Hydraulics Research
Wallingford

ALLUVIAL PROCESSES: PHYSICAL PROCESSES

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ABSTRACT

This report describes work that was undertaken as part of a research programme to improve the understanding of the physics of sediment - related phenomena. The investigations reported here have dealt with frictional resistance and local scour.

Desk and laboratory studies have been carried out to investigate the effect of a sand-silt mixture and also of bed features on alluvial friction. Two distinct relationships have been observed, one for lower regime and one for upper regime.

Desk and laboratory studies have been carried out to investigate the development of local scour with time. This has shown the importance of the flow geometry to model tests and also that later scour may be predicted from initial scour development.

CONTENTS

1 INTRODUCTION

This report describes work partly supported under Contract PECD 7/7/29-204/83, funded by the Department of the Environment. This was part of a programme of work designed to:

(a) predict the hydraulic performance of certain engineering structures in rivers and their consequences to the environment; to optimise the design of the structures in terms of safety and economy;

(b) improve techniques for prediction of: sediment loads entering reservoirs; the aggradation, degradation and local scour associated with dams and other structures; sediment exclusion at intakes; and the long term stability of flood relief channels or other man-made waterways.

The work described in this report was undertaken to improve the understanding of the physics of sediment-related phenomena. The terms of reference for the study were:

(a) Frictional resistance. Laboratory experiments in an existing installation will be undertaken to investigate the effect of armouring or fine material on the frictional resistance of alluvial channels.

(b) Local scour. A desk study and laboratory experiments in an existing installation will be carried out to investigate rates of local scour of alluvial materials and soft rock.

(c) Scaling laws for mobile-bed physical models. Information will be exchanged with other researchers as part of an IAHR task force.

1.1 Frictional resistance

To determine the flow or sediment transport in an alluvial channel an estimate of the friction losses is required. In rigid boundary channels the frictional resistance is determined by the channel geometry and the roughness of the channel boundary. In alluvial channels the roughness of the boundary depends upon the nature of the bed forms, such as ripples or dunes, that are present. As these bed forms change with the flow the frictional resistance changes. Thus in alluvial channels it is difficult to estimate the frictional resistance as it depends upon both the flow and the sediment transport. The work carried out and described below has greatly extended the range of flow conditions for which reliable predictions of frictional resistance can be made.

1.2 Local scour

Flowing water exerts a shear stress on the bed of a channel. Due to the geometry and nature of the flow this shear stress may be increased in a local area, for example, when a bridge pier is placed in a river. This increase in shear stress affects the sediment movement in the area and may lead to the removal of sediment from the area of increased shear stress. This is known as local scour. Despite having been studied for many years structures still are frequently endangered or fail due to local scour; bridges collapse due to piers being undermined (Holmes, 1974) and there are numerous cases of serious scour problems associated with dams (Mason, 1984). Frequently when dealing with local scour problems only the final depth of scour is important but there are cases when it is important to know the time evolution of this scour. For example, during the closing of part of an estuary scour would not have time to develop to its ultimate depth. In cases of scour associated with permanent structures then, because of the unsteady nature of the flow, it may be important to be able to predict the dependence of scour on time.

1.3 Scaling laws for mobile-bed physical models

It was intended that this part of the contract would support an HR input to an IAHR (International Association for Hydraulic Research) Task Force on the design of mobile-bed physical models. Unfortunately, despite pressure from HR there have been no developments on this since April 1984, the start of the present contract.

2 FRICTIONAL RESISTANCE

To calculate flow or sediment transport in an alluvial channel an engineer is faced with the problem of determining the frictional losses on the boundary of the channel.

For artificial, regular channels which are fixed in shape and carry little sediment there is data readily available which can be used as a basis for the estimation of appropriate friction factors. When natural channels are considered the problems of estimating the friction losses grow. In this case, not only must the frictional losses due to the composition of the banks and bed of the channel be estimated but also due allowance must be given for the effects of channel irregularities and other factors. If one considers channels with movable beds the problems are compounded. The frictional losses are dependent on the bed features present, but these are influenced by the transport of the sediment. The sediment transport, however, depends on the fluid motion and is hence inseparable from the determination of the frictional losses.

The work undertaken for this contract started with some experiment on the frictional resistance of sand-silt mixtures. This experimental work was predominantly carried out by A Bassi, a visitor to HR. The analysis of the results of these experiments lead to doubts about the previously derived relationships which described alluvial friction for uniform sands. A further series of experiments were, therefore, instituted to look further at the alluvial friction of uniform sands. Some of these experiments were carried out at higher Froude numbers than had previously been considered. The results lead to an extension of the existing theory for alluvial friction to include these higher Froude number flows. This latter series of experiments and subsequent analysis was predominantly carried out by Mr Wang Shiqiang, a visitor to HR. Separate accounts of these investigations are now given.

2.1 Frictional resistance of sand-silt mixtures

There are a number of theories for predicting the frictional losses in alluvial channels (Einstein and Barbarossa, 1952; Engelund, 1966; Raudkivi, 1967; White et al, 1980). Most of these theories are based on data, the vast majority of which is from laboratory experiments. Laboratory experiments are almost invariably characterised by the use of narrow-graded, clean sand, that is, sand with a small range of sizes from which both the larger sizes and any smaller silt or clay material has been removed. The finer silt and clay sizes frequently show very different properties to those of sand since these materials demonstrate cohesive properties whereas sands are non-cohesive. The silts and clays are sufficiently small that the physico-chemical properties associated with the surface of the particles become significant. In applications to practical problems, however, it is rare that the sediments which are encountered are similar to the narrowly graded sands used in laboratory experiments. Much more frequently sediments are widely graded and contain varying quantities of silts and clays.

This is an account of a simple, steady-state laboratory investigation to discover if, under these circumstances, the presence of significant proportions of silt mixed with a sand bed have a discernible effect on the alluvial roughness in terms of the methods used to predict alluvial friction. The results were analysed to determine if the theories for predicting alluvial friction based on clean sand needed modification before they could be applied to channels where a proportion of silt is present. The work was confined to predominantly sand beds, as frequently found in rivers. It does not cover the case, which is more frequently found in estuaries than rivers, in which the predominant sediment is cohesive but contains some non-cohesive sand sizes.

Experimental apparatus and procedure

The experiments were performed in a 2.44m wide, 24m long, recirculating, tilting flume. The sediment bed was 16m in length. At the downstream end of the flume a tailgate was used to control the depth of flow. Details of the experimental apparatus and procedure are given in Bassi (1985).

A total of 29 experiments were performed; the first six runs, with a sand only bed, were used to test the equipment and the range of possible flows. Runs 7 to 14 were also carried out with a sand only bed. The grading curve of the sand is shown in Figure 1. The D_{50} size is 0.24mm and $D_{85}/D_{15} = 2.33$. For the remaining three series of experiments (Runs 15 to 20, 21 to 24 and 25 to 29) increasing quantities of silt were added to the sand bed. The discharges for the experiments varied from 0.13 to 0.21 cumecs.

To add the silt to the sand bed the water in the flume was drained down without draining the water from the bed and then the silt was poured onto the surface of the bed as a thick slurry.

The bed was sampled periodically during the experiments. The grading curves of the sediments changed very slightly. The D_{35} varied from 0.21mm to 0.19mm. No systematic change of D_{35} with the silt content of the flume was observed.

At the end of each sand only experiment the pumps were quickly stopped and the water was allowed to overflow from the stilling basin at the downstream end of the flume. The overflowing water did not carry any sediment in suspension. The sand bed was never drained between the experiments. At the end of each experiment with silt the water was retained in the flume to avoid the loss of the finer part of the suspended sediments.

At least once a day the average sediment concentration was measured from samples taken from the recirculating pipes via Pitot tubes.

During the experiments involving silt some velocity and concentration profiles were measured. All the measurements were taken along the axis of the channel and approximately half way along the sediment bed. The velocity profiles were determined by placing a miniature current meter, 10mm in diameter, at a given distance from the water surface and recording the velocity. Each profile consisted of 7 to 10 velocity measurements; the local depth of flow was also measured by lowering a probe with a flat base onto the bed of the channel. Sediment movement in suspension was obtained by taking simultaneous measurements of velocity and sediment concentration at 6 different depths. A small plastic tube of 0.6mm diameter was used to take samples of water and sediment at the same location as the propeller meter.

It was not possible to control the temperature of the water which varied from 14°C at the beginning of an experiment to 23°C at the end. The temperature was measured for every test with a thermometer reading to 0.1°C.

Data Summary

The average water surface slope was calculated from the measured water levels by using a least-squares linear regression. Two values of the slope were determined, the first from the three central levels only, the second using all five points. The first value was used for all calculations because it was less affected by end effects. The second value of slope was used as a control.

The average flow depth was calculated by averaging the six depths measured in the central part of the flume. The standard deviation of the measurements was always less than 1cm, being greater when the sediment transport rate was higher and the bed less regular.

A summary of the measured data for the 146 tests is given in Bassi (1985). For each test the following data is provided:

- time from the beginning of the experiment, in hours,
- water temperature, in degrees Celsius;
- average water surface slope, calculated using the 3 central water levels;
- average water surface slope, calculated using all 5 measured levels;
- average flow depth, in metres
- discharge, in litres per second
- average flow velocity, calculated from the measured discharge and mean cross-section;
- average concentration of sediments, if measured, in parts per million by weight, obtained from samples taken from the water and sediment return system. The concentration values refer to the mixture of sand and silt. It is also indicated if a velocity profile or a velocity and sediment concentration profiles were recorded during the test.

The observed velocity profiles are also detailed in Bassi (1985). For each water depth (measured in metres from the free surface) there is:

- the average flow velocity, in metres per second;
- the standard deviation of the recorded data, in metres per second;
- the standard deviation of the recorded data, in percentage of the mean value

The last depth of each profile indicates the bed level.

The recorded concentration profiles are also given in Bassi 1985. In addition to the same values as for the velocity profiles, there is also the sediment concentration, in parts per million by weight.

Data analysis

The experimental data was analysed by using four different theories on alluvial friction: Einstein and Barbarossa (1951), Engelund (1966), Raudkivi (1967) and White et al (1980). The basic theory of these approaches is outlined together with the data analysis procedures in Bassi (1985).

Einstein and Barbarossa

Using the theory of Einstein and Barbarossa less than 2% of the predictions were within 20% of the observed value. Not all the experimental points could be utilised as some fell outside the range described by the theory. There was no discernible difference between the results with sand and silt mixtures and those with sand alone.

Engelund

The predictions provided by the Engelund method were very good; 65% of the predictions were within 20% of the observed value and all were within a factor of 2. This was a better performance than that reported by White et al (1980). Again there seemed to be no discernible difference between the experimental results with sand and silt mixture and those for sand alone.

Raudkivi

50% of the predictions were within 20% of the observed values. The agreement was better than that reported by White et al (1980). Again, within the results, there is no discernible difference between the results for sand and silt mixtures and those for sand alone.

White et al

The comparison of the White et al theory (1980) with the observations was disappointing. The theory consistently over predicted and none of the predictions were within 20% of the observations. This behaviour was considerably worse than that reported by White et al (1980). Again, there is no discernible difference between the results with the sand and silt mixtures and those with sand above.

2.2 Frictional resistance of uniform sand beds

Following the work of A Bassi it was decided to carry out experiments to investigate further the friction due to a sand bed. The experiments described here were designed to provide more information on the alluvial friction due to sand beds and so act as a basis for comparison with experiments performed with sediments containing a mixture of sand and silt.

Experimental apparatus and procedure

The experiments were performed in the same flume as that used in the earlier series of experiments. Details are given in Wang Shiqiang et al (1986).

The bed forms and their size were observed for each test. The velocity of the bed forms was measured for selected tests.

The grading curve of the sand used is shown in Fig 1. The D_{35} and D_{50} sizes were 0.21mm and 0.23mm respectively.

A total of 31 tests were performed, the results of which are summarised on Wang Shiqiang et al (1986).

Data summary

For each test the following data is provided:

- flume slope
- average water surface slope, calculated using the 3 central water levels.
- average water surface slop, calculated using all 5 measured levels
- average flow depth, in metres
- discharge, in litres per second
- average flow velocity, calculated from the measured discharge and mean cross section
- bed features
- temperature

Discussion of results

Following the work of White et al (1980) and Bassi (1985) the results were analysed in terms of the sediment mobility, F_{gr}, the ratio of the effective shear forces to the immersed weight of the particles. The mobility number was defined in such a way that only the relevant shear forces were used, that is, total shear for fine sediments, grain shear for coarse sediments and an intermediate value depending upon the dimensionless grain size for the transitional sediments.

