HR Wallingford

# FLUIDISATION OF MUD BY WAVES

## Laboratory and field experiments

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Report SR 268
April 1991

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CONTRACT

This report describes work funded by the Department of the Environment under Research contract PECD 7/6/143, for which the nominated officer was Mr P Woodhead, and by HR Wallingford. It is published on behalf of the Department of the Environment, but any opinions expressed in the report are not necessarily those of the funding Department. The work was carried out at HR Wallingford in Dr E A Delo's Section of the Tidal Engineering Department, under the management of Mr M F C Thorn.



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ABSTRACT

Previous studies at HR Wallingford have investigated the effect of monofrequency waves on a mud bed, in a laboratory wave flume. In reality, the sea bed is subjected to a pattern of random waves, which generate much more variable conditions in the wave boundary layer. An extension of the previous research has been to investigate the effect of a more realistic sequence of random waves on a mud bed.

This report describes laboratory tests conducted at HR Wallingford on mud beds, under monofrequency waves and random waves. It describes the recent modifications to the wave flume which make the apparatus more effective. Laboratory tests conducted by University College of Swansea to investigate the structure of the bed under waves are also included.

The report also describes the field measurement of the combined effect of tides and waves, following the development of field equipment for measurement of near-bed cohesive sediment processes.

A series of tests on Fawley mud compared the critical shear stress for erosion and erosion rate for equivalent monofrequency and random waves. The critical shear stress for erosion of a bed of Fawley mud of density $300kgm^{-3}$ was around $0.1Nm^{-2}$. The erosion rate appeared to be higher for the monofrequency tests than for the random tests on the same mud, with $1.0\ 10^{-4}kgm^{-2}s^{-1}$ to $1.3\ 10^{-4}kgm^{-2}s^{-1}$ for monofrequency tests, and $4\ 10^{-5}$ to $6\ 10^{-5}kgm^{-2}s^{-1}$ for the random tests. These erosion rates were for wave periods of around 1.2 seconds. An estimate for erosion rates at different wave periods was made, based on the assumption that erosion occurs for the same proportion of the wave cycle.

The density of the mud bed was not observed to change much during a test. Transport of mud from the paddle end of the flume towards the beach end was observed. Transport rates per unit width of the flume were calculated to be $1\ 10^{-5}kgm^{-1}s^{-1}$ to $4\ 10^{-5}kgm^{-1}s^{-1}$.

Laboratory measurements and in-situ rheological measurements of mud under waves were made by University College of Swansea. A rheometer, which used the sensitivity of the shear wave velocity to the structure of the bed, was incorporated into a wave flume. Under the action of waves, fluidisation of the upper material occurred, with the bed sediment losing its elastic properties and changing from a viscoelastic material to a shear-thinning non-Newtonian suspension. The weakening effect of repeated cyclical stressing was observed at moderate amplitudes, with complete loss of measurable structure occurring at larger amplitudes. After a recovery period of 2 hours (no waves), the structure of the bed had only recovered about 70%.

A bed frame was used to make field measurements of wave parameters at the same time as near-bed cohesive sediment processes. Wave periods of 1.5 to 2.5 seconds were recorded, with wave heights up to 0.2m. The wave period increased as the depth increased over the inter-tidal mudflats, where the frame was deployed. The wave induced shear stress was highest when the water depth was very shallow; however, this was not high enough to erode the consolidated mudflat.

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# 1 INTRODUCTION

The erosion of mud by waves plays a particularly important role in the cohesive sediment transport along muddy coasts and in wide shallow estuaries close to the sea. So far, the research has mainly been to investigate the effect of tidal currents on the erosion and deposition of mud. This includes laboratory tests and, more recently, field measurements. However, there are many shallow areas, such as inter-tidal mudflats, where waves may have a significant influence on the erosion and deposition of cohesive sediment.

Previous studies at HR Wallingford (Ref 1) have investigated the effect of monofrequency waves on a mud bed, in a laboratory wave flume. In reality, the sea bed is subjected to a pattern of random waves, which generate much more variable conditions in the wave boundary layer. An extension of the previous research has been to investigate the effect of a more realistic sequence of random waves on a mud bed. This has been carried out in laboratory wave flumes and by development of field equipment for measuring cohesive sediment near-bed processes.

This report describes laboratory tests conducted at HR Wallingford on mud beds, under monofrequency waves and random waves. It describes the recent modifications to the wave flume which make the apparatus more effective. Laboratory tests conducted by University College of Swansea to investigate the structure of the bed under waves are also included.

The report also describes the field measurement of the combined effect of tides and waves, following the development of field equipment which was described in an earlier report (Ref 2).

## 2 LABORATORY TESTS

### 2.1 Objectives

The objectives of the laboratory wave tests were to determine a critical shear stress for erosion under random waves, and to compare the effect of random waves with that of equivalent monofrequency waves. The parameters of wave height, wave period and water depth were chosen to give a range of resulting bottom orbital velocities similar to that found in the field. A series of tests was run on Fawley mud with monofrequency waves. Modifications were made to the flume to generate random waves, and a further series of tests was run on Fawley mud. In addition, several other muds have been tested in the wave flume under random waves. These tests were not part of this contract, but a summary of the results is included, in terms of wave conditions, shear stresses and erosion rates.

### 2.2 Description of apparatus

The tests on Fawley mud were carried out in a wave flume 23m long and 0.75m wide, with a maximum water depth of 0.55m. There was a trough in the floor of the flume to hold the test bed, starting 8.5m from the end of the flume with the wave paddle. The trough was 10m long, 0.2m wide and 0.1m deep. At each end of the test bed section there was a raised lip to prevent mud escaping from the bed as bed load transport (Fig 1, with current flume configuration).

The flume was modified to allow for two wave generators, which could be set for monochromatic and random waves. For monochromatic waves, the user controlled the wave frequency and wave height manually, by altering the stroke length and frequency on the generator. The random wave generator was controlled by a micro-computer. For random waves, the user input a zero crossing period and a significant wave height; the program generated a random wave spectrum which satisfied these input conditions. The wave spectrum generated in this case was the JONSWAP spectrum. At the other end of the flume to the generators there was a shingle spending beach to absorb the wave energy and hence minimise wave reflections back into the flume.

An ultrasonic probe mounted on a moving platform above the bed was used to monitor the surface level of the mud bed at intervals along the length of the bed.

A turbidity sensor was used to measure suspended sediment concentrations at various depths and positions along the flume, at time intervals throughout each test. The instrument used was an Analite Nephelometer, which operates on the principle of backscatter of infra red light. The degree of backscatter is proportional to the reflection coefficient and concentration of suspended sediment.

At positions throughout the test section water pressure transducers were mounted below the water surface and the signals logged onto a computer. The wave spectrum was determined by analysis of the water pressure signals.

A conductivity probe was used to determine the vertical bed density profile. The overall conductivity of the mud is a function of the conductivity of the pore water and the volume of voids. It was assumed that the conductivity of the pore water was constant and, hence, the overall conductivity of the mud was a function of its pore volume ie. its density. The instrument was calibrated using samples of known dry density of the mud under investigation and with saline solutions of known density. The calibration samples had dry densities typically in the range $50kgm^{-3}$ to $500kgm^{-3}$.

For the later tests on Marsden, Poole and Hongkong muds, the flume was modified again. For these tests, the flume was 23m long, 0.3m wide, 0.55m maximum water depth. The test section was 8m long, 0.3m wide, 0.1m deep. Both monofrequency and random wave generators were available. The bed level readings from the ultrasonic probe and the concentration readings from the turbidity were automated. These were measured using a computer controlled robot, which travelled the length of the test section at various depths. The data recorded was logged on to a micro-computer. This automatic monitoring system allowed much easier collection of data, at more points along the test section.

## 2.3 Details of tests

After several preliminary tests to check the operation of the equipment and instruments, the following sets of tests were run:

- 10 tests on Fawley mud, monofrequency waves
- 6 tests on Fawley mud, random waves
- 3 tests on Wentlooge mud, monofrequency waves
- 2 tests on Wentlooge mud, random waves

For each test, a uniform slurry was made up from the bulk mud sample by dilution with saline water. This slurry had a dry density in the range 300kgm$^{-3}$ - 400kgm$^{-3}$. This was poured into the test section of the flume. The flume was filled with saline water at 30kgm$^{-3}$.

The placed mud bed was then subjected to a pattern of waves, either monochromatic or random waves. The input wave period was fixed for the duration of all the monofrequency tests at 1.2 seconds. For the random wave tests the input wave period was fixed in some tests at 1.2s, in others it varied between 1s and 2s. The input parameter of wave height (significant wave height for random waves) was changed up to three times during a test; each wave height was imposed for one hour, then it was changed so that the resulting peak bed shear stress was increased. Wave heights were in the range 0.0 - 0.12m.

The vertical density profile of the bed was determined with the conductivity probe before the start of a test and at half-hourly intervals during a test. This monitored changes in bed density caused by consolidation or wave action. The bed levels at 0.5m intervals along the test section were measured at the start of the test and at half-hourly intervals with the ultrasonic probe. The concentration of suspended sediment was measured every 30 minutes at several heights in the water column at positions along the entire length of the flume.

The conditions of water depth, wave height, wave period and resulting bottom orbital velocity and peak bed shear stress are summarised for these tests in Table 2.

## 2.4 Results

### 2.4.1 Shear stress on the bed

The parameters of water depth, wave height and wave period for these tests were chosen to give resulting peak bed shear stresses similar to those in the field. The effect of varying these parameters individually has not been extensively investigated.

For monochromatic waves, the peak bed shear stress can be calculated from the water depth and wave characteristics using first order linear wave theory (Ref 3) such that:

$$U_m = \frac{\pi H}{T \sinh (2\pi d/L)} \qquad (1)$$

where

$U_m$ = maximum bottom orbital velocity ($ms^{-1}$)

H = wave height (m)

T = wave period (s)

L = wave length (m)

d = water depth (m)

The magnitude of the wave length is determined iteratively from:

$$\omega^2 = gk \tanh (kd) \qquad (2)$$

where

$\omega = 2\pi/T$ ($s^{-1}$)

g = acceleration due to gravity ($ms^{-2}$)

$k = 2\pi/L$ ($m^{-1}$)

The peak bed shear stress is estimated using the following relationship :

$$\tau_m = 0.5 \ \rho_w f_w U_m^2 \tag{3}$$

where

$\tau_m$ = peak bed shear stress ($Nm^{-2}$)

$\rho_w$ = fluid density ($kgm^{-3}$)

$f_w$ = wave friction factor

$U_m$ = maximum bottom orbital velocity ($ms^{-1}$)

The wave friction factor is dependent upon the wave reynolds number and the relative roughness (Ref 4).

For random waves, the surface elevation spectrum generated is the JONSWAP spectrum:

$$S(f) = 4.732 \ 10^{-4} \exp(-1.25 \ f_m^4 \ f^{-4}) \ f^{-5} \ 3.3^x \tag{4}$$

with

$x = \exp(-(f \ f_m^{-1} - 1)^2 \ y^{-1})$

$y = 0.0162$ if $f > f_m$

$= 0.0098$ if $f \leq f_m$

where

$f$ = frequency of ordinate

$f_m$ = frequency at which spectral peak occurs

$= 0.87T_z^{-1}$ or $= 0.217H_s^{-0.5}$

$T_z$ = mean zero crossing period (seconds)

$H_s$ = significant wave height (m)

$S(f)$ = value of spectral ordinate at frequency f

Where both $T_z$ and $H_s$ are input, $T_z$ is used as the defining value and the spectrum magnitude is defined with an implied gain. The near-bottom velocity cannot now be described by a single $U_m$, and is usually described by the standard deviation, $U_{rms}$, of the time-series of instantaneous velocities.

$U_{rms}$ can be approximated as a function of $H_s$ and $T_z$ (Ref 3):

$$U_{rms}T_n/H_s = 0.25/(1 + At^2)^3 \qquad (5)$$

where

$A = (6500 + (0.56 + 15.54t)^6\ )^{1/6}$

$t = T_n\ /\ T_z$

$T_n = (d/g)^{0.5}$ scaling period

d = water depth

g = acceleration due to gravity

This fits the JONSWAP curve to an accuracy of better than 1% in the range $0 < t < 0.55$.

For equivalent monochromatic and random waves with the same variance of bottom orbital velocities

$$U_m = \sqrt{2}\ U_{rms} \qquad (6)$$

A peak significant shear stress for random waves can therefore be calculated according to equation 3, using $\sqrt{2}\ U_{rms}$ in place of $U_m$. The equations used to calculate a bed shear stress from the bottom orbital velocity are given in Table 1.

