Hydraulics Research
Wallingford

SEDIMENT MOVEMENT IN SEWERS

Interim Report

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SUMMARY

Following from earlier work on sediment in sewers carried out at the Hydraulic Research Station (HRS), a new programme of work, funded by Construction Industry Directorate of Department of the Environment (DoE), is under way at Hydraulics Research (HR). A 300 mm diameter concrete pipe is being studied with a 0.72 mm sand, at concentrations up to the limit of deposition, over a range of flow velocities from 0.5 to 1.3m/s. Pipe-full and part- full conditions are being studied, and head-loss due to the presence of the sediment is being measured. In this report the experimental rig and methodology are described in detail, and data so far collected at the limit of deposition are compared with existing formulae. Measured values of hydraulic gradient without sediment present are analysed to assess the effect of shape factor on resistance. Measured values of velocity at the threshold of movement are compared with existing theories.

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List of variables

A	flow area
B	free surface width
Cd	drag coefficent (Durand-Condolios equation)
Cv	volumetric sediment concentration (=Qs/Q)
Cvp	predicted value of Cv
D	pipe diameter
Dgr	dimensionless grain size (after Ackers-White)
d	particle size
d50	sieve size for which 50% by weight passing
E	specific energy on centre line
Fr	open-channel Froude number ($=V\sqrt{B}/\sqrt{g.A}$)
g	gravity (=9.8066 m/s^2)
i	hydraulic gradient
k	roughness size
k_s	Nikuradse equivalent sand roughness
L	pipe invert level
$\bar{L}$	mean value of L along profile
M	sediment sensor reading
Mc	clear water sediment sensor reading
m	energy gradient (+ive increasing E in d/s direction)
N	number of readings
n	Manning's n
P	wetted perimeter
Q	volumetric flow rate of water
Qs	volumetric flow rate of sediment
Qs^*	transport parameter in Macke equation ($= Qs.\gamma.g.(s-1).w^{1.5}$)
R	hydraulic radius
Re	Reynold's number ($= 4VR/\nu$)
Rey	modified Reynold's number ($= Vy/\nu$)
So	invert slope (+ive for fall in d/s direction)
s	specific gravity of particle
T	water temperature (°C)

V mean velocity at cross-section

$\bar{V}$ mean of all V values along profile

V_t threshold velocity

V_{tr} predicted Vt (Novak and Nalluri - rough pipes)

V_{ts} predicted Vt (Novak and Nalluri - smooth pipes)

w fall velocity of particle in water

y maximum depth at cross-section

$\bar{y}$ mean of y values along profile

γ specific weight of water

λ Darcy-Weisbach friction factor

λ_o value of λ for clear water when comparing with limit of deposition

λ_p predicted value of λ from Colebrook-White

θ proportional increase in λ $(=\lambda-\lambda_o/\lambda_o)$

Ψ shape effect correction to λ

Ψ_{kc} predicted Ψ from Kazemipour (semi-circular)

Ψ_{kr} predicted Ψ from Kazemipour (rectangular)

Ψ_{na} predicted Ψ from Nalluri and Adepoju

τ_o shear stress

σ_s standard deviation of sample

σ_n estimated standard error of mean

1. INTRODUCTION

1.1 Background

Recent research on the movement of sediment in pipes has shown that the velocity necessary to maintain deposit-free conditions is dependent on a number of factors including the type and diameter of the pipe, and the concentration and characteristics of the sediment. A program of experimental work was carried out by May (11) at the Hydraulics Research Station (HRS) with the intention of producing design recommendations applicable to storm and combined sewerage systems. Pipe diameters of 77mm and 158mm were studied with non-cohesive sediments of sizes in the range 0.6 to 8mm with a specific gravity of 2.65. May developed a new theoretical model for the transport of sediment at low concentrations in pipes, and a design formula was proposed, based on this and on the experimental results.

A number of earlier studies, including those of Robinson and Graf (16), Laursen (9) and Novak and Nalluri (14) each produced their own formula for predicting the limit of deposition. These were compared with the new data, and each gave widely varying predictions of concentration for the pipe sizes and sediment sizes covered by the HRS study. This highlights the limitations of design formulae which are empirically derived, when it is attempted to extrapolate beyond their measured range.

The HRS formula had a theoretical basis, but it was necessary to introduce an empirical factor to account for the effect of the sediment size relative to the pipe diameter. Pipes used in storm sewerage systems are typically of larger diameter than those used for the HRS study, and this has been a drawback to most of the studies of sediment transport in pipes so far carried out.

CIRIA (2) commissioned a report aimed at reviewing the present problem of sediment in storm and combined sewers and identifying what research was necessary to close the principal gaps in present knowledge. A specific need which the report identified was for "laboratory studies to clarify scale effects in modelling non-cohesive sediment movement".

1.2 Present study

A new programme of experimental work is under way at Hydraulics Research, extending the scope of the earlier experiments to larger pipe diameters, and to those of the type more commonly used in sewerage systems. Experiments are currently in progress on a section of 300mm diameter concrete sewer pipe mounted in a tilting flume which will allow pipes up to 450mm diameter to be studied. The aim of this research is to determine how accurate are the design criteria already proposed when applied to pipes of larger diameter than those previously studied. If necessary, new design criteria will be developed which can be applied to pipe sizes covering the full range of both studies. It is intended to further extend the study to sediment-laden flow over a deposited bed.

In this report, the results so far obtained at the limit of deposition are compared with the original HRS formula and with those proposed by Macke (10) and most recently by Nalluri and Mayerle (13). The latter two formulae are also compared with the earlier HRS data. Measured values of the flow velocity at threshold of movement are also compared with the formulae derived by Novak and Nalluri (14),

(15) from their extensive studies of the incipient motion of sediments over a wide range of bed roughness and particle sizes. The measured values of hydraulic gradient without the presence of sediment are analysed to assess the effect on resistance of changing shape of cross-section at different flow-depths. Head-loss data at the limit of deposition are compared with existing theories.

2 TEST ARRANGEMENT

2.1 General layout

The experiments are being carried out in a converted 2.44m wide tilting flume (see fig 1) in which flow is supplied to the test pipe by up to three pumps having a total capacity of around 0.25 m^3/s. Pipes up to 450mm diameter can be studied, but initially a 300mm dia. concrete pipe has been installed. The pipe is mounted in one side of the flume, the other half of the flume acting as an overflow channel. Flow into the head of the system can pass into the sewer pipe over a 1.22m wide rectangular thin plate weir or, depending upon the setting of a tilting weir, a controlled proportion can be allowed to pass down the overflow channel. This system allows flow into the sewer pipe to be varied rapidly for accurate simulation of floods. In addition to the main flow, sediment is recirculated with a small proportion of the liquid discharge, by a slurry pump whose discharge is measured using an electro-magnetic current meter. The sediment concentration in the recirculation pipe is measured using a infra-red sensor.

The pipe being tested has a total length of 21m and nominal internal diameter of 300mm. It is comprised of 2.52m long sections of ROCLA spun-concrete sewer pipe and was measured to have a mean internal diameter of 298.83mm, with a standard deviation of σ_s = 2.89mm (see fig 2). The individual lengths of pipe have a spigot-and-socket type of joint, and should be laid with spigots pointing downstream. For practical reasons it was necessary to fix the pipe to the bulkhead at the upstream end, so the pipe was laid with sockets pointing downstream. This has the effect that the joints present a small (approx 2-3mm) expansion in the downstream direction, and thus deposition will not be encouraged at the joints. Internal gaps between the pipe lengths vary from zero to approximately 20mm depending on the fit at individual joints. The pipe was laid on wooden blocks such that the invert was as level as possible when the flume was level. Subsequently invert levels were checked along the pipe and, at gauge positions, deviations from mean invert level were found to lie in the range $-0.2 \leq L-\bar{L} \leq 2.1$mm. For all points measured, $-4.9 \leq L-\bar{L} \leq 2.4$mm. Pipe slope can be varied up to around 1/100.

Each pipe length has two 900 x 90mm slots cut in the top to allow observation of bed conditions along the length of the pipe. Flush-fitting, transparent lids were built to re-seal the pipe for tests at pipe-full flow whilst still allowing observation of the bed.

Depth in the pipe can be controlled using an adjustable sluice gate at the downstream end, flow from the pipe falling freely into a hopper where sediment is allowed to settle. The sediment is extracted, with a small proportion of the flow, from the bottom of the hopper, and recirculated by the slurry pump to the head of the sewer. The remaining discharge spills over the sill of the hopper into an outer tank, thus maintaining a constant head over the slurry pump. Mesh screens around the sill prevent sediment in suspension from escaping into the outer tank.

The hydraulic gradient along the pipe is measured using five digital depth gauges mounted above the pipe at 2.52m intervals along part of its length. The point gauges are fitted with a battery powered electronic detector circuit, which emits an audible "squeak" when the tip of the gauge is in contact with the water. This is of particular assistance when measuring the depth for part-full flow tests, when the water surface is measured directly, and fluctuations in level cause the gauge to "dip" into and out of the water. For pipe-full flow the level is measured in stilling wells mounted on the transparent perspex lids. These stilling wells are in communication with the pipe via 0.5mm diameter holes, being small enough to reduce periodic fluctuations in water level to around ±1mm.

2.2 Sediment concentration measurement

In the earlier HRS tests, dry sand was added at the upstream end of the test pipes using a vibrating screw sediment injector, and removed at the downstream end by collecting it in a hopper. This was found to have a number of drawbacks. The screw-injector was tending to grind the sand, reducing its d50 size as tests went on, also it was necessary to keep drying large quantities of sand after every test before it could be reused, and the injector could not be relied upon to give a constant rate of supply.

In order to achieve the much higher transport rates expected in the larger diameter pipes, without demanding huge quantities of dry sand for every test, a new method has been devised for the present set of experiments. This uses a re-circulating sediment system which measures sediment concentration by the interruption of an infra-red light beam. It is an application of the widely used technique for measuring concentration of silt particles in suspension using a light beam, and allows wet sand to be continuously re-circulated without the need for drying and weighing of samples.

At the downstream end of the sewer pipe, mixed sediment and water falls into a hopper, where the sand is allowed to settle to the bottom. Clear water overflows into a separate, surrounding tank, through mesh screens which prevent any sediment in suspension from escaping. A slurry pump collects the sediment from the bottom of the hopper and recirculates mixed sand and water at a pre-set velocity to the head of the system via a 75 mm diameter pipe. The flow velocity in this sediment return pipe is measured using an electro-magnetic current-meter (ECM) which is not affected by small sediment concentrations. The sediment return pipe has a 1m long perspex section part way along, with an infra-red light source mounted against the invert on the outside of the pipe (see fig 3). The source shines a "pencil" beam across the diameter of the pipe to a sensor opposite it, also mounted on the outside of the pipe. Sand passing along the pipe interrupts the beam, increasingly with increasing concentration, and reduces the strength of signal arriving at the sensor. The output signal is fed to an amplifier unit which converts the sensor signal to a voltage, nominally in the range 0-1 volt, but variable using gain and balance settings. For the sediment tests these were set to give an output of .988 V for no signal reaching the sensor, down to around 0.1V for clear water. From the amplifier, the signal is fed both to a chart recorder to produce a hard copy, and through a voltage-frequency converter to a counter. The counter can be set to count over a given time period from 1 second up to 9999 minutes to give a mean reading for that period. After passing the sensor the sand and water is fed back into the head of the sewer pipe, thus maintaining a uniform mean sediment transport rate through the system. Fig 4 shows schematically the layout of the measurement and recording system.

The response of the infra-red device is dependent on both sediment size and flow velocity in the sediment pipe. The dependence on flow velocity is advantageous, in that a wide range of transport rates can be covered by only a few pipe velocities: increasing the pipe velocity reduces sediment concentration for a given transport rate, and therefore reduces the response of the infra-red sensor. The flow velocity and sediment concentration in the sediment pipe can be altered to suitable values without affecting the corresponding conditions in the sewer pipe.

Before the system could be used, it was necessary to calibrate the infra-red sensor over a range of sediment concentrations and sediment pipe flow velocities. Initially, a 0.72mm, narrow graded sand was chosen for investigation at limit of deposition. Based on expected transport rates for the range of sewer velocities to be studied, initially two calibration velocities were chosen, and tested over the full response range of the sensor.

Before and after each calibration a sensor reading was taken with no sediment present. This, the "clear-water" reading, was found to vary by a few percent from the one test to another. At the other end of the scale, a reading was taken with the infra-red source switched off. This reading was found to be constant, confirming that ambient light levels were not affecting the readings.

The sediment sensor calibrations were carried out using a 2 litre plastic beaker, with holes of various sizes drilled in the base to allow a range of injection rates. The holes were taped over, and the beaker filled with sand and weighed. It was then mounted above the hopper at the downstream end of the sewer pipe, with a funnel and vertical pipe to catch the sand from the beaker and carry it directly down to the slurry pump offtake. With the return pipe set at the required calibration velocity, tape was removed from one or more holes, and a stop watch was started. Sand was then added to the beaker from a pre-weighed supply, to keep it topped up to a constant level. When all the pre-weighed sand had been used, the holes were resealed and the stopwatch stopped. The beaker was then reweighed, and mean injection rate calculated as

$$\frac{\text{initial beaker sand + pre-weighed sand - final beaker sand}}{\text{duration of test}}$$

The amount of pre-weighed sand was chosen to give a test duration of at least five minutes - at the very highest injection rates the amount of sand required make a longer calibration impracticable. A hard copy of the calibration output was retained from the chart recorder, but actual sensor readings were obtained from the counter, which was set at 100s counting period. The chart record served only as a check on the counter output, and was useful in determining how steady the sand supply rate remained during the test.

It was found that the lowest transport rates (below approx 2g/s) could not be achieved using this arrangement as the sand tended to arch above the hole in the beaker if it was smaller than about 4mm, and a steady rate of supply could not be relied upon. Therefore, a simple vibrating wire, driven by s small electric motor was used to allow a smaller beaker and hole diameter to be used. This allowed calibrations to be carried down to 0.16 g/s, which is of the same order at the lowest expected transport rate to be studied in the 300mm pipe.

It was necessary to normalise the sensor output in some way to account for variations in the clear-water reading. These variations could be ascribed to two main causes:

1 Changes in the sensitivity of the sensor, due to temperature and power fluctuations.

2 Changes in the transmissivity of the water due to presence of fines and air

Other possible factors include electrical interference from other equipment and physical movement of the heads, but these are not thought to be significant.

If one introduces a theoretical "pure water" reading - ie. the reading which is obtainable from water with no air or fines present - then the normalised reading will be equal to the change in signal due to the present of sediment, divided by the full range of the instrument.

ie.

$$\frac{\text{clear water reading - actual reading}}{\text{source off reading - pure water reading}}$$

If the sensitivity of the sensor changes, the all readings below "source off" should change proportionally, including the "pure water" reading. Therefore, if it were assumed that all fluctuations in clear water reading were due to changes in sensitivity only, then the quantity

$$\frac{\text{source off reading - clear water reading}}{\text{source off reading - pure water reading}}$$

should be constant, and it is appropriate to normalize the output as

$$\frac{\text{actual reading - clear water reading}}{\text{source off reading - clear water reading}}$$

If all fluctuations are due only to changes in the transmissivity of the water, then the "pure water" reading would be constant and it is appropriate to use normalize readings as

actual reading - clear water reading

Early calibrations tests yielded a very non-linear relationship between sensor output and concentration, with the sensor showing a tendency to "saturate" at low concentrations. This non-linear response was unacceptable because, if short-term variations were meaned with respect to time, the calculated mean concentration would be distorted from its true value. The character of the sediment transport in the sediment pipe - widely spaced particles passing rapidly across the beam - means that such a distortion could be very considerable. This would be still worse if dunes were passing along the sewer pipe, causing longer-term variations in concentration. By reducing the strength of the source it was possible to achieve an approximately linear response over about 70% of the sensor range. (i.e from clear water up to a concentration giving 70% of the signal for "source off"). For concentrations beyond the linear range, increases in concentration produced progressively smaller changes in output signal. This was expected, as once the thickness of the sediment layer increases significantly, some of the sand particles will lie in the shadow of others, reducing their net effect on the signal strength reaching the sensor. This does not present a problem because, as mentioned, by moving to a higher pipe velocity, the higher transport rates can be measured within the linear range of the sensor.

Calibrations were obtained at velocities of 0.69, 1.39 and 2.15m/s which gave consistent results covering transport rates from 0.16 up to around 50g/s. The calibrations were determined using both the normalizing techniques described above, and very little difference was found between them, both giving a response which could be regarded as approximately linear over 70% of full range, and both having a standard deviation of 8.4%. It was decided that there was more evidence that fluctuations were due to changes in the transmissivity of the water, than to changes in the sensitivity of the measuring system, so the calibrations chosen for use in the experiments to measure the limit of deposition were those calculated using

actual reading - clear water reading

Figure 5. shows the calibration obtained at velocity = 1.39m/s in the sediment return pipe.

Once some initial problems were overcome, the infra-red sensor proved to work well, and is saving a great deal of work in the shape of drying and weighing sand samples. By allowing the system to settle for around 30 minutes before using it for measurements, variations in clear-water reading have been reduced to below 5%, and if a clear-water reading is obtained immediately before or after each sediment test, these variations should not adversely affect the accuracy of the concentration measurement.

