LOCAL SCOUR AT A BRIDGE PIER

CAUSED BY FLOOD WAVES

A report submitted in partial fulfilment
of the requirements for the degree of
Master of Engineering at the University
of Canterbury, Christchurch, New Zealand.

by

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ABSTRACT

Results of an investigation (Walker (1975)) of scouring processes during the passage of a flood wave past a vertical cylinder are extended. Scour depths are measured visually on the cylinder.

For waves producing clear water scour Walker's observations that:

(i) The scour depth at any time is less than the steady state scour depth corresponding to the wave discharge at that time.

(ii) That scouring continues after the wave peak are confirmed. An empirical method for predicting the final scour depth caused by a clear water flood wave is proposed.

Flood waves in which all flows were capable of causing bed motion showed no effect on the equilibrium scour depth existing before the wave passage.

Long flood waves with initial clear water conditions and peak flows capable of causing bed motion would develop a scour hole depth corresponding to the equilibrium depth for steady flows with upstream bed motion. In such cases scouring can cease before the wave peak arrives.

Recommendations for further research are made.

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CONTENTS

Page

LIST OF FIGURES AND TABLES

TABLE

CHAPTER I

INTRODUCTION

The problem of predicting and thus of adequately allowing for the effect of scour around bridge piers is one of major concern for the bridge designer. By introducing an obstruction, such as a bridge pier, into flow over an alluvial bed the interaction between the flow and the bed may be modified. Local scour is one of the possibilities.

Local scour can be defined as the scour in the vicinity of an obstruction caused by the local flow patterns around the obstruction. Local scour is to be distinguished from constriction scour which is scour caused by a significant constriction of the flow and which can influence a large area of the bed.

There are two types of local scour. Firstly, clear water scour which occurs when there is no general bed movement, and secondly, scour with "continuous sediment transport" or "upstream bed motion" where the scour hole is continuously supplied with sediment from upstream.

Many formulae in use for predicting scour depths have been based on laboratory experiments with steady flows supplemented at times with limited field data. These formulae, which have been reviewed by Melville (1975), do not recognise that in nature there are seldom steady flows but rather, continuously changing flows with most of the scour failures occurring during floods. Laursen and Toch (1956) suggest that their conclusions and results, which were based on steady flow conditions, would not be applicable under flood conditions.

PURPOSE OF INVESTIGATION

The purpose of the experiments described herein was to investigate scour hole formation by translation waves and to compare results with those obtained from steady flows.

CHAPTER II

LITERATURE REVIEW

2.1 INTRODUCTION

This review will concentrate on previous research pertinent to the current investigation. More comprehensive reviews have been given elsewhere, the one by Melville (1975) being of particular relevance to New Zealand conditions. Melville's criterion for selecting satisfactory formulae for New Zealand conditions was that the formula predicted the scour depth within -20% to +50% of the scour depth recorded at failure. However it must be realised that the scour depth at failure may not be the maximum possible under the given flow conditions.

2.2 STEADY FLOW EXPERIMENTS

i. Clear Water Scour

The formula recommended by Melville on the basis of comparison with field data from New Zealand is the equation put forward by Shen, *et al.* (1969):

$$d_s = 0.000223 \quad Re^{0.619} \qquad 2.1$$

where d_s = scour depth measured from the upstream bed level (m).

$$Re = \frac{Vb}{\nu} \qquad 2.2$$

where V = mean approach velocity. (m/s).

b = pier diameter. (m).

ν = kinematic viscosity. (m^2/s)

This equation forms an envelope to most of the available experimental and field data. It thus overestimates the scour depth and also has the disadvantage of not including such variables as grain size and bed slope.

Experimenters have plotted their results in many different ways, e.g. Leclerc (1971) who plotted:

$$\frac{d_s}{b} = f\left(\frac{D_{50}\, V_*}{\nu}\right) \qquad 2.3$$

where D_{50} = geometric mean size of the bed material. (m).

V_* = shear velocity (m/s)

and Nicollet and Ramette (1971) who plotted:

$$\frac{d_s}{b} = f\left(\frac{V\, D_{50}}{\nu}, \quad \frac{b}{D_{50}}\right) \qquad 2.4$$

These two groups of researchers have shown that D_{50} and V_* are important with the results showing distinct trends. No explanation of the trend was offered.

Hancu (1971) stated that:

$$d_s = f\left(\left(\frac{2V}{V_{cr}} - 1\right), \; V_{cr}, \; g, \; b\right) \qquad 2.5$$

where V_{cr} = velocity to cause bed motion. (m/s).

g = acceleration due to gravity (m/s^2).

Hancu found that scour at the pier started when:

$$V = \frac{V_{cr}}{2} \qquad 2.6$$

and that scour depths increased linearly with V in the range $\frac{V_{cr}}{2}$ to V_{cr} so that the term:

$$\frac{2\,V}{V_{cr}} - 1 \qquad 2.7$$

expressed the scouring potential.

Equation 2.5 was modified by Ettema (1976) to:

$$\frac{d_s}{b} = f\left(\frac{2V_* - V_{*c}}{V_{*c}}, \quad V_{*c}, \quad \frac{\sigma}{D_{50}}, \quad \alpha\right) \qquad 2.8$$

where V_{*c} = critical shear velocity to cause bed motion.

α = static angle of grain repose.

σ = inclusive graphic standard deviation of the bed material.

Ettema (1976) investigated the effect of bed material gradation, i.e. changes in σ, and concluded that for bed materials with increasing standard deviation and the same D_{50} value, the scour hole is reduced due to the effect of armouring in the base of the scour hole. Ettema takes this into account with the term:

$$\frac{\sigma}{D_{50}} \qquad 2.9$$

thus including a better description of the bed material in his analysis. The author considers Equation 2.8 to be the best combination of the parameters involved in the situation of clear water scour with steady uniform flows.

ii. <u>Scour With Continuous Sediment Motion</u>

Melville recommends on the basis of comparison with New Zealand field data that the larger of the scour depths predicted by the following equations be used for design purposes:-

i. Laursen II (Laursen 1963)

$$\frac{b}{y} = 5.5 \frac{d_s}{y} \left(\left(\frac{d_s}{r\,y} + 1 \right)^{1.7} - 1 \right) \qquad 2.10$$

where $r = 11.5$,

y = upstream flow depth.

ii. Shen II (2) (Shen <u>et</u> <u>al</u>. 1969)

$$\frac{d_s}{b} = 3.4 \left(Fr_p \right)^{0.67} \qquad 2.11$$

where Fr_p = Pier Froude number

$$= \frac{V}{\sqrt{gb}}$$

The simplest formula in use is Breusers (1965):

$$d_s = 1.4b \qquad 2.12$$

which Breusers (1972) modified to:

$$d_s = (1.3 - 1.5)b \qquad 2.13$$

Breusers however made the reservation that if too large a portion of the sediment load is carried in suspension, then his formula (2.13) may under estimate the depth of scour.

Coleman (1971), using dimensional analysis and plotting scour Euler number, $\frac{V}{\sqrt{2gd_s}}$, versus pier Froude number, $\frac{V}{\sqrt{gb}}$, obtained the predictive formula:

$$d_s = 1.49b^{0.9} \left(\frac{V^2}{2g}\right)^{0.1} \qquad 2.14$$

for circular piers in sand beds.

Equation 2.12 is used in New Zealand for design purposes, but it is possible that if sediment is carried as suspended load that equation 2.14 will predict the scour depth more accurately. This condition implies higher flow velocities and equation 2.14 includes the velocity explicitly.

2.3 TIME DEVELOPMENT OF LOCAL SCOUR (CONSTANT DISCHARGE)

For a given steady flow the time required to reach the final equilibrium scour depth is strongly dependent on the flow conditions. Bonasoundas (1973) suggests, for laboratory work, with clear water scour 72 - 96 hours are required while for scour with continuous motion he suggests that only one hour is necessary. Problems in which the time development of scour becomes important are the design of temporary protective works and the consideration of the effect of flood waves.

Many formulae have been suggested for predicting local scour depths as a function of time for scour near bridge piers, abutments and spur dykes, and for scour downstream of spillways. Although clarifying the time development problem none of these formulae can be used with confidence for design work. Some are only general relations without specific values given for the coefficients (Ahmad (1953), Liu et al.

(1961)) while others, (Shen et al. (1965), Shen et al. (1966)) are well defined relations based on a small number of tests with a limited range of variables.

Shen et al. (1965) studied circular bridge piers in flows with continuous sediment motion and defined graphically a function of the form:

$$\frac{d_s}{d_{sm}} = f\left(\frac{t}{t_m}, \ Fr\right) \qquad 2.15$$

where d_s = scour depth at time t.

d_{sm} = maximum scour depth which occurs at $t = t_m$

Fr = Froude number

$= \frac{V}{\sqrt{g\ y}}$

Shen, et al. (1966) on the basis of new tests expressed the time evolution of scour for clear water conditions as:

$$\frac{d_s}{y} = 2.5\ Fr^{0.4} \left(\frac{b}{y}\right)^{0.6} \left(1 - e^{-mE}\right)^2 \qquad 2.17$$

where $m = 0.026\ e^{2.932y}$ 2.18a

$E = \left(\frac{b}{y}\right)^{0.33} Fr^{0.33} \ln\left(\frac{V\ t}{y}\right)$ 2.18b

where y in equation 2.18a is in feet and t in equation 2.18b is in minutes. With equations 2.17 - 2.18b being so complex there is doubt that they can be generalised to conditions other than those of Shen's tests. It would be unwise to use these equations for practical applic- ations.

Ahmad (1953), obtained the expression, for clear water scour:

$$\frac{h_s}{h_{sm}} = \left(1 - m\ e^{-nt}\right) \qquad 2.19$$

where h_s = scour depth at time t measured from the water surface.

h_{sm} = maximum scour depth.

m, n are coefficients related to the diameter of the bed material.

Liu, et al. (1961) presented an equation of the same form for clear water scour:

$$d_s = d_{sm}\,(1-e^{-c\frac{t}{t_o}}) \qquad 2.20$$

where

$$t_o = e^{-B_o} \qquad 2.21a$$

$$c = e^{A\sqrt{1-\frac{d_s}{d_{sm}}}} \qquad 2.21b$$

where A and B_o depend on the flow characteristics, bed materials, and the geometry of the obstruction. The authors do not define these relationships.

For scour with continuous bed motion Liu et al. (1961) state that:

$$d_s = d_{sm}\,\frac{1-e^{-c\frac{t}{t_o}}}{1-\frac{t}{t_m}} \qquad 2.22$$

where t_m is the time to reach maximum scour depth. For the case of clear water scour Liu considers t_m to equal infinity so that equation 2.22 will revert to equation 2.20.

Breusers (1968) developed a formula for two dimensional local scour:

$$\frac{d_s}{y} = \left(\frac{t}{t_1}\right)^{0.38} \qquad 2.23$$

where $t = t_1$ when $d_s = y$.

Thomas (1975) considers a variety of cases involving local scour and puts forward the expression:

$$d_s = a\,t^{1-b} \qquad 2.24$$

where a, b are constants depending on the situation in which local scour is being studied, but no means of evaluating these constants in a particular situation was presented.

Cunha (1975) developed a general law for the time evolution of local scour near spur dykes:

$$\frac{d_s}{y} = a\, t^b \qquad 2.25$$

For a given spur dyke of length 0.2m Cunha found that:

$$\frac{d_s}{y} = 1.89 \left(\frac{1}{y}\right)^{0.69} \left(\frac{V_*}{W}\right)^{3.25} t^{0.16} \qquad 2.26$$

where W = fall velocity of the mean sediment size in water. (m/s)

Ettema (1976) developed a formula which states that:

$$d_s \operatorname{Cosec} \alpha = K_1 \ln \frac{K_2 D_{50} t}{b} \qquad 2.27$$

where

$$K_1 = f\left(\frac{\sigma}{D_{50}}\right) \qquad 2.28a$$

$$K_2 = f\left(\frac{2 V_* - V_{*c}}{V_{*c}}\right) \qquad 2.28b$$

From the above three types of equation emerge

$$d_s = a\, t^b \qquad 2.29a$$

$$d_s = a \log bt \qquad 2.29b$$

$$\frac{d_s}{d_{sm}} = (1 - e^{-bt}) \qquad 2.29c$$

where the constants a, b are determined by the flow and sediment variables in conjunction with the geometry of the obstruction. The limited quantity of available data and the variability inherent in data from scour experiments combine to prevent a decision as to which of the equation best represents the phenomenon.

2.4 UNSTEADY FLOW EXPERIMENTS

i. Introduction

The unsteady flow experiments to be mentioned involve continuously changing flow properties (i.e. flow rate, velocity, depth) with time as opposed to steady flow experiments (discussed in the previous sections) where the discharge is held constant for the duration of each experiment.

ii. Clear Water Scour

Walker (1975) studied local scour caused by triangular flood hydrographs and drew the following conclusions applicable for symmetrical waves:

i. Scour depth increases after the peak of the flood wave and scouring ceases on the receeding stage of the floodwave when the scouring mechanisms are too weak to remove particles from the scour hole.

ii. The final scour depth is always less than the steady state equilibrium scour depth corresponding to the peak flow of the wave.

Walker also proposed a method for predicting the final scour depth. The calculations require data defining the wave (period, maximum and initial discharge) and the particle size. Only one pier size was tested and no reference is made to the bed slope. The generality of this method has not been investigated.

iii. Scour With Upstream Bed Motion

Walker (1975) also studied the effect of a flood wave where the initial flow caused continuous motion of the bed material. His conclusion was that the passage of the flood wave had no effect on the equilibrium depth of scour.

Bogomolov, Zheleznyakov, Petrov, Altunin, and Proudovsy (1975) experimented with symmetrical flood waves and concluded:

i. "That during the accelerated flow the intensity of scour processes is sharply increasing and after flood crest passing it, is decreasing."

ii. "At the recession of flood partial filling of the hole takes place."

iii. The maximum scour hole depth does not coincide with the velocity maximum but this is the time when the maximum sediment transport occurs.

Bogomolov also gave relationships for the time development of scour during the wave passage.

i. For the rising portion of the flood.

$$\frac{d_s}{d_{sm}} = \cosh\left(A\frac{t}{t_n}\right) - 1 \qquad 2.30a$$

ii. For the flood recession

$$\frac{d_s}{d_{sm}} = \frac{t}{t_n} B^{\left(\frac{t}{t_n} - 1\right)} \qquad 2.30b$$

where d_s = scour depth at time t.

d_{sm} = scour depth at time t_n.

A and B are constants depending on the "characteristics of flood wave as a function of it's duration and the maximum discharge." It is not stated whether equations 2.30a and 2.30b are experimental or theoretical. The main criticism of this work is that neither the wave properties (shapes and magnitudes) nor the relations between A and B and the wave properties are given. It is thus impossible to compare new data with the published curves.

CHAPTER III

EXPERIMENTAL EQUIPMENT AND PROCEDURES

3.1 THE FLUME AND PIER

The flume, which has been described in detail by Hill (1967), consists of a flat steel plate deck 27.5m in length with a working section of 23.7 m and sidewalls which can be adjusted to give widths up to 2.6 m. These are constructed of steel plate except for a 10 m section of 19 mm plate glass 11 m downstream from the inlet.

Slope changes are made by adjusting screwjacks at the midpoint and upper end of the flume, the downstream end being freely hinged. Slope measurements were taken using a Zeiss NI 007 precise automatic level and staff. Readings were taken on both sides of the flume, ensuring that the bed would be level in section, to an accuracy of ± 0.5 mm. The slopes chosen were low to ensure that the constricted section at the pier caused no backwater effects: The slopes were 0.001 (± 3.0%), 0.0015 (± 1.5%), and 0.004 (± 1.0%).

The pier, which has been described in detail by Walker (1975), consisted of P.V.C. cylindrical pipe, 60 mm in outside diameter, and 750 mm high. To enable the scour depth to be recorded periodically at the front and side of the pier, two vertical rows of brass electrodes 4.76 mm in diameter spaced at 10 mm centres were put in the pier. The pier was modified after the first series of runs by engraving lines at 2 mm centres and smoothing them with wax to improve measurement accuracy.

The flume width was chosen as 600 mm to avoid constriction scour. Ministry of Works and Development current design practice states that constriction scour can be ignored if the pier causes a constriction of less than 10%, and Shen <u>et al</u>. (1966) recommend a minimum ratio of pier size to flume width of 1:8 to avoid the effects of contraction and side wall confinement.

3.2 FLOW AND FLOW MEASUREMENT

Flow was taken directly from the laboratory constant head recirculating system. Water is pumped from a storage sump to the constant head towers, which gave a supply head of approximately 10 m. Two supply lines, 15.25 cm and 7.125 cm diameter, capable of supplying a maximum flow of 0.105 m^3/s were used. The flow entered the channel through a coarse screen followed by a series of vertical steel plate flow straighteners. The flow was raised 25 cm from the flume floor to the bed surface by a sloping ramp. (Figure 3.1).

After passing over the test bed the water and any eroded sediment passed over a 2.0 m section of rough concrete and a 2 cm weir, (which was removed after the first series of experiments). The flow was taken to a calibrated pit (approximately 22 m^3) which acted both as a sediment trap and as a flow measuring device.

Several determinations were made during each steady state run thus calculating the flow rate to within 1%.

During these runs the supply line valves were calibrated so that the valves could be set at given positions with the resulting flow known exactly. Hence translation waves with known discharge versus time characteristics could be generated using a valve setting versus time curve. All waves had triangular shaped discharge/time curves with the same rates of rise and fall.

3.3 DEPTH MEASUREMENT

There were two independent systems for measuring flow depth. Firstly two water level gauges were used providing a record on a Hewlett-

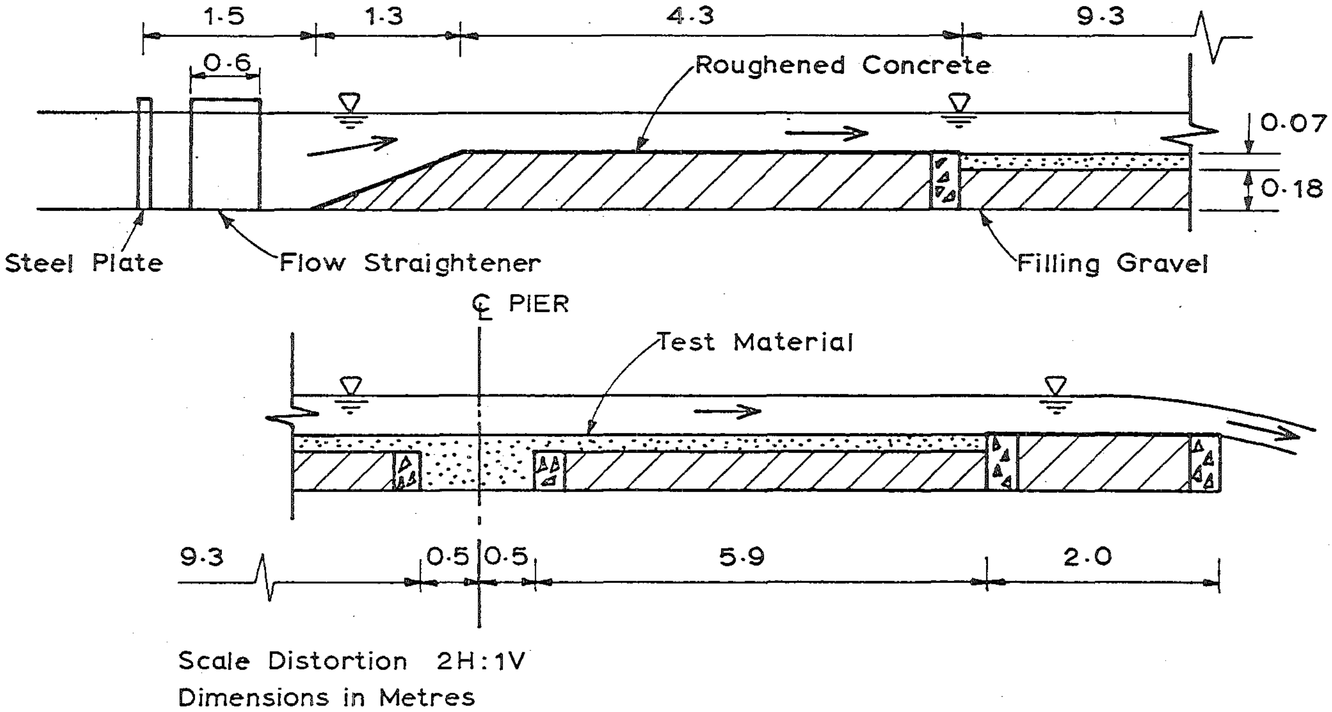


Fig. 3·1 FLUME SECTION SHOWING PIER POSITION AND PLACEMENT OF MATERIAL

Packard chart recorder. One gauge was placed 50 cm upstream of the pier and the other 10 m upstream. Each gauge consisted of two stainless steel rods, 3.175 mm diameter and 455 mm long, at a spacing of 19.0 mm. Each rod was connected in an electrical circuit which changed in impedance as the water level between the two adjacent rods changed. The charts recorder recorded these changes giving a continuous record of the water surface level to within ± 10 mm.

The second method was to measure the bed and water surface elevations with plate gauges at three stations upstream of the pier and two stations downstream of the pier. These gauges were point gauges modified by fixing a 3 cm square flat plate onto the tip. The accuracy of measurement, which was affected by water turbulence, was ± 2.0 mm. For low flows, which have less turbulence, it was possible to determine the depth within ± 1.0 mm.

3.4 SCOUR HOLE MEASUREMENT

The basic requirements for scour measurement were that the scour depth could be measured at any desired time and that an accurate recording be made. This was achieved by monitoring the impedance between adjacent pairs of electrodes mounted in the pier surface. There is a marked change in impedance between the states of water (electrodes uncovered) and saturated bed material (electrode covered) which by means of digital circuitry, (Walker 1975) was converted to a two level voltage output. The voltages were recorded by a Solatron Data Logger (Compact Logger Series Two). Scour hole depths were thus recorded to within ± 2.6 mm (the distance between electrodes). This represents approximately 1.5 and 0.65 grain diameters for the two sediments used in the experiments. This figure was improved by observing the development of the hole through a small glass viewer and noting the number of engraved rings exposed on the pier.

3.5 SEDIMENT SUPPLY

During experiments with upstream bed motion it was necessary to add sediment at the upstream end of the moveable bed to avoid bed degradation.

A hopper, constructed of 3 mm mild steel in the shape of a four sided truncated pyramid was used. A variable width opening allowed material to be moved from the bottom of the hopper at different rates. A lug mounted constant speed Sinex rotary electric vibrator (A series) which was mounted to the base of the hopper maintained a constant flow rate of material.

