silt factor, f = 0.757
Width of the river along E-W Interconnector = 11.0 km
Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	21.79	5299	187001.71	5.18	6.21	0.83	0.62
Feb-97	20.71	3795	133925.55	3.71	4.29	0.87	0.59
mar-97	21.83	5081	179308.49	4.97	5.21	0.95	0.62
Apr-97	22.30	8567	302329.43	8.38	7.38	1.14	0.64
may-97	23.67	25650	905188.50	25.09	15.32	1.64	0.68
Jun-97	26.63	27600	974004.00	27.00	16.09	1.68	0.76
Jul-97	26.54	62150	2193273.50	60.79	27.64	2.20	0.76
Aug-97	25.08	41336	1458747.44	40.43	21.06	1.92	0.72
Sep-97	24.12	35282	1245101.78	34.51	18.95	1.82	0.69
oct-97	23.46	28345	1000295.05	27.72	16.38	1.69	0.67
Nov-97	23.08	13593	479696.97	13.30	10.03	1.33	0.66
Dec-97	15.68	6545	230973.05	6.40	6.16	1.04	0.45
Jan-96	3.79	5106	180190.74	4.99	5.22	0.96	0.11
Feb-96	4.71	3699	130537.71	3.62	4.21	0.86	0.13
mar-96	3.92	5081	179308.49	4.97	5.21	0.95	0.11
Apr-96	6.92	8467	298800.43	8.28	7.32	1.13	0.20
may-96	21.17	23650	834608.50	23.13	14.51	1.59	0.60
Jun-96	27.71	27648	975697.92	27.04	16.11	1.68	0.79
Jul-96	26.79	61154	2158124.66	59.81	27.34	2.19	0.77
Aug-96	25.13	43356	1530033.24	42.41	21.74	1.95	0.72
Sep-96	24.96	36584	1291049.36	35.78	19.41	1.84	0.71
oct-96	23.71	27648	975697.92	27.04	16.11	1.68	0.68
Nov-96	23.04	12694	447971.26	12.42	9.59	1.30	0.66
Dec-96	22.92	7040	248441.60	6.89	6.47	1.06	0.65
Jan-95	12.79	5186	183013.94	5.07	5.28	0.96	0.37
Feb-95	12.71	4321	152488.09	4.23	4.67	0.90	0.36
mar-95	8.75	4447	156934.63	4.35	4.76	0.91	0.25
Apr-95	7.79	5599	197588.71	5.48	5.55	0.99	0.22
may-95	21.79	22730	802141.70	22.23	14.14	1.57	0.62
Jun-95	26.09	41009	1447207.61	40.11	20.95	1.91	0.75
Jul-95	24.09	67344	2376569.76	65.87	29.16	2.26	0.69
Aug-95	32.34	45994	1623128.26	44.99	22.61	1.99	0.92
Sep-95	30.17	30971	1092966.59	30.29	17.37	1.74	0.86
oct-95	17.50	28026	989037.54	27.41	16.25	1.69	0.50
Nov-95	5.07	16589	585425.81	16.23	11.46	1.42	0.14
Dec-95	0.04	7966	281120.14	7.79	7.03	1.11	0.00
Jan-94	6.17	7750	273497.50	7.58	6.90	1.10	0.18
Feb-94	4.63	1230	43406.70	1.20	2.02	0.59	0.13

Table 1.6 (continued)

mar-94	4.79	1650	58228.50	1.61	2.46	0.66	0.14
Apr-94	11.50	1960	69168.40	1.92	2.76	0.69	0.33
may-94	15.25	2850	100576.50	2.79	3.54	0.79	0.44
Jun-94	24.63	34522	1218281.38	33.77	18.68	1.81	0.70
Jul-94	24.88	32147	1134467.63	31.44	17.81	1.77	0.71
Aug-94	26.79	32980	1163864.20	32.26	18.12	1.78	0.77
Sep-94	24.46	26379	930914.91	25.80	15.61	1.65	0.70
oct-94	24.29	20121	710070.09	19.68	13.03	1.51	0.69
Nov-94	24.37	9145	322727.05	8.94	7.70	1.16	0.70
Dec-94	12.63	8320	293612.80	8.14	7.23	1.13	0.36
Jan-93	12.38	6261	220950.69	6.12	5.98	1.02	0.35
Feb-93	8.09	5223	184319.67	5.11	5.30	0.96	0.23
mar-93	9.71	6101	215304.29	5.97	5.88	1.01	0.28
Apr-93	12.92	8588	303070.52	8.40	7.39	1.14	0.37
may-93	15.25	20357	718398.53	19.91	13.13	1.52	0.44
Jun-93	19.75	45200	1595108.00	44.21	22.35	1.98	0.56
Jul-93	24.84	58719	2072193.51	57.43	26.61	2.16	0.71
Aug-93	28.75	51989	1834691.81	50.85	24.54	2.07	0.82
Sep-93	25.38	34664	1223292.56	33.91	18.73	1.81	0.73
oct-93	23.80	27331	964510.99	26.73	15.98	1.67	0.68
Nov-93	20.92	13400	472886.00	13.11	9.94	1.32	0.60
Dec-93	8.46	7996	282178.84	7.82	7.04	1.11	0.24

