

# Development of Scour Near Bridge Piers and Abutments

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**SUMMARY** A method is presented for predicting the depth of scour which may occur in the vicinity of bridge piers and abutments. Both the clear water and equilibrium conditions are capable of interpretation by the method, which represents a marked improvement on a number of methods currently in use. Experimental justification is provided.

## 1 INTRODUCTION

In order to design the foundations for a bridge over an alluvial stream it is necessary to know or be able to predict the elevation of the bed in the vicinity of the piers and abutments. Usually the design elevation corresponds to that caused by the design flood with suitable allowance being made for scour around the proposed structure.

The sequence of events as the velocity past an obstacle such as a bridge pier is gradually increased, while maintaining all other factors such as depth, etc. constant, is well documented (Chabert and Engeldinger (1956), Shen 1971). No scour occurs until a critical velocity is reached; then scour increases until the threshold velocity in the main channel is attained; thereafter the bed is "live" and equilibrium scour prevails. Such are the qualitative aspects. However, in common with many other sedimentary phenomena, the quantitative prediction leaves much to be desired.

The first author of this paper has previously proposed a means of predicting progressive scour near channel constrictions (Field, 1969, 1971). This paper presents experimental evidence in support of the method; with most predictions of scour depth being within 20% of observed values. Although being far from completely satisfactory, the method represents a marked improvement on a number of methods currently in use.

## 2 PREDICTIVE METHOD

Owing to the complexity of the scour problem, investigators have reached little agreement on many factors affecting the maximum depth of local scour. Karaki and Haynie (1963) compiled a list of no less than 307 references pertaining either directly or indirectly to the basic understanding of local scour. In spite of the vast amount of data available, little unification of thought has been achieved. Field (1971) has taken the view that so many variables affect the local scour phenomenon that only a macroscopic picture of the scour process can be presented at this time. He further assumed that the scour at an abutment is essentially the same as scour around a pier of the same shape as the abutment and its mirror image in the wall of the channel so that scour at piers and abutments could be examined collectively. Shen, Schneider, and Karaki (1966) had tested this hypothesis and found it to be approximately correct.

Since most streams carry some sediment load, the equilibrium scour depth is defined as that pertaining when rate of sediment supply to the scour hole is equal to the rate of removal. The passage of bed forms through the scour hole causes variations in scour depth so the time average is used to define the equilibrium condition. Based upon the observations of Liu, Chang and Skinner (1961) using vertical board and vertical wall abutment models and Laursen and Toch's (1956) design curve for rectangular piers at zero angles of attack, Field (1971) produced a design chart for the prediction of equilibrium scour with maximum errors of the order of  $\pm 20\%$ . His design chart is reproduced as Figure 1, upon which the variables of the problem are also defined.

It is instructive to replot the variables of Figure 1 in the form of Figure 2. The significance of the dashed curve will be discussed later. If all independent variables except velocity are held constant, Figure 2 may be regarded as showing the variation of depth of scour with increasing velocity. As velocity increases, scour increases until a limiting value is obtained, dependent on the width of obstruction and so on. Increases in velocity thereafter produce no appreciable increase in scour depth. Qualitatively, this is more or less in agreement with the observations of Chabert and Engeldinger (1956) and Shen, Schneider and Karaki (1966).

Clear water scour refers to the condition where no sediment is supplied to the scour hole, and two separate conditions may occur. The first arises when the approach flow is insufficient to carry any sediment load and the second when the area around the scour hole is armoured. Field (1971) has proposed a method for estimating clear water scour depth from the curves for equilibrium scour. In clear water scour, the scouring process continues until the velocity in the scour hole has been reduced to a value which can no longer transport sediment. Therefore, clear water scour may be regarded as being the threshold condition of equilibrium scour. If an imaginary scour profile is considered as in Figure 3, an appropriate interpretation of approach Froude number and depth will allow the use of Figure 2 to predict the scour. An imaginary bed level is defined by depth at which the given discharge must flow in order that particle movement just occurs. If this depth is denoted by  $y_0$ , then