3.6 BED MATERIAL AND PLACEMENT

Two gravels were used with geometric mean sizes of 4.02 mm and 1.8 mm determined by sieving using a series of standard square mesh sieves.

The 1.8 mm material was normally distributed with a geometric standard deviation of 1.37 (Figure 3.2) and was observed to be subrounded by direct examination (Figure 3.4).

The 4.02 mm material was normally distributed (Figure 3.3) with a geometric standard deviation of 1.16 and was observed to be rounded to well rounded. (Figure 3.4). The inclusive graphic standard deviation of the two sediments was also calculated and was found to be 0.59 mm for the 1.8 mm sediment and 0.58 mm for the 4.02 mm sediment. This shows that the 1.8 mm bed material was not as uniform as the 4.02 mm material so armouring effects would be more pronounced with the finer bed material. Both bed materials had specific gravities of 2.65 ± 0.01.

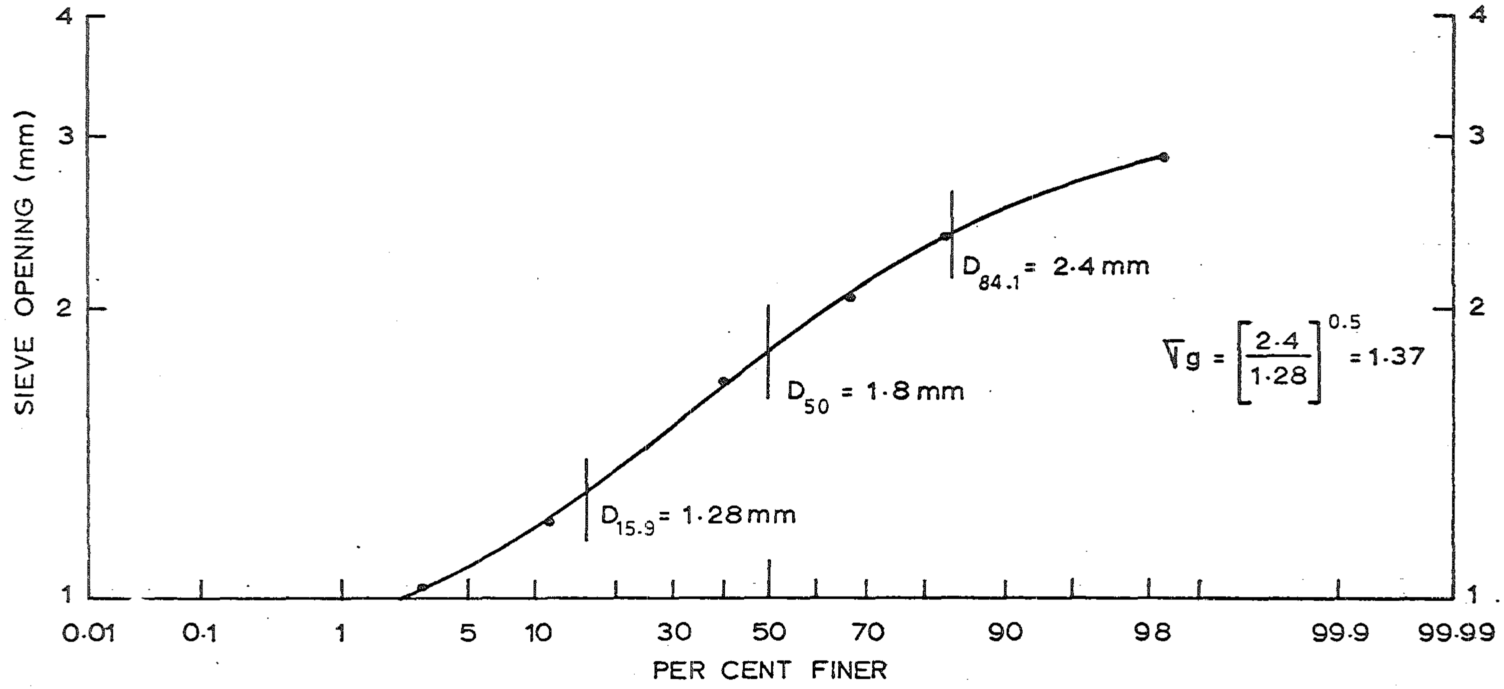


Fig.3.2 CUMULATIVE LOGARITHMIC-PROBABILITY SIZE FREQUENCY GRAPH FOR 1.8mm SEDIMENT.

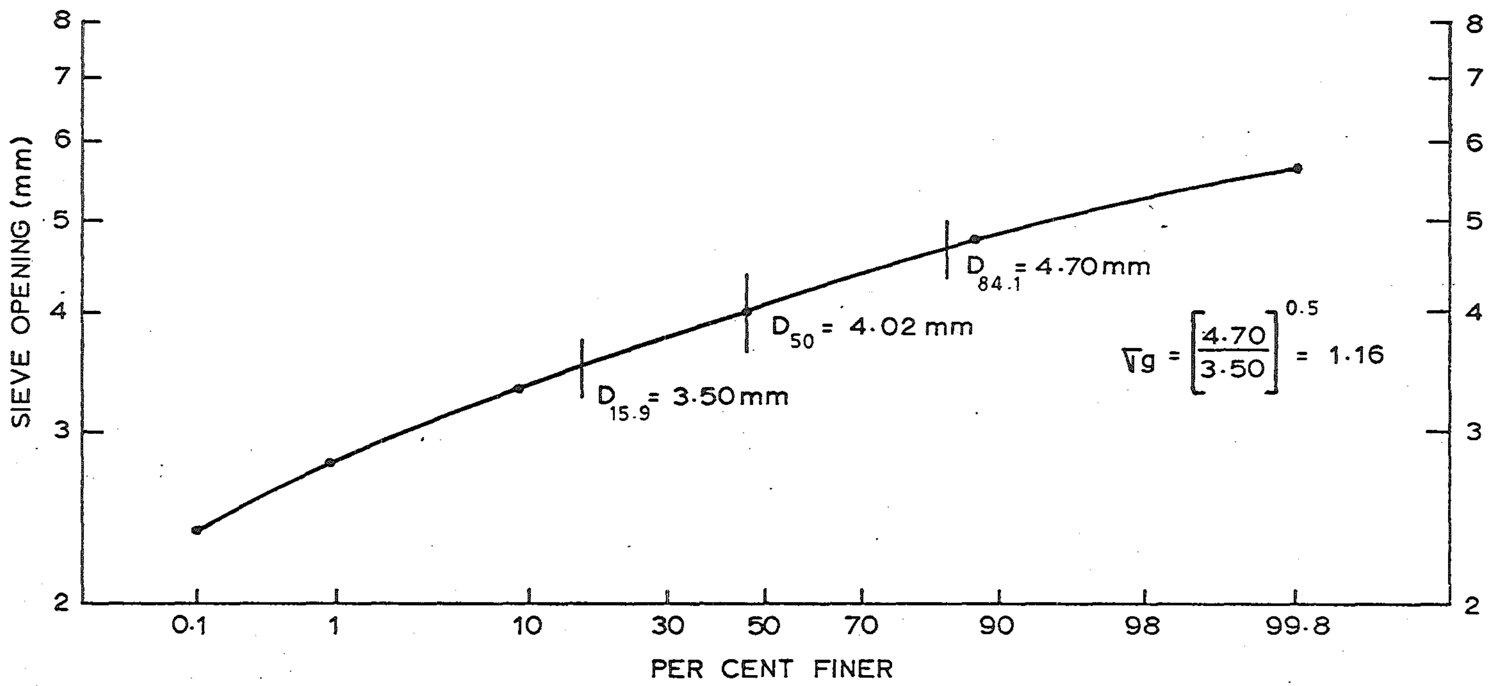
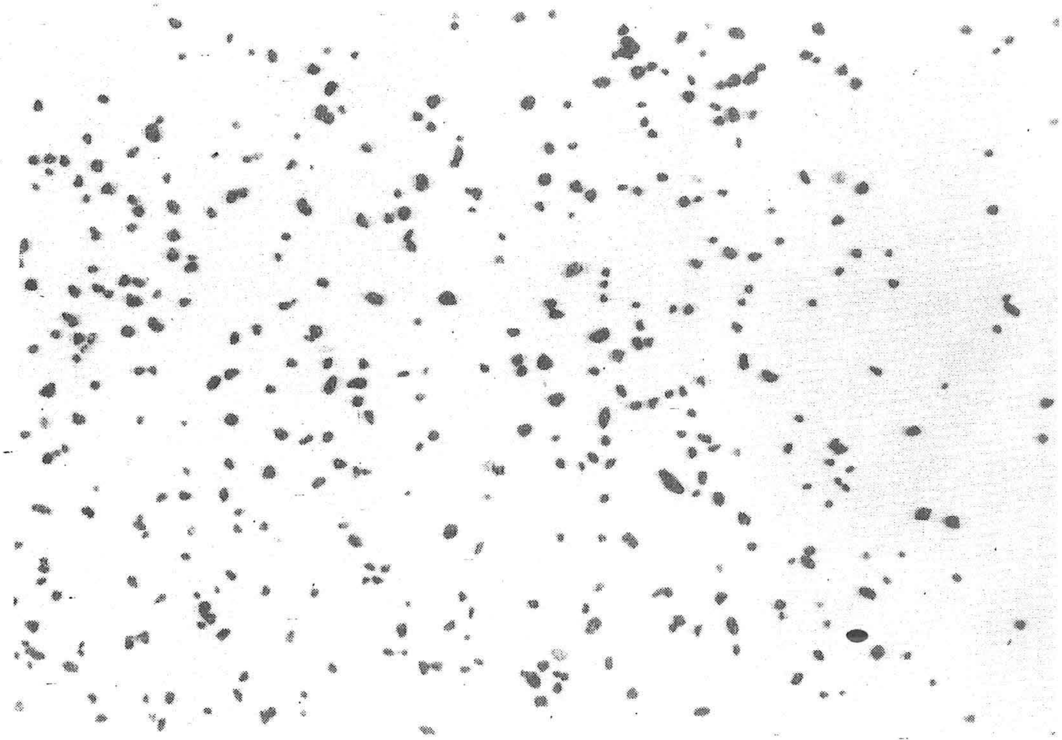
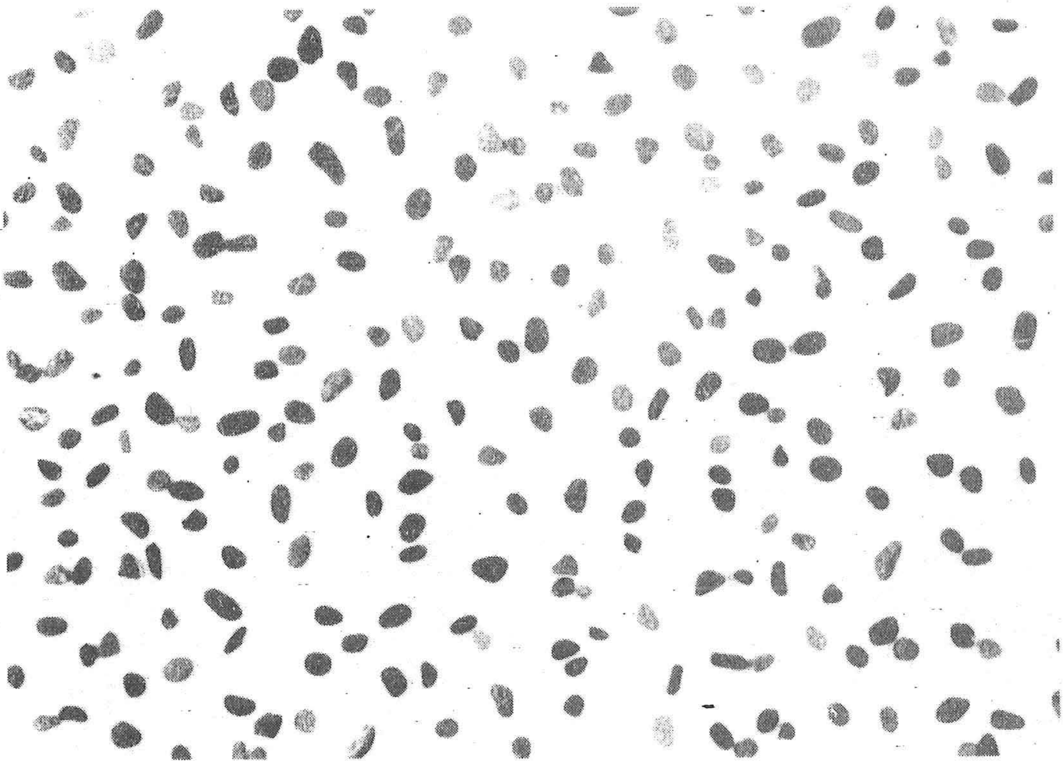


Fig.3·3 CUMULATIVE LOGARITHMIC-PROBABILITY SIZE FREQUENCY GRAPH FOR 4·02 mm. SEDIMENT.



D_{50} = 1.0mm



D_{50} = 4.02mm

Fig.3.4 GRAIN SIZE SHAPES

(NOT ACTUAL SIZE)

The test material was laid in two different depths along 16.20 m of the flume (Figure 3.1). At the pier section, the bed, consisting entirely of the test material, was 250 mm deep for a distance of 500 mm either side of the pier. On either side of this section there was a filling gravel 180 mm thick with 70 mm of test material overlying it. At the upstream end a roughened cement wash was laid for a length of 4.3 m. The scouring at the transition from the fixed to loose bed was used as a guide for the sediment input from the hopper once bed movement began. There was a fixed bed section for a length of 2.0 m downstream of the loose bed which was also made of roughened cement wash. This was to prevent scouring of the bed material caused by the M2 water surface profile which extended upstream of the outflow for approximately 2.0 m.

3.7 EXPERIMENTAL PROCEDURES

i. Introduction

Two series of tests were run in this study. The first series used the 4.02 mm material and the second series the 1.8 mm material. In each series steady state runs were made at different bed slopes to investigate the effect of velocity and slope on scour depth and for later comparison with translation wave runs.

ii. Steady State Runs

Before each clear water run the bed was levelled with a blade and a 600 mm x 150 mm high gate was placed at the downstream end of the flume.

Water was run slowly from the upstream end until the required depth was obtained, care being taken not to disturb the bed. The scour meter was checked and adjusted if necessary as were the depth recorders. The valve was then opened to a predetermined setting producing a surge down the still water in the flume. Scanning of the electrodes and the stop watches were started as the surge reached the pier. As the surge reached the end of the flume the downstream gate was removed thus rapidly producing steady state flow conditions. The runs lasted for varying durations (Appendix III). During each run the following series of observations and measurements was done:

i. The depth of scour was recorded periodically to give a time history record of scour development.

ii. A visual record of selected runs was kept in the form of sketches, notes, and photographs.

iii. Water temperature was monitored during each run.
Because the water was being passed through the laboratory's constant head system, it was possible to change the water during the experiments. This enabled the water temperature variation to be controlled within $4^{o}C$ during each run.

iv. The flow, bed and water surface elevations were checked periodically to make sure that they were constant and that the flow was uniform upstream of the pier.
The runs were continued until there was no noticeable change in scour depth for a period of at least two hundred minutes.
In runs with bed motion the flow was increased from zero to the desired amount. This was done as it was impractical to use the clear water procedure for the large flows required for continuous bed motion: These runs were continued for at least one hour as the equilibrium scour depth was obtained in a very short period of time.

iii. Non Steady Runs - Translation Waves

These runs were to investigate the scouring induced by the passage of a flood wave past the pier. After raking and levelling the bed an initial condition of a flow insufficient to cause scouring at the pier was set up.

All waves were symmetrical and were created by the gradual opening (and shutting) of the control valve (150 mm line) to predetermined settings at given times. The 75 mm line was used to set the base flow for waves with a high peak flow.

The scour meter and wave were started simultaneously and during the passage of the wave the scour hole depth was observed and recorded every minute. The water surface elevations were also recorded periodically to check that the desired wave was in fact being obtained.

The runs with sediment motion upstream had the additional procedure of adding sediment at the upstream transition between the fixed and loose bed in the same manner as described in the steady state runs. The rate of addition was increased as the wave discharge increased always ensuring that no scour hole formed at the upstream end of the moveable bed.

CHAPTER IV

PRESENTATION AND ANALYSIS OF RESULTS

4.1 INTRODUCTION

Complete lists of the final records of all experiments are given in Appendix III. Of the steady state experiments twenty (No. 1-7, 12-24) were done with clear water scour conditions, and six (No. 8-11, 25, 26) with upstream bed movement. For both bed materials two bed slopes were used (0.001 and 0.0015) plus a third slope (0.004) was used with the 4.02 mm material. The flows associated with the steepest slope were non-uniform with an energy slope of 0.003 ± 15%.

One hundred flood wave experiments were completed. Each series of experiments had the same peak flow and base times ranging from ten to one hundred and twenty minutes. Eighty two experiments were run with clear water conditions, and eighteen with upstream bed motion. Detailed records of scour development with time are on file in the Civil Engineering Department at the University of Canterbury.

At this point it is convenient to define two variables. Firstly the limiting scour depth is designated d_s, and secondly the equilibrium scour depth, d_{se} of a time development record with upstream bed movement conditions. The distinction between the two is that scour depths with upstream bed motion fluctuate with time so that d_{se} is the average scour depth, while the limiting clear water scour depth d_s is constant.

All the scour depths shown in Figures 4.8 - 4.11 were recorded at the front of the pier. These depths are greater than side scour depths for the case of upstream bed movement and also for the higher clear water scour flows.

4.2 STEADY FLOW

Graphs of limiting scour depth versus the steady velocity which formed the scour hole are given in Figures 4.1 and 4.2.

Detailed records of scour development with time were kept for runs 7, 16 - 19 (clear water scour) and runs 10 and 11 (scour with upstream bed motion) only. Typical examples of clear water scour development with time are shown in Figures 4.3 and 4.4 and a typical section of a time development record of scour with upstream bed motion in Figure 4.5. The recordings of scour depth with time were not plotted for the first ten minutes in Figure 4.3 and the first three minutes in Figure 4.4 as the initial scouring rates are very high as the bed responds to the step change in discharge.

4.3 TRANSLATION WAVES

i. Clear Water Scour

All the clear water scour waves started with no initial scour hole.

Typical examples of scour development with time are given in Figures 4.6 and 4.7; the steady state scour depth shown at each time is that which would be caused by the corresponding wave discharge had that discharge been a steady flow of long duration. Plots of the final scour depth for a series of flood waves versus the duration of the flood waves are given in Figures 4.8 to 4.11.

ii. Scour With Upstream Bed Motion

Two types of experiments were performed with upstream bed movement. One had a critical flow too weak to cause any scouring and a peak flow capable of causing bed movement. Examples of these are Figures 4.12 and 4.13. The other experiments had an initial flow that caused bed movement and an initial scour depth equal to the equilibrium scour depth for that flow as seen in Figure 4.14. Therefore upstream bed motion exists for the duration of the wave. Plots of equilibrium scour depth versus wave base time are given in Figures 4.15 and 4.16.

4.4 ANALYSIS OF RESULTS

i. Clear Water Scour - Translation Waves

The following grouping of variables provides information relating the depth of scour to the bed and wave properties:

$$d_s = f\ (D_{50}, S, b, y_p, T) \qquad 4.1$$

where S = energy slope of the flow.

T = base time of flood wave (minutes) (see Figure 4.17)

y_p = water depth at the peak of the flood wave.

As scour depth increases with flood wave base time for a given peak discharge, equation 4.1 can be considered analogous to an equation describing scour development with time.

Section 2.3 of Chapter II shows that, in the past, researchers have considered scour development formulae to be either semilogarithmic or logarithmic with respect to time. Figure 4.11 shows the results for one set of waves plotted in both of the above forms. From Figure 4.11 and Figures 4.8 to 4.10 it can be seen that the relationship between scour depth and base time is best represented by an equation of the form:

$$d_s = A_o\ T^c \qquad 4.2$$

Here the coefficient and the exponent may both be functions of D_{50}, b, S and y_p in accord with equation 4.1.

By defining T_E (see Figure 4.17) as the time for which scouring is

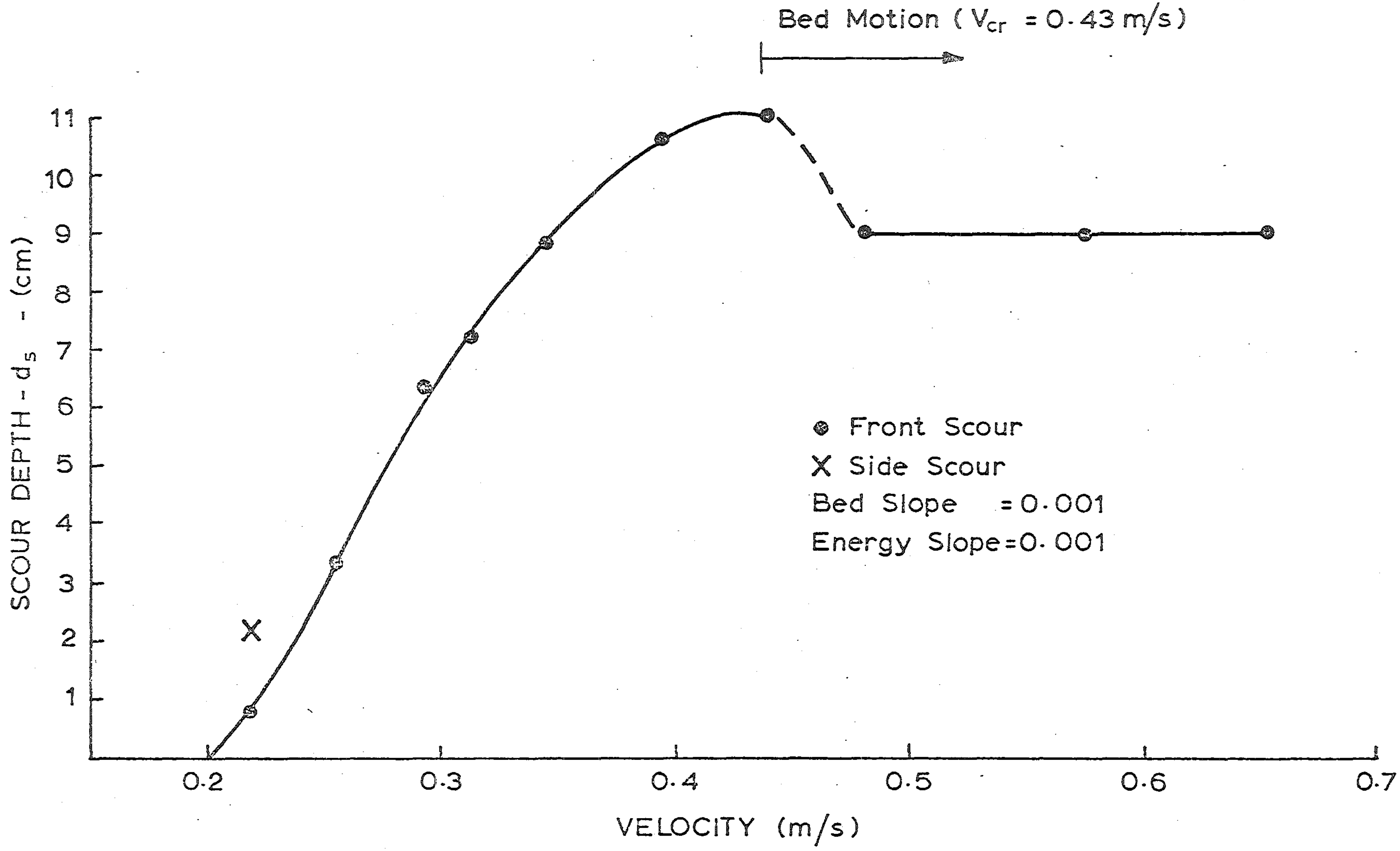


Fig. 4·1 EQUILIBRIUM SCOUR DEPTH AS A FUNCTION OF VELOCITY FOR STEADY FLOWS. (D_{50} = 1·8 mm.)