Table 1.7: Scour and related data for the E-W Interconnector (Caisson no. 8)
Diameter of the caisson, b = 35 ft.
Lacey's silt factor, f = 0.757
Width of the river along E-W Interconnector = 11.0 km
Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	20.83	5299	187001.71	5.18	6.21	0.83	0.60
Feb-97	19.83	3795	133925.55	3.71	4.29	0.87	0.57
mar-97	20.96	5081	179308.49	4.97	5.21	0.95	0.60
Apr-97	21.46	8567	302329.43	8.38	7.38	1.14	0.61
may-97	22.79	25650	905188.50	25.09	15.32	1.64	0.65
Jun-97	25.75	27600	974004.00	27.00	16.09	1.68	0.74
Jul-97	25.71	62150	2193273.50	60.79	27.64	2.20	0.73
Aug-97	24.21	41336	1458747.44	40.43	21.06	1.92	0.69
Sep-97	23.17	35282	1245101.78	34.51	18.95	1.82	0.66
oct-97	22.63	28345	1000295.05	27.72	16.38	1.69	0.65
Nov-97	22.13	13593	479696.97	13.30	10.03	1.33	0.63
Dec-97	8.56	6545	230973.05	6.40	6.16	1.04	0.24
Jan-96	4.71	5106	180190.74	4.99	5.22	0.96	0.13
Feb-96	4.13	3699	130537.71	3.62	4.21	0.86	0.12
mar-96	3.92	5081	179308.49	4.97	5.21	0.95	0.11
Apr-96	7.01	8467	298800.43	8.28	7.32	1.13	0.20
may-96	19.92	23650	834608.50	23.13	14.51	1.59	0.57
Jun-96	26.66	27648	975697.92	27.04	16.11	1.68	0.76
Jul-96	25.80	61154	2158124.66	59.81	27.34	2.19	0.74
Aug-96	24.09	43356	1530033.24	42.41	21.74	1.95	0.69
Sep-96	23.80	36584	1291049.36	35.78	19.41	1.84	0.68
oct-96	22.58	27648	975697.92	27.04	16.11	1.68	0.65
Nov-96	22.12	12694	447971.26	12.42	9.59	1.30	0.63
Dec-96	21.71	7040	248441.60	6.89	6.47	1.06	0.62
Jan-95	12.05	5186	183013.94	5.07	5.28	0.96	0.34
Feb-95	11.96	4321	152488.09	4.23	4.67	0.90	0.34
mar-95	7.96	4447	156934.63	4.35	4.76	0.91	0.23
Apr-95	6.96	5599	197588.71	5.48	5.55	0.99	0.20
may-95	20.62	22730	802141.70	22.23	14.14	1.57	0.59
Jun-95	24.96	41009	1447207.61	40.11	20.95	1.91	0.71
Jul-95	22.71	67344	2376569.76	65.87	29.16	2.26	0.65
Aug-95	27.08	45994	1623128.26	44.99	22.61	1.99	0.77
Sep-95	24.92	30971	1092966.59	30.29	17.37	1.74	0.71
oct-95	13.20	28026	989037.54	27.41	16.25	1.69	0.38
Nov-95	2.13	16589	585425.81	16.23	11.46	1.42	0.06
Dec-95	1.62	7966	281120.14	7.79	7.03	1.11	0.05
Jan-94	7.88	7750	273497.50	7.58	6.90	1.10	0.23
Feb-94	5.58	1230	43406.70	1.20	2.02	0.59	0.16

