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SCOURING AROUND CIRCULAR COMPOUND PIER BRIDGE

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Abstract:-

The knowledge of the maximum depth of scour around such structures is essential from the point of view of safety of these structures; excessive scour can undermine the foundations and lead to the failure of the structure. Alluvial streams are sometimes partially obstructed by hydraulic structures such as spurs, bridge piers, abutments, guide banks etc. In some other cases high velocity sheets of water from spillways and sluice gates flow over loose alluvial material. In all these cases the bed level in the vicinity of the structures is lowered as a result of interaction between the high velocity flow and the loose bed and consequent modification in the flow pattern, such local drop in the bed level is known as local scour. Proper design requires that the foundation be taken down to a level lower than the anticipated level of the scour hole. It has been reported that since 1950 over 500 bridges have failed in U.S.A. and majority of failures were due to scour of foundation material. In the Indian context, such data on number of failed bridges due to scour are not easily available. However it is known that scour has been the matter of concern to the railways, and detailed hydrologic and scour studies have been undertaken by RDSO, Lucknow. Three major reasons for such a concern can be mentioned namely-

(i) Inadequate knowledge of scour phenomenon when these bridges were constructed; (ii) these bridges were designed probably for maximum observed flood upto the time of design. However, since flood is a probabilistic phenomenon, floods of higher magnitude of larger return period must have occurred and for which foundation was no designed.

(iii) lastly the size of wagons , trucks and their frequency has significantly increased which has increased the load on the bridge.

Keywords:*Introduction,factors affecting scour, codal provisions and flow structure*

Introduction

The process of scour around bridge elements like piers, abutment sand spur dikes is complex due to three-dimensional flow the flow and sediment transport. A number of investigations have been carried out on this top it mainly with the objective of developing relationships for maximum depth of scour. As a result, large amount of literature is available on the subject of bridge scour and its control. However, only few studies are available so far on the flow field around the bridge elements. The flow pattern within the scour hole around circular uniform bridge piers has been studied through laboratory experimentation amongst to their by Melville and Raudkivi (1977), Deyetal.(1995),Ahmed and Rajaratnam (1998), Graf and Istiarto (2002), Muzzammil and Gangadhariah (2003)and Dey and Raikar (2007). These investigations have mostly focused on scour around such piers which have uniform cross-section (orgeometry) along their height. Previous investigations didn’t payany attention to the effects of foundation geometry on scour Parola etal. (1996). But actual bridge piers are constructed in various types of geometries and many of them can have non-uniform cross sections along their heights (Melville and Raudkivi, 1996). Such piers are termed here as compound piers (ornon-uniform piers).

A circular compound bridge pier may be defined as circular bridge pier resting on larger diameter circular well orcaisson. Such geometries of bridge foundations are mostly used in bridge sub-structures in the Indian-sub continent. Fig.1.1 shows the definition diagram of such a circular compound bridge pier. Here b is the diameter of the pier, b∗ is the diameter of the foundation, his the depth off low and Y isthe depth of the top surface of the well (foundation) below the initial bed level. Thus Yshall attain positive values when the top surface of the well is below the general level of river bed. The investig at orsnamely: Chabert and Engel dinger1956) Tsujimotoet al. (1987), Joneset al. (1992),Fother by and Jones (1993), Melville and Raudkivi (1996), Coleman (2005) and Ashtiani etal. (2010) have demonstrated that foundation geometry significantly affects the scour. All the above studies are carried out with uniform sediments. Thus there is urgent need to investigate effect of sediment non uniformity on scour in compound bridge piers.