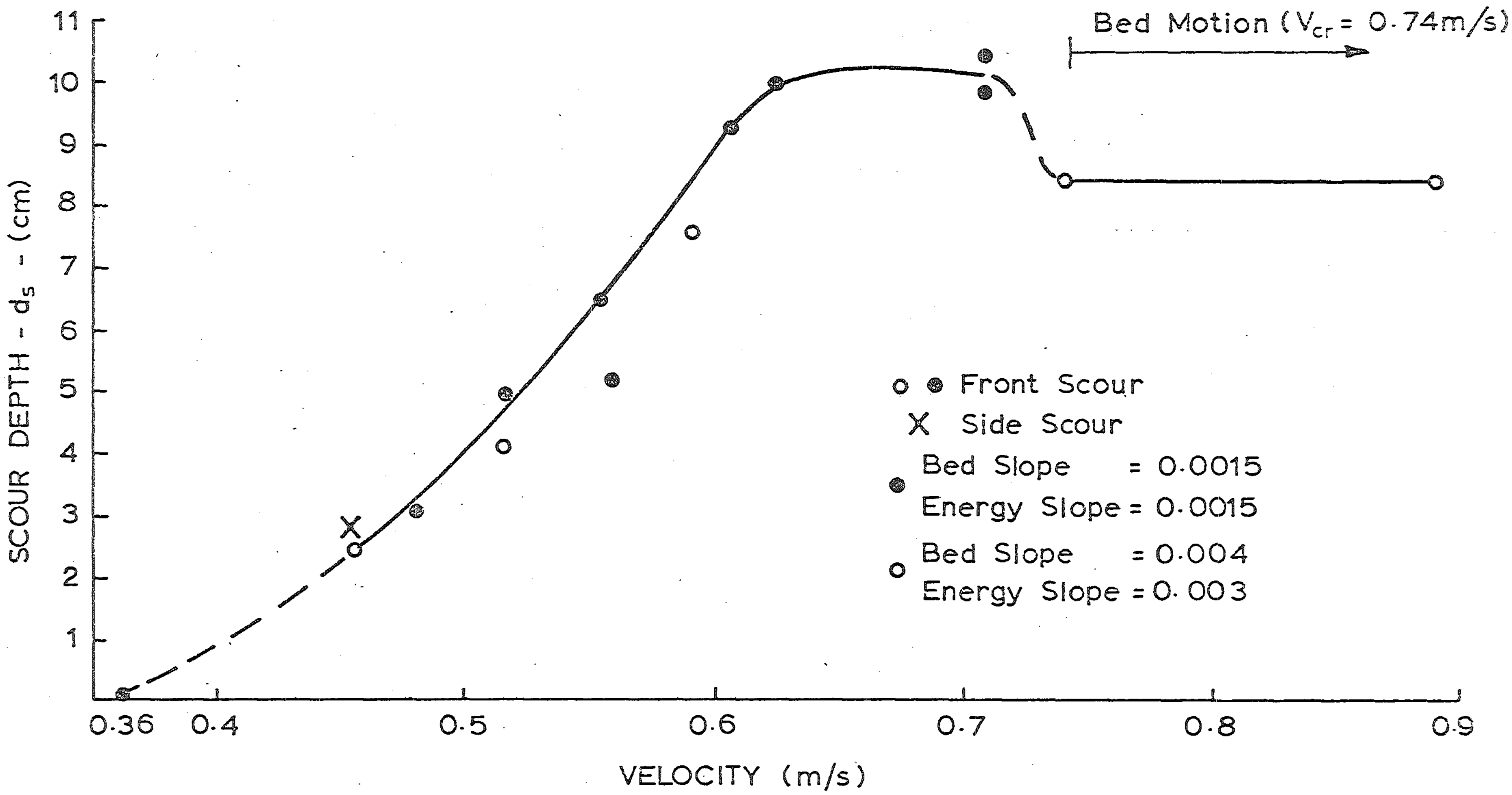


Fig. 4·2 EQUILIBRIUM SCOUR DEPTH AS A FUNCTION OF VELOCITY FOR STEADY FLOWS. (D_{50}= 4·02mm)

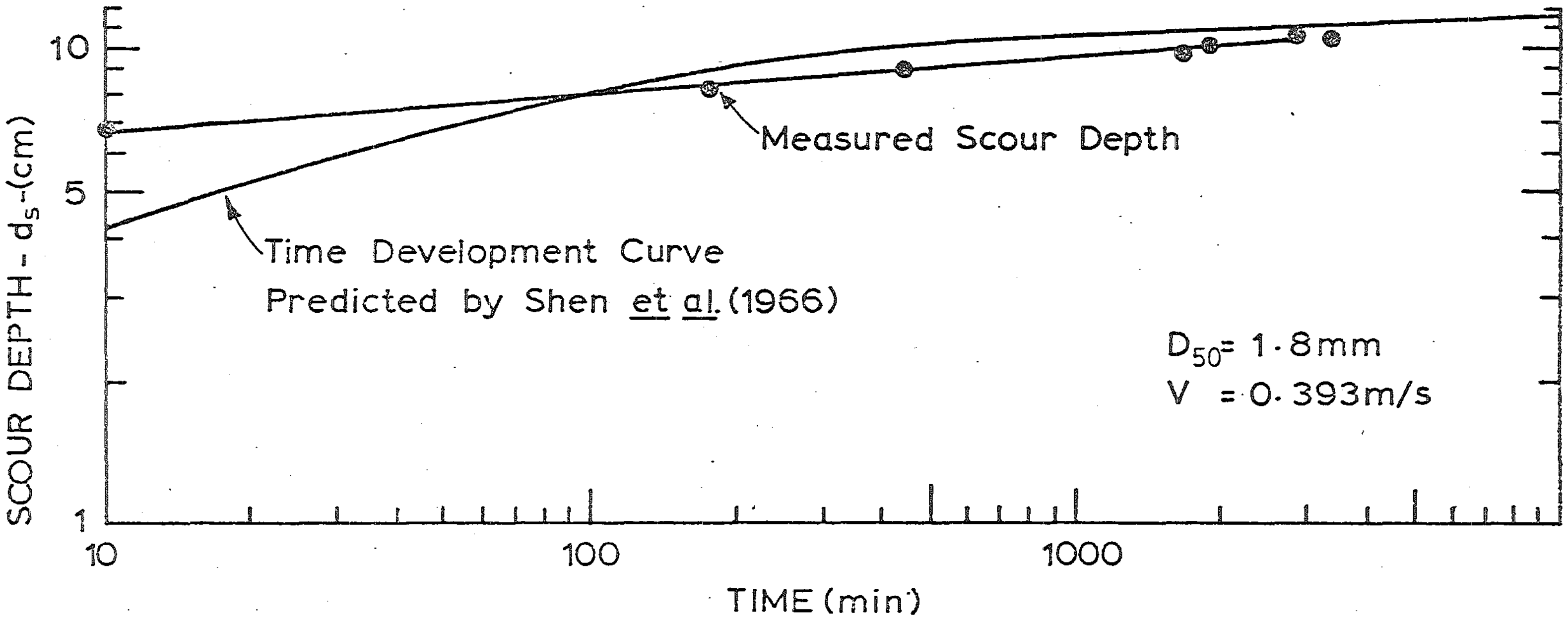


Fig.4.3 DEVELOPMENT OF SCOUR DEPTH WITH TIME. CLEAR WATER SCOUR WITH STEADY FLOW. (EXPERIMENT No.7)

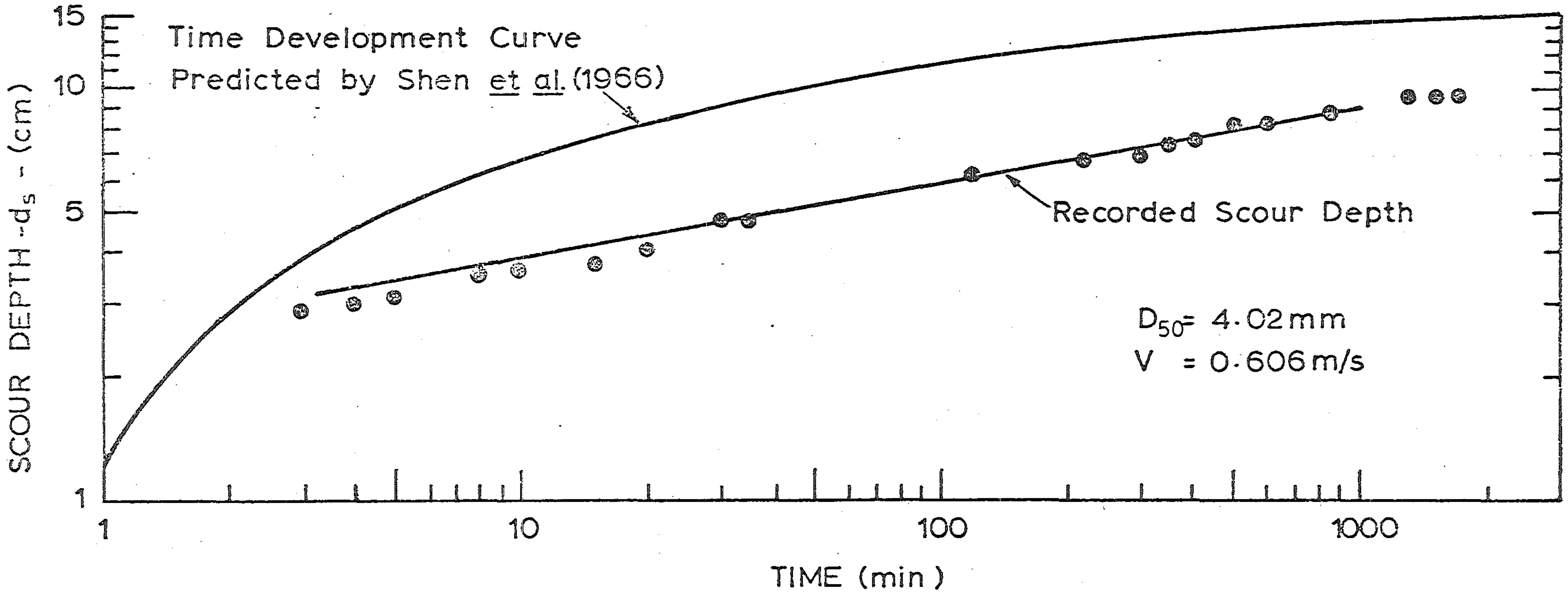


Fig.4.4 DEVELOPMENT OF SCOUR DEPTH WITH TIME. CLEAR WATER SCOUR WITH STEADY FLOW. (EXPERIMENT No.16).

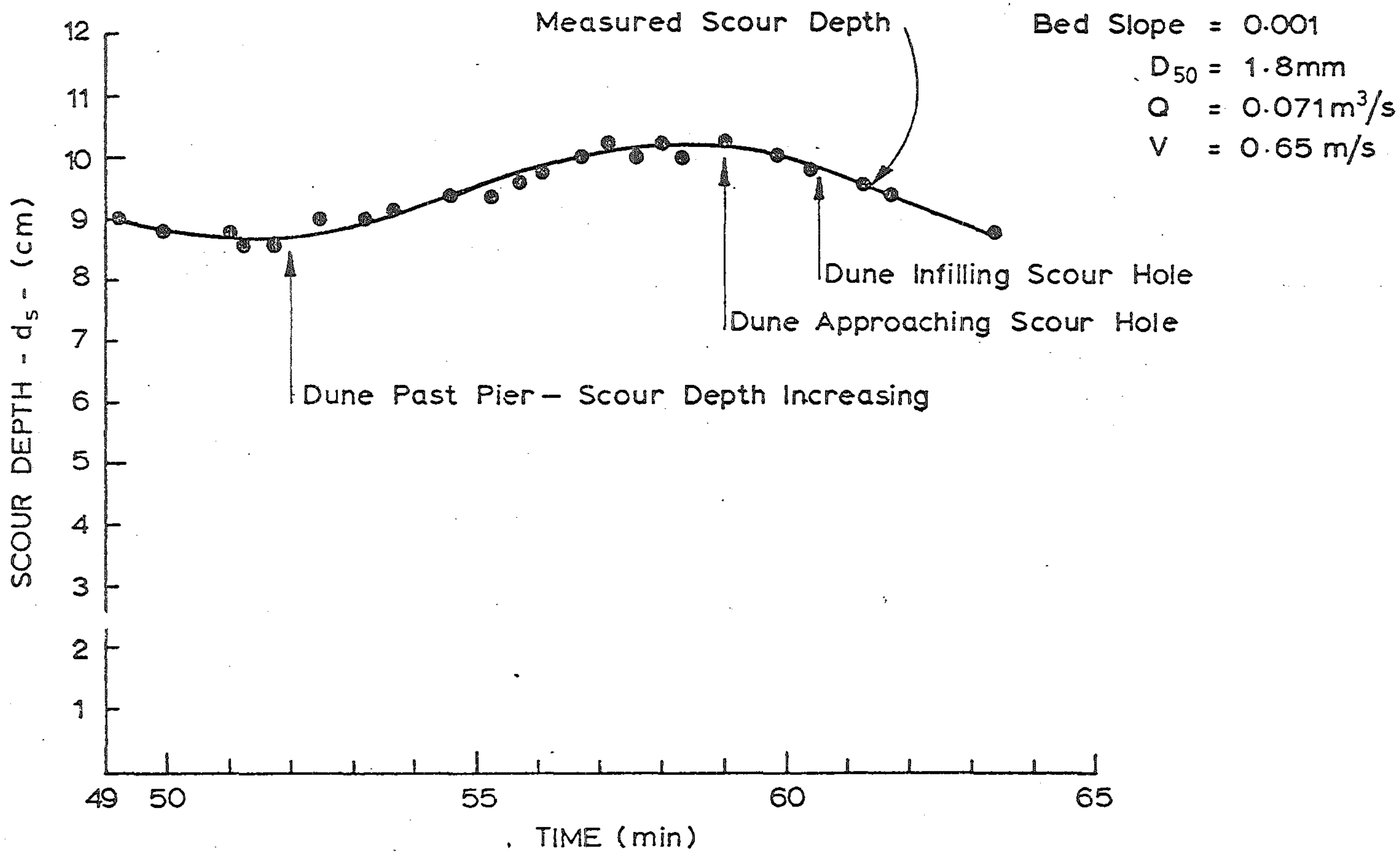


Fig.4.5 TIME DEVELOPMENT OF SCOUR DEPTH. UPSTREAM BED MOVEMENT WITH STEADY FLOW. PASSAGE OF DUNE.

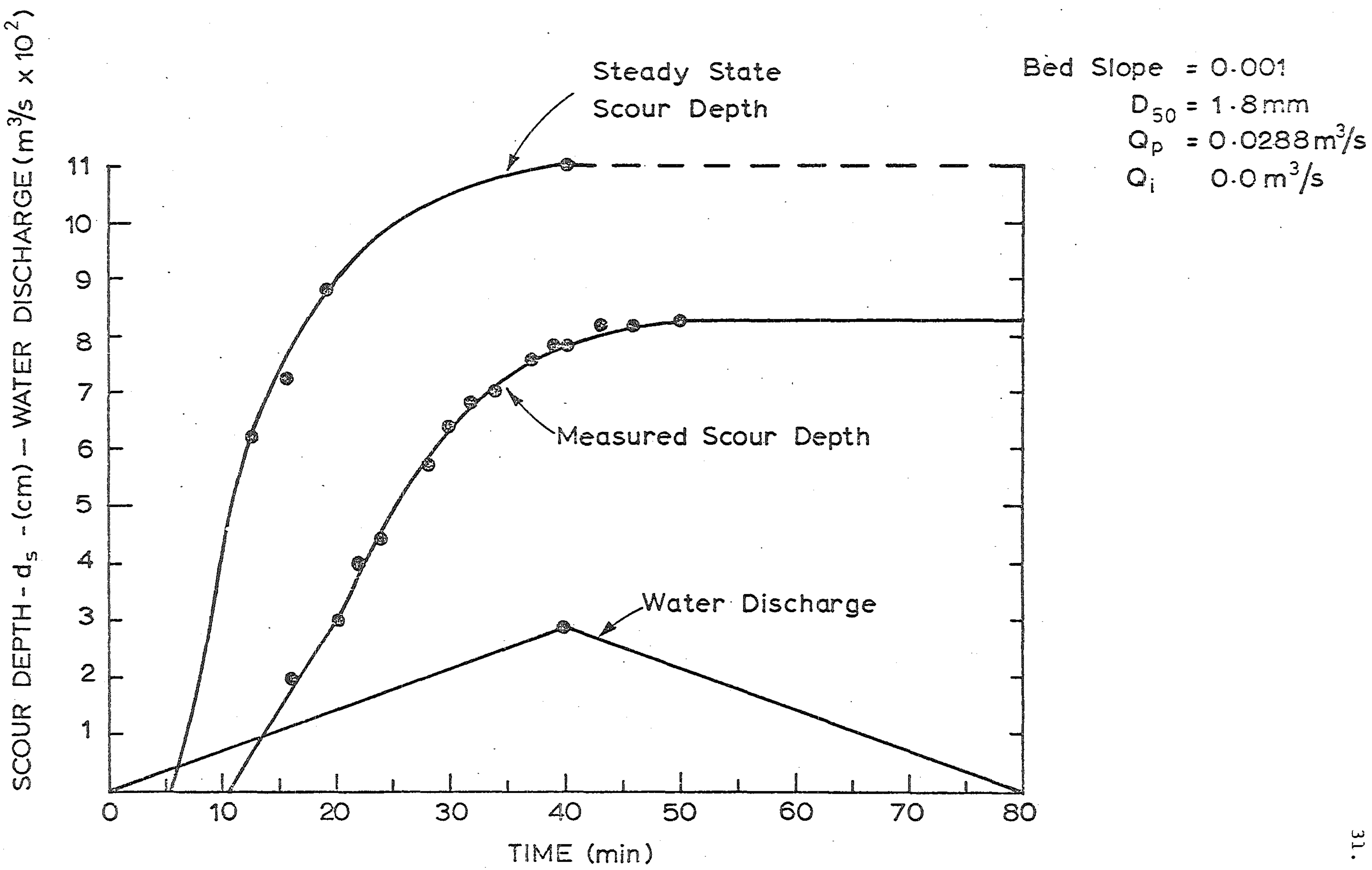


Fig.4·6 SCOUR DEVELOPMENT DURING A CLEAR WATER TRANSLATION WAVE.