Table 1.7 (continued)

mar-94	5.75	1650	58228.50	1.61	2.46	0.66	0.16
Apr-94	10.59	1960	69168.40	1.92	2.76	0.69	0.30
may-94	14.88	2850	100576.50	2.79	3.54	0.79	0.43
Jun-94	22.50	34522	1218281.38	33.77	18.68	1.81	0.64
Jul-94	24.75	32147	1134467.63	31.44	17.81	1.77	0.71
Aug-94	26.75	32980	1163864.20	32.26	18.12	1.78	0.76
Sep-94	23.96	26379	930914.91	25.80	15.61	1.65	0.68
oct-94	23.92	20121	710070.09	19.68	13.03	1.51	0.68
Nov-94	23.63	9145	322727.05	8.94	7.70	1.16	0.68
Dec-94	12.04	8320	293612.80	8.14	7.23	1.13	0.34
Jan-93	32.55	6261	220950.69	6.12	5.98	1.02	0.93
Feb-93	26.12	5223	184319.67	5.11	5.30	0.96	0.75
mar-93	24.34	6101	215304.29	5.97	5.88	1.01	0.70
Apr-93	27.59	8588	303070.52	8.40	7.39	1.14	0.79
may-93	29.46	20357	718398.53	19.91	13.13	1.52	0.84
Jun-93	36.29	45200	1595108.00	44.21	22.35	1.98	1.04
Jul-93	38.63	58719	2072193.51	57.43	26.61	2.16	1.10
Aug-93	40.88	51989	1834691.81	50.85	24.54	2.07	1.17
Sep-93	36.71	34664	1223292.56	33.91	18.73	1.81	1.05
oct-93	26.42	27331	964510.99	26.73	15.98	1.67	0.75
Nov-93	22.25	13400	472886.00	13.11	9.94	1.32	0.64
Dec-93	16.53	7996	282178.84	7.82	7.04	1.11	0.47

Table 1.8: Scour and related data for the E-W Interconnector (Caisson no. 9)

Diameter of the caisson, b = 35 ft.

Lacey's silt factor, f = 0.757

Width of the river along E-W Interconnector = 11.0 km

Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	25.63	5299	187001.71	5.18	6.21	0.83	0.73
Feb-97	24.29	3795	133925.55	3.71	4.29	0.87	0.69
mar-97	25.37	5081	179308.49	4.97	5.21	0.95	0.72
Apr-97	25.96	8567	302329.43	8.38	7.38	1.14	0.74
may-97	27.04	25650	905188.50	25.09	15.32	1.64	0.77
Jun-97	29.92	27600	974004.00	27.00	16.09	1.68	0.85
Jul-97	29.91	62150	2193273.50	60.79	27.64	2.20	0.85
Aug-97	28.33	41336	1458747.44	40.43	21.06	1.92	0.81
Sep-97	27.29	35282	1245101.78	34.51	18.95	1.82	0.78
oct-97	26.63	28345	1000295.05	27.72	16.38	1.69	0.76
Nov-97	26.12	13593	479696.97	13.30	10.03	1.33	0.75
Dec-97	15.37	6545	230973.05	6.40	6.16	1.04	0.44
Jan-96	5.26	5106	180190.74	4.99	5.22	0.96	0.15
Feb-96	4.00	3699	130537.71	3.62	4.21	0.86	0.11
mar-96	3.79	5081	179308.49	4.97	5.21	0.95	0.11
Apr-96	37.29	8467	298800.43	8.28	7.32	1.13	1.07
may-96	23.04	23650	834608.50	23.13	14.51	1.59	0.66
Jun-96	31.84	27648	975697.92	27.04	16.11	1.68	0.91
Jul-96	31.04	61154	2158124.66	59.81	27.34	2.19	0.89
Aug-96	29.33	43356	1530033.24	42.41	21.74	1.95	0.84
Sep-96	28.88	36584	1291049.36	35.78	19.41	1.84	0.83
oct-96	27.79	27648	975697.92	27.04	16.11	1.68	0.79
Nov-96	27.00	12694	447971.26	12.42	9.59	1.30	0.77
Dec-96	26.87	7040	248441.60	6.89	6.47	1.06	0.77
Jan-95	12.63	5186	183013.94	5.07	5.28	0.96	0.36
Feb-95	12.58	4321	152488.09	4.23	4.67	0.90	0.36
mar-95	8.63	4447	156934.63	4.35	4.76	0.91	0.25
Apr-95	7.63	5599	197588.71	5.48	5.55	0.99	0.22
may-95	32.92	22730	802141.70	22.23	14.14	1.57	0.94
Jun-95	34.67	41009	1447207.61	40.11	20.95	1.91	0.99
Jul-95	32.83	67344	2376569.76	65.87	29.16	2.26	0.94
Aug-95	32.79	45994	1623128.26	44.99	22.61	1.99	0.94
Sep-95	30.71	30971	1092966.59	30.29	17.37	1.74	0.88
oct-95	21.50	28026	989037.54	27.41	16.25	1.69	0.61
Nov-95	9.67	16589	585425.81	16.23	11.46	1.42	0.28
Dec-95	5.26	7966	281120.14	7.79	7.03	1.11	0.15
Jan-94	13.38	7750	273497.50	7.58	6.90	1.10	0.38
Feb-94	13.71	1230	43406.70	1.20	2.02	0.59	0.39