$$y_0 = y_n \frac{v_n}{V} \quad (1)$$



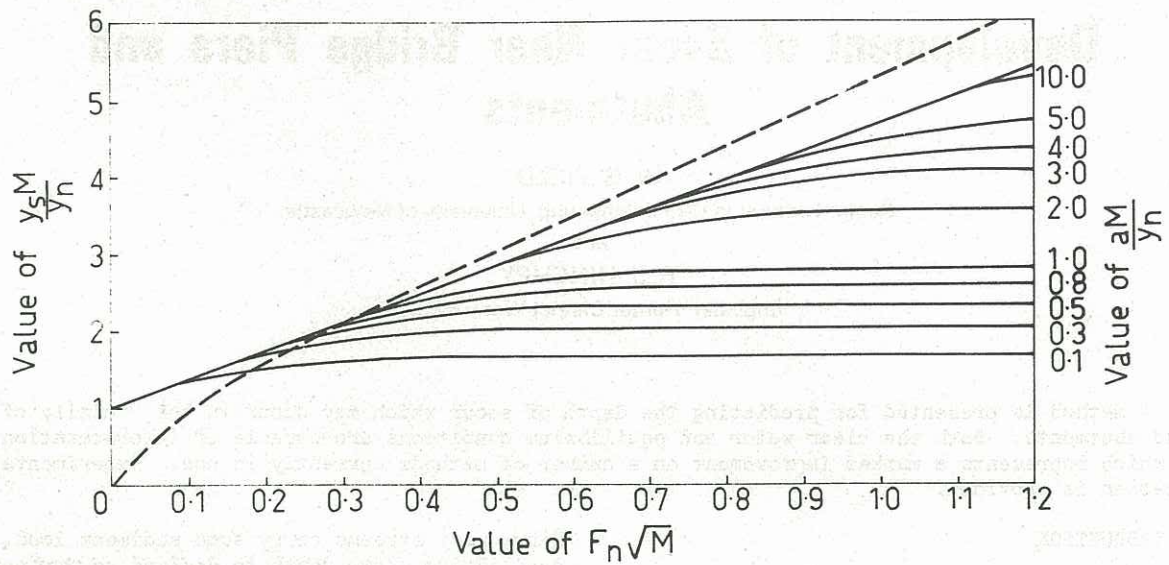


Figure 1 Design Chart for Equilibrium Scour

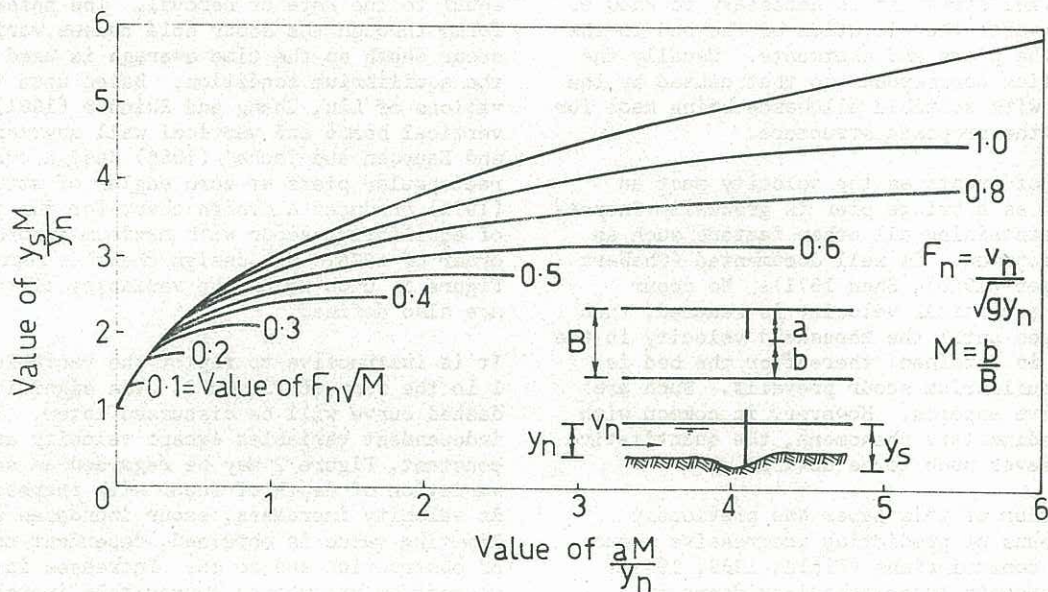


Figure 2 Variation of Scour with Flow Parameters

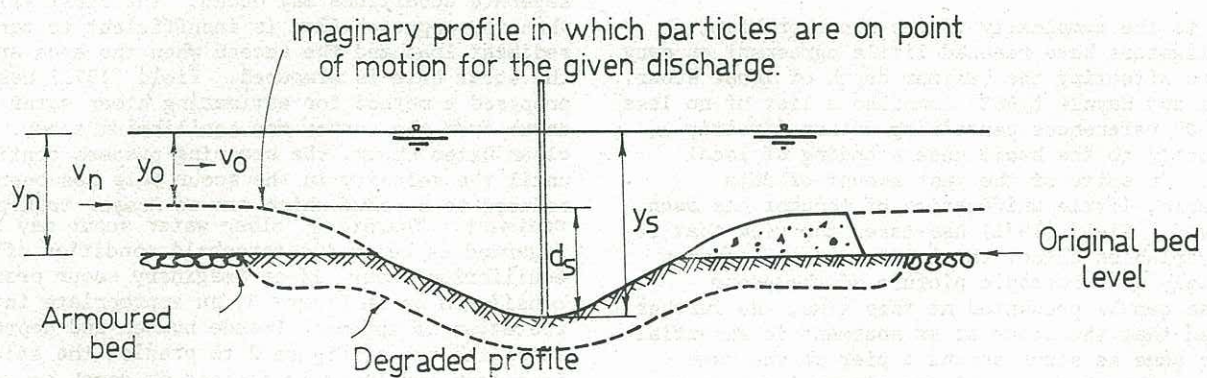


Figure 3 Diagrammatic Representation of Clear Water Scour



wherein  $y_n$  = normal depth of flow in the approach channel;  
 $V_n$  = normal mean velocity of flow in approach channel,  
 $V_o$  = critical mean velocity which just causes particle movement on a level bed.

Similarly 
$$F_o = \frac{V_o}{\sqrt{gy_o}} = F_n \left( \frac{V_o}{V_n} \right)^{3/2} \quad (2)$$

in which  $F_n$  = Froude number of approach flow  

$$= \frac{V_n}{\sqrt{gy_n}}$$

and  $g$  = acceleration due to gravity.

Substitution of  $y_o$  for  $y_n$  and  $F_o$  for  $F_n$  in Figures 1 or 2 should then provide a good estimate of the scour depth  $y_s$ .

Returning now to Figure 2, we examine the sequence of events as the velocity past an obstruction is progressively increased whilst maintaining every other factor constant. We shall arbitrarily assume that the critical velocity which just causes particle movement corresponds to a value of  $F_n \sqrt{M} = 0.3$ , that the value of  $\frac{aM}{y_n}$  is large, and that the scour and flow parameters are to be expressed in terms of the original flow depth. Using the method outlined above, we will find that we will progress along the dashed line commencing at the origin. No