FINAL PROJECT REPORT

"COMPOSITE FOUNDATION FOR FLOOD PRONE AREAS SUBJECTED TO SCOURING OF FOUNDATIONS''

SUBMITTED TO

University Grants Commission
New Delhi

PREPARED BY

Prof. ShreeRam
Co-Principal Investigator

Prof. Syed Mohd.Ali Jawaid
Principal Investigator

STATION

Department of Civil Engineering
Madan Mohan Malaviya University of Technology,
(Formerly M.M.M. Engg College, Gorakhpur)
GORAKHPUR -273010 (U.P.) INDIA

July 2020

Table of Content

ACKNOWLEDGEMENTS

We wish to express our sincere gratitude to Prof. S. N. Singh, Hon’ble Vice Chancellor, M. M. M. University of Technology, Gorakhpur for his high appreciation, positive attitude and suggestions.

Weare also thankful to my colleagues Dr. R. K. Shukla, Er. Ram Dular, Dr. G. Pandey, Er. S. N. Chaudhary, and Dr. A. K. Mishra for help and suggestions at various level of this research work. Appreciation are due to all members of Geotechnical Engineering laboratory and Hydraulics Engineering laboratory, late Shri Ashok Kumar Yadav, Shri Rajiv Kr Srivastava, Shri Ram Nagina and Shri Raj Kumar Yadav for their friendly help and sincere efforts.

The authors would also like to express their deep gratitude and appreciation to the members of their families whom love, for bearance and understanding provides a source of inspiration.

(ShriRam) (S. M. Ali Jawaid)

CHAPTER-1

INTRODUCTION

1.1 GENERAL

Indo-gangetic plains of India experience large floods regularly and are frequently subjected to flooding damage. The flood in the year 1998 in eastern Uttar Pradesh was the greatest since independence of India. Floods have affected more than 400 villages. The various means of communication (roads, electricity, telephone etc.)had been completely cut off from 23 August 1998 to September 15, 1998. A large number of houses were reported to have been damaged mostly due to scouring of the ground leading to foundation failures (Fig. 1).

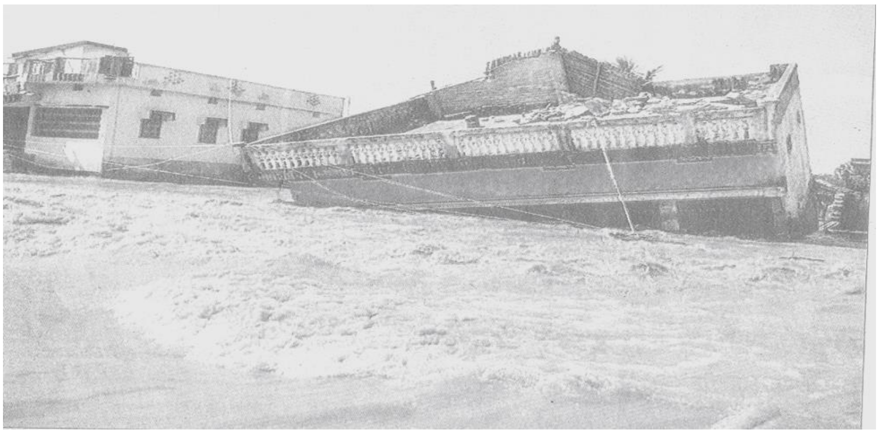


Fig. 1. Foundation failure due to scouring of ground at Begusarai, Distt. Kushinagar
(Courtesy: DainikJagran, Gorakhpur Edition)

Therefore, besides investigating the flood control and flood prevention, in order to protect the life and property of people, it was felt at that time that there is a need to develop suitable, economical and appropriate foundation system and to transfer it to the population living in flood plain areas.

Jawaid and Madhav (2008) had developed short rigid composite foundation for lowlands where shallow foundations are difficult to construct due to the flooding of the foundation pit. It consists of shallow pipes or well steinings (outer diameter, d_0, of 1.0 to 1.5 m, thickness, t, of 10-15 cm and length, L, 1.0-3.0 m) with a granular core inside. The steining is sunk to the desired depth by conventional sinking techniques. Soil within the steining is removed and the granular material filled in and compacted to enhance the stability and load carrying capacity of the foundation.

During the process of scouring, the removal of the materials from the stream bed takes place thanks to the erosive action of the flowing water. On the other hand the scour phenomenon may also be described as the process of lowering the bed level of the channel due to erosive action of the flowing water such that the foundation of the building in flood plain area or hydraulic structure are exposed and the amount of such decrement below an assumed natural level is basically called as the scour depth.

When an alluvial stream is partially obstructed by hydraulic structures such as bridge piers, abutments, shallow/composite foundations in flood plain area etc, scour takes place around the hydraulic structures. This causes formation of a system of vortices which wraps around the composite foundation in the shape of a horse-shoe in plan view and hence generally called as the horse-shoe vortex. Due to such type of the vortex system around the hydraulic structures, a very high level of the local shear stresses on the stream bed or on the channel bed are developed and it causes the high sediment transport capacity around the hydraulic structures such as composite foundation in the flood plain.

The scour also occurs at composite foundation for houses constructed in flood plain area. For the safety of the houses constructed in flood plain area correct prediction of scour depth at composite foundation is essential. The foundation should be taken deeper than the possible maximum scour and should be placed on firm stratum, wherever, there is a possibility of river scouring. The underestimation of scour depth may lead to failure due to design of too shallow foundation and consequent exposure of the foundation endangering the safety of buildings. However, the over-estimation of the scour depth results in uneconomical design of the composite foundation.

In India, Lacey-lnglis method for the prediction of the scour depth is recommended for use by the Indian railways and Indian road congress. However, variable's range and conditions under which this empirical method was devised was narrow and specific than the general case for which it has been recommended. Similar drawbacks can be found to occur in other methods of scour depth prediction.

1.2 TYPES OF SCOUR AND SCOUR PROCESSES

The types of scour that can occur around the composite foundations in flood plain area are generally divided as general scour, contraction scour and local scour.

1.2.1 General scour

General scour can occur either as long-term scour or short term scour. The two types being differentiated by the time taken for the development of the scour.

Short-term general scour: Short-term general scour is type of scour which is caused due to a single or various closely spaced floods.

Long-term general scour: Long-term scour is a type of scour which is generally caused at a longer time scale of considerably the order of the several years. This type of scour is generally may not be important during the design life of a hydraulic structure in the flood plain area, if the rate of the development of the scour is relatively slow. Long-term general scour can be caused by natural changes in the catchment or due to the activities of the man.

Human causes: 1. Channel alterations

-dredging

-channelization

-straightening

-cut-off formation

2. stream-bed mining

3. dam/reservoir construction

4. land use changes

-urbanisation

-deforestation

-agricultural activities

Natural causes: -cut-off formation

-land slide, mud flows

-liquefaction

-climate changes

1.2.2 Contraction scour

Contraction scour at the hydraulic structure is generally caused due to the constriction of the flow around the hydraulic structures.

1.2.3 Local Scour

Local scour is a type of scour which is caused generally due to interference of the composite foundation in flood plain area with the approaching flow. Generally the local scour around the composite foundation in the flood plain area is characterised by the formation of the scour holes immediately around the composite foundation.

Generally, the combined form of the contraction scour and the local scour is termed as Localised scour and the localised scour is of two types such as clear-water scour and live-bed scour.

Clear-water scour: clear-water scour is a type of scour which occurs when the bed materials are at rest along the direction of upstream flow means in such types of flow there is no any movement of the bed materials. And maximum depth of the local scour is achieved when the flow is not able to remove the bed materials from the scour area.

Live-bed scour: live-bed scour is a type of scour which generally occurs due to the movement of the bed materials from the scour area.

1.3 Objective of the Present research

From the above discussions it is evident that the any real treatment of any type of scour problem requires analytical understanding of its basic mechanism and its causes. Incomplete knowledge of the data of two-dimensional flow field and three-dimensional flow field around the composite foundation will hinders the analysis of the flow field of the problem. The present investigation has been taken up by keeping in view the above gaps in the knowledge.

The objective of this study is to develop a semi-empirical method to determine the time variation of the clear-water scour depth at the composite foundation and thus to develop the design formula for the same. The main objective of this study is to develop experimentally the relation between the variation of the scour depth with time for the clear-water condition.

CHAPTER-2

LITERATURE REVIEW

2.1. INTRODUCTION

2.2.1. Composite Foundation for Flood Prone Areas

Jawaid (2006) had proposed a suitable and economic foundation for alluvial lowlands christened it as "Composite rigid caisson with granular core (abbreviated as Composite Foundation). It consists of shallow pipes or well steinings (outer diameter, d_0, of 1.0 to 1.5 m, thickness, t, of 10-15 cm and length, L, 1.0-3.0 m) with a granular core inside. The steining is sunk to the desired depth by conventional sinking techniques. Soil within the steining is removed and the granular material filled in and compacted to enhance the stability and load carrying capacity of the foundation (Fig. 2.1). The moduli, Poisson's ratios and angles of shearing resistance of soil and granular core are E_s, ν_s, ϕ and E_{gp}, ν_{gp}, ϕ_{gp} respectively. A vertical load, Q, is applied at the top of the proposed foundation.

The proposed composite rigid caisson with granular core functions similar to a short pipe pile except that the granular infill is much stronger and stiffer than the original ground. Hence, it can carry and transfer part of the applied load. The steining is relatively incompressible and hence settles more than the granular core. Therefore, the outer and inner surfaces of the pipe or caisson resist the applied load by positive resistance, the granular infill would be subjected to down-drag or negative skin resistance because of which larger loads are transferred through its base. Granular material, if confined, deforms one- dimensionally with stiffness increasing with confining stress.

The analysis of the proposed foundation - soil interaction is carried andreported elsewhere (Jawaid and Madhav, 2008).

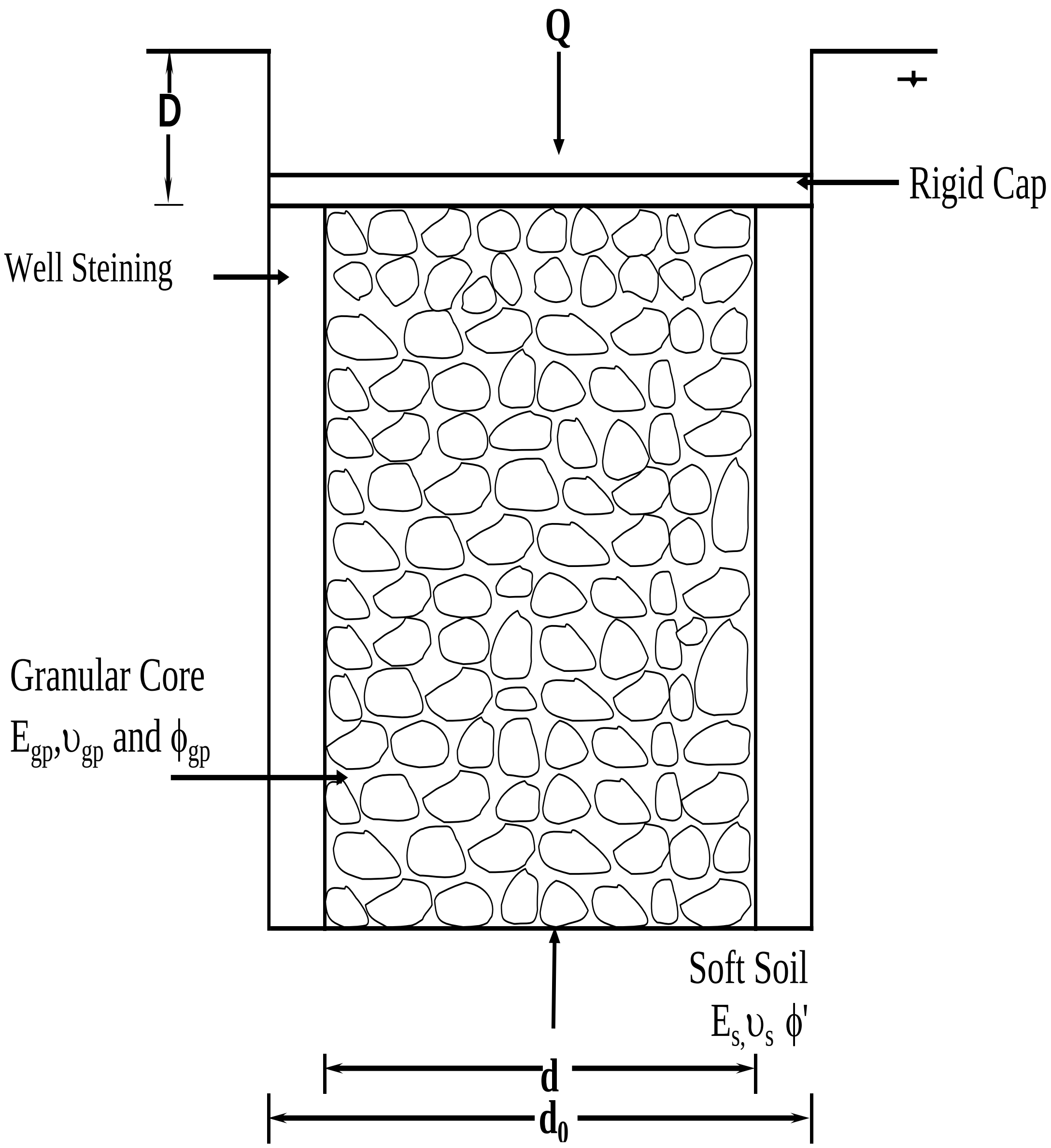


Fig. 2.1. Composite Foundation

2.1.2. Scouring of Foundation

A huge amount of the literature is available related to the problem of scour around the foundation and very few data are available for composite/composite foundations and methods of protection. Although the different aspects of the local scour at bridge foundation/bridge pier have been studied from time to time but a large difference exist between actual scour depth measured in field and the depth of local scour calculated from the different relationships currently in use [Dargahi (1982) and Dey (1997), Melville (1975), Breusers et al. (1977), RDSO (1987,1992)]. These all the field studies which have been given above emphasises the requirement for good understanding of local scour phenomenon, which will lead to more rational methods for the estimation of the depth of scour at the composite foundation in the flood plain area.

The mechanism of scouring around a composite foundation is associated with a three-dimensional flow separation to upstream of the composite foundation and the periodical vortex shedding in the region of the downstream and the phenomenon related to the problem of the local scouring have been studied for more than the eight decades. For the first time in the history, Keutner (1932) was the first researcher who investigate that the vortex systems are the basic cause for the mechanism of the local scouring. Since many of the experimental studies also exist related to the estimation of local scouring. However, in most of these studies the emphasis has been made on formulation of scour depth equations, rather than the modelling of the problem of the local scouring. Many important experiments related to the investigation of the local scouring have been given by Dargahi (1982), and Dey (1997), Breusers et al. (1977).

Local scour is very complex in nature due to its large variation in field data from one place to another place because flow characteristics and channel geometry and sediment characteristic and other parameter describing the scour depth change place to place and have their own individual effect over the scour depth from one site to the other site.

In this chapter, it has been tried to give the brief review of the previous works of different investigators, experimental as well as theoretical, which have connected time to time to the different parameters used for the prediction of the depth of scour at different locations.