Table 1.8 (continued)

mar-94	15.38	1650	58228.50	1.61	2.46	0.66	0.44
Apr-94	22.54	1960	69168.40	1.92	2.76	0.69	0.64
may-94	26.00	2850	100576.50	2.79	3.54	0.79	0.74
Jun-94	34.88	34522	1218281.38	33.77	18.68	1.81	1.00
Jul-94	25.42	32147	1134467.63	31.44	17.81	1.77	0.73
Aug-94	27.00	32980	1163864.20	32.26	18.12	1.78	0.77
Sep-94	24.63	26379	930914.91	25.80	15.61	1.65	0.70
oct-94	24.67	20121	710070.09	19.68	13.03	1.51	0.70
Nov-94	11.67	9145	322727.05	8.94	7.70	1.16	0.33
Dec-94	12.79	8320	293612.80	8.14	7.23	1.13	0.37
Jan-93	28.09	6261	220950.69	6.12	5.98	1.02	0.80
Feb-93	25.21	5223	184319.67	5.11	5.30	0.96	0.72
mar-93	22.63	6101	215304.29	5.97	5.88	1.01	0.65
Apr-93	33.37	8588	303070.52	8.40	7.39	1.14	0.95
may-93	35.00	20357	718398.53	19.91	13.13	1.52	1.00
Jun-93	36.58	45200	1595108.00	44.21	22.35	1.98	1.05
Jul-93	39.71	58719	2072193.51	57.43	26.61	2.16	1.13
Aug-93	41.92	51989	1834691.81	50.85	24.54	2.07	1.20
Sep-93	35.59	34664	1223292.56	33.91	18.73	1.81	1.02
oct-93	34.29	27331	964510.99	26.73	15.98	1.67	0.98
Nov-93	31.21	13400	472886.00	13.11	9.94	1.32	0.89
Dec-93	17.46	7996	282178.84	7.82	7.04	1.11	0.50

Table 1.9: Scour and related data for the E-W Interconnector (Caisson no. 10)

Diameter of the caisson, b = 35 ft.