The dimensionless grain size D_{gr} was defined by

$$D_{gr} = \left[g\frac{(s-1)}{\nu^2}\right]^{1/3} D \qquad (1)$$

where g is acceleration due to gravity

s is specific gravity of the sediment

ν is kinematic viscosity

and D is sediment diameter

The dimensionless sediment mobility was defined by

$$F_{gr} = \frac{v_*^{\,n}}{\sqrt{(gD(s-1))}} \left[\frac{V}{\sqrt{(32)}\log_{10}(10d/D)}\right]^{1-n} \qquad (2)$$

where v_* is the shear velocity

V is the average flow velocity

d is the depth

and n is an exponent which varies from 1.0 for fine sediments (D_{gr} = 1.0) to 0.0 for coarse sediments (D_{gr} = 60).

Thus for fine sediments

$$F_{fg} = \frac{v_*}{\sqrt{(gD(s-1))}} \qquad (3)$$

and for coarse sediments

$$F_{cg} = \frac{V}{\sqrt{(gD(s-1))}\sqrt{(32)}\log_{10}(10d/D)} \qquad (4)$$

Values of F_{fg} and F_{gr} are shown plotted in Figure 1, together with the equation given by White et al (1980). It can be seen that there are two distinct relationships, one for a flat bed, ripples and dunes, termed lower regime, and another relationship for the upper regime of plane bed and anti-dunes with a transition between the two curves. This leads to the postulation that the method for predicting alluvial friction described by White et al (1980) applies to the lower regime case and that the method could be extended to predict flows in the upper regime by the inclusion of a new relationship for upper regime case.

The use of two different relationships, one for lower regime and one for upper regime was first suggested by Engelund (1966). Engelund plotted the dimensionless bed shear due to skin friction against the total dimensionless bed shear for a sequence of flume data, see Figure 2. He postulated two relationships, the lower regime curve for dunes, ripples being excluded from Engelund's analysis, while the upper regime curve applies to flat beds, standing waves and anti-dunes.

A comparison of the present results with those from Bassi (1985) and Guy et al (1966) are shown in Figure 3. It can be seen that the present results and those of Bassi (1985) are close to each other but that they are both above those of Guy et al (1966) and the theoretical relationship of White et al (1980). In the upper regime the present data is also above that of Guy et al (1966).

Criterion for upper or lower regime

The use of two separate relationships for lower and upper regime creates two problems. The first is the determination of which is the appropriate regime to use in particular circumstances and the second is determining the transition that must take place from one regime to the other. In the account so far the distinction between the two regimes has been provided by a description of the bed features associated with them. It, therefore, seems reasonable that the criterion used to define the upper and lower regime conditions should be related to those used to specify the occurrence of different bed forms. Simons and Richardson (1963) distinguished different bed features by plotting stream power, τV against median fall diameter, see Figure 4. Further consideration of the problem has lead us to prefer the use of a non-dimensionalised unit stream power U_E, in the form

$$U_E = \frac{VS}{(g\nu)^{1/3} D_{gr}} \tag{5}$$

Figure 8 shows U_E plotted against D_{gr} for a range of data from Guy et al (1966), Gilbert (1914) and Wang Shiqiang (1986). It can be clearly seen that different areas of the diagram correspond to different bed features. For values of U_E less than 0.00035 the bed is plane. Ripples occur for values of U_E between 0.00035 and 0.011 provided the D_{gr} value is less than approximately 15. Otherwise, for values of U_E between 0.00035 and 0.011 the bed feature is predominantly dunes. The transition region is approximately for values of U_E between 0.011 and 0.02 while flat bed and anti-dunes occur for U_E values greater than 0.02. Thus the lower regime curves are appropriate if U_E is less than 0.011 and the upper regime curve if U_E is greater than 0.011.

We will now consider how this method of determining the nature of the bed features can be used in the calculation of alluvial friction. If there is no a priori information as to the nature of the flow then one must assume in turn that the flow corresponds to lower and upper regime and then determine which assumption leads to consistent results. In the following, we will denote by a superscript L values of variables calculated assuming lower regime and by a superscript U values of variables calculated assuming upper regime. Since $U_E^U > U_E^L$ three different cases must be considered.

(a) $U_E^L < 0.011$ and $U_E^U < 0.011$

In this case since the use of the upper regime equation leads to a solution that implies lower regime conditions the only consistent assumption is that the system is in lower regime.

(b) $U_E^L > 0.011$ and $U_E^U > 0.011$

Similarly since the use of the lower regime equation leads to an inconsistency the system must be in upper regime.

(c) $U_E^L < 0.011$ and $U_E^U > 0.011$

In this case both assumptions lead to consistent results. Our interpretation is that either result represents a stable solution and that the form adopted in practice will depend upon the previous history of the flows.

There is little or no experimental evidence available, but we suggest that the system might display an hysteresis effect. If the system is in lower regime it will remain in lower regime until $U_E^L = 0.011$ at which point the system will make the transition to upper regime. If the system is in upper regime then it will remain in the upper regime until $U_E^U = 0.011$ when it will make the transition to lower regime. There is not enough evidence for a complete interpretation of other people's experiments, but it seems likely that experimental results lying between the upper and lower regime curves do not represent stable, steady solutions, but are unsteady solutions in transition between the upper and lower regimes. A careful series of experiments is required to resolve these problems.

In the interim an algorithm is required which can be used to determine the alluvial friction in any particular case. If information is available about the nature of the flow or its past history then this should be used to determine whether upper or lower regime conditions should be assumed. In the absence of any such information we suggest that the following criterion is used:

$U_E^L + U_E^U < 0.022$ use lower regime

$U_E^L + U_E^U > 0.022$ use upper regime

This is represented graphically in Figure 9. Since U_E^U is always greater than U_E^L only the area below the line OA is considered. Points to the left of the line EB represent upper regime. Two possible stable solutions exist for points in the region DEB. The above criterion is equivalent to assuming that points to the left and below the line EC are lower regime and points to the right and above the line EC are in upper regime.

Conclusions

A set of experiments carefully measuring the alluvial friction due to sand-silt mixture has been performed. The analysis of the experimental data indicates that the presence of silt fractions in concentrations of up to 3000ppm has no effect on the determination of alluvial friction using accepted theories for predicting alluvial friction of sand beds. It is, therefore, recommended that in situations where silt is present in a sand bed but the sediment concentration in the flow does not exceed 3000ppm theories for predicting alluvial friction of sand beds are used without modification. The results further show that the theory of Einstein and Barbarossa provides poor predictions of alluvial roughness as has been reported elsewhere and that of the theories tested that by Engelund provided the best predictions.

A set of experiments carefully measuring the alluvial friction due to a sand bed have been performed.

The type of bed features developed has been related to the dimensionless unit stream power U_E.

Two distinct relationships have been observed describing alluvial friction, one for lower regime and one for upper regime. The observations in the lower regime are similar to those of Bassi (1985) but both sets of results differ from those of Guy et al (1966) and the theoretical relationship of White et al (1980).

An extension of the White et al (1980) method for predicting alluvial friction has been proposed for upper regime flows. The method has been developed on flume experiments. This removes the restriction on the White et al method to flows whose Froude number is less than 0.8. More data, however, is required to elucidate the behaviour of large sediment sizes under higher Froude number flows.

A criterion to determine when the flow in a channel is in upper or lower regime has been suggested. It has been further suggested that there is a range of conditions under which both the solutions in the upper and lower regime are stable. It is postulated that the solution that is achieved in practise depends upon the history of the flow and that the system may well exhibit hysteresis. Further experiments are required to investigate these suggestions.

The application of the extended method is described in Appendix 1.

3 LOCAL SCOUR

Flowing water exerts a shear stress on the bed of a channel. Due to the geometry and nature of the flow this shear stress may be increased in a local area, for example, when a bridge pier is placed in a river. This increase in shear stress affects the sediment movement in the area and may lead to the removal of sediment from the area of increased shear stress. This is known as local scour. Despite having been studied for many years structures still are frequently endangered or fail due to local scour; bridges collapse due to piers being undermined (Holmes, 1974) and there are numerous cases of series scour problems associated with dams (Mason, 1984). Frequently when dealing with local scour problems only the final depth of scour is important but there are cases when it is important to know the time evolution of this scour. For example, during the closing of part of an estuary scour would not have time to develop to its ultimate depth. In cases of scour associated with permanent structures then, because of the unsteady nature of the flow, it may be important to be able to predict the dependence of scour on time.

3.1 Local scour due to jets

A feature of high dams is the discharge of water with high energy either from spillways or low-level outlets, or both. Such a discharge may lead to erosion downstream of the structure which may threaten the safety of the dam or may even affect the stability of the abutments. The importance of such effects is increasing as the size of projects considered becomes larger. To prevent serious erosion there is a need to dissipate as much as possible of the energy of the

the energy of the flow. For dam structures this is frequently achieved by using free overfalls, ski jumps or flip buckets. All these structures direct a jet of water at some point of impact selected so that, hopefully, scour in that particular area will not lead to serious problems. To ensure that problems do not develop during the life of a project it is important to be able to predict the size of scour holes that will be developed. In this review a number of case histories are summarised where the scour holes that developed were either larger than anticipated or developed in unexpected forms.

Case histories

The following case histories are summarised to give an indication of the type and magnitude of the problems that can develop. The material has been taken from Mason (1984).

1. **Alder Dam USA**

   The 100m high Alder Dam was constructed in 1945 with a spillway capacity of 2,265m^3/s. The spillway ends in a flip bucket that directs the flow to an area of blocky andesite. After seven years operation, during which the maximum recorded spillway discharge was only 566m^3/s, a plunge pool had developed approximately 30m x 45m x 24m deep. Remedial work carried out involved the placing of 6000m^3 of mass and reinforced concrete.

2. **Nacimcento Dam USA**

The 82m high Nacimcento dam was constructed in 1957 with a spillway with a flip bucket. A flood in 1969 caused major erosion downstream of the spillway that could have undermined the embankment. Remedial work carried out at a cost of approximately US $6 million included demolishing the existing flip bucket, extending the spillway chute by 83m and constructing a new flip bucket at the downstream end; reconstruction of the fill either side of the extended chute and excavation of a new stilling pool downstream.

3. **Picote Dam**

The Picote Dam is a 100m high arch dam. It has tapering chute spillway with flip bucket which directs the flow into a narrow granite canyon. After a flood 4 years after construction a 20m deep pit had developed together with a 15m high bar of eroded material downstream. The bar caused a significant reduction in head and consequent loss in output at the power station. Remedial work carried out at a cost of US $4 million included extending an existing diversion tunnel so that flows could bypass the plunge pool and bar.

4. **Grand Rapids**

The spillway of the Grand Rapids dam, completed in 1962, is designed to give a free-overfall jet. After four years of operation the depth of scour was 30% more than that anticipated for the complete life of the structure. Remedial work involving extending the chute 20m to the upstream face of the scour hole cost US $4 million.

5. **Kariba Dam**

Kariba is a 130m high arch dam with flood gates which provide a free-overfall jet impacting on to gneiss immediately downstream of the dam. Five years after completion the removal of 400,000m^3 of rock had formed a plunge pool 50m deep. Spray from the jet had also resulted in slides of material on the banks threatening the stability of the abutments. Further erosion was prevented by the construction in 1967 of a second power station, as originally designed.

6. **Tarbela Dam**

Tarbela is a 143m high embankment dam. There is both a service spillway and an auxilliary spillway, both having flip buckets. Within a year of construction 400,000m^3 of material was removed from the plunge pool. Remedial work on the service spillway necessitated increased use of the auxiliary spillway which resulted in excessive erosion of the plunge pool for that spillway. Remedial work involved stabilisation of the high slopes around the plunge pool, the post tensioning of the bucket structure into the rock upstream, lowering bed levels at the downstream end of the plunge pool and lining the sides of the pool with rolled concrete. The cost of repairs to the service and auxilliary plunge pools was US $120 million and US $90 million respectively.