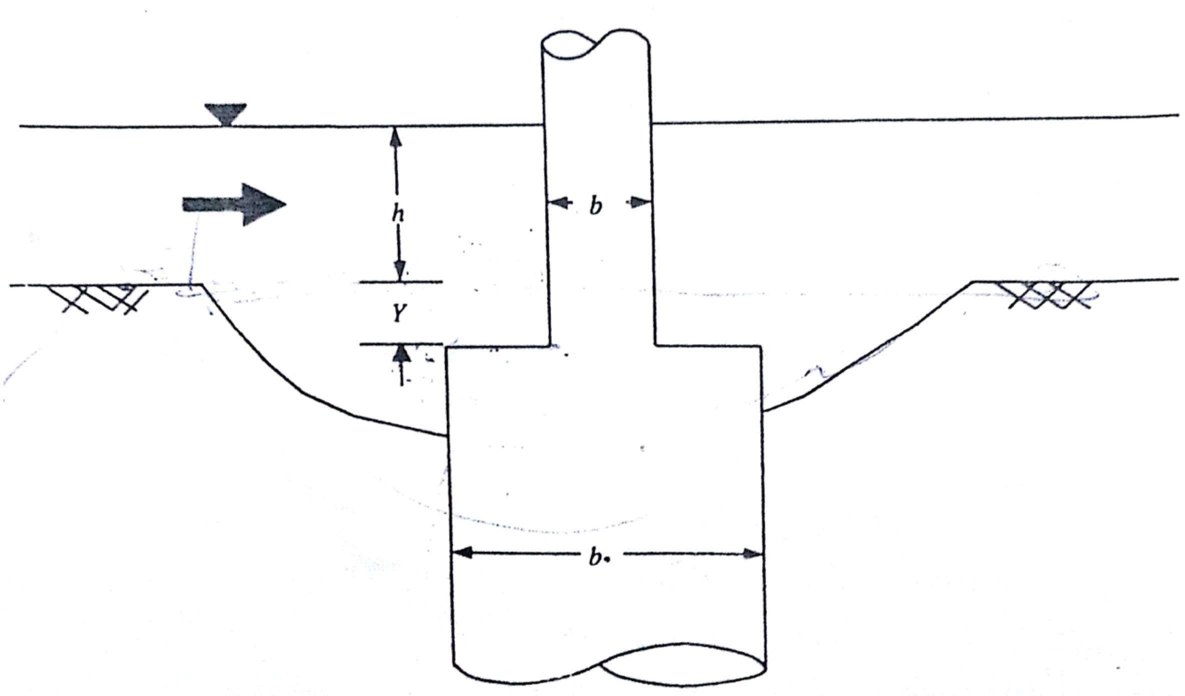

Fig1.1 Definition diagram of Circular Compound Bridge Pier (Kumar et al.2012)

Fig. 1.2 Bridge on river Ganga at Allahabad havingcircular compound bridge pier

As in the case of compound piers, the pier geometry is not uniform long the pier height there fore in comparison to the uniform piers the flow structure around the uniform pier becomes more complicated even in case of circular geometries. Several empirical and semi-empirical relations areavailable for computation of scour depth around uniform and non-uniform piers. But in many cases these ams not yield realistic results. A lack of complete understanding of the flow pattern around the bridge piers is one the problem has remained unsolved even after decades of work. More recently only Beheshti and Ashtiani (2010) and Kumar and Kothyari (2012) have experimentally studied the flow structure around compound bridge piers. How ever low structure around the circular compound bridge piers is only studied by Kumar and Kothyari (2012). The topic of scour around compound piers is studied more recently and this topic has importance in the design of bridge foundations.

Literature review

The mechanics of local scour around bridge piers has been discussed in detail by Shen et al. (1975), Ettema (1980), Melville (1975). The placement of the pier in the stream creates an adverse pressure gradient in the longitudinal direction ahead of the pier and a favourable pressure gradient in the downward direction on its front face. This results in a complex three-dimensional flow system consisting of down flow at the nose of the pier and a horseshoe vortex in front of the pier and extending past it. Additionally, a wake –vortex or Karman vortex system is formed by the rolling up of the shear layers generated at the surface of the pier. Furthermore, a surface roller also forms as shown in Fig 2.1 reproduced from Raudkivi (1990). There are two schools of thought as to the main cause of scour around the pier. Shen et al (1975), Kothyari et al. (1992), and Muzzammil and Gangadhraiah (1995) consider the horseshoe vortex to be the main cause of scour, on the other hand, Melville (1977)and Ettema (1980) consider that scour is caused mainly by the down flow . A combination of analytical and experimental work has, however, revealed that down flow may not be the only agent responsible for scour, and the horseshoe vortex also contributes significantly to the scour.

Factors Affecting Scour

On the basis of review of Literature Garde (2006) the factors affecting scour depth can be summarized as follows.

1. Type of Incoming flow- Clear water flow or Sediment transporting flow, when u_*/u_{*c} is less than unity clear water flow occurs when it is greater than unity sediment transporting flow occurs. Here $u_* = \sqrt{\tau_0 / \rho_f}$ where τ_0 average shear stress on the bed and ρ_f is mass density of fluid is average shear velocity in the channel and u_{*c} is its value when the bed material just starts moving. Other conditions remaining same clear water scour is 10% more than sediment transporting flows.

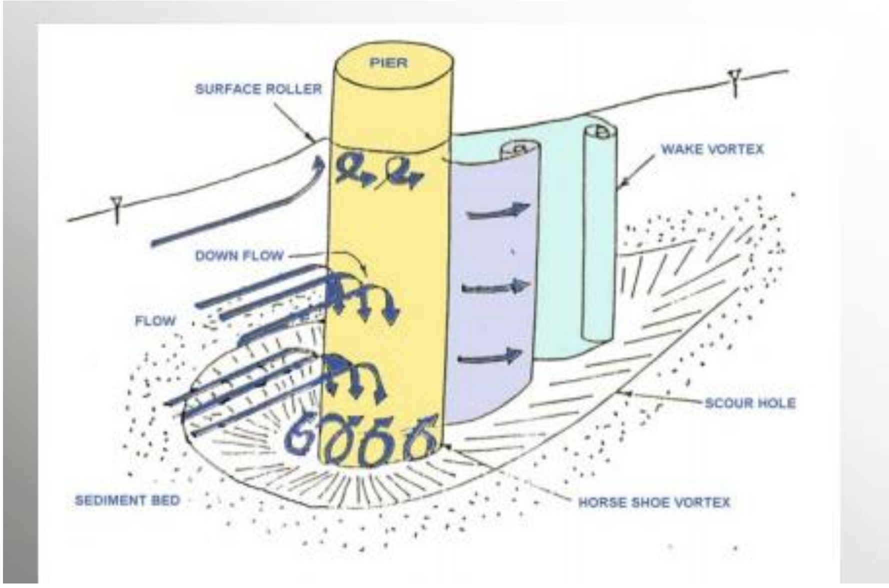

Fig 2.1 Flow Structure at a Bridge pier (Raudkivi 1990)

2. Depth of Flow- Melville and Sutherland (1988) have shown that when depth of flow to pier width ratio D/b is greater than 2.6 scour depth does not depend on depth of flow; however for small depth ,depth of flow affects scour depth.