Lacey's silt factor, f = 0.757

Width of the river along E-W Interconnector = 11.0 km

Type of caisson = Circular

Date of observation	Dt (obs.) (ft.)	Q (m3/sec)	Q (ft.3/sec)	q (ft3/sec/ft)	Depth of Water, y (ft.)	Velocity V, (ft./sec)	Dt/b (obs.)
Jan-97	20.54	5299	187001.71	5.18	6.21	0.83	0.59
Feb-97	19.38	3795	133925.55	3.71	4.29	0.87	0.55
mar-97	20.54	5081	179308.49	4.97	5.21	0.95	0.59
Apr-97	21.17	8567	302329.43	8.38	7.38	1.14	0.60
may-97	22.75	25650	905188.50	25.09	15.32	1.64	0.65
Jun-97	25.79	27600	974004.00	27.00	16.09	1.68	0.74
Jul-97	25.59	62150	2193273.50	60.79	27.64	2.20	0.73
Aug-97	24.29	41336	1458747.44	40.43	21.06	1.92	0.69
Sep-97	23.29	35282	1245101.78	34.51	18.95	1.82	0.67
oct-97	22.66	28345	1000295.05	27.72	16.38	1.69	0.65
Nov-97	22.21	13593	479696.97	13.30	10.03	1.33	0.63
Dec-97	20.50	6545	230973.05	6.40	6.16	1.04	0.59
Jan-96	30.96	5106	180190.74	4.99	5.22	0.96	0.88
Feb-96	25.79	3699	130537.71	3.62	4.21	0.86	0.74
mar-96	27.50	5081	179308.49	4.97	5.21	0.95	0.79
Apr-96	29.59	8467	298800.43	8.28	7.32	1.13	0.85
may-96	47.74	23650	834608.50	23.13	14.51	1.59	1.36
Jun-96	54.38	27648	975697.92	27.04	16.11	1.68	1.55
Jul-96	53.26	61154	2158124.66	59.81	27.34	2.19	1.52
Aug-96	51.63	43356	1530033.24	42.41	21.74	1.95	1.48
Sep-96	50.75	36584	1291049.36	35.78	19.41	1.84	1.45
oct-96	49.29	27648	975697.92	27.04	16.11	1.68	1.41
Nov-96	48.54	12694	447971.26	12.42	9.59	1.30	1.39
Dec-96	21.21	7040	248441.60	6.89	6.47	1.06	0.61
Jan-95	4.88	5186	183013.94	5.07	5.28	0.96	0.14
Feb-95	4.85	4321	152488.09	4.23	4.67	0.90	0.14
mar-95	1.88	4447	156934.63	4.35	4.76	0.91	0.05
Apr-95	1.12	5599	197588.71	5.48	5.55	0.99	0.03
may-95	18.88	22730	802141.70	22.23	14.14	1.57	0.54
Jun-95	21.17	41009	1447207.61	40.11	20.95	1.91	0.60
Jul-95	19.46	67344	2376569.76	65.87	29.16	2.26	0.56
Aug-95	67.83	45994	1623128.26	44.99	22.61	1.99	1.94
Sep-95	65.54	30971	1092966.59	30.29	17.37	1.74	1.87
oct-95	48.75	28026	989037.54	27.41	16.25	1.69	1.39
Nov-95	40.09	16589	585425.81	16.23	11.46	1.42	1.15
Dec-95	34.46	7966	281120.14	7.79	7.03	1.11	0.98
Jan-94	15.88	7750	273497.50	7.58	6.90	1.10	0.45
Feb-94	14.71	1230	43406.70	1.20	2.02	0.59	0.42

Table 1.9 (continued)

mar-94	12.88	1650	58228.50	1.61	2.46	0.66	0.37
Apr-94	21.55	1960	69168.40	1.92	2.76	0.69	0.62
may-94	20.83	2850	100576.50	2.79	3.54	0.79	0.60
Jun-94	30.50	34522	1218281.38	33.77	18.68	1.81	0.87
Jul-94	32.71	32147	1134467.63	31.44	17.81	1.77	0.93
Aug-94	34.54	32980	1163864.20	32.26	18.12	1.78	0.99
Sep-94	16.50	26379	930914.91	25.80	15.61	1.65	0.47
oct-94	16.50	20121	710070.09	19.68	13.03	1.51	0.47
Nov-94	16.12	9145	322727.05	8.94	7.70	1.16	0.46
Dec-94	14.71	8320	293612.80	8.14	7.23	1.13	0.42
Jan-93	17.54	6261	220950.69	6.12	5.98	1.02	0.50
Feb-93	12.38	5223	184319.67	5.11	5.30	0.96	0.35
mar-93	12.04	6101	215304.29	5.97	5.88	1.01	0.34
Apr-93	17.05	8588	303070.52	8.40	7.39	1.14	0.49
may-93	18.84	20357	718398.53	19.91	13.13	1.52	0.54
Jun-93	33.42	45200	1595108.00	44.21	22.35	1.98	0.95
Jul-93	32.84	58719	2072193.51	57.43	26.61	2.16	0.94
Aug-93	36.75	51989	1834691.81	50.85	24.54	2.07	1.05
Sep-93	36.71	34664	1223292.56	33.91	18.73	1.81	1.05
oct-93	35.38	27331	964510.99	26.73	15.98	1.67	1.01
Nov-93	32.38	13400	472886.00	13.11	9.94	1.32	0.93
Dec-93	19.00	7996	282178.84	7.82	7.04	1.11	0.54

Table 2 : Scour and related data for the pier of Hardinge Bridge
Width of the pier, b = 37 ft.
Type of pier = Round-nose
Lacey's silt factor, f = 0.757