2.2. CLASSIFICATION AND MECHANISM OF SCOUR

The process of the local scour around the composite foundation in flood plain area involves the complexities of both three-dimensional flow and the sediment transport. The main characteristic of the flow around composite foundation is vortex system which are featured as the two types as horse-shoe vortex and wake vortex. On the behalf of the experimental and theoretical studies, the formation of the horse-shoe vortex and scouring mechanism can be described in the following lines.

2.2.1 Classification of Scour around piles

The types of scour around the composite foundation in flood plain area can be divided as follow:

(i) General Scour: General scour is a scour which can be either long-term scour or short-term scour.T he two types can be distinguished by the time taken for the development of the scour.

(ii) Contraction Scour: The scour which may occur because of the contraction of the waterway by the composite foundation.

(iii) Local Scour: Local scour is the type of scour which is caused by the interference of the piles with the approaching flow and is generally characterised by the formation of scour hole around the composite foundation.

Thus the total scour is the combined effect of the general scour, constriction scour and the local scour.

2.2.2 Mechanism of scour around pile

The flow approaching to the composite foundation becomes zero at the upstream face of the composite foundation, in the plane having the vertical symmetry, and due to this the approaching flow velocity decreases from the free surface to the downward and becomes zero at the bed level and thus the decrement in the stagnation pressure occurs at the face of the composite foundation. And this downward pressure at the face of the composite foundation causes the down-flow.

The stagnation pressure causing the down-flow and sideward acceleration of the flow past the pile. The flow separation at side of the composite foundation generates cast-off vortices with wake at the main stream interface. These cast-off vortices are approaching downstream with the flow and interact with the horse-shoe vortex at the stream bed causing it’s oscillation to the horizontal and the vertical direction at the face of the composite foundation. And thus cast-off vortices acts as the little tornado’s which lifts the sediments from the stream bed causing the scouring around the composite foundation.

According to Melville (1975), local scour holes at the pile or piers are developed due to the combined effect of down-flow and horse-shoe vortex. Kwan et al. (1994) have also shown that the primary vortex is the principle scouring agent.

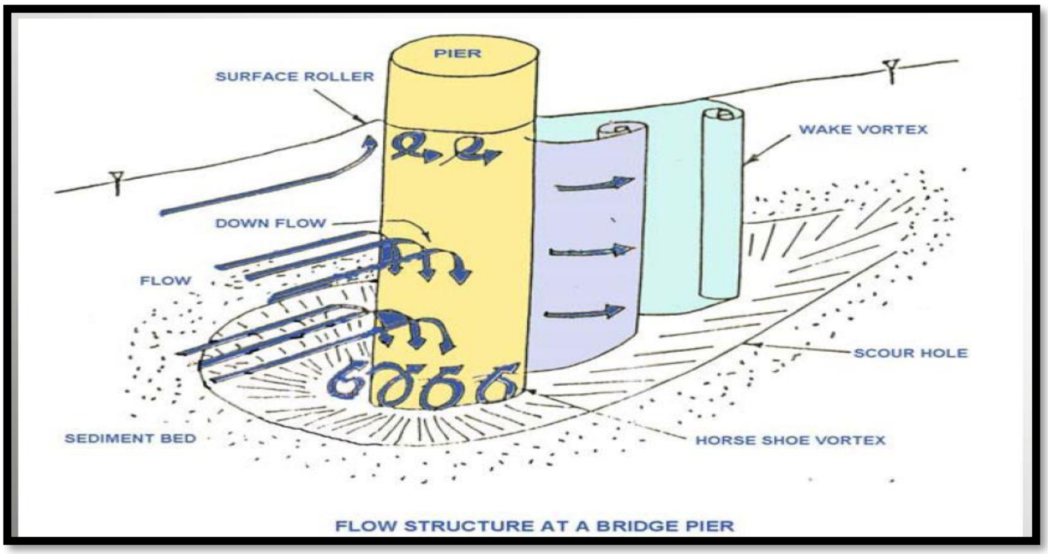


Figure 2.1 Components causing the scouring

2.3. FLOW CHARACTERISTICS OF CLEAR-WATER HORSE-SHOE VORTEX

About more than last three decades, many researchers have investigated about the formation of the horse-shoe vortex and their characteristics. On the other hand Dargahi (1990) said that, the previous studies about the formation of the formation of the horse-shoe vortex and their characteristics provide a very little information about the flow field that is related to the scour mechanism.

Earle investigation about the horse-shoe vortex and their physical and theoretical aspects have been given by Squire and winter (1951), Hawthorne (1964) and Taylor (1967). Firstly, Shen et al. (1969) concluded that the circulation of the vortex is proportional to pier or pile

Reynolds number, IRp, and the observation of horse-shoe vortex before the commencement of the scouring have been reported by many investigators, including Schwind (1962), Roper (1967), Kikkawan (1971), Nakagawa (1974), Yano (1975), Baker (1979) and Dargahi (1989). It has helped in understanding the subject in depth but individually the approach and objective of the study differed. So in this literature review based on the experimental result, the term which have been discussed are as position of the horse-shoe vortex, number of horse-shoe vortices etc.

(a). Position of Horse-shoe vortex

The investigation related to the position of the horse-shoe vortex have been given by many researchers but in this literature review two of them have been discussed. Baker (1979, 1980b) and Kubo and Takezawa (1988) have investigated about the position of the horse-shoe vortex are discussed here.

Baker (1979, 1980b): Baker investigated into the laminar and the turbulent horse-shoe vortex system and determined the position of the horse-shoe vortex from the pressure measurement for each case at the plane of symmetry upstream of the circular pier.

Kubo and Takezawa (1988): He showed that the core of the vortex is highly unstable but the mean value of the core of the vortex variation range is approximately given as 0.15D. The author assumed that 'the vortex core variation is caused by the interactions of the horse-shoe vortex, the wake vortex and the bursting phenomenon peculiar to a turbulent flow interaction with one another.

(b). Number of Horse-shoe Vortices

There are some differences on the opinion that whether a single or a multiple number of vortices are formed. Certain investigation given by the researchers such as Baker (1979, 1980b), Sharma (1988) and Dargahi (1989).

Baker (1979, 1980b): Baker observed that even number of the vortex system existed showing a regular oscillating motion and an irregular unsteady behaviour. In the turbulent vortex system the number of vortices was 4 while 2, 4 or 6 vortices were reported in the laminar case which may increased in number with an increase in pier Reynolds number, IR_p.

Sharma (1988): According to the investigation of the Sharma, the number of the vortices is dependent on Reynolds number and the leading edge geometry and size of the obstruction.

Dargahi (1989): On the basis of his investigation around the circular pier, Dargahi reported a multiple vortex system containing four main vortices and one corner vortex. And also it was found that the increase in the Reynolds number resulted in an increase in the number of the vortices.

(c). Vortex shape

There are few investigators who have reported the shape of the vortex system. These investigators are as Kubo and Takezawa (1988), Devenport and Simpson (1990), and Muzzammil (1992).

Kubo and Takezawa (1988): On the basis of the photographs, it was reported that the shape of the vortex is an elliptical one with the major axis in the direction of the flow.

Devenport and Simpson (1990): Also stated the elliptical shape of the horse-shoe vortex.

Muzzammil (1992): He presented a series of the photographs related to the horse-shoe vortex formation using the technique of a mud flow visualisation. He investigated that the shape of the vortex system is dependent on the pier Reynolds number and reported that it is circular at the low Reynolds number and becomes more and more elliptical with the increase in the Reynolds number.

(d). Vortex Size

The two investigations are reported here expressing the detail about the size of the vortex.

Qadar (1981): He found that the vortex size depends on D (pier diameter) and noted as D_V.

Dargahi (1989): He found that the vortex size is independent of the Reynolds number and dependent upon the pile or pier diameter (D).

(e). Vortex Velocity

Qadar (1981): He reported that the tangential velocity of the vortex system,U_θ is a function of approach velocity (U) and the size of the obstruction (i.e. Diameter of the pile D).

Devenport and Simpson (1990): Investigated the behaviour of the turbulent boundary layer near a wings mounted normal to a flat plate using laser anemometer. They reported that the tangential velocity of the vortex was half of the free stream velocity.

Kubo and Takezawa (1988): He conducted the measurement of the velocity of the vortex at a height of 0.01D above the bed and gave a range of U_θ/U as: 0.3≤ U_θ/U ≤0.55. further, the analysis for approach velocity U was made by studying circumferential velocity values from a couple of photographs. The final expression for the vortex velocity (U_θ) which depends over the diameter of the obstruction D, mean velocity of the main flow U, mean depth, H of the approach flow is given by;

$$U_\theta = 0.57\mathrm{U}[0.217D/(H-0.033D)]^{1/6} \text{ ; for } IR_e > 10^5$$

And the author suggested a value $U_\theta/U \cong 0.5$ for the practical application in the Reynolds number range of greater than 10^5.

Muzzammil (1992): He counted the frequency of the rotation of the vortex by using the vortex probe and then used it to find the vortex tangential velocity, $U_\theta=\pi N D_V$, where N is the number of rotations of the vortex per second and D_V is the size of the vortex and is given by the average of sum, of the two axes of the ellipse formed by vortex rotation.

(f). Vortex strength

Vortex strength is an important parameter. And the Qadar (1981) expressed it as “the maximum scour depth is functionally related to the vortex strength which develops ahead of the pile or pier.”

Melville (1975): According to the Melville, the vortex strength would be a function of the pile diameter and the mean velocity of the approach flow. However, the relation of vortex strength with U and D was not investigated by the author.

Muzzammil (1992): He calculated the vortex strength,Γas $\pi D_V\, U_\theta$. And a non-dimensional parameter of $\Gamma/\pi UD$ alongwith data from the other researchers was analysed. All the showed are better applied of $\Gamma/\pi UD$for $IR_P \geq 10^4$.

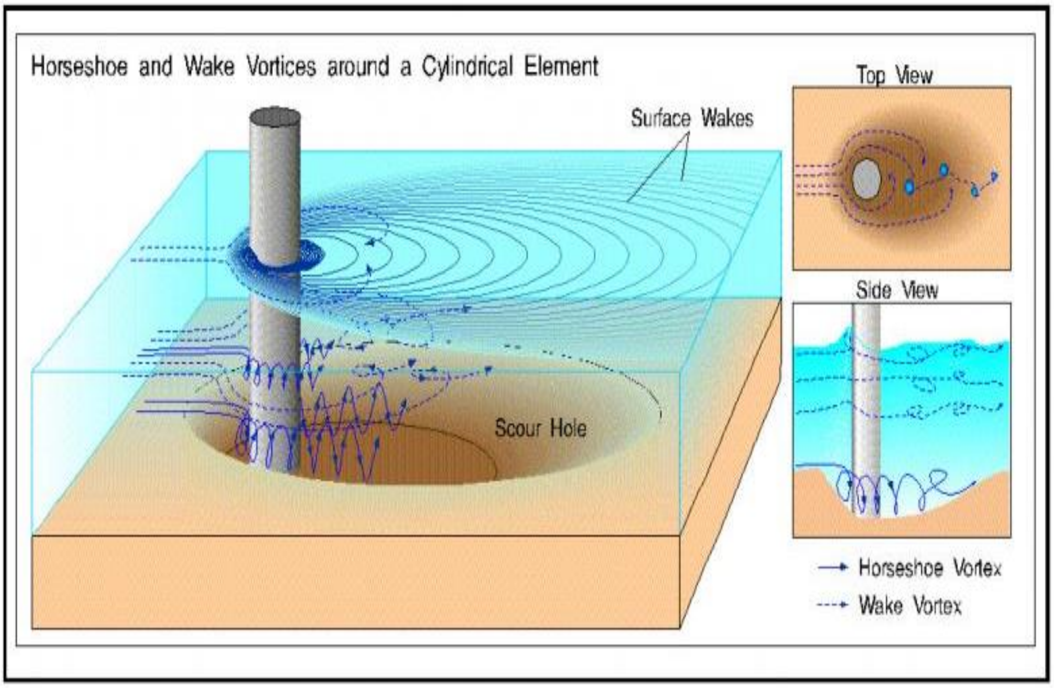


Figure 2.2 Horse-Shoe Vortex and wake vortex

2.5. FORMATION OF THE SCOUR HOLE AROUND THE PILES

Scour hole around the piles is that area from where the removal of the sediment from the stream bed takes place. Formation of scour hole is generally caused by the interference of the upstream face of the pile with upstream flow upward or downward.The downward component causing the erosion of sediment from the stream bed at the toe of the composite foundation. Scour holes formed at the base of the composite foundation has dimensions governed by the type of structure face, nature of the attacking wave, and resistance of the bed sediments. Extreme scouring will cause undermining of the structure that may cause structural failure. Prevention of scouring and thus scour hole development requires protection to the seaward toe of the structure utilizing armour stone of adequate size to prevent displacement.

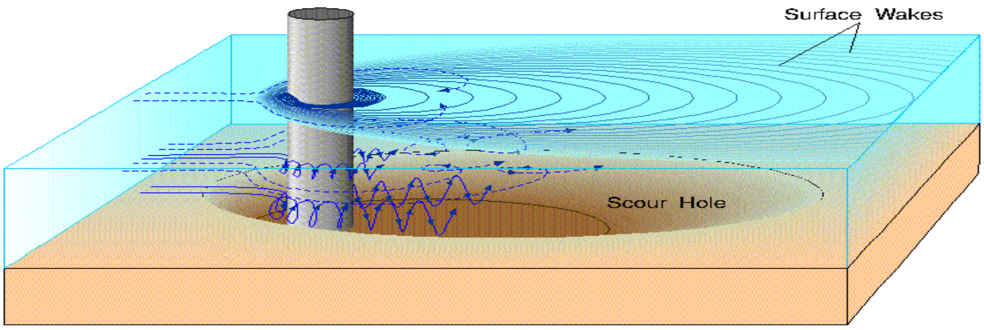


Figure 2.3 Scour hole around the single pile

2.5 NECESSITY OF SCOUR DEPTH ESTIMATION

Before the construction of any hydraulic structure, the estimation of the scour depth for the purpose of the design is very-very crucial part. So before the use of any mathematical model for the prediction of the scour depth at composite foundation in flood plain area, it is very essential that the model must be proved by means of the actual prototype scour observations under the same conditions of the flow and the other parameters used for the prediction of the scour depth. In the past, the various methods have been given by different investigators for the prediction of the scour depth. So from safety and economy point of view it is very necessary to design the scour depth very accurately for the hydraulic structures. Any damage in the hydraulic structures may lead to the bigger looses and it needs a large expense over its maintenance.

2.6 FACTORS AFFECTING THE SCOUR DEPTH AROUND THE COMPOSITE FOUNDATION

The factors affecting the scour depth are very crucial to study. The factors which affect the scour depth at the composite foundation at the bridge site are as: The geomorphic factor, flood-flow transport, bed sediment and bridge-geometric factors that may affect the scour at the bridge foundations. A brief discussion of some of these factors is discussed here.