3. Effect of shape of pier nose: The shape of the pier nose affects the strength of horse vortex as well as separation around the bridge pier; hence it affects the scour depth. This effect is quantified by the coefficient Ks which is defined as the ratio of scour around the pier of given shape to that around cylindrical pier under identical conditions.

4. Angle of Inclination of Pier with Flow: When the pier axis makes an angle with the general direction of flow, two major changes take place in the flow field. First is that the separation Pattern is drastically changed except in the case of cylindrical pier. Secondly the open width between piers, perpendicular to the flow direction is reduced as the angle of inclination is increased. This effect is incorporated by introducing a coefficient K_θ for non-circular piers which is defined as the ratio of scour around the bridge pier at a given angle of inclination that at 0^0 angle of inclination under identical conditions.

5. Opening Ratio: The opening ratio α is defined as α = (B-b)/B where B is centre to centre Spacing of piers and b is pier diameter, or width. When α is close to unity i. e. spacing between piers is large, the flow around one pier does not affect that around the other. However as α decrease the interference effect becomes more pronounced. Analysis of extensive data by Garde et al. (1987) has shown that (D_{sc} or D_{se})/D ~ $\alpha^{-0.3}$. Here D_{sc} or D_{se} are scour depths measured below water surface for clear water and sediment transporting conditions respectively.

6. Bed Material Characteristics: Scour depth is affected by relative density of sediment its median size and geometric standard deviation. For all field problems, relative density of sediment can be taken as 2.65. According to Lacey's approach $D_{se} \sim d^{-1/6}$ where d is the sediment size. The size distribution affects scour depth more significantly. When standard deviation is large and bed material contains more non moving sizes of sediments, the coarser particles tend to accumulate in the scour hole and form an armour layer which tend to inhibit further scour.

Codal Provisions

As regards scour estimation around bridge piers, two codal provisions are available in India, both of which depend on Lacey-Inglis method with only a slight variation. Indian Railway Standard (IRS - 1985) and IRC (1999, 2000) stipulate that in case where, due to constriction of waterway, the width is less than Lacey's width, or where it is narrow and deep as in the case of incised rivers, Lacey depth be calculated using the equation,

$$D_{Lq} = 1.338\left(\frac{q^2}{f}\right)^{\frac{1}{3}} \qquad (2.1)$$

In using Eq. (2.1) for computing D_{Lq}(Laceys depth based on equation using q) one must compute q as Q divided by effective linear waterway, and f is Laceys silt factor. Then

$$D_{se} = 2D_{Lq} \qquad (2.2)$$

Where D_{se} – Equilibrium scour depth below water surface.

The Indian Road Congress Standard Specifications and Code of Practice for Road Bridges (1999, 2000) recommend use ofEq. (2.1) for computingD_{Lq} and then $D_{se} = 2D_{Lq}$ (Further, both codal provisions give guidelines for estimating Lacey's silt factor f.

As indicated by Chitale (1988) for Inglis data on 17 bridges, if D_{Lq} is computed and then compared with the observed D_{se} the magnitude of K varies from 1.23 to 8.39 with a mean of 3.78. Hence, it would be wrong to take K=2.0. In fact there is a strong need for reviewing codal provision for estimation of scour in order to make more rational recommendations which are based on results of recent investigations.

Lacey-Inglis method is meant for non-cohesive sandy material with mean sediment size of about 0. 15 mm to 0.43 mm. In this size range the geometric standard deviation of the bed material would vary between 1.4 to 1.8 orso. The method should not be used outside this range. In the case of coarser material which has larger standard deviation, as scour progresses armouring may occur by selective removal of finer material from scour hole and smaller scour depth will occur. For finer material, having cohesion, there will be greater resistance to scour and hence reduced scour depth will result. Both these effects are not considered in Lacey-Inglis method.

As pointed out by Chitale (1988), LaceyInglis method is for sandy rivers of meandering type, and should be used only in such cases. Further, it is known that in the case of rivers, scour at the bridge pier can be due to three reasons, these are:

(i) Pier scour because of modification of the flow due to presence of pier.

(ii) Scour due to contraction when channel width is reduced at the bridge site by road embankment and guide bunds; and

(iii) Scour due to non-uniform distribution of flow in the bridge waterway, due to presence of bend, non-uniform cross section, and obstructions.