Date of scour data observation	Average velocity, V (fps)	Approach depth, y (ft.)	Discharge per unit width, q	Observed scour depth from water level, d_t (ft.)	Observed scour depth below bed level, d_s (ft.)	Froude No. Fr	y/b	dt/b	ds/b
24.07.76	5.080	53.000	269.240	66.60	13.60	0.12	1.43	1.80	0.37
10.08.76	5.870	60.500	355.135	74.50	14.00	0.13	1.64	2.01	0.38
02.09.76	8.310	44.500	369.795	71.30	26.80	0.22	1.20	1.93	0.72
21.07.77	6.070	55.000	333.850	61.80	6.80	0.14	1.49	1.67	0.18
04.08.77	6.950	56.500	392.675	70.80	14.30	0.16	1.53	1.91	0.39
25.08.77	6.110	57.500	351.325	66.10	8.60	0.14	1.55	1.79	0.23
02.09.77	6.210	59.000	366.390	79.00	20.00	0.14	1.59	2.14	0.54
09.09.77	6.520	70.500	459.660	82.40	11.90	0.14	1.91	2.23	0.32
01.07.78	3.530	35.500	125.315	64.00	28.50	0.10	0.96	1.73	0.77
09.07.78	3.990	30.000	119.700	63.80	33.80	0.13	0.81	1.72	0.91
19.07.78	4.470	36.000	160.920	69.90	33.90	0.13	0.97	1.89	0.92
19.07.78	4.870	40.500	197.235	69.90	29.40	0.13	1.09	1.89	0.79
02.08.78	5.710	49.000	279.790	79.30	30.30	0.14	1.32	2.14	0.82
30.08.78	7.650	65.500	501.075	87.30	21.80	0.17	1.77	2.36	0.59
06.09.78	6.930	55.000	381.150	73.50	18.50	0.16	1.49	1.99	0.50
13.09.78	8.570	55.000	471.350	82.50	27.50	0.20	1.49	2.23	0.74
20.09.78	7.290	62.000	451.980	75.60	13.60	0.16	1.68	2.04	0.37
05.10.78	4.230	49.000	207.270	73.90	24.90	0.11	1.32	2.00	0.67
12.10.78	3.320	47.500	157.700	80.50	33.00	0.08	1.28	2.18	0.89

Table-3.1 : Laboratory test data of scour depth (Kabir, 1984)

Type of pier : Circular
Width of pier (b) : 0.164 ft.
Width of channel (B) : 2.0 ft.

Run No.	Sediment size, D_{50} (mm)	Discharge Q_{max} (cfs)	Av. approach depth, y (ft.)	Average velocity, V (ft./ sec)	Max. scour depth from W.L. d_t (ft)	d_t/b	y/b	d_t/y
1	0.94	0.1295	0.082	0.7896	0.1595	0.973	0.500	1.95
2	0.94	0.1648	0.0942	0.8747	0.182	1.110	0.574	1.93
3	0.94	0.2001	0.1011	0.9896	0.2001	1.220	0.616	1.98
4	0.94	0.2354	0.1073	1.0969	0.2221	1.354	0.654	2.07
17	0.4	0.1295	0.0984	0.6580	0.1926	1.174	0.600	1.96
18	0.4	0.1648	0.103	0.8000	0.228	1.390	0.628	2.21
19	0.4	0.2001	0.1142	0.8761	0.2592	1.580	0.696	2.27
20	0.4	0.2354	0.1247	0.9439	0.2871	1.751	0.760	2.30
33	0.19	0.1295	0.1155	0.5606	0.2083	1.270	0.704	1.80
34	0.19	0.1648	0.1267	0.6504	0.251	1.530	0.773	1.98
35	0.19	0.2001	0.1362	0.7346	0.2887	1.760	0.830	2.12
36	0.19	0.2354	0.1493	0.7883	0.3199	1.951	0.910	2.14

Table- 3.2 : Laboratory test data of scour depth (Kabir, 1984)

Type of pier : Round-nose
Width of pier (b) : 0.164 ft.
Length of pier (L) : 0.492 ft.
Width of channel (B) : 2.0 ft.