scour will actually occur until a value of  $\frac{y_s M}{y_n} = M$ , the opening ratio, is attained. Scour will then vary along the dashed line until the value of  $F_n \sqrt{M} = 0.3$  is reached. At this point, the approach flow is such that the whole bed is on the point of motion. If sediment is now supplied continuously at an adequate rate to the upstream flow, the scour will progress along the full line to its limiting condition. If, however, no sediment is supplied to the flow, degradation will continue along the dashed line. Should the bed be armoured, a limiting condition will develop depending more or less on the proximity of the armour to the deepest part of the scour hole and the angle of repose of the bed material.

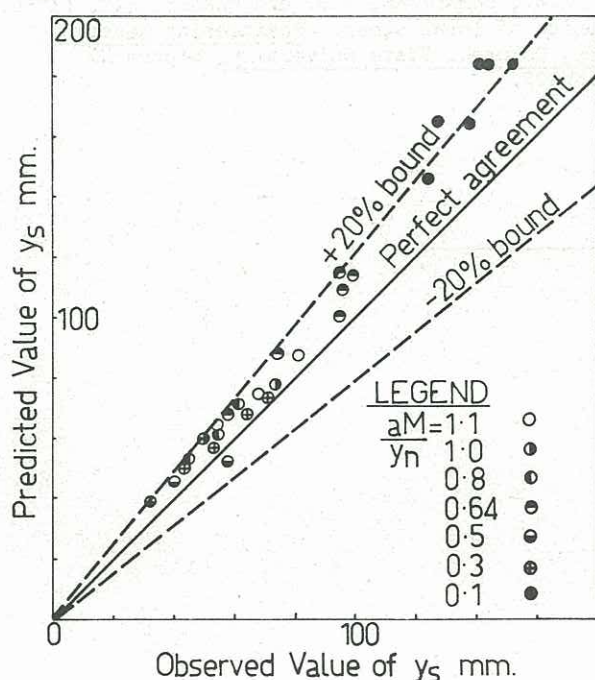


Figure 4 Clear Water Scour Results

### 3 EXPERIMENTAL OBSERVATIONS

The major aim of the experimentation undertaken was to determine whether or not the method outlined above could be used to estimate clear water scour depths within the limits of  $\pm 20\%$  imposed by the equilibrium scour design curves. To this end a glass-sided tilting flume 5m long, 150mm wide was used in which both water and sediment were recirculated. Discharge was measured by a standard venturi meter in the delivery line to the flume inlet.

Sand with a  $d_{50}$  size of 0.37mm, uniformity coefficient  $d_{60}/d_{10}$  of 1.8 and placed to a depth of 150mm in the bottom of the flume was used throughout the experiment.