3 EXPERIMENTAL PROCEDURE

3.1 Clear water roughness

Before any experiments with sediment took place, a series of clear-water tests were carried out, in order to obtain an estimate of the value of k_s (Nikuradse equivalent sand roughness) for the actual pipe being studied, and in order to develop a workable system for setting uniform flow conditions at part-full. Clear-water roughness was also measured immediately prior to each limit of deposition test, at the same flow conditions as those to be studied at limit of deposition.

The procedure adopted in all these tests was to set a particular discharge without sediment present, then adjust the flow depth to the required value, and take a measurement of hydraulic gradient using the digital point gauges. For pipe-full tests, the slope of the pipe was set at some convenient value such that water surfaces at the gauging points were within the stilling wells, and as low as possible to minimise leakage around the lids. The pipe was surcharged by lowering the downstream sluice gate gradually until this state was reached. The slope was not changed from one test to the next unless necessary for this reason. A "still-water" reading was taken at each flume slope setting, this reading acting as a datum for calculation of hydraulic gradient. The reading was obtained by stoppering the sewer at the down-stream end and filling it slowly until a still water level could be measured in each of the gauged stilling wells. These still-water readings were checked periodically to ensure that the gauges had not moved.

For part-full tests the slope was adjusted to achieve conditions as close as possible to uniform flow. This was not always easy, particularly if normal depth was close to, or less than critical depth for the required velocity. Disturbances of the flow at entry and exit from the pipe cause the water surface to fluctuate periodically by 2-3mm at the gauge positions, and irregularities in pipe section at the joints and elsewhere cause standing waves with amplitudes of up to 15mm for subcritical flows, and as much as 20mm when the flow is supercritical. This means that some gauge readings are not representative of the average depth in the vicinity of that gauge, and that the mean velocity can vary significantly from one section to another. The criteria for adjustment of flume slope and gate setting were therefore necessarily flexible, and no strict formula could be applied which would define the "best" approximation of uniform flow conditions. Generally, it was attempted to get at least three of the five gauges reading the required depth to within ±2mm, but this was not always possible, and the gauges used would vary according to the individual flow conditions. Once the slope was set, all five gauges were read, and average hydraulic gradient along the test section was usually calculated from all the readings. The only exception to this was for tests at 1m/s for y/D = 0.5, when it was found that the depth at gauge 3 was some 20mm lower than elsewhere. In this case, the hydraulic gradient was calculated from the other four gauge readings.

In the measurements made immediately preceding the additions of sediment for study of limit of deposition, two sets of depth gauge readings were taken. For pipe-full tests, the readings were taken in the same way, by lowering the depth gauge gradually until it touched the water surface in the stilling well. The second test was therefore a simple independent repeat of the first. For part-full tests, because of fluctuations in the water surface, determining the mean depth is more subjective. For this reason, a different criterion was adopted for the two sets of readings. For the first measurement the depth gauge was gradually lowered until it was considered that the tip was submerged for approximately 50% of the time. For the repeat measurement the tip was raised again, the lowered until the tip was continuously submerged. In the initial set of tests, using clear water only, the depth gauges were read once only, using the "50% submerged" criterion.

3.2 Threshold of movement

Tests were carried out to obtain an approximate value for the velocity at which isolated sand particles would start to move in the sewer pipe. The procedure was to set a flow depth and velocity, then add a few sand particles by hand and see whether they continued to move having fallen to the bed. If the particles failed to move the velocity was marginally increased and slope re-set in order to obtain approximately uniform flow conditions. This process was continued until movement was observed. The rig was not specifically designed for such measurements, and it was not practicable to carefully position particles on the invert, nor to carry out tests for pipe-full flow. Two readings were obtained, at approximately $\frac{1}{4}$ full and $\frac{1}{2}$ full conditions.

3.3 Limit of deposition

Once the gradient and sediment sensor reading had been recorded for clear water conditions, sediment was gradually added to the system. The sediment used was a narrow-graded sand with $d50=0.72$mm and specific gravity of 2.62. A grading is shown in Fig. 6. In order to prevent immediate formation of dunes it was found that the best method was to throw sand by hand into the jet as it fell into the hopper from the downstream end of the sewer pipe. This allowed the sand to mix with the water in the hopper before being extracted by the slurry pump, rather than travelling along the sediment pipe as one "slug". At first it was found that large quantities of sediment were escaping over the sill of the hopper, so the mesh screens were added, and the back of the tank was raised to accommodate the additional head difference across the mesh. Another difficulty was that some of the sand deposited on the sides of the hopper rather than falling to the bottom, this despite the steep (45°) sides and considerable turbulence within the hopper. This became most apparent with the part-full tests, where the discharge from the sewer pipe was less, and therefore less turbulent mixing occurred in the hopper. In order to minimise this deposition, water jets were added in the corners of the hopper to wash the sand off the sides and back into suspension where it could be collected by the pump.

Sand was added until the limit of deposition was observed. At low flow velocities (below around 1m/s) this was taken to be the point at which particles would bunch together and cease to move for a few seconds before being dispersed and carried away by the flow. This condition could be satisfactorily observed from above - very easily for pipe full tests through the transparent windows in the top of the pipe. At higher velocities, as with the earlier HRS tests it was found that as sand was added there was a gradual transition from flume traction to flow over a continuous moving bed. In this case, as concentration is increased, although particles on the invert may be in continuous motion, they are not being moved directly by the flow, but are being carried along by shear forces transmitted by the layer of particles above. The limit of deposition is taken to be the state when particles on the invert are still just being moved directly by the flow. A small increase in concentration will cause the particles on the invert to become closely packed and move only due to forces transmitted by the layer above. Eventually, when the concentration in the flow is high enough, the moving deposit will thicken until the shear force exerted on the particles on the invert is equal to the frictional resistance and they will cease to move. Clearly in this instance it is not possible to judge when limit of deposition has occurred, solely by observations from above, as the slow-moving deposit on the invert is obscured by a continuous moving bed above it. For this reason it was necessary to install windows along the invert of the pipe to allow observation from below.

It was necessary to decide which section of the pipe should be used for determining when the flow was at the limit of deposition, as clearly the local disburbances in the flow caused certain sections to deposit before others. A particular section around mid-length was chosen, which seemed to be "typical" in terms of how soon it would form a deposit relative to other parts of the pipe. Judgement was primarily based on the conditions at this point, but the full length of the pipe was always checked to ensure that local dunes had not formed elsewhere.

Once it was decided that the flow was at the limit of deposition, a minimum of about 15 minutes was allowed for the system to reach equilibrium. A series of 5-10 consecutive readings were recorded from the concentration sensor, each reading representing the mean concentration for a 100s period. The hydraulic gradient was measured again, and for part-full tests the slope was adjusted to restore uniform flow conditions if necessary. Two sets of water level readings were taken for each test, as with the clear-water measurement. The fluctuations in water level already described tended to make it difficult to detect the small increases in roughness between clear water and the limit of deposition. This was even so for the pipe full tests where an additional problem was the presence of air travelling along the pipe and escaping into the stilling wells, causing further oscillation of the column of water.

In most of the tests, limit of deposition conditions were maintained for the ten to fifteen minutes whilst concentration and head loss readings were being recorded. If this was the case, limiting concentration was calculated as above from the mean of all readings. Sometimes however, it was not easy to identify the limit of deposition and achieve steady conditions. An inevitable consequence of reaching limit of deposition is that the flow experiences some decrease in sediment concentration, so that the downstream portion of the pipe is starved of sediment and the flow there will not be at the limit of deposition. The system employed for these tests, of recirculating the sediment to the head of the pipe means that inevitably there is a certain degree of unsteadiness in the rate of sediment supply, and it is only the mean concentration over several minutes that remains constant. In some cases it was found that limit of deposition would be observed, but that due to this starvation effect, subsequently the concentration would fall to a lower value. If this was the case only the readings taken when the flow was actually observed to be at limit of deposition were included in the calculation. Similarly, water level readings were only used with the corresponding concentration reading taken for the same period.

Once the necessary readings had been taken at the limit of deposition, the sluice gate was positioned across the lower half of the outfall in order to trap the sediment in the sewer pipe. The slurry pipe was then allowed to continue running until clear water flowed past the infra-red sensor. This clear-water reading was recorded for comparison with the equivalent reading at the beginning of the test, and for calculation of actual sand concentration at the limit of deposition, using the normalizing procedure that was used for calibrating the sensor (section 2.2)

4 EXPERIMENTAL RESULTS

Details of the results of the experiments carried out so far are shown in Tables 1 and 2. Table 1 comprises: test no., date, proportional depth, mean flow velocity, volumetric sediment concentration, water temperature, Darcy-Weisbach friction factor, proportionate increase in fraction factor from clear water. Table 2 shows results at the limit of deposition only.

4.1 Friction analysis

For pipe-full tests, the hydraulic gradient was taken to be the mean water surface slope with respect to the still-water reading. This was calculated directly from water levels in the stilling wells, using least squares regression. If one point gauge was clearly in disagreement with the others it was excluded from the regression.

The method used to determine the hydraulic gradient i for part-full tests was a follows. Mean flow velocity was calculated at each gauge position, based on the recorded flow depth and total discharge. Specific energy, E at each point could then be determined from:-

$$E = Y + V^2/2g \qquad (1)$$

where Y is depth at the centreline. V is mean velocity at the section, calculated as discharge divided by flow area at the section, and g is the gravity constant.

The best-fit energy gradient, m (positive for E increasing in the downstream direction) was determined using least-squares regression on all the points.

All points were used to avoid distorting the calculation of mean velocity, and because it was found that calculated roughness values were more consistent. An exception to this was the case mentioned in 5.1 with V=1.0m/s, and y/d=0.5, where only four gauge readings were used. The hydraulic gradient i was then found from:-

$$i = So - m \tag{2}$$

where So is the slope of the bed.

In both pipe-full and part-full cases, Darcy-Weisbach friction factor is calculated from:-

$$\lambda = 8gRi/V^2 \tag{3}$$

where R is the hydraulics radius, calculated from the mean depth along the profile.

A "measured" value of k_s could then be determined from the Colebrook-White formula for commercial pipes

$$k_s = 14.8R\ (10^{-1/2\sqrt{\lambda}} - 2.51/Re\sqrt{\lambda}) \tag{4}$$

where Re is Reynolds number (=4VR/ ν)

For comparison, values of Manning's n were also calculated from

$$n = R^{2/3}\ i^{\frac{1}{2}}\ /V \tag{5}$$

4.2 Results for clear water

Before tests with sediment were initiated, a number of measurements were made to determine the clear-water roughness value, k_s to be used in the Colebrook-White resistance formula. It was expected that the value of k_s would remain approximately constant over the full range of flow conditions to be studied. These measurements covered flow velocities in the range 0.18 to 2.09 m/s, at flow depths approximating to ¼ full, ½ full, ¾ full, pipe full, and just below pipe full (y/D = 0.95). The measurements at y/D = 0.95 were made in an attempt to assess any influence the lids might have on the roughness. Hydraulic gradient was also measured immediately before each limit of deposition test, with the same flow velocity and depth as that to be studied. The previous HRS tests had shown that if any increase in head loss due to sediment was to be observed, this measurement was necessary, and it was not sufficient to rely only on a predicted value of λ, or even on values measured at the same flow conditions but at a different time.

All 80 clear-water results are included in Table 1, and calculated values of k_s are shown plotted against Re in Fig 7. The results are reasonably consistent over a wide range of Reynolds numbers, although there are a number of outliers, occurring particularly at low velocities. The overall mean value of k_s, from all measurements is k_s = 0.177mm with σ_s = .235mm. This suggests that it is highly improbable that any roughness measurement greater than 0.647mm (being ks + $2\sigma_s$) is correct and it is therefore justifiable to exclude the four outlying measurements from the analysis.

In the pipe-full tests the second head loss reading is effectively a repeat of the first, taken 2-3 minutes later in exactly the same way, so any difference between the readings should result only from unsteadiness in the flow or from experimental error, and the two readings are thus essentially independent. For the later part-full tests however, the second reading, taken with the gauge tip continuously submerged, inevitably shows a lower depth and hence higher predicted mean velocity than the first. For this reason the predicted friction factor is invariably lower using the second method. The true mean depth should lie somewhere between the two readings, as surface tension causes the 50% submerged readings to give a slightly high setting, and the continuously submerged reading clearly estimates too low. It was hoped that the second method might give more consistent results, but from the readings obtained so far, this does not appear to be the case, with the first method giving results with less scatter about the mean. For this reason the values obtained with the gauge tip fully submerged have been entirely excluded from the analysis. All the readings are included in Table 1, those obtained using the second method being indicated *.

The mean of all the clear water measurements included in the analysis is $\bar{k}_s$ = .1340mm, with σ_s =.0792mm. The estimated standard error of the mean is therefore σ_n=.00975mm, giving $.1275 \leqslant k_s \leqslant .1405$mm at a 50% confidence level. The predicted values of Manning's n and λ for k_s = 0.134mm with pipe-full conditions are shown along with the measured values for comparison on figs. 8 and 9.

By treating the results for different values of y/D separately, the mean value of k_s can be observed to vary from 0.258mm at y/D = 0.95 down to 0.105mm at y/d = 0.25 (see Table 3). These variations, whilst determined from only a few values, are statistically significant at a 50% confidence level, and demanded further investigation. (see 5.1)

4.3 Threshold of movement

Only two tests were carried out and were intended only as an approximate measure of the threshold velocity for the pipe and sediment being studied. The results are shown in table 4, along with predicted threshold velocities from the formulae derived by Novak and Nalluri. The original formula derived for smooth pipes (14) is

$$Vts = 0.61(g(s-1)d)^{\frac{1}{2}} (d/R)^{-0.27} \quad (6)$$

Where s is specific gravity of the sediment.

It predicts a slightly lower value of threshold velocity than those measured in the concrete pipe. Novak and Nalluri (15) extended the study to include groups of particles on smooth and rough beds. They found that threshold velocity for a single particle on a rough bed (Vtr) was higher than a smooth bed, and that the velocities were related as

$$Vtr/Vts = 1 + 1.43 (d/k)^{-0.4} \quad (7)$$

In carrying out the study they roughened the bed surface artificially by glueing sand or sandpaper to it, k in equation (7) being the size of roughness. The smallest roughness size that was studied was 0.3mm, and the sediment size was always greater than the roughness size. By considering the concrete pipe being studied here to be "rough" rather than "smooth" it is possible to calculate what k size is needed in the formula to bring predicted and measured values together. These k values come out to be only .0013 and .0016mm, in both cases far below the roughnesses studied by Novak and Nalluri, and by their criteria the pipe should therefore be considered "smooth". This suggests that further study of incipient motion on "intermediate" surfaces such as concrete would be valuable.

4.4 Results at limit of deposition

Readings of sediment concentration and hydraulic gradient were obtained for both part-full and pipe-full flow conditions. Tests with the pipe flowing half-full were carried out over a range of flow velocities from 0.7 to 1.3m/s. One test was carried out with y/D=0.75 at 1m/s and, for pipe-full flow, tests covered the range 0.5 to 1.2m/s. All these results are presented in Table 2. In some cases it was decided at the time the observations were made that conditions were either slightly above or slightly below the limit of deposition as defined in section 3.1, and these results are labelled $>$LD OR$<$LD as appropriate. As previously mentioned, once the limit of deposition was reached, two readings of hydraulic gradient were taken in order to determine any head-loss resulting from the presence of the sediment. In the first test to be carried out (Test A1), a number of readings were also taken at concentrations below the limit of deposition, in the hope that the increase in head-loss with increasing concentration would be observable. The results from this first test made it clear that the increases would be so small relative to the scatter in readings that this would be unrealistic, so in subsequent tests readings of hydraulic gradient were taken only at limit of deposition.

For the majority of part-full tests, all the gauge readings were included when calculating the mean energy gradient, even if there was considerable disagreement between individual gauges. This is because otherwise, if one is to be consistent and exclude the gauge reading entirely from the analysis, the calculated mean velocity and hydraulic radius are also affected. Generally it was found that including all the gauge readings gave roughness values which were more consistent with the overall mean, than by selectively excluding gauge readings which showed higher or lower levels than the others. The only exception to this was for the tests at 1m/s for y/D=0.5, where the level at gauge 3 was over 15mm lower than for the other four readings. This could not be explained as merely a random fluctuation in level, but seemed to indicate that there was a short stretch of supercritical flow with subcritical flow either side of it - obviously far from uniform flow conditions. It was decided therefore that this gauge reading was wholly unrepresentative at this velocity, and the analysis was based on the other gauge readings where flow was more uniform.

As was the case with clear-water roughness measurements, readings of hydraulic gradient taken with the gauge tip continuously submerged were excluded from the analysis. This means that the increases in resistance included in Table 1 are from one pair of readings for part-full tests, and are the average of two pairs of readings for pipe full tests. A number of these results show a decrease in resistance at the limit of deposition compared with that for clear water. Such an effect is improbable, and is almost certainly due to the high variability in measurements due to fluctuating water levels, non-uniform flow, and non-uniformity of pipe section.

5 ANALYSIS AND DISCUSSION

5.1 Analysis of clear-water results

The Colebrook-White formula was used with the calculated values of friction factor to determine a value of k_s for the pipe in sediment-free conditions. For part-full tests the rearranged form of the formula was as in equation 4.

$$k_s = 14.8R\ (10^{-1/2\sqrt{\lambda}} - 2.51/Re\sqrt{\lambda}) \quad (4)$$

This is the form used in the HR tables for hydraulic design of pipes (6).