Flow structure around a Compound Pier

A detailed understanding of the flow modified due to presence of pier is essential to model the phenomenon of scour around the piers. As stated earlier, a number of studies were reported before on the flow pattern around uniform piers in the presence or absence of a scour hole. Relatively fewer investigations have been carried out on the flow structure around compound bridge piers (Beheshti and Ashtiani, 2010; Kumar and Kothyari, 2012).Beheshti and Ashtiani (2010) conducted experiments in a 15.0 m long, 1.254 m wide and 0.9 m deep flume to study the three-dimensional turbulent flow field around a complex pier foundation placed on a rough fixed bed. The complex pier foundation consisted of a rectangular column at top, a rectangular pile cap in the middle portion, and a 2 x 4 circular pile group at the bottom. The entire pier foundation was exposed to the approaching flow. The measurement of time-averaged velocity components, turbulent intensity components at specified points around the complex pier placed on fixed rough sand bed were measured by an acoustic Doppler velocity meter (ADV) while the pier was in position. The profiles and contours of time averaged velocity components, turbulent kinetic energy, and Reynolds stresses, as well as velocity vectors at different vertical and horizontal planes were mapped and analyzed. The characteristics of flow pattern around the complex pier were discussed with reference to the geometry of the complex pier foundation. Kumar and Kothyari (2012) conducted experiments on the flow pattern and turbulence characteristics around the circular uniform and the compound piers in the presence of a scour hole using an ADV. The experiments were conducted in a 30.0 m long, 1.0 m wide and 0.3 m deep rectangular channel. Four series of experimental runs were conducted in which the scour hole was allowed to develop before observing for flow field under the clear-water approach flow conditions. One experimental run was conducted around a uniform circular pier of diameter 114 mm for reference, while the other three runs were conducted around the circular compound pier having pier diameter 114 mm and footing diameter 210 mm. In the series with circular compound pier, the top of the foundation level was placed at different elevations with respect to the original bed level of the channel i.e. above the bed level (i.e. $Y = b^*/10$) , at the bed level (i.e. $Y= 0$) and below the bed level (i.e. $Y= b^*/10$) Kumar and Kothyari (2012) summarized their observations on flow pattern around compound piers as below.

(a) Diameter of the principal vortex upstream of the compound pier is 1.11 times as large as that for circular uniform pier when the top surface of the footing is above the general level of the channel bed due to larger exposure of the footing to the flow, whereas it is 0.85 times its size for uniform pier when the top surface of the footing was below the general level of the channel bed which is attributed to the vortex supporting ability of the footing.

(b) The measurements of velocity, turbulence intensities and Reynolds shear stresses made round each of the pier models at different vertical planes exhibit almost similar profiles along the flow depth. However the measurements close to the pier revealed that a significant change occurs in the vertical profile of the flow parameters when the position of the top surface of the footing varied with respect to general level of the channel bed.

Temporal variation of scour around circular compound piers

Only a few studies related to temporal variation of scour around circular compound bridge piers are available so far (Melville and Raudkivi, 1996; Kumar et al., 2003; Lu et al.,2011). Melville and Raudkivi (1996) conducted an experimental investigation of local scour at non-uniform circular piers. The experiments were conducted in a 0.44 m wide,0.38 m deep and 11.8 m long laboratory flume. Eight series of experiments were conducted. For each series, b and b* were held constant while the value of Y was systematically changed. Each experiment was run until the rate of development of the scour hole became effectively zero i.e., until the equilibrium scour depth had been attained. This condition was attained over a period of 24 h. All experiments were run at the threshold condition for the sediment motion on the approach channel bed. On the basis of results obtained, Melville and Raudkivi (1996) defined three zones for scour, namely: zone1; where the foundation is below the bottom of the scour hole and does not affect the process of scour, zone 2; where the top of the foundation is within the scour hole and reduces the scour depth, and zone 3; where the top of the foundation is above the bed level and increases the scour depth compared to that of a uniform pier. They concluded that for $Y/b \geq 2.4$ (zone1), the scour depth is the same as that at a uniform circular pier of the same size and is independent of foundation size since the top of the footing was always below the maximum scour depth. For $2.4 \geq Y/b \geq 0$ (zone 2), the scour depth is less than that at the upper uniform circular pier, while for $Y/b \leq 0$ (zone the scour is more than that at the upper uniform circular pier. In zones 2 and 3, the scour depth ratio depended on b/b* and Y/b.

METHODS TO ESTIMATE SCOUR

Many researchers had suggested different methodologies to estimate scour, some of these equations which commonly used are listed below. Methods to Estimate scour are classified into two categories as i) Clear Water Scour and ii) Live bed scour.