Table 2 summarises the wave conditions during each test and the resulting bottom orbital velocities and shear stresses. For the random wave tests, the root mean square bottom orbital velocity, $U_{rms}$, has been calculated as well as a peak significant orbital velocity, for comparison with the monofrequency tests. For most of the Fawley tests, the peak shear stress (monofrequency equivalent) generated in the random waves was much lower than in the monofrequency tests. Only test FR6 had shear stresses similar to those in some of the monofrequency tests, particularly tests FM20, FM21 and FM25. The shear stresses in these tests were in the range 0.23 - 0.27Nm$^{-2}$. In the Wentlooge tests, the shear stresses in tests WM1, WM3, WR1 and WR2 were all similar around 0.24 - 0.26Nm$^{-2}$.

Details of the wave tests on Poole, Hong Kong and Marsden muds are given in Table 3. The shear stresses generated in the Poole tests covered a lower range than those in the HongKong tests. The monofrequency waves generated in the Marsden tests resulted in the highest shear stresses.

A critical shear stress for erosion by waves, for a bed of Fawley mud of density of around 300kgm$^{-3}$, has been estimated by considering test FM18; the monofrequency waves were increased in wave height at each step, resulting in a stepped increase in the peak bed shear stress. The concentration of sediment in suspension at 0.1m above the bed, along the length of the flume, is shown at half hour intervals in Figure 2. Distances along the flume have been measured starting at the paddle end of the test section. Some mud has been eroded after an hour (peak shear stress 0.14Nm$^{-2}$), but more material has been eroded after 1.5 hours (peak shear stress 0.24Nm$^{-2}$). The material in suspension also appears to be moving down the flume towards the beach, particularly at the higher shear stresses, where the concentration at the beach end of the flume is much higher than at the paddle end. Based on the point of observed increase in concentration in suspension, the critical peak shear stress for erosion was below 0.14Nm$^{-2}$. A better estimate for critical shear stress can be made by considering the erosion rate.

### 2.4.2 Erosion rate

For each test, an erosion rate for each half hour period was calculated from the increase in suspended sediment concentration. A second erosion rate was calculated from the depth of erosion measured with the ultrasonic probe. The second method assumes that there is no consolidation of the bed. The lower part of Figure 2 shows the erosion rate against peak bed shear stress for test FM18. This shows erosion rates in the range 0.00002 - 0.00007$kgm^{-2}s^{-1}$. If one assumes that the erosion rate is proportional to the excess shear stress, then an estimate for the critical shear stress can be made from Figure 2. This suggests a critical shear stress of around 0.1$Nm^{-2}$.

Figure 3 shows the results of peak shear stress against erosion rate for all the Fawley monofrequency tests and all the Fawley random wave tests. The erosion rates for the monofrequency tests are, in general, higher than those for random wave tests, with erosion rates of 0.00010 - 0.00013$kgm^{-2}s^{-1}$ (monofrequency) and 0.00004 - 0.00006$kgm^{-2}s^{-1}$ (random).

These erosion rates are for a wave period of 1.2s, whereas in reality the wave period is more likely to be longer. However, if one assumes that the critical shear stress is exceeded for the same percentage of the time during a complete wave period, then corresponding erosion rates for a longer period wave of period T (with the same peak shear stress) could be estimated by multiplying by 1.2/T. For a wave period of 7s, this would give erosion rates of about 2 $10^{-5}kgm^{-2}s^{-1}$ for monofrequency waves and 1 $10^{-5}kgm^{-2}s^{-1}$ for random waves.

The measured erosion rates from all the wave tests are summarised in Table 4. These range from 0.00003kgm$^{-2}$s$^{-1}$ for the Marsden tests (which had some of the highest shear stresses, but was a high density bed) to 0.0004kgm$^{-2}$s$^{-1}$ in the Wentlooge tests (a much lower density bed).

### 2.4.3 Bed density

During the wave tests the density of the bed was measured at half hour intervals with a conductivity meter. This instrument indicates the shape of the density profile, and can be used to see if the profile changes significantly during a test. For these tests, each mud bed was placed at a constant density. The effect of the waves on the density profile can be seen from subsequent profiles.

Figure 4 shows the density profiles recorded during test FM19. These density profiles are typical of this set of wave tests. Test FM19 shows that the constant initial density was retained throughout the test. There is no change in density at the surface of the bed. However, the surface of the bed may be fluidised (a change in structure) without a change in density.

The bed density profiles were measured at a point in the centre of the test section. From the density profiles, the bed actually appeared to increase in depth during the test. A rise in bed levels at one end of the test section was also shown with the readings from the ultrasonic transducer. The bed levels were recorded along the length of the flume, at half hour intervals. Figure 5 shows the bed levels for test FM19, indicating that material has moved from one end of the test section to the other.

At the end nearest the paddle (0m) the bed level had dropped by approximtely 1mm after the first hour and around 2mm after three hours. At the other end of the test section the bed level had risen by around 1mm. This is very typical of the wave tests conducted in this series. It shows the effect of a finite length test section, and uni-directional waves.

### 2.4.4 Mud mass transport rate

The bed level measurements made during wave tests may be used to estimate mass transport rates along the flume (Ref 5). Typically during a test, the bed levels went down at the paddle end of the flume, and up at the beach end. The concentrations in suspension at the beach end were also higher, indicating transport of material down the flume.

An average bed level was calculated at each time interval. The part of the profile above this average level was then calculated to be the volume transported. A mass transport rate was calculated by multiplying the volume of material by the density of the mud surface. Volumes from successive profiles were subtracted to give the volume transported during a time interval. Volumes were calculated per unit width of the flume. The total volume transported during each test is shown in Figure 6. After an initial sharp rise during the first 30 minutes, the volume transported in each test remains roughly constant. Test FM24 shows the largest volume transported at around $0.08m^3$ per metre width of the flume, which is two to three times larger than the volume transported in the other tests. This gives a mass transport rate of around $4\ 10^{-5} kgm^{-1}s^{-1}$ for the first 30 minutes of test FM24.

### 2.4.5 Bed structure

The structure of a mud bed under wave action has been studied by the University College of Swansea (U C Swansea). Laboratory measurements and in-situ rheometrical measurements of the mechanical response of cohesive sediment under waves were made prior to and during bed deformation and loss of structure. A full account of the work is given in Appendix 1, and summarised here.

An in-situ rheometer was developed, which utilised the sensitivity of shear wave velocity to the structure of the bed (through mechanical rigidity). The dimensions of the rheometer allowed it to be incorporated into flume experiments with minimal disturbance.

Shear wave transducers were mounted in the test section part of a wave flume. The test section was 100mm deep, with the transducers mounted at 23mm, 53mm and 80mm above the base. The transducers respond to shear wave arrival and dynamic pressure changes within the bed. Shear wave velocities and attenuation coefficients were measured at the three vertical locations. The dynamical bed response was investigated for two test procedures: waves of constant frequency and constant amplitude, and constant frequency and increasing amplitude.

A pronounced non-homogeneous distribution of increasing bed rigidity with depth was found. This was due to the non-linear dependence of the rigidity modulus, G, on the fractional volume of solids for cohesive sediments.

Fluidisation of the upper material occurred, with the bed sediment losing its elastic properties and changing from a viscoelastic material to a shear-thinning non-Newtonian suspension (the flocs being fully supported by the fluid). This fluidisation of the upper material also corresponded with weakening of the lower layers. The weakening effect of repeated cyclical stressing was observed at moderate amplitudes, with complete loss of measurable structure occurring at larger amplitudes. After a recovery period of 2 hours (no waves), the structure of the bed had not completely recovered (recovery only ~70%).

These flume tests indicated that in-situ rheometry, based on shear wave propagation techniques, can be used to determine the dynamical behaviour of a visco-elastic sediment under uni-directional wave loading.

# 3 FIELD MEASUREMENTS

## 3.1 Description of apparatus

A bed frame was designed to enable simultaneous field measurements of tide and wave induced flow and near-bed cohesive sediment processes (Ref 2). The frame was triangular in shape with an open structure to offer as little resistance to the flow as possible. It had three legs which could be pushed into the bed to prevent movement (Fig 7).

The following instruments were mounted on the frame:

- 3 Braystoke current meters
- 4 electromagnetic current meters (EMCM)
- 3 Partech turbidity sensors
- 1 pressure sensor
- 3 ultrasonic probes

The Braystoke current meters were mounted at 0.1m, 0.5m and 1.0m above the bed and in such a way that they could swivel freely through 360 degrees to align themselves with the current. These were used to measure tidal flows and to give a field calibration of the EMCMs.

The EMCMs measured 2 components of velocities. These were mounted at 0.1m and 0.5m above the bed and set to measure in the x,y and x,z planes at each height. The output from these was logged at 5Hz to enable high frequency wave and turbulence fluctuations to be analysed.

The Partech turbidity sensors were mounted at 0.1m, 0.5m and 1.0m above the bed in positions that would not interfere with flow around the EMCMs.

The pressure sensor was mounted at 0.8m above the bed. The output was logged at 5Hz and enabled water depth and wave conditions to be found.

The ultrasonic probes were mounted at approximately 0.1m above the bed, either attached to a spur on each leg of the bedframe or about 3 metres away from the main bedframe. These enabled the bed level change during the tide to be monitored.

The output from each of the instruments was logged onto a computer over the monitoring period. Full details of the bed frame, instrumentation and analysis of data are given in Reference 6.

## 3.2 Details of deployments

The frame was deployed on seven occasions between November 1989 and December 1990. The first five deployments were on spring tides at a site on inter-tidal mudflats at the mouth of the River Usk, near Newport, Gwent. The Usk is a tributary of the River Severn, and therefore has a spring tidal range of around 12m. The bed frame was submerged for about 3 hours on each tide. Details of the deployments and a summary of results are given in Table 3. During some of the deployments difficulties were experienced with the pressure transducer and no information about wave conditions was recorded. Some wave activity was recorded during the April and May 1990 deployments and on one tide during the October deployment.

Two further deployments were made at Eastham Docks in the Mersey estuary, on spring and neap tides. This was a sheltered site with very little wave activity during these deployments. Details of these deployments are given in Table 4.

## 3.3 Results

Wave heights of up to 0.2m were recorded during the April and May deployments at West Usk. Wave periods of 1.5s to 2.5 seconds were recorded over the same period. Figures 8 and 9 show the wave height, wave period and water depth for two of the recorded tides. As expected, the wave period increased as the water depth increased, and then decreased again as the water depth decreased. There was little change in the wave height over the same period.

The wave induced shear stress was calculated from the spectral analysis of the high frequency velocity fluctuations ($\tau_{ew}$) and from the pressure variations ($\tau_w$) (see Ref 6, Appendix A). Very high values of $\tau_{ew}$ were obtained in a large proportion of the measurements. These were thought to be unrealistic and were most likely due to measurement errors caused by either flexing of the instrument mountings or disturbance of the flow around the current meters. Nevertheless, it was clear that even for quite small waves, the wave induced shear stress can be higher than the tide induced shear stress. This means that waves are an important factor in determining whether material is eroded or deposited. The mud may then be transported away from the site by the tidal velocities.

Figure 10 shows the wave induced shear stress for 25 May 1990 am, which gave reasonable values of the wave induced shear stress, calculated from both methods. The values of shear stress decreased as the water depth increased at the start of the monitoring period and then increased as the water depth decreased towards the end of the monitoring period.

The wave induced shear stress was not high enough to erode any of the bed in this case. This is probably because at the West Usk site, the mudflats consist of a base of very consolidated mud, on top of which there is sometimes a soft unconsolidated deposit. This soft deposit is easily eroded, and on the occasions when it has been seen at the West Usk site it has been eroded very quickly when the mudflats are first submerged, and before monitoring begins (as this requires at least 1m depth of water). It is thought that erosion of the consolidated mudflats would only take place during storms. So far monitoring has not taken place during a stormy period.

# 4 CONCLUSIONS AND RECOMMENDATIONS

## 4.1 Conclusions

1. The wave flume was modified to generate either monofrequency or random waves. A series of tests on Fawley mud compared the critical shear stress for erosion and erosion rate for equivalent monofrequency and random waves. The critical shear stress for erosion of a bed of Fawley mud of density $300 kgm^{-3}$ was around $0.1 Nm^{-2}$.