The substitution in the formula of 4R=D for open channel flow is widely used to design pipes for part-full flow conditions. Hydraulic radius is straightforward to calculate, and by using 4R there is no inconsistency between part-full and pipe-full conditions. However, this assumes that the shear stress is uniformly distributed around the section, as it is for the pipe flowing full, and that the altered velocity distribution in the part-full pipe has no effect on resistance. The results obtained here seem to show significant variations between the determined values of k_s for different flow depths. There have been a number of past studies of the effect of shape factor on hydraulic resistance. Engelund (4) proposed the use of a "resistance radius" in place of hydraulic radius, and developed a theory based on certain assumptions about the distributions of velocity and shear stress. The method involves rather lengthy calculations, and Engelund made a number of simplifying assumptions applicable to wide channels in the fully rough region. These assumptions would not hold for the part-circular section considered here. Kazemipour (7) carried out numerous experimental studies on channels of various cross sections, and also incorporated data from other researchers in deriving an essentially empirical correction to be applied to the friction factor for open channels which would allow the use of standard pipe resistance formulae. One such study concentrated on semi-circular channels (8), and following on from this Nalluri and Adepoju (12) used this data, along with data from May (11) and a large quantity of further data of their own to develop a formula which was applicable to flow depths greater than 0.5D. The drawback to both these studies on pipe channels is that they were empirically derived from smooth pipe data. The Kazemipour formula shifts values of λ to fit the Karman-Prandtl equation

$$1/\sqrt{\lambda} = 2\log Re\sqrt{\lambda} - 0.8 \qquad (8)$$

and Nalluri and Adepoju compared their data with the Blasius equation

$$\lambda = 0.316/Re^{0.25} \qquad (9)$$

Kazemipour also carried out work on non-circular channels (7), and verified this with a limited amount of data in the transition region, so that his correction shifts the data onto the Colebrook-White formula for open channels (substituting 4R=D). The correlation he found was not very high, however, so it is by no means certain that his method is applicable in the transition region. The Nalluri and Adepoju method suggests an equation of the same form as the Blasius equation, but incorporating a shape factor y/P, and using a modified Reynolds number, Rey = Vy/ν.

Comparing the k_s values determined for each flow depth and shown in Table 3., the overall trend is of increasing resistance with depth up to almost pipe-full, then falling again for pipe-full flow. A surprising result is that resistance appears to be minimum for ¼ full, but it should be stressed that the data at flow depths other than pipe-full and half-full are limited. By adopting the mean value of k_s = .121mm at pipe-full, and calculating friction factor at each of the other flow depths using Colebrook-White with 4R=D, these values can be compared with the measured values to obtain the required "correction factor". These factors, which are applied to λ, appear in Table 5, alongside the calculated Kazemipour factors for semi-circular pipes. Kazemipour publishes a curve which allows the correction factor to be calculated for flow depths greater than half-full, but does not suggest that the method is applicable in this range. The calculated factors for $\bar{y}/D$ = 0.75 and 0.95 are included only for interest. Also included are the factors derived from his earlier work on non-circular channels, and perhaps surprisingly these factors seem to match the measured data for flow depths above half-full reasonably well. At flow depths of half-full and below the picture is more unclear, with the semi-circular method predicting corrections much greater than those observed, and very different from those generated from the non-circular (rectangular) method.

Although Nalluri and Adepoju's formula is intended for direct use only in smooth pipes, a correction factor can be calculated by comparing their formula with the Blasius equation. These correction factors are given in the table and suggest a much greater shift than Kazemipour's method.

In Table 3 are the recalculated values of k_s after correcting λ using Kazemipour's method for non-circular channels, which of all the schemes seems to fit the data best. The results are acceptably uniform for all but the half-full results, giving an overall mean of k_s=.136mm, but suggesting that the value of k_s=.121mm for the pipe-full data may be more appropriate. It appears that the variations in resistance are smaller than could be expected, and the overall mean value of k_s=.134mm seems to be a good estimate of the roughness of the pipe being studied.

It would seem that none of the available methods for incorporating shape effects in the calculation of hydraulic resistance match the data obtained here from a circular pipe flowing part-full in the transition region. The results do show that a simple substitution of 4R=D is not necessarily most appropriate, but that variations are generally smaller than those found in studies of smooth pipes.

More data are necessary for part-full conditions before conclusions can be drawn regarding the effect of shape factor on the roughness, but a mean value of k_s has been measured which is consistent with the design value of k_s=.15mm given in HR Tables (6) for spun-concrete pipes.

5.2 Limit of deposition

For each of the tests a mean concentration was determined from the output from the infra-red sensor. A corresponding mean flow depth which was nominally $\bar{y}/D$=1, 0.75, or 0.5 was calculated accurately from the five gauge readings. From this a mean velocity could be determined, and these values are plotted in fig. 10. A number of prediction formulae are added to the figure for comparison in the case of pipe-full flow.

The formulae consider:-

Laursen (9)

$$V/(2g(s-1)y)^{\frac{1}{2}} = 7\ Cv^{1/3} \qquad (10)$$

where Cv = volumetric concentration of sediment

Equation 10 is a best-fit carried out in SI units by May (11) to Laursen's graphical representation.

Macke (10)

$$Qs* = 1.64x10^{-4}\ \tau o^{3} \qquad (11)$$

which is equivalent to

$$V = 1.98\ \lambda^{-0.6}\ w^{0.3}\ \left[(s-1).A.Cv\right]^{0.2} \quad (12)$$

Where w is the fall velocity in m/s, V is in m/s and A in m^2.

May (11)

$$Cv = \frac{2.05}{100}\ (D^2/A).(d/R)^{0.6}(V^2/g(s-1)D)^{1.5}(1-Vt/V)^4 \quad (13)$$

where Vt is the threshold velocity.

Nalluri and Mayerle (13)

$$V/(g(s-1)d)^{\frac{1}{2}}=0.86\ Cv^{-0.18}\lambda^{-1.06}(R/d)^{-0.2}Dgr^{-0.4} \quad (14)$$

Where Dgr is the dimensionless grain size proposed by Ackers and White (1).

$$Dgr = (g(s-1)/\ \nu^2)^{1/3}\ .\ d \quad (15)$$

Formulae for the calculation of kinematic viscosity and fall velocity are given in Appendix 1.

The results at pipe-full show a consistent trend of increasing concentration with increasing velocity, the actual concentrations being somewhat lower than those predicted by all the formulae under consideration. Over the range of values studied it can be seen that the slope of the Laursen equation does not seem to match the data well, and the values it predicts are up 20 times higher than those measured. The formula by Nalluri and Mayerle was derived by regression analysis of recent data from a study of a 152mm perspex pipe with sediment sizes in the range $0.5 \leqslant d \leqslant 8.74$mm. It predicts higher concentrations than those measured, but follows the overall trend of the new data.

Macke derived his formula from a large quantity of data both from his own studies and those of other researchers. It can be seen that this curve lies close to that of May, both of these following the data closely, but consistently overpredicting the measured concentrations. The May formula includes a transition parameter incorporating the threshold velocity, which allows transport to cease at a velocity greater than zero. May's recommendation was that the threshold velocity be calculated using Novak and Nalluri's formula for incipient motion on a smooth bed. If this procedure is followed, the higher of the two curves shown on the plot is generated. The approximate measurements of threshold velocity for the concrete pipe studied here suggested that the smooth pipe prediction of Novak and Nalluri was too low. By putting the measured threshold velocity into May's formula the lower curve is produced (labelled May′), which demonstrates an improved fit to the measured values.

When considering the results at half-full, it can be seen that they are less consistent, measured concentrations being scattered over a much wider range for a given velocity than for the pipe-full results. Some of these show higher concentrations than for pipe-full, others lower, with no one set predominant. The formulae of Macke and of May both predict that the transport rate at half full will be the same as for the same velocity at pipe-full. i.e. the concentration will be double. The Nalluri and Mayerle formula, which does not include flow area as a parameter in the regression, predicts that the same concentration of sediment can be carried at half full (i.e. transport rate is halved). This would seem to be more in line with measurements so far obtained for the concrete pipe, if the "mean" effect is considered. It is, however, difficult to draw conclusions based on such widely varying results.

A statistical summary of the goodness of fit of the various formulae is given in Table 6. It can be seen that none of the formulae fits both the pipe-full and part-full data well. The May formula, using the measured threshold velocity, fits the pipe-full data best, with a mean Cv-measured/Cv-predicted of .892 and standard deviation of 17.6%. The over-prediction of half-full concentrations is much worse, with a mean of .515 and σ_s = 39.4%. The best fit for the half-full data is from Nalluri and Mayerle, with a mean of .653. The scatter is considerable, but inherent in the data rather than highlighting an inadequacy of the formula.

A visual comparison of the formulae can be achieved by plotting the data in a different format, of a transport parameters against a velocity parameter, or shear stress parameter in the case of Macke. These are shown in figures 11, 12, 13. The actual scatter as observed on these plots is somewhat deceptive as the various transport parameters contain Cv raised to different powers, but the figures serve to show how well each prediction follows the trend in the new data.

In the earlier HRS study, and data collected then were compared with those prediction formulae which were available. It was found that no existing formulae was close to the new data set. The formulae of Macke and of Nalluri and Mayerle have been published since that study, so a comparison of these with the original HRS data is included here. A statistical summary is given in Table 7., and the data is presented graphically in figures 14, 15, 16. The exact data set used here differs slightly from that used by May to derive his formula, in that the Macke and Nalluri and Mayerle formulae include friction factor λ in the prediction, and this was not always measured by May. May grouped some of the HRS data to simplify the analysis and this grouping is also removed from the present comparison.

The most obvious finding is that Macke's formula is a long way from the results obtained for gravel sized particles. This is not surprising, as Macke's theory is based on particles in suspension, and the gravel was certainly being transported as bed-load. This is also true however of much of the sand data, which the Macke formula fits very well. The range of sediment sizes included in Macke's study was $0.1 \leqslant d \leqslant 3$mm, and sediment size appears in the formula only in terms of fall velocity. In the present comparison this has been estimated from an empirical formula, (see Appendix 1) and is therefore a potential source of error, but it seems unlikely to account for such a large discrepancy. One possibility is that the gravel was being transported in such a way that particles never left the invert, but simply slid or rolled along. This would certainly be a mode of transport which Macke's formula would not be applicable to, but this information was not recorded when the HRS experiments were carried out, and so cannot be confirmed.

If the gravel data are excluded, then the Macke formula clearly performs well, particularly for the 152mm pipe data, where the mean and standard deviation are comparable with those from May's own formula. Some of the data falls in Macke's "Region II" ($Q_s^* \leqslant 2\text{x}10^{-4}$), being lower transport rates where his formula does not apply. By removing these few points the fit is almost unaffected.

Nalluri and Mayerle's formula does not give such a good fit, although following the trends in the data quite well. As previously mentioned, it predicts the same concentration of sediment at half-full flow as at pipe-full. The HRS data did not follow that pattern, so the formula under-predicts markedly at half-full. The fit with gravel data is good, and if results at pipe-full flow are compared, the effect of pipe size is also accounted for satisfactorily.

Overall, the broad agreement between these formulae is encouraging. The new results for the concrete pipe are showing lower concentrations at the limit of deposition than expected. This could be accounted for by the uncertainty of definition of the limit and the method of observation used here. However, these are not thought to vary significantly from the criteria used in the HRS study, although the measurement technique is different. If the changes are due to the larger pipe diameter and the higher roughness of the pipe then clearly attention must be turned to those aspects of the formulae which account for these effects - typically (R/d50), and threshold velocity or friction factor.

5.3 Head-loss at limit of deposition

The changes in friction factor from clear-water to limit of deposition are shown in fig. 17. There are 12 cases in which measured value of λ increased with the addition of sediment, and 6 instances when it apparently fell. Overall, there is a mean increase in λ of 4.27E-05 or 0.61%. If the negative results are assumed to be aberrations, and assigned zero values the increase is 4.63E-04 or 2.55%. Quite clearly, the degree of scatter in the readings is large, meaning that it is not possible to deduce statistically significant results from such a small data set. Nonetheless, by neglecting the negative values and presenting the others as proportionate increases in resistance divided by concentration, a comparison can be made with the well known equation derived by Durand and Condolios (3) from studies of sediment in flume traction.

$$\Theta/Cv = 648.\ (g(s-1)R/V^2.Cd)^{1.5} \qquad (16)$$

where $\theta = (\lambda - \lambda o)/\lambda o$

This comparison is shown in fig. 18, assuming Cd=1, and it can be seen that the correlation is very poor. There is no apparent trend of increasing resistance with increasing concentration in the results collected so far, the systematic variation in Θ/Cv being solely due to the dependence of Cv on V at the limit of deposition, hence why the data follow a much steeper line in than that of the formula. This finding of smaller increases in head-loss with the concrete pipe than with, for instance, the smooth pipes used in the HRS study is to be expected. The effect of adding sediment in small concentrations is to add a drag force (exerted by the particles on the flow), causing an additional hydraulic resistance to that exerted by the pipe on the clear-water flow. If the resistance of the pipe is already significant, the effect of adding the sediment will clearly be less than if the pipe were hydraulically smooth.

6 CONCLUSIONS

After a lengthy period of equipment development, particularly for the sediment sensor, the measurement apparatus is working well. It is now possible to gather reliable data rapidly over a wide range of flow conditions.

The measurements of hydraulic resistance taken with clear water have been consistent over a wide range of velocities, and have confirmed a k_s value for the 300mm dia. spun-concrete pipe which is within the range recommended for pipes in clean condition. There appears to be a small effect upon resistance due to variations in cross-sectional shape under part-full conditions, apparently showing resistance to be lowest at quarter-full flow. When these variations are compared with results of previous researchers for smooth pipes they appear to be less than expected, and the variations do not follow the same pattern with changing depth of flow.

The approximate measurements of threshold velocity suggest that the velocity required for incipient motion in the concrete pipe is a little higher than that for a smooth pipe as studied by Novak and Nalluri. However, the roughness of the concrete pipe is too small to allow their correction for rough pipes to be applied. Further studies of incipient motion on such surfaces of intermediate roughness would be of value.

The increase in resistance with the addition of sediment is too small to be measured satisfactorily with the present arrangement, and the scatter in measured values is high. A number of the tests have shown decreases in resistance at limit of deposition compared with clear-water, but this is almost certainly due to measurement errors.

The conclusion to be drawn is that changes in resistance due to the presence of sediment (prior to deposition) are probably not significant compared with other effects due to joints and non-uniformity of pipe section.

The concentration measurements at the limit of deposition in the 300mm concrete pipe are consistently lower than those predicted by any of the recently derived formulae. The results obtained in pipe-full flow show a consistent increase with increasing velocity, and the rate of increase follows the pattern predicted by those formulae. The reason for the lower concentrations is not known certainly, but is likely to be due to one of three reasons. Firstly, the criterion for assessing the limit could be different; alternatively, the pipe diameter, being larger than almost all those studied previously could be causing the discrepancy, and this is the scale-effect particularly highlighted by CIRIA as requiring attention. Finally, the different surface texture of the pipe could be the cause, although May's formula takes some account of this by using threshold velocity, and the other formulae under consideration incorporate friction factor in their analyses.

The part-full flow results show a disappointingly large degree of scatter, the cause of which is still unknown. This scatter makes it difficult to deduce whether the rate of sediment transport at half-full flow is equal to that at pipe-full for the same velocity, or if it is less. It is hoped that further tests will resolve this uncertainty.

The analysis of the old HRS data in comparison with the formulae of Macke and of Nalluri and Mayerle is encouraging. The very close correlation of Macke's formula with the HRS sand data provides a useful independent verification of Macke's formula, and suggests that there is now a better understanding of the interaction of the various parameters governing sediment transport in pipes. The formula by Nalluri and Mayerle gives a good overall fit with all the HRS data, including gravel. As far as the new HR data is concerned, there is little to choose between the three formulae considered, all giving $\sigma_s \approx 35\%$ and slightly overestimating concentration at the limit of deposition. The contribution of results from the pipe being studied here, and from the 450mm pipe when they become available should be substantial.

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TABLES.