3.2 Literature review on modelling of local scour

This literature review describes work that has been carried out on the development of local scour with time. Two of the significant parameters associated with local scour are the time development of the scour and the ultimate scour depth. In the past most attention has been given to the prediction of ultimate scour depths. The time development may also be important, however, for example where a structure is not sufficiently permanent for the ultimate scour depth to develop or where the flow is predominantly unsteady so that an ultimate scour depth predicted on the basis of a steady flow is unrepresentative.

In this review we consider scour due to both horizontal and vertical jets. There are two essential areas of interest, one is to determine the functional relationship between scour and time for a given set of circumstances and the second is the method required to transfer from one scale to another, most commonly from a model scale to a prototype.

Scour due to horizontal jets

Breusers (1965, 1967) at the Delft Hydraulics laboratory carried out work on the scour generated downstream by flow over a low sill. Tests were performed in three different flumes 0.5, 1.0 and 3.0m wide with water depths of 0.25, 0.5 and 1.5m respectively. Different mean velocities of flow were used and measurements were obtained using a 15mm diameter miniature current meter. The bed materials used were sand, bakelite and polystyrene with specific gravities of 2.65, 1.35 and 1.05 respectively. The bed materials varied in size from 0.1 to 2.6mm.

Van der Meulen and Vinje (1975) extended the work of Breusers by having the sill extend only part of the way across the flume so that a three-dimensional flow pattern was created.

Further work carried out at Delft compared scour at a prototype structure with the scour developed during model tests (de Graauw and Pilarczyk, 1980). The study was carried out in connection with the closing of the Eastern Schelde by means of a storm surge barrier which will be kept closed only during a storm surge. The reliable prediction of the future scour depths was of great importance for the stability of the construction. It was, therefore, decided to perform special prototype tests to verify the time-scour depth relationships. The results for the prototype and a 1:30 scale model were compared. The prototype sediment had a D_{50} size of 0.25mm with a specific gravity of 2.65. The model sediment was polystyrene with a D_{50} size of 0.13mm and a specific gravity of 1.05. The predicted scour was slightly greater than the prototype, the difference being put down to the non-uniform bed material of the prototype.

Farhoudi and Smith (1982) modelled scour downstream of a hydraulic jump produced on a dissipator apron at the bottom of an overflow dam structure. To verify that no scale effects occurred three sizes of model were used, together with six bed materials:

Material	D_{50} (mm)	Specific gravity
Fine bakelite	0.25	1.40
Coarse bakelite	0.52	1.41
Sand	0.15	2.68
Sand	0.25	2.68
Sand	0.52	2.68
Sand	0.85	2.65

Cunha (1975) measured the development of scour with time in a flume. The experiments were carried out in a flume 10m long, 2m wide and 0.6m deep. The scour was developed downstream of protrusions consisting of flat plates of 0.1, 0.2 and 0.3m lengths. Two bed materials were used.

	D_{50} (mm)	Critical velocity of motion (m/s)
Sand	1.60	0.38
Gravel	5.8	0.70

Vertical jets

Early experimental work on the scour due to two-dimensional submerged jets is due to Rouse (1940). In these experiments a constant head tank was used to drive a vertical jet into a six inch wide flume. The duration of the tests was from three to twenty four hours, depending upon the rate of scour. The shape of the scour hole was traced out on the glass wall of the flume at various time intervals while the flow was still running.

Doddiah, Albertson and Thomas (1953) carried out a series of tests on scour due to circular jets, one for circular jets and one for a free overfall.

For the circular jet series there are two types of jet; solid and hollow. The geometry of the tests was as follows, a fixed height of jet at 0.7m, with area variable from 580 to 1330mm. This fell on to a 1.22 by 1.22 by 0.64m deep bed of gravel. There were two sizes of gravel, both of narrow grade, 3.18-6.35mm and 6.35-12.7mm, with mean fall velocities of 0.219 and 0.292m/s respectively. The depth of the tailwater could be kept constant for each run but was varied between 0.05 and 0.41m for various runs. At the start of each run of the model the bed was covered until equilibrium of the tailwater was reached. Then the cover was removed and time begun. Measurements were made by diverting the flow and using a point gauge, only one line of measurements was required as the shape of the hole was nearly conical.

The stopping and starting of flows for measurement caused the side slopes to fall in - hence the conical shape of the scour hole, giving walls at angle of 27-29 degrees.

For the free overfall jet a sharp crest was used which could be varied in height from 0.3 to 1.22m. The width of the flume was 0.85m. Discharges were varied from 0.0115 to 0.0461m^3/s per m of crest. Two gravels both having the same mean size of about 6.35mm, but different standard deviations of 1.17 and 1.33 were used.

As with the start of the circular jet tests the tailwater was allowed to steady before the bed was uncovered and time begun. Measurements were made while the water still ran, they were made from 0.5 minutes up to 18 hours after the start. Photographs of the scour progression were taken periodically through the glass wall of the flume.

Rajaratnam carried out work with two dimensional submerged jets of both air and water impinging on sand beds. The experiments were carried out in a flume

0.15m wide x 0.3m deep and 1.8m long, with a sand bed 100mm deep. The jet was produced by a "well designed nozzel" which could be raised or lowered from the bed. Jet velocities were measured with a 1.07mm total head probe placed in the core of the jet. The range of parameters varied for the runs are given below.

	Jet width mm	Height mm	Grain diameter mm	Jet velocity cm/s
Water series	2.5	12.5 to 127.0	1.2 to 2.38	2.53 to 9.10
Air series	2.03 and 4.7	100 to 178	1.2	74.6 to 159.8

Photographs of the scour profile when the jet is on show a large amount of turbulence. Because of this turbulence scour profiles were measured when the jet was turned off - which would appear to be drastically different from the dynamic state.

Analysis of experimental work

1. Horizontal jets

In analysing the time development of scour there are two fundamental problems. One is for a given geometry and flow and using information on the initial rate of scour development to predict how the scour will continue to develop. The second problem is if one knows the development of scour with time at one scale to predict the rate of scour at a different scale. Typically, if one measures the rate of scour on a small scale physical model how can one then predict the rate of scour on the prototype.

Breusers (1965, 1967) and those that extended his work (Van de Meulen and Vinje, 1975; de Graauw and Pilarczyk, 1980) assumed that the scour dependence with time could be described by the equation:

$$\frac{h_{max(t)}}{h_o} = \left(\frac{t}{t_1}\right)^p \tag{6}$$

where $h_{max(t)}$ is the maximum scour depth at time t
h_o is the value of h_{max} at time t_1
t is time
P is exponent

Thus knowing the maximum depth of scour at one instant the maximum depth of scour can be found for all times. Breusers found that the exponent p was almost constant at 0.38 for a wide range of values of h_{max}/h_o with different velocities of flow. de Graauw and Pilarczyk (1980) used a value for p of 0.4, Farhoudi and Smith (1982) found good correlation with an exponent value of 0.19 for scour downstream of a hydraulic jump. Van der Meulen and Vinje (1975) found that for three-dimensional scour the exponent p varied with both longitudinal section and time.

Since from equation (6):

$$h_{max(t)} = h_o \left(\frac{t}{t_1}\right)^p \tag{7}$$

it follows that if p is strictly positive and constant that the depth of scour increases indefinitely. This cannot be true for all time but may be a reasonable approximation during the initial development.

Shen et al (1965) carried out an analysis of the time-development of local scour around bridge piers.

In their analysis they derived a relationship of the form:

$$\frac{h_{max(t)}}{h_o} = A \log \frac{t}{t_1} + B \; , \qquad (8)$$

where A seemed to depend upon the Froude number of the approach flow.

Novak (1981) proposed an expression for the time dependence of scour in stilling basins of the form:

$$\frac{A}{A_1} = a - b\, e^{-c(t/t_1 + d)} \qquad (9)$$

where a, b, c and d constants depending upon the type of stilling basins

A_1 is the depth of scour at time t_1

and A is the depth of scour at time t (for $t > t_1$)

Local scour occurs when, due to the particular geometry and flow, the shear stress on the bed is locally increased above the general level. Whether the mean shear stress on the bed, that is, the shear stress away from the local increase, exceeds that for threshold of motion or is less is unimportant but for scour to result the local increase must be such that in that area the threshold of motion is exceeded. As a result sediment transport takes place. The non-uniform spatial distribution of shear stress will imply a non-uniform sediment transport rate and as a result the bed level will change.

We can imagine a simple one-dimensional example in which the shear stress is locally increased, see Figure 7a. As a result the sediment transport rate increases, Figure 7b. Using the sediment continuity equation

$$B \frac{\partial z}{\partial t} + \frac{\partial G}{\partial x} = 0 \tag{10}$$

we can see that instantaneously the most rapid change of bed level will correspond to the point where the transport rate varies most rapidly. The changes in bed level will affect the flow and hence lead to a change in the shear stress distribution. The rate at which the scour will develop will depend upon how the shear-stress distribution is affected by changes in bed level. Returning to the example, if the shear stress distribution was independent of the depth of flow then scour would take place at a constant rate indefinitely. If, however, a small change in bed level resulted in the return of the shear stress to the mean level then the rate of scour would rapidly diminish and scour would very soon cease. This implies that the rate of scour must depend to a large degree on the relationship between flow geometry, in particular bed level, and shear stress on the bed. As inevitably this must be extremely sensitive to the particular geometry of flow it is inconceivable that there can be one relationship which describes the time development of scour for all geometries. All the relationships developed must be specific to the particular geometry considered. This implies that it is extremely unlikely, for example, that an equation used to describe the time development of local scour downstream of a hydraulic jump would be applicable to scour around a bridge pier. It also implies that to determine the rate of local scour for a particular geometry specific studies must be made for that geometry. In general this suggests that resort must be made to a physical model with that geometry.

The problem remains, if a model test is carried out and a rate of scour measured at the model scale, how can this be scaled to give the rate of scour at the prototype scale. As the problem is intimately related to sediment transport it is obviously important that sediment transport in both model and prototype be considered. Consideration of equation 10 immediately reveals that the important quantity is $\frac{\partial G}{\partial x}$ or the spatial rate of change of sediment transport rate. For fine sediments a small increase in bed shear stress will lead to a significant increase in sediment transport rate but for coarse sediments the same increase in shear stress would lead to a much more modest increase in sediment transport rate. It follows that the rate of local scour is sensitive to sediment diameter. It should be noted that we are only considering the rate at which scour develops. One can provide quite general arguments that suggest that the ultimate scour depth achieved is more or less independent of sediment diameter. Here, we are considering the time that is taken to achieve that ultimate scour.

By considering the sediment continuity equation

$$B \frac{\partial z}{\partial t} + \frac{\partial G}{\partial x} = 0 \qquad (11)$$

and a sediment transport relationship Breusers concluded that the time scale n_t was given by:

$$n_t = n_L^2 \, n_\Delta^{1.5} \, n_D^{0.5} \, n_{(u-u_{crit})}^{-4} \qquad (12)$$

This was on the assumption of an undistorted model. For the case of scour downstream of a long horizontal bottom protection this equation was confirmed by extensive experiments (Breusers 1965, 1967). Farhoudi and Smith (1982) carried out a similar series of experiments for scour downstream of a hydraulic jump. They developed a similar form of equation to Breusers but replaced $n_{(u-u_{crit})^{-4}}$ by $n_{(u-u_{crit})^{-3}}$.

Obviously the form of the scaling relationship (12) depends upon the sediment transport equation used to define G.

Since the velocity u in the model and prototype varies in space it follows that n_t is not a constant. This implies that at a fixed time the scour in the model and prototype will not necessarily be geometrically similar. If in one or other or both the slope of the scour hole approaches the angle of repose of the sediment then similarity between the model and prototype may be violated.

3.3 Scour Development in time

Effect of varying discharge

A number of equations have been formulated to describe the development of scour through time. These have been derived from laboratory experiments carried out with constant discharges. They all take the form:

$$d(t) = d_e \quad f(t) \tag{13}$$

where d = scour depth at time t

d_e = ultimate scour depth

f(t) = some function of t

Since for large t

$$d(t) = d_e \tag{14}$$

we have $f(t) \to 1$ as $t \to \infty$

The value of d_e can be given by a number of expressions of the form:

$$d_e = d_e\ (q,\ h,\ h_2,\ g,\ D) \tag{15}$$

where q is the unit discharge
h is the fall of the jet
h_2 is the tailwater depth
g is acceleration due to gravity
and D is sediment diameter

These equations are all based on constant discharges. In practical applications, however, the unit discharge q is not constant but varies in time. It is of interest to ask how the scour hole will develop for a particular sequence of discharges. If we can approximate the discharge record by a sequence of steady discharges, Q_1, Q_2, Q_3, ... and corresponding times, t_1, t_2, t_3, ... then it is possible to use equations (11) and (15) to calculate the time development of the scour hole.