2.6.1 Geomorphic Factors

Geomorphic factors affecting the scour at composite foundation in flood plain area can be divided into two types such as catchment characteristics and river characteristics. Generally these effects are more important for the general scour rather than for the localised scour. The catchment characteristics of the geomorphic factor consist of the climate factors and the topographic, vegetative and characteristics of the soil of the catchment. These factors are of the primary importance because they determine the water and sediment transport rates at the composite foundation in flood plain area. Also, the capacity for the accumulation of debris at the composite foundations and inflaming any scour depends on the type and the quantity of catchment vegetation.

2.6.2 Flood Flow Transport Factors

Water transport, sediment transport and debris transport are related to the transport factors affecting the scour at the composite foundation in flood plain areas. General scour and localised scour can be easily determined by using these factors. Condition of the live-bed scour or clear-water scour can be determined easily by considering the sediment transport factors. For the case of the live-bed conditions, the sediment transport rate and bed-form characteristics are important factors. Scour at composite foundation in flood plain area can be significantly increased by the accumulation of floating debris at composite foundation.

2.6.3 Bed Sediment Factors

Bed sediment factors at composite foundation includes the particle size distribution and, for the cohesion-less sediments, the spatial distribution of the size of the sediments in areally and vertically. Scour at the composite foundation constructed on cohesive soils is a very complex phenomenon. The state of scour of very fine grained soils cannot be check on the basis of grain-size characteristics because of its compound physio-chemical interaction between the colloidal particles of small size, and the pore water pressure effect, and the preloading effect.

2.6.4 Bridge Geometric Factors

For the estimation of the localised scour at the composite foundation in flood plain area, the foundation and channel geometry are very important factors. They includes the degree of contraction of the channel caused by the restriction of the flow caused by the channel in the flood plain area, the geometry of the composite foundation, the orientation of the composite foundation in relation to the bends of the channel, and the presence of scour protection and scour reduction works. Flow contraction can be horizontal due to encroachment of the channel approaches and vertical due to the submergence of the superstructures. The geometry of the composite foundation can be explained by type, shape, length, width and orientation with the flow of individual composite foundation in flood plain area.

Figure 2.4 Bank erosion just downstream (Geomorphic Factors affecting the scour depth)

Figure 2.5 Floating debris accumulation at the bridge pies (Flood flow transport factors)

Figure 2.6 Bed sediment factors (causing bridge failure)

2.7 SCOUR PROTECTION MEASURES

Scour Protection measures are also termed as scour control measures. And it can be classified on the basis of the type of the scour they are going to control. The arrangements such as retards, riprap such as willow planting, spurs and dikes etc are the control measures for the lateral bank erosion.

For the safe and economical design, scour around the composite foundation is required to controlled and reduced. The performance of any device used to countermeasure the scour depth depends upon the process that how the device is resisting the phenomenon of the scouring around the hydraulic structure. To reduce the scour around the composite foundation various efforts have been made such as installation of the riprap around the pile and by providing an array of the piles in front of the pier and a arrangement of the collar around the piles or piers. Some time the submerged vanes and a delta-wing-like fin in front of the pier are use to reduce the scour depth.

2.8 GENERAL REMARKS AND CONCLUSIONS

2.8.1 General Remarks

Structure induced formation of the horse-shoe vortex has been studied by many investigators over the rigid bed as well as mobile-bed but they fail to give the position, number, size and shape, velocity and strength of the scouring vortex. There is a difference of opinion regarding the number of horse-shoe vertices. But majority view is for multiple vortex system. Thus the characteristics of the horse-shoe vortex as its size, velocity and strength which are directly related to scouring are still not in the established state.

Existing modelling or analytical solutions to the problem of local scouring are not satisfactory, since due to the over simplification made, the physics of the problem is poorly represented. Barring the empirical formulae, the main target of the existing models is the estimation of the equilibrium scour depth in terms of the approach flow velocity, flow depth and pier width, by modelling the horse-shoe vortex. But no attempts are made to get the exact information regarding the flow field inside the scour hole for modelling it around the pier in equilibrium scour hole. Without any doubt, Melville and Raudkivi (1977) have taken the measurements of velocities inside the scour hole for describing the flow characteristics, required attention is yet to be paid for representing the flow structure in mathematical expressions but studying the horse-shoe vortex characteristics. A mathematical solution of equations of fluid motion despite of its complexity is a more rational approach for representing the flow structure. Because of the complexity of the problem there seems to be a great deal of controversy in the literature regarding the basic non-dimensional parameters affecting local-scour around the composite foundation in the flood plain area.

There are also some different views regarding the effect of sediment size on scour. Inglis has concluded from laboratory studies that gradation of the sediment material is a factor affecting local scour. Ahmad (1953) found that only the rate of scour is affected by variation of sediment size but not the maximum scour. However, he concluded that large variation in sediment size may lead to changes in the maximum scour depth. Shen et al. (1966, 1969) analysed scour data in sand range excluding the effect of size, Laursen (1963) considered that the size of the bed sediment does not affect the scour when the channel is transporting sediment. As against this, studies at Roorkee shows a significant effect of sediment size. Sarma’s (1969) investigation also reveals a considerable influence of sediment size on scour. Shen et al. and Laursen consider the width dimension to be important and not the opening ratio. According to Laursen (1962), the degree of contraction does not affect the scour as long as the scour holes of two piers do not overlap. Omission of the opening ratio may be justified only for large opening ratios. However, when the obstruction is significant, the opening ratio becomes important. Ahmad, Liu et al., Grade and Kothyari (1990) included the opening ratio as a significant parameter in their scour relations.

From the above literature review it is clear that the total scour depth at the composite foundation is the summation of degradation, contraction and local scour. But most of the investigators have studied only one type of the scour at one time. So the prediction of total scour becomes difficult. Although the above scour are based on the different scouring mechanism.

For the local scour depth, different equations have been given for clear-water and live-bed scour and for uniformly and non-uniformly graded sediments. Although single equation can be modelled.

2.8.2 Conclusions

The following broadconclusions are made on the basis of the above literature review.

Local scour around the compsite foundation/pile is very complex phenomenon resulting from the strong interaction of the three-dimensional turbulent flow field around the pile and the erodible sediment which results in the formation of primary horse-shoe vortex.

The shape, size, number and strength of the primary vortex is still not a settled issue and the system of vortices consist of primary vortex, bow vortex and wake vortex.

There is a strong justification for assuming for the horse-shoe vortex to be mainly responsible for the scour around bridge piles.

CHAPTER-3

EQUIPMENT AND EXPERIMENTAL PROCEDURE

3.1 INTRODUCTION

After going through the literature review, it is concluded that there are many methods given by the various investigators regarding depth of scour in clear-water scour and live-bed scour. These reviewed methods performed by various investigators have been done in different conditions by different investigators. In the present work, an experimental study of clear-water scour depth have been done by using the locally available sand and laboratory flume.

Keeping these in view, experiment was planned and conducted in Hydraulic and water resource laboratory of the Civil Engineering Department at Madan Mohan Malaviya University of Technology, Gorakhpur for the collection of the data related to the above mentioned aspect of the clear-water scour depth at the composite foundation.

In this chapter, experimental setup and procedure has been discussed in the following paragraphs.

3.2 DETAILS OF EXPERIMENTAL SETUP

3.2.1 Flume

The flume was a non-recirculating glass walled flume, having the length of 4.2 meters and width of 1.5 meters and depth of 0.405 meters. The fine sand bed was filled 0.152 m deep and packed with gravel curtain at the upstream face to prevent the sand getting eroded at the inlet. A sediment trap arrangement was provided at the upstream and downstream faces of the channel.

The flume gets its water supply continuously throughout the experiment from the reservoir inbuilt with the flume situated below the channel section of the flume. Water was supplied in to the channel through a valve on the upstream side. After the water level builds up in the channel section, water level is controlled by gates provided at the inlet and outlet of the channel. In order to maintain the uniform flow depth and discharge, the downstream gate was operated suitably. The flow depth in the flume channel was measured by using a pointer gauge with a flat sand bottom. The gauge could be moved over adjustable rails mounted onthe walls of the flume. The rail was kept parallel to the bed of the flume. The depth of the flow in the flume channel could be adjusted by operating the tail gate located at the downstream end of the flume to obtain a uniform depth of flow. The cannel was operated with zero longitudinal slope. The general section of the experimental flume is shown in the following Fig-3.1 (a) and Fig-3.1 (b).

Fig. 3.1 (a): Longitudinal section of the experimental flume

Fig. 3.1 (b): Width of the experimental flume

3.3 SOURCE OF SEDIMENT AND ITS CHARACTERISTICS

The sediment used was non-cohesive natural sand of relative density 2.65. The flume was filled with fine sediment having uniform grain size distribution. The sediment has been collected from the Rapti river near Gorakhpur city. The used sediment has mean size as $d_{50} = 0.2mm$. which has been used for the calculation. For the sieve analysis 10kg of the sediment has taken and by using the following relation the sieve analysis graph is obtained.

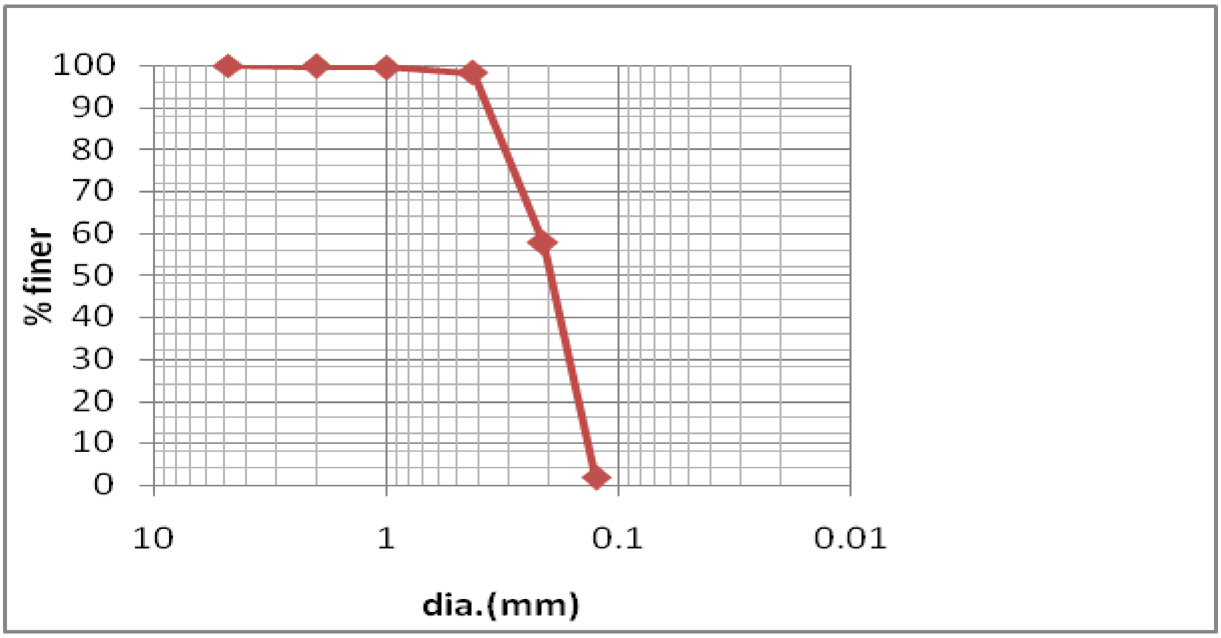
1. $Percentage\ retained\ on\ any\ sieve\ number = \frac{weight\ of\ soil\ retained}{total\ soil\ weight} \times 100\%$

2. $Cumulative\ percentage\ retained\ on\ any\ sieve\ number = \Sigma\ percentage\ retained$

3. $Percentage\ Finer\ than\ any\ sieve\ size = 100\% - \Sigma\ percentage\ retained$

Table 3.1 Data For Sieve Analysis of Fine Sand

Sieve no.	Dia. of each sieve (mm)	Mass of soil retained on each sieve (gm)	%age retained on each sieve	Cumulative %age retained	%age finer
10	4.75	0	0	0	100
16	2	9	0.09	0.09	99.91
30	1	35	0.36	0.45	99.55
40	0.425	136	1.36	1.81	98.19
60	0.212	4013	40.38	42.19	57.81
100	0.125	5579	56.14	98.33	1.67
200	0.75	165	1.66	99.99	0
		Sum=9937 gm			



Graph: 3.1- Sieve Analysis

3.4 DIMENSION OF THE MODEL

For the calculation of the scour depth, a composite foundation model was prepared and a scale ratio was fixed for horizontal and the vertical dimensions.

3.4.1 Size and Scale Ratio

A Composite foundation (similar to pile) model for the estimation of the depth of scour for the clear-water condition was used. The composite foundation model was having the dimension such as outer diameter =51.2 cm and length=71.2 cm. The iron pipe having the diameter of 2.8 cm was used for making the Composite foundation (pile) of length 15.24cm. The diameter of the pile was considered to be 2.8cm. The Composite foundation model was having 12 piles of equal diameter and length and every pile was welded with the metal sheet of 20 gauges to form the Composite foundation model.

The scale ratio which was used for the preparation of the Composite foundation model for setting the horizontal and vertical dimensions was such as:

$$\text{Composite foundation } spacing = 24.2\ cm,\ \ Horizontal\ scale\ ratio = 1{:}\,10, Vertical\ scale\ ratio = 1{:}\,50\ \ L/B\ Scale\ ratio = 1{:}\,1.75$$

Fig.3.2 Experimental Composite foundation model

3.5 STATE OF EXPERIMENT

1. Experiment was performed in a rectangular glass walled channel;
2. The flow was considered to be uniform throughout the experiment;
3. The depth of flow was maintained constant throughout the duration of the experiment;
4. The interference of the walls of the channel being wide and was neglected; and
5. The slope of the channel bed was very small and hence considered to be horizontal.

3.6 MEASUREMENT OF DATA

3.6.1 Discharge Measurement (Q)

The following steps have been considered for the calculation of the discharge (Q) such as:

1. The flow pipe for carrying the discharge in the flume is considered as the orifice meter. An **orifice meter** is a device used for the measurement of the rate of flow (discharge) Q of thewater flowing through the pipe. It consists of a flat circular plate with circular sharp edge hole called orifice, which is concentric with the pipe.

The diameter of the orifice was kept generally 0.5times the pipe diameter and it may vary between 0.4 to 0.8 times the diameter of the pipe.

Two differential manometers have been used for the measurement of the pressure of the fluid passing through the pipe. One was connected at the section (1), which was at a distance of 1.5 to 2.0times the diameter of the pipe upstream from the orifice plate, and the second wasconnected at the section (2) which was at a distance of about half the diameter of the orifice on the downstream side from the orifice plate.