TABLE 1 : Experimental results

Test No	Date	$\bar{y}/D$	$\bar{V}$ (m/s)	F_r	T (°C)	Cv (ppm)	λ, λ_o	Θ $(\frac{\lambda - \lambda_o}{\lambda_o})$	Θ/Cv
1	29/05/87	.935	1.286	.604	13.6	0	.019349		
2	29/05/87	.944	.925	.418	14.4	0	.017311		
3	29/05/87	.949	.545	.241	14.8	0	.021857		
6	04/06/87	.504	.520	.482	13.5	0	.020572		
7	04/06/87	.506	.520	.481	14.2	0	.021256		
8	04/06/87	.507	.730	.674	15.7	0	.017583		
9	04/06/87	.496	1.020	.956	15.9	0	.017777		
10	04/06/87	.506	1.382	1.278	16.4	0	.018998		
11	04/06/87	.508	1.950	1.799	16.9	0	.020079		
12	05/06/87	.248	1.735	2.417	14.2	0	.020855		
13	05/06/87	.249	1.370	1.905	14.5	0	.018108		
14	05/06/87	.249	.757	1.052	14.8	0	.021481		
15	05/06/87	.252	.757	1.046	14.8	0	.020955		
16	05/06/87	.251	.493	.682	14.7	0	.020322		
17	08/06/87	.754	.458	.316	13.7	0	.024491		
18	08/06/87	.753	.461	.314	14.2	0	.028341		
19	08/06/87	.752	.731	.499	14.6	0	.018691		
20	08/06/87	.759	.981	.663	15.0	0	.016751		
21	08/06/87	.728	1.396	.983	15.5	0	.015685		
22	09/06/87	.747	2.090	1.435	14.9	0	.015148		
23	23/06/87	1.0	.494		14.3	0	.022491		
23B	24/06/87	1.0	.494		(15)	0	.026460		
24B	07/07/87	1.0	1.524		(15)	0	.015593		
25B	07/07/87	1.0	1.023		13.4	0	.017651		
26B	07/07/87	1.0	.63		(15)	0	.018816		
27A	08/07/87	1.0	.470		(15)	0	.018025		
27B	08/07/87	1.0	.469		14.5	0	.015601		
28A	08/07/87	1.0	1.638		15.5	0	.015776		
28B	08/08/87	1.0	1.643		16.5	0	.014968		
R27A	04/08/87	1.0	.470		13.0	0	.018855		
R27B	04/08/87	1.0	.470		13.0	0	.019645		
R28A	04/08/87	1.0	1.069		13.1	0	.015456		
R28B	04/08/87	1.0	1.608		13.1	0	.015753		
R26A	05/08/87	1.0	.643		13.6	0	.019906		
R26B	05/08/87	1.0	.639		13.6	0	.018574		
R25A	05/08/87	1.0	1.021		13.6	0	.017554		
R25B	05/08/87	1.0	1.022		13.4	0	.017424		
N3	28/10/87	1.0	.180		11.4	0	.032688		
N4	30/10/87	.489	.779	.736	11.7	0	.019115		
N5	04/11/87	.253	.494	.681	11.4	0	.021387		
SED1	17/11/87	1.0	.460		10.7	0	.021000		
A1CLR	18/01/88	1.0	.901		10.0	0	.019624		
A1CLR1	18/01/88	1.0	.901		10.0	0	.018078		
A1SED	19/01/88	1.0	.901		12.5	6.68	.018027	-.020591	-3082.5
A1SED1	19/01/88	1.0	.892		12.5	6.68	.018464	.003151	471.7

\+ Gauge 3 omitted (See 3.1)
* Gauges 'fully immersed' (See 4.2)
() Estimated temperature

TABLE 1 : Cont'd

Test No	Date	$\bar{y}/D$	$\bar{V}$ (m/s)	F_r	T (°C)	Cv (ppm)	λ, λ_o	θ $\left(\frac{\lambda - \lambda_o}{\lambda_o}\right)$	θ/Cv
A1SED2	19/01/88	1.0	.898		12.5	11.89	.018253	-.008313	-699.2
A1SED3	19/01/88	1.0	.898		12.5	11.89	.018977	.031022	2609.1
A1SED4	19/01/88	1.0	.898		13.0	26.01	.018036	-.020102	-773.9
A1SED5	19/01/88	1.0	.898		13.0	26.01	.018404	-.000109	-4.2
A1SED6	19/01/88	1.0	.899		13.5	26.71	.018694	.015647	585.8
A1SED7	19/01/88	1.0	.899		13.5	26.71	.018766	.019559	732.3
A1SED8	20/01/88	1.0	.893		13.5	26.34	.017971	.023634	897.3
A1SED9	20/01/88	1.0	.893		13.5	26.34	.018320	-.004672	-177.4
A1SED1	20/01/88	1.0	.893		13.5	29.86	.018747	.018527	620.5
A1SED1	20/01/88	1.0	.893		13.5	29.86	.018262	-.007824	-262.0
A1CLR2	20/01/88	1.0	.898		10.5	0	.018562		
A1CLR3	22/01/88	1.0	.898		10.5	0	.017795		
A1CLR4	22/01/88	1.0	.898		11.3	0	.017945		
A1CLR5	22/01/88	1.0	.898		11.3	0	.018429		
A2CLR	27/01/88	1.0	1.003		10.2	0	.018063		
A2CLR1	27/01/88	1.0	1.003		10.2	0	.018367		
A2SED	27/01/88	1.0	1.006		10.4	45.50	.017948	-.014658	-322.2
A2SED1	27/01/88	1.0	1.006		10.4	45.50	.018398	.010047	220.8
A3CLR	29/01/88	1.0	.799		10.3	0	.018685		
A3CLR1	29/01/88	1.0	.799		10.3	0	.018539		
A3SED	29/01/88	1.0	.800		10.3	14.50	.019260	.034816	2401.1
A3SED1	29/01/88	1.0	.800		10.3	14.50	.019421	.043467	2997.7
A4CLR	29/01/88	1.0	.698		10.0	0	.018195		
A4CLR1	29/01/88	1.0	.698		10.0	0	.019901		
A4SED	29/01/88	1.0	.698		10.0	7.56	.019368	.016800	2222.2
A4SED1	29/01/88	1.0	.698		10.0	7.56	.020064	.053339	7055.4
A5CLR	02/02/88	1.0	.499		9.8	0	.019901		
A5CLR1	02/02/88	1.0	.499		9.8	0	.019461		
A5SED	02/02/88	1.0	.500		9.8	0.67	.019437	-.012398	-18504.5
A5SED1	02/02/88	1.0	.500		9.8	0.67	.020148	.023728	35414.9
A6CLR	02/02/88	1.0	.603		9.9	0	.019433		
A6CLR1	02/02/88	1.0	.603		9.9	0	.019832		
A6SED	02/02/88	1.0	.603		9.9	4.07	.018785	-.043142	-10600.0
A6SED1	02/02/88	1.0	.603		9.9	4.07	.019022	-.031070	- 7633.9
A9SED	08/02/88	1.0	1.196		9.8	69.75	.017462	-	-
A9SED1	08/02/88	1.0	1.196		9.8	69.75	.017577	-	-
A10CLR	09/02/88	1.0	1.101		10.2	0	.017911		
A10CLR1	09/02/88	1.0	1.101		10.2	0	.017853		
A10SED	09/02/88	1.0	1.099		10.2	55.29	.017811	-.003970 -	71.8
A10SED1	09/02/88	1.0	1.099		10.2	55.29	.017882	.000000	0
A11CLR	09/02/88	1.0	.549		9.7	0	.019763		
A11CLR1	09/02/88	1.0	.549		9.7	0	.020058		
A11SED	09/02/88	1.0	.549		9.6	2.13	.020786	.043946	20631.9
A11SED1	09/02/88	1.0	.549		9.6	2.13	.020910	.050173	23555.4
A12CLR	25/02/88	.501	.997	.928	10.4	0	+.018636		

\+ Gauge 3 omitted (See 3.1)
* Gauges 'fully immersed' (See 4.2)
() Estimated temperature

TABLE 1 : Cont'd

Test No	Date	$\bar{y}/D$	$\bar{V}$ (m/s)	F_r	T (°C)	Cv (ppm)	λ, λ_0	θ $\left(\frac{\lambda - \lambda_0}{\lambda_0}\right)$	θ/Cv
A12SED	25/02/88	.511	.972	.893	9.5	70.10	+.018968	.017815	254.1
A13CLR	15/03/88	.504	.894	.828	11.0	0	.018412		
A13CLR1	15/03/88	.497	.910	.851	11.0	0	.017477*		
A13SED	15/03/88	.504	.896	.831	11.0	22.18	.018865	.024604	1109.3
A13SED1	15/03/88	.497	.909	.850	11.0	22.18	.017696*	.012531	565.0
A14CLR	15/03/88	.251	.994	1.377	10.8	0	.018960		
A14CLR1	15/03/88	.246	1.022	1.430	10.8	0	.017714*		
A15CLR	16/03/88	.741	1.019	.705	10.2	0	.020274		
A15CLR1	16/03/88	.734	1.028	.718	10.2	0	.018977*		
A15SED	16/03/88	.734	1.016	.709	10.2	42.54	.017849	-.119611	-2811.7
A15SED1	16/03/88	.730	1.023	.718	10.2	42.54	.017121*	-.097803	-2299.1
A16CLR	17/03/88	.493	1.019	.959	10.2	0	+.017444		
A16CLR1	17/03/88	.487	1.035	.982	10.2	0	+.016800*		
A16SED	17/03/88	.490	1.021	.963	10.5	32.67	+.017691	.014160	433.4
A16SED1	17/03/88	.481	1.046	.999	10.5	32.67	+.016749*	.003036	-92.9
A17CLR	18/03/88	.498	.704	.658	10.9	0	.016673		
A17CLR1	18/03/88	.490	.718	.678	10.9	0	.016369*		
A17SED	18/03/88	.499	.702	.655	10.9	6.68	.018035	.081035	12228.9
A17SED1	18/03/88	.485	.728	.693	10.9	6.68	.014388*	.121021	-18116.9
A18CLR	21/03/88	.492	.814	.766	10.3	0	.018654		
A18CLR1	21/03/88	.488	.821	.777	10.3	0	.018319*		
A18SED	21/03/88	.494	.812	.763	10.3	27.52	.018157	-.026646	-968.2
A18SED1	21/03/88	.489	.823	.778	10.3	27.52	.017075*	-.067908	-2467.6
A19SED	22/03/88	.513	.870	.798	10.0	30.47	.020275		
A19SED1	22/03/88	.507	.883	.816	10.0	30.47	.019023*		
A20CLR	23/03/88	.491	1.131	1.067	11.0	0	.019548		
A20CLR1	23/03/88	.485	1.148	1.091	11.0	0	.015479*		
A20SED	23/03/88	.500	1.104	1.029	11.0	46.06	.016551	-.153315	-2975.8
A20SED1	23/03/88	.498	1.108	1.035	11.0	51.52	.016380	-.162063	-3145.6
A21CLR	24/03/88	.502	1.195	1.111	10.9	0	.015454		
A21CLR1	24/03/88	.496	1.213	1.137	10.9	0	.014954*		
A21SED	24/03/88	.502	1.191	1.107	10.9	55.73	.015834	.024589	441.2
A21SED1	24/03/88	.497	1.208	1.131	10.9	55.73	.015019*	.004347	78.0
A22CLR	25/03/88	.508	1.270	1.171	11.0	0	.018176		
A22CLR1	25/03/88	.502	1.290	1.199	11.0	0	.017517*		
A22SED	25/03/88	.519	1.237	1.126	11.4	135.54	.019327	.063325	467.2
A22SED1	25/03/88	.514	1.251	1.145	11.4	142.21	.020263	.114822	847.1

\+ Gauge 3 omitted (See 3.1)

* Gauges 'fully immersed' (See 4.2)

() Estimated temperature

TABLE 2 : Results at limit of deposition

D = 298.8mm

d50 = 0.72mm

Test	Date	y/D	V	Cv	T	λ	
A1SED10	20/01/88	1.0	.893	29.86	13.5	.018747	
A1SED11	20/01/88	1.0	.893	29.86	13.5	.018262	
A2SED	27/01/88	1.0	1.006	45.50	10.4	.017948	
A2SED1	27/01/88	1.0	1.006	45.50	10.4	.018398	
A3SED	29/01/88	1.0	.800	14.50	10.3	.019260	
A3SED1	29/01/88	1.0	.800	14.50	10.3	.019421	
A4SED	29/01/88	1.0	.698	7.56	10.0	.019368	
A4SED1	29/01/88	1.0	.698	7.56	10.0	.020064	
A5SED	02/02/88	1.0	.500	.67	9.8	.019437	
A5SED1	02/02/88	1.0	.500	.67	9.8	.020148	
A6SED	02/02/88	1.0	.603	4.07	9.9	.018785	
A6SED1	02/02/88	1.0	.603	4.07	9.9	.019022	
A9SED	08/02/88	1.0	1.196	69.75	9.8	.017462	
A9SED1	08/02/88	1.0	1.196	69.75	9.8	.017577	
A10SED	09/02/88	1.0	1.099	55.29	10.2	.017811	
A10SED1	09/02/88	1.0	1.099	55.29	10.2	.017822	
A11SED	09/02/88	1.0	.549	2.13	9.6	.020786	
A11SED1	09/02/88	1.0	.549	2.13	9.6	.020910	
A12SED	25/02/88	.511	.972	70.10	9.5	.018968	
A13SED	15/03/88	.504	.896	22.18	11.0	.018865	
A13SED1	15/03/88	.497	.909	22.18	11.0	.017696	*
A15SED	16/03/88	.734	1.016	42.54	10.2	.017849	
A15SED1	16/03/88	.730	1.023	42.54	10.2	.017121	*
A16SED	17/03/88	.490	1.021	32.67	10.5	.017691	
A16SED1	17/03/88	.481	1.046	32.67	10.5	.016749	*
A17SED	18/03/88	.499	.702	6.68	10.9	.018035	
A17SED1	18/03/88	.485	.728	6.68	10.9	.014388	*
A18SED	21/03/88	.494	.812	27.52	10.3	.018157	
A18SED1	21/03/88	.489	.823	27.52	10.3	.017075	*
A19SED	22/03/88	.513	.870	30.47	10.0	.020275	
A19SED1	22/03/88	.507	.883	30.47	10.0	.019023	*
A20SED	23/03/88	.500	1.104	46.06	11.0	.016551	<LD
A20SED1	23/03/88	.498	1.108	51.52	11.0	.016380	
A21SED	24/03/88	.502	1.191	55.73	10.9	.015834	
A21SED1	24/03/88	.497	1.208	55.73	10.9	.015019	*
A22SED	25/03/88	.519	1.237	135.54	11.4	.019327	
A22SED1	25/03/88	.514	1.251	142.21	11.4	.020263	>LD

* Gauges 'fully immersed' (See 4.2)

<LD Slightly below limit of deposition (see 4.4)

>LD Slightly above limit of deposition (see 4.4)

TABLE 3 : Measured and corrected ks values by Kazemipour's method for clear water

		Measured			Corrected by Kazemipour method		
$\bar{y}/D$	N	k_s (mm)	σ_s	σ_n	k_s	σ_s	σ_n
1	37	.121	.0668	.0111	.121	.0668	.0111
0.95	3	.258	.1144	.0809	.117	.0682	.0482
0.75	5	.152	.0962	.0481	.123	.0845	.0423
0.5	15	.151	.0772	.0206	.196	.0884	.0228
0.25	7	.105	.0463	.0189	.105	.0463	.0189
all	67	.134	.0792	.0189	.136	.0789	.00972

TABLE 4 : Measured and predicted threshold velocities

$\bar{y}/d$	Measured Vt (m/s)	Predicted Vts (1) (m/s)	Vt/Vts	Predicted "k" (2) (mm)
.268	.224	.201	1.114	.0013
.489	.256	.228	1.123	.0016

1. Novak and Nalluri (Equation 6) smooth pipes
2. Novak and Nalluri (Equation 7) rough pipes

TABLE 5 : Measured and predicted variations in friction factor

$\bar{y}/D$	λ/λ_p	Ψ_{kc}	Ψ_{kr}	Ψ_{na}
.95	1.104	.94	1.13	1.24
.75	1.013	1.03	1.03	1.27
. 5	1.023	1.14	.96	1.00
.25	.981	1.25	1.00	.56

Ψ_{kc} - Kazemipour correction factor - semi-circular channels

Ψ_{kr} - " " " " " " - rectangular channels

Ψ_{na} - Nalluri & Adepoju correction factor - circular sections

λ_p - Predicted λ from Colebrook-White

TABLE 6 : Measured and predicted concentrations at limit of deposition - new data

May

$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)
1	.729	.135	18.5%
0.5	.448	.177	39.5%
all	.594	.207	34.8%

May' (using measured threshold velocity)

$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)
1	.892	.157	17.6%
0.5	.515	.203	39.4%
all	.706	.255	36.1%

Macke

				$Q_s^* \geqslant 2x10^{-4}$		
$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)	C_v/C_{vp}	σ	σ (%)
1	.746	.199	26.7%	.810	.132	16.3%
0.5	.481	.161	33.5%	.490	.169	34.5%
all	.617	.219	35.5%	.630	.214	34.0%

Nalluri & Mayerle

$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)
1	.498	.131	26.3%
0.5	.653	.246	37.7%
all	.584	.209	35.8%

TABLE 7 : Measured and predicted concentrations at limit of deposition - HRS data

May

D	d_{50}	$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)
76	.57	1	.987	.110	11.1%
158	.64	1	1.054	.197	18.7%
158	.64	.75	.868	.180	20.7%
158	.64	.5	.804	.170	21.1%
158	5.8	1	1.027	.127	12.4%
158	7.9	1	.842	.181	21.5%
	all		.979	.197	20.1%

Macke

D	d_{50}	$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)
76	.57	1	.730	.130	17.8%
158	.64	1	1.079	.243	22.5%
158	.64	.75	.981	.150	15.3%
158	.64	.5	1.020	.212	26.7%
158	5.8	1	(30.38	3.61)	11.9%
158	7.9	1	(35.62	5.11)	14.3%
all (except gravel)			.992	.259	26.1%
all (except gravel) $Q_s^* > 2x10^{-4}$			1.023	.259	25.3%

Nalluri & Mayerle

D	d_{50}	$\bar{y}/D$	C_v/C_{vp}	σ	σ (%)
76	.57	1	1.424	.249	17.5%
158	.64	1	1.227	.416	33.9%
158	.64	.75	2.155	.319	14.8%
158	.64	.5	2.878	.874	30.4%
158	5.8	1	.859	.123	14.3%
158	7.9	1	.595	.113	19.0%
	all		1.398	.741	53.0%

FIGURES.