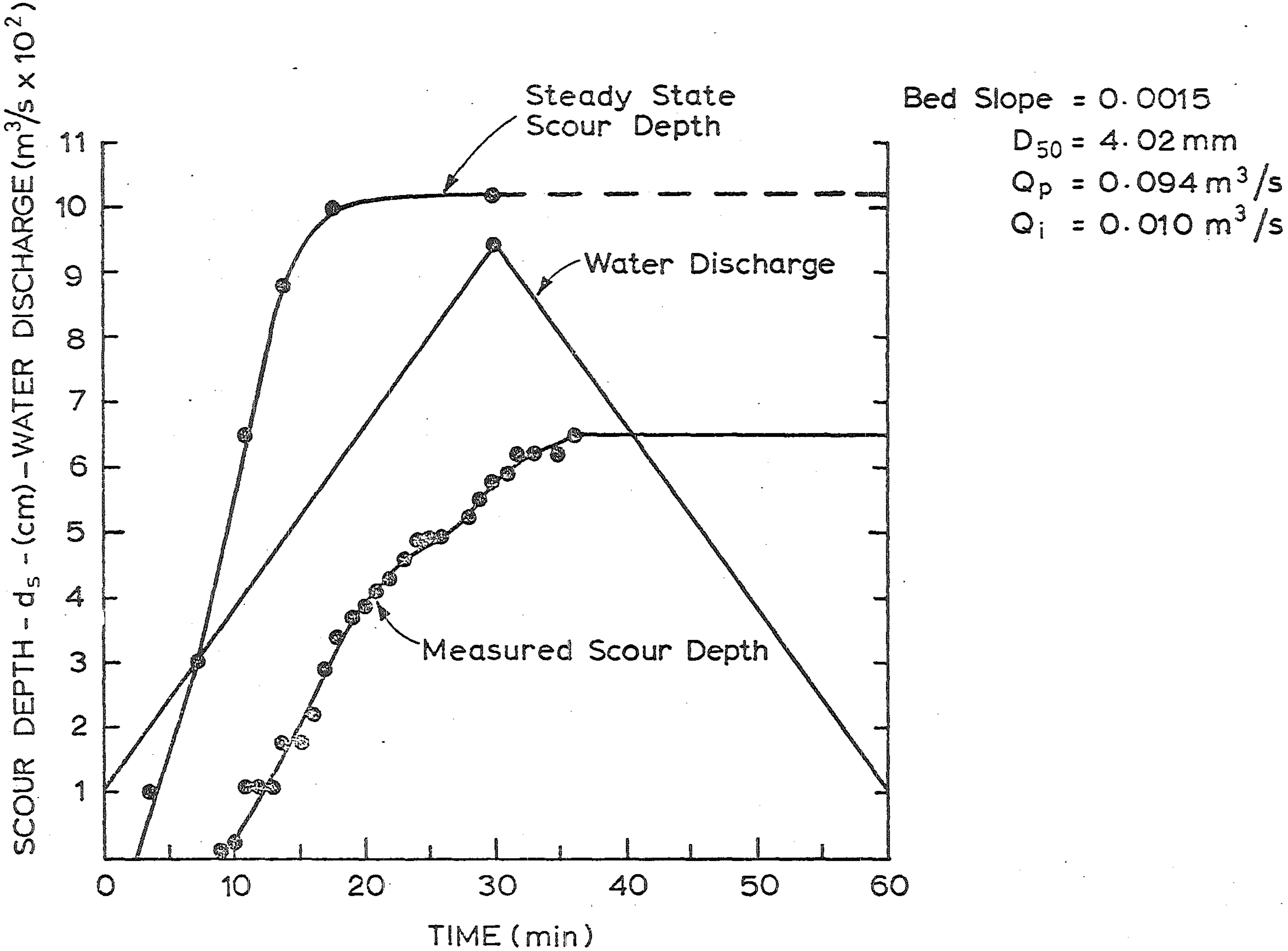


Fig. 4·7 SCOUR DEVELOPMENT DURING A CLEAR WATER TRANSLATION WAVE

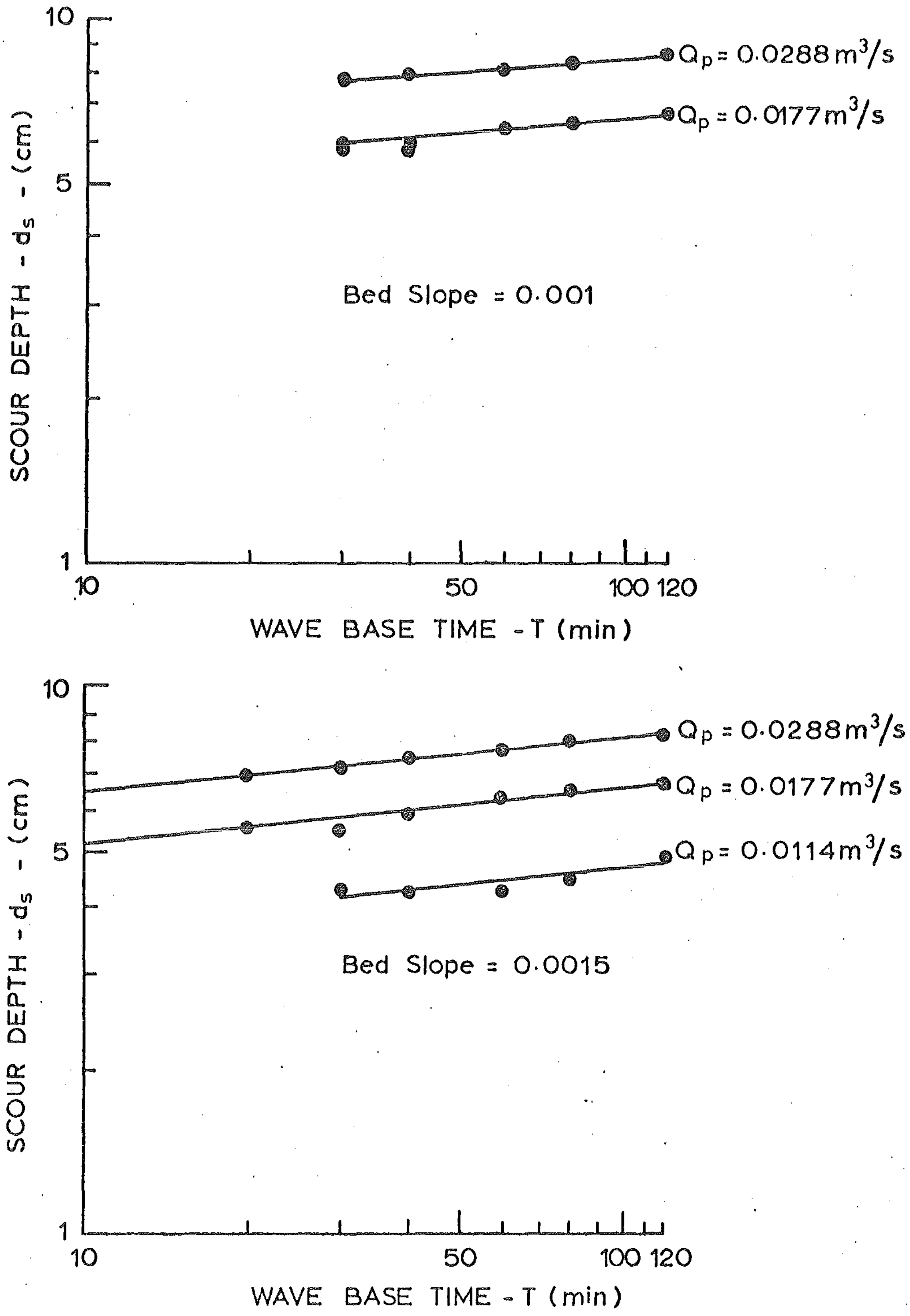


Fig.4.8 FINAL SCOUR DEPTH FOR CLEAR WATER TRANSLATION WAVES. (D_{50} = 1.8mm.)

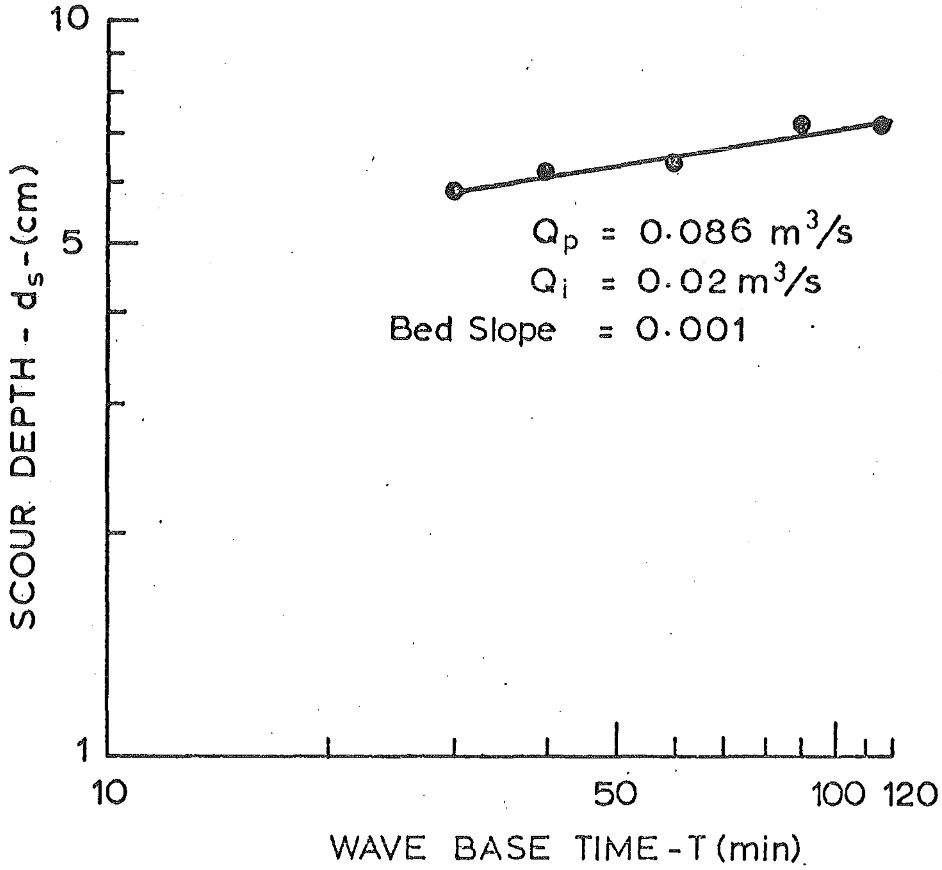


Fig.4.9 FINAL SCOUR DEPTH FOR CLEAR WATER TRANSLATION WAVES. (D_{50}= 4.02mm.)

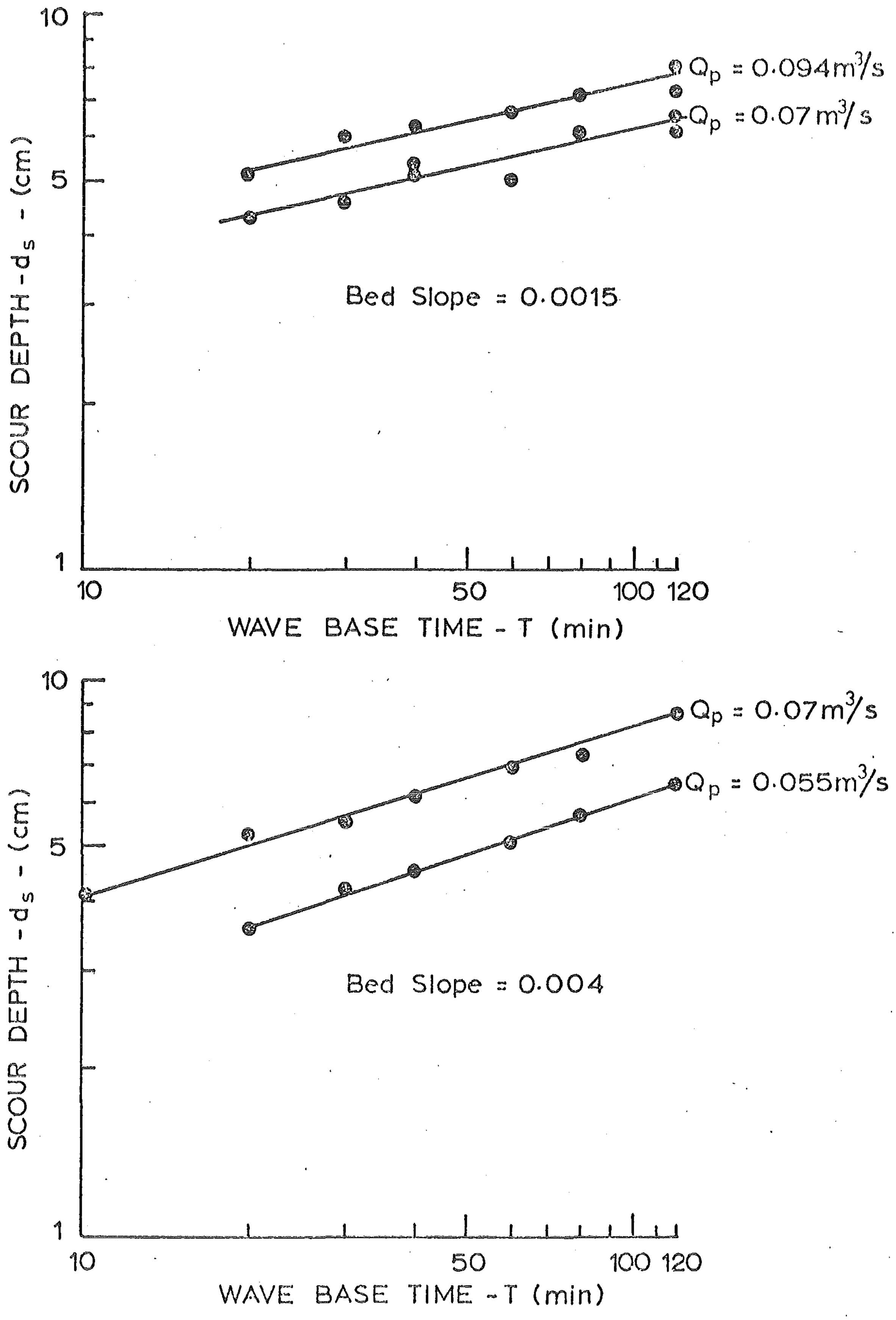


Fig.4.10 FINAL SCOUR DEPTH FOR CLEAR WATER TRANSLATION WAVES. (D_{50} = 4.02 mm)

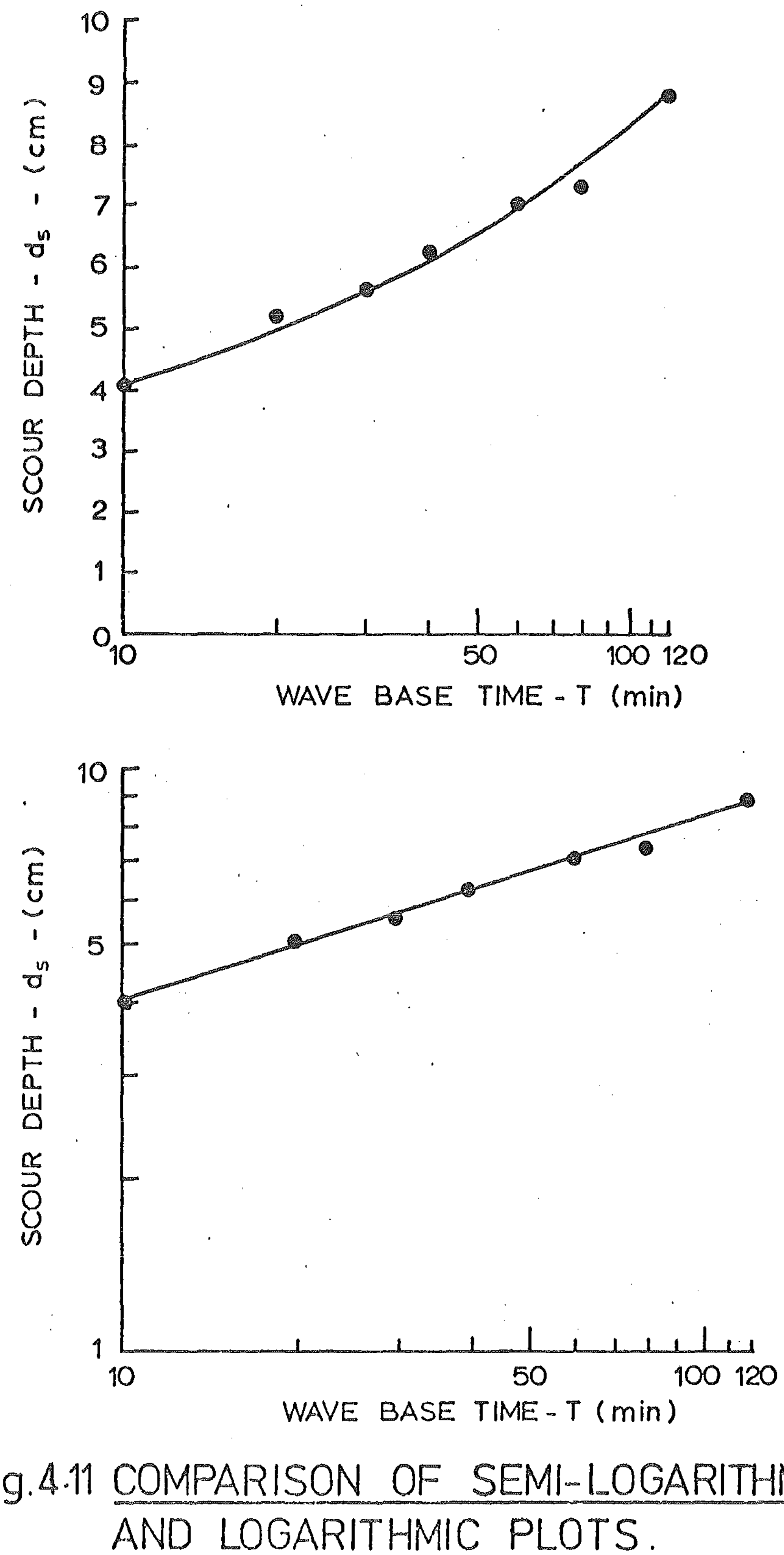


Fig.4.11 COMPARISON OF SEMI-LOGARITHMIC AND LOGARITHMIC PLOTS.
(D_{50}= 4·02 mm., Q_p= 0·07m^3/s; Bed Slope =0·004)

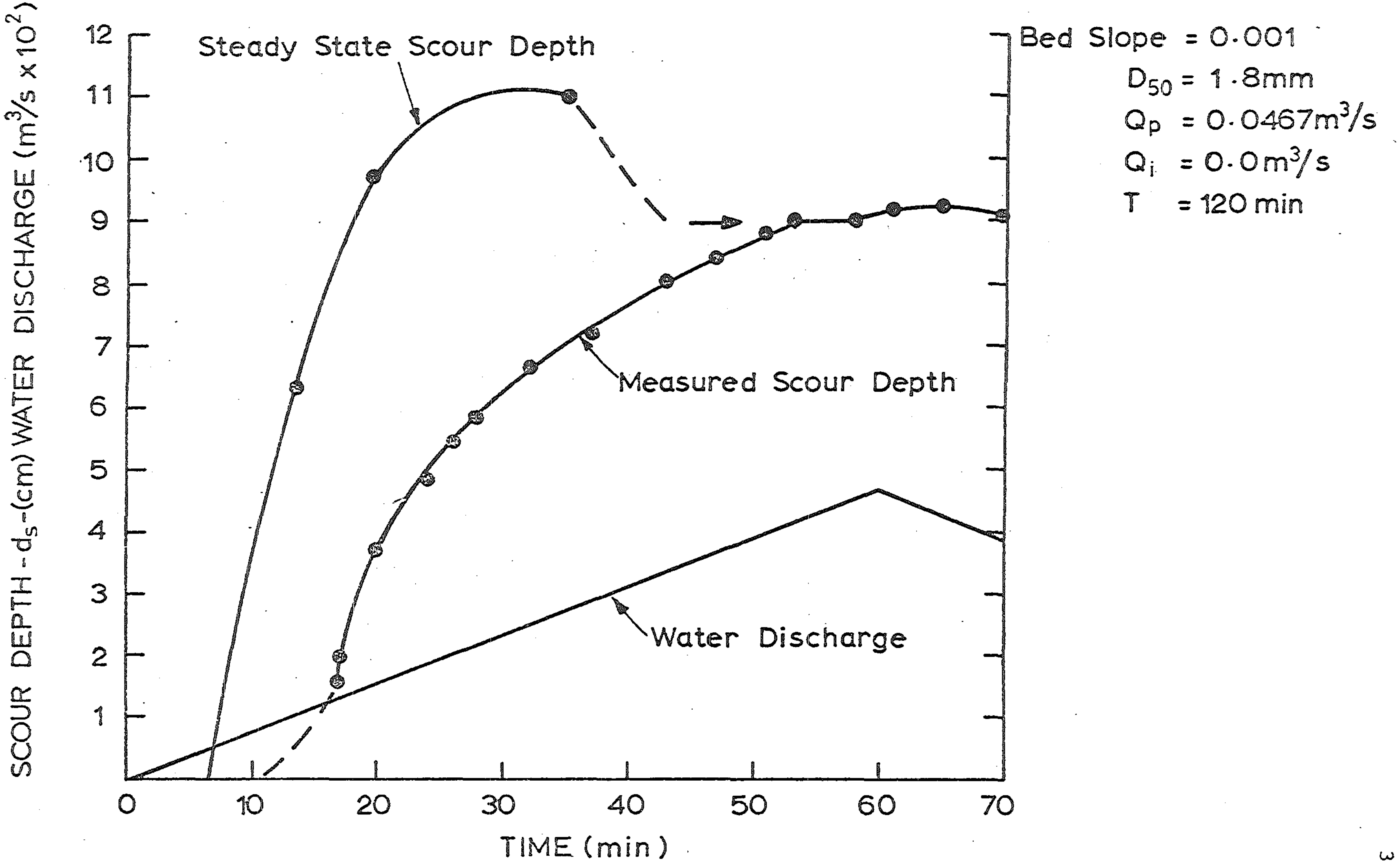


Fig. 4·12 SCOUR DEPTH DURING A TRANSLATION WAVE. CLEARWATER TO UPSTREAM BED MOVEMENT CONDITIONS.

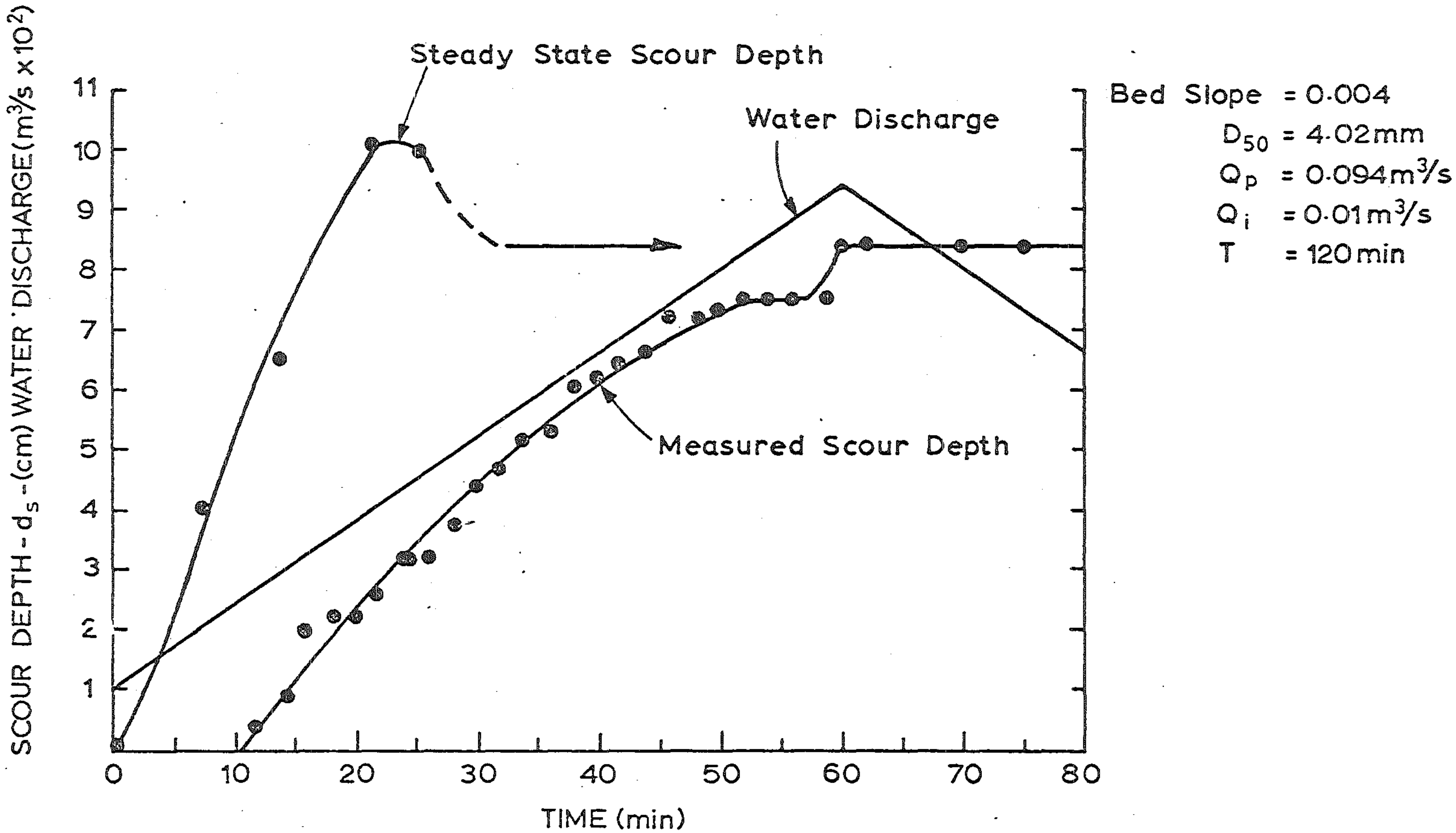


Fig.4.13 SCOUR DEPTH DURING A TRANSLATION WAVE. CLEAR WATER TO UPSTREAM BED CONDITIONS.