Run No.	Sediment size, D_{50} (mm)	Discharge Q_{max} (cfs)	Av. approach depth, y (ft.)	Average velocity, V (ft./ sec)	Max. scour depth from W.L. d_t (ft)	d_t/b	y/b	d_t/y
1	0.94	0.1295	0.0824	0.7858	0.1469	0.896	0.502	1.78
2	0.94	0.1648	0.0935	0.8813	0.1624	0.990	0.570	1.74
3	0.94	0.2001	0.1017	0.9838	0.187	1.140	0.620	1.84
4	0.94	0.2354	0.1083	1.0868	0.21	1.280	0.660	1.94
17	0.4	0.1295	0.0921	0.7030	0.1854	1.130	0.562	2.01
18	0.4	0.1648	0.1024	0.8047	0.2231	1.360	0.624	2.18
19	0.4	0.2001	0.1125	0.8893	0.2575	1.570	0.686	2.29
20	0.4	0.2354	0.1247	0.9439	0.2831	1.726	0.760	2.27
33	0.19	0.1295	0.1165	0.5558	0.1985	1.210	0.710	1.70
34	0.19	0.1648	0.1266	0.6509	0.2369	1.445	0.772	1.87
35	0.19	0.2001	0.1362	0.7346	0.2759	1.682	0.830	2.03
36	0.19	0.2354	0.1482	0.7942	0.3084	1.880	0.904	2.08

Table- 4 : Laboratory test data of scour depth (RRI)

Depth of Flow, y = 0.10 m
Width of pier, b = 0.102 m
D_{50} = 0.23 mm
Type of pier = Circular

Test No.	Q (m3/sec)	q (m3/sec/m)	Observed depth of scour (m) from WL	Velocity V (m/sec)	Froude no. Fr	dt/b (observed)
1	0.035	0.01	0.103	0.10	0.101	1.010
2	0.053	0.015	0.124	0.15	0.151	1.216
3	0.071	0.02	0.169	0.20	0.202	1.657
4	0.083	0.024	0.189	0.24	0.242	1.853
5	0.106	0.03	0.203	0.30	0.303	1.990
6	0.124	0.035	0.215	0.35	0.353	2.108
7	0.141	0.04	0.214	0.40	0.404	2.098

Table 5 : Scour Data for Groyne (RRI)

Name of model	Type of Groyne	Groyne No.	h0 (m)	hs (obs.) (m)	Q (m3/sec)	B (m)	b (m)	Fr	d50 (mm)	Silt factor (f)	hs/ho	[Q/(B-b)]^2/3	(q^2/f)^1/3	(b/ho)^0.4*Fr^0.33
Kazipur (add. test)	St. spur	S5	3.57	17	6677	1460	222	0.22	0.185	0.757	4.76	3.08	3.11	3.17
Simla (Add. Test)	St. spur	S1	5.21	19	15067	1666	312	0.23	0.18	0.747	3.65	4.98	2.61	3.16
Kazipur	T-head	G2	5.92	23	36000	3000	350	0.28	0.185	0.757	3.89	5.69	2.71	3.36
Simla	T-head	G2	7.5	32	35000	2800	350	0.25	0.18	0.747	4.27	5.89	2.90	2.94
Paka Narayanpur	St. groyne	G1	10.15	21	18000	1600	50	0.28	0.16	0.704	2.07	5.13	1.83	1.24*
Bairagirhat & Chilmari	T-head	G2	6.7	25	35600	4400	490	0.55	0.18	0.747	3.73	4.36	2.65	4.57
Bairagirhat	T-head	G2	5.81	25.89	11806	1850	460	0.143	0.18	0.747	4.46	4.16	2.98	3.02
Chilmari	T-head	G7	4.54	20.14	11295	1850	400	0.202	0.18	0.747	4.44	3.93	2.98	3.54

Table 6.1 : Equilibrium scour (mPWD) at u/s transect for different angle of pier with direction of flow of Paksey bridge pile configuration flume study

Discharge in the model = 384 lps

Depth of flow = 30 m (proto)