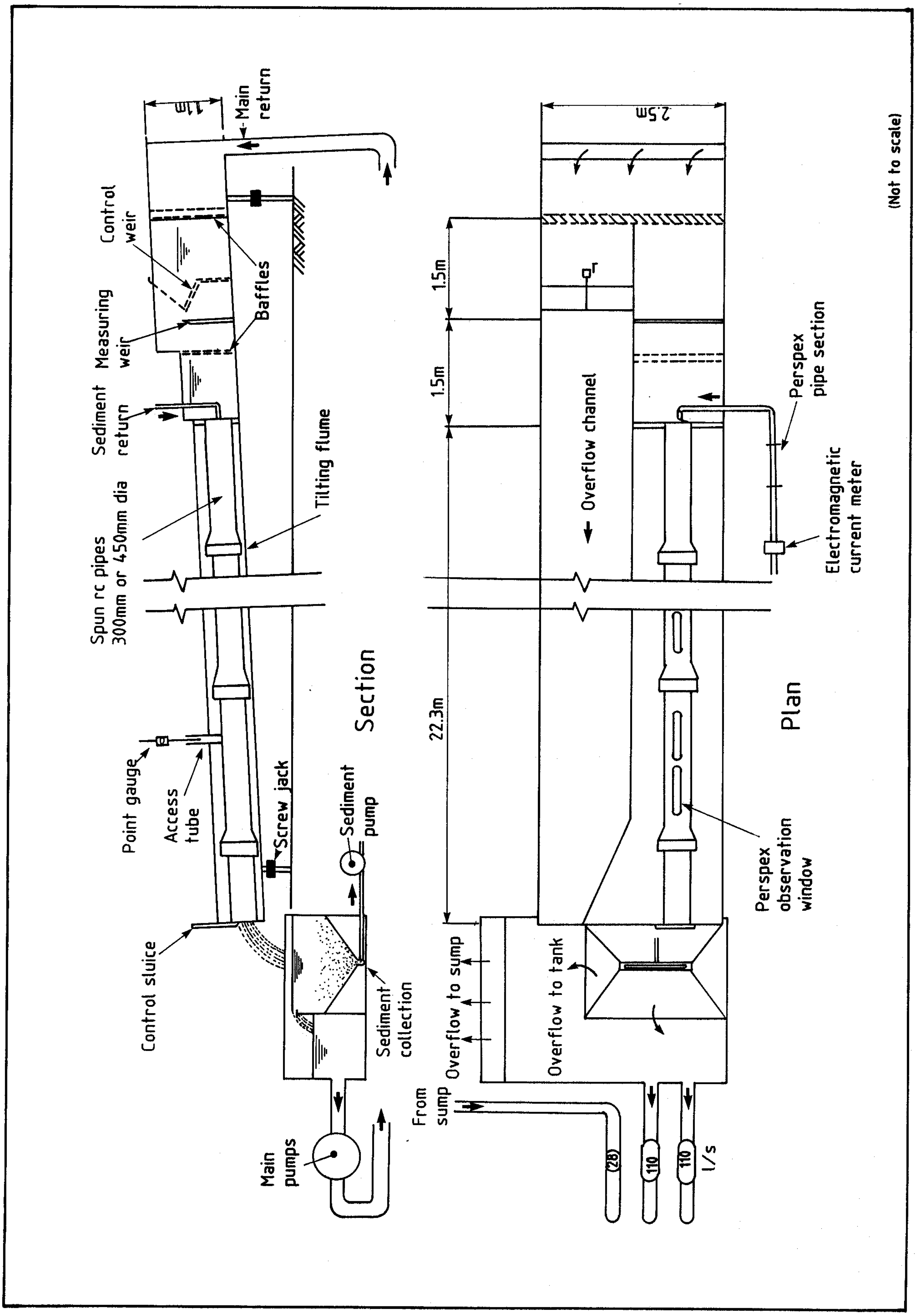

Fig 1 Layout of test rig

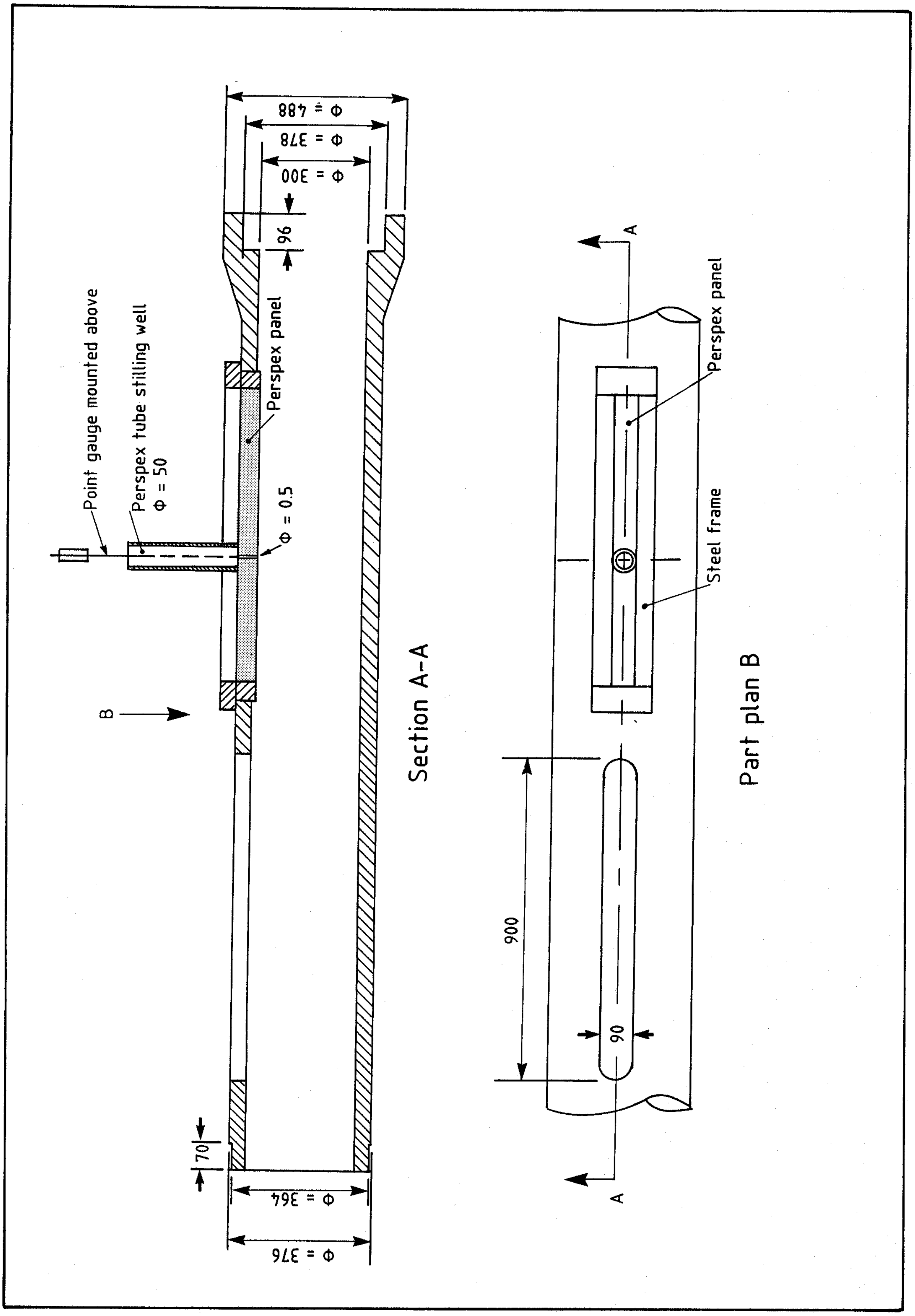

Fig 2 Detail of concrete pipe section showing slots and removable lids

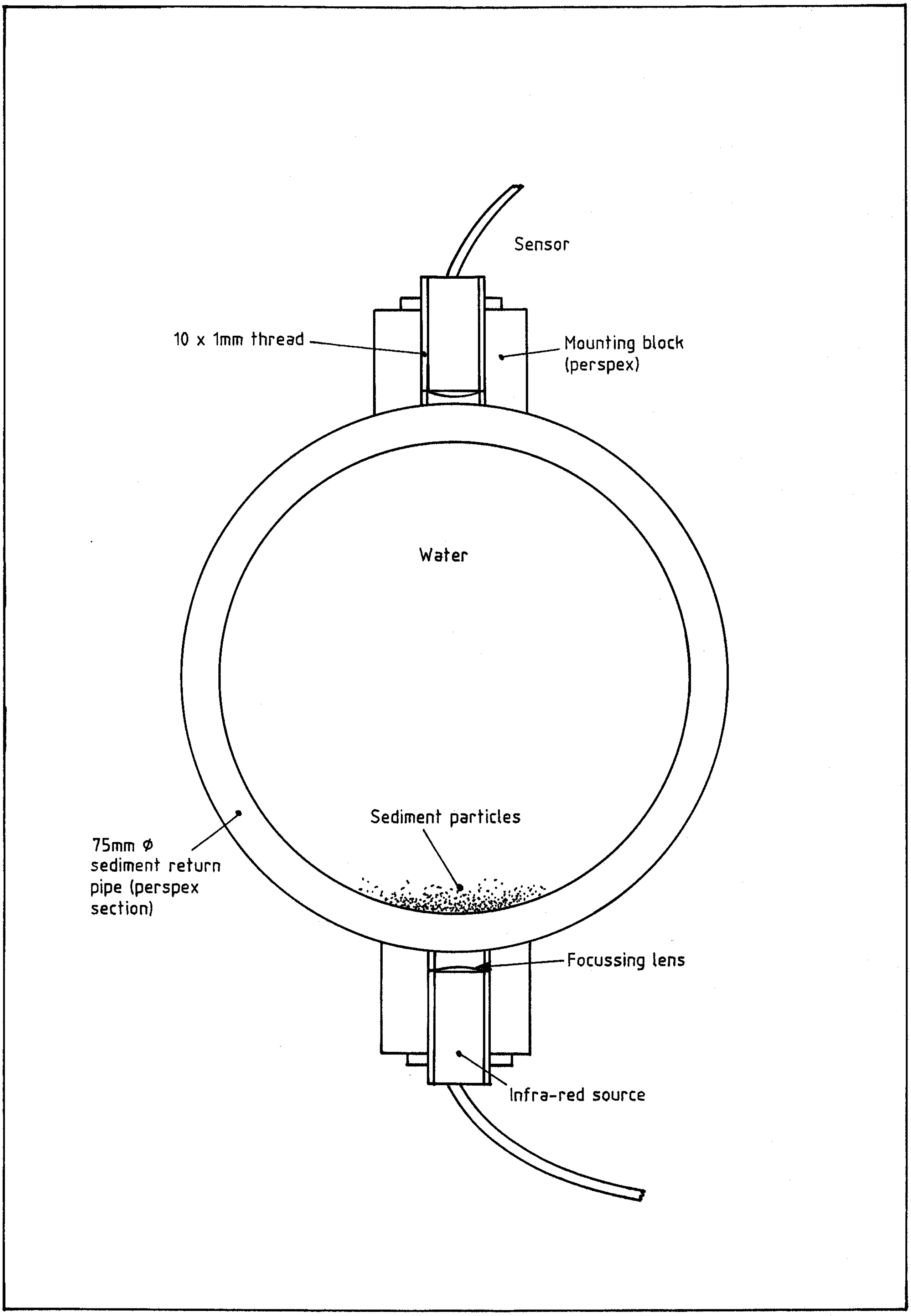

Fig 3 Cross-section through sediment sensor

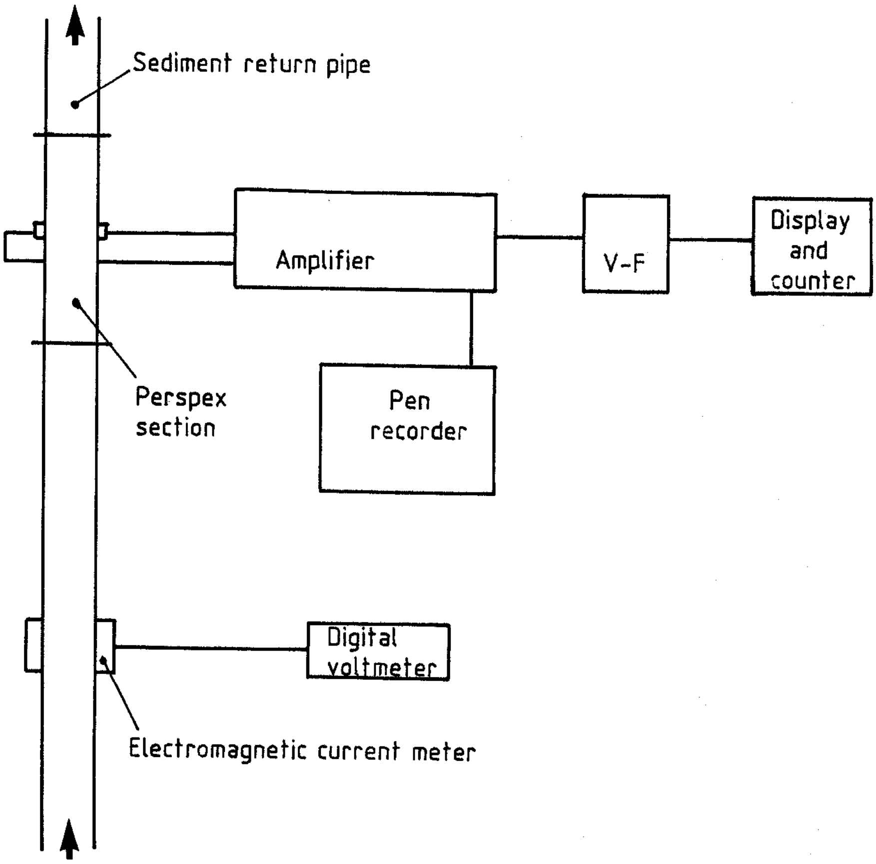

Fig 4   Schematic layout of sensor equipment

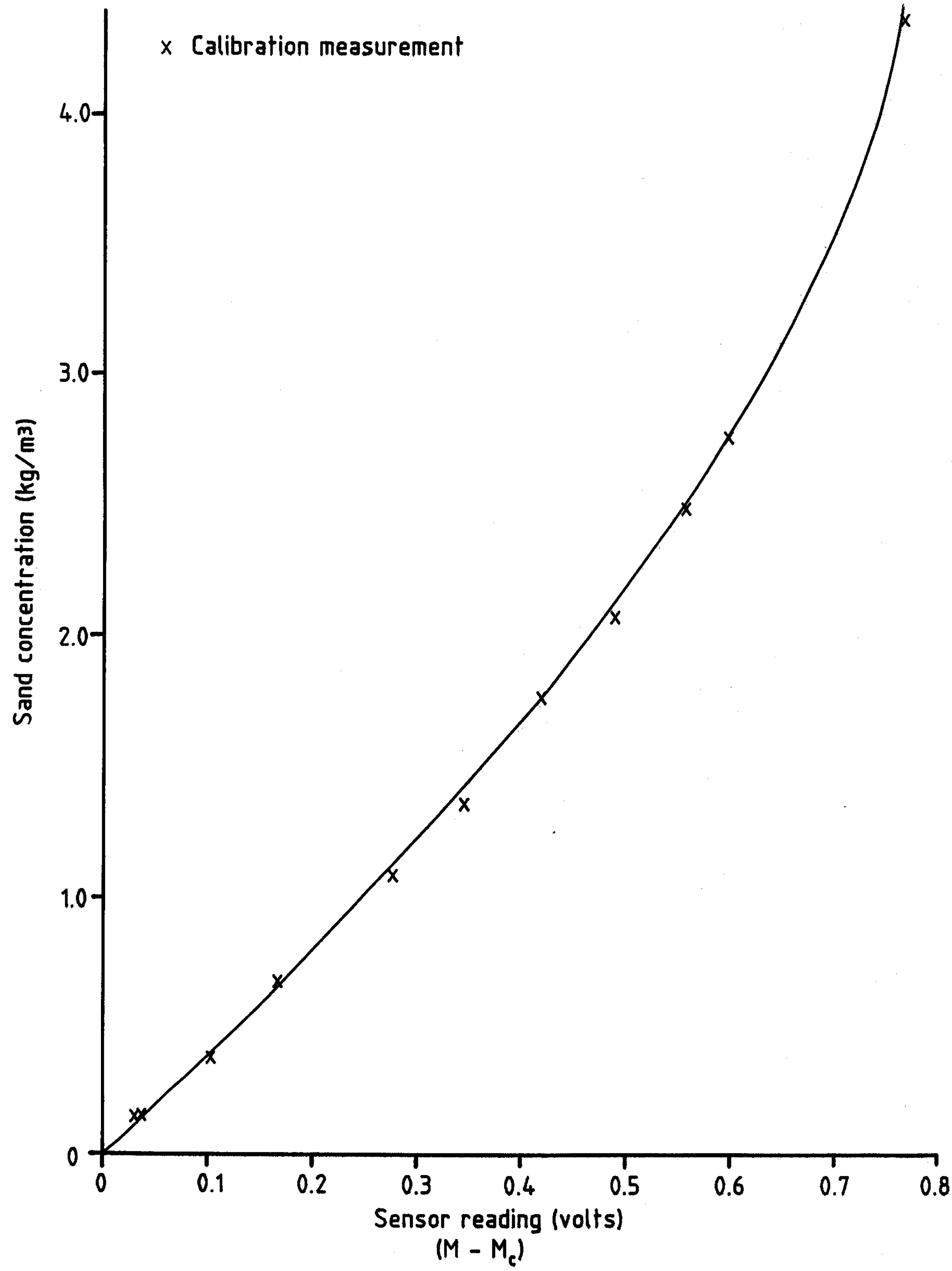