2. The erosion rate appeared to be higher for the monofrequency tests than for the random tests on the same mud, with $1.0\ 10^{-4} kgm^{-2}s^{-1}$ to $1.3\ 10^{-4} kgm^{-2}s^{-1}$ for monofrequency tests, and $4\ 10^{-5}$ to $6\ 10^{-5} kgm^{-2}s^{-1}$ for the random tests. These erosion rates were for wave periods of around 1.2 seconds. If one assumes that erosion takes place for the same proportion of the wave cycle even for a longer period wave, then erosion rates of around $1\ 10^{-5} kgm^{-2}s^{-1}$ were estimated for a random wave of period around 7s (a more realistic wave period for the field).

3. The density of the mud bed did not change much during a test. However, the density profiles were measured half way along the test section and some changes in bed thickness were observed. Transport of mud from the paddle end of the flume towards the beach end was observed. Transport rates per unit width of the flume were calculated to be $1\ 10^{-5} kgm^{-1}s^{-1}$ to $4\ 10^{-5} kgm^{-1}s^{-1}$.

4. Laboratory measurements and in-situ rheological measurements of mud under waves were made by University College of Swansea (Appendix 1). A rheometer, which used the sensitivity of the shear wave velocity to the structure of the bed, was incorporated into a wave flume. Under the action of waves, fluidisation of the upper material occurred, with the bed sediment losing its elastic properties and changing from a viscoelastic material to a shear-thinning non-Newtonian suspension. The weakening effect of repeated cyclical stressing was observed at moderate amplitudes, with complete loss of measurable structure occurring at larger amplitudes. After a recovery period of 2 hours (no waves), the structure of the bed had only recovered about 70%.

5. A bed frame was used to make field measurements of wave parameters at the same time as near-bed cohesive sediment processes. Wave periods of 1.5 to 2.5 seconds were recorded, with wave heights up to 0.2m. The wave period increased as the depth increased over the inter-tidal mudflats, where the frame was deployed. The wave induced shear stress was highest when the water depth was very shallow; however, this was not high enough to erode the consolidated mudflat although it probably eroded all soft unconsolidated deposits on the surface before the bed frame monitoring started.

## 4.2 Recommendations

In-situ rheometry, based on shear wave propagation techniques has been successfully used to determine the dynamical behaviour of mud under uni-directional waves. It is recommended that this technique is used more extensively to cover wider ranges of the wave parameters and the resulting effect. The structure of the mud appears to be one of the most important factors in determining the behaviour of the mud. Other parameters, such as density, are not so useful as tt has been shown that the density can remain unchanged even though the structure has changed.

The bed frame has been used on a number of occasions to measure wave parameters at inter-tidal sites. During the deployments the wave induced shear stresses have not been high enough to erode the consolidated mudflats, although thin layers of soft unconsolidated deposits have been eroded very quickly in the shallow water as the tide rises (and before the monitoring equipment can be started). It is recommended that the bed frame is deployed during a period of high activity when the consolidated mudflat might erode. In addition it would be useful to deploy the bed frame sub-tidally, so that the effects of the high wave shear stresses in very shallow water were not observed.

## 5 ACKNOWLEDGEMENTS

HR Wallingford gratefully acknowledges the rheological measurements of Dr D J A Williams and Dr P R Williams of the Department of Chemical Engineering, University College of Swansea.

HR Wallingford also acknowledges the work of Ing. Ismael Piedra Cueva, who continued the research on wave fluidisation at HR with the aid of grants from the Uruguayan Government and the EEC (DG XII).

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# Tables

TABLE 1 Equations used in calculation of bed shear stress from bottom orbital velocity

Input parameters

| | |
|---|---|
| $U_{rms}$ | Root-mean-square bed orbital velocity for random waves ($ms^{-1}$) |
| $T_z$ | Mean zero crossing period (s) |
| $\nu$ | Viscosity of fluid ($m^2s^{-1}$) |
| $\rho$ | Density of fluid ($kgm^{-3}$) |
| $k_s$ | Nikuradse equivalent grain roughness (m) |

Calculated parameters

| | |
|---|---|
| $U_m$ | Orbital velocity for equivalent monochromatic wave ($ms^{-1}$) |
| A | Semi-orbital excursion length (m) |
| $R_w$ | Wave Reynolds number |
| r | Relative roughness ($A/k_s$) |
| $F_s$ | Smooth friction factor |
| $F_r$ | Rough friction factor |
| $F_w$ | Wave friction factor |
| $\tau$ | Representative bed shear stress for random wave spectrum($Nm^{-2}$) |

Equations used

$$U_m = \sqrt{2}\, U_{rms}$$

$$\tau = 0.5\ \rho\ F_w\ U_m^2$$

$$A = U_m\ T_z\ /\ 2\pi$$

$$R_w = U_m\ A\ /\ \nu$$

$F_s = 2\ R_w^{-0.5}$ if $Rw < 5\ 10^5$ Laminar

$F_s = 0.0521\ R_w^{-0.187}$ if $Rw > 5\ 10^5$ Smooth turbulent

$F_r = 0.3$ if $r < 1.57$ Rough turbulent

$F_r = 0.00251\ \exp(5.21\ r^{-0.19})$ if $r > 1.57$ Rough turbulent

$$F_w = \max(F_s, F_r)$$

TABLE 2 Wave conditions during Fawley and Wentlooge laboratory tests

| Mud | Test no. | Waves | Average bed dry density | Water depth | Wave period (note 1) | | | Wave height (note 2) | | | Urms | | | Um | | | $\tau$(peak) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins |
| | | | kg/m3 | m | seconds | | | metres | | | m/s | | | m/s | | | N/m2 | | |
| Fawley | FM16 | mono | 300 | 0.5 | 1.2 | 1.2 | | 0.098 | 0.120 | | | | | 0.116 | 0.142 | | 0.27 | 0.33 | |
| Fawley | FM17 | mono | 300 | 0.5 | 1.2 | 1.2 | | 0.098 | 0.120 | | | | | 0.116 | 0.142 | | 0.27 | 0.33 | |
| Fawley | FM18 | mono | 300 | 0.5 | 1.2 | 1.2 | 1.2 | 0.050 | 0.088 | 0.110 | | | | 0.059 | 0.104 | 0.130 | 0.14 | 0.24 | 0.30 |
| Fawley | FM19 | mono | 300 | 0.5 | 1.2 | 1.2 | 1.2 | 0.056 | 0.098 | 0.110 | | | | 0.066 | 0.116 | 0.130 | 0.15 | 0.27 | 0.30 |
| Fawley | FM20 | mono | 300 | 0.5 | 1.2 | 1.2 | 1.2 | 0.098 | 0.098 | 0.098 | | | | 0.116 | 0.116 | 0.116 | 0.27 | 0.27 | 0.27 |
| Fawley | FM21 | mono | 290 | 0.5 | 1.2 | 1.2 | 1.2 | 0.098 | 0.098 | 0.098 | | | | 0.116 | 0.116 | 0.116 | 0.27 | 0.27 | 0.27 |
| Fawley | FM22 | mono | 300 | 0.5 | no waves | | | no waves | | | | | | - | - | - | - | - | - |
| Fawley | FM23 | mono | 290 | 0.5 | 1.2 | 1.2 | 1.2 | 0.110 | 0.110 | 0.110 | | | | 0.130 | 0.130 | 0.130 | 0.30 | 0.30 | 0.30 |
| Fawley | FM24 | mono | 390 | 0.5 | 1.2 | 1.2 | 1.2 | 0.098 | 0.110 | 0.110 | | | | 0.116 | 0.130 | 0.130 | 0.27 | 0.30 | 0.30 |
| Fawley | FM25 | mono | 390 | 0.5 | 1.2 | 1.2 | 1.2 | 0.098 | 0.098 | 0.098 | | | | 0.116 | 0.116 | 0.116 | 0.27 | 0.27 | 0.27 |
| Fawley | FR1 | random | 430 | 0.5 | low | 1.60 | 2.05 | small | 0.078 | 0.082 | - | 0.067 | 0.078 | - | 0.095 | 0.110 | - | 0.19 | 0.20 |
| Fawley | FR2 | random | 420 | 0.5 | 1.05 | 1.18 | 1.78 | 0.035 | 0.042 | 0.042 | 0.021 | 0.029 | 0.038 | 0.030 | 0.041 | 0.054 | 0.08 | 0.10 | 0.10 |
| Fawley | FR3 | random | 320 | 0.5 | 1.15 | 1.20 | 1.90 | 0.033 | 0.042 | 0.040 | 0.022 | 0.030 | 0.037 | 0.032 | 0.042 | 0.052 | 0.08 | 0.10 | 0.10 |
| Fawley | FR4 | random | 330 | 0.3 | 0.88 | 1.06 | 1.12 | 0.027 | 0.031 | 0.035 | 0.023 | 0.030 | 0.035 | 0.033 | 0.043 | 0.050 | 0.09 | 0.11 | 0.12 |
| Fawley | FR5 | random | 310 | 0.3 | 1.06 | 1.02 | 1.08 | 0.028 | 0.027 | 0.028 | 0.027 | 0.026 | 0.027 | 0.039 | 0.037 | 0.039 | 0.10 | 0.09 | 0.10 |
| Fawley | FR6 | random | 310 | 0.3 | 1.08 | 1.11 | 1.14 | 0.072 | 0.068 | 0.068 | 0.071 | 0.068 | 0.070 | 0.100 | 0.096 | 0.099 | 0.25 | 0.23 | 0.24 |
| Wentlooge | WM1 | mono | 300 | 0.3 | 1.2 | 1.2 | 1.2 | 0.055 | 0.055 | 0.055 | | | | 0.113 | 0.113 | 0.113 | 0.26 | 0.26 | 0.26 |
| Wentlooge | WM2 | mono | 300 | 0.3 | 1.2 | 1.2 | 1.2 | 0.028 | 0.028 | 0.028 | | | | 0.057 | 0.057 | 0.057 | 0.13 | 0.13 | 0.13 |
| Wentlooge | WM3 | mono | 300 | 0.3 | 1.2 | 1.2 | 1.2 | 0.055 | 0.055 | 0.055 | | | | 0.113 | 0.113 | 0.113 | 0.26 | 0.26 | 0.26 |
| Wentlooge | WR1 | random | 290 | 0.3 | 1.08 | 1.08 | 1.11 | 0.073 | 0.075 | 0.071 | 0.072 | 0.073 | 0.071 | 0.101 | 0.103 | 0.100 | 0.25 | 0.25 | 0.24 |
| Wentlooge | WR2 | random | 300 | 0.3 | 1.11 | 1.10 | 1.12 | 0.075 | 0.072 | 0.072 | 0.075 | 0.071 | 0.073 | 0.106 | 0.101 | 0.103 | 0.26 | 0.25 | 0.25 |

Note 1: Wave period constant T for monofrequency waves, zero crossing period Tz for random waves

Note 2: Wave height constant H for monofrequency waves, significant wave height Hs for random waves

TABLE 3 Wave conditions during Poole, Hongkong and Marsden laboratory tests

| Mud | Test no. | Waves | Average bed dry density | Water depth | Wave period (note 1) | | | Wave height (note 2) | | | Urms | | | Um | | | $\tau$ | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins | 0-60 mins | 60-120 mins | 120-180 mins |
| | | | kg/m3 | m | seconds | | | metres | | | m/s | | | m/s | | | N/m2 | | |
| Poole | PW1 | random | 450 | 0.3 | 0.8 | 0.9 | 1.1 | 0.040 | 0.051 | 0.069 | 0.032 | 0.045 | 0.068 | 0.046 | 0.064 | 0.097 | 0.13 | 0.17 | 0.24 |
| Poole | PW2 | random | 475 | 0.3 | 0.8 | 0.9 | 1.0 | 0.045 | 0.050 | 0.065 | 0.036 | 0.044 | 0.063 | 0.051 | 0.063 | 0.090 | 0.14 | 0.17 | 0.22 |
| Poole | PW3 | random | 475 | 0.3 | 0.9 | 0.9 | 1.0 | 0.048 | 0.050 | 0.059 | 0.043 | 0.044 | 0.057 | 0.060 | 0.063 | 0.081 | 0.16 | 0.17 | 0.20 |
| HongKong | HKW1 | random | 350 | 0.4 | 1.2 | 1.2 | 1.2 | 0.067 | 0.093 | 0.118 | 0.056 | 0.078 | 0.099 | 0.080 | 0.111 | 0.140 | 0.19 | 0.26 | 0.33 |
| HongKong | HKW2 | random | 350 | 0.45 | 1.2 | 1.2 | 1.2 | 0.082 | 0.101 | 0.113 | 0.064 | 0.079 | 0.088 | 0.091 | 0.112 | 0.125 | 0.21 | 0.26 | 0.29 |
| HongKong | HKW3 | random | 350 | 0.3 | 1.2 | 1.2 | 1.2 | 0.054 | 0.076 | 0.088 | 0.057 | 0.080 | 0.094 | 0.083 | 0.116 | 0.135 | 0.19 | 0.27 | 0.31 |
| Marsden | MW1 | mono | 540 | 0.3 | 1.2 | 1.2 | 1.2 | 0.053 | 0.069 | 0.080 | | | | 0.109 | 0.141 | 0.164 | 0.25 | 0.32 | 0.38 |
| Marsden | MW2 | mono | 530 | 0.3 | 1.2 | 1.2 | 1.2 | 0.055 | 0.080 | 0.103 | | | | 0.113 | 0.164 | 0.211 | 0.26 | 0.38 | 0.48 |