Let d_i denote the scour depth after time period t_i then

$$d_1 = d_e\ (q_1)\ f(t_1) \tag{16}$$

Define:

$$t_2' = f^{-1}\ [\frac{d_1}{d_e(q_2)}] \tag{17}$$

i.e. t_2' is the length of time it would have taken a discharge of q_2 to develop a scour hole of depth d_1, then

$$d_2 = d_e(q_2)f(t_2' + t_2) \quad (18)$$

More generally

$$t_n' = f^{-1}\left[\frac{d_{n-1}}{d_e(q_n)}\right] \quad (19)$$

and $d_n = d_e(q_n)\ f\ (t_n' + t_n)$ (20)

An example of the process is explained graphically in Figure 8.

3.4 Experimental work

The object of the experimental work was to measure the evolution of a 2-dimensional scour hole produced by a vertical jet of water.

Apparatus and method

The experiment was set up in a perspex-sided flume of 0.4m width. The apparatus is shown diagrammatically in Figure 9.

A mobile bed of 3m length was retained in the flume at a thickness of 0.5m. The vertical jet of water was supplied by a rectangular jet nozzle of 0.39m across the flume and 0.04m width, positioned above the mid-point of the bed. A depth of water was maintained above the bed and controlled by adjustable tail gates.

Water was circulated by pumping from a sump, through the jet nozzle, over the tail gates either side of the retained bed and back to the sump. The flow rate was controlled by a gate valve and measured by a bend flow meter. A baffle was installed inside the jet nozzle to ensure uniform flow across the width of the jet.

The bed material used was a uniform medium-coarse oblate gravel with mean particle size (D_{50}) of 9mm. Figure 10 shows the particle size distribution for a sample of this material. The angle of repose of the bed material under water was approximately 45°.

A grid with 0.1m inter-sections was marked on the perspex wall of the flume. A camera was set up at the side of the flume and the development of the scour hole was recorded photographically, the grid providing a scale.

For each test, the bed was protected with wooden boards while the required water depth and jet velocity were set and steady conditions established. The boards were then removed quickly and a stopwatch was started. A series of photographs of the scour hole were then taken at increasing time intervals. Each test was run for a period of at least one hour, allowing the scour hole to become fully developed.

Tests carried out

A total of sixteen tests were carried out; thirteen with the jet nozzle set at 0.22m above the bed and three runs with the nozzle at 0.32m above the bed. Figure 11 summarises the water level above the bed and the jet velocity used for each of the tests.

The jet velocities used ranged from 0.8 to 2.8ms^{-1}. The water depths ranged from 0.15m to 0.53m above the bed; from below the jet nozzle to above the jet nozzle as can be seen from Figure 11.

The experimental results are given in Tables 1 to 16.

Scour hole shape

The development of the shape of the scour hole is demonstrated in Figure 12. The vertical scour depth has been non-dimensionalised by the total depth of scour at that time while the horizontal distance has been non-dimensionalised by the horizontal distance to the point of maximum height. The results for test 12 clearly show the impact of reduced tailwater levels, this leads to the development of an elongated, flattened ridge. Under deep tailwater conditions the downstream slope of the ridge corresponds to the angle of repose of the sediment under water.

Following the work of Breusers, Farhoudi and Smith (1982) developed scaling laws for scour downstream of a hydraulic jump. The development, however, is very dependent upon having appropriate equations to describe both critical conditions and the sediment transport rate when critical conditions are exceeded. Unfortunately, in the case of vertical jets no such equations are available. It is therefore difficult to see how such an approach could be used for vertical jets.

Development of scour with time

To study the development of scour with time:

$t \dfrac{u_o^3}{sgDB}$ was plotted against $\dfrac{d}{B}$

where:

t is time
u_o is jet velocity
s is submerged specific gravity of sediment

g is acceleration due to gravity

D is D_{50} sediment size

B is the jet width

and d is the maximum depth of scour at time t.

Though the absolute values of these variables differ between tests they show a consistent behaviour within each test, see Figure 13. Thus the initial scour development can be used to predict the scour that will be developed at later periods. The time dependent behaviour of the various tests is similar for both the deep and the shallow tailwater cases.

In most cases there was a significant difference in the depth of the scour hole during the test while the jet was operating and at the end of the test when the flow was switched off. Sediment maintained in suspension by the flow settled and sediment from the sides of the scour hole slumped into the scour hole thereby reducing the slope of the sides and the total depth. Figure 14 shows the profiles of scour holes during and after two tests. These suggest that observations of scour taken after a flood period may significantly under-estimate the total depth of scour developed during a high flow period.

4 CONCLUSIONS

4.1 Frictional resistance

A set of experiments carefully measuring the alluvial friction due to sand-silt mixture has been performed. The analysis of the experimental data indicates that the presence of silt fractions in concentrations of up to 3000ppm has no effect on the determination of alluvial friction using accepted theories for predicting alluvial friction of sand beds. It is, therefore, recommended that in situations where silt is present in a sand bed but the sediment concentration in the flow does not exceed 3000ppm theories for predicting alluvial friction of sand beds are used without modification. The results further show that the theory of Einstein and Barbarossa provides poor predictions of alluvial roughness as has been reported elsewhere and that of the theories tested that by Engelund provided the best predictions.

A set of experiments carefully measuring the alluvial friction due to a sand bed have been performed.

The type of bed features developed has been related to the dimensionless unit stream power U_E.

Two distinct relationships have been observed describing alluvial friction, one for lower regime and one for upper regime. The observations in the lower regime are similar to those of Bassi (1985) but both sets of results differ from those of Guy et al (1966) and the theoretical relationship of White et al (1980).

An extension of the White et al (1980) method for predicting alluvial friction has been proposed for upper regime flows. The method has been developed on flume experiments. This removes the restriction on the White et al method to flows whose Froude number is less than 0.8. More data, however, is required to elucidate the behaviour of large sediment sizes under higher Froude number flows.

A criterion to determine when the flow in a channel is in upper or lower regime has been suggested. It has been further suggested that there is a range of conditions under which both the solutions in the upper and lower regime are stable. It is postulated that the solution that is achieved in practise depends upon the history of the flow and that the system may well exhibit hysteresis. Further experiments are required to investigate these suggestions.

The application of the extended method is describe in Appendix 1.

4.2 Local scour

The rate of scour development is dependent upon the flow geometry so that results for one flow geometry may not, necessarily, be applicable to another. In practise this implies that model tests must be carried out for any specific geometry.

Because of the nature of the scaling from model to prototype, model/prototype similarity may be violated if the angle of repose is exceeded in either the model or the prototype.

The dynamic scour developed while the jet is operating may significantly exceed the static scour observed when the jet flow ceases.

The time development of scour may be predicted by plotting $t \frac{u_o^3}{sgDB}$ against $\frac{d}{B}$.

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TABLES.

TABLE 1

Test Number 1

Water depth, H 37cm

Height of jet, h 22cm

Jet velocity, u_o 106cm/s

t (s)	Lp (cm)	D (cm)
7	14	2
15	14	2.5
30	14.5	2.5
60	14.5	2.3
300	15	4
600	15	5
1200	16	5
2700	18	6

TABLE 2

Test Number 2

Water depth, H 22cm

Height of jet, h 22cm

Jet velocity, u_o 85cm/s

t (s)	Lp (cm)	D (cm)
10	12	1.5
30	12	1.5
60	12	1.5
300	12.5	1.5
600	12.5	2
1200	13	2
2400	14	2
3600	14	2

TABLE 3

Test Number 3

Water depth, H 15cm

Height of jet, h 22cm

Jet velocity, u_o 87cm/s

t (s)	Lp (cm)	D (cm)
10	10	5
20	11	5
900	17	7.5
1800	19	7
3600	20	8

TABLE 4

Test Number 4

Water depth, H 22cm

Height of jet, h 22cm

Jet velocity, u_o 207cm/s

t (s)	Lp (cm)	D (cm)
10	26	18
30	28	18
60	30	18
240	35	21
600	38	22
1800	41	23.5
3600	45	25
7200	47	26

TABLE 5

Test Number 5

Water depth, H 37cm

Height of jet, h 22cm

Jet velocity, u_o 260cm/s

t (s)	Lp (cm)	D (cm)
7	25	21
15	27	25
45	30	25
300	38	25
600	40	25
1800	43	25
7200	48	30

TABLE 6

Test Number 6

Water depth, H 36cm

Height of jet, h 22cm

Jet velocity, u_o 215cm/s

t (s)	Lp (cm)	D (cm)
10	24	18
30	25	18
60	26	18
120	28	18
300	30	18
600	33	18
1800	35	18
3600	38	19

TABLE 7

Test Number 7

Water depth, H 34cm

Height of jet, h 22cm

Jet velocity, u_o 166cm/s

t (s)	Lp (cm)	D (cm)
10	22	11
30	23	13
60	24	13
120	26	13
300	27	13.5
900	30	14
3600	33	14

TABLE 8

Test Number 8

Water depth, H 53cm

Height of jet, h 22cm

Jet velocity, u_o 273cm/s

t (s)	Lp (cm)	D (cm)
10	25	23
30	35	24
60	39	25
120	40	27
300	44	27
600	47	30
3600	51	30

TABLE 9

Test Number 9

Water depth, H 51cm

Height of jet, h 22cm

Jet velocity, u_o 207cm/s

t (s)	Lp (cm)	D (cm)
10	20	17
30	27	18
60	29	18
120	32	18
300	32.5	19
1200	35	19
3600	38	19

TABLE 10

Test Number 10

Water depth, H 49cm

Height of jet, h 22cm

Jet velocity, u_o 159cm/s

t (s)	Lp (cm)	D (cm)
10	22	13
30	24	13
60	24	13
120	26	13
300	26	14
1500	29	14
3600	29	14

TABLE 11

Test Number 11

Water depth, H 15cm

Height of jet, h 22cm

Jet velocity, u_o 207cm/s

t (s)	Lp (cm)	D (cm)
10	23	17.5
30	30	18
60	32	18
120	32	19
300	33	20
1200	36	20
3600	42	22

TABLE 12

Test Number 12

Water depth, H 19cm

Height of jet, h 22cm

Jet velocity, u_o 279cm/s

t (s)	Lp (cm)	D (cm)
5	25	24
10	38	28
30	44	29
60	46	29
120	46	30
1200	57	33
3600	58	35

TABLE 13

Test Number 13

Water depth, H 15cm

Height of jet, h 22cm

Jet velocity, u_o 155cm/s

t (s)	Lp (cm)	D (cm)
10	20	13
30	24	15
60	27	15
120	27	15
300	28	17
1200	28	17
3600	33	18

TABLE 14

Test Number 14

Water depth, H 15cm

Height of jet, h 32cm

Jet velocity, u_o 207cm/s

t (s)	Lp (cm)	D (cm)
10	24	15
30	31	17
60	34	18
120	38	20
300	42	20
600	42	22
3600	47	22

TABLE 15

Test Number 15

Water depth, H 38cm

Height of jet, h 32cm

Jet velocity, u_o 205cm/s

t (s)	Lp (cm)	D (cm)
10	24	13
30	29	17
60	30	18
180	35	19
600	39	20
1800	43	20
3600	47	20

TABLE 16

Test Number 16

Water depth, H 54cm

Height of jet, h 32cm

Jet velocity, u_o 215cm/s

t (s)	Lp (cm)	D (cm)
10	41	17
30	42	18
60	43	19
120	44	20
360	49	21
1800	55	23
3600	59	24

FIGURES.