Fig 5 Sensor calibration at 1.39m/s Q = 0.00613 m^3/s

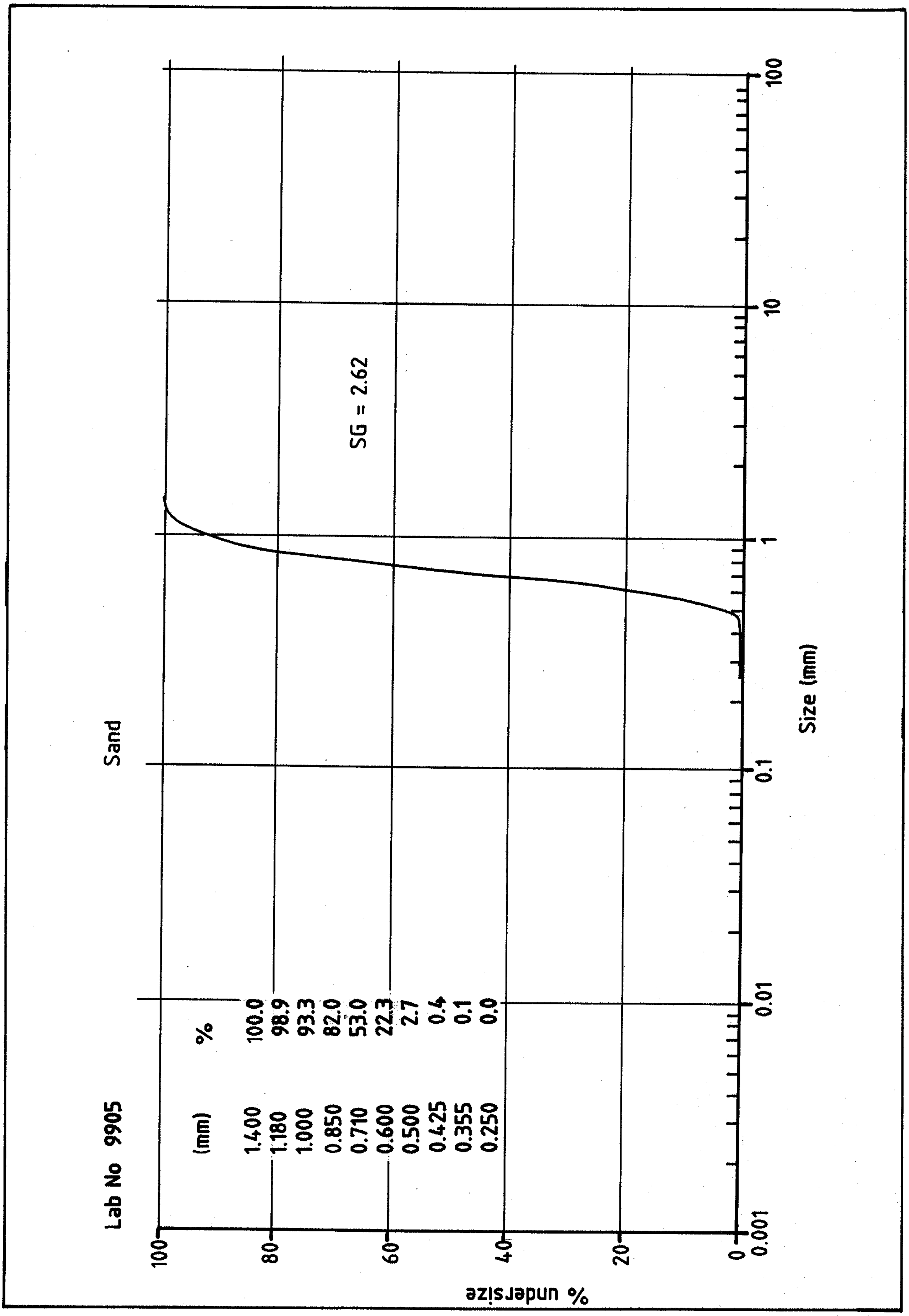

Fig 6 Sand grading

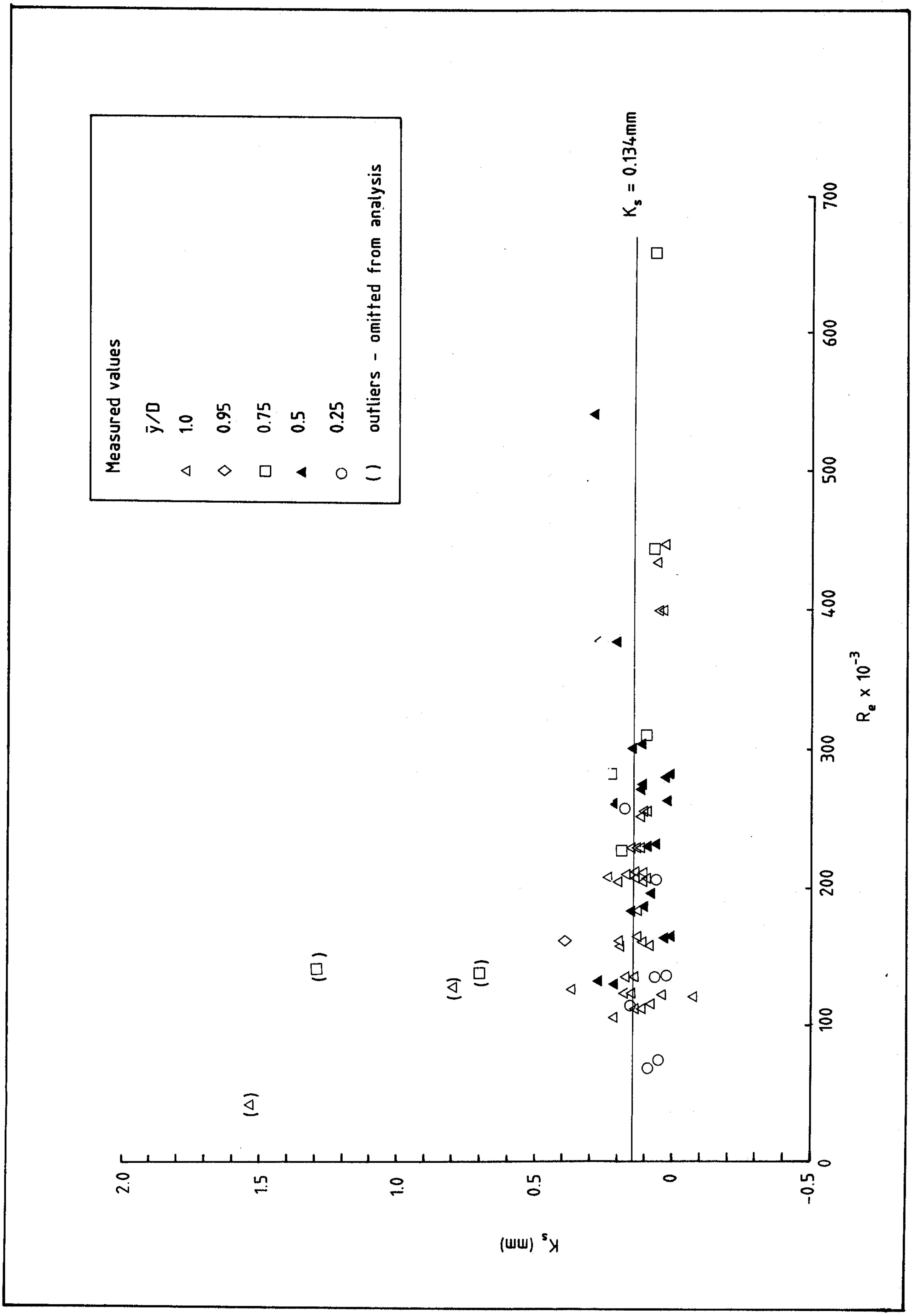

Fig 7 K_s v Reynolds Number for clear water

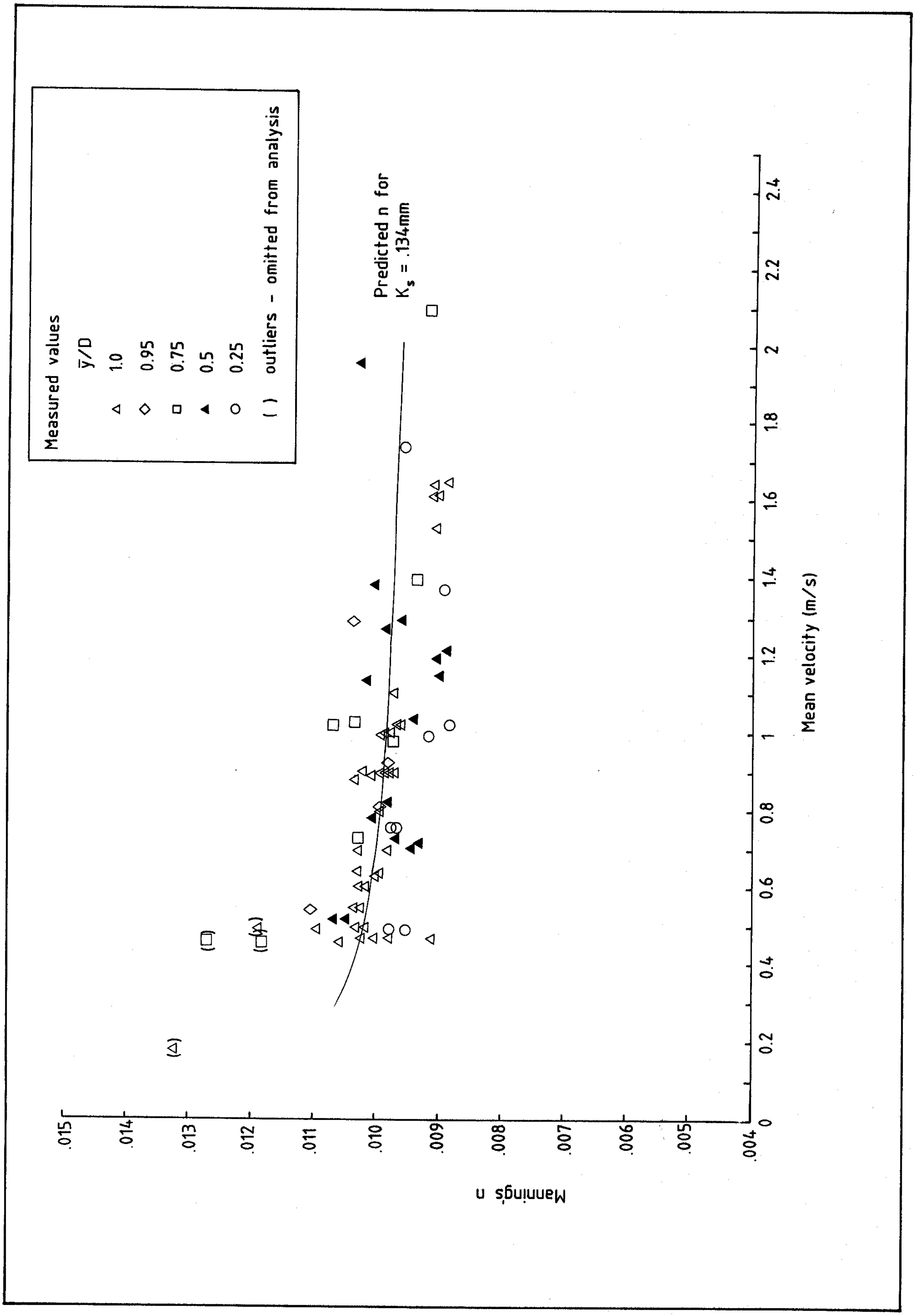

Fig 8 Manning's n v mean velocity

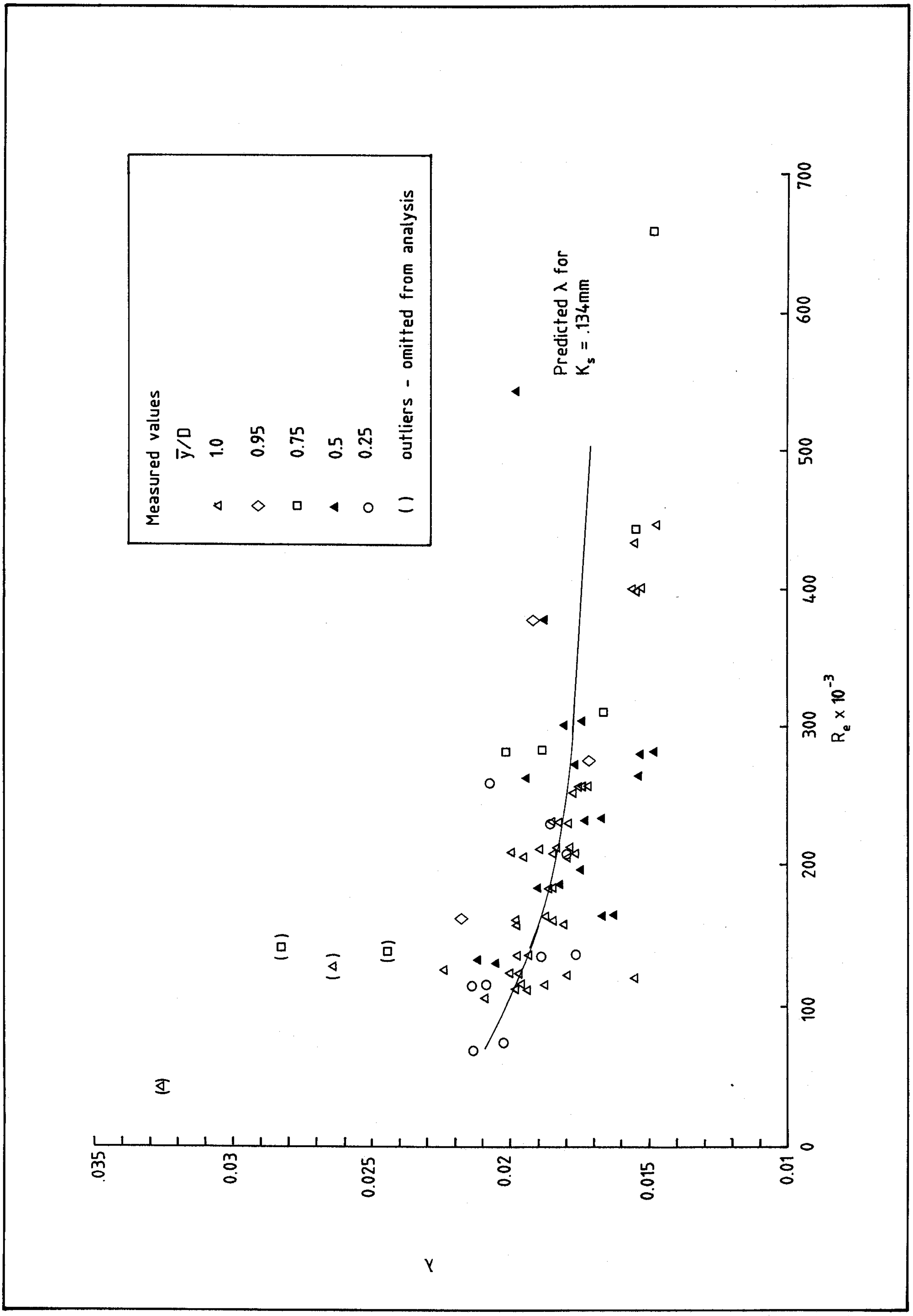

Fig 9 Friction factor λ v Reynolds Number