Note 1: Wave period constant T for monofrequency waves, zero crossing period Tz for random waves

Note 2: Wave height constant H for monofrequency waves, significant wave height Hs for random waves

TABLE 4 Erosion rates in laboratory wave tests

| Mud | Waves | Average bed dry density | Water depth | $\tau$ (peak) | $\tau$ critical | Erosion rate (maximum) from turbidity measurements |
|---|---|---|---|---|---|---|
| | | kg/m3 | m | N/m2 | N/m2 | kg/m2/s |
| Fawley 16-23 | mono | 300 | 0.5 | 0.27 - 0.33 | 0.1 - 0.14 | 0.00010 - 0.00013 |
| Fawley 24-25 | mono | 400 | 0.5 | 0.27 - 0.30 | | |
| Fawley FR6 | random | 310 | 0.3 | 0.23 - 0.25 | < 0.08 | 0.00004 - 0.00006 |
| Wentlooge | mono | 300 | 0.3 | 0.26 | | 0.0001 - 0.0003 |
| Wentlooge | random | 300 | 0.3 | 0.24 - 0.26 | | 0.0001 - 0.0004 |
| Poole | random | 475 | 0.3 | 0.13 - 0.20 | 0.14 | 0.00001 - 0.00005 |
| HongKong | random | 350 | 0.3-0.45 | 0.19 - 0.33 | 0.15 | 0.00008 - 0.0001 |
| Marsden | mono | 530 | 0.3 | 0.25 - 0.48 | 0.1 - 0.4 | 0.00003 |

TABLE 5 Summary of bed frame deployments

| Date | Time of HW | Height of HW (m) | Velocity maxima 0.1m (m/s) | Velocity maxima 0.5m (m/s) | Water Depth max (m) | Suspended concentration ($kgm^{-3}$) 0.1m max | 0.1m min | 0.5m max | 0.5m min | 1.0m max | 1.0m min | Shear stress maximum $\tau_p$ ($Nm^{-2}$) | Bed level change (mm) | Wave characteristics Hs max (m) | Hs min (m) | Tz max (s) | Tz min (s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| West Usk deployments | | | | | | | | | | | | | | | | | |
| 29/11/89 | 1947 | 12.2 | - | - | - | - | - | - | - | | | - | - | - | - | - | - |
| 30/11/89 | 0804 | 12.2 | 0.21 | 0.25 | 2.31 | 0.500 | 0.200 | 0.300 | 0.100 | | | - | 0 | - | - | - | - |
| 30/11/89 | 2019 | 12.0 | - | - | - | - | - | - | - | | | - | - | - | - | - | - |
| 01/12/89 | 0837 | 12.0 | - | - | - | - | - | - | - | | | - | - | - | - | - | - |
| 24/04/90 | 1838 | 13.4 | 0.30 | 0.36 | 3.90 | - | - | - | - | - | - | 0.35 | 2.5 | - | - | - | - |
| 25/04/90 | 0700 | 13.7 | 0.28 | 0.36 | 3.91 | - | - | - | - | - | - | 0.36 | 1.5 | 0.05 | 0.03 | 3.1 | 1.6 |
| 25/04/90 | 1923 | 13.7 | 0.30 | 0.38 | 3.90 | - | - | - | - | - | - | 0.35 | - | - | - | - | - |
| 26/04/90 | 0747 | 13.9 | 0.29 | 0.38 | 3.90 | 0.800 | 0.500 | 0.500 | 0.100 | - | - | 0.30 | - | 0.17 | 0.08 | 2.9 | 2.1 |
| 27/04/90 | 0832 | 13.7 | 0.30 | 0.44 | 3.92 | 0.600 | 0.200 | 0.430 | 0.100 | 0.200 | 0.100 | 0.40 | - | 0.16 | 0.10 | 3.3 | 1.9 |
| 23/05/90 | 1812 | 13.0 | 0.30 | 0.21 | 2.90 | 0.500 | 0.380 | 0.200 | 0.150 | 0.100 | 0.500 | 0.21 | - | 0.80 | 0.05 | 2.7 | 1.5 |
| 24/05/90 | 0639 | 13.2 | 0.32 | 0.24 | 3.30 | 0.510 | 0.400 | 0.200 | 0.150 | 0.150 | 0.100 | 0.27 | 16 | 0.70 | 0.04 | 2.6 | 1.5 |
| 24/05/90 | 1903 | 13.3 | 0.31 | 0.24 | 3.40 | 0.700 | 0.500 | 0.50 | 0 150 | 0.100 | 0.500 | 0.26 | -2 | 0.15 | 0.10 | 2.7 | 1.5 |
| 25/05/90 | 0730 | 13.2 | 0.29 | 0.22 | 3.50 | 1.100 | 0.600 | 0.110 | 0.020 | 0.100 | 0.150 | 0.23 | 12 | 0.10 | 0.05 | 2.7 | 1.5 |
| 08/08/90 | 2030 | 13.1 | 0.32 | 0.39 | - | 0.700 | 0.400 | 0.200 | 0.100 | 0.150 | 0.500 | 0.65 | -5 | - | - | - | - |
| 09/08/90 | 0849 | 13.0 | 0.28 | 0.35 | - | 0.800 | 0.600 | - | - | 0.200 | 0.100 | 0.16 | 0 | - | - | - | - |
| 09/08/90 | 2107 | 13.3 | 0.26 | 0.35 | - | 0.800 | 0.600 | 0.200 | 0.100 | 0.100 | 0.050 | 0.36 | - | - | - | - | - |
| 10/08/90 | 0925 | 13.1 | 0.26 | 0.36 | - | 0.800 | 0.600 | 0.200 | 0.100 | 0.200 | 0.100 | 0.17 | 0.15 | - | - | - | - |
| 03/10/90 | 1818 | 13.0 | 0.25 | 0.16 | - | 1.100 | 0.500 | 0.500 | 0.300 | 0.300 | 0.150 | 0.23 | - | - | - | - | - |
| 04/10/90 | 0641 | 13.1 | 0.27 | 0.17 | - | 1.450 | 0.700 | 0.600 | 0.350 | 0.350 | 0.100 | 0.25 | - | - | - | - | - |
| 04/10/90 | 1900 | 13.6 | 0.30 | 0.22 | - | 1.750 | 0.500 | 1.550 | 0.400 | 0.500 | 0.100 | 0.28 | 1.5 | - | - | - | - |
| 05/10/90 | 0721 | 13.5 | 0.31 | 0.21 | 3.30 | 1.800 | 0.300 | 1.850 | 0.450 | 1.750 | 0.050 | 0.23 | 2.5 | 0.38 | 0.27 | 2.3 | 3.3 |
| Eastham Docks deployments | | | | | | | | | | | | | | | | | |
| 20/11/90 | 0018 | 8.7 | 0.35 | 0.42 | 5.50 | 0.830 | 0.200 | 0.870 | 0.180 | 0.560 | 0.090 | 0.90 | - | - | - | - | - |
| 20/11/90 | 1232 | 8.9 | 0.42 | 0.45 | 5.50 | 0.950 | 0.200 | 0.870 | 0.180 | 0.560 | 0.150 | 0.90 | 23 | - | - | - | - |
| 21/11/90 | 0050 | 8.5 | 0.44 | 0.45 | 5.20 | 0.820 | 0.200 | 0.810 | 0.180 | 0.540 | 0.070 | 0.85 | 25 | - | - | - | - |
| 26/11/90 | 1716 | 7.7 | 0.38 | 0.44 | 3.80 | 0.250 | 0.070 | 0.200 | 0.065 | 0.160 | 0.045 | 0.6 | 11 | - | - | - | - |
| 27/11/90 | 1824 | 7.9 | 0.36 | 0.40 | 4.00 | 0.275 | 0.080 | 0.080 | 0.075 | 0.160 | 0.050 | 0.5 | - | - | - | - | - |