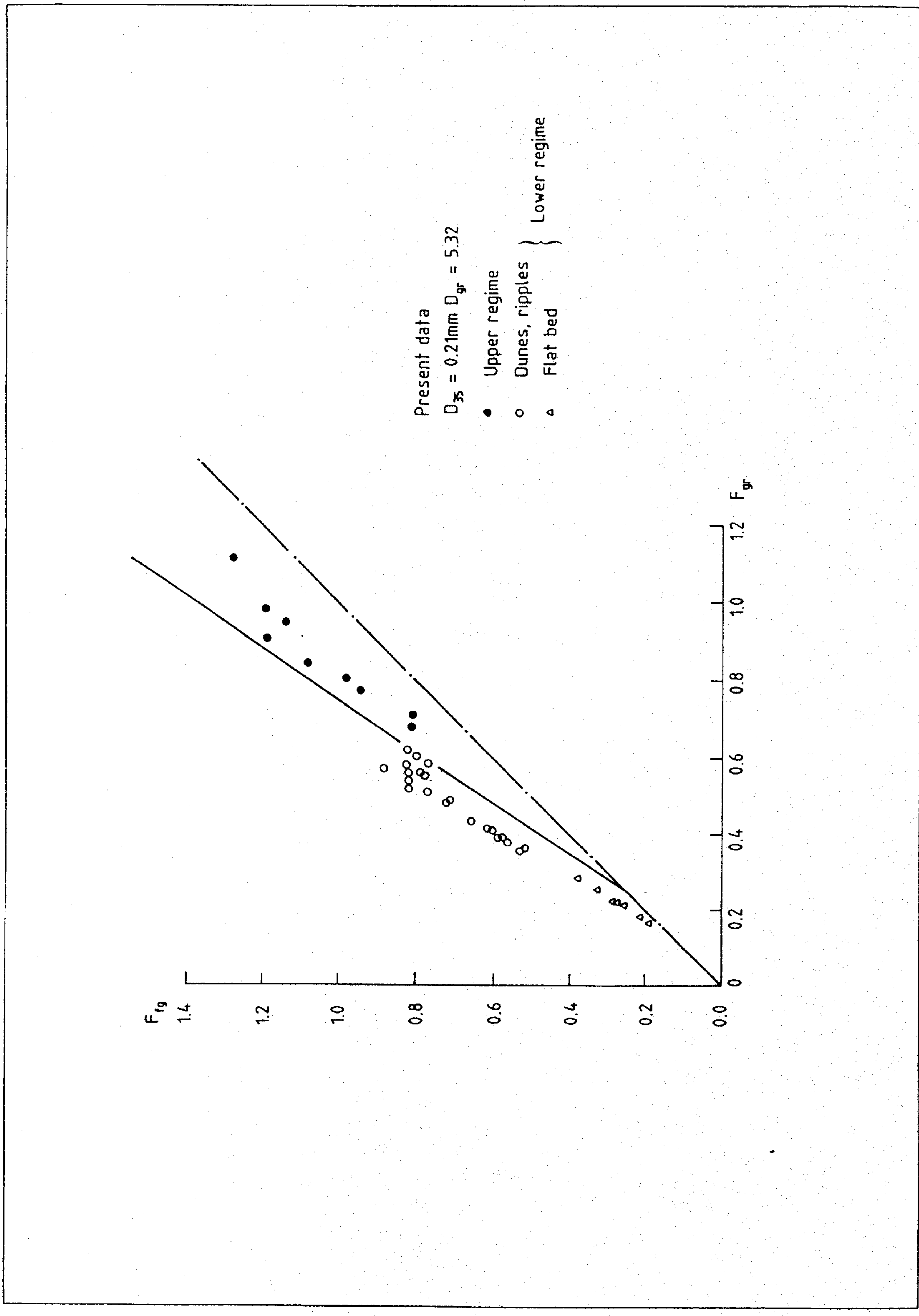

Fig 1 F_{gr} against F_{fg}, present results

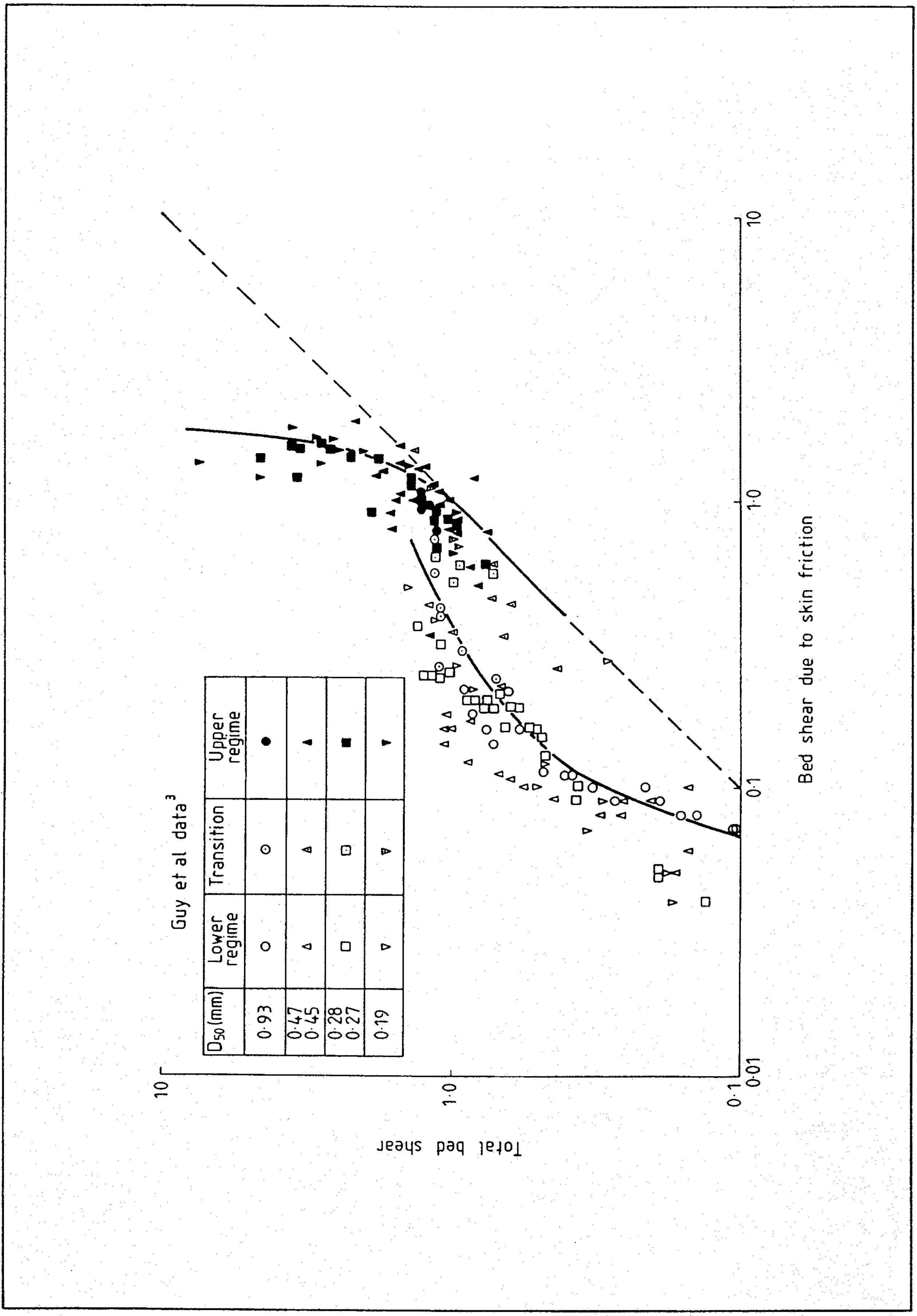

Fig. 2 Frictional relationship by Engelund

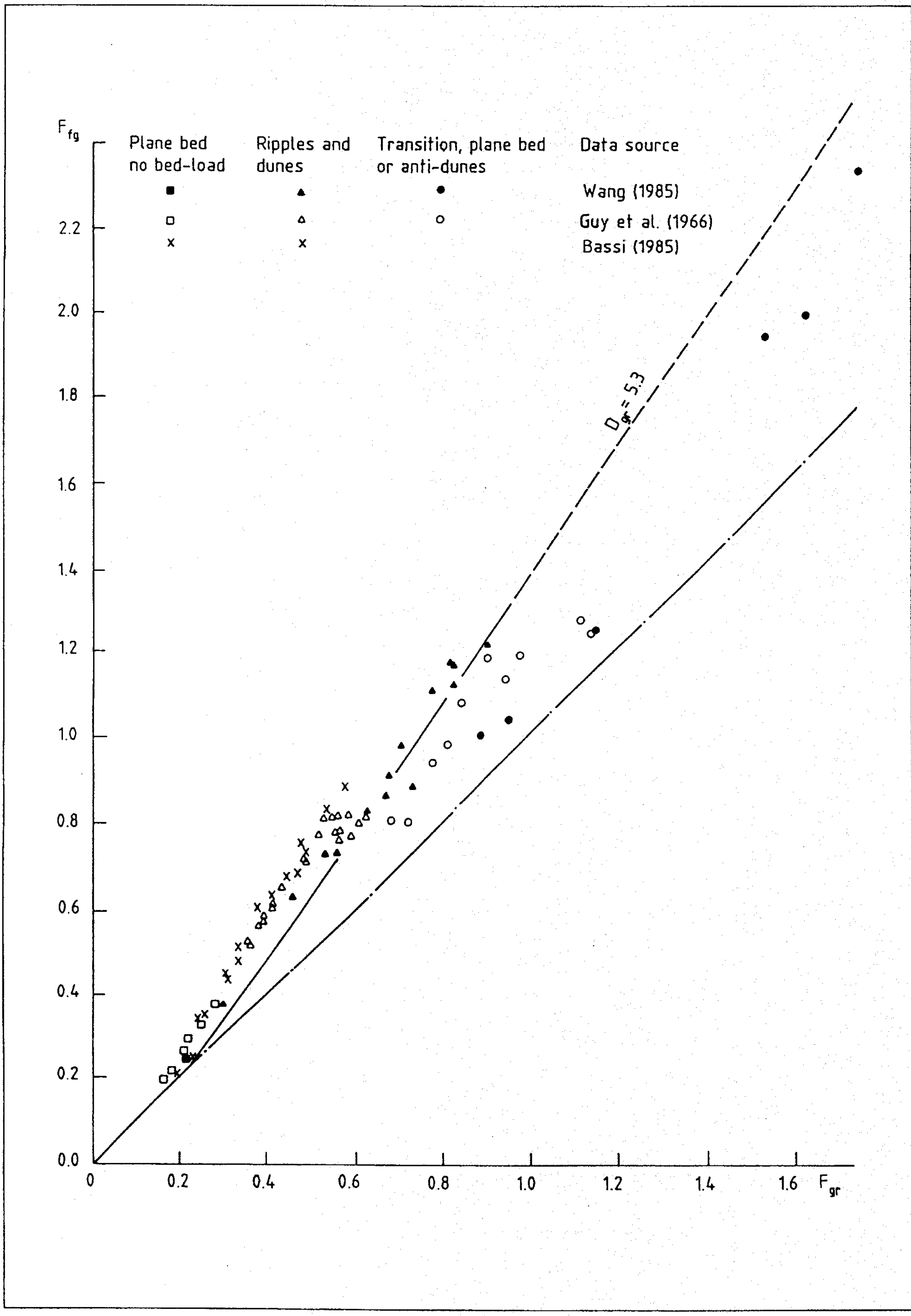

Fig 3 F_{gr} against F_{fg}, all data