Model scale = 1:90

Point no.	dis.fr. L/B (m)	Ini. Bed (m PWD)	Equilibrium scour (mPWD) for different angle of attack				
			0^0 (T2)	5^0(T3)	10^0(T4)	20^0(T5)	30^0 (T6)
1	0.0	-15.00	-20.005	-18.925	-19.375	-23.875	-25.225
2	0.2	-15.00	-20.275	-18.925	-22.075	-23.425	-25.225
3	0.4	-15.00	-22.075	-22.525	-22.075	-24.325	-25.675
4	0.5	-15.00	-21.985	-21.625	-23.875	-24.775	-26.125
5	0.6	-15.00	-25.675	-22.525	-23.875	-27.835	-26.845
6	0.7	-15.00	-24.325	-22.075	-26.485	-26.125	-28.825
7	0.8	-15.00	-25.675	-24.775	-26.125	-27.025	-27.025
8	0.9	-15.00	-23.065	-24.325	-22.975	-26.125	-25.675
9	1.0	-15.00	-22.795	-20.725	-23.875	-23.335	-26.575
10	1.2	-15.00	-23.425	-21.625	-22.525	-23.425	-28.375
11	1.3	-15.00	-24.325	-25.225	-24.325	-26.575	-28.825
12	1.4	-15.00	-27.925	-27.925	-27.925	-27.925	-31.705
13	1.5	-15.00	-25.945	-26.575	-28.105	-27.925	-33.325
14	1.6	-15.00	-27.925	-25.225	-28.375	-31.525	-31.525
15	1.7	-15.00	-23.425	-23.425	-26.125	-27.925	-28.825
16	1.8	-15.00	-23.245	-21.175	-23.875	-26.035	-28.645
17	2.0	-15.00	-22.975	-20.725	-21.625	-26.845	-27.205
18	2.2	-15.00	-20.995	-18.925	-22.525	-25.405	-27.025

Table 6.2 : Equilibrium scour (mPWD) at d/s transect for different angle of pier with direction of flow of Paksey bridge pile configuration flume study

Point no.	dis.fr. L/B (m)	Ini. Bed (m PWD)	Equilibrium scour (mPWD) for different angle of attack				
			0^0	5^0	10^0	20^0	30^0
1	0.0	-15.00	-21.625	-20.725	-20.725	-25.225	-25.225
2	0.2	-15.00	-20.905	-19.825	-21.175	-24.325	-25.225
3	0.4	-15.00	-20.725	-21.175	-21.985	-24.775	-25.945
4	0.5	-15.00	-23.875	-21.625	-22.525	-26.305	-27.025
5	0.6	-15.00	-25.675	-24.325	-25.405	-27.925	-27.745
6	0.7	-15.00	-27.025	-26.125	-27.025	-28.375	-29.275
7	0.8	-15.00	-27.025	-26.845	-27.925	-31.525	-30.625
8	0.9	-15.00	-26.575	-25.675	-26.575	-26.665	-27.925
9	1.0	-15.00	-24.325	-23.875	-22.525	-26.125	-28.105
10	1.2	-15.00	-23.425	-24.595	-25.405	-26.125	-27.745
11	1.3	-15.00	-27.925	-25.675	-23.425	-26.575	-28.825
12	1.4	-15.00	-29.005	-27.925	-29.725	-29.725	-34.675
13	1.5	-15.00	-30.175	-29.725	-30.625	-32.425	-35.125
14	1.6	-15.00	-28.825	-29.275	-30.175	-31.525	-35.575
15	1.7	-15.00	-25.225	-23.875	-25.225	-29.275	-33.325
16	1.8	-15.00	-25.225	-21.625	-25.405	-28.645	-29.005
17	2.0	-15.00	-23.425	-19.825	-23.875	-25.225	-28.375
18	2.2	-15.00	-23.425	-19.825	-22.525	-25.225	-28.825

Table 6.3 : Maximum scour level (mPWD) at individual pile for Pier Group - A with different angle of attack for Paksey bridge flume study

Test No.	Angle of attack	Pile name	Max. scour (mPWD)
2	0^0	A1	-24.775
		A2	-27.025
		A3	-26.575
		A4	-25.405
3	5^0	A1	-24.775
		A2	-26.125
		A3	-25.585
		A4	-25.225
4	10^0	A1	-26.125
		A2	-27.295
		A3	-27.925
		A4	-26.125
5	20^0	A1	-28.375
		A2	-28.825
		A3	-30.715
		A4	-29.275
6	30^0	A1	-29.995
		A2	-30.625
		A3	-31.705
		A4	-29.455

Table 6.4 : Maximum scour level (mPWD) at individual pile for Pier Group - B with different angle of attack for Paksey bridge flume study

Expt. No.	Angle of attack	Pile name	Max. scour (mPWD)
2	0^0	B1	-27.025
		B2	-28.825
		B3	-28.645
		B4	-28.105
3	5^0	B1	-28.105
		B2	-29.005
		B3	-28.015
		B4	-27.745
4	10^0	B1	-28.825
		B2	-31.075
		B3	-29.275
		B4	-28.825
5	20^0	B1	-32.605
		B2	-33.685
		B3	-34.405
		B4	-32.425
6	30^0	B1	-34.675
		B2	-36.925
		B3	-36.475
		B4	-36.025