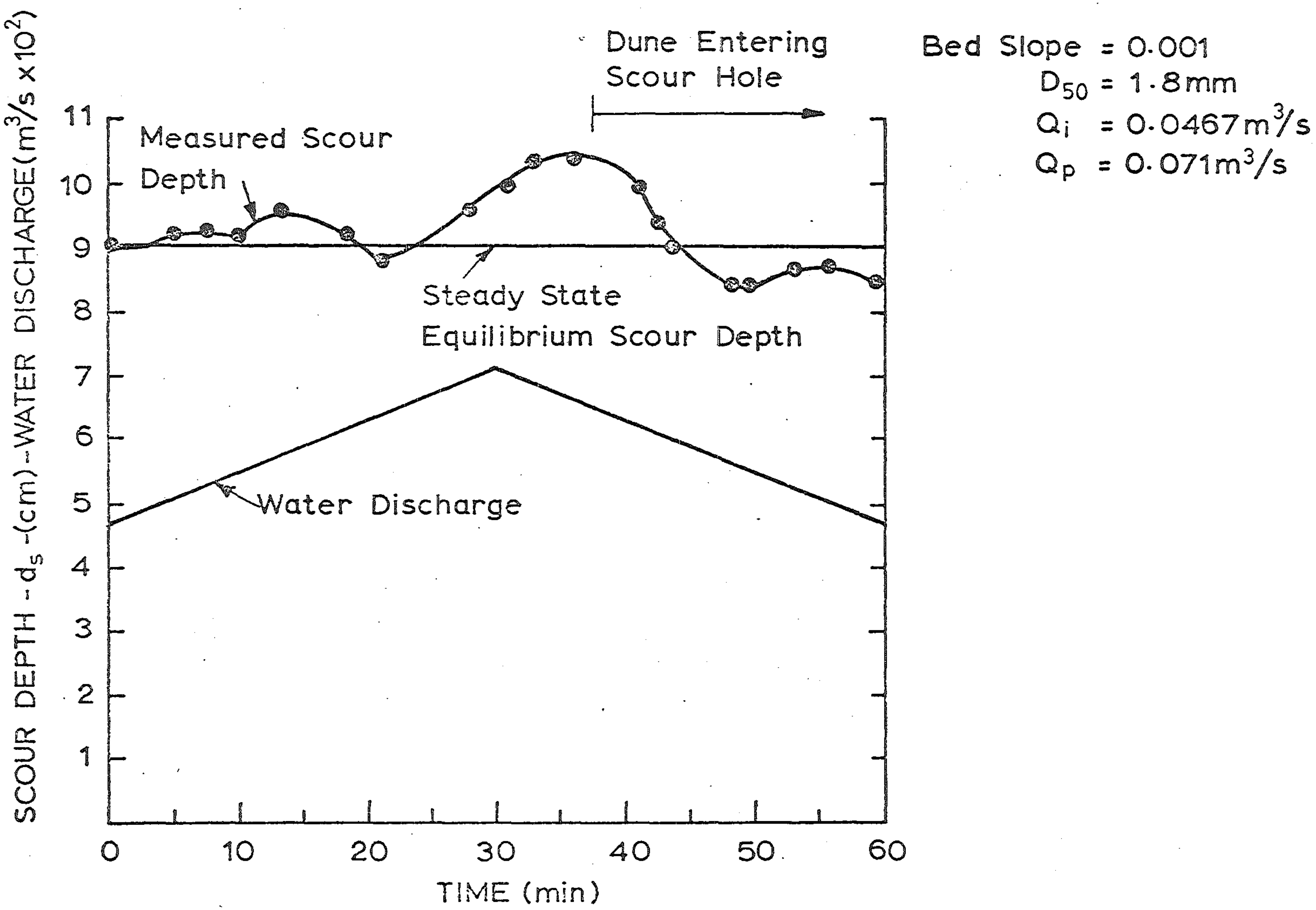
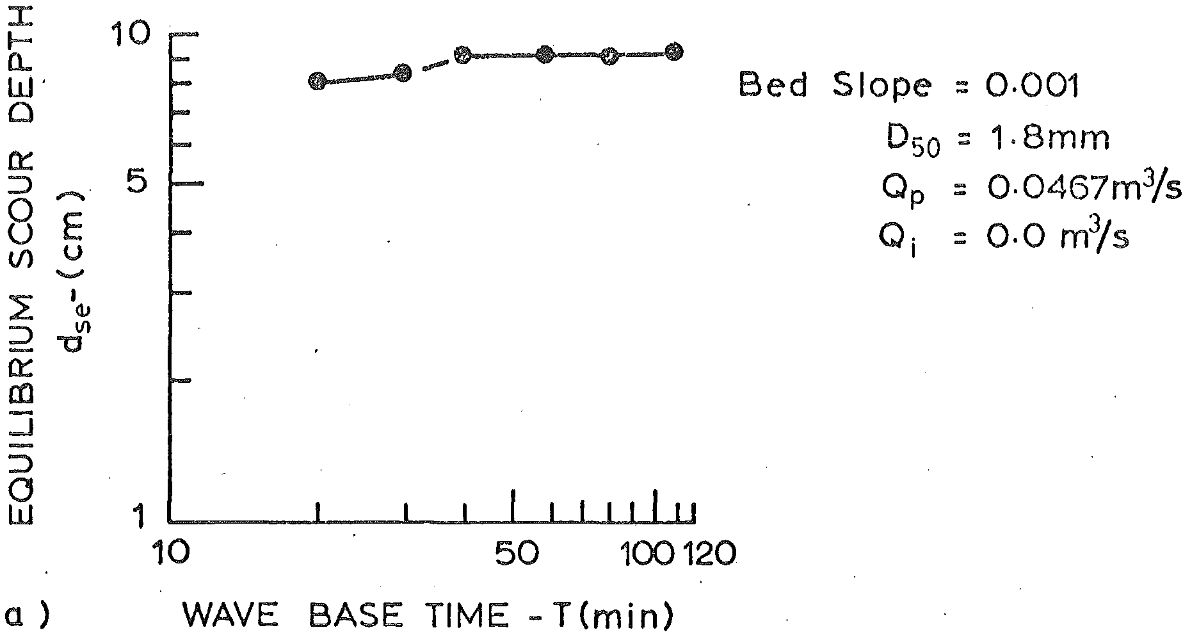
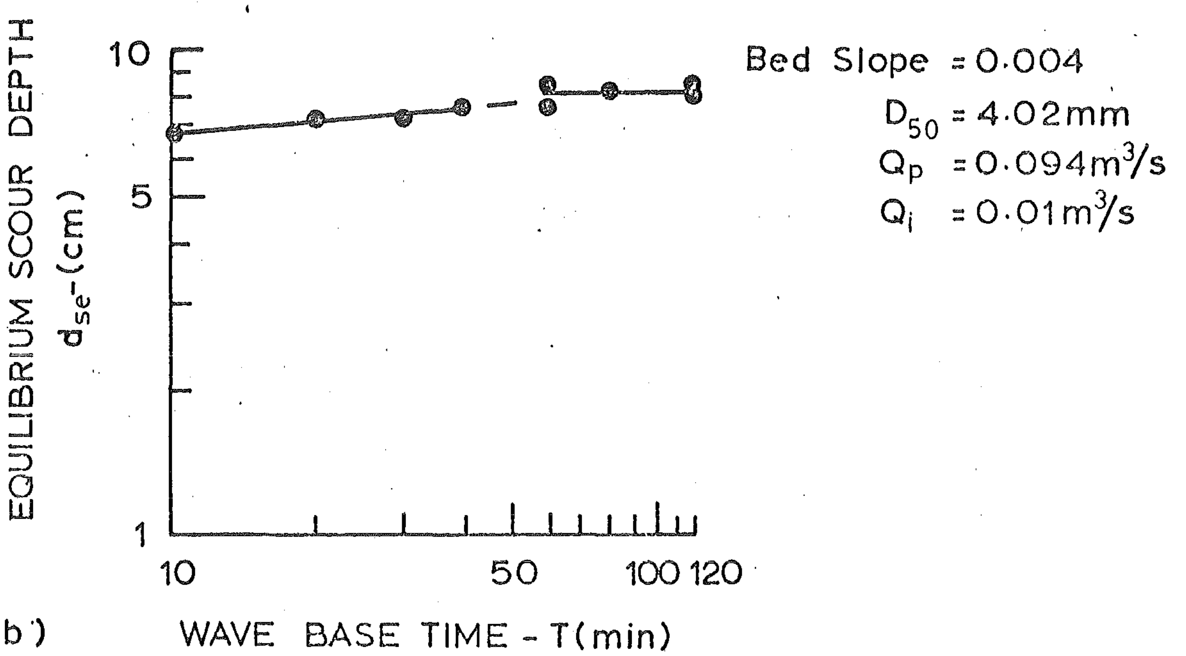


Fig.4·14 SCOUR DEPTH DURING A TRANSLATION WAVE WITH UPSTREAM BED MOVEMENT. INITIAL SCOUR DEPTH 9·0cm.



(a)



(b)

Fig. 4.15 EQUILIBRIUM SCOUR DEPTH FOR WAVES WITH UPSTREAM BED MOVEMENT. INITIAL SCOUR DEPTH = 0.0

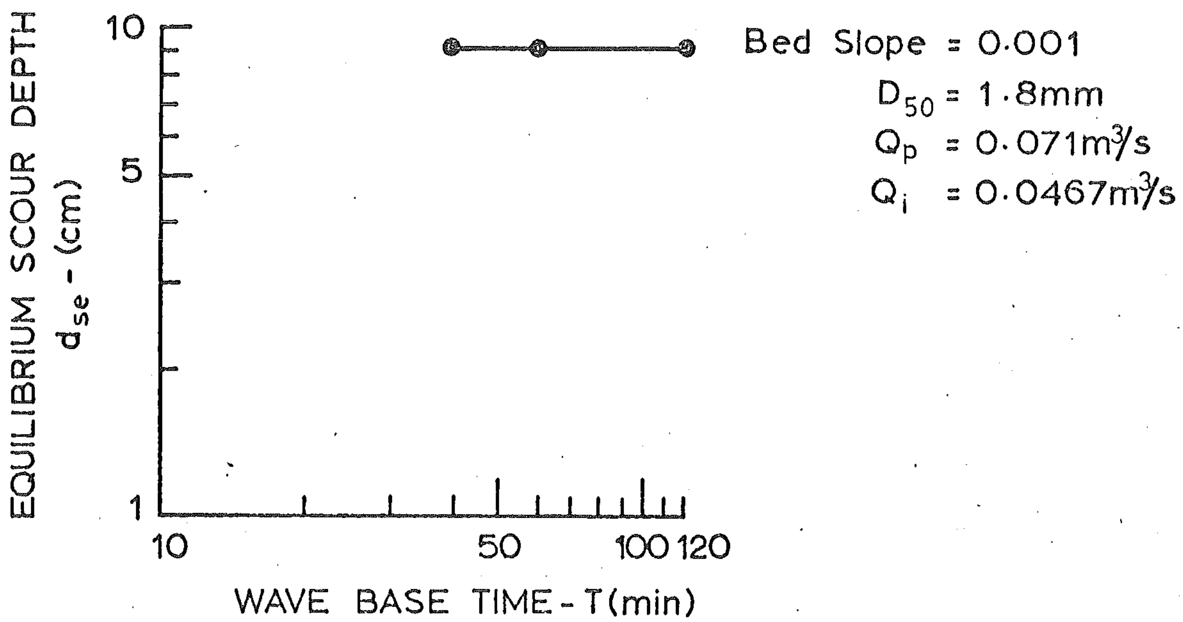


Fig.4.16 EQUILIBRIUM SCOUR DEPTH FOR WAVES WITH UPSTREAM BED MOVEMENT. INITIAL CONDITION IS UPSTREAM BED MOVEMENT.

possible (the time that elapses before scouring starts is of no interest) equation 4.2 becomes:

$$d_s = A_1 \, T_E^{\,c} \qquad 4.3$$

It was found that by dividing the final scour depth by the maximum approach depth the coefficient and the exponent became constant for a given energy slope and grain size so that:

$$\frac{d_s}{y_p} = A_2 \, T_E^{\,c} \qquad 4.4$$

where $A_2 = f\,(D_{50}, S, b)$ 4.5a

and $c = f\,(D_{50}, S, b)$ 4.5b

(see Appendix IV for the values of A_2 and c for the different wave situations).

Plotting A_2 as a function of slope (Figure 4.18) on logarithmic graph paper, gives two parallel lines separated according to grain size. From Figure 4.18 and using the other characteristic length in the problem, viz. b, to form a nondimensional parameter, one obtains:

$$A_2 = S^{-0.29} \; f\left(\frac{b}{D_{50}}\right) \qquad 4.6$$

Plotting $f\left(\frac{b}{D_{50}}\right)$ against $\left(\frac{b}{D_{50}}\right)$

(Figure 4.19) gives the equation

$$A_2 = 2.36 \times 10^{-6} \; S^{-0.29} \left(\frac{b}{D_{50}}\right)^{1.67} \qquad 4.7$$

Plotting now c as a function of slope (Figure 4.20) and non-dimensionalising grainsize again with pier diameter leads to:

$$c = S^{0.6} \; f\left(\frac{D_{50}}{b}\right) \qquad 4.8$$

Plotting $f\left(\frac{D_{50}}{b}\right)$ against $\left(\frac{D_{50}}{b}\right)$

(Figure 4.21) leads to:

$$c = 1.17 \times 10^{2} \; S^{0.6} \left(\frac{D_{50}}{b}\right)^{0.9} \qquad 4.9$$

Plots of equation 4.3 and actual data are shown in Figures 4.22 to 4.25.

ii. Translation Waves With Upstream Bed Movement

Figure 4.15a shows an increase in equilibrium scour depth with time up to the 40 minute wave and then constant depth at the value shown in Figure 4.1 for the 1.8 mm material, similarly for the 4.02 mm material, the equilibrium scour depth increases up to the 60 minute wave and then is constant (Figure 4.15b). Figure 4.16 shows no increase in equilibrium scour depth with flood wave time for the initial conditions of continuous bed motion and an initial scour hole.

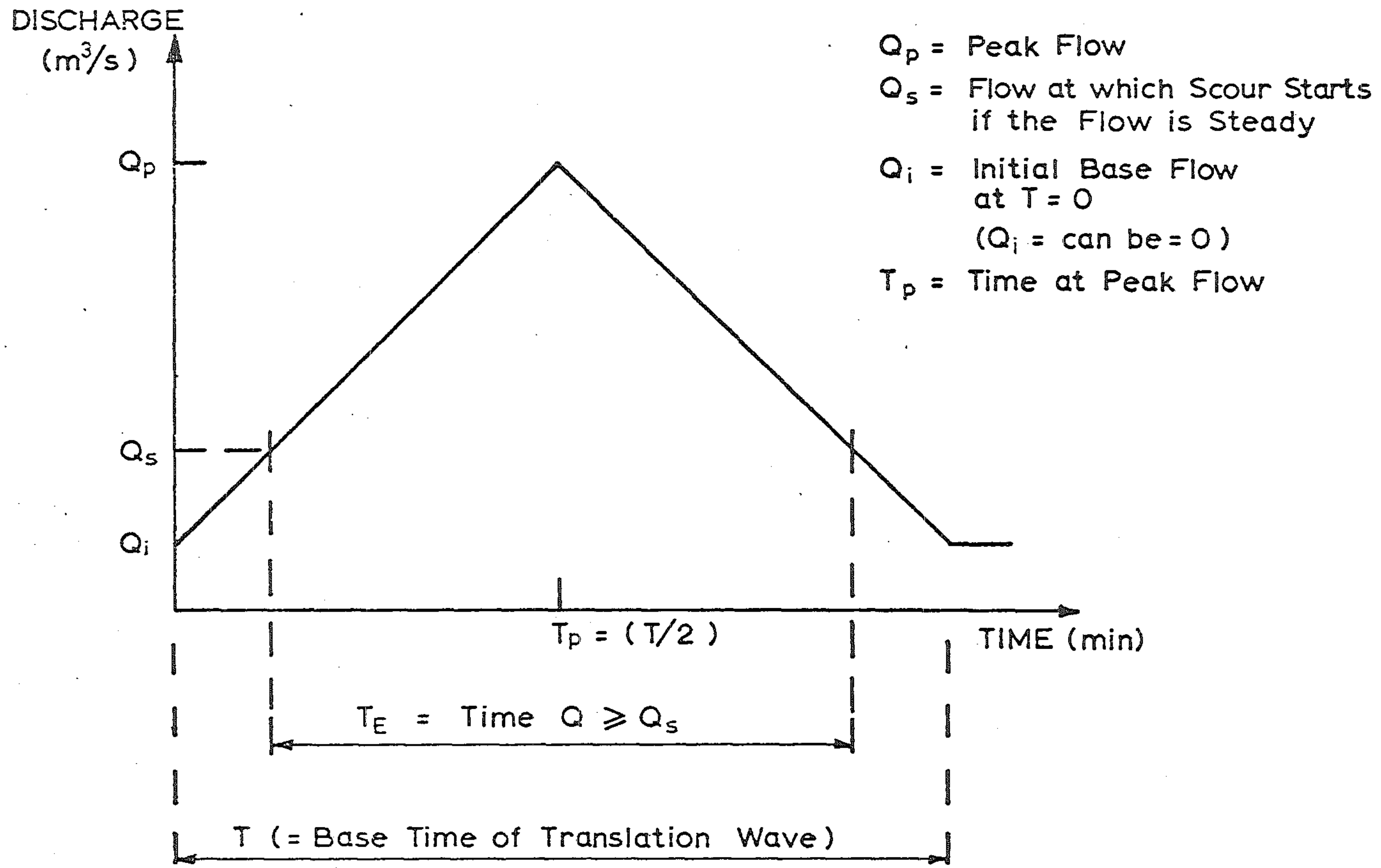


Fig. 4.17 DISCHARGE-TIME CHARACTERISTICS OF A TRANSLATION WAVE.

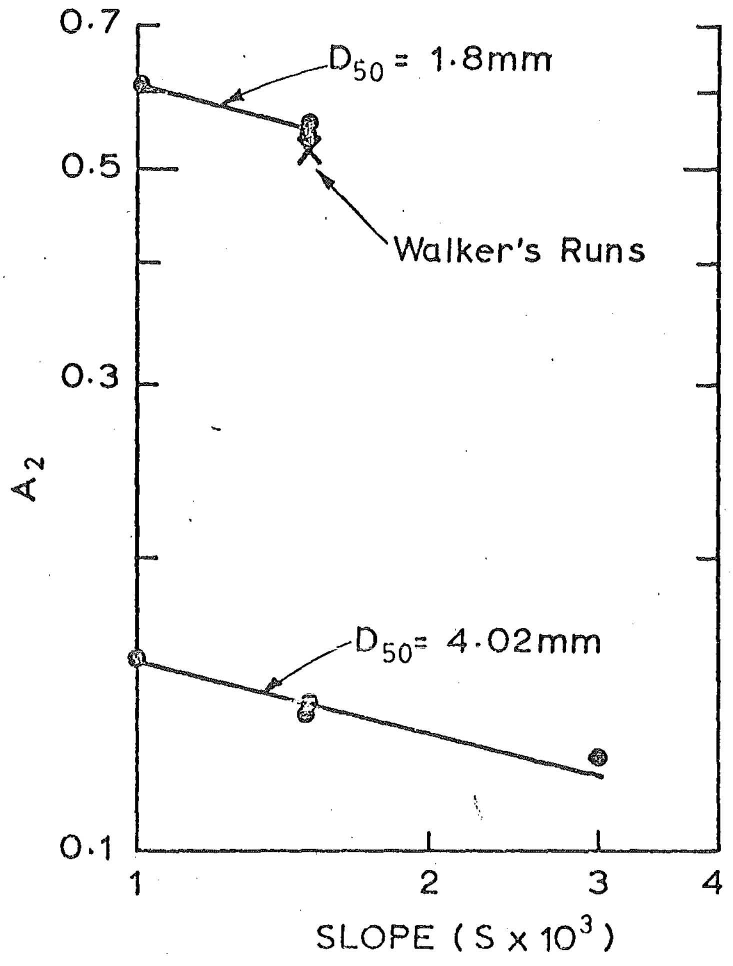


Fig. 4.18 A_2 AS A FUNCTION OF SLOPE.