2. After measuring the pressure difference at the two sections (1) and (2), applying ***the Bernoulli's equation*** at sections (1) and (2) at the upstream and the downstream.

$$\frac{P_1}{\rho g} + \frac{{V_1}^2}{2g} + Z_1 = \frac{P_2}{\rho g} + \frac{{V_2}^2}{2g} + Z_2 \qquad (3.1)$$

$$P_1 = upstream\ pressure\ and\ P_2 = downstream\ pressure \quad \left[\frac{P_1}{\rho g} + Z_1\right] - \left[\frac{P_2}{\rho g} + Z_2\right] = \frac{{V_2}^2}{2g} - \frac{{\mathrm{V}_1}^2}{2\mathrm{g}} \qquad (3.2)$$

$$\left[\frac{P_1}{\rho g}+Z_1\right]-\left[\frac{P_2}{\rho g}+Z_2\right]=h=differential\ head \tag{3.3}$$

After solving the above equation, the formula for calculating the rate of flow (discharge) Q is obtained as:

$$\boldsymbol{Q}=\frac{\boldsymbol{a_o c_c\sqrt{2gh}}}{\sqrt{\boldsymbol{1-\left(\frac{a_o}{a_1}\right)^2 C_c{}^2}}} \tag{3.4}$$

Again the above equation can be simplified as:

$$\boldsymbol{Q}=\frac{\boldsymbol{c_d a_o a_1\sqrt{2gh}}}{\sqrt{\boldsymbol{a_1{}^2-a_o{}^2}}} \tag{3.5}$$

Where;

$a_o = area\ of\ the\ orifice \cong {}^{\pi}/_{4}\,(d_o)^2$

$d_o = diameter\ of\ the\ orifice\ \cong [0.4d_1\ to\ 0.8d_1]$

$a_1 = area\ of\ section\ (1) of\ the\ pipe \cong {}^{\pi}/_{4}\,(d_1)^2$

$d_1 = diameter\ of\ the\ pipe\ of\ the\ flume$

$h = differential\ head\ or\ pressure\ head$

$c_d = co-efficient\ of\ discharge\ for\ orifice\ meter \cong 0.64\ to\ 0.76$

$\rho = density\ of\ the\ water = 1000\ kg/m^3.$

$g = acceleration\ due\ to\ gravity = 9.81$

The calculated discharge (Q) for each run is given in table number 3.3.

3.6.2 Approaching Flow Velocity Measurement (U)

Then after calculating the value of discharge (Q), the approaching flow velocity (U) of the fluid entering in to the channel section of the flume is calculated by the following relation as:

$$Q = A \times U \tag{3.6}$$

$A = area\ of\ the\ channel\ section\ of\ the\ flume$

$$\text{Then,} \quad U = Q/A \tag{3.7}$$

Where;

$U = Approaching\ flow\ velocity$

$Q = discharge\ entering\ into\ the\ channel\ section\ of\ the\ flume$

$A = area\ of\ the\ channel\ section\ of\ the\ flume\ and\ A = B \times h$

$B = width\ of\ the\ channel\ section\ of\ the\ flume$

$and \quad h = Average\ depth\ of\ the\ flow\ into\ the\ channel\ section\ of\ the\ flume$

The calculated approaching flow velocity (U) for each run is given in the table (3.4).

Table -3.2: Area Calculation

Run no.	Orifice dia. $(d_o)cm$	Pipe dia.$(d_1)cm$	$a_o = {}^{\pi}/_{4}\,(d_o)^2 cm^2$	$a_1 = {}^{\pi}/_{4}\,(d_1)^2 cm^2$
1	$0.4 \times 10 = 4$	10	12.56	78.5
2	$0.5 \times 10 = 5$	10	19.625	78.5
3	$0.6 \times 10 = 6$	10	28.26	78.5
4	$0.6 \times 10 = 6$	10	28.26	78.5
5	$0.7 \times 10 = 7$	10	38.465	78.5
6	$0.8 \times 10 = 8$	10	50.24	78.5

Table -3.3: Pressure head, discharge (Q) for the water entering into the channel section of the flume

Run no.	Run time (hrs)	Upstream pressure= $p_1(kg/cm^2)$	Downstream pressure=P_2 (kg/cm^2)	Pressure head=h(cm)	$discharge$ $Q(cm^3/s)$
1	10	2.4	2.2	20	384.023
2	10	2.5	2.1	40	274.01
3	10	2.6	2.4	22	158.62
4	10	2.3	2.0	30	534.76
5	10	2.2	2.0	10	578.23
6	10	2.1	2.0	11	653.15

Table-3.4: Approaching flow velocity (U) into the channel section of the flume

Run no.	Time of run (10hrs)	B=Width of the channel section (cm)	h=Average Depth of Flow in the channel section (cm)	A=Area of the channel section $(B \times h)$	Q=Discharge entering in to the channel (cm^3/s)	U=Q/A= Approaching flow velocity (cm/s)
1	10	150	2.6	390	384.023	0.98
2	10	150	2.4	360	274.01	0.77
3	10	150	2.5	375	158.62	0.43
4	10	150	2.7	405	534.76	1.33
5	10	150	2.8	420	578.23	1.38
6	10	150	3.0	450	653.15	1.46

3.6.3 Threshold Approaching Flow Velocity Measurement (U_{cr})

Clear-water scour occurs for mean flow velocitiy up to the threshold velocity for bed materials entrainment. i.e.,$U \leq U_{cr}$. While live-bed scour takes place for $U > U_{cr}$ and the maximum equilibrium scour depth occurs at, $= U_{cr}$. Here in this present investigation, the condition of the clear-water scour depth at the composite foundation in the flood plain area has been discussed only.

The threshold shear velocity (u_{*c}) and threshold Approaching flow velocity (U_{cr}) are calculated as follows ***(by Sheppard et al.'s method):***

$$u_{*c} = \left\{16.2 d_{50}\left[\frac{9.09\times10^{-6}}{d_{50}} - d_{50}(38.76 + 9.6 ln d_{50}) - 0.005\right]\right\}^{0.5} \tag{3.8}$$

$$U_{cr} = u_{*c} 2.5 ln\left(2.21\frac{h}{d_{50}}\right) \tag{3.9}$$

Here to find the clear-water condition, we have to compare the calculated value of approaching flow velocity (U) with the threshold approaching flow velocity (U_{cr}) . Then we can find the ratio as $U/U_{cr} < 1$ (clear-water scour condition).

Calculating the threshold approaching flow velocity $(U_{cr})$from equation (3.9) for the $d_{50} = 0.2mm$.

$$u_{*c} = \left\{16.2d_{50}\left[\frac{9.09\times10^{-6}}{d_{50}} - d_{50}(38.76 + 9.6lnd_{50}) - 0.005\right]\right\}^{0.5}$$

$$= \left\{16.2 \times 0.2 \times 10^{-3}\left[\frac{9.09\times10^{-6}}{0.2\times10^{-3}} - 0.2\times10^{-3}[38.76 + 9.6ln(0.2\times10^{-3})] - 0.005\right]\right\}^{0.5}$$

$$= 0.012\ m\ s^{-1}$$

Then the value of U_{cr} is calculated for the different values of the flow depth (h) for the six runs by using the equation 3.9

$$U_{cr} = u_{*c}2.5ln\left(2.21\frac{h}{d_{50}}\right)$$

For the first run (h=0.026 m)

$$U_{cr} = 0.012 \times 2.5ln\left(2.21 \times \frac{0.026}{0.2\times10^{-3}}\right) = 0.169\ m\ s^{-1}$$

For the second run (h=0.024 m)

$$U_{cr} = 0.012 \times 2.5ln\left(2.21 \times \frac{0.024}{0.2\times10^{-3}}\right) = 0.168\ m\ s^{-1}$$

For the third run (h=0.025 m)

$$U_{cr} = 0.012 \times 2.5ln\left(2.21 \times \frac{0.025}{0.2\times10^{-3}}\right) = 0.169\ m\ s^{-1}$$

For the fourth run (h=0.027 m)

$$U_{cr} = 0.012 \times 2.5ln\left(2.21 \times \frac{0.027}{0.2\times10^{-3}}\right) = 0.171\ m\ s^{-1}$$

For the fifth run (h=0.028 m)

$$U_{cr} = 0.012 \times 2.5ln\left(2.21 \times \frac{0.028}{0.2 \times 10^{-3}}\right) = 0.173\, m\, s^{-1}$$

For the sixth run (h=0.03 m)

$$U_{cr} = 0.012 \times 2.5ln\left(2.21 \times \frac{0.03}{0.2 \times 10^{-3}}\right) = 0.175\, m\, s^{-1}$$

Now comparing the approaching flow velocity (U) have been calculated for the six run with the threshold approaching flow velocity (U_{cr}) and then comparing the ratio as $U/U_{cr} < 1$ (for clear-water scour).

Run number-1

$$U = 0.0098\, ms^{-1} and U_{cr} = 0.169\, m\, s^{-1}\, thus U < U_{cr}$$

$$For\ \ U/U_{cr} = 0.0098/0.169 = 0.058 < 1\ \ (clear - water\ scour)$$

Run number-2

$$U = 0.0077\, m\, s^{-1} and\ U_{cr} = 0.168\, m\, s^{-1}\ thus\ U < U_{cr}$$

$$For\ \ U/U_{cr} = 0.0077/0.168 = 0.046 < 1\ \ (clear - water\ scour)$$

Run number-3

$$U = 0.0043\, m\, s^{-1} and\ U_{cr} = 0.169\, m/s\ thus\ U < U_{cr}$$

$$For\ \ U/U_{cr} = 0.0043/0.169 = 0.026 < 1\ (clear - water\ scour)$$

Run number-4

$$U = 0.0133\, m\, s^{-1}\ and\ U_{cr} = 0.171\, m\, s^{-1}\ thus\ U < U_{cr}$$

$$For\ \ U/U_{cr} = 0.0133/0.171 = 0.078 < 1\ \ (clear - water\ scour)$$

Run number-5

$$U = 0.0138\, m\, s^{-1}\ and\ U_{cr} = 0.173\, m\, s^{-1}\ thus\ U < U_{cr}$$

$$For\ \ U/U_{cr} = 0.0138/0.173 = 0.079 < 1\ \ (clear - water\ scour)$$

Run number-6

$U = 0.0146\ m\ s^{-1}\ and\ U_{cr} = 0.175\ m\ s^{-1}\ thus\ U < U_{cr}$

$For\ \ U/U_{cr} = 0.0146/0.175 = 0.084 < 1\ \ (clear - water\ scour)$

The Value of approaching flow velocity(U)and threshold approaching flow velocity(U_{cr}) and their relation for $d_{50} = 0.2\ mm$ is given in the table (3.5).

Table -3.5: Values of approaching flow velocity (U) and threshold approaching flow velocity (Ucr) and their relation for $d_{50} = 0.2mm$.

Run no.	Threshold shear velocity $(u_{*c})\ m\ s^{-1}$	Approaching flow velocity $(U)\ m\ s^{-1}$	Threshold approaching flow velocity $(U_{cr})\ m\ s^{-1}$	$For\ U/U_{cr} < 1$ $(clear - water\ scour)$
1	0.012	0.0098	0.169	0.058
2	0.012	0.0077	0.168	0.046
3	0.012	0.0043	0.169	0.026
4	0.012	0.0133	0.171	0.078
5	0.012	0.0138	0.173	0.079
6	0.012	0.0146	0.175	0.084

3.7 EXPERIMENTAL PROCEDURE

The following experimental procedures have been followed in different phases as:

1. After finding the discharge and approaching flow velocity of the water entering in to the channel section of the flume, the relation between the clear-water local scour-depth for the composite foundation model with time at the different-different locations is calculated. Six readings for the scour depth for six run of the flume (10hrs of each run) have been predicted for the pile model by shifting the pile model at three different locations in the flume over the sand bed.

2. For each run the values of discharge and approaching flow velocity was kept different but for that particular run these values were kept constant.

3. The duration of each run was 10hrs. And for the experiment six runs have been performed.

4. During the experiment, the scour depth variation was calculated at three different locations by shifting the composite foundation model inside the channel section of the flume over the sand bed.

5. First location of the composite foundation model was near the glass wall of the flume and the scour depth variation with time was calculated for the six runs. Values of scour depth are given in table (3.6).

6. In the Second location of the composite foundation model, the model was shifted by 30 cm away from the side face of the glass wall of the flume and the scour depth variation with time was calculated for the six runs. Values of scour depth are given in table (3.7).

7. In the third location of the composite foundation model, the model was shifted by 49.5 cm away from the side face of the glass wall of the flume and then the scour depth variation with time was calculated for the six runs. Values of scour depth are given in table (3.8).

8. Again by keeping the same values of the discharge and Approaching flow Velocity, the Clear-water scour depth variation with time was estimated by tilting the composite foundation model by $15°, 30°\ and\ 45°$ with respect to the side face glass wall of the experimental flume. The variation of the clear-water depth with time was calculated for the six runs for each location. Values of the scour depth have been given in the table (3.9), (3.10), and (3.11).

9. All the values and calculated data were kept same for all the calculations. Only the location of the composite foundation model was changed to obtain the variation of clear-water scour depth with time.

10. On the basis of data obtained from the table 3.6, 3.7, 3.8, 3.9, 3.10 and 3.11, graphs were drawn showing the variation of the clear-water scour depth with time.

11. The scour depth for each run at every pile was measured by using Digital Pointer Gauge arrangement as shown in the figure (3.3).

Figure: 3.3 - Digital Pointer Gauge Arrangement

3.8 FRAMEWORK OF ANALYSIS

The process of local scour at Composite foundation is time dependent phenomenon. An equilibrium between the erosive tendency of the flowing water and the stresses to motion of the bed sediments was continuously attained through erosion of the flowing boundary. In fine-grained sediments such as sand and gravel, the scouring process attained rapidly for live-bed condition but it occurs gradually for the clear-water condition.

3.8.1 Mathematical Modelling for clear-water Scour Depth

The clear-water scour depth around Composite foundation depend on the variables characterising the fluid, flow, bed sediment and time from the beginning of the scour. The functional relationship describing the clear-water local scour depth, Ds, at a composite foundation for a particular time, t, can be written as

$$D_s = f_1(\rho, \nu, U, H, \rho_s, d_{50}, g, D, t) \qquad (3.10)$$

Where, ρ, ρ_s is the mass density of fluid and sediment, respectively, U= mean velocity of flow, H = depth of flow, d_{50}= the median size of bed materials, D = the pile Diameter, t= time from beginning of scour, ν= Kinematic viscosity of water, and g = acceleration due to gravity.

By using $\boldsymbol{Buckingham's\ \pi - Theorem}$ "if there are n numbers of variables (independent and dependent variables) in a physical phenomenon and if these variables contain m number of fundamental dimensions (M, L, T), then these variables can be arranged into (n-m) dimensionless terms. Each term is called $\pi - term$.

Then equation 3.10 can be written as,

$$f_1(D_s, \rho, \nu, U, H, \rho_s, d_{50}, g, D, t) = 0 \qquad (3.11)$$

Total number of variables, n=10

Number of fundamental dimensions=3

m is the number of fundamental dimensions in the problem and obtained by writing dimensions of each variable as:

$$\rho = ML^{-3}, \nu = L^2T^{-1}, U = LT^{-1}, \rho_s = ML^{-3}, g = LT^{-2}$$

Thus as fundamental dimensions in the problem are M, L, T, and hence m=3.