Clear –Water scour around bridge piers:

Based on Roper’s work, Shen et al (1975) contend that the strength of the horseshoe vortex system is a function of pier Reynolds number $\frac{Ub}{v}$, where b is the pier width. Therefore, they argue that the scour depth d_{se} also will depend on $\frac{Ub}{v}$. Based on the available data for bridge piers ranging from 50 mm to 152 mm in width (or diameter) and sediment size from 0.17 mm to 0.68 mm, they proposed the envelope equation

$$d_{se} = 2.23 \times 10^{-4} \left(\frac{Ub}{v}\right)^{0.619} \qquad 3.1$$

Here d_{se} is expressed in m. Assuming the horseshoe vortex to be responsible for the scour around blunt-nosed piers, Baker (1980) has shown that the scour depth for clear-water flow is given by

$$\frac{d_{se}}{b} = (a_1 N - a_2)\tanh\left(a_3 \frac{D}{b}\right) \qquad 3.2$$

In which $N = \frac{U}{\sqrt{\frac{\Delta\gamma_s d}{\rho_f}}}$and a_1, a_2, $a_3$are functions of $\frac{\Delta\gamma_s d^3}{\rho_f v^2}$. The analysis of data collected by Baker yielded $a_1 = 1.0$, $a_2 = 1.3$ to 2.2 and $a_3 = 1$. Support to this form of equation was given by Breussers et al. (1977) who proposed the equation

$$\frac{d_{se}}{b} = f_1\left(\frac{U}{U_c}\right)\left[2\tanh(\frac{D}{b})\right] f_2\left(\alpha, \frac{l}{b}\right) \qquad 3.3$$

Where

$$f_1\left(\frac{U}{U_c}\right) = 0 \text{ for} \frac{U}{U_c} < 0.5$$

$$f_1\left(\frac{U}{U_c}\right) = \left[2\frac{U}{U_c} - 1\right] \text{for } 0.5 < \frac{U}{U_c} < 1$$

and$f_1 \frac{U}{U_c} = 1$ for $\frac{U}{U_c} > 1.0$ i.e. sediment transporting flows. In view of the fact that clear water scour is slightly greater than scour in sediment transporting channels, Eq. (3.1) can be considered as an envelope.

Hancu (1971) and several other investigators have found that there is negligible Scour if $\frac{u_*}{u_{*c}}$or$\frac{U}{U_c}$ is less than 0.50 and if the contraction due to the bridge pier is not appreciable. This implies that the bed shear (note that $\tau_0 = \rho u^2$) in the vicinity of the pier is about four times the shear stress in the approach flow as was confirmed by measurements . If $\frac{u_*}{u_{*c}}$or$\frac{U}{U_c}$ lies between 0.50 and 1.0, there is little sediment transport in the channel and the scour can be termed clear – water scour. In his case Hancu found that there is almost a linear relationship between the scour depth and $\frac{u_*}{u_{*c}}$or$\frac{U}{U_c}$. At $\frac{u_*}{u_{*c}}$equal to unity the scour was found to be maximum. For $\frac{u_*}{u_{*c}}$ Values greater than unity the scour was found to be nearly independent of the shear stress or the mean velocity; other conditions remaining the same, the scour is about 10% smaller than the maximum scour at such $\frac{u_*}{u_{*c}}$values.

By analysis of a large volume of the laboratory data Jain (1981) proposed the following enveloping equations for the maximum depth of scour in clear – water flows

$$\frac{d_{se}}{b} = 1.84\left(\frac{D}{b}\right)^{0.3} (F_{rc})^{0.25} \qquad 3.4$$

Where F_{rc} is the Froude number of the flow corresponding to the critical velocity for incipient motion of the given sediment and the actual depth of flow.

Kothyari et al. (1992) propose the following equation for the maximum depth of scour around circular piers in clear water flows.

$$\frac{d_{se}}{d} = 0.66\left(\frac{b}{d}\right)^{0.75}\left(\frac{D}{d}\right)^{0.16} \propto^{-0.30} \left\{\frac{U^2 - U_c^2}{\left(\frac{\Delta\gamma_s}{\rho_f}\right)d}\right\}^{0.4} \qquad 3.5$$

In which $\alpha = \frac{B-b}{b}$

$$\frac{U_c^2}{\left(\frac{\Delta\gamma_s}{\rho_f}\right)d} = 1.2\left(\frac{b}{d}\right)^{-0.11}\left(\frac{D}{d}\right)^{0.16} \qquad 3.6$$

Here U_c is the average velocity in the channel at which scour around the pier is initiated. Equation (3.5) is obtained b regression analysis as the mean through a large volume of data and if the constant in this equation is changed to 1.0, the equation becomes an enveloping equation.

Objectives

- To carry out experimental investigations to decide the elevation of top surface of well of circular compound pier below general bed level for minimum scour condition.
- To study the scouring effect around circular compound pier for non uniform sediments under clear water conditions.
- To study the scaling effect of pier to foundation width ratio on scouring.

EXPERIMENTATION

The experiments were conducted in a tilting flume 10.0 m long, 0.3 m wide and 0.5 m deep located in the Hydraulics laboratory of Bharati Vidyapeeth Deemed University College of Engineering, Pune, India. The flume has working section 0.7m long, 0.3m wide and 0.1m deep was located 4.5m downstream of flume entrance. The working section was filled with desired sediment to the level of flume bed. Tilting flume is shown in Fig 3.1The discharge in the flume was measured with the help of sharp crested calibrated weir placed in the return channel. The flow depth in the flume was measured with the help of vernier pointer gauge.