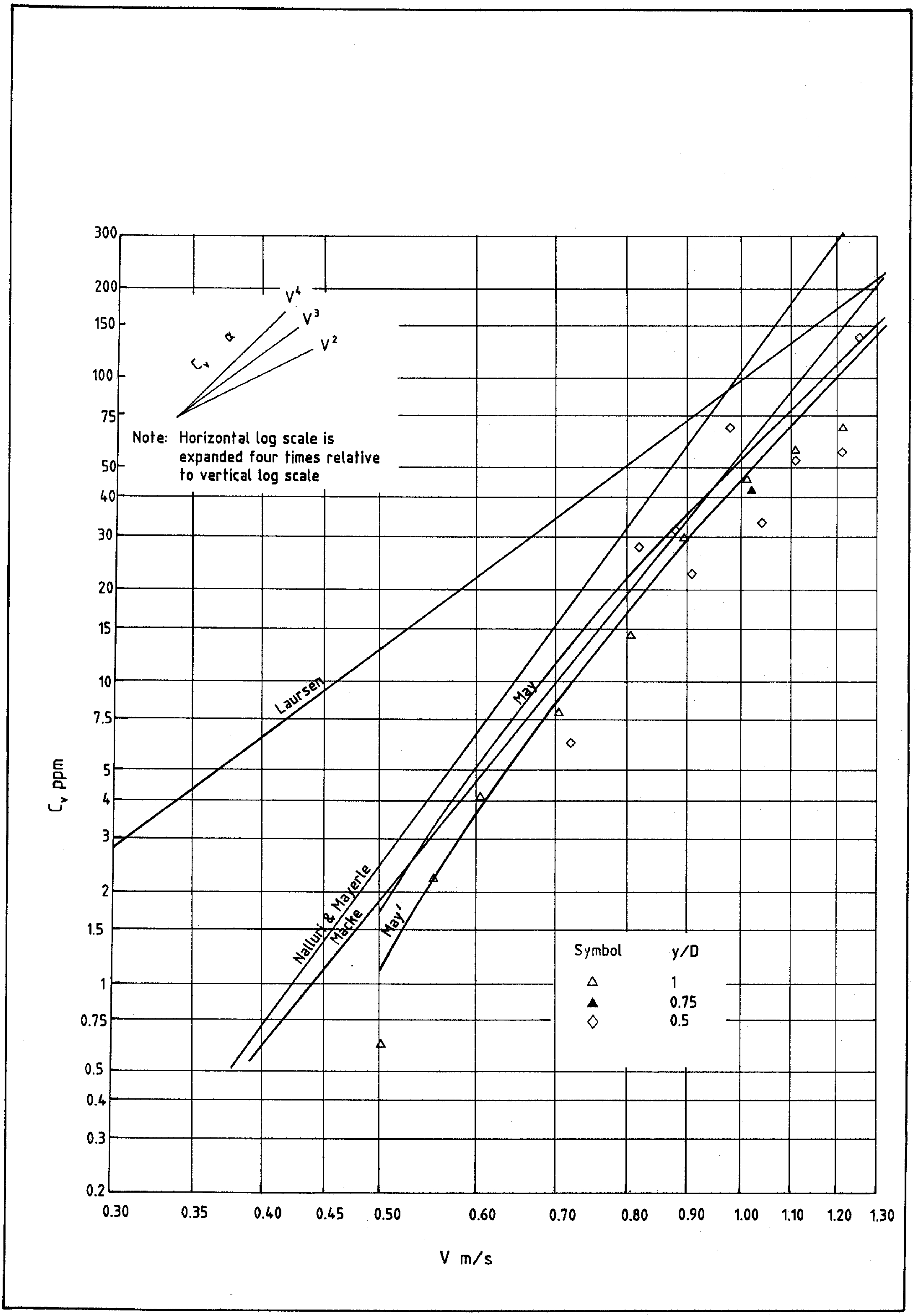

Fig 10 Limit of desposition: Experimental data and comparison with prediction formulae

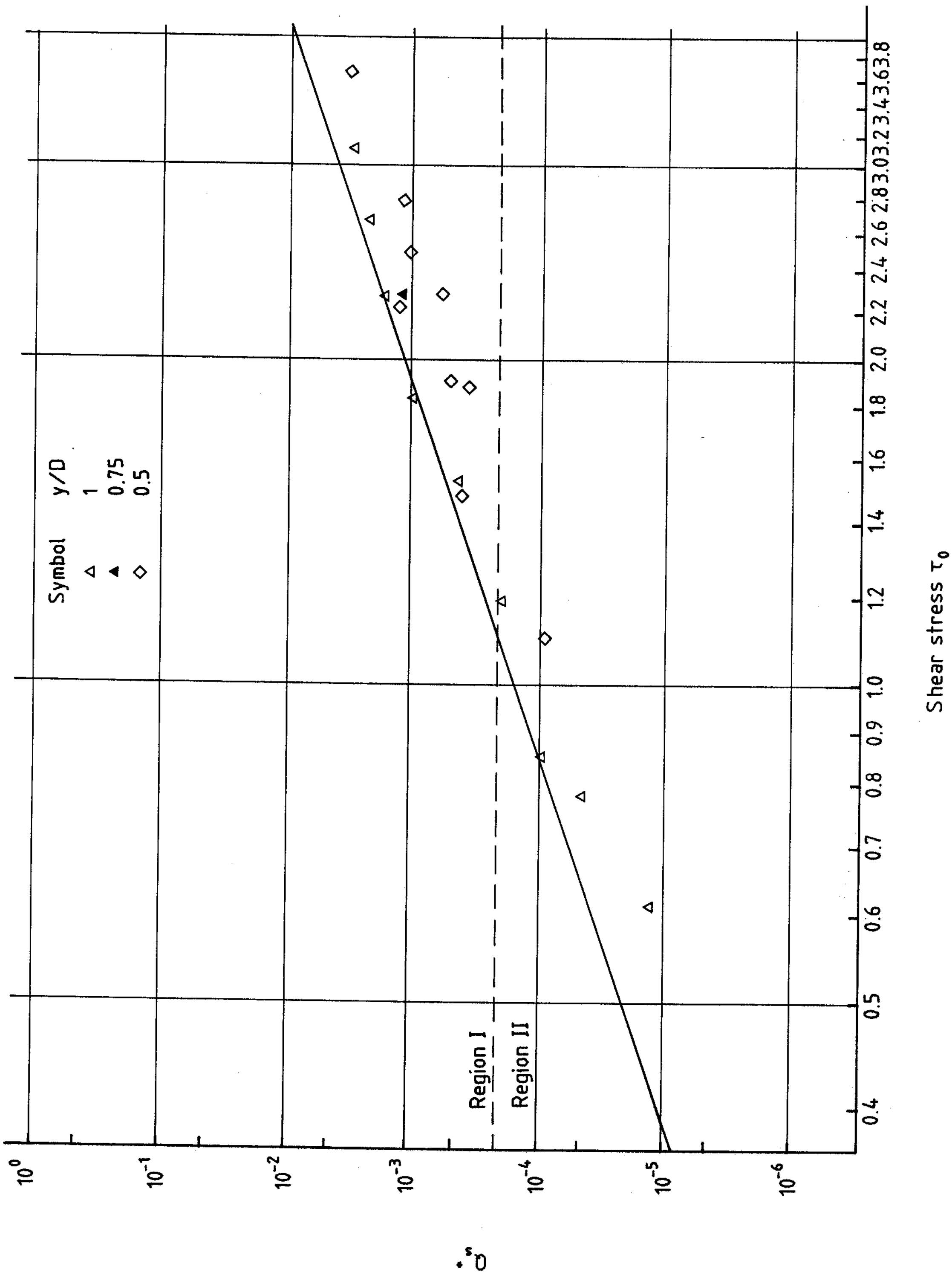

Fig 11 Limit of deposition data: Macke format

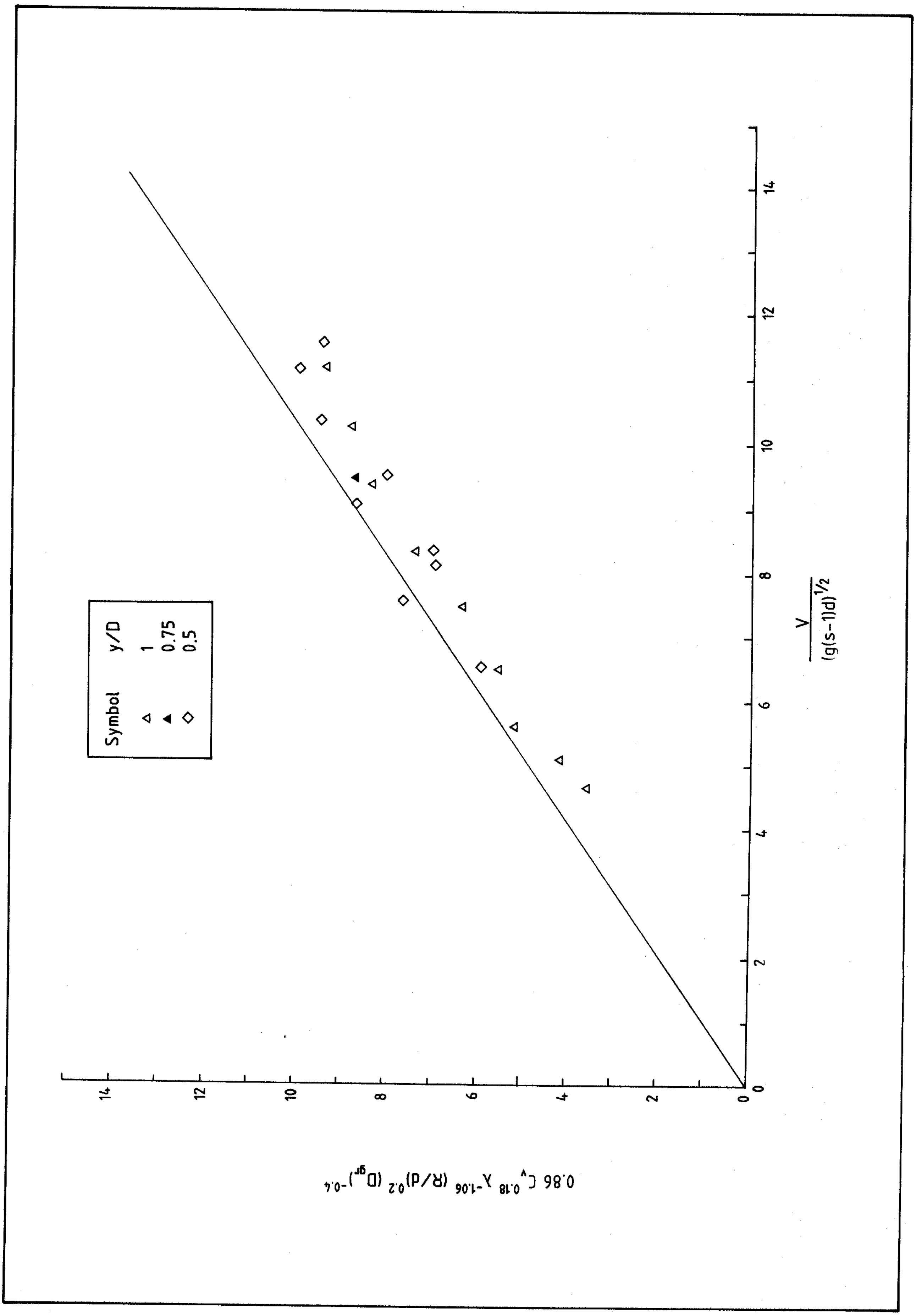

Fig 12 Limit of deposition data: Nalluri format

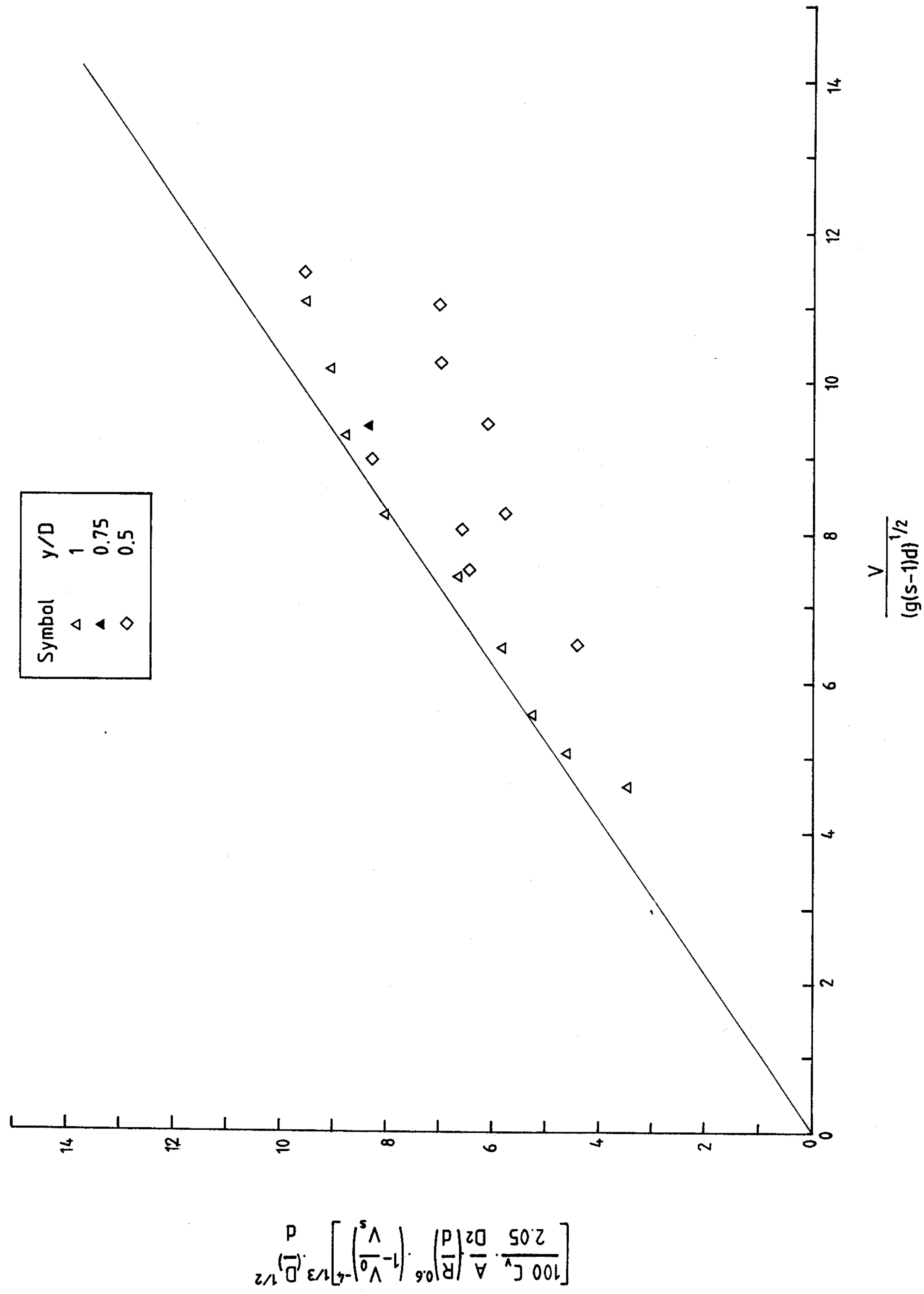

Fig 13 Limit of deposition data: May format.