Number of dimensionless $\pi - terms = n - m = 10 - 3 = 7$

Thus, seven $\pi - terms\ say\ \pi_1, \pi_2, \pi_3, \pi_4, \pi_5, \pi_6, \pi_7$ are formed, hence equation 3.11 can be written as

$$f_1 = (\pi_1, \pi_2, \pi_3, \pi_4, \pi_5, \pi_6, \pi_7) = 0 \qquad (3.12)$$

Choosing D, U and ρ as repeating variables, we have

$$\pi_1 = D^{a_1}.U^{b_1}.\rho^{c_1}.D_s$$

$$\pi_2 = D^{a_2}.U^{b_2}.\rho^{c_2}.\nu$$

$$\pi_3 = D^{a_3}.U^{b_3}.\rho^{c_3}.H$$

$$\pi_4 = D^{a_4}.U^{b_4}.\rho^{c_4}.\rho_s$$

$$\pi_5 = D^{a_5}.U^{b_5}.\rho^{c_5}.d_{50}$$

$$\pi_6 = D^{a_6}.U^{b_6}.\rho^{c_6}.g$$

$$\pi_7 = D^{a_7}.U^{b_7}.\rho^{c_7}.t$$

First $\pi - term$

Substituting the dimensions on both sides of π_1,

$M^0L^0T^0 = L^{a_1}.(LT^{-1})^{b_1}.(ML^{-3})^{c_1}.M^0L^1T^0$

Equating powers of M, L and T on both the sides

Power of M, $\quad 0 = c_1 \therefore \quad c_1 = 0$

Power of L, $\quad 0 = a_1 + b_1 - 3c_1 + 1 \therefore \quad a_1 = -1$

Power of T, $\quad 0 = -b_1 \therefore \quad b_1 = 0$

Substituting the values of $a_1, b_1 and\ c_1 in\ \pi_1$,

$$\pi_1 = D^{-1}.U^0.\rho^0.D_s \qquad \therefore \quad \boldsymbol{\pi_1 = \frac{D_s}{D}}$$

Second $\pi - term$

Substituting the dimensions on both sides of π_2,

$M^0L^0T^0 = L^{a_2}.(LT^{-1})^{b_2}.(ML^{-3})^{c_2}.M^0L^2T^{-1}$

Equating powers of M, L and T on both the sides

Power of M, $0 = c_2 \therefore \quad c_2 = 0$

Power of L, $0 = a_2 + b_2 - 3c_2 + 2 \therefore \quad a_2 = -1$

Power of T, $0 = -b_2 - 1 \therefore \quad b_2 = -1$

Substituting the values of $a_2, b_2 and\ c_2 in\ \pi_2$,

$$\pi_2 = D^{-1}.U^{-1}.\rho^0.\nu \qquad \therefore \qquad \boldsymbol{\pi_2 = \frac{\nu}{DU}}$$

Third π − term

Substituting dimensions on both sides of π_3,

$M^0L^0T^0 = L^{a_3}.(LT^{-1})^{b_3}.(ML^{-3})^{c_3}.M^0L^1T^0$

Equating powers of M, L and T on both the sides

Power of M, $0 = c_3 \therefore \quad c_3 = 0$

Power of L, $0 = a_3 + b_3 - 3c_3 + 1 \quad \therefore \quad a_3 = -1$

Power of T, $0 = -b_3 \therefore \quad b_3 = 0$

Substituting the values of $a_3, b_3 and\ c_3 in\ \pi_3$,

$$\pi_3 = D^{-1}.U^0.\rho^0.H \qquad \therefore \quad \boldsymbol{\pi_3 = \frac{H}{D}}$$

Fourth π − term

Substituting the dimensions on the both sides of π_4,

$M^0L^0T^0 = L^{a_4}.(LT^{-1})^{b_4}.(ML^{-3})^{c_4}.M^1L^{-3}T^0$

Equating the powers of M, L and T on both sides

Power of M, $0 = c_4 + 1 \therefore \quad c_4 = -1$

Power of L, $0 = a_4 + b_4 - 3c_4 - 3 \therefore \quad a_4 = -6$

Power of T, $0 = -b_4 \therefore \quad b_4 = 0$

Substituting the values of $a_4, b_4 and\ c_4 in\ \pi_4$,

$$\pi_4 = D^{-6}.U^0.\rho^{-1}.\rho_s \qquad \therefore \quad \boldsymbol{\pi_4 = \frac{\rho_s}{D^6\rho}}$$

***Fifth* $\boldsymbol{\pi - term}$**

Substituting dimensions on both sides of π_5,

$M^0L^0T^0 = L^{a_5}.(LT^{-1})^{b_5}.(ML^{-3})^{c_5}.M^0L^1T^0$

Equating the powers of M, L and T on both sides

Power of M, $\qquad 0 = c_5 \therefore \quad c_5 = 0$

Power of L, $\qquad 0 = a_5 + b_5 - 3c_5 + 1 \therefore \quad a_5 = -1$

Power of T, $\qquad 0 = -b_5 \therefore \quad b_5 = 0$

Substituting the values of $a_5, b_5 and\ c_5 in\ \pi_5$,

$$\pi_5 = D^{-1}.U^0.\rho^0.d_{50} \qquad \therefore \quad \boldsymbol{\pi_5 = \frac{d_{50}}{D}}$$

***Sixth* $\boldsymbol{\pi - term}$**

Substituting dimensions on both sides of π_6,

$M^0L^0T^0 = L^{a_6}.(LT^{-1})^{b_6}.(ML^{-3})^{c_6}.M^0L^1T^{-2}$

Equating the powers of M, L and T on both sides

Power of M, $\qquad 0 = c_6 \therefore \quad c_6 = 0$

Power of L, $\qquad 0 = a_6 + b_6 - 3c_6 + 1 \therefore \quad a_6 = 1$

Power of T, $\qquad 0 = -b_6 - 2 \therefore \quad b_6 = -2$

Substituting the values of $a_6, b_6 and\ c_6 in\ \pi_6$,

$$\pi_6 = D^1.U^{-2}.\rho^0.g \qquad \therefore \quad \boldsymbol{\pi_6 = \frac{Dg}{U^2}}$$

$\boldsymbol{Seventh\ \pi - term}$

Substituting dimensions on both sides of π_7,

$M^0L^0T^0 = L^{a_7}.(LT^{-1})^{b_7}.(ML^{-3})^{c_7}.M^0L^0T^1$

Equating the powers of M, L and T on both sides

Power of M, $\quad 0 = c_7 \therefore \quad c_7 = 0$

Power of L, $\quad 0 = a_7 + b_7 - 3c_7 \therefore \quad a_7 = -1$

Power of T, $\quad 0 = -b_7 + 1 \therefore \quad b_7 = 1$

Substituting the values of $a_7, b_7 and\ c_7 in\ \pi_7$,

$$\pi_7 = D^{-1}.U^1.\rho^0.t \qquad \therefore \quad \boldsymbol{\pi_7 = \frac{Ut}{D}}$$

Substituting the values $\pi_1, \pi_2, \pi_3, \pi_4, \pi_5, \pi_6\ and\ \pi_7$ in equation 3.12

$$f_1 = \left(\frac{D_s}{D}, \frac{\nu}{DU}, \frac{H}{D}, \frac{\rho_s}{D^6\rho}, \frac{d_{50}}{D}, \frac{Dg}{U^2}, \frac{Ut}{D}\right) = 0$$

Again, the above expression can be written as

$$\frac{D_s}{D} = \phi\left(\frac{\nu}{DU}, \frac{H}{D}, \frac{\rho_s}{D^6\rho}, \frac{d_{50}}{D}, \frac{Dg}{U^2}, \frac{Ut}{D}\right) \quad (3.13)$$

The above equation can be written as:

$$\frac{D_s}{D} = \phi\left(\frac{UD}{\nu}, \frac{H}{D}, \frac{D}{d_{50}}, \frac{U^2}{gD}, \frac{Ut}{D}\right) \quad (3.14)$$

The different parameters used in the equation (3.14) have the following significance:

1. The term $UD/\nu$is defined as pier Reynolds number and is usually not a important parameter and so it can be neglected from equation (3.14) as the ` flow around the pier and composite foundation is completely turbulent.

2. The term,H/D,is a necessary geometrical ratio expressed in part the geometry of the vortex system. H/Dratio is also very useful to describe the action of the down-flow into the scour hole, the horse-shoe vortex at the base of the composite foundation, and the counter rotating water surface roller near the top of the composite foundation.

3. The term, D/d_{50}, is the pile diameter relative to sediment mean size, termed the sediment coarseness.

4. The term, U^2/gD,$[i.e. IF_P = U/(gD)^{0.50}, defined\ as\ pier\ Reynolds\ number]$.

5. The term, Ut/D, expresses similarity in time development of scour. To establish relationship for scour depth for any time, t, it is a important parameter.

Thus from the above discussion following non-dimensional parameters describing the scour depth at any time, t has been found to be important and can be written as-

$$D_S/D = f_2(H, d_{50}, g, t) \tag{3.15}$$

These parameters can be replaced by the following ones given below as:

$$D_S/D = \ f_3(H/D, D/d_{50}, U^2/gD, Ut/D) \tag{3.16}$$

Analysing the results of fine sand $(d_{50} = 0.2mm)$ for 10-hours flow duration, the relation between the scour depth versus time have been shown through the graphs for the same diameter of the piles. It can be observed that the increment in the scour depth is a time dependent phenomenon and increasing gradually with increasing time for the clear-water condition.

As an initial attempt to correlate scour depth with time, a curve obtained from regression analysis in functional form-

$$D_S/D = f_4(H/D, U^2/gD, Ut/D) \tag{3.17}$$

$$D_S/D = f_4(H/D, IF_P, H/D) \tag{3.18}$$

is plotted in figure (3.3). This figure shows that the scour depth is tending to reach equilibrium at long hours of run, also the relationship shows that scour depth varies with time in logarithmic scale in the mid range. As given the following equations i.e.

$$D_S/D = 1.05\ln(X) - 1.47(1 - e^{0.17X^2}) \tag{3.19}$$

where $X = \ln\,(Ut/D)(U^2/gD)^{0.21}(H/D)^{0.24}$

In the initial and at the final stages, the scour depth follows nonlinear pattern with logarithmic time as a variable. This leads to observe the rate of scour as a function of time.

By regression analysis of the data, the best fit curve passing through the maximum points is given by the curve,

$$Y = 0.00045X^{-0.46} \tag{3.20a}$$

$$\text{where } X = D_S\left(H/D^{2.18}\right)^{8.25} \tag{3.20b}$$

Figure 3.4 shows a curve relating the rate of scour with non-dimensional time. It may be observed that the rate of scour is very high in the beginning and it goes on decreasing as the scour depth increases with time. An expression for rate of scour depth from regression analysis is obtained as-

$$\left[\frac{d(D_S/D)}{d(Ut/D)}\right] = 0.365\left(\frac{Ut}{D}\right)^{-1.12} \tag{3.21}$$

In order to predict the values of relative scour depth (D_s/D), equation (3.21) was integrated and the following equation was obtained-

$$\frac{D_S}{D} = -3.04\left(\frac{Ut}{D}\right)^{-0.12} + C \tag{3.22}$$

where C is the integration constant.

With the condition that t=0 the constant C was not possible to be evaluated and the other condition i.e. equilibrium condition for which time, t, is very large and for that time scour depth, D_s, is not known. Due to this, scour depth at intermediate value of time, t, is chosen for the evaluation of constant C. Evaluating the integration constant C at a mid range value of $Ut/D = 10^5$ for the diameter of the pile used in the experiment i.e. D_S/D at $Ut/D = 10^5$

with the other parameter such as IF_P from regression analysis. One may have the following relation,

$$D_s/D = 10.52(IF_P)^{0.62} \qquad ; \text{at} Ut/D = 10^5 \qquad (3.23)$$

From the equation (3.22)

$$D_S/D = -0.75 + C \qquad ; \text{at} Ut/D = 10^5 \qquad (3.24)$$

and the constant

$$C = 0.75 + D_S/D = 10.52(IF_P)^{0.62} + 0.75 \qquad ; \text{at} Ut/D = 10^5 \qquad (3.25)$$

Using the above relations the scour depth may be calculated as-

$$D_S/D = 10.52(U^2/gD)^{0.62} - 3.04(Ut/D)^{0.12} + 0.75 \qquad (3.26)$$

To verify these relations derived as given in equation (3.19). Agreement between observed values and the computed values from equation (3.19) found to be reasonably good as shown through the graphs.

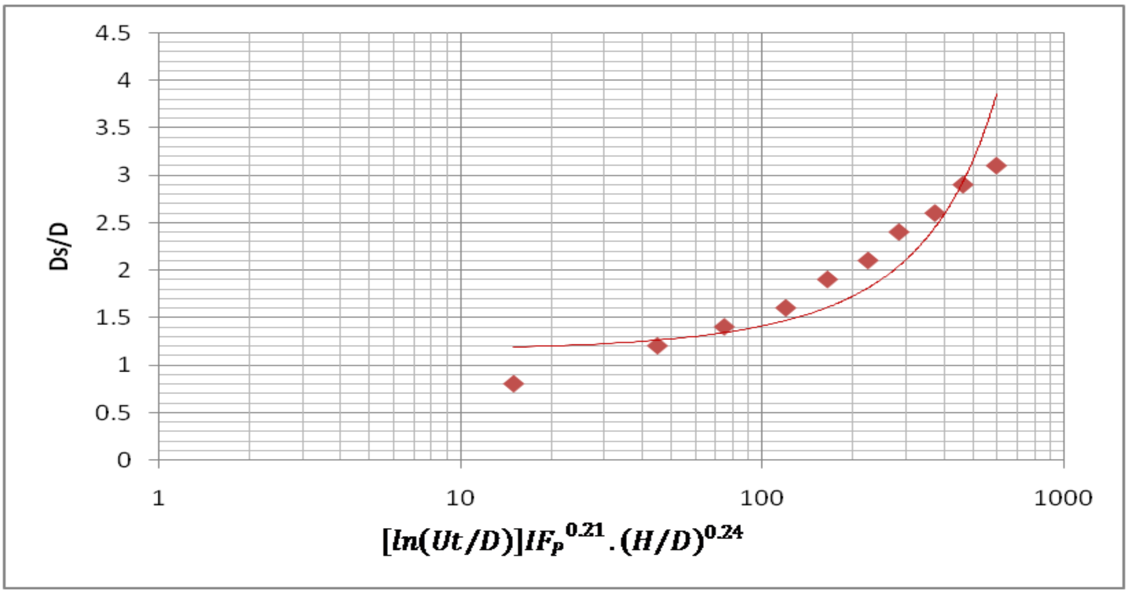


Figure.3.4: Prediction of Scour Depth as a Function of Time

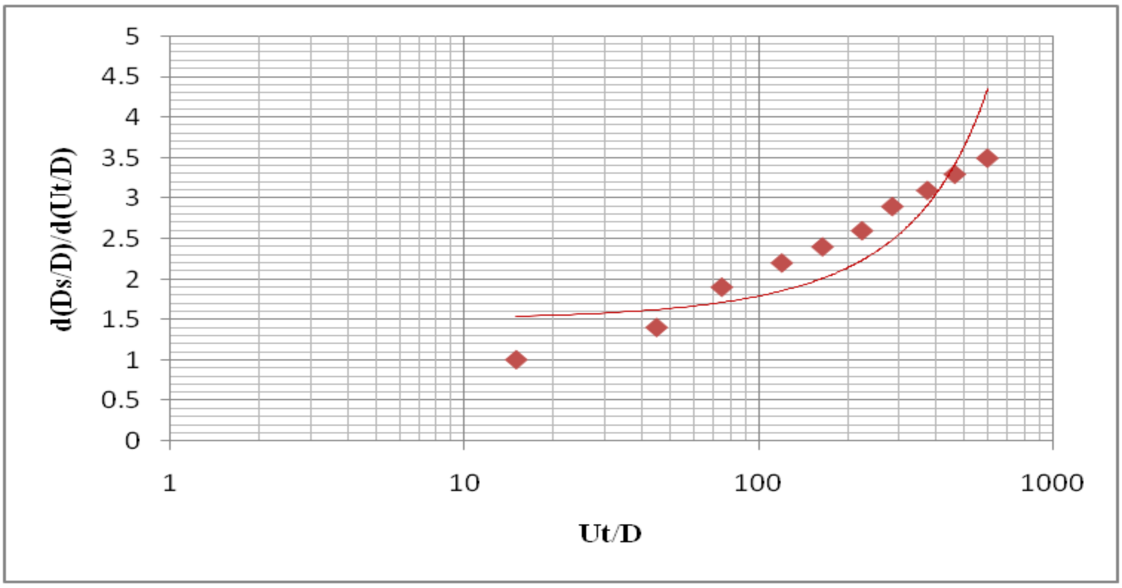


Figure.3.5: Rate of Scour Variation with Time for Fine Sand

DIFFERENT LOCATIONS OF THE COMPOSITE FOUNDATION MODEL

Figure: 3.6 Composite foundation Model near side wall of the channel

Figure: 3.7 Composite foundation model placed 30cm away from side face of the channel

Figure: 3.8 Composite foundation Model placed 49.5cm away from side face of the channel

Figure: 3.9 Composite foundation Model Tilted by 15° $w.r.to$ side Face of the channel

Figure: 3.10   Composite foundation Model Tilted by 30° $w.r.to$ side Face of the channel

Figure: 3.11   Composite foundation Model Tilted by 45° $w.r.to$ side face of the channel

TABLES AND GRAPHS

Observed Scour Depth Vs Time for the Clear-water Condition

TABLE -3.6

Data for the variation of the clear-water scour depth (D_s) with Time (t) for 10 hrs of run when the pile model is placed near the side wall of the channel.