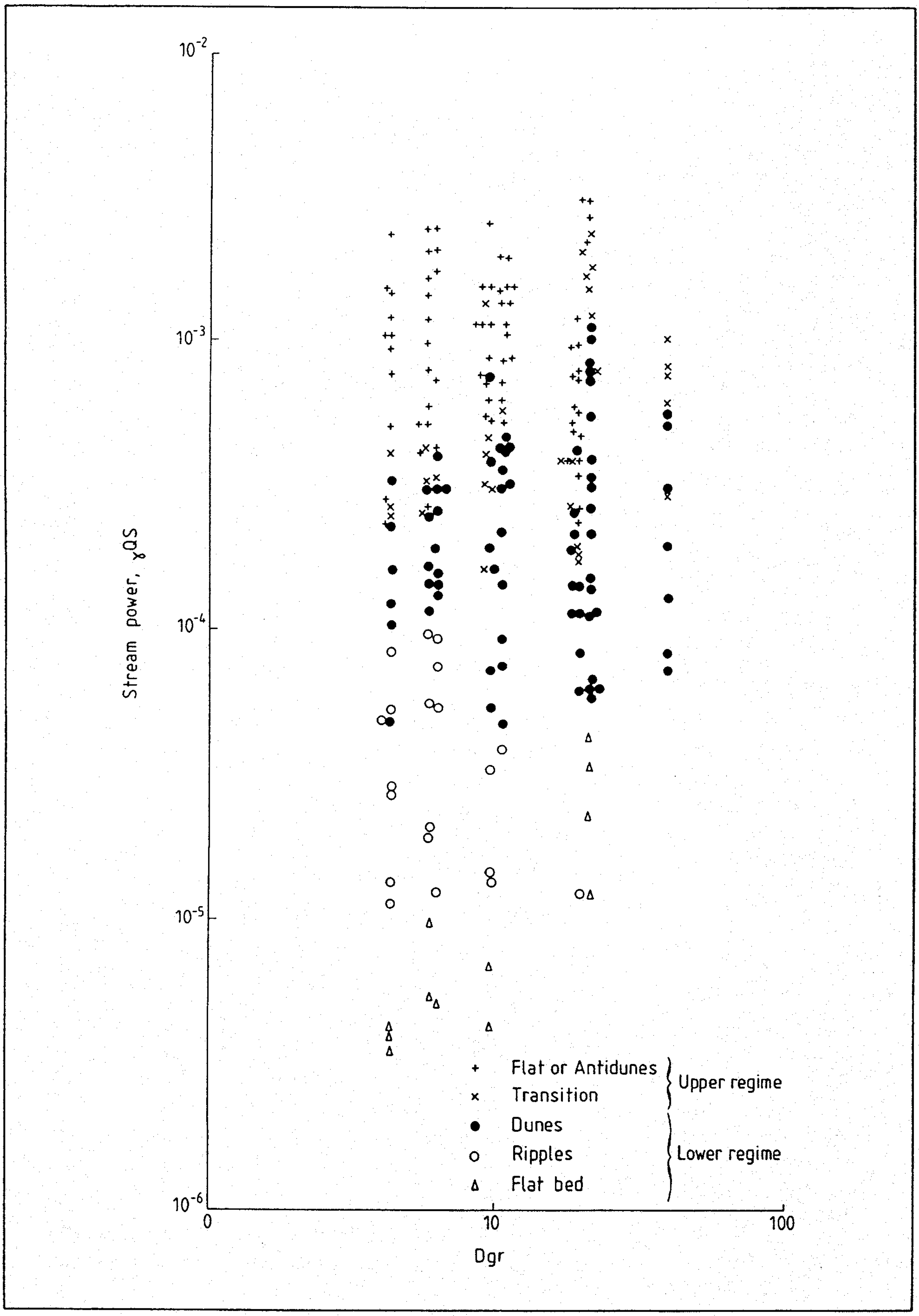

Fig. 4 Stream power against sediment size

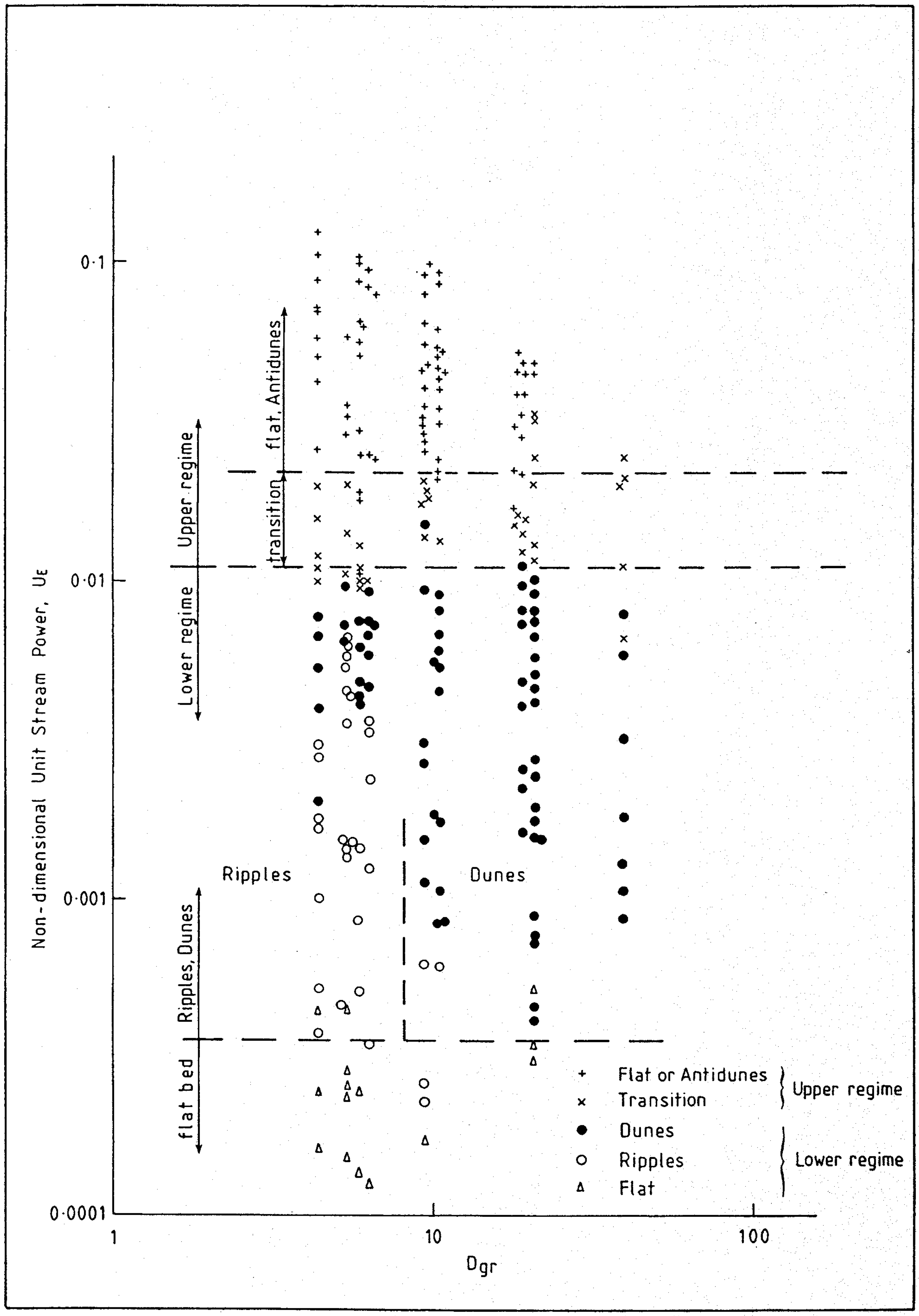

Fig 5 U_E against D_{gr}

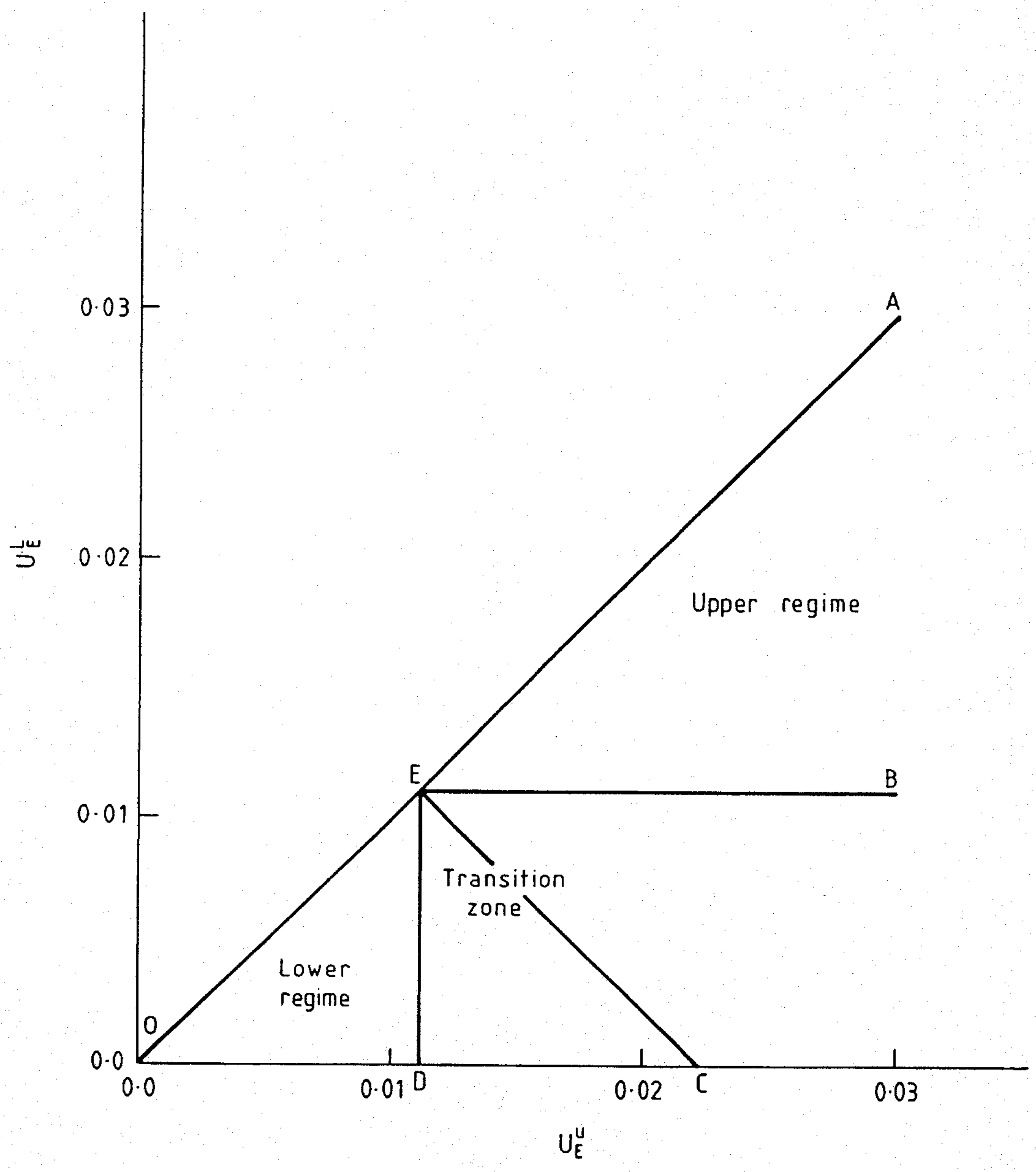

Fig 6 Criterion for selecting Upper or Lower regime

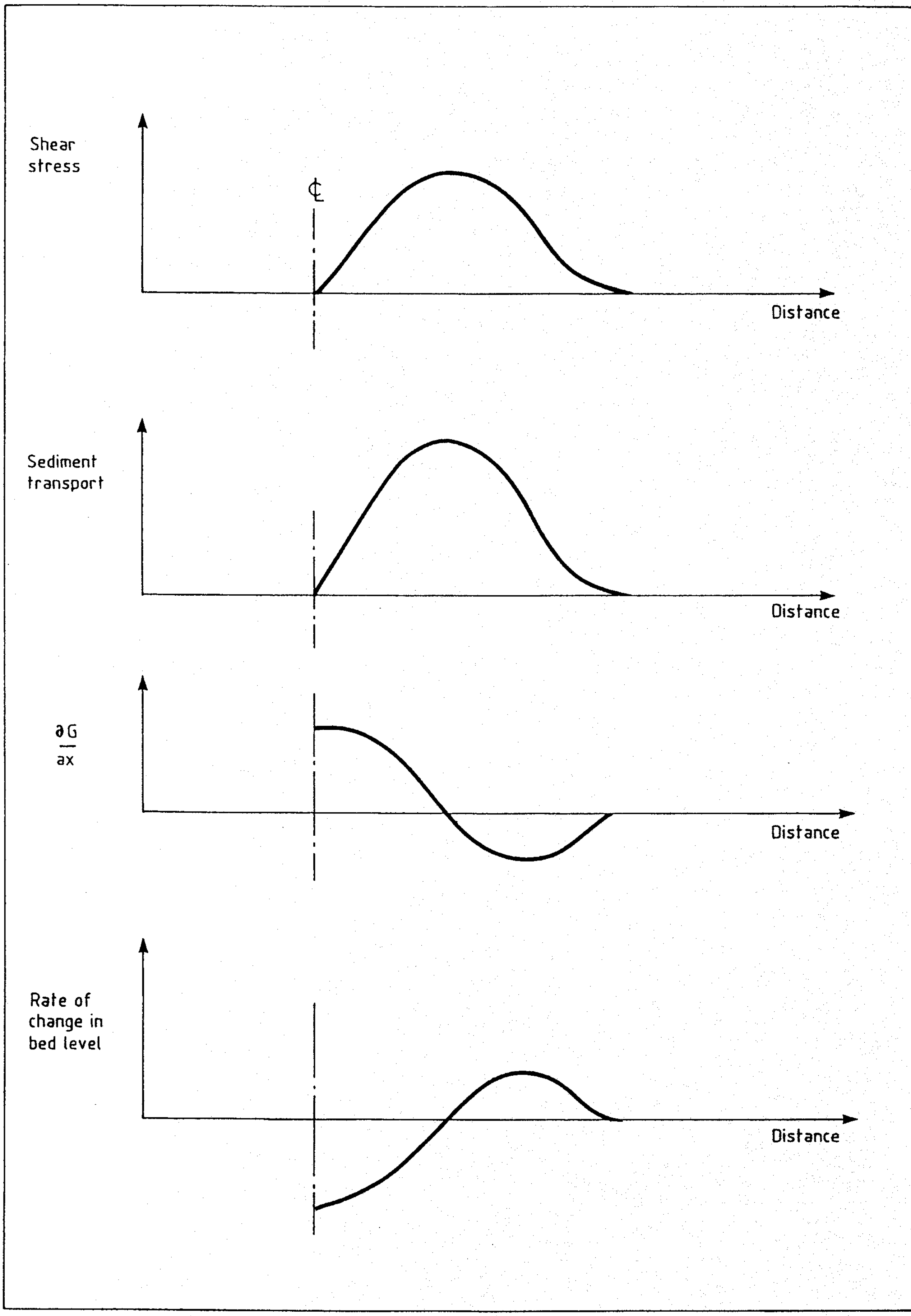