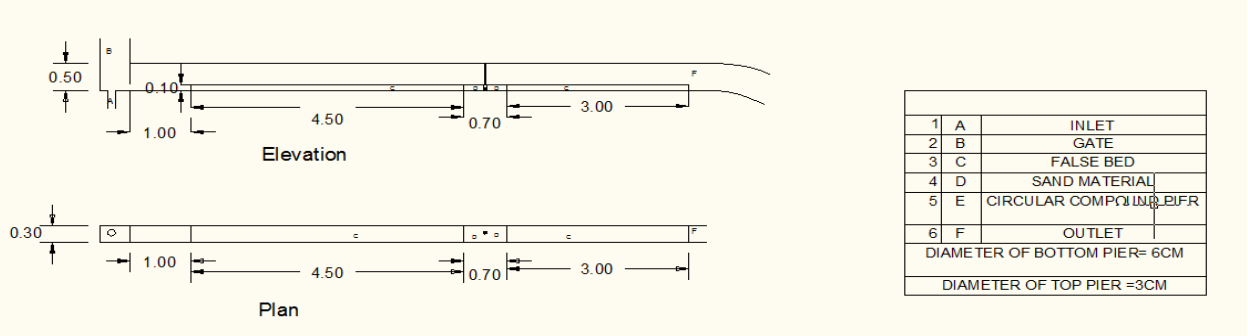

Fig 3.1 Elevation and plan of Tilting flume used for experimentation

Sediment Sample

Non cohesive river bed sand is used as sediment in the experiment. Two samples of sediments are prepared having d_{50} size of 1 mm and standard deviation (σ_g) of 1.14 and 1.68.Initially grain size distribution curve to get required σ_g was drawn. Percentage finer differences between two consecutive sieve sizes were obtained from grain size distribution curve. That percentage multiplied by total weight of sample required is computed. The procedure is repeated for all consecutive sieve sizes. Similar analysis is carried out for non uniform sediments.

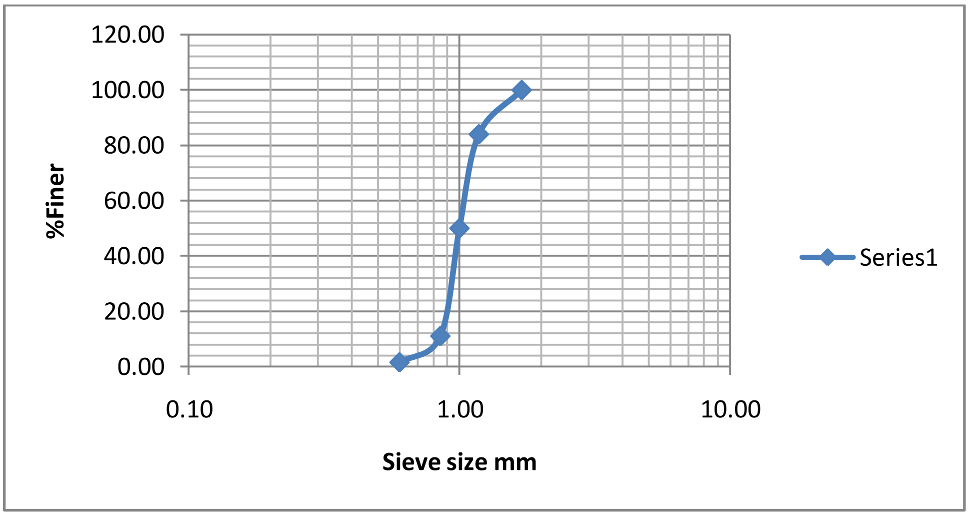

Fig 3.3 Grain size Distribution curve for Uniform sediments.
(σ_g=1.14 , d_{50} =1 mm)

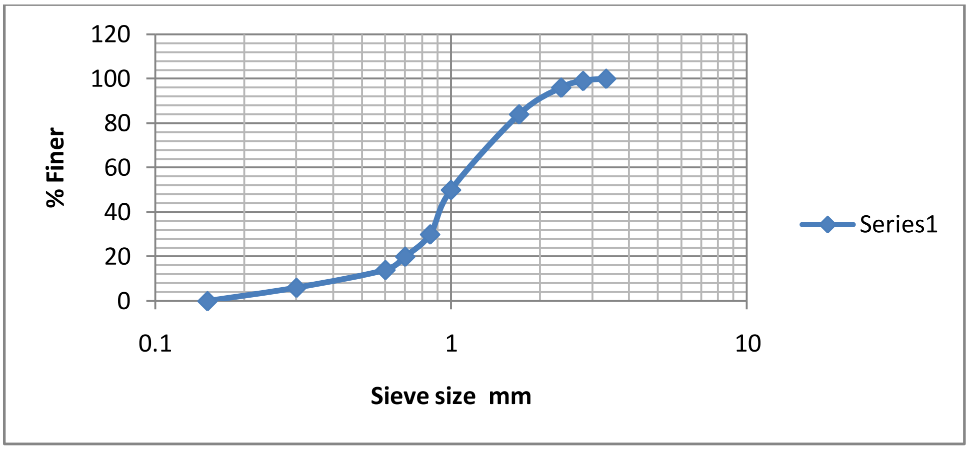

Fig 3.4 Grain size Distribution curve for Non Uniform sediments.
(σ_g=1.68 , d_{50} =1 mm)

Table 3.1 Experimental Data for Uniform sediments.