Run No.	Time in Minutes	Scour depth D_s (cm)			Dia. of pile (cm)
		Pile no.- 1	Pile no.- 2	Pile no.- 3	
1	15	0.8	0.5	0.6	2.8
	45	1.2	0.9	0.9	2.8
	75	1.4	1.1	1.2	2.8
	120	1.6	1.3	1.4	2.8
	165	1.9	1.5	1.6	2.8
	225	2.1	1.8	1.8	2.8
	285	2.4	2.0	2.0	2.8
	375	2.6	2.2	2.2	2.8
	465	2.9	2.4	2.5	2.8
	600	3.1	2.6	2.8	2.8
2	15	0.4	0.5	0.4	2.8
	45	0.6	0.9	0.8	2.8
	75	0.8	1.2	1.1	2.8
	120	1.1	1.4	1.5	2.8
	165	1.3	1.6	1.7	2.8
	225	1.5	1.8	1.9	2.8
	285	1.8	2.0	2.0	2.8

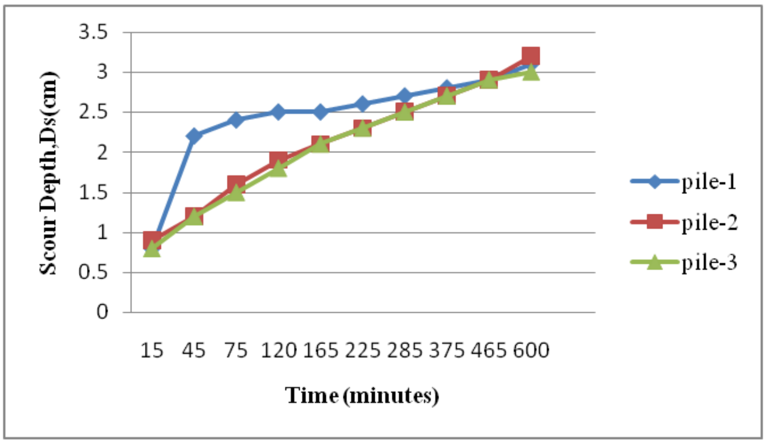
	375	2.0	2.3	2.2	2.8
	465	2.2	2.5	2.4	2.8
	600	2.5	2.8	2.6	2.8
3	15	0.8	0.6	0.6	2.8
	45	1.1	0.9	1.0	2.8
	75	1.5	1.2	1.5	2.8
	120	1.7	1.4	1.8	2.8
	165	1.9	1.6	2.0	2.8
	225	2.1	1.9	2.3	2.8
	285	2.3	2.0	2.5	2.8
	375	2.5	2.3	2.8	2.8
	465	2.8	2.5	3.0	2.8
	600	3.0	2.8	3.2	2.8
4	15	1	0.9	0.8	2.8
	45	1.4	1.2	1.1	2.8
	75	1.9	1.6	1.6	2.8
	120	2.2	1.9	1.8	2.8
	165	2.4	2.1	2.0	2.8
	225	2.6	2.4	2.2	2.8
	285	2.9	2.6	2.5	2.8
	375	3.1	2.8	2.8	2.8
	465	3.3	3.0	3.0	2.8
	600	3.5	3.2	3.2	2.8
5	15	0.9	1.0	0.8	2.8
	45	1.3	1.3	1.1	2.8

	75	1.6	1.6	1.3	2.8
	120	1.9	1.9	1.6	2.8
	165	2.3	2.1	1.9	2.8
	225	2.6	2.4	2.2	2.8
	285	2.9	2.6	2.4	2.8
	375	3.1	2.9	2.6	2.8
	465	3.4	3.1	2.9	2.8
	600	3.6	3.3	3.1	2.8
6	15	1.1	0.9	1.1	2.8
	45	1.6	1.2	1.6	2.8
	75	2.0	1.6	1.9	2.8
	120	2.4	1.9	2.0	2.8
	165	2.6	2.1	2.3	2.8
	225	2.9	2.4	2.5	2.8
	285	3.1	2.6	2.8	2.8
	375	3.4	2.8	3.0	2.8
	465	3.6	3.0	3.3	2.8
	600	3.9	3.3	3.5	2.8

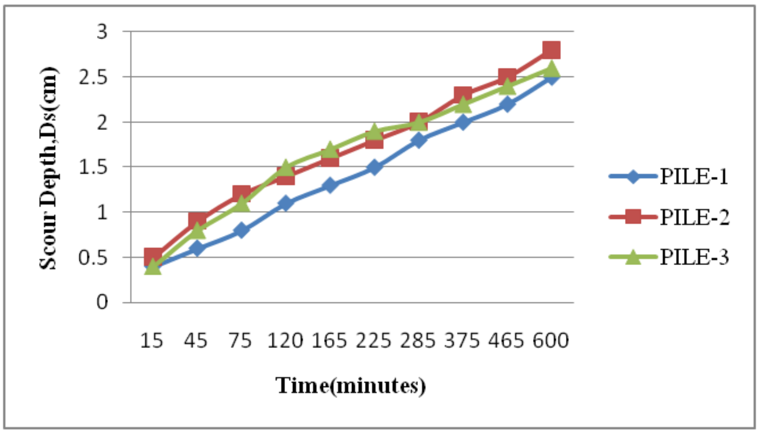
Graph-3.2

Graphs for the variation of the clear-water scour depth (D_s) with Time (t) for 10 hrs. of Run when the pile model is placed near the side wall of the channel

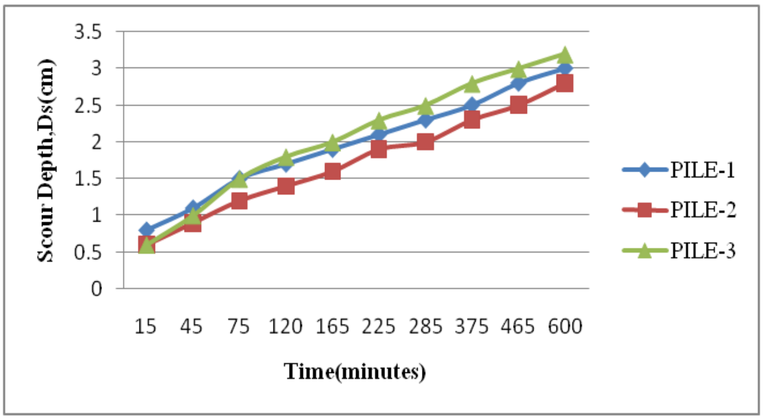
First run



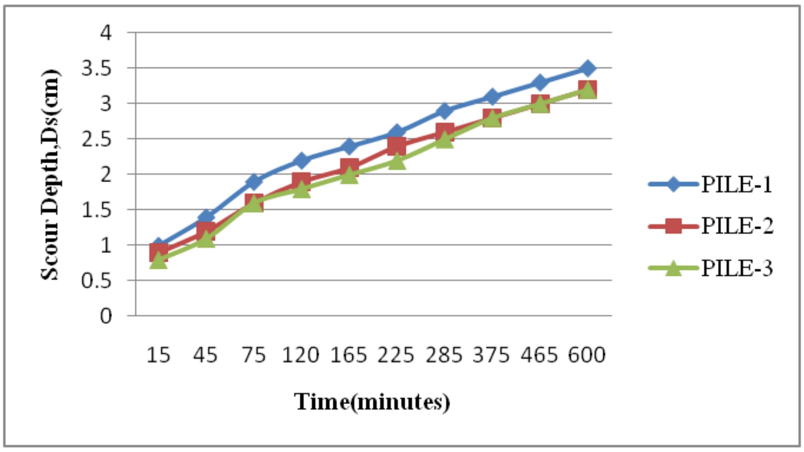
Second run



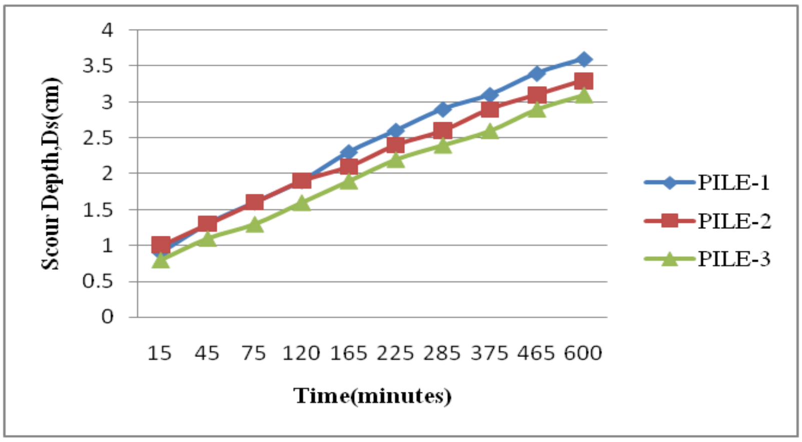
Third run



Fourth run



Fifth run



Sixth run

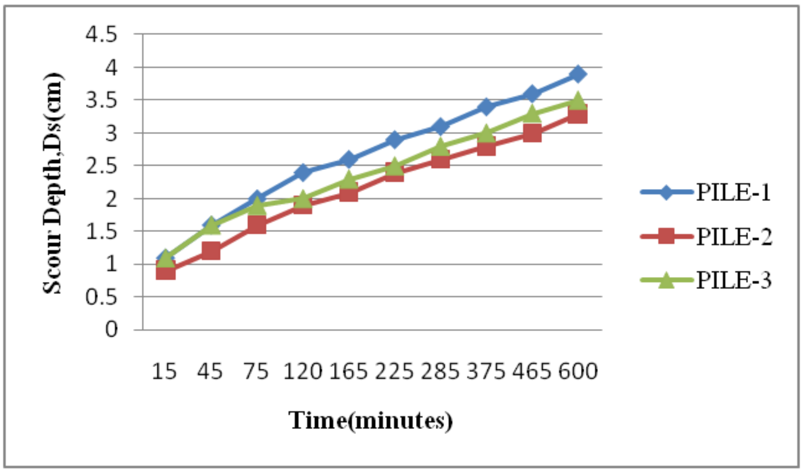


TABLE -3.7

Data for the variation of the clear-water scour depth (D_S) with Time (t) for 10 hrs of run when the pile model is placed 30cm away from the side wall of the channel.

Run No.	Time in Minutes	Scour depth D_s (cm)			Dia. of pile (cm)
		Pile no.- 1	Pile no.- 2	Pile no.- 3	
1	15	1	0.9	1.1	2.8
	45	1.4	1.3	1.6	2.8
	75	1.9	1.6	2.0	2.8
	120	2.2	1.9	2.4	2.8
	165	2.4	2.3	2.6	2.8
	225	2.6	2.6	2.9	2.8
	285	2.9	2.9	3.1	2.8
	375	3.1	3.1	3.4	2.8
	465	3.3	3.4	3.6	2.8
	600	3.5	3.6	3.9	2.8
2	15	0.8	0.4	0.8	2.8
	45	1.2	0.6	1.1	2.8

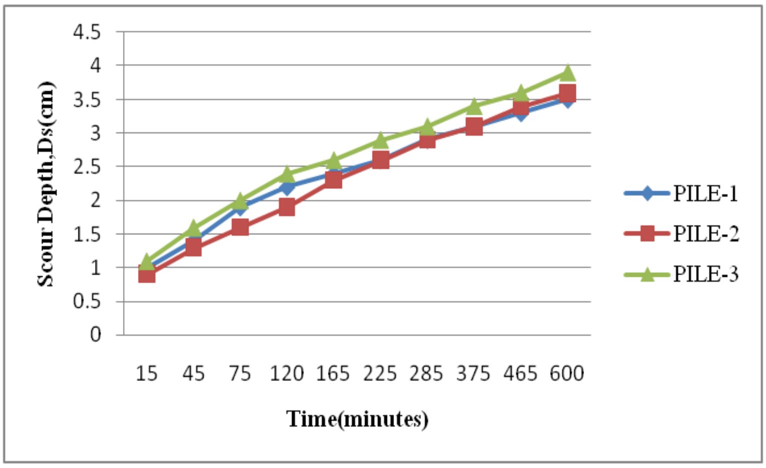
	75	1.4	0.8	1.5	2.8
	120	1.6	1.1	1.7	2.8
	165	1.9	1.3	1.9	2.8
	225	2.1	1.5	2.1	2.8
	285	2.4	1.8	2.3	2.8
	375	2.6	2.0	2.5	2.8
	465	2.9	2.2	2.8	2.8
	600	3.1	2.5	3.0	2.8
3	15	0.9	1.0	0.9	2.8
	45	1.2	1.3	1.2	2.8
	75	1.6	1.6	1.6	2.8
	120	1.9	1.9	1.9	2.8
	165	2.1	2.1	2.1	2.8
	225	2.4	2.4	2.4	2.8
	285	2.6	2.6	2.6	2.8
	375	2.8	2.9	2.8	2.8
	465	3.0	3.1	3.0	2.8
	600	3.2	3.3	3.3	2.8
4	15	0.5	0.5	0.6	2.8
	45	0.9	0.9	0.9	2.8
	75	1.1	1.2	1.2	2.8
	120	1.3	1.4	1.4	2.8
	165	1.5	1.6	1.6	2.8
	225	1.8	1.8	1.9	2.8