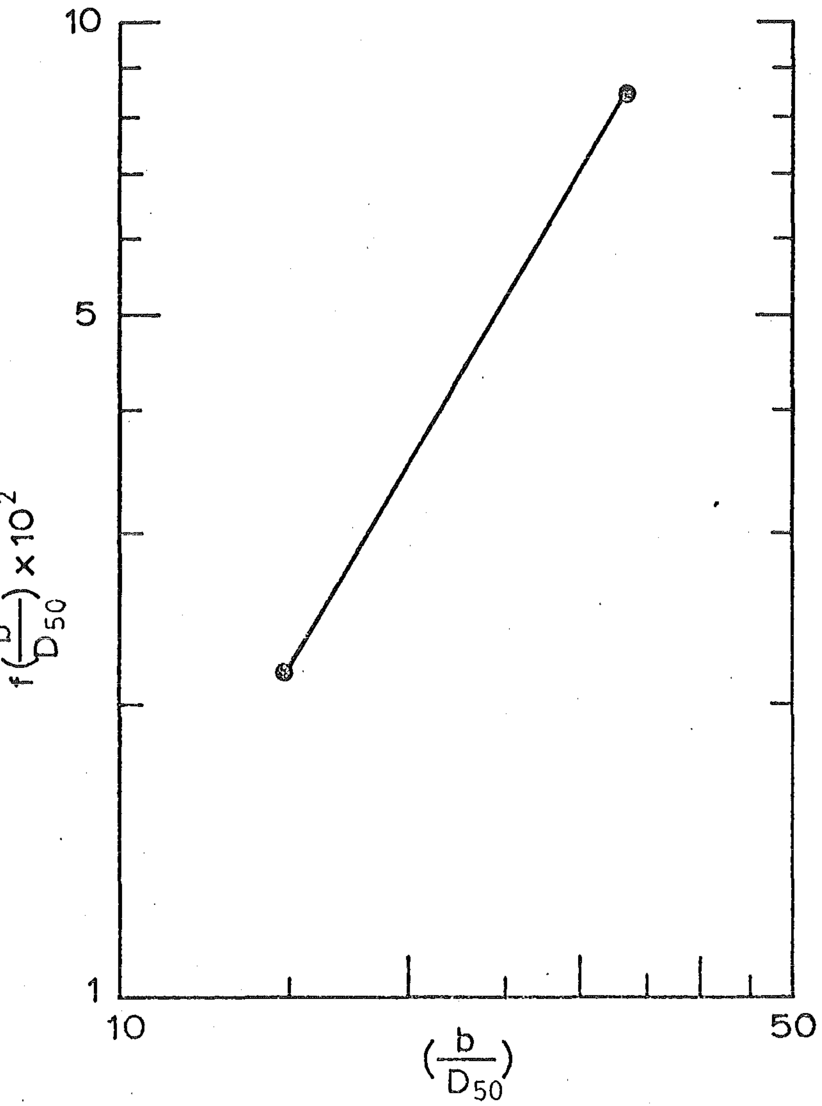


Fig. 4.19 $f(\frac{b}{D_{50}})$ AS A FUNCTION OF $\frac{b}{D_{50}}$

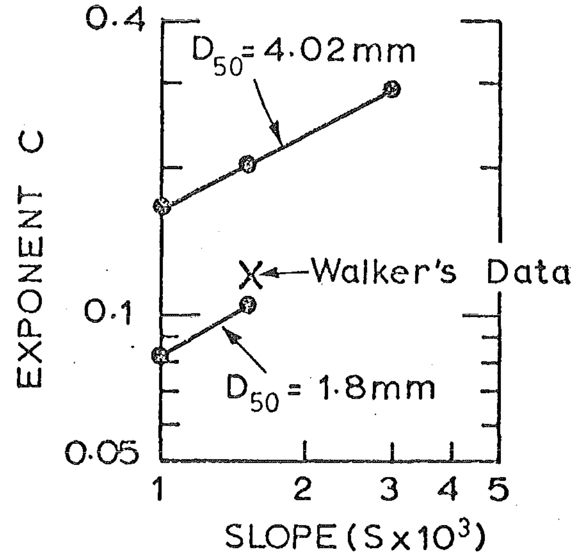


Fig.4·20 C AS A FUNCTION OF SLOPE.

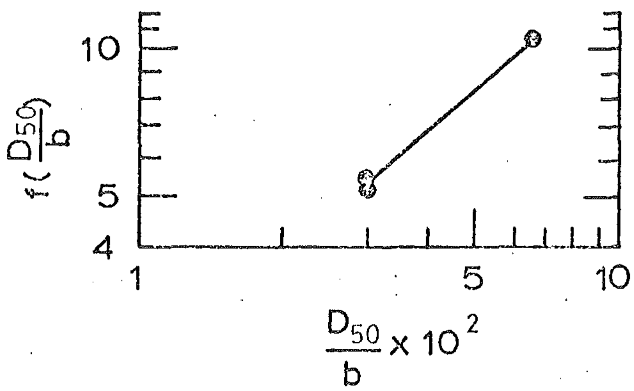


Fig.4·21 $f\left(\frac{D_{50}}{b}\right)$ AS A FUNCTION OF $\frac{D_{50}}{b}$.

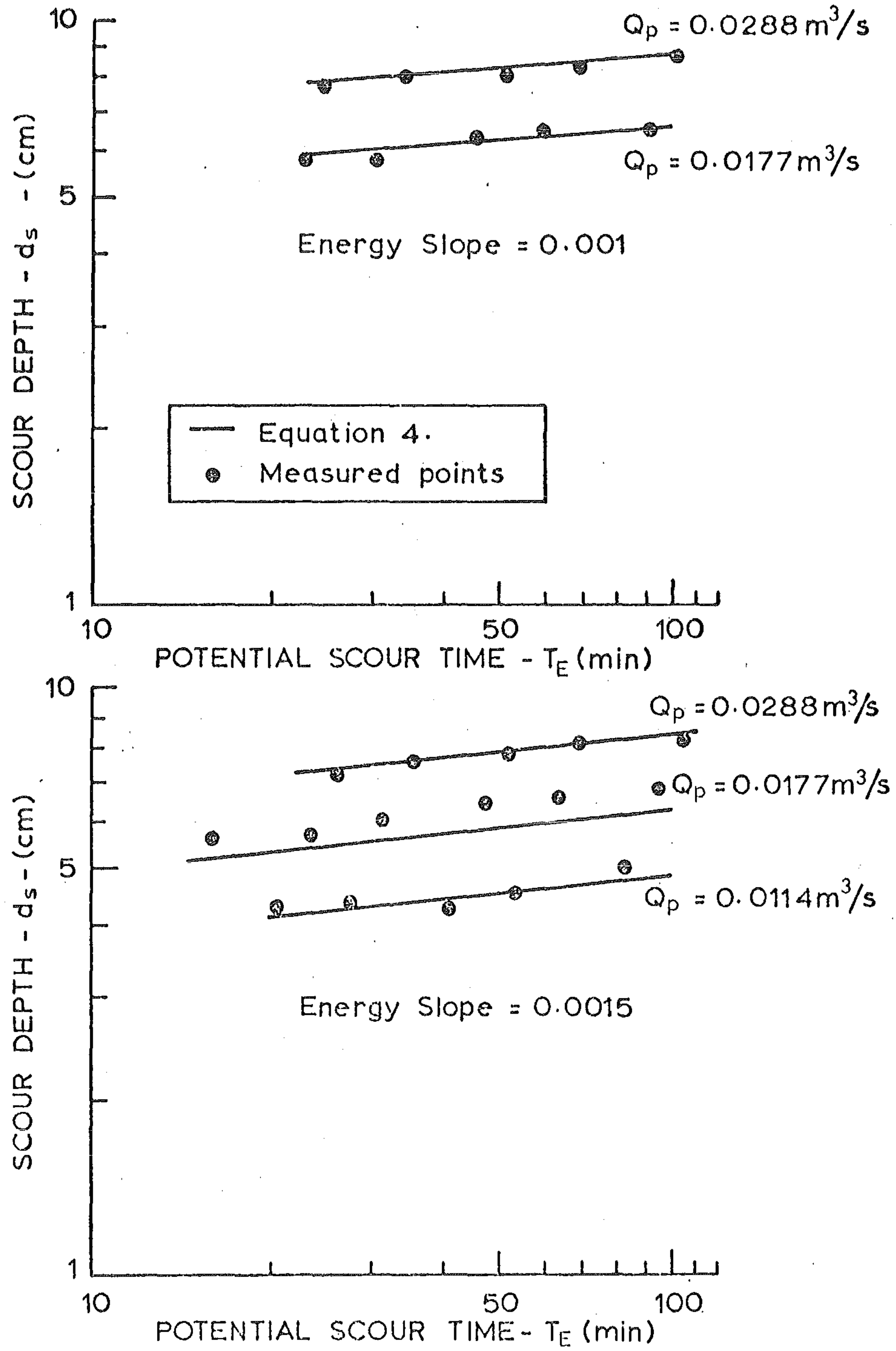


Fig.4.22 EQUATION 4.4 FOR D_{50}= 1.8mm.

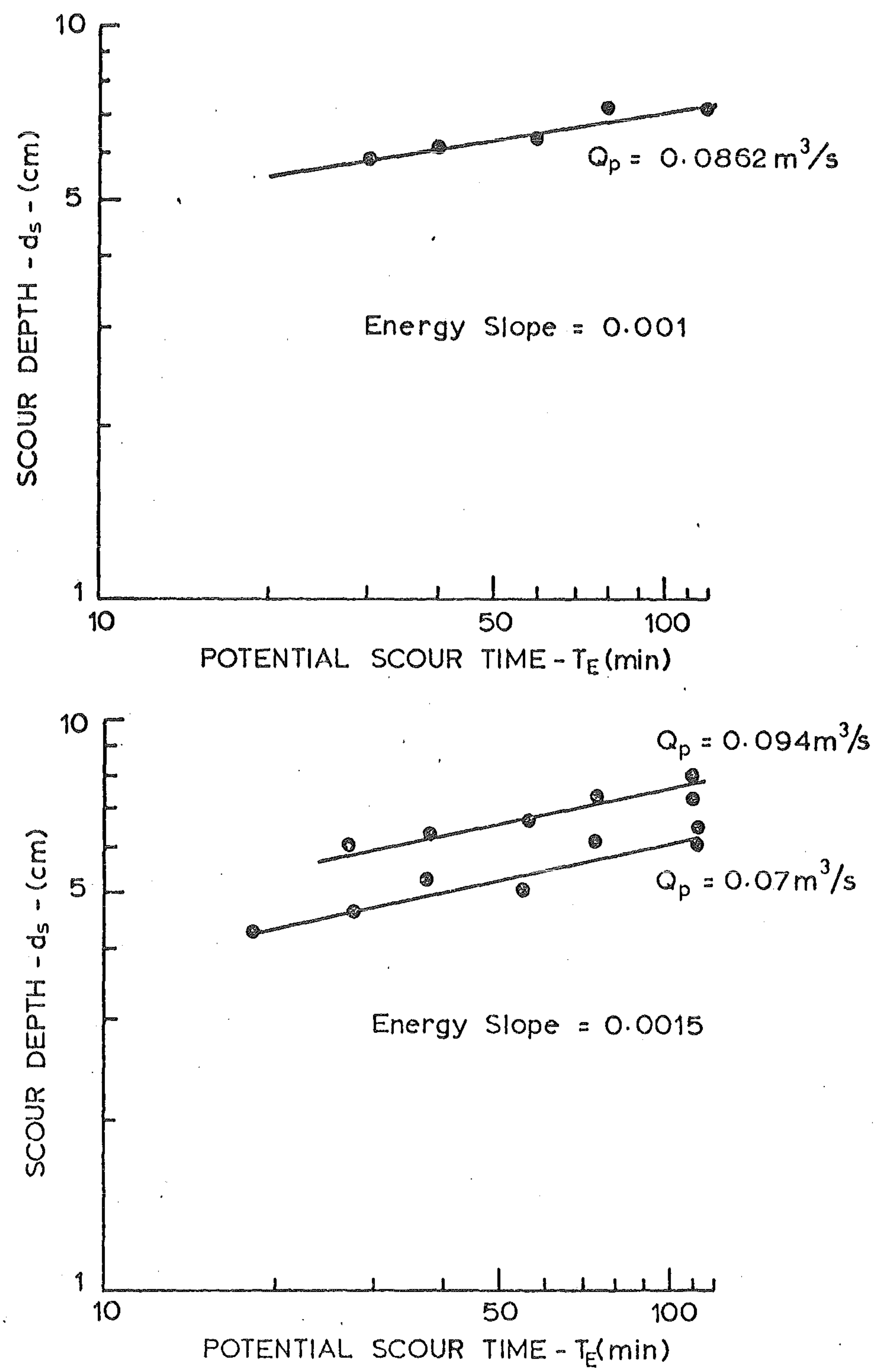


Fig.4·23 EQUATION 4·4 FOR D_{50} = 4·02mm

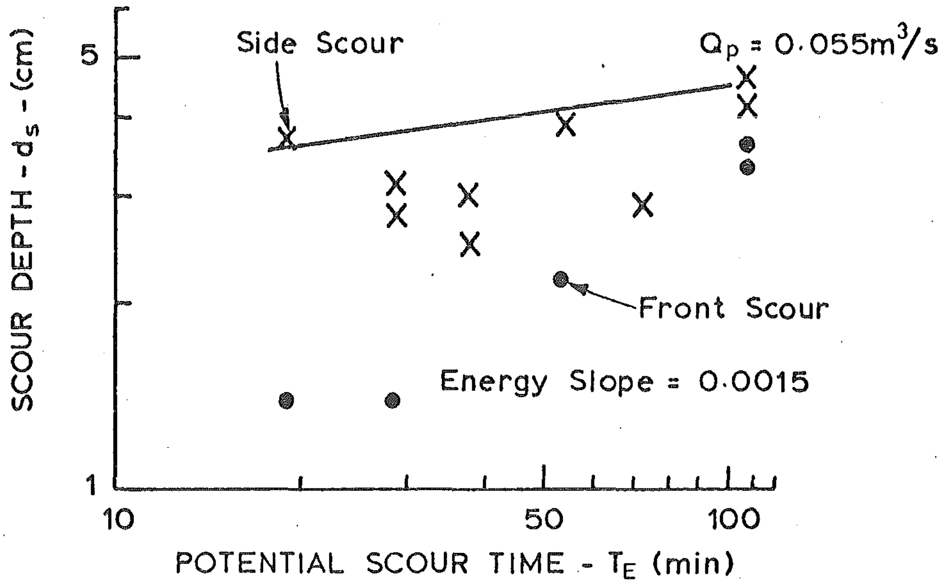


Fig.4.24 EQUATION 4.4 FOR D_{50}= 4.02mm

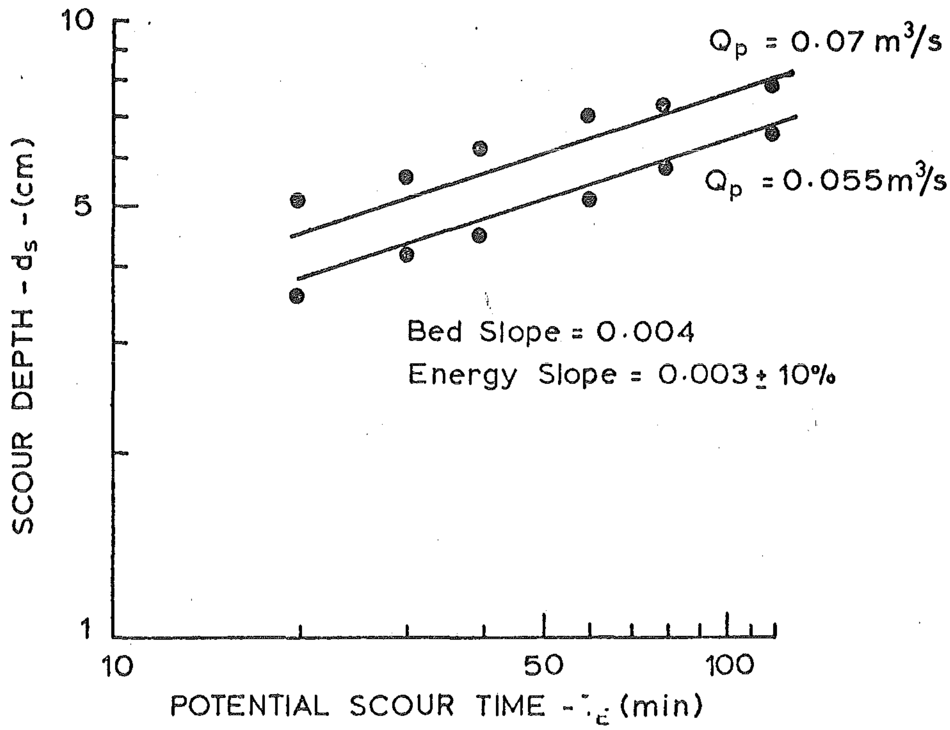


Fig.4.25 EQUATION 4.4 FOR D_{50}= 4.02mm

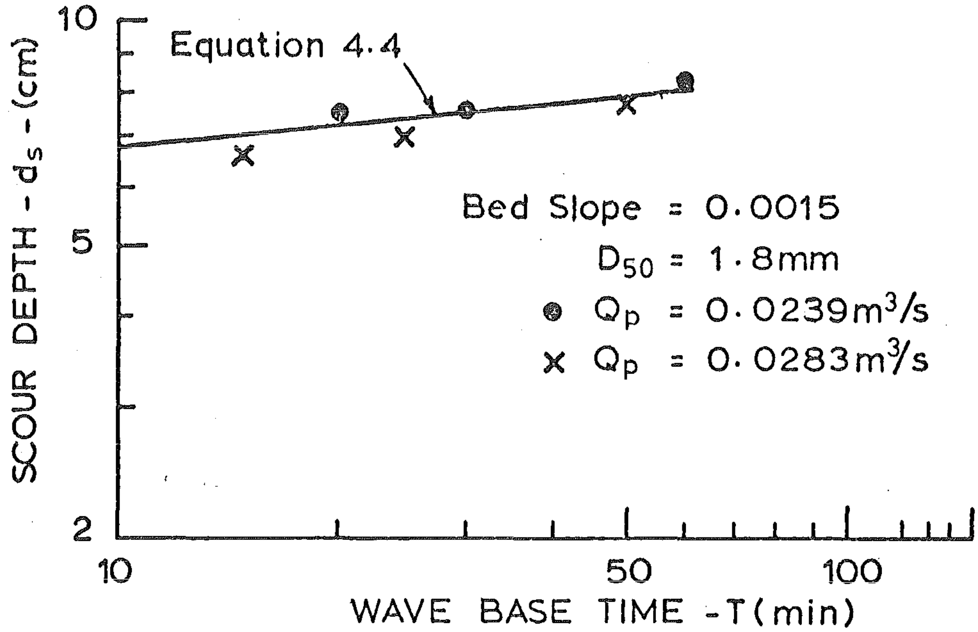


Fig. 4.26 COMPARISON OF EQUATION 4.4 AND WALKER'S DATA.

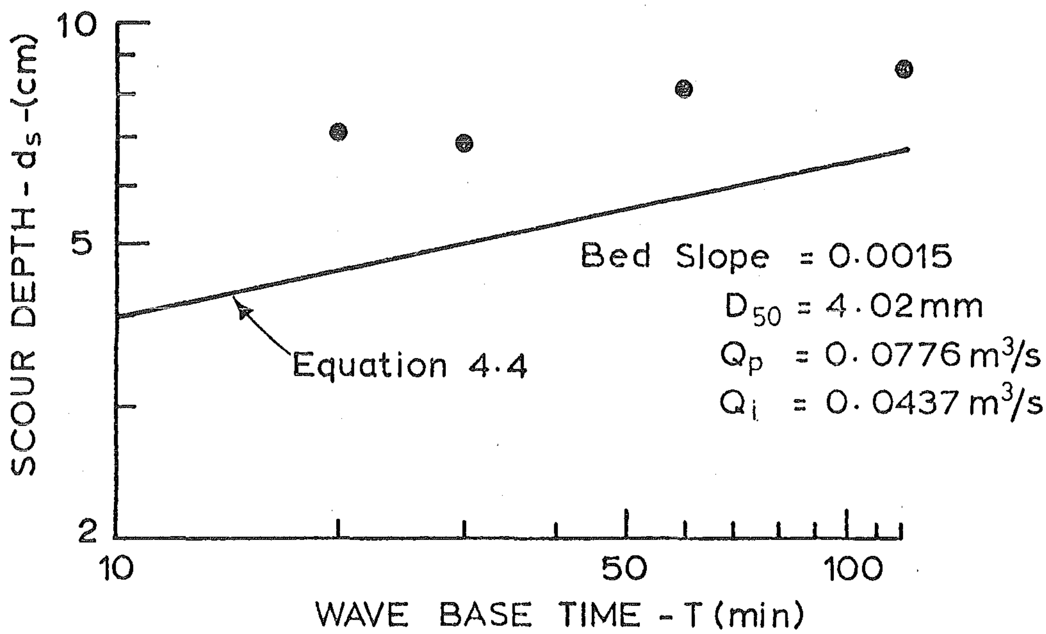


Fig. 4.27 COMPARISON OF EQUATION 4.4 AND WALKER'S DATA.
INITIAL SCOUR HOLE 5.0cm.

CHAPTER V

DISCUSSION OF RESULTS

5.1 DEVELOPMENT OF SCOUR HOLE

i. Clear Water Scour

The scouring mechanisms and the way in which the scour hole develops are the same for steady state flows and for translation waves. There is, however, a significant difference in the rates of scour hole development. Figures 4.3 and 4.4 show that in steady flows the scour hole develops quickly during the first ten minutes after which the scour depth is approximately 40% of the final scour depth. The initial scouring during a flood wave is much slower (Figures 4.6 and 4.7) because the imbalance between the scouring potential of the leading portion of the wave and the bed geometry, is not as great as in the steady flow cases which started with a step increase to the discharge being investigated.

The scour is initiated around the upstream side of the pier lying at 45° to the direction of flow (Figure 5.1). This is as observed by Carstens and Sharma (1975) but contrary to Melville's (1976) observations that scouring started at an angle of 100° to the flow where he found shear stress to be highest. The scouring next occurs at the sides of the pier and it is not until the horseshoe vortex formed around the base of the pier becomes strong enough to scour material that a hole develops at the front of the pier. The front scour depth increases more rapidly than that at the sides and except for small flow rates the former is larger when equilibrium is reached.

Particles from the base of the hole are either swept around the side of the pier or uplifted from the trench (Figure 5.2) formed around the front of the pier by the horseshoe vortex and deposited at the back of the pier. The wake behind the pier is very unsteady as evidenced by particles

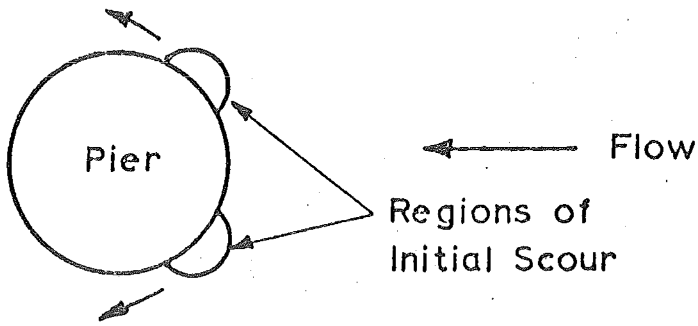


Fig. 5·1 POSITION OF INITIAL SCOUR.