Fig 7 Idealised scour in one-dimension

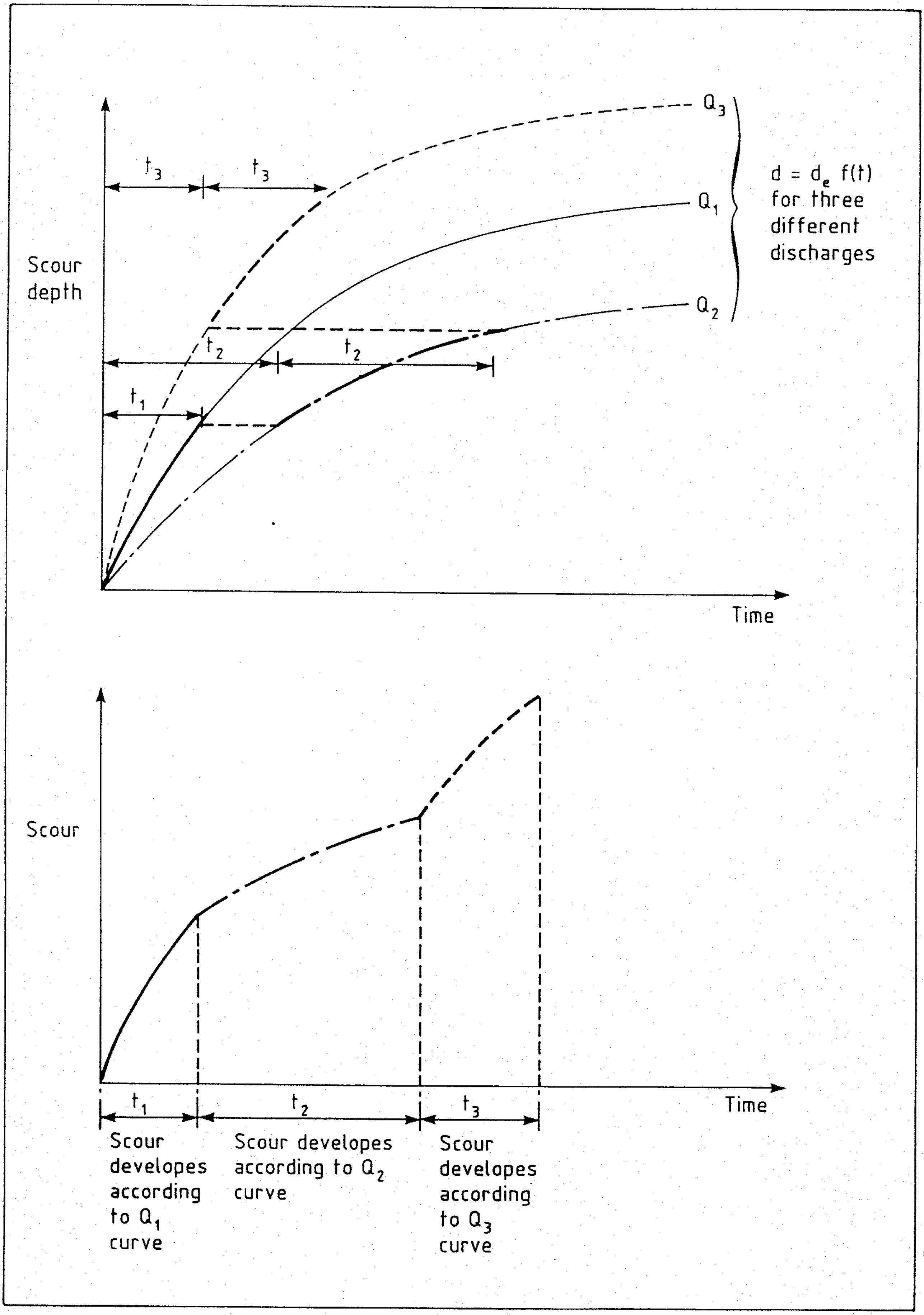

Fig 8 Development of scour with time

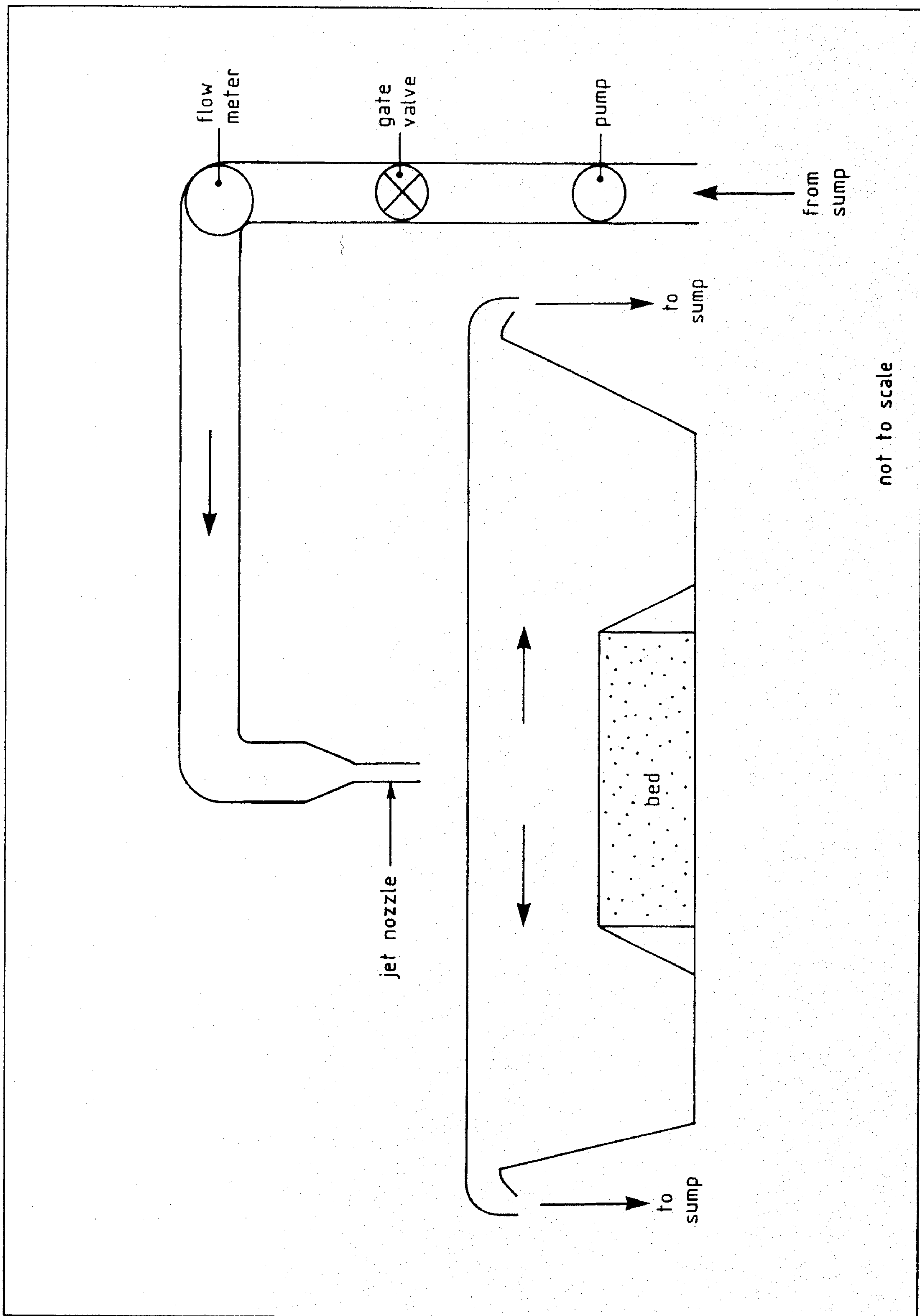

Fig 9 Diagrammatic sketch of apparatus

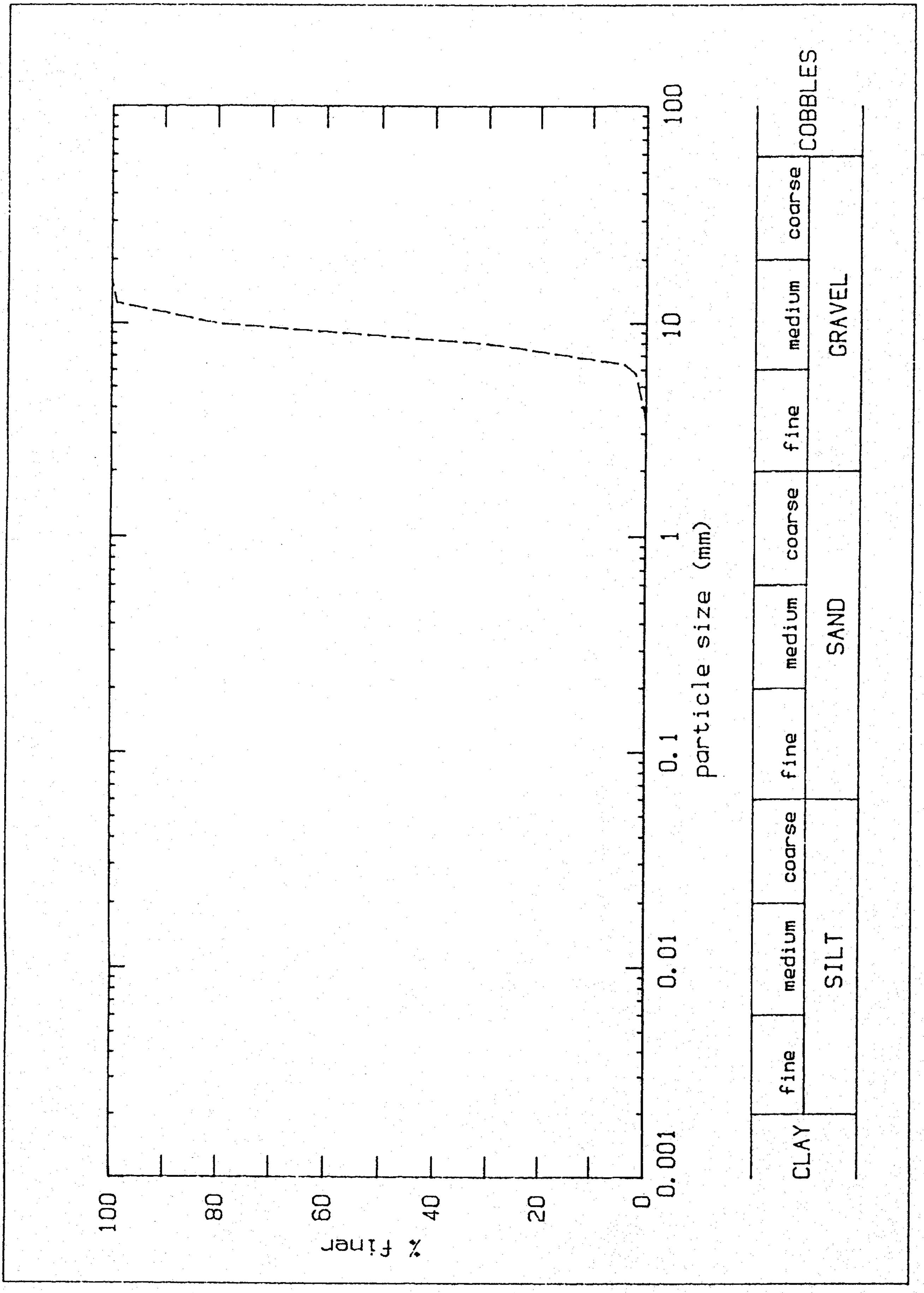