Figures

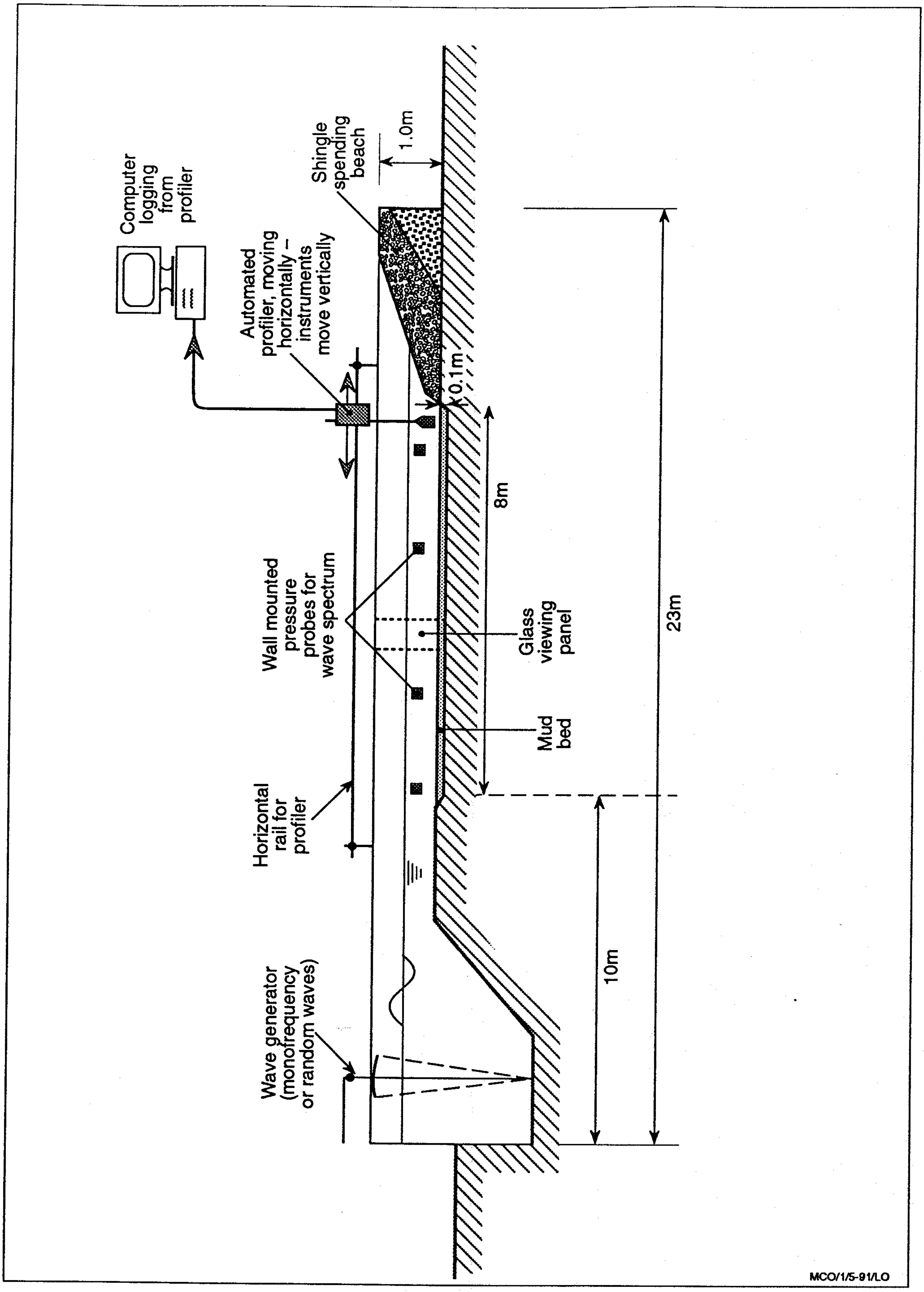


**Fig 1 The wave flume**

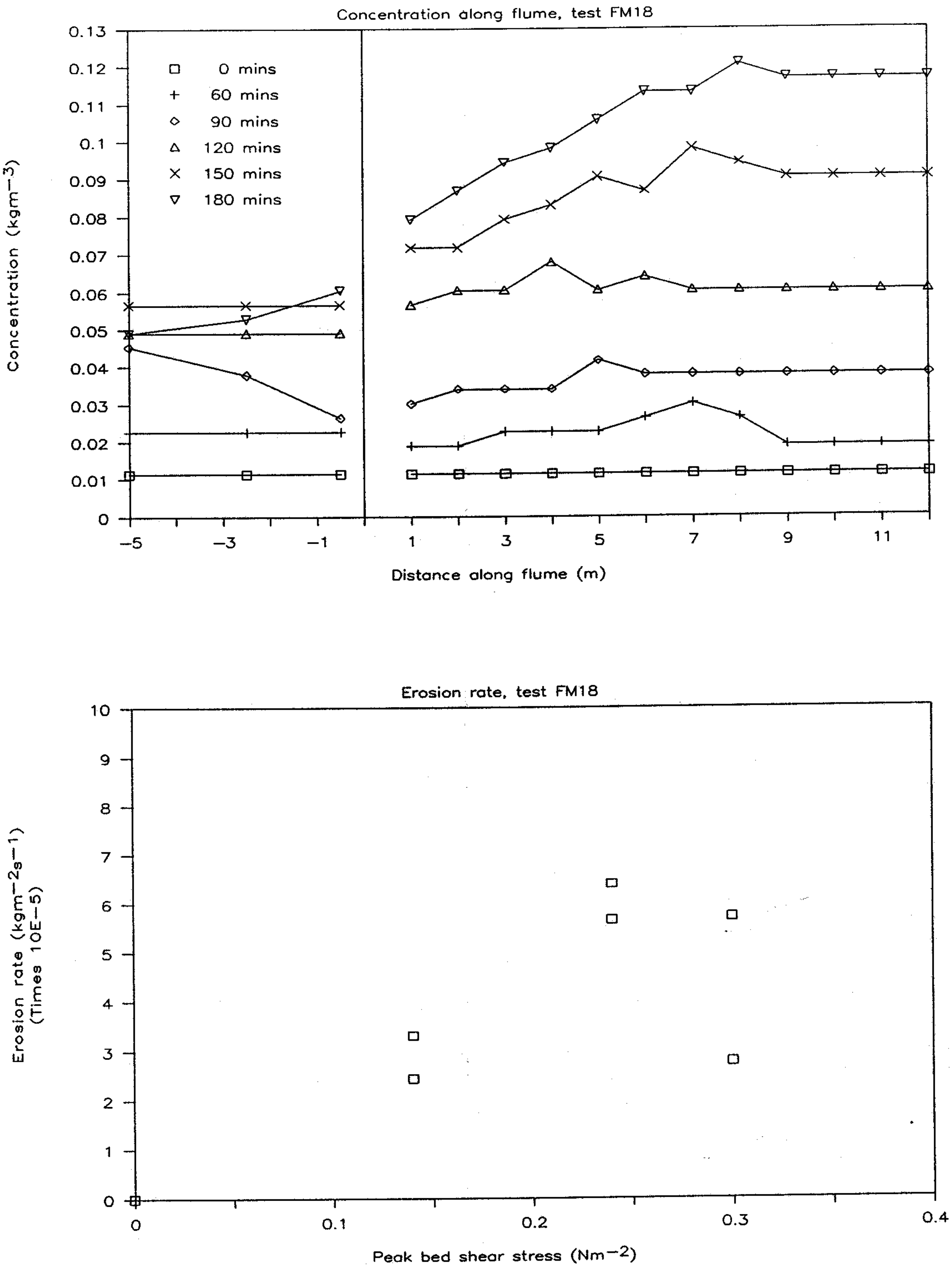


Fig 2 Suspended sediment concentration along flume with time, and erosion rate against peak bed shear stress, Fawley test FM18

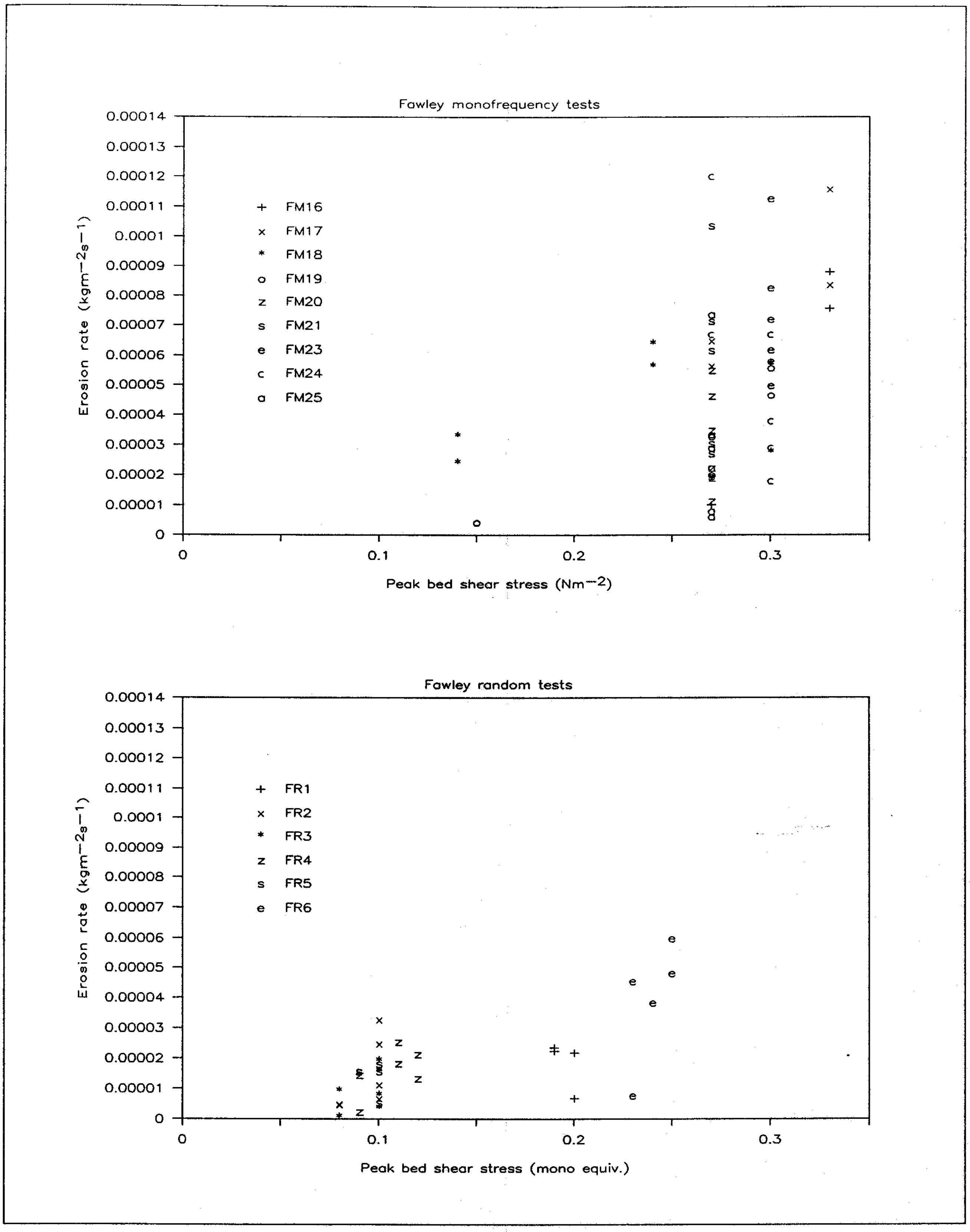


Fig 3 Erosion rate against peak bed shear stress, Fawley monofrequency and random wave tests