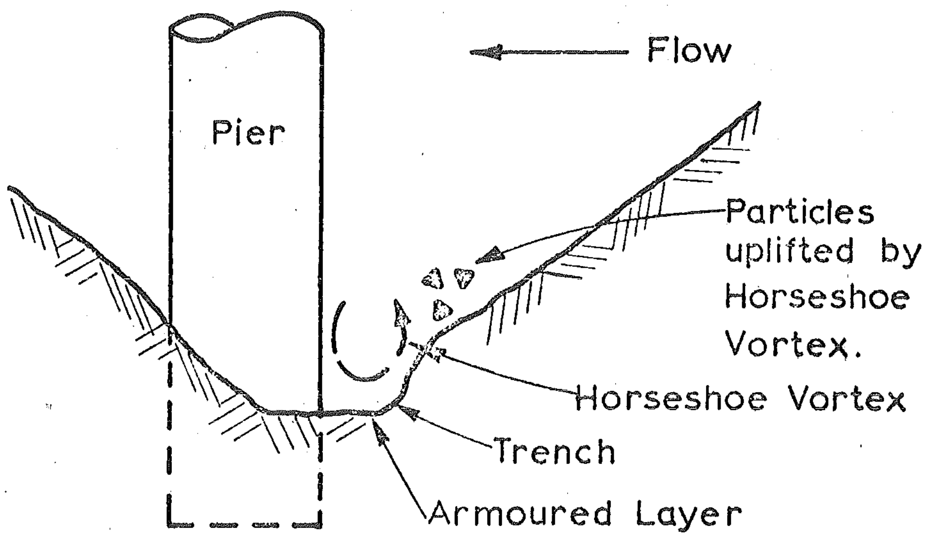


Fig. 5·2 SCOUR HOLE SHOWING ACTION OF HORSESHOE VORTEX.

being periodically lifted and tossed around at the back of the pier, (Figure 5.3). The material scoured from around the pier forms a downstream mound which moves downstream from the pier as more material is added to it, (Figure 5.4).

After a long period of time the scouring stops as the scouring mechanism becomes too weak to remove particles. There is still motion in the bottom of the trench as particles are disturbed by the flow in the trench. An armouring effect was noticeable for the 1.8 mm sediment. Larger stones could be seen forming the base of the trench in these experiments.

ii. Scour With Upstream Bed Movement

Initially the scour hole with upstream bed motion develops in the same manner as clear water scour but at a faster rate. The equilibrium depth is reached quickly and then the actual depth fluctuates about the equilibrium value as dunes pass the scour hole; the size of the fluctuations being of the order of half the dune height.

5.2 STEADY STATE EXPERIMENTS

Figures 4.1 and 4.2 show scouring to start when the velocity equals one half the critical velocity for upstream motion. Clear water scour thus occurs between the limits $V_{cr}/2$ and V_{cr} as proposed by Hancu (1971).

Data shown in Figure 4.2 are derived from two different slopes and define a single line. Thus both the critical velocity for initial motion of a particular grain size and the scour depth caused by a given velocity are relatively insensitive to the doubling of slope.

The criterion adopted herein for the limiting clear water scour depth being reached, was that there was no increase in scour depth over

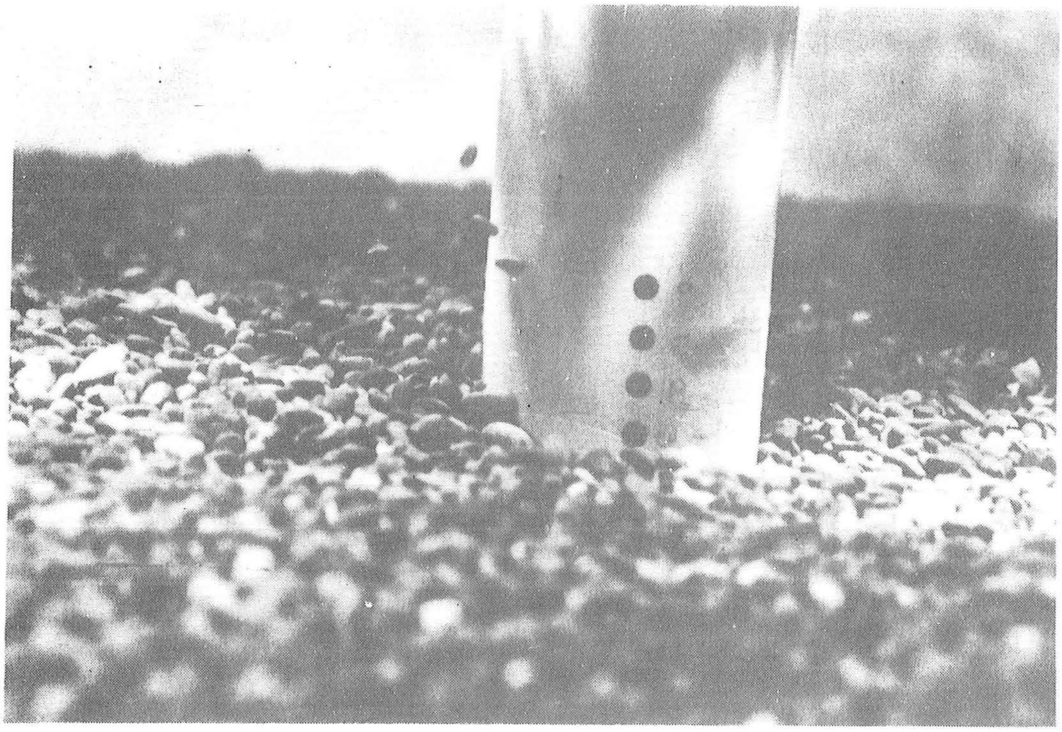


Fig. 5·3 PARTICLES UPLIFTED BY UNSTEADY WAKE.

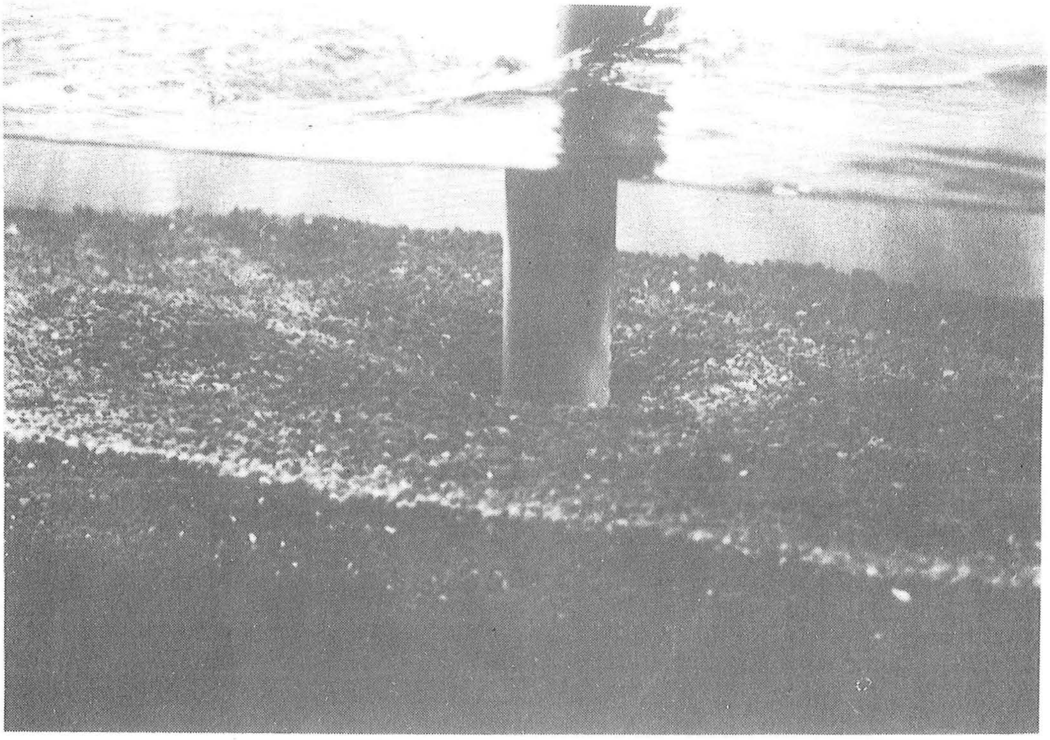


Fig. 5·4 SCOUR HOLE AND MOUND OF MATERIAL DOWNSTREAM.

a time of two hundred minutes. Values of d_s/b up to 1.85 were obtained with this criterion. Other researchers (Nicollet and Ramette (1971), Ettema (1976)), have observed values up to 2.2 with both fine and coarse grained bed materials. Both groups of researchers used a pier diameter of 100 mm which is greater than the author's (60 mm) which suggests that scour depth in clear water flows is a function of pier diameter to a power greater than 1.0.

Equilibrium scour depths are quickly reached for scour with upstream bed motion, the scour depths being independent of water velocity and sediment size as suggested by Breusers formula, equation 2.13.

5.3 PREDICTIVE EQUATION COMPARISON

i. Clear Water Scour

Shen *et al*. equation 2.1 consistently overpredicts the final scour depth for both sediments. This is to be expected as the equation was intended to form an envelope of all data. Figures 4.3 and 4.4 show that equation 2.17 fits the time development curve for the 1.8 mm bed material but not the 4.02 mm bed material. This is because Shen *et al*. (1966) used fine grained materials (maximum D_{50}= 0.46 mm) and that the durations of their experiments were 100 - 400 minutes. If Shen's experiment durations had been longer, the time development curves may well have been better approximated by a logarithmic, rather than a semilogarithmic, relationship as is the present data.

ii. Scour With Upstream Bed Movement

Table 5.1 shows a comparison between scour depths predicted by those equations recommended by Melville and chosen by the author, and actual values.

TABLE 5.1 PREDICTED AND ACTUAL SCOUR DEPTHS

(Scour depths in cm)

Equation	D_{50} = 1.8 mm Run 9	Run 11	D_{50} = 4.02 mm Run 25	Run 26
Laursen II (2.10)	9.1	11.4	9.4	11.6
Shen II (2.11)	15.0	18.3	19.9	22.0
Coleman (2.14)	8.2	8.4	8.5	8.5
Breusers (2.13)	7.8 - 9.0	7.8 - 9.0	7.8 - 9.0	7.8 - 9.0
Actual	9.0	9.2	8.4	8.4

It is clear that equation 2.13 (Breusers (1972)) fits the results best despite it's simplicity. Equation 2.11 (Shen et al. (1969)) consistently overpredicts, but as this equation was derived from experiments with fine materials it will give better results for lower velocities. Equation 2.10 (Laursen (1963)) was also derived from small grain sizes so it gives a better prediction for the 1.8 mm material. Equation 2.14 (Coleman (1971)) underpredicts for the 1.8 mm material but agrees closely with the results from the 4.02 mm material.

All equations bar equation 2.11 meet Melville's criterion (-20% to + 50% of measured scour depth) with Breusers (1972) equation 2.13 being the easiest to apply, and giving consistently accurate results.

The scour depth varied approximately by ± ½ dune height in the same manner described by Shen et al. (1969) and Walker (1975).

5.4 TRANSLATION WAVE RESULTS

i. Clear Water Scour

Most of the scour occurs on the rising limb of the flood waves, with the scour hole growing at a slower rate during the flood recession until the scouring mechanisms are too weak to remove particles from the scour hole.

Figures 4.22, 4.23, and 4.25 show that equation 4.4 fits the measured data reasonably. One limitation of Equation 4.4 is that front scour must be greater than or equal to side scour which is not the case in Figure 4.24. Figure 4.24 also shows that for low flows and a large grainsize a large scatter of results is possible. As side scour is greater than front scour for the case of low flows only, this situation is of little importance.

During a flood wave scouring starts at a higher discharge than the steady state discharge required to initiate scouring and the scour depths are always less than the corresponding steady state scour depths corresponding to the wave discharge at that time, (Figures 4.6 and 4.7). This implies that the steady state scour depths form an envelope for flood wave scour development.

ii. Scour With Upstream Bed Motion

Figure 4.13 shows that the scour depth develops quickly and equilibrium scour depth is reached in this case at the peak of the wave. Figure 4.12, which shows results from the finer bed material, shows that equilibrium scour depth can be reached before the flood peak which emphasises the rapid rate of scour hole development. As the flow decreased and the vortex weakened, the particles in suspension (Figure 5.2) dropped into the trench at the bottom of the scour hole causing slight infilling.

Figure 4.14 shows that a flood wave has no effect on the equilibrium scour depth if bed motion exists prior to the flood wave, the fluctuations in scour depth being due to the passage of dunes past the pier.

Therefore the flood waves of both types, initial conditions clear water or bed motion, have no effect on the equilibrium scour depth.

5.5 COMPARISON WITH WALKER'S FLOOD WAVE RESULTS

i. Clear Water Scour

Figure 4.26 shows that Walker's data (which was not used in the derivation of equation 4.4) for the fine grained material compared well with predictions made using equation 4.4. Data from the coarse grained material did not compare as well with prediction. A possible reason is the inherent scatter of results possible (Figure 4.24) when using the larger material.

Figure 4.27 shows that if an initial scourhole formed by the initial flow exists, then equation 4.4 underpredicts the scour depth. This suggests that a series of flood waves could deepen the scour hole caused by one flood wave.

ii. Scour With Upstream Bed Movement

Walker found that a flood wave had no effect on the equilibrium scour depth if upstream bed motion existed prior to the flood wave. This result has been confirmed by the author (Figure 4.14) for both bed materials.

CHAPTER VI

CONCLUSION

This study was designed to extend an earlier study (Walker 1975)) on the effects of a flood wave passing a bridge pier.

For clear water scour Walker's (1975) observations have been confirmed using a wider range of flood hydrographs. These observations were that:

i. The scoured depths formed by the flood wave are less than those caused by corresponding steady state flows.

ii. Scour depths increase when the flood wave is receding until the scouring mechanisms are too weak to remove particles from the scour hole.

An empirically derived formula to predict final scour depths resulting from the passage of a clear water wave has been presented. The formula adequately describes the author's data and Walker's data for his experiments with fine grained material.

Walker's (1975) observation that when bed load transport is fully developed before a wave then no change occurs in the equilibrium scour depth during a wave was confirmed.

It was also observed that if a wave started in clear water conditions and had peak flows capable of causing upstream bed motion, the steady state equilibrium depth would be reached. If the wave was long enough the scouring could stop before the wave peak. This implies that if bed motion exists then steady state formula give reasonable predictions, far more so than equation 2.1 (Shen *et al.*(1966)) does for clear water scour.

For clear water scour if an initial scour hole existed the prediction formula underestimated the scour depth caused by the flood wave.

Further research along the following lines is recommended to extend the author's findings:

1. Extend the range of data (base times, slopes, material size, and pier diameter) to give a more adequate foundation to the predictive equation 4.4 for clear water scour. It is essential to change the pier diameter so that the role of the parameter d_s/b can be explored.

2. Extend the range of data for continuous bed motion experiments (steady and non steady) for higher rates of sediment transport.

3. Investigate the effect of a series of flood waves on scour hole development to determine if significant changes in depth occur.

4. Change the contraction ratio of the pier diameter to flume width to study the effect of vortex suppression by the flume walls. (See Appendix I).

5. Investigate the effects of ground water flow on scour hole development and final scour depths for steady flows. (See Appendix II).

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APPENDIX I

CHOICE OF FLUME WIDTH

Shen et al. (1966) recommended a minimum ratio of pier diameter to flume width of 1.8. Neil (1975), as a result of experiments with spur dykes, suggests if the contraction ratio for a spur dyke is too large then the development of local scour is suppressed.

Figures I.1 and 1.2 show a scour hole of dimensionless scour depth $d_s/b = 2.0$ in a bed material with an angle of repose of 30°. In Figure I.1 the contraction ratio is too high as the flume walls will effect the scour hole development. Figure I.2 shows that a contraction ratio of 1:10 is better but as scour depths of the order $d_s/b = 2.2$ have been reached (Ettema (1976) contraction rate = 1.15) a smaller contraction ratio would be desirable.

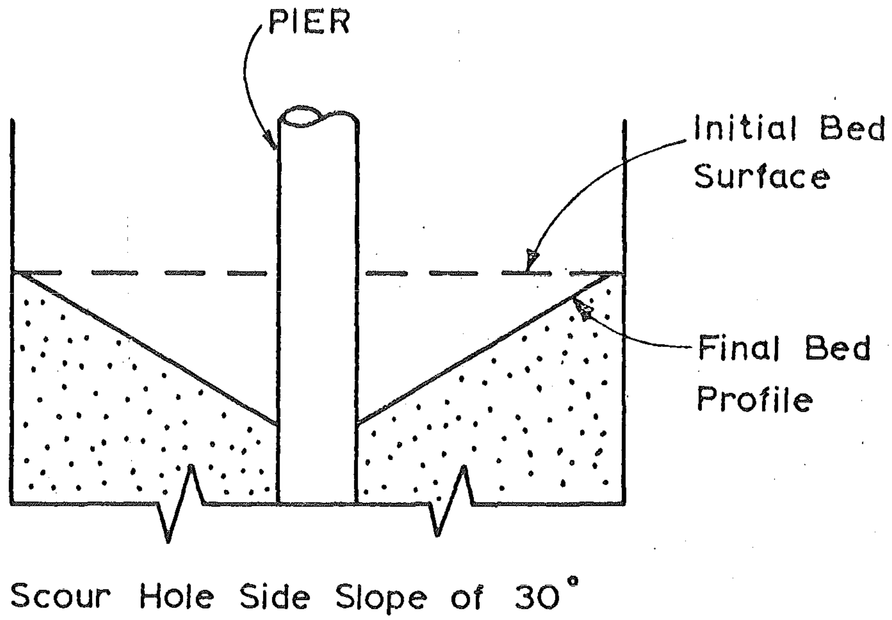


Fig.I·1 CROSS SECTION OF FLUME SHOWING CONTRACTION RATIO 1:8

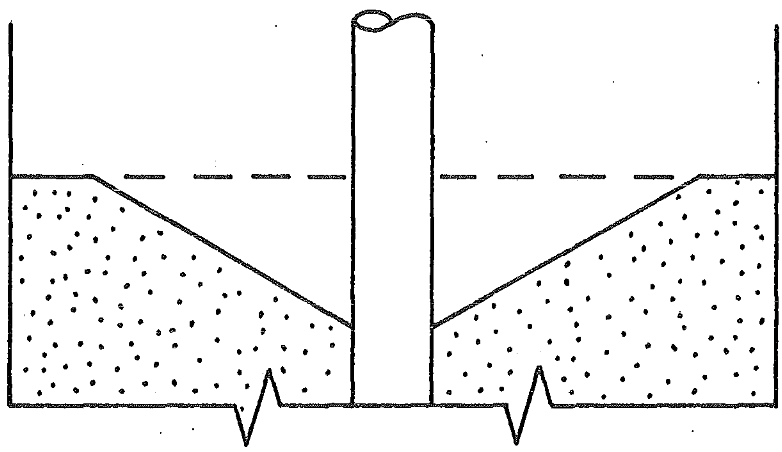
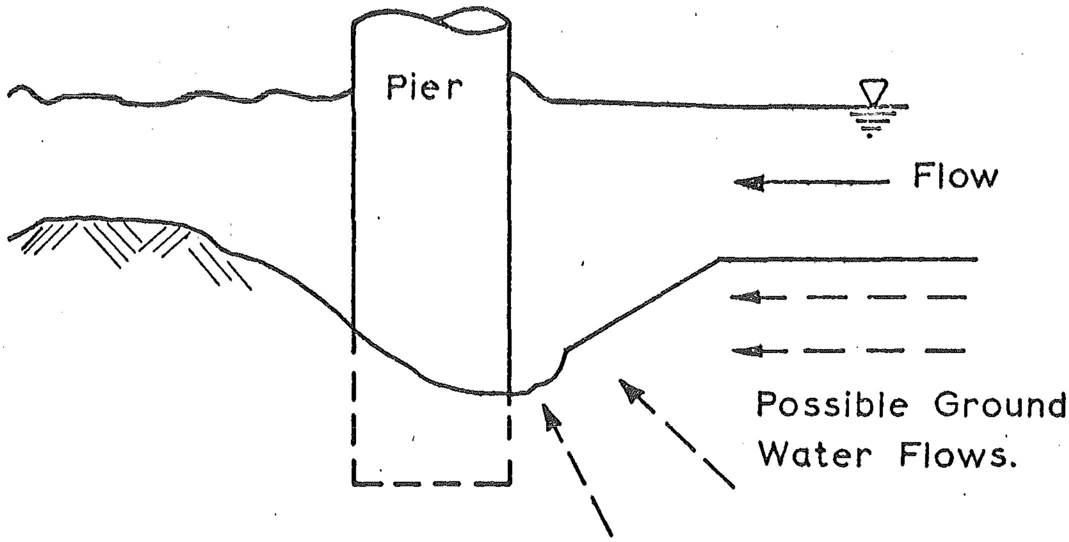


Fig.I·2 CROSS SECTION OF FLUME SHOWING CONTRACTION RATIO 1:10

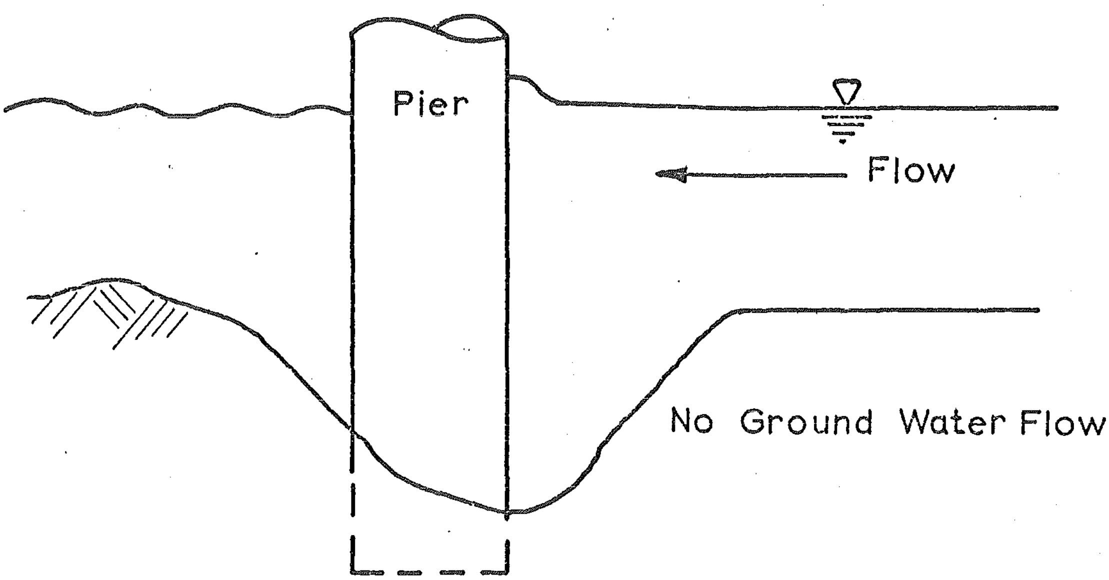
APPENDIX II

POSSIBLE EFFECT OF GROUND WATER FLOW

In the past, laboratory experiments investigating the effects of local scour have differed from the river situation in that there is no flow through the bed in the laboratory situation (Figure II.1). During floods ground water flows can be surprisingly large so a method of investigating possible effects of ground water flow (change in time development of scour, change in final scour depths for steady flows) is given in Figure II.2.