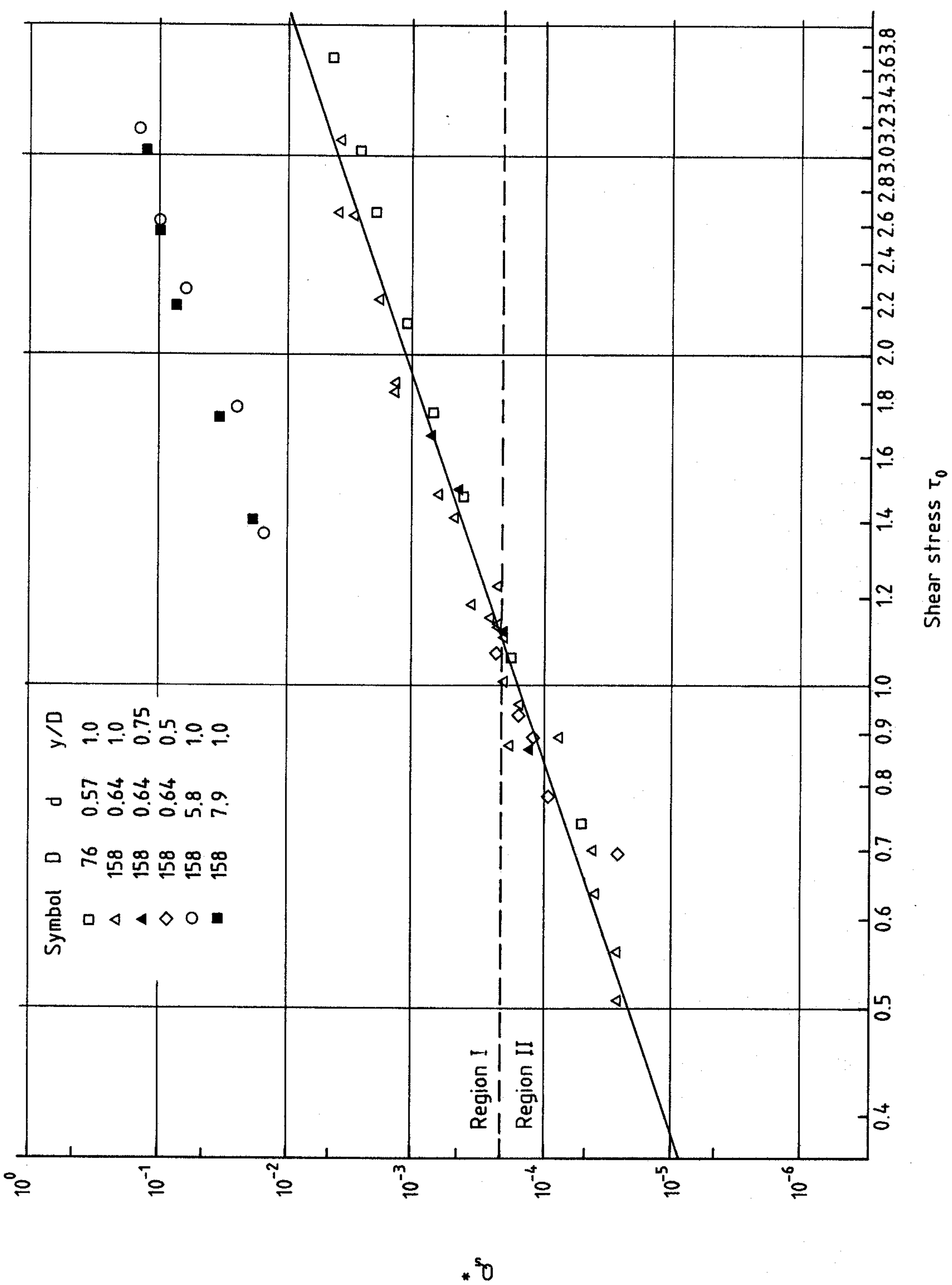

Fig 14 Limit of deposition data (HRS study): Macke format

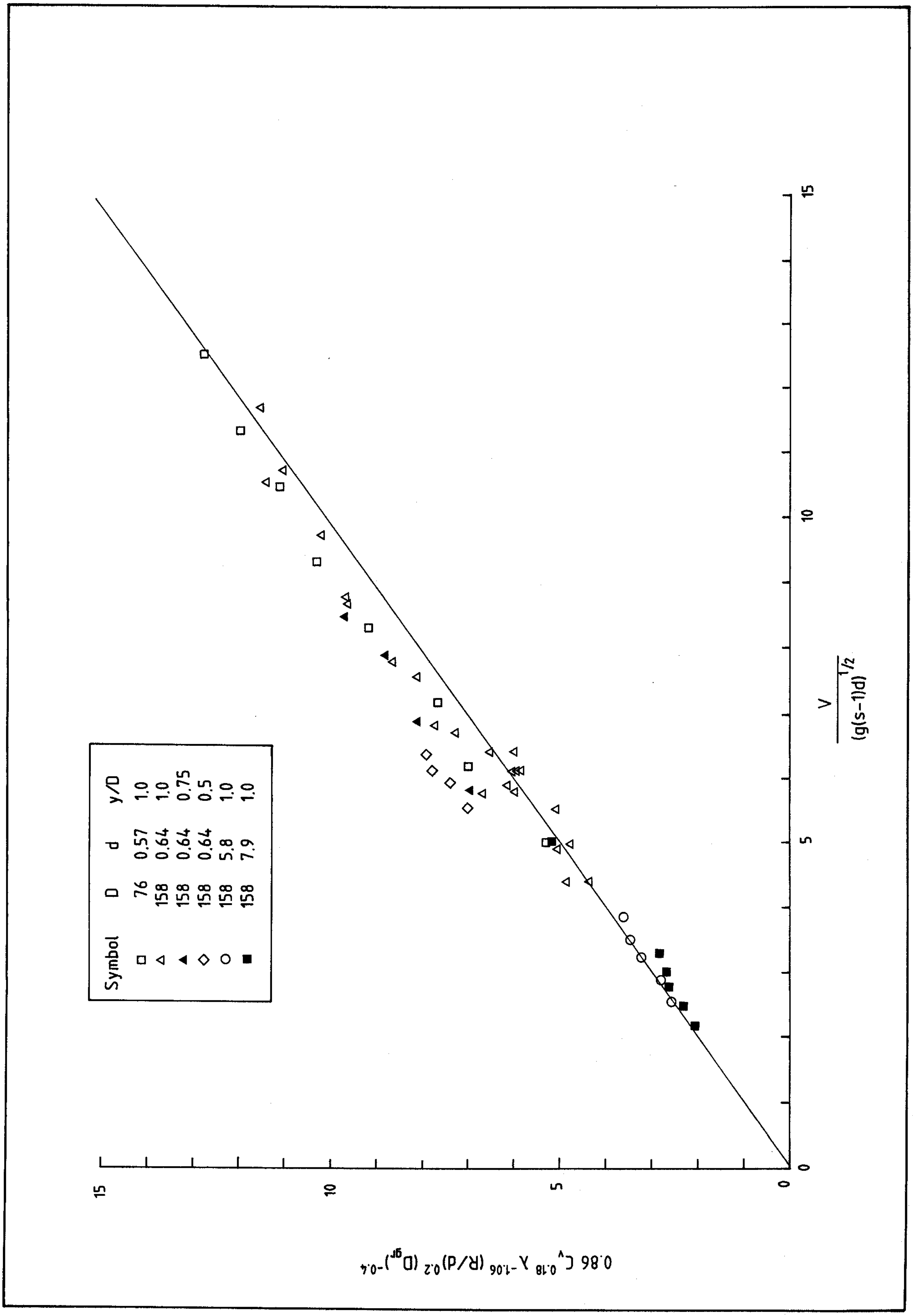

Fig 15 Limit of deposition data (HRS study): Nalluri format

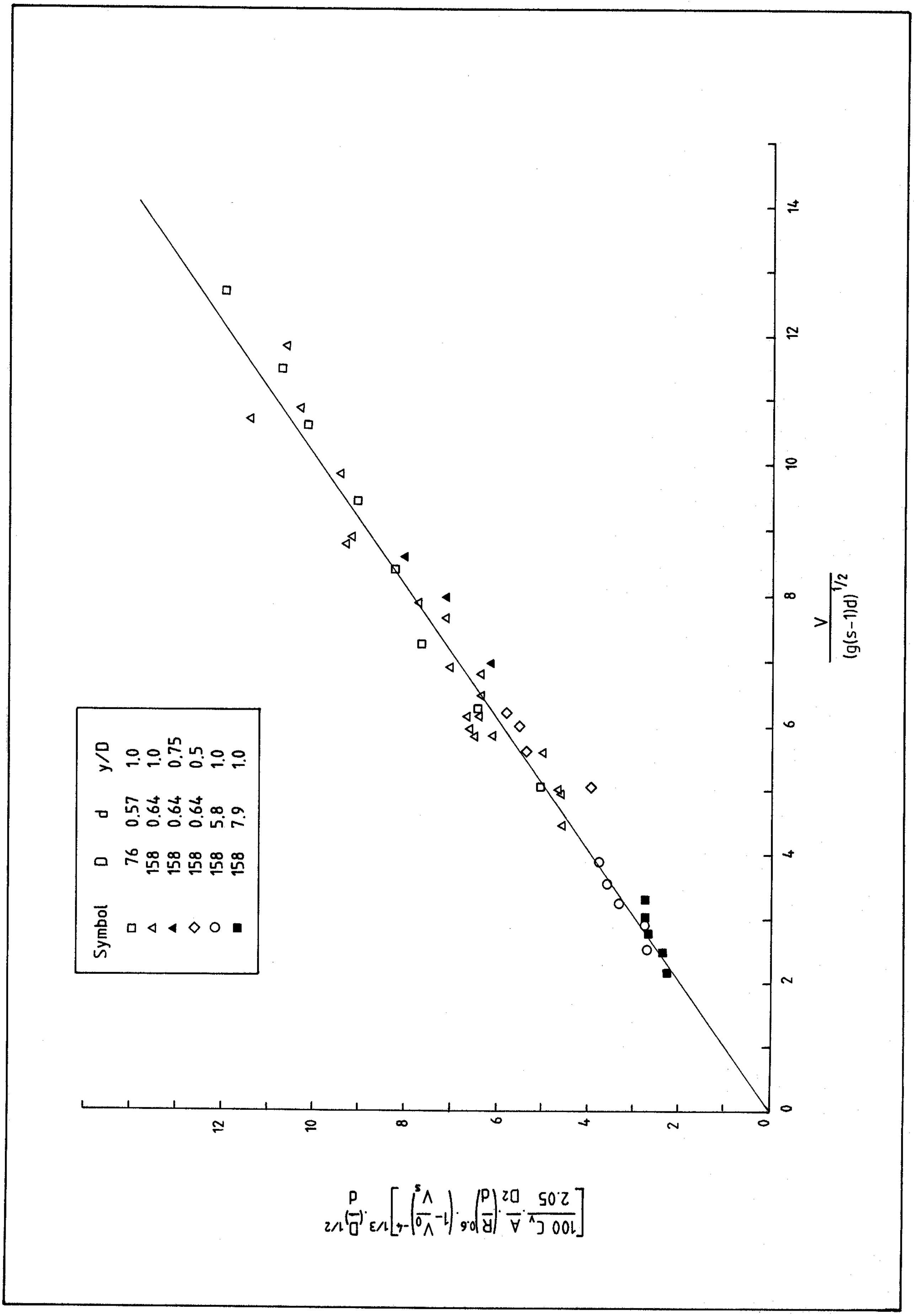

Fig 16 Limit of deposition data (HRS study): May format

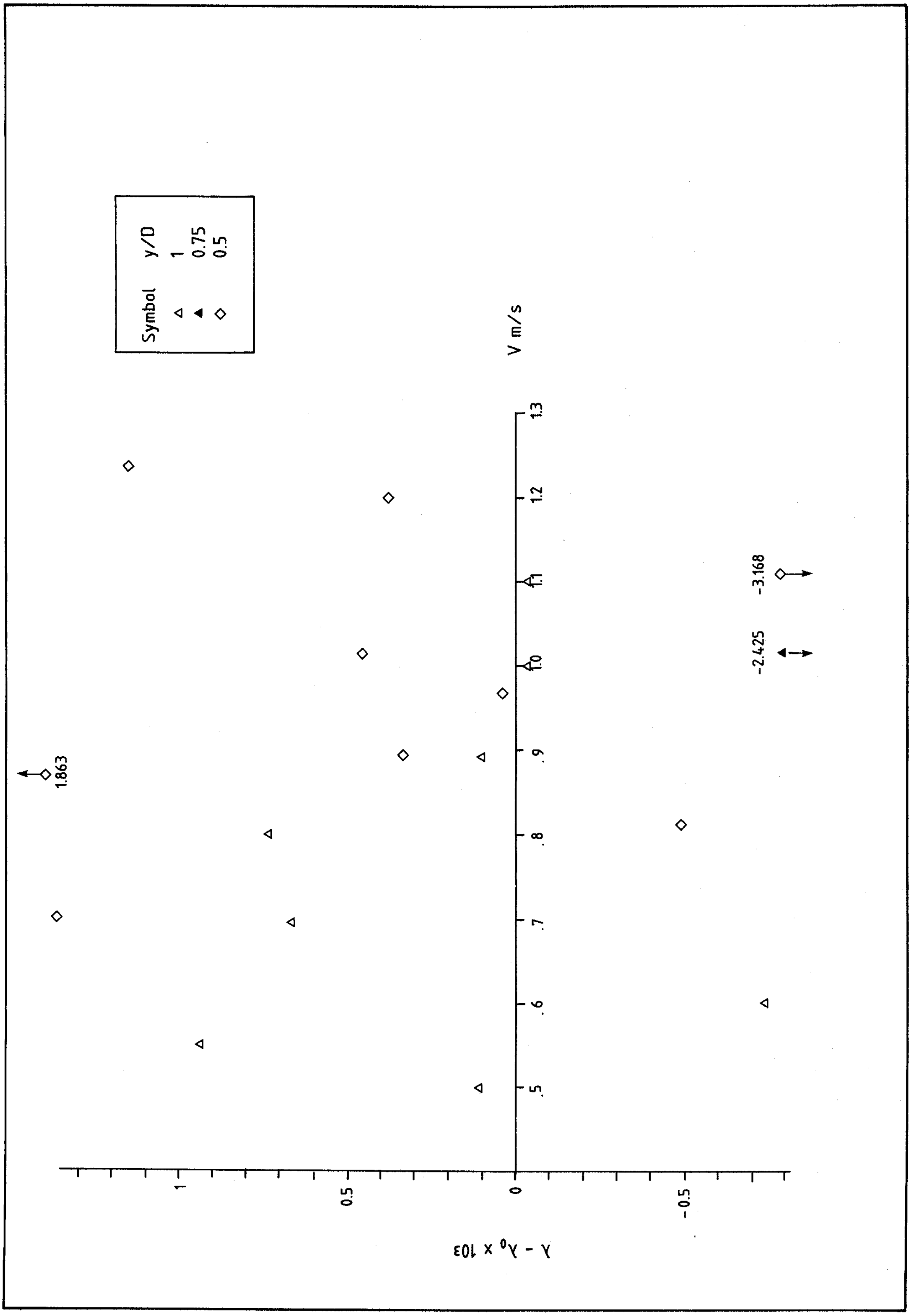

Fig 17 Head loss at limit of deposition

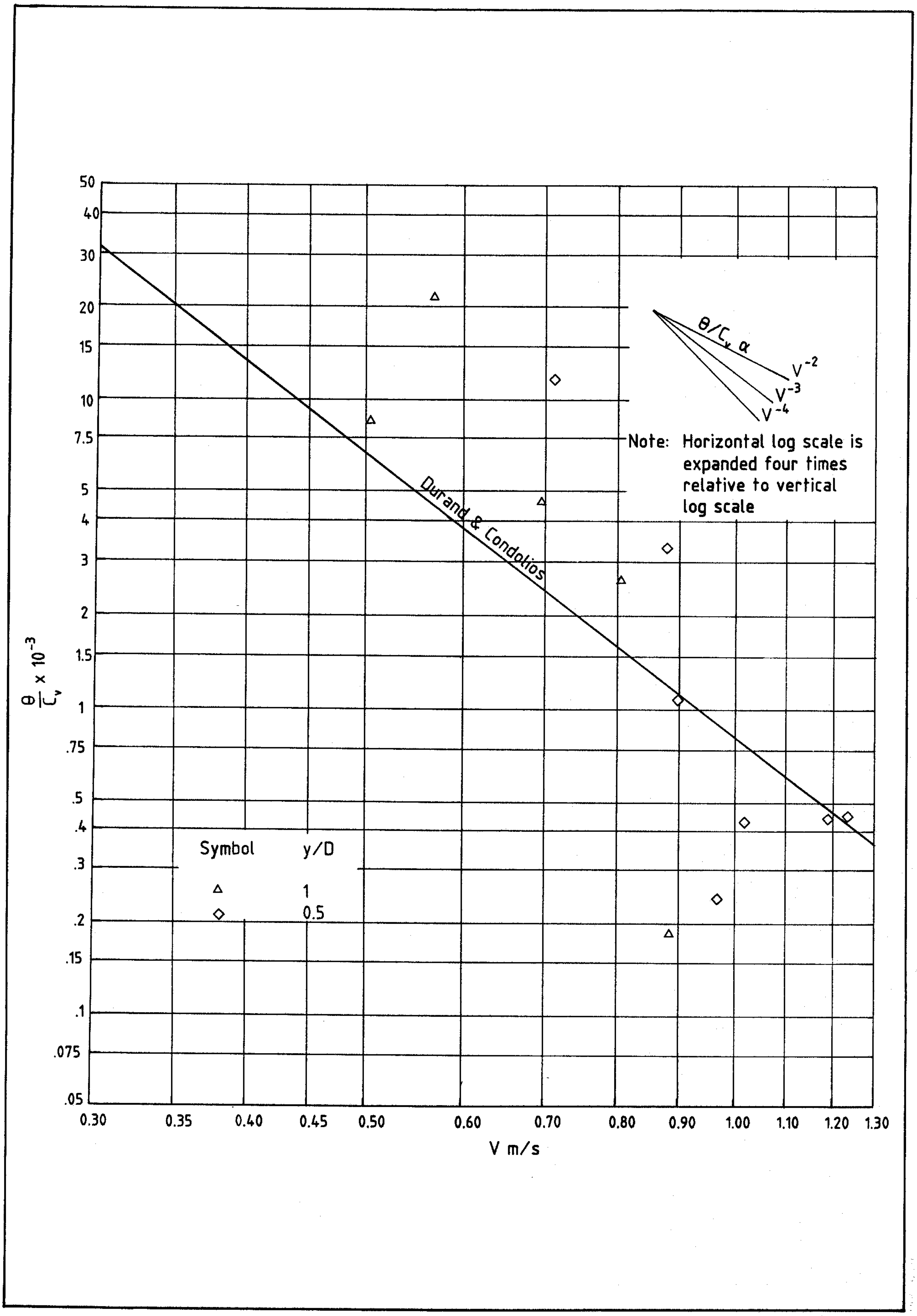

Fig 18 Increase in head loss due to sediment: Experimental data

APPENDIX.

APPENDIX 1 : Formulae for viscosity and settling velocity

1. Kinematic viscosity, ν

$$\nu = \frac{1.79 \times 10^{-6}}{1 + 0.03368T + 0.000221T^{2}}$$

where T is the temperature in degrees centigrade.

2. Fall velocity of the particle, w, in m/s:

$$w = \frac{\{9\nu^{2} + 10^{-9} d^{2} g (s - 1)(0.03869 + 0.0248d)\}^{1/2} - 3\nu}{[0.11607 + 0.074405\, d] \times 10^{-3}}$$

Here ν = kinematic viscosity of fluid in m^{2}/s

d = sediment size in mm

and s = specific gravity of sediment