Fig 10 Particle size distribution of bed material

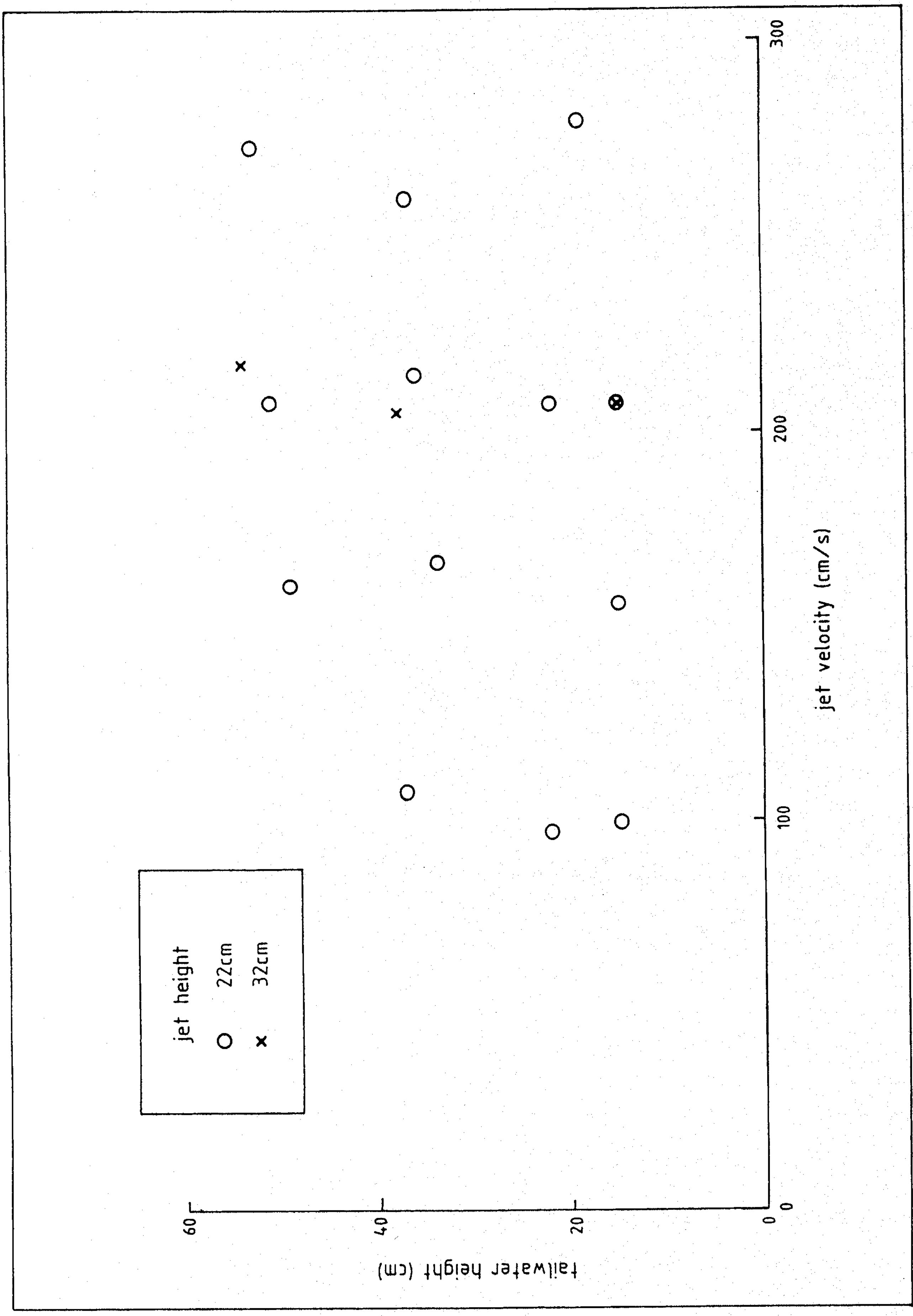

Fig 11 Details of tests carried out

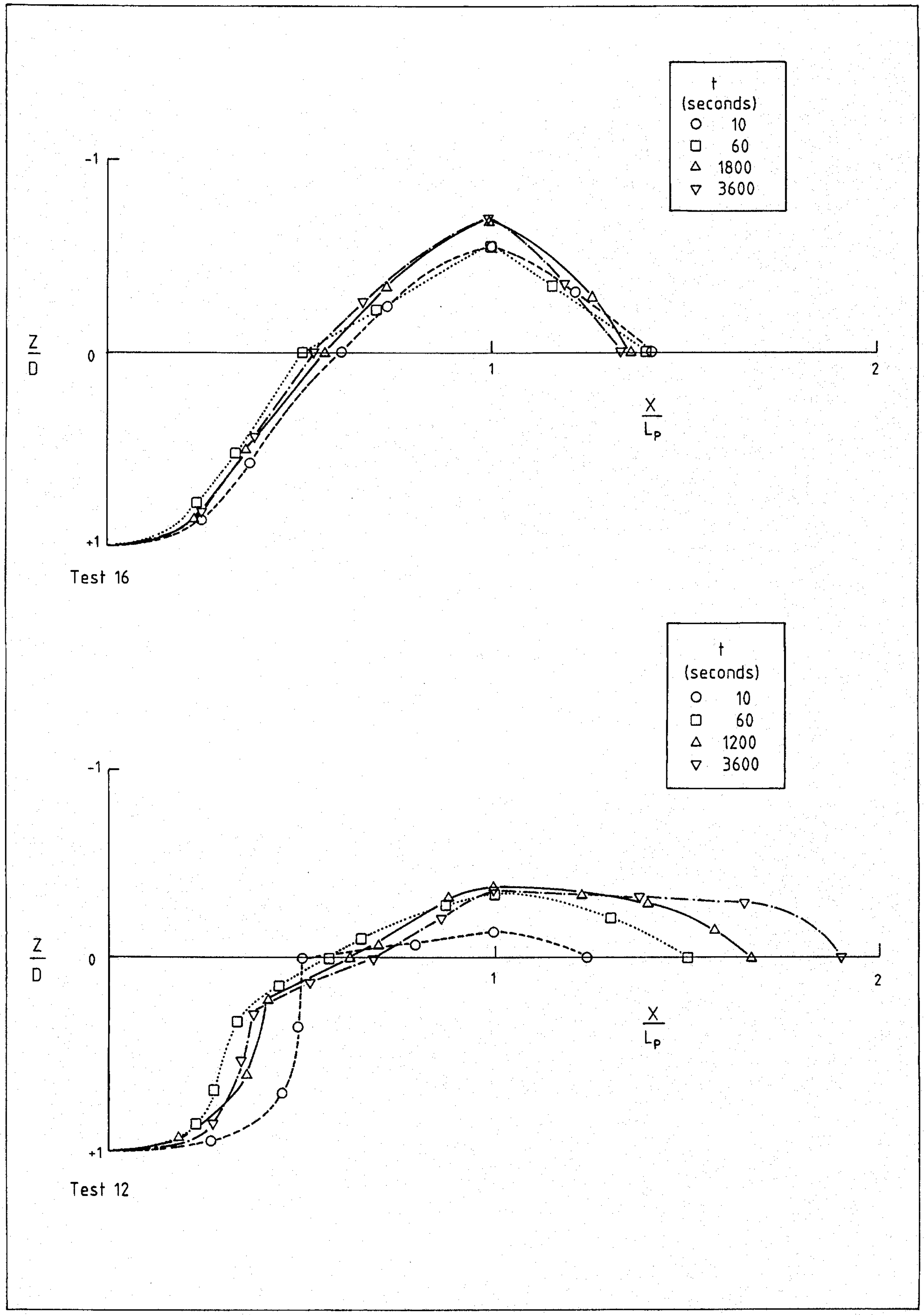

Fig 12 Non-dimensional scour hole shape

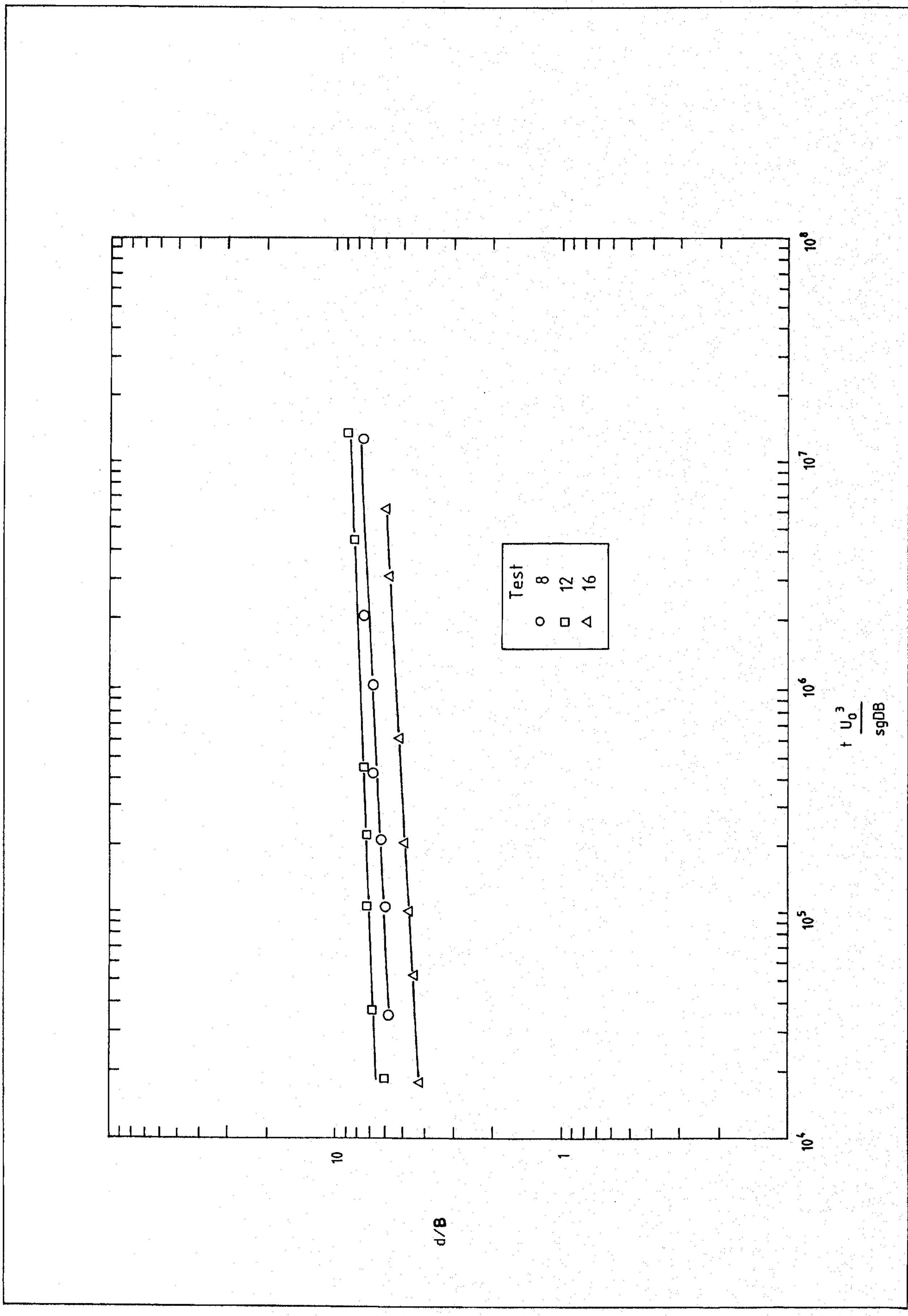

Fig 13 Time development of scour