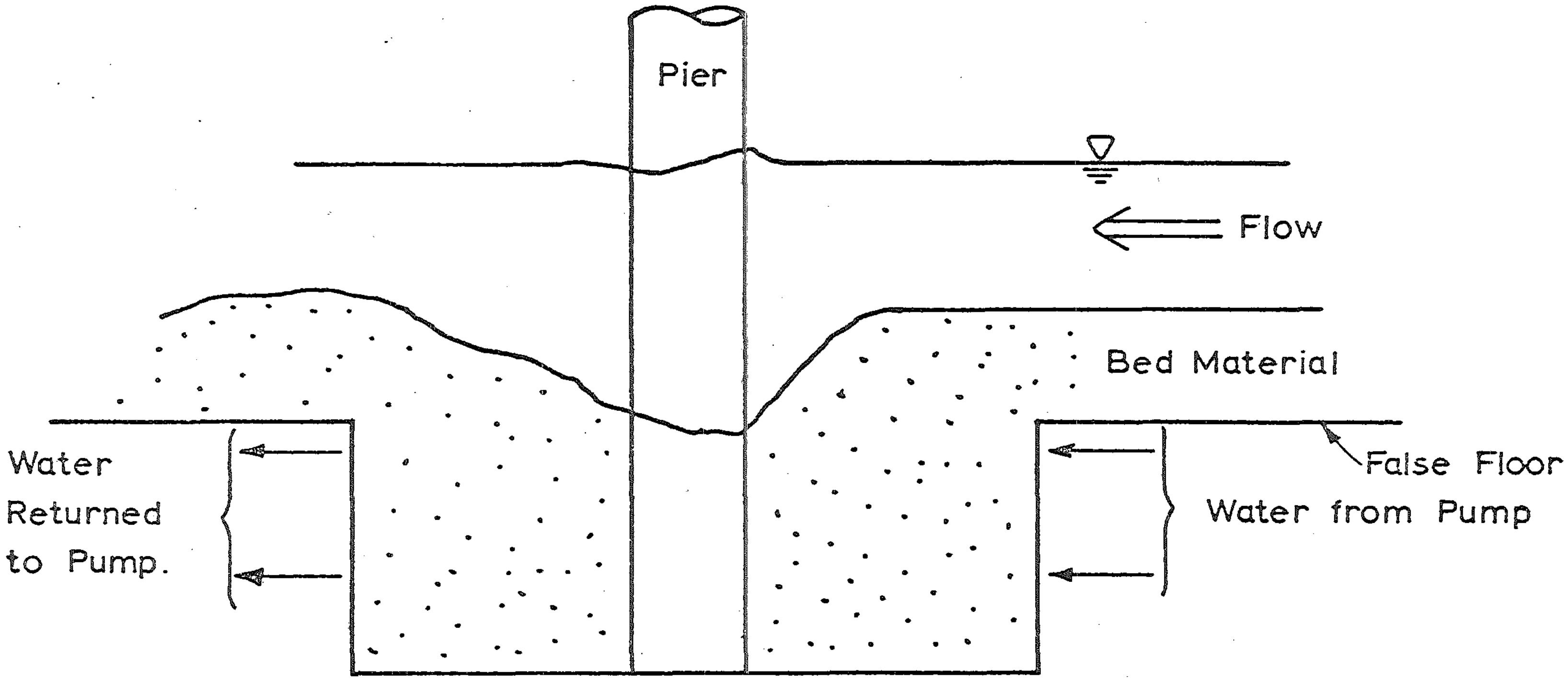


RIVER SITUATION



LABORATORY SITUATION

Fig.II·1 COMPARISON OF LABORATORY AND RIVER SITUATIONS.



METHOD: After Establishing Groundwater Flow, Run Flow in Flume.

Fig. II·2 LABORATORY METHOD FOR INVESTIGATING EFFECT OF GROUNDWATER FLOW.

APPENDIX III

FINAL RESULTS OF ALL EXPERIMENTS

TABLE i. Clear Water Steady State Runs - D_{50} = 1.8 mm, S = 0.001

Run No.	Fluid Temp. (^{o}C.)	Time (Min.)	Discharge Q (m^3/s)	Fluid Depth y (cm)	Mean Fluid Velocity V (m/s)	Front Scour Depth d_s (cm)	Side Scour Depth d_{ss} (cm)
1	19.6 - 22.0	900	0.0055	4.0	0.229	0.8	2.2
2	19.8 - 20.2	1245	0.0058	4.06	0.238	1.2	2.0
3	19.4 - 20.0	1600	0.0070	4.51	0.257	3.4	3.4
4	19.0 - 20.4	1320	0.0092	5.21	0.294	6.4	5.2
5	19.4 - 20.0	1350	0.0114	6.00	0.316	7.2	6.2
6	18.0 - 20.4	1440	0.0141	6.75	0.348	8.8	8.0
7*	18.8 - 20.8	3400	0.0176	7.45	0.393	10.6	9.6

TABLE ii. Scour With Upstream Bed Movement - D_{50} = 1.8 mm, S = 0.001

8	19.6 - 21.0	1920	0.0285	10.79	0.440	10.8	10.2
9	19.4	60	0.0340	11.70	0.484	9.0	8.4
10*	19.0	60	0.0467	13.39	0.581	9.0	8.2
11*	19.0	74	0.0710	18.12	0.653	9.2	8.2

TABLE iii. Clear Water Steady State Scour - D_{50} = 4.02 mm. (Side Scour Depths not Recorded).

Run No.	Fluid Temp. (^{o}C.)	Time (Min.)	Discharge Q (m^3/s)	Fluid Depth (cm)	Mean Fluid Velocity V (m/s)	Bed Slope S	Front Scour Depth d_s (cm)
12	18.0 - 20.6	1700	0.029	10.17	0.478	0.0015	3.1
13	18.0 - 19.0	1400	0.039	12.80	0.516	0.0015	5.0
14	20.0 - 20.6	550	0.041	12.20	0.560	0.0015	5.2
15	18.4 - 20.2	1240	0.047	14.09	0.554	0.0015	6.5
16*	15.0 - 20.0	1700	0.051	13.90	0.606	0.0015	9.4
17*	20.0 - 21.0	1500	0.058	15.50	0.626	0.0015	10.0
18*	20.6 - 24.0	1640	0.075	17.50	0.717	0.0015	9.9
19	18.8 - 21.0	1980	0.075	17.57	0.713	0.0015	10.4
20	19.2 - 21.0	2940	0.093	21.20	0.731	0.0015	11.5
21	16.0 - 18.2	960	0.0158	6.60	0.395	0.004	0.2
22	17.0 - 19.2	735	0.021	7.71	0.454	0.004	2.4
23	15.2 - 19.4	1065	0.029	9.02	0.525	0.004	4.2
24	15.4 - 18.2	1380	0.039	10.88	0.597	0.004	7.5

TABLE iv. Scour With Upstream Bed Movement - D_{50} = 4.02 mm

25	19.4 - 21.0	1350	0.055	12.40	0.740	0.004	8.4
26	17.4 - 20.0	1200	0.096	18.70	0.855	0.004	8.4

* Indicates detailed time development records on file in Civil Engineering Department.

TABLE v. Clear Water Translation Waves - D_{50} = 4.02 mm, Bed Slope = 0.0015.

Run No.	Temp. (^{o}C.)	Initial Discharge Q_i (m^3/s)	Peak Discharge Q_p (m^3/s)	Base Time T (Min.)	Potential Scour Time T_E (Min.)	Max'm Front Scour d_s (cm)	Time Front Scour Stops (Min.)	Max'm Side Scour d_{ss} (cm)	Time Side Scour Stops (Min.)
27	17.6	0.01	0.055	10	9.1	1.6	8	3.7	7
28	17.6	0.01	0.055	10	9.1	1.6	6	3.7	6
29	17.6	0.01	0.055	20	18.2	1.4	6	3.7	11
30	15.6	0.01	0.055	30	27.3	1.4	17	2.8	17
31	15.6	0.01	0.055	30	27.3	1.4	17.5	3.2	16
32	17.6	0.01	0.055	40	36.4	2.5	22	3.0	22
33	16.5	0.01	0.055	60	54.6	2.2	30.5	3.7	32
34	16.0	0.01	0.055	80	72.9	2.9	51	2.9	42
35	15.6	0.01	0.055	120	109.3	3.6	64	4.7	64
36	15.6	0.01	0.055	120	109.3	3.4	75	4.2	70
37	17.8	0.01	0.07	10	9.42	2.6	6	3.4	6
38	17.8	0.01	0.07	20	18.85	4.3	12	4.6	12
39	17.8	0.01	0.07	30	28.28	4.6	21	4.8	17
40	17.5	0.01	0.07	40	37.7	5.3	25	5.7	26
41	17.8	0.01	0.07	40	37.7	5.2	23	5.1	21
42	17.8	0.01	0.07	40	37.7	5.2	21	5.2	25

Run No.	Temp. ($^\circ$C.)	Initial Discharge Q_i (m^3/s)	Peak Discharge Q_p (m^3/s)	Base Time T (Min.)	Potential Scour Time T_E (Min.)	Max'm Front Scour d_s (cm)	Time Front Scour Stops (Min.)	Max'm Side Scour d_{ss} (cm)	Time Side Scour Stops (Min.)
43	-	0.01	0.07	60	56.57	5.1	44	5.9	33
44	17.6	0.01	0.07	80	75.4	6.1	47	5.8	58
45	18.0	0.01	0.07	120	113.1	6.1	72	6.1	74
46	-	0.01	0.07	120	113.1	6.5	63	6.1	65
47	20.6	0.01	0.094	10	9.58	4.4	6	5.2	7
48	20.6	0.01	0.094	20	19.15	5.2	12	6.0	10
49	-	0.01	0.094	30	28.75	6.2	22	6.0	17
50	20.0	0.01	0.094	40	38.3	6.3	26	5.4	19
51	19.8	0.01	0.094	60	57.5	6.6	36	5.2	32
52	18.8	0.01	0.094	80	76.6	7.2	51	6.8	46
53	18.6	0.01	0.094	120	114	7.2	71	7.0	62
54	20.0	0.01	0.094	120	114	8.0	70	7.2	70

TABLE vi. Clear Water Translation Waves - D_{50} = 4.02 mm, Bed Slope = 0.004

55	-	0.01	0.055	10	10	3.3	5.5	4.0	6.0
56	20.4	0.01	0.055	20	20	3.5	12.0	3.9	11.0
57	20.0	0.01	0.055	30	30	4.8	18.0	5.0	16.0
58	18.2	0.01	0.055	30	30	4.2	16.75	4.8	18
59	19.6	0.01	0.055	40	40	4.5	25	5.0	21
60	19.4	0.01	0.055	60	60	5.0	32	5.8	29

Run No.	Temp. (^{o}C.)	Initial Discharge Q_i (m^3/s)	Peak Discharge Q_p (m^3/s)	Base Time T (Min.)	Potential Scour Time T_E (Min.)	Max'm Front Scour d_s (cm)	Time Front Scour Stops (Min.)	Max'm Side Scour d_{ss} (cm)	Time Side Scour Stops (Min.)
61	18.0	0.01	0.055	80	80	5.5	40	6.0	40
62	19.8	0.01	0.055	120	120	6.5	68	6.2	66
63	19.8	0.01	0.055	120	120	6.5	70	5.8	59
64	16.7	0.01	0.070	10	10	4.2	6	4.0	5.5
65	16.6	0.01	0.070	20	20	5.3	15	5.8	11
66	16.8	0.01	0.070	30	30	5.6	19	6.2	16
67	16.4	0.01	0.070	40	40	6.2	27	6.5	30
68	16.4	0.01	0.070	60	60	7.0	35	6.8	35
69	17.0	0.01	0.070	80	80	7.3	43	6.3	47
70	19.6	0.01	0.070	120	120	7.8	71	7.2	64

TABLE vii. Clear Water Translation Waves - D_{50} = 4.02, Bed Slope = 0.001

71	17.0	0.02	0.086	30	30	5.8	22	6.0	22
72	19.0	0.02	0.086	40	40	6.2	24	5.2	32
73	17.4	0.02	0.086	60	60	6.4	36	6.2	32
74	18.0	0.02	0.086	80	80	7.2	53	5.8	45
75	19.0	0.02	0.086	120	120	7.2	75	6.2	75

TABLE viii. Clear Water Translation Waves - D_{50} = 1.8, Bed Slope = 0.0015

Run No.	Temp. (^{o}C.)	Initial Discharge Q_i (m^3/s)	Peak Discharge Q_p (m^3/s)	Base Time T (Min.)	Potential Scour Time T_E (Min.)	Max'm Front Scour d_s (cm)	Time Front Scour Stops (Min.)	Max'm Side Scour d_{ss} (cm)	Time Side Scour Stops (Min.)
76	20.0	0.0	0.0114	20	13.8	3.0	13	3.2	13
77	18.8	0.0	0.0114	30	20.8	4.3	20	3.6	20
78	-	0.0	0.0114	40	27.7	4.3	30	4.0	22
79	18.0	0.0	0.0114	60	41.6	4.3	39	4.0	39
80	49.4	0.0	0.0114	80	55.4	4.5	54	3.9	41
81	19.0	0.0	0.0114	120	83.2	5.0	64	4.2	64
82	18.8	0.0	0.0177	10	8.0	4.8	-	-	-
83	18.8	0.0	0.0177	10	8.0	5.6	-	-	-
84	18.4	0.0	0.0177	20	16.0	5.6	12	5.0	12
85	18.2	0.0	0.0177	30	24.0	5.6	16	5.0	20
86	18.0	0.0	0.0177	40	32	6.0	22	5.2	28
87	-	0.0	0.0177	60	48	6.4	36.5	5.6	36
88	18.4	0.0	0.0177	80	64.2	6.6	48	5.4	50
89	19.0	0.0	0.0177	120	96.2	6.8	68	6.0	68
90	18.0	0.0	0.0288	10	8.8	6.4	8.25	5.8	7.0
91	17.4	0.0	0.0288	20	17.6	7.0	13	6.2	13
92	17.0	0.0	0.0288	30	26.4	7.2	21	6.8	16
93	17.2	0.0	0.0288	40	35.1	7.6	21	6.6	23
94	17.4	0.0	0.0288	60	52.7	7.8	29	7.0	31

Run No.	Temp. (^{o}C.)	Initial Discharge Q_i (m^3/s)	Peak Discharge Q_p (m^3/s)	Base Time T (Min.)	Potential Scour Time T_E (Min.)	Max'm Front Scour d_s (cm)	Time Front Scour Stops (Min.)	Max'm Side Scour d_{ss} (cm)	Time Side Scour Stops (Min.)
95	17.8	0.0	0.0288	80	70.3	8.1	45	7.1	41
96	18.0	0.0	0.0288	120	105.4	8.3	64	7.3	56
<u>TABLE ix.</u>	<u>Clear Water Translation Waves</u> - D_{50} = 1.8 mm, Bed Slope = 0.001								
97	19.5	0.0	0.0177	30	23.2	5.8	18.5	4.8	18.5
98	-	0.0	0.0177	30	23.2	5.9	21.5	5.0	19.0
99	18.8	0.0	0.0177	40	30.9	5.8	28	5.0	26
100	20.2	0.0	0.0177	40	30.9	5.9	25	-	-
101	-	0.0	0.0177	60	46.4	6.3	36	5.3	50
102	19.6	0.0	0.0177	80	61.9	6.4	44	5.2	42
103	19.0	0.0	0.0177	120	92.9	6.5	80	5.6	83
104	19.2	0.0	0.0288	30	25.8	7.7	22	6.8	20
105	19.4	0.0	0.0288	40	34.4	5.0	25	7.0	25
106	20.6	0.0	0.0288	60	51.7	8.1	35	7.4	35
107	19.6	0.0	0.0288	80	68.9	8.3	50	7.4	55
108	-	0.0	0.0288	120	103.3	8.7	70	7.6	66
<u>TABLE x.</u>	<u>Translation Waves With Upstream Bed Movement</u> - D_{50} = 4.02, Bed Slope = 0.004								
109	16.4	0.010	0.094	10	10	6.8	5	6.8	5
110	16.4	0.010	0.094	20	20	7.2	11	6.8	11
111	16.8	0.010	0.094	30	30	7.3	16	7.2	17

Run No.	Temp. (°C.)	Initial Discharge Q_i (m^3/s)	Peak Discharge Q_p (m^3/s)	Base Time T (Min.)	Potential Scour Time T_E (Min.)	Front Scour d_{se} (cm)	Time Front Scour Stops (Min.)	Side Scour d_{sse} (cm)	Time Side Scour Stops (Min.)
112	17.4	0.010	0.094	40	40	7.6	24	7.6	21
113	17.8	0.010	0.094	60	60	7.7	34	7.5	32
114	16.2	0.010	0.094	60	60	8.4	34	8.0	29
115	-	0.010	0.094	80	80	8.2	53	8.2	53
116	20.2	0.010	0.094	120	120	8.4	60	8.0	62
117	20.6	0.010	0.094	120	120	8.0	60	7.9	65

TABLE xi. Translation Waves With Upstream Bed Movement - D_{50} = 1.8 mm, Bed Slope = 0.001

118	19.6	0.0	0.0467	20	17.9	8.0	12	6.8	10
119	20.0	0.0	0.0467	30	26.8	8.3	26	7.4	20.5
120	19.6	0.0	0.0467	40	35.7	9.0	24	8.0	24
121	19.0	0.0	0.0467	60	53.6	9.0	40	8.0	38
122	20.0	0.0	0.0467	80	71.4	9.0	40	8.0	40
123	19.6	0.0	0.0467	120	107.2	9.2	61	8.2	58
124	18.0	0.0467	0.0710	40	40	9.2	-	8.2	-
125	18.4	0.0467	0.0710	60	60	9.2	-	8.2	-
126	19.0	0.0467	0.0710	120	120	9.2	-	8.2	-

APPENDIX IV

SCOUR DEVELOPMENT AND WAVE PROPERTIES

i. Bed Properties: $D_{50} = 1.8$ mm, $s = 0.001$

Wave Properties:

$Q_i = 0.0\ m^3/s$	$Q_i = 0.0\ m^3/s$
$Q_p = 0.0177\ m^3/s$	$Q_p = 0.0288\ m^3/s$
$Y_p = 0.0745$ m	$Y_p = 0.0994$ m
$\frac{d_s}{Y_p} = 0.61\ T_E^{0.08}$	$\frac{d_s}{Y_p} = 0.60\ T_E^{0.08}$

$$\text{average } \frac{d_s}{Y_p} = 0.60\ T_E^{0.08}$$

---

ii. Bed Properties: $D_{50} = 1.8$ mm, $s = 0.0015$

Wave Properties:

$Q_i = 0.0\ m^3/s$	$Q_i = 0.0\ m^3/s$	$Q_i = 0.0\ m^3/s$
$Q_p = 0.0114\ m^3/s$	$Q_p = 0.0177\ m^3/s$	$Q_p = 0.0288\ m^3/s$
$Y_p = 0.056$ m	$Y_p = 0.072$ m	$Y_p = 0.097$ m
$\frac{d_s}{Y_p} = 0.54\ T_E^{0.10}$	$\frac{d_s}{Y_p} = 0.56\ T_E^{0.11}$	$\frac{d_s}{Y_p} = 0.57\ T_E^{0.106}$

$$\text{average } \frac{d_s}{Y_p} = 0.543\ T_E^{0.105}$$

---

iii. Bed Properties: $D_{50} = 4.02$ mm, $s = 0.001$

Wave Properties:

$Q_i = 0.0\ m^3/s$ $\quad Q_p = 0.0862\ m^3/s$ $\quad Y_p = 0.210$ m

$$\frac{d_s}{Y_p} = 0.159\ T_E^{0.210}\ m$$

---

iv. Bed Properties: $D_{50} = 4.02$ mm, $s = 0.0015$

Wave Properties:

$Q_i = 0.01\ m^3/s$	$Q_i = 0.01\ m^3/s$
$Q_p = 0.07\ m^3/s$	$Q_p = 0.094\ m^3/s$
$Y_p = 0.170$ m	$Y_p = 0.212$ m
$\frac{d_s}{Y_p} = 0.138\ T_E^{0.21}$	$\frac{d_s}{Y_p} = 0.141\ T^{0.20}$

average $\frac{d_s}{Y_p} = 0.14\ T^{0.205}$

---

v. Bed Properties: $D_{50} = 4.02$ mm $s_{BED} = 0.004$

Energy Slope $= 0.003 \pm 10\%$

Wave Properties:

$Q_i = 0.01\ m^3/s$	$Q_i = 0.01\ m^3/s$
$Q_p = 0.055\ m^3/s$	$Q_p = 0.07\ m^3/s$
$Y_p = 0.130$ m	$Y_p = 0.152$ m
$\frac{d_s}{Y_p} = 0.125\ T_E^{0.29}$	$\frac{d_s}{Y_p} = 0.138\ T_E^{0.29}$

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vi Figures 4.26 and 4.27 show data from Walker's experiments (1975) that have been compared with equation 4.4 but not used in equation's 4.4 derivation.

v.ii Equation 4.4 gives d_s in the units of meters if T_E is expressed in minutes.

Classn:

LOCAL SCOUR AT A BRIDGE PIER CAUSED BY FLOOD WAVES.

N. J. HARWOOD

Abstract: A continuation of previous work (Walker (1975)) is presented. An empirical formula for clear water flood wave scour depths involving peak flow depth, wave times, pier and grain diameters, and energy slope is given. For flood waves with upstream bed motion, steady state formulae give acceptable results.

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