	285	2.0	2.0	2.0	2.8
	375	2.2	2.3	2.3	2.8
	465	2.4	2.5	2.5	2.8
	600	2.6	2.8	2.8	2.8
5	15	0.8	0.8	1.1	2.8
	45	1.1	1.1	1.6	2.8
	75	1.6	1.3	1.9	2.8
	120	1.8	1.6	2.0	2.8
	165	2.0	1.9	2.3	2.8
	225	2.2	2.2	2.5	2.8
	285	2.5	2.4	2.8	2.8
	375	2.8	2.6	3.0	2.8
	465	3.0	2.9	3.3	2.8
	600	3.2	3.1	3.5	2.8
6	15	0.6	0.4	0.6	2.8
	45	0.9	0.8	1.0	2.8
	75	1.2	1.1	1.5	2.8
	120	1.4	1.5	1.8	2.8
	165	1.6	1.7	2.0	2.8
	225	1.8	1.9	2.3	2.8
	285	2.0	2.0	2.5	2.8
	375	2.2	2.2	2.8	2.8
	465	2.5	2.4	3.0	2.8
	600	2.8	2.6	3.2	2.8

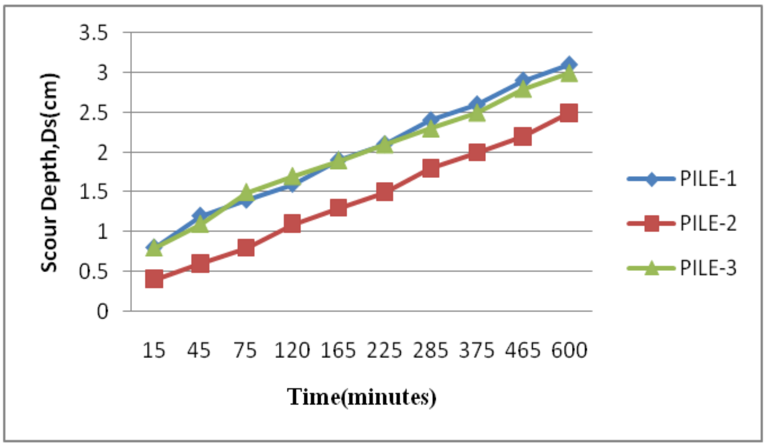
Graph -3.3

Graph for the variation of the clear-water scour depth (D_s) with Time (t) for 10 hrs of run when the pile model is placed 30cm away from the side wall of the channel.

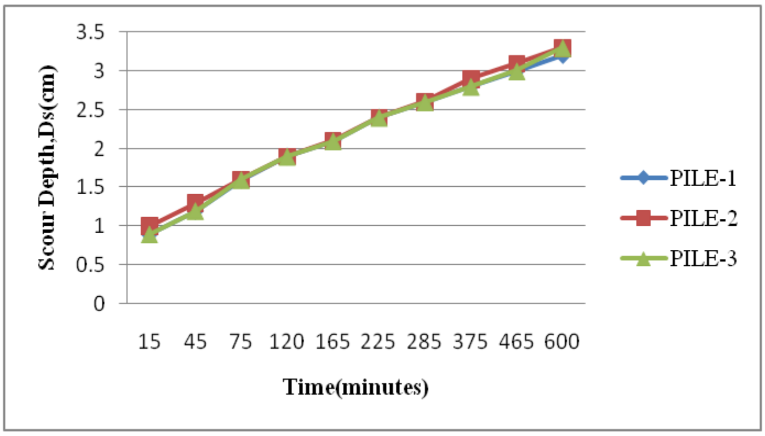
First run



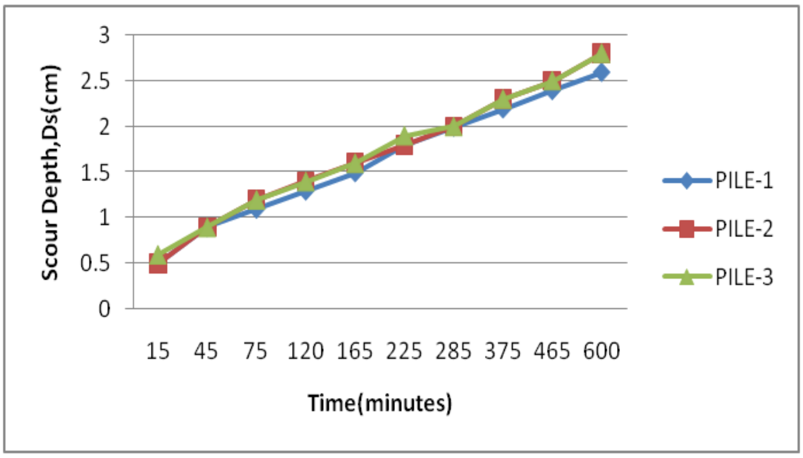
Second run



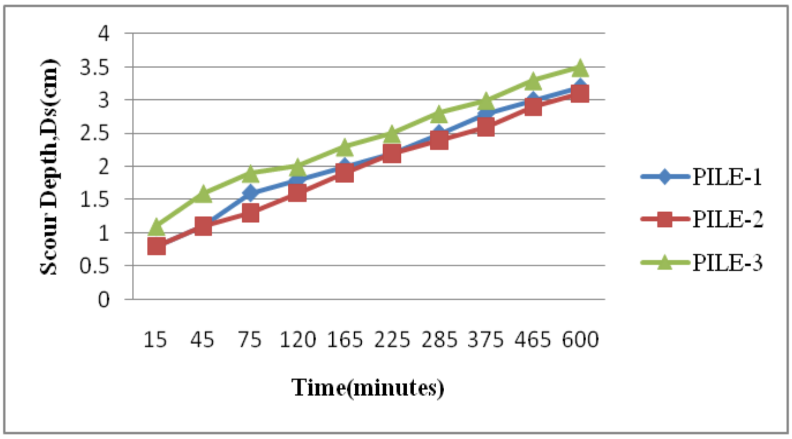
Third run



Fourth run



Fifth run



Sixth run

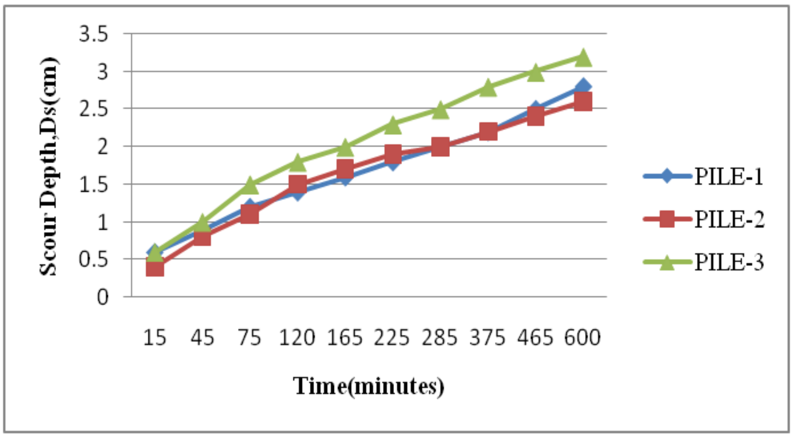


TABLE -3.8

Data for the variation of the clear-water scour depth (D_s) with Time (t) for 10 hrs of run when the pile model is placed at 49.5cm away from the side wall of the channel.

Run No.	Time in Minutes	Scour depth D_s (cm)			Dia. of pile (cm)
		Pile no.- 1	Pile no.- 2	Pile no.- 3	
1	15	0.8	1.0	1.1	2.8
	45	1.1	1.3	1.6	2.8
	75	1.3	1.5	1.9	2.8
	120	1.5	1.8	2.0	2.8
	165	1.9	1.9	2.3	2.8
	225	2.2	2.0	2.5	2.8
	285	2.6	2.2	2.8	2.8
	375	2.8	2.5	3.0	2.8
	465	3.0	2.9	3.3	2.8
	600	3.3	3.3	3.5	2.8
2	15	0.8	1.1	0.9	2.8
	45	1.1	1.4	1.3	2.8
	75	1.5	1.6	1.6	2.8
	120	1.8	1.8	1.9	2.8
	165	2.0	2.0	2.1	2.8
	225	2.3	2.3	2.5	2.8
	285	2.5	2.6	2.8	2.8
	375	2.9	2.8	3.0	2.8
	465	3.1	3.0	3.3	2.8
	600	3.4	3.3	3.5	2.8

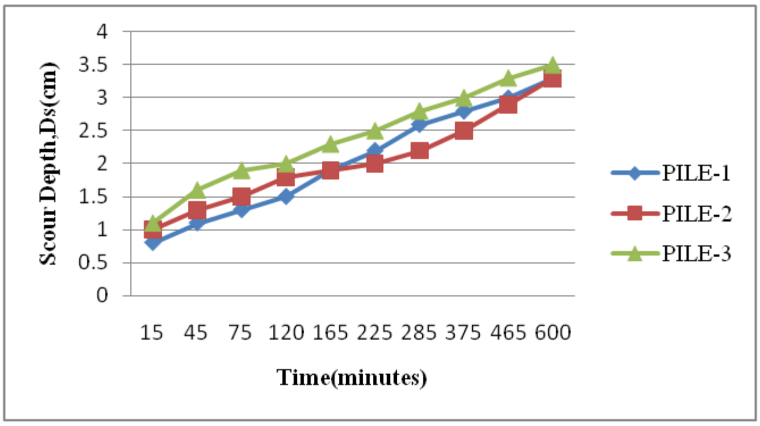
3	15	1.1	0.8	1.1	2.8
	45	1.4	1.1	1.6	2.8
	75	1.6	1.3	1.9	2.8
	120	1.8	1.5	2.0	2.8
	165	2.0	1.9	2.3	2.8
	225	2.2	2.2	2.5	2.8
	285	2.5	2.6	2.8	2.8
	375	2.8	2.8	3.0	2.8
	465	3.0	3.0	3.3	2.8
	600	3.3	3.3	3.5	2.8
4	15	0.9	1.1	0.8	2.8
	45	1.2	1.4	1.1	2.8
	75	1.5	1.6	1.5	2.8
	120	1.8	1.8	1.7	2.8
	165	2.0	2.0	1.9	2.8
	225	2.3	2.2	2.1	2.8
	285	2.5	2.4	2.3	2.8
	375	2.8	2.6	2.5	2.8
	465	3.0	2.9	2.8	2.8
	600	3.3	3.0	3.0	2.8
5	15	0.8	0.5	0.8	2.8
	45	1.2	0.9	1.1	2.8
	75	1.4	1.1	1.3	2.8
	120	1.6	1.3	1.5	2.8
	165	1.9	1.5	1.8	2.8

	225	2.1	1.8	2.0	2.8
	285	2.4	2.0	2.1	2.8
	375	2.6	2.2	2.3	2.8
	465	2.9	2.4	2.5	2.8
	600	3.1	2.6	2.8	2.8
6	15	0.4	0.5	0.9	2.8
	45	0.6	0.9	1.2	2.8
	75	0.8	1.2	1.6	2.8
	120	1.1	1.4	1.9	2.8
	165	1.3	1.6	2.1	2.8
	225	1.5	1.8	2.4	2.8
	285	1.8	2.0	2.6	2.8
	375	2.0	2.3	2.8	2.8
	465	2.2	2.5	3.0	2.8
	600	2.5	2.8	3.2	2.8

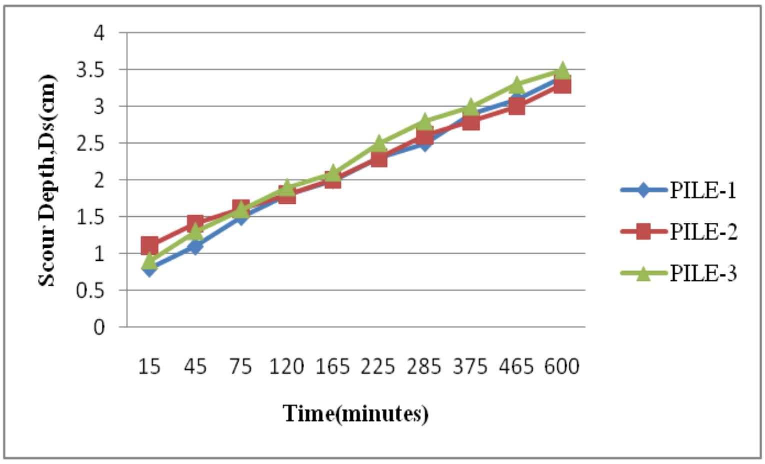
Graph -3.4

Graph for the variation of the clear-water scour depth (D_s) with Time (t) for 10 hrs of run when the pile model is placed at 49.5cm away from the side wall of the channel.