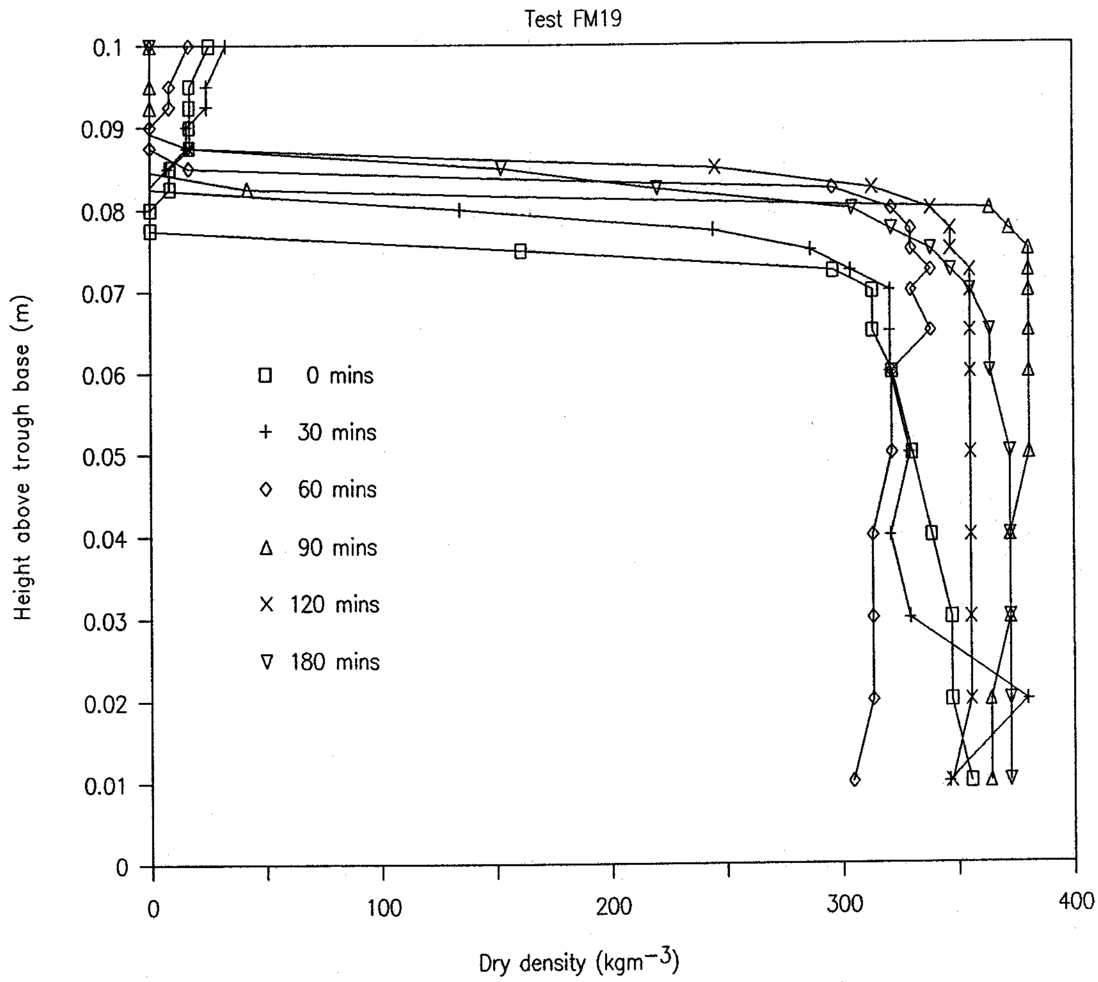


Fig 4 Density profiles during Fawley test FM19

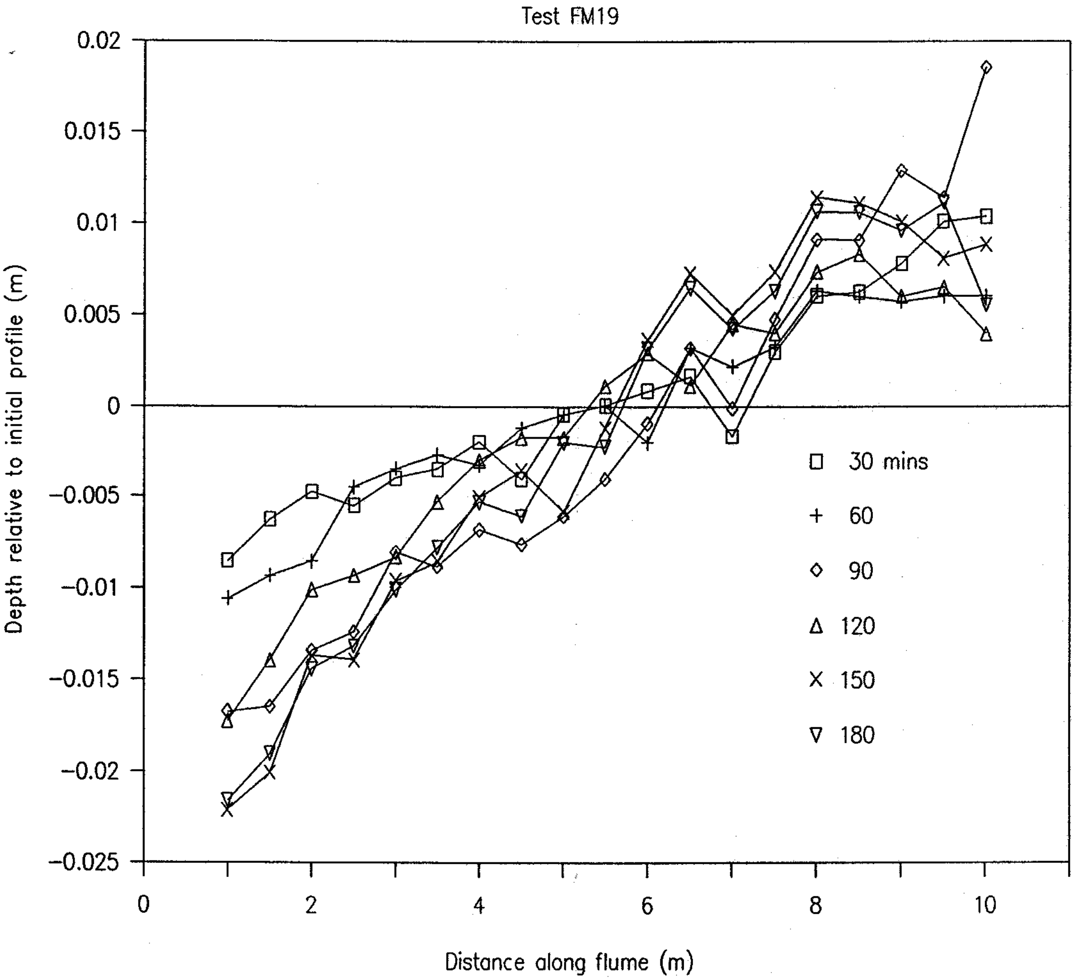


Fig 5 Bed levels along flume during Fawley test FM19

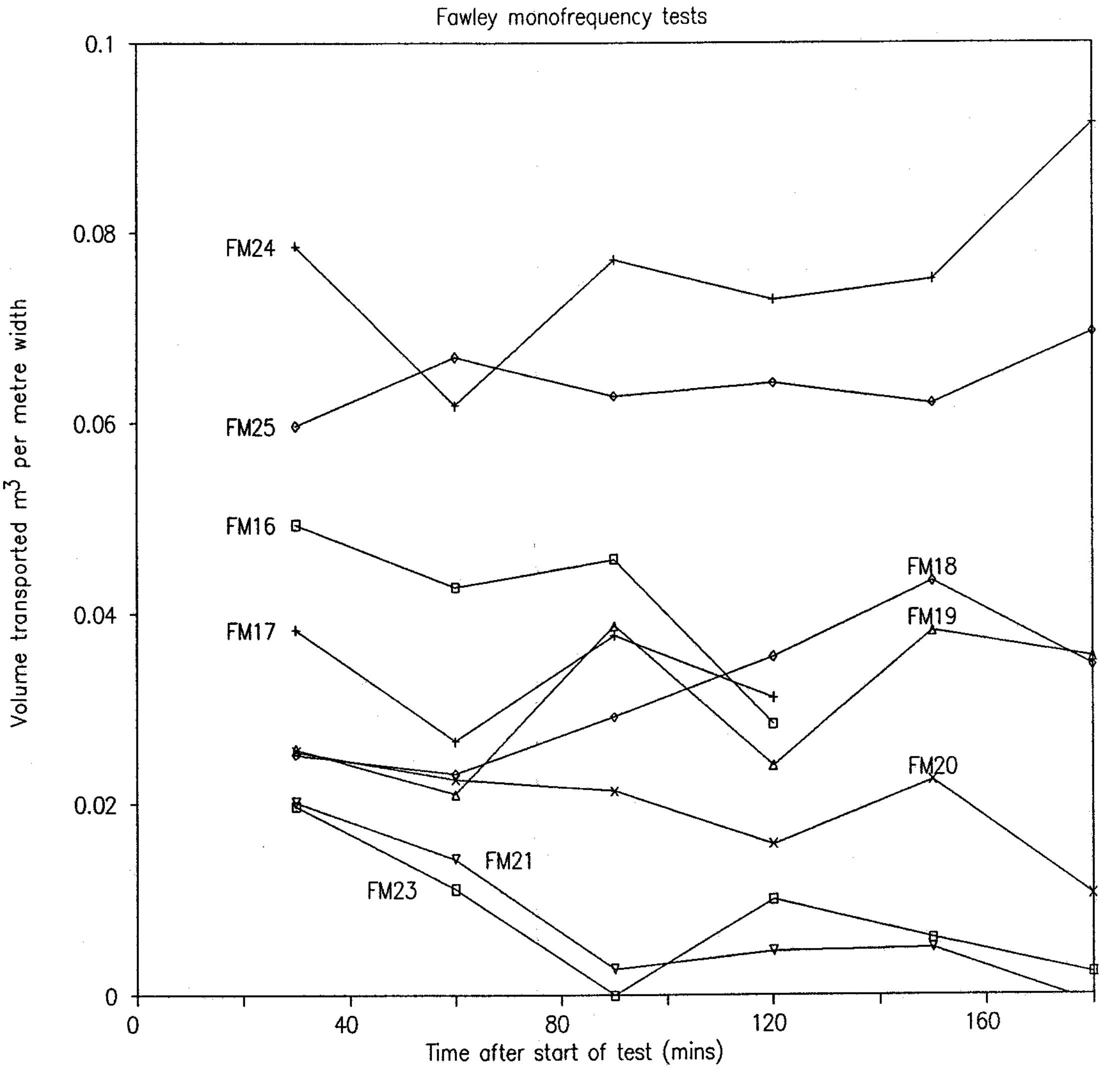


Fig 6 Volume transported during Fawley monofrequency tests

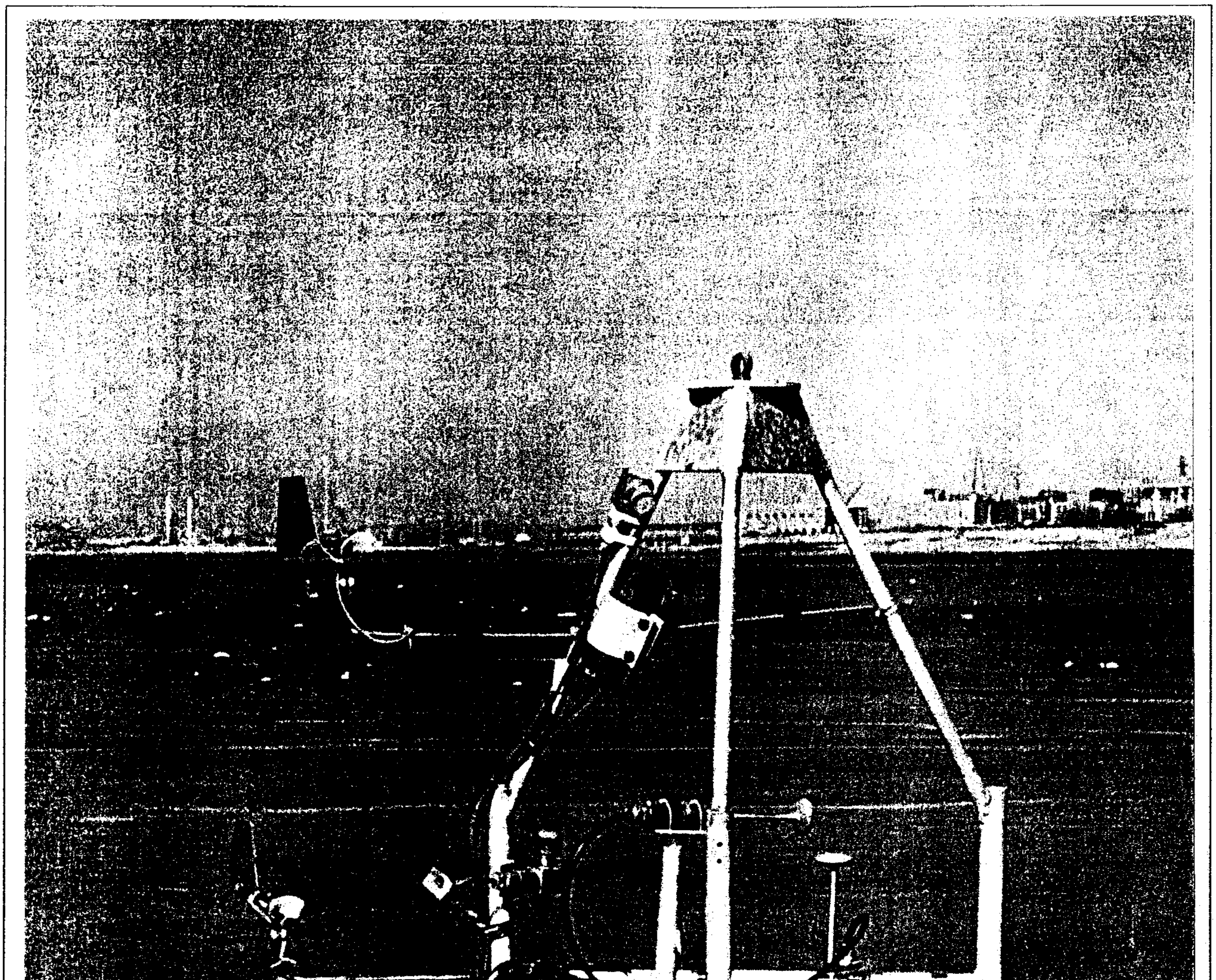

Fig 7    The bed frame

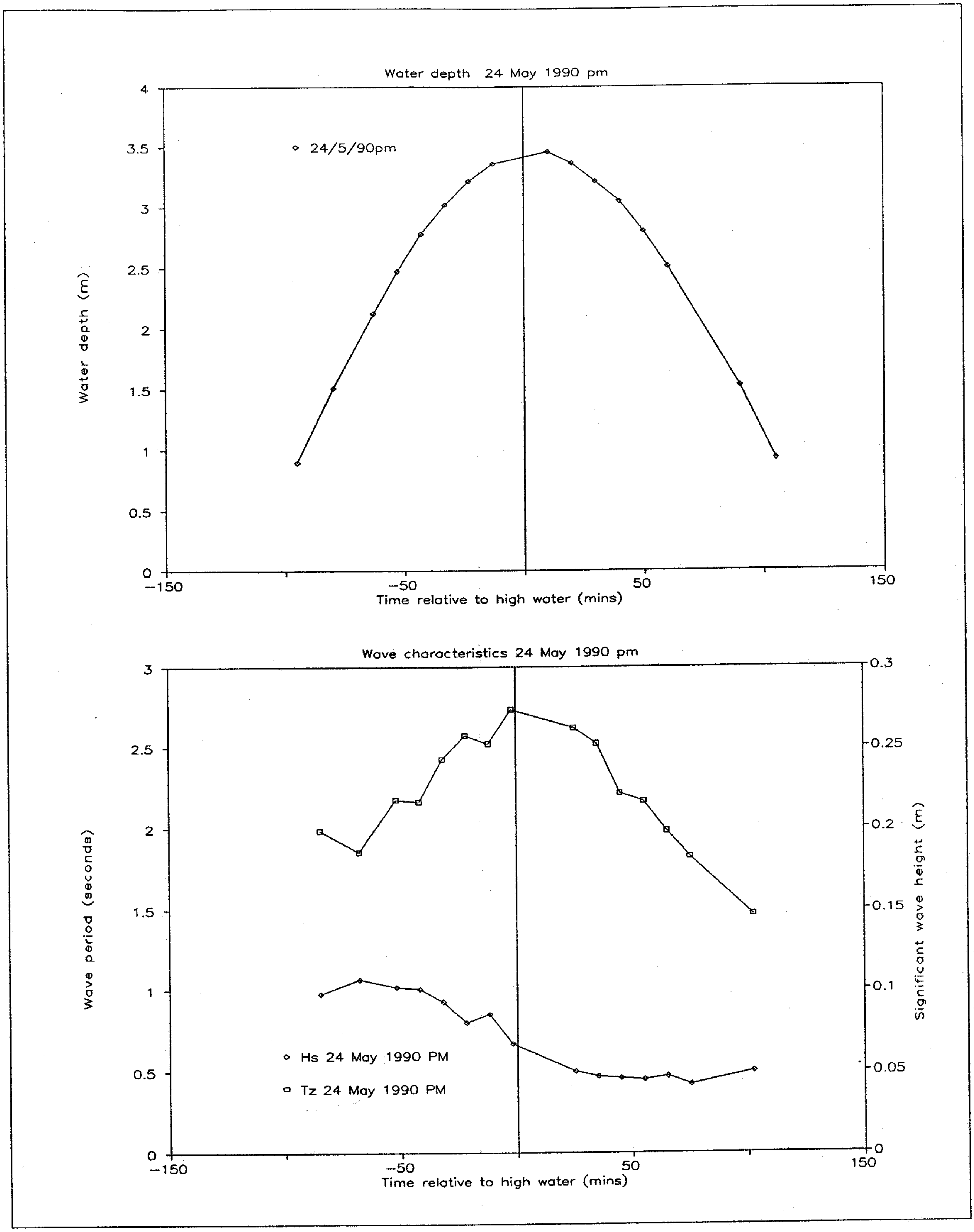


Fig 8 Water depth, wave height and wave period at West Usk field site, 24 May 1990 pm