Half-cylindrical tubing was used to represent the piers, being placed flush against the glass side of the flume. Hence, in effect, the flume represented half a channel, and the tubing, half a pier. Half-widths of pier used in the experiments were  $a = 12.5\text{mm}, 25\text{mm}, 37.5\text{mm}, 50\text{mm}$ . With a range in  $y_n$  from say 25mm to 150mm then the range in  $\frac{aM}{y_n}$  obtainable from the apparatus was from approximately 0.1 to 1.3. Values of  $\frac{aM}{y_n}$  for which tests were conducted were 0.1, 0.3, 0.5, 0.64, 0.8, 1.0 and 1.1. The effect of having a full pier situated in the centre of the flow channel was checked for a value of  $\frac{aM}{y_n} = 0.3$ . No significant difference in scour depths was observed.

The experimental procedure followed was one of assigning a value of  $\frac{aM}{y_n}$ , choosing a pier size  $a$ , and evaluating the required normal flow depth  $y_n$ . Then the velocity past the pier was increased stepwise and the variation of  $\frac{y_s M}{y_n}$  with  $F_n \sqrt{M}$  determined. At any particular velocity  $V_n$ , scour depth measurements were taken over a period of 3 to 6 hours depending on the time taken to fully develop the scour.

### 4 RESULTS

Bearing in mind that the aim of this work was to determine whether or not the method proposed for

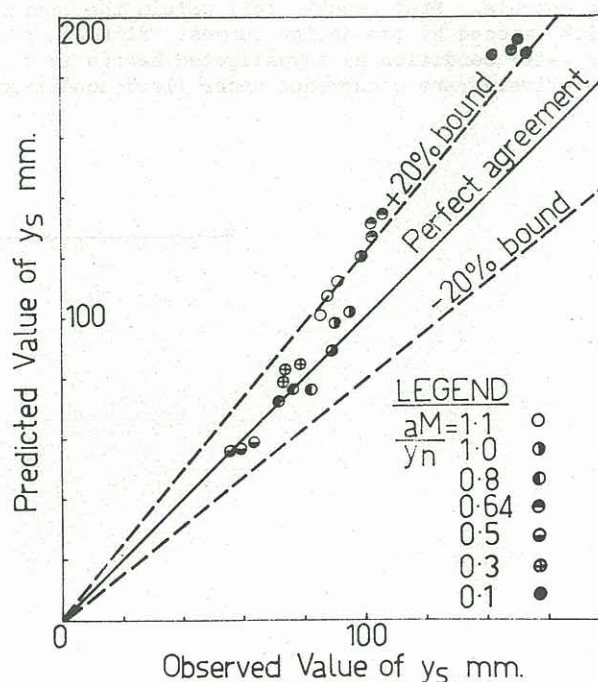


Figure 5 Equilibrium Scour Results



estimating clear water scour from the design curves for equilibrium scour could be applied with an accuracy of  $\pm 20\%$ , the relevant comparison is a plot of  $y_s$  predicted versus  $y_s$  observed. Before the comparison can be made two factors need to be assessed. Firstly, the experimental observations were made on cylindrical piers whereas the design curves are for rectangular or vertical board models. Laursen and Toch (1956) suggest that the depth of scour at a cylindrical pier is only 90% of that of a rectangular pier where the depth of scour  $d_s$  is measured below the original bed level rather than below the original surface level as has been the practice herein. Consequently in predicting the scour depth the value of  $d_s$  was appropriately reduced to account for pier shape. Secondly, observations of the threshold of movement indicated that a value of  $V_0 = 0.23 \text{ ms}^{-1}$  was most appropriate and assumed to apply to all tests.

The comparison for all clear water scour results is shown in Figure 4 while all the equilibrium scour results are plotted in Figure 5. As well, it should be stated that the shapes of all curves corresponded closely with those of Figure 2. It is quite clear that the vast majority of results are contained within the  $\pm 20\%$  limits anticipated. In general, observed values are less than those predicted. This may be the result of unaccounted for factors or simply not allowing sufficient time for equilibrium to be reached in spite of the 3 to 6 hours employed.

The observations were also compared with predictions based on the recommendations of other experimenters. The enveloping curve of Shen, Schneider and Karaki (1969) was found to overestimate the actual observed values of equilibrium scour by amounts varying between about  $-10\%$  and  $+60\%$ ; Laursen and Toch's (1956) design curve overestimated the equilibrium scour results by  $+30\%$  to  $+150\%$ ; Ahmad's (1953) method overestimated the clear water scour results by  $+10\%$  to  $+50\%$ .

## 5 CONCLUSIONS

The results of the experimentation have indicated that the proposed method of predicting clear water scour from the design curves for equilibrium scour is acceptable. Most results fell within the limits of  $\pm 20\%$  imposed by the design curves. Although the clear water condition as investigated herein is a comparatively rare occurrence under flood conditions

in the field, effective armouring adjacent to a scour hole may result from grassed flood plains in which case the method should have application. In addition the method may be important in interpreting model results and their subsequent translation to field predictions, as well as adding to a greater overall understanding of the local scour phenomenon.

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