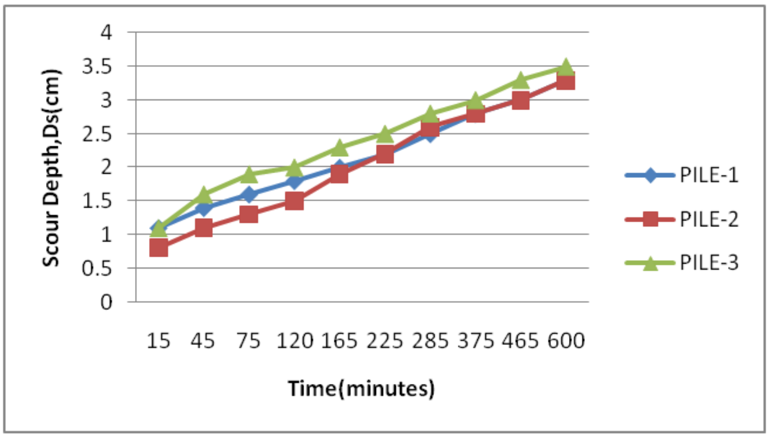
First run



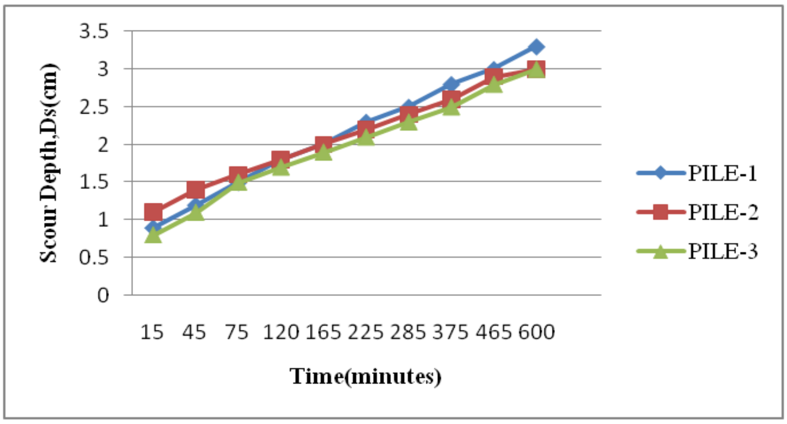
Second run



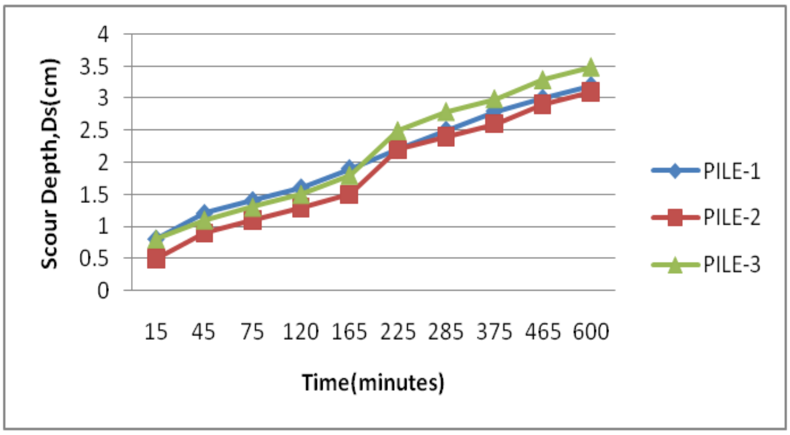
Third run



Fourth run



Fifth run



Sixth run

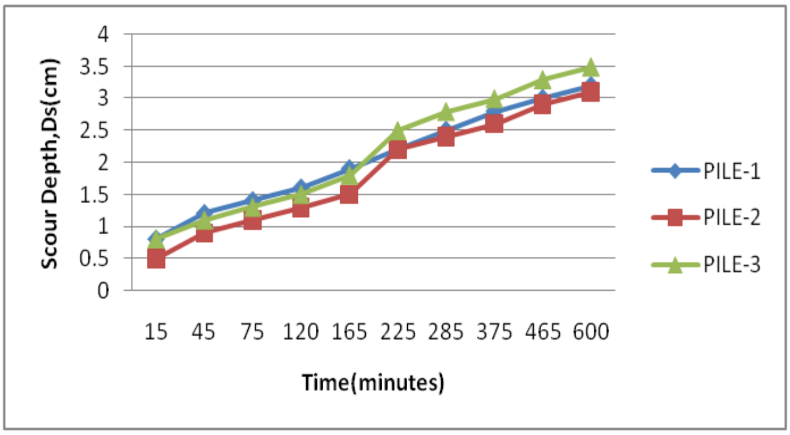


Table -3.9

Data for the variation of scour depth (Ds) with time by tilting the pile model is fixed at an angle of attack of15° $w.r.t.$ side wall of the channel

Run number	Time of run (minutes)	Scour depth, Ds (cm)			Pile dia. (cm)
		Pile-1	Pile-2	Pile-3	
1	15	0.4	0.5	0.9	2.8
	45	0.6	0.9	1.2	2.8
	75	0.8	1.2	1.6	2.8
	120	1.1	1.4	1.9	2.8
	165	1.3	1.6	2.1	2.8
	225	1.5	1.8	2.4	2.8
	285	1.8	2.0	2.6	2.8
	375	2.0	2.3	2.8	2.8
	465	2.2	2.5	3.0	2.8
	600	2.5	2.8	3.2	2.8
2	15	0.9	1.1	0.8	2.8
	45	1.2	1.4	1.1	2.8
	75	1.5	1.6	1.5	2.8
	120	1.8	1.8	1.7	2.8
	165	2.0	2.0	1.9	2.8
	225	2.3	2.2	2.1	2.8
	285	2.5	2.4	2.3	2.8
	375	2.8	2.6	2.5	2.8
	465	3.0	2.9	2.8	2.8
	600	3.3	3.0	3.0	2.8
3	15	0.8	1.1	0.9	2.8

	45	1.1	1.4	1.3	2.8
	75	1.5	1.6	1.6	2.8
	120	1.8	1.8	1.9	2.8
	165	2.0	2.0	2.1	2.8
	225	2.3	2.3	2.5	2.8
	285	2.5	2.6	2.8	2.8
	375	2.9	2.8	3.0	2.8
	465	3.1	3.0	3.3	2.8
	600	3.4	3.3	3.5	2.8
4	15	0.8	1.0	1.1	2.8
	45	1.1	1.3	1.6	2.8
	75	1.3	1.5	1.9	2.8
	120	1.5	1.8	2.0	2.8
	165	1.9	1.9	2.3	2.8
	225	2.2	2.0	2.5	2.8
	285	2.6	2.2	2.8	2.8
	375	2.8	2.5	3.0	2.8
	465	3.0	2.9	3.3	2.8
	600	3.3	3.3	3.5	2.8
5	15	1.1	0.8	1.1	2.8
	45	1.4	1.1	1.6	2.8
	75	1.6	1.3	1.9	2.8
	120	1.8	1.5	2.0	2.8
	165	2.0	1.9	2.3	2.8
	225	2.2	2.2	2.5	2.8

	285	2.5	2.6	2.8	2.8
	375	2.8	2.8	3.0	2.8
	465	3.0	3.0	3.3	2.8
	600	3.3	3.3	3.5	2.8
6	15	1.1	0.8	0.9	2.8
	45	1.4	1.1	1.2	2.8
	75	1.6	1.3	1.4	2.8
	120	1.9	1.4	1.6	2.8
	165	2.0	1.6	1.9	2.8
	225	2.2	1.9	2.1	2.8
	285	2.4	2.1	2.4	2.8
	375	2.6	2.3	2.6	2.8
	465	2.9	2.5	2.9	2.8
	600	3.3	2.8	3.2	2.8

CHAPTER-4

CONCLUSIONS AND SCOPE FOR FURTHER STUDY

CONCLUSIONS

On the basis of present study, the following broad conclusions can be drawn:

The clear-water local scour depth at the Composite foundation, which was related to the components that are exposed to the flow in the channel, was studied experimentally for the steady clear-water condition as the clear water scour is maximum.

The experiment was performed for the different locations of the composite foundation model in the flume and for all the calculations, discharge (Q), approaching flow velocity (U) was kept constant for a particular run during the experiment. Pile diameter for every case was kept same.

The present experimental study focuses on the variation of the clear-water local scour depth (Ds) with time (t) for the case of steady flow. For the experimental setup, a composite foundation model composed of the pile groups was inserted in uniform, steady flows in wide rectangular channel with flat bed composed of uniform, non-ripple forming sand. The effect of time on the process of scouring at the composite foundation was investigated. It was concluded that the scour experiment at composite foundation model was done for six run (i.e. each run of 10 hours) may essentially connected with the essential uncertainties on the clear-water local scour depth.

A mathematical model for the variation of the scour depth around the composite foundation has been developed and from this model analysis, an equation for the prediction of clear-water local scour been developed. A dimensional analysis of scour depth variation with time and other parameters has been made around the composite foundation, and also on the basis of dimensional analysis of different parameters affecting scour depth around composite foundation has been made and scour predictors around the composite foundation has been developed.

After finding the predicted value of the clear-water scour depth and observe value of the clear-water scour depth, a regression analysis graphs have been prepared comparing these two values of clear-water scour depth around the composite foundation model. A satisfactory value of the regression coefficient (r) have been found which satisfies the experimental data and procedure of finding the clear-water scour depth at the composite foundation model in the flood plain area.

Scope for further study

On the behalf of the experimental work done on the composite foundation in flood plain area for the locally available fine sand, there is a lot of aspect for the further studies on this topic. Now a days, due to the drastically increasing population of country, a situation of scarcity of the land for the residential area have been taking place. So due to the scarcity of the land a huge mass of our country has caused the deforestation and gradually migrating towards the bank of the river in search of the residential land. So near the river bank a very high level flood occur and due to this building constructed near the river bank are not safe and damages occur due to the high scouring at the foundation level caused by the high floods. So my study about the prediction of scour depth at composite foundation in flood plain area focuses over the prediction of scour depth at composite foundation in flood plain area. So by properly designing the composite foundation in the flood plain area near the river bank can minimize the damages of the building foundations caused due to the high floods.

REFERENCES/BIBLOGRAPHY

1. Ahmad M. (1935) “Experiments on design and behavior of spurdikes”. Proc. of International Hydraulic Convention, ASCE, New York, NY, pp.145-159.
2. Baker, C.J. (1979). “Laminar horse-shoe vortex” Journal of Fluid Mechanics, Vol.95, Part-2, pp.347-365.
3. Baker, C.J. (1980a). “The turbulent horse-shoe vortex” Journal of wind engineering and Industrial Aerodynamics, No.6, pp.9-23.
4. Baker, C.J. (1980b). “Theoretical approach to prediction of local scour around bridge piers” Journal of Hydraulic Research, IAHR, Vol. 18, No. 1, pp.1-12.
5. Breusers, H.N.C., Nicollet G. and Shen H.W. (1977). “Local scour around cylindrical piers.” Journal of Hydraulic Research, IAHR, Vol. 15, No. 3, pp.211-252.
6. Dargahi B. (1982). “Local scouring around bridge pier- a review of practice and theory.” Bulletin No.114, Hydraulics Laboratory, Research Institute of Technology, Stockholm, Sweden.
7. Dargahi B. (1990). “Controlling mechanism of local scouring.” Journal of Hydraulic Engineering, ASCE, Vol.116, No.10, pp.1197-1214.
8. Dey S. (1997). “Local scour at pier, Part-1&2: A review of developments of research”, International Journal of Sediment Research, Vol-12, No.2, pp.29-55.
9. Devenport, W.J. and Simpson, R.L. (1990). “Time-dependent and time-averaged turbulence structure near the nose of wind body junction.” Journal of Fluid Mechanics, Vol.210, pp.23-55.
10. Keuther C. (1932), “Stream flow pattern around the pier of different horizontal cross-sediment forms and their effect on stream bed.” Die Bauetechnic, Vol.10, No.12, pp.161-170.

11. Kikkawa H., Fukuoka, S. Iwama, N and Soogawa H. (1971). "Study of scouring around bridge pier and its prevention" Proc. Of JSCE, Vol.194, pp.83-90.

12. Laursen, E.M. (1962). "Scour at bridge crossing". Transaction of ASCE, Vol.127, Part-1, pp.166-209.

13. Melville, B.W. (1975). "Local scour at bridge site". Report No.117, University of Auckland, School of Engineering, Auckland, New Zealand.

14. Melville, B.W., and Raudkivi, A.J. (1977) "Flow characteristics in local scour at bridge piers". Journal of Hydraulic Research, IAHR, Vol.15, No.4, pp.373-380.

15. Melville, B.W. and Raudkivi, A.J. (1996). "Effect of foundation geometry on bridge pier scour". Journal of Hydraulic Engineering, ASCE, Vol.122, No.4, pp.203-209.

16. Muzzammil, M. Gupta, A.K., Gangadharaiah T. And Subramanaya K. (1989). "Vorticity characteristics of scouring horse shoe vortex". Proc. 3rd International workshop on Alluvial River Problems, University of Roorkee, Roorkee, India.

17. Nakagawa H. and Suzuki K. (1975). "An application of stochastic model of sediment of local scour around bridge pier". Proc. Of 16th Congress, IAHR, Sao Paulo, Brazil, Vol-2, pp.285-289.

18. Roper A.T., Schneider, V.R. and Shen H.W. (1967). "Analytical approach to local scour", Proc. 12th Congress, IAHR, Fort Collins, Colo., USA, Vol-3, pp.151-161.

19. "Scour around bridge piers in alluvial rivers". Report No.1/87, Ministry of Railways, Govt of India, RDSO, Lucknow, India, 1987.

20. "Scour around bridge piers". Bridges and Floods Report No. RBF-10, Govt. of India, Ministry of Railways, RDSO, Lucknow, July-1992.

21. Shen H.W., Schneider, V.R. and Karaki S. (1996). "Mechanics of local scour" Rep. No. CER-66-HWS-SK22; Colorado State University, Colorado.

22. Shen H.W., Schneider, V.R. and Karaki S. (1969). "Local scour around bridge piers". Journal of the Hydraulics Division, Proc. of ASCE, Vol.95, No.HY6, pp.1919-1940.

Paper Published

Scouring of Composite Foundation with Granular Pile Foundation in Alluvial Floodplain

Shri Ram[1] and S M Ali Jawaid[2]
[1]Associate Professor, [2]Professor
Civil Engineering Department, Madan Mohan Malaviya University of Technology, Gorakhpur, India
src_gkp@rediffmail.com, smaj@rediffmail.com

Abstract

Lowlands are influenced by fluctuating surface water levels like tides, surges, floods, etc. Netherlands, Japan, Bangladesh, India, Thailand, etc. have large tracts of flood plains. At the time of floods, the foundation soil has affirmative possibilities. Foundation pit remains flooded due to high water table round the year thus, it is strenuous to construct conventional shallow foundations. Thus, composite foundation with granular core is being proposed for lowlands based on the concept of skirted granular pile along with well steining by Jawaid and Madhav (2006). The proposed foundation was alike to a short pipe pile apart from that the granular infill is much stronger and harder than the original ground. Due to flooding problem in this region, it was found necessary to study the scouring effect on present pipe pile foundation. In this paper, results of model study of above-mentioned foundation using laboratory flume is documented. It is derived that the scour depth varies with time and scour pile is more economical and reliable in comparison to other piles in floodplain.

Keywords: Pile Foundations, model testing, lowlands and scouring.

1. INTRODUCTION

In many of the developing countries of the world, due to heavy population pressure, human settlements are encroaching on to the flood plains of major rivers which are inundated periodically. Some of the lowlands of the world are reviewed and difficulties encountered in their development with respect to geotechnical aspects are presented elsewhere (Madhav and Miura, 1994). The alluvial rivers such as Ganges, Brahmaputra, Godavari, Ghaghra, Rapti, Gandak etc., flood vast tracts of land almost every year during rainy season. Particularly, the Indo-gangetic plains of India experience large floods regularly and are frequently subjected to flooding damage. The flood in the year 1998 in Gorakhpur division of eastern Uttar Pradesh was the greatest since independence of India. Floods have affected more than 400 villages. The lifeline of Gorakhpur District (highway, railways, electricity, telephone etc.) had been completely disrupted from 23 August 1998 to Sept 15, 1998. A large number of houses were reported to have been damaged mostly due to scouring of the ground leading to foundation failures in floodplain area.

Therefore, besides investigating the flood control and flood prevention, in order to protect the life and property of people, it was felt at that time that there is a need to develop suitable, economical and appropriate foundation system and to transfer it to the population living in floodplain areas. Composite rigid caisson with granular core (Fig. 1) is being proposed as an appropriate foundation for this region. It consists of shallow pipes length, L, (1.5-3.5 m) with a 100-150 mm dia. granular core inside. Using conventional sinking methods, the steining is lowered to anecessary depth. To increase load carrying capacity and stability of the foundation, the soil in the steining is replaced and filled by compacted granular material.

After literature review, it is concluded that there are many methods given by the various investigators regarding depth of scour in clear-water scour and live-bed scour. These reviewed methods performed by various investigators have been done in different conditions by different investigators. In the present work, an experimental study of clear-water scour depth has been done by using the locally available river sand and laboratory flume. Keeping these in view, experiment was planned and conducted in Hydraulics and Water Resources laboratory of the Civil Engineering Department at M.M.M. University of Technology, Gorakhpur for