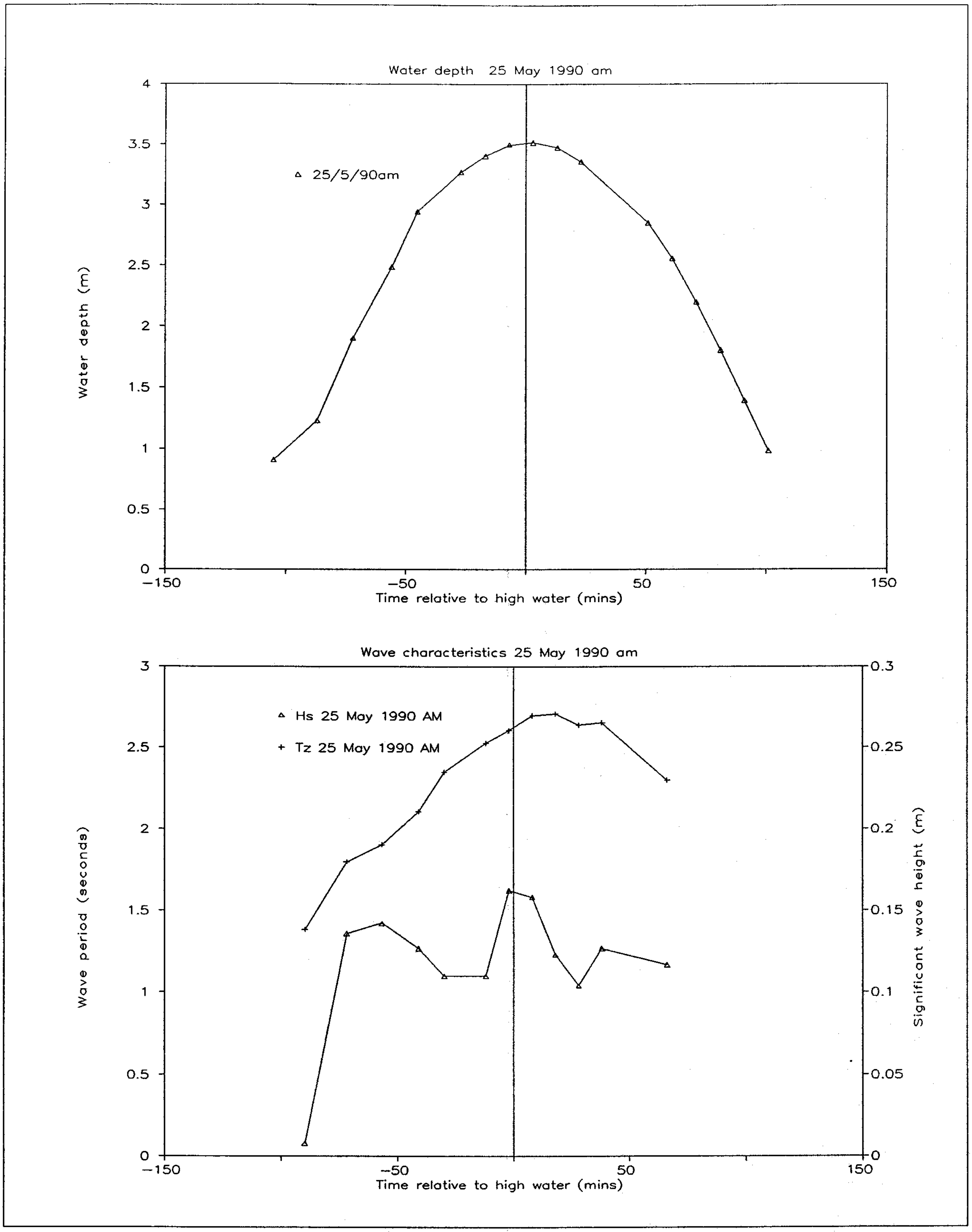


Fig 9 Water depth, wave height and wave period at West Usk field site, 25 May 1990 am

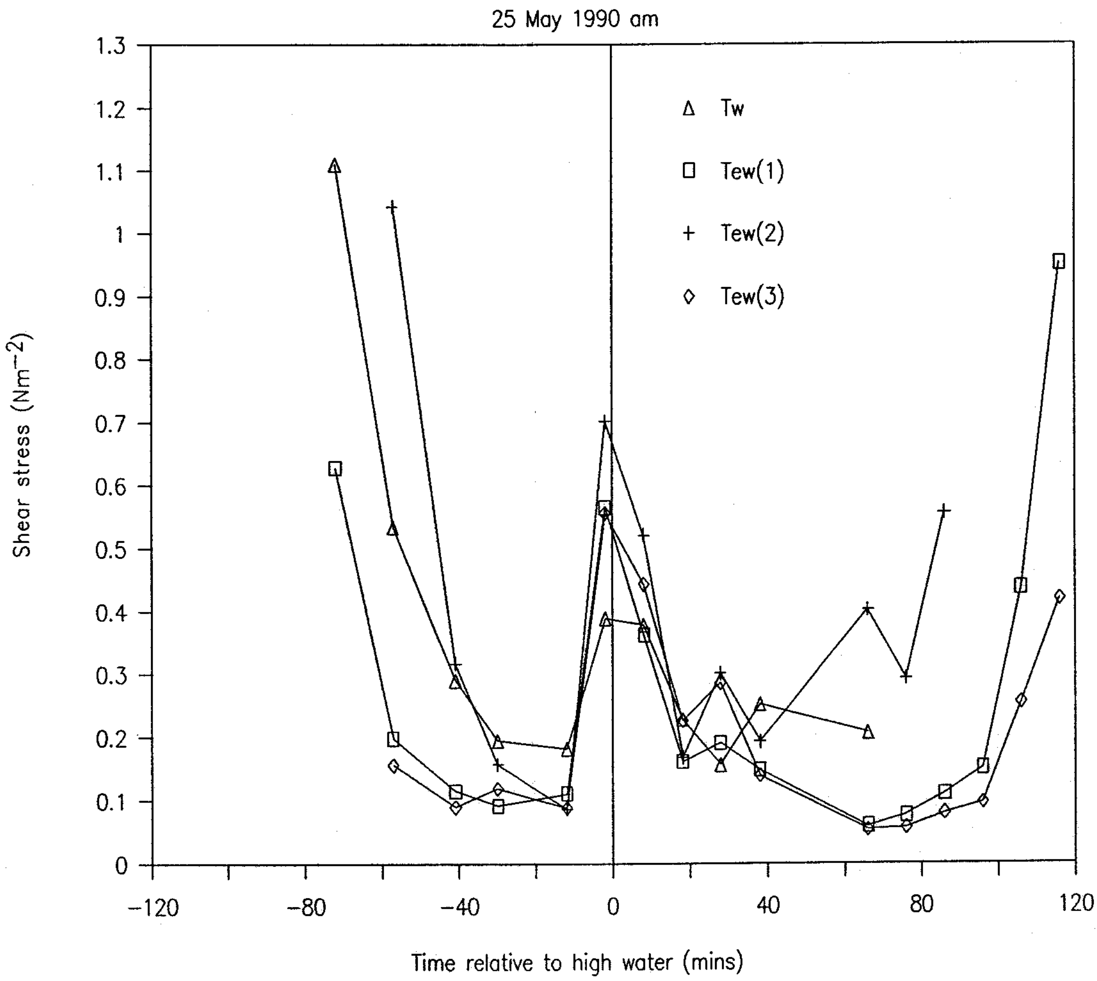


Fig 10 Wave induced bed shear stress at West Usk site, 25 May 1990 am

# Appendix

# RHEOLOGY OF COHESIVE SEDIMENTS UNDER WAVE ACTION

## REPORT


by

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FEBRUARY 1990


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CONTENTS

## 1. Introduction

The significance and importance of understanding the mechanism involved in wave-cohesive sediment interaction in natural aquatic environments is well established , e.g. see Mehta (1988). The rheological behaviour of cohesive sediment under wave action is of particular importance in this context and the elastico-viscous nature of mud is recognised (James et al 1988).

Laboratory rheometry incurs sample disturbance and is in general unable to re-create in-situ stresses with respect to a cohesive sediment bed. Data from such studies may be unrealistic in terms of bed properties. However laboratory rheometry is a useful adjunct to in-situ testing. The present work provides an account of both laboratory measurements and in-situ rheometrical measurements of the mechanical response of cohesive sediment in the presence of uni-directional waves of varying amplitude prior to and during bed deformation and eventual loss of cohesive structure.

The in-situ rheometer is based upon and incorporates developments in shear wave propagation studies at U.C. Swansea. The rheometer utilises the sensitivity of shear wave velocity to the structure (through mechanical rigidity) and changes induced by strain of the bed, e.g. under wave loading. The device has physical dimensions which allow its incorporation into flume experiments while causing minimum disturbance of sediment.

## 2. Experimental

### 2.1 Materials and flume details

#### Materials

The sediment used in flume tests was Parrett estuary mud of grey-green appearance. Inspection of a stock sample of slurry (nominal density "$1300 kg/m^3$" revealed the presence of fibrous material and an attempt was made to remove such by high speed stirring (the fibre becoming entangled in the stirrer shaft and subsequently being removed).

Flume details

The 75 ft wave flume contained a 3.55m long rectangular sediment trap test section of depth 100mm and width 150mm. The in-situ rheometer was placed halfway along the trap and its base plate bonded by silicon adhesive to the trap floor. Cables to facilitate data logging and control of the rheometer were routed through recessed channels in the rheometer's base plate and sealed to the observation window at the side of the trap.

The sediment bed was laid down by pouring the sample slurry around and over the rheometer to a depth of 10cm; the flume was subsequently filled with water. Relevant dimensions are given in fig. (1).

In this preliminary investigation lack of time imposed three major restrictions on experimental methodology:

(1) All shear wave path lengths were preset and fixed at the time of rheometer installation. No attempt was made to recover the rheometer to reset the path lengths and thereby change the instrument's sensitivity.

(2) Complete electrical failure of some transducers occurred shortly after emplacement of the rheometer: thus no data were available from the measuring location at mid-bed.

(3) Equilibrium times between runs and changes of experimental conditions were arbitrarily restricted to 2 hours (maximum) unless left overnight ($\sim$ 17 hrs).

All density profiles and density data including calibrations were provided by Hydraulics Research Ltd., Wallingford.

## 2.2 In-situ rheometry

In-situ rheometry used herein involves the generation and direction of small amplitude plane shear waves by the piezoelectrically stimulated wave generating /detecting surfaces of transducers. Each shear wave transducer is comprised of ceramic 'bimorph' elements bonded to steel plates (length 12 mm: width 4mm: thickness 100 μm).

In the present work three such transducers were mounted in series horizontally with preset relative dispositions and three sets of three transducers were mounted at preset vertical locations above a perspex base plate bonded to the flume test section base.

Each transducer has a dual facility in that shear wave generation and detection is accomplished by an element which simultaneously responds to shear wave arrival and dynamic pressure changes within the sediment bed (induced by uni-directional water wave motion). The transducers thereby afford direct study of strain induced variation of elastic and dissipative bed properties under a range of pre-erosional dynamic conditions.

Measurement of shear wave velocity and attenuation

Shear wave velocities ($V_s$) and attenuation coefficients ($\alpha$) were measured over preset wave path lengths of 4mm at three vertical locations throughout a sediment bed of depth 100mm, at levels 23mm, 53mm and 80mm above the flume test section base.

The test frequencies employed were between 900 Hz and 1.6 kHz at shear strain amplitudes ($\gamma$) within the linear viscoelastic region at each vertical location (i.e. $\gamma < \gamma_c$ where $\gamma_c$ is the critical strain at the upper limit of linear viscoelastic response). Linearity was tested by employing the excitation voltage amplitude (hence wave generator displacement amplitude) ranges which produced no measurable change in $V_s$.

Shear wave frequency and displacement amplitude of the wave generating surfaces were controlled by a Phillips PM 5133 function generator using continuous and tone burst (8 cycle) sinusoidal output up to a maximum of 20 volts. Data logging and signal processing was by twin channel notch filter with 40 dB gain per channel and 12-bit analog to digital converter and PC-AT microcomputer. Sampling frequency was 25 kHz (per channel).

The wave rigidity modulus G was obtained using

$$G = \rho_B V_s^2 \tag{1}$$

where $\rho_B$ is the bulk density of the material. This is a simplified form of the following expression for $G^1$ the dynamic rigidity (see Ferry 1980)

$$G^1 = \frac{\rho V_s^2 (1-r^2)}{(1 + r^2)^2} \qquad \text{...........(2)}$$

where $r = \frac{\lambda\alpha}{2\pi}$ and $\lambda$ is the shear wavelength. Attenuation was assessed by driving adjacent pairs of transducers in 8-cycle bursts and noting the variation of measured shear wave amplitude with distances. For the measurements reported herein $\lambda\alpha < 1$ (typically in range 0.1 to 0.4) and therefore $G \approx \rho_B V_s^2$ as may be expected for $\gamma \ll \gamma_c$ and high frequencies.

All reported $V_s$ values are related to the instantaneous peak pressure at each vertical location (under water wave action) in the bed by recording the pressure induced modulation of the shear wave signals.

### 2.3 Laboratory rheological tests

All laboratory measurements were made on mud samples withdrawn from the same bulk samples used in flume tests. Measurements of G were made using a modified RANK Pulse Shearometer (operating frequency ca. 220 Hz); sample volume was 50 ml with path lengths up to 3 cm.

Additional laboratory tests were performed using a CARRI-MED CSL controlled stress rheometer to provide dynamic moduli from oscillatory shear tests. The test geometry employed was a 4cm diameter parallel plate system with gap widths 2mm and 4mm. All tests were performed at $20^\circ$C.

## 3. Flume tests

Results are presented in two ways, these being analogous to two widely used rheometrical test procedures and are intended to elucidate the dynamical bed response to

a) water waves of constant frequency and amplitude (to study the effect of time (number of cycles of loading) on the distributed rheological properties), and

b) water waves of constant frequency but varying (gradually increasing) amplitude.

((a) and (b) may be combined by studying the temporal evolution of rheological response in (b) at each successive water wave amplitude).

### 3.1 Results

In a series of tests (runs 1-7) the water wave frequency was 0.83 Hz and the water wave generator displacement setting was 10cm.

Before beginning the tests the bed was allowed to consolidate overnight (in total, 17 hours).

The density profile obtained for the bed prior to run 1 is shown in fig. (5), curve (a), taken 10cm upstream of the rheometer location. The results obtained from location #1 (uppermost) and #3 (nearest flume base) are presented (as $V_s^2$ against elapsed time of experiment following start-up of water wave) in fig. (2). Note that for a constant bulk density of homogeneous bed material the variation of $V_s^2$ would directly represent the variation of the wave rigidity modulus (assuming negligible dispersion between 900 Hz and 1.6 kHz).

At the upper measuring location, #1, (2cm below the bed surface at the beginning of the experiment) the shear wave velocity is seen to decline linearly with number of cycles of loading from $\sim$ 11 $ms^{-1}$ to $\sim$ 3 $ms^{-1}$ following an indunction period of $\sim$ 500 seconds. If, for the sake of illustration, it is assumed that $\rho \sim 1400$ kg/m$^3$ (see curve (a), fig. (5)), this represents a decline in rigidity from $\sim$ 170 $kNm^{-2}$ to $\sim$ 12 $kNm^{-2}$.

At the lower measuring location, #3, $V_s$ is independent of the number of cycles of loading for $500 \leq t \leq 1200$ seconds despite the continual reduction in $V_s$ at #1. Subsequently, however, as $V_s \to 0$ at #1 for $t > 1200$ seconds, $V_s$ at #3 declines rapidly from $\sim$ 20 $ms^{-1}$ to $\sim$ 14 $ms^{-1}$. Thereafter a quasi-equilibrium response is obtained over $\sim$ 1200 seconds, $V_s$ eventually declining to $\sim$ 11 $ms^{-1}$.

Note that as $V_s \rightarrow 0$ bed sediment loses its elastic properties and changes from a viscoelastic material with continuous structural rigidity to a shear thinning non-Newtonian suspension, i.e. locally the bed material tends towards a fluidised state (the flocs are fully supported by the fluid).

In terms of estimated rigidity (taking $\rho \sim 1450$ kg/m$^3$) the reduction of rigidity at #3 throughout the tests is from $\sim$ 580 kNm$^{-2}$ to $\sim$ 175 kNm$^{-2}$, i.e. the rigidity at #3 at the end of the experiment has declined to the initial value of the upper level.

The experiments reported above were repeated after a bed recovery time of $\sim$ 2 hours at the same water wave frequency (0.83 Hz) but larger water wave generator displacement setting, 32 cm (runs 8-13). The results are shown in fig. (3).

The initial values of $V_s$ for #1 and #3 are lower than the initial values for runs 1-7 indicating incomplete recovery of bed structure. Comparison of data from #3 shows a recovery of $\sim$ 70%. This weakened material displays a greatly reduced induction period ($\sim$ 600 seconds) concomitant with the decline in $V_s^2$ from 400 to 180 m$^2$s$^{-2}$ for #3. It is noteworthy that at #1, $V_s$ declines after the first few loading cycles in this test and, after 900 seconds no further response is measurable for the preset shear wave path lengths and shear wave amplitudes employed. This coincides with a rapid decline in bed structure at the lower level slowing to a quasi equilibrium behaviour thereafter. Note that significant amounts of bed material were seen suspended in the water column above the test section at times coincident with the rapid decline of upper bed rigidity.

The density profile obtained for the bed following this series of runs is shown in curve (b) of fig. (5) taken 10cm. downstream of the rheometer.

In fig.(4) data are presented from #1 from five runs at constant water wave frequency (0.83 Hz) but varying wave generator displacement, 2 to 35 cm settings. Data Coded $t_o$ is the first measurement of $V_s$ under each changed condition of test and $t_1$, after 600 seconds. It should be noted that this does not imply a test with fully equilibrated bed material. $V_s^2$ is nearly constant for small wave displacement with a rapid decline at large amplitudes; this latter rheological response is typical of concentrated cohesive sediment behaviour at $\gamma > \gamma_c$.

It is interesting to note the considerable weakening effect of repeated cyclical stressing over $\sim$ 500 cycles at moderate amplitudes ($V_s^2$ from $\sim$ 55 to 35 $m^2s^{-2}$). At larger amplitude complete loss of measurable structure occurs. These results in terms of number of stress cycles, wave amplitudes and range of $V_s$ are similar to those obtained in runs 8-13 and imply a return of the upper bed layers to a near equilibrium given sufficient time (overnight in this case).

## 4. Results of laboratory rheological tests

### 4.1 Shear wave propagation tests

Measurements of shear wave velocity and attenuation on 3 aliquots of mud taken from the flume test bed gave $V_s = 4\ ms^{-1}$ ($\pm$ 0.2 $ms^{-1}$) with attenuation coefficients sufficiently low such that G was obtained from equation (1). Thus with $\rho \sim 1300\ kg/m^3$, $G \sim 21 kNm^{-2}$ at 220 Hz and $\gamma \sim 10^{-3}$. This is seen to be within the lower ranges found by in-situ measurements. An equilibration time of 30 minutes was allowed in each test.

### 4.2 Oscillatory shear tests

$G^1$ found from oscillatory shear tests at 0.83 Hz ($\gamma \sim 10^{-3}$) was $\sim$ 2800 $Nm^{-2}$ ($V_s \sim$ 1.5-1.6 $ms^{-1}$ for $\rho \sim 1300\ kg/m^3$). With increasing frequency a gradual increase in $G^1$ was recorded rising to 3500 $Nm^{-2}$ at 10 Hz. An equilibration time of 30 minutes was allowed throughout these tests.

## 5. Discussion

These preliminary flume and laboratory tests have identified several features of interest in the context of the dynamical response of a cohesive sediment bed under uni-directional wave loading.

(i) A pronounced non-homogeneous distribution of (increasing) bed rigidity with depth is found. This increase is greater than may at first be expected from consideration of the bed density profile (a) of fig. (5) due to the highly non-linear dependence of G on solids volume fraction $\phi$ for cohesive sediments. The consequences of this for geotechnical test procedures (e.g. shear vane tests) should be considered.

(ii) At sufficient depth the bed may behave (locally) as a linear visco-elastic material while the overlying material is non-linear and continually weakening. When fluidisation of the upper material occurs lower layers of material show a marked decline in rigidity until a critical depth of overlying fluidised, non-Newtonian suspension is available to modify the downward energy transfer. Under such circumstances a quasi-equilibrium response is seen although it is to be expected that due to progressive straining of the lower layer and thixotropy of the suspension overlayer (s), this will be of finite duration and will be followed by further reduction in bed rigidity.

The results of runs 1-7 and 8-13 can be considered in terms of the evolution of an increasing depth of fluidised material at $\phi < \phi_c$ (where $\phi_c$ is the critical concentration for formation of a 3-dimensional network structure) from higher $\phi$ values at any depth with time under stress. As the depth of fluidised material increases, the lower layers weaken (possibly with quasi-equilibrium periods) and, locally, $\phi \to \phi_c$. For a bed composed of homogeneous material, of varying bulk density, it is to be expected that $V_s^2$ determined at the lowest layer will ultimately pass through the initial $V_s^2$ values possessed by its overlying layers as locally $\phi \to \phi_c$. This is seen to be the case for runs 1-7 and 8-13; note also that $V_s^2$ in fig. (4) at #1 at t = 0 tends to the same value as the ultimate value for #3 in fig. (3).

For linear viscoelastic behaviour it may be anticipated that the evaluation of bed mechanical response at layers overlying #1 and between

#1 and #3 in fig. (2) will be given closely by the data of fig. (3) and thus the data of both sets of runs may be combined to illustrate the evolution of structure throughout the bed. Alternatively the data may be interpreted in terms of the structural change of a single layer given sufficient amplitude and number of cycles of stress.

## 6. Conclusions

(a) Laboratory rheological tests show that Parret sediment ($\sim$ 1300 kg/m$^3$ density) is a relatively weakly structured visco-elastic material ($G^1 < 5 \times 10^3$ N/m$^2$ at low frequencies (< 10 Hz) and moderate strain amplitudes $0(10^{-3})$).

(b) Preliminary flume tests indicate that the dynamical behaviour of this visco-elastic sediment under uni-directional wave loading is determined qualitatively and quantitatively using in-situ rheometry based on shear wave propagation techniques.

## References


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'Viscoelastic Properties of Polymers', 3rd Edition, Wiley.

## Figures

Fig. 1. Experimental arrangements showing position of Bed Rheometer within Flume Test Section.

Fig. 2. Results of runs 1-7 (Variation of $V_s^2$ with elapsed time of experiment). Data from bottom and top of bed shown in curves #3 and #1 respectively. Water wave frequency is 0.83 Hz, small wave amplitude.

Fig. 3. Results of runs 8-13 (as in Fig. 2). Water wave frequency is 0.83 Hz, large wave amplitude.

Fig. 4. Results of runs at constant water wave frequency (0.83 Hz) and a range of water wave amplitude. Results are shown for readings taken at 0 and 600 sec for a range of wave amplitudes. All data obtained at location #1.

Fig. 5. Bed density profiles before and after flume tests.

FIGURE (1)