Sr.No.	Pier diameter b mm	Footing diameter b_* mm	Footing elevation w.r.t. Bed level .Y mm	Flow depth h mm	Velocity of approach flow V m/s	Critical velocity for sediment motion V_c m/s	Median grain size d_{50} mm	Geometric standard deviationσ_g
1	30	60	-10	84	0.32	0.378	1	1.14
2	30	60	0	84	0.32	0.378	1	1.14
3	30	60	10	84	0.32	0.378	1	1.14

Inthis V is the velocity of approach flow computed from known discharge and depth of approach flow for single discharge value. Using Shields chart for threshold condition of uniform sediments in water (Melville and Sutherland 1988) the shear velocity u_{*c} corresponding to d_{50} is obtained .The shear velocity u_{*c} is converted to mean shear velocity V_c by the logarithmic form of velocity profile given in equation 3.1

$$\frac{V_c}{u_{*c}} = 5.75 \log\left(5.53 \frac{h}{d_{50}}\right) \qquad (3.1)$$

Similarly required data is obtained for non uniform sediments and given below in table 3.2

Table 3.2 Experimental Data for Non Uniform sediments.

Sr.No.	Pier diameter b mm	Footing diameter b_* mm	Footing elevation w.r.t. Bed level .Y mm	Flow depth h mm	Velocity of approach flow V m/s	Critical velocity for sediment motion Va m/s	Median grain size d_{50} mm	Geometric standard deviation$_g$
1	30	60	-10	78	0.36	0.397	1	1.68
2	30	60	0	78	0.36	0.397	1	1.68
3	30	60	10	78	0.36	0.397	1	1.68

The method to determine V_a is given in Melville (1997) .Thus Va =0.8 V_{ca}
V_{ca} can be determined from logarithmic form of velocity profile

$$\frac{V_{ca}}{u_{*ca}} = 5.75 \log\left(5.53 \frac{h}{d_{50a}}\right) \qquad (3.2)$$

In this u_{*ca} is critical shear velocity for d_{50a} size and d_{50a} is median armour size. Shear velocity is determined from Shields diagram for respective sizes.
The particle size d_{50a} is found using the Expression as given by Chin(1985)

$$d_{50a} = d_{max}/1.8$$

(3.3)
d_{max} is the maximum particle size determined from particle size distribution.
Using these experimental procedures , observations were taken and recorded for further analysis and given in next chapter.

EXPERIMENTAL RESULTS AND DISCUSSIONS

Table 4.1 and Table 4.2 shows the results of temporal variation of scour depth in uniform and non uniform sediments respectively for three different conditions of footing top level. Scour depth measurements were carried out with help of vernier pointer gauge for three hours duration. Similarly Fig 4.1 and Fig 4.2 shows the graphical representation of these observations.

Table 4.1 Temporal variation of scour depth in uniform sediments

Time in Minutes	Scour depth in mm		
	Footing top 1 cm above bed level	Footing top at bed level	Footing top 1 cm below bed level
0	0	0	0
5	15	2.4	0
10	21.3	15.3	0
15	26.5	15.3	5.2
20	30.1	18.5	5.2
30	35.3	19	8.3
40	40.8	21.4	8.9
50	48.3	22.6	11.5
60	51.2	23.8	16.8
80	52.4	25.4	24.1
100	52.4	31	28.3
120	53	36.2	31.6
140	53.2	43.4	31.8
160	53.2	43.4	32.0
180	53.4	43.6	32.0

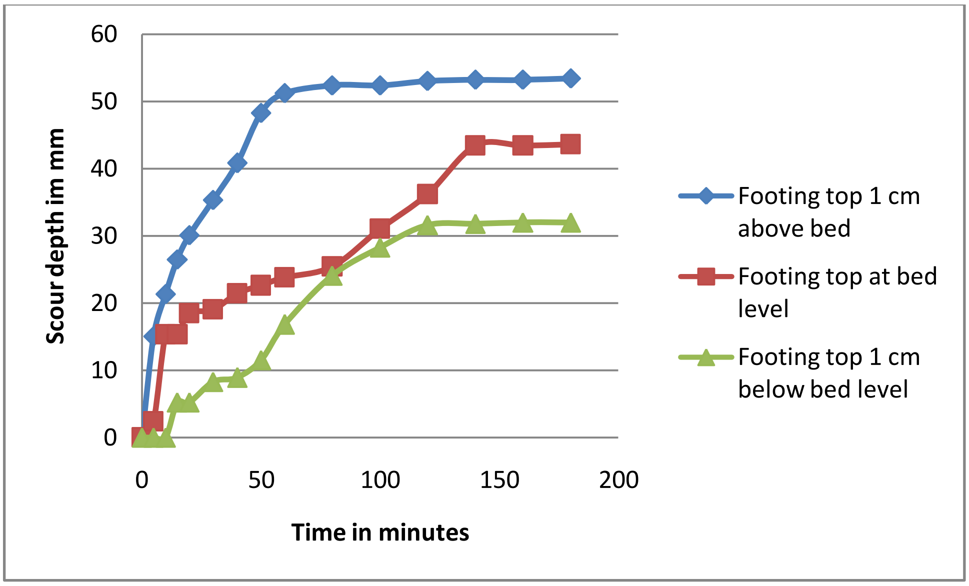

Fig 4.1 Temporal variation of scour depth in uniform sediments

Table 4.2 Temporal variation of scour depth in non uniform sediments

Time in Minutes	Scour depth in mm		
	Footing top 1 cm above bed level	Footing top at bed level	Footing top 1 cm below bed level
0	0	0	0
5	1.7	0	0
10	2.5	1.4	0
15	4.5	2.6	0.5
20	8.6	5.4	1.8
30	12.3	6.2	1.8
40	15.9	8.1	2.4
50	16.4	12.4	4.5
60	19..2	14.4	6.8
80	22.5	18.3	8.6
100	26.2	19.1	10.5
120	28.4	22.4	12.2
140	30.7	25.9	13.8
160	31.4	25.9	14.1
180	32.1	26.2	15.3

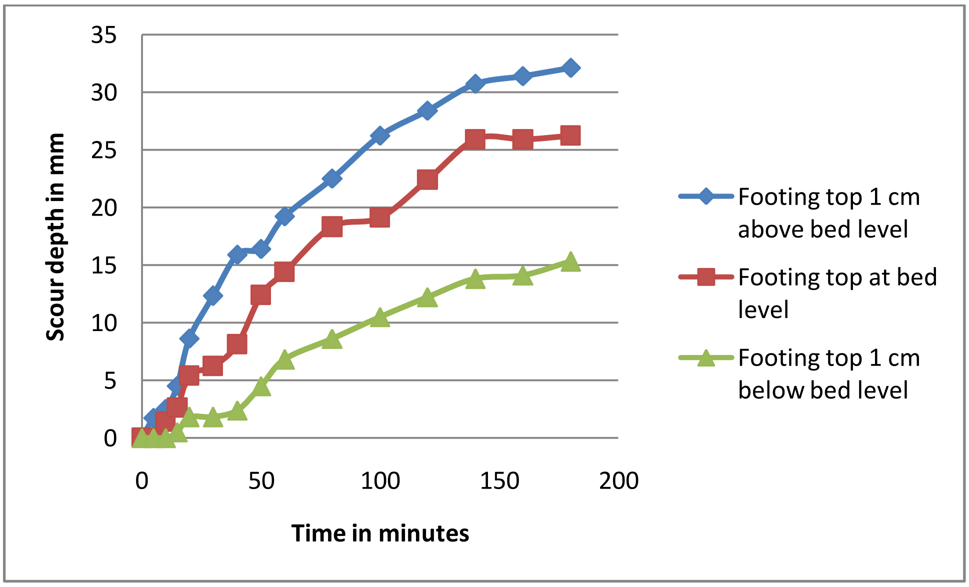

Fig 4.2 Temporal variation of scour depth in non uniform sediments

RESULTS AND DISCUSSIONS

The temporal variation of scour depth around the circular compound piers was measured for the b* / b ratio as 2. The position of the top surface of the footing was systematically varied and the same was placed, respectively, 1 cm above the channel bed level, at the bed level, and 1cm below the channel bed level. The following features of the scour process were noticed. The depth of the scour is greater around the compound pier when the top of the footing is placed above the general bed level of the channel. The extent of scoured area is also larger in this condition because of the exposure of the larger height of footing to the flow. The deepest scour in this case occurred at the upstream nose of the pier/footing.

When the top of the footing is placed at the level of the channel bed, the depth and extent of scour is reduced as compared to the earlier condition as the obstruction to flow was reduced.

When the top surface of the footing is placed below the level of the channel bed, a reduction in scour depth was observed . The reduction of the maximum depth of scour is attributed to the reason that in this condition the principle vortex rests on the projected top surface of the footing, which is rigid and this will reduce further scouring. This characteristic has been termed as the vortex supporting ability of the footing (Kumar and Kothyari 2012).

CONCLUSIONS

1 For uniform sediments when footing top was above bed level for the duration of 50 minutes linear variation of scour was observed after words it was almost constant. For this case maximum scour was observed because of exposure of larger height of footing to flow.

2 For the second case of uniform sediments when footing top was placed at bed level there was reduction in scour depth as obstruction to flow was reduced.

3 For the third case when footing top was placed below bed level further reduction was observed in the scour depth. The reduction of the maximum depth of scour is attributed to the reason that in this condition the principle vortex rests on the projected top surface of the footing, which is rigid. This characteristic has been termed as the vortex supporting ability of the footing.

4 Similar pattern was observed in Non uniform sediments for the three cases mentioned above but there is overall reduction on scour depth as compared to uniform sediments due to armouring effect of non uniform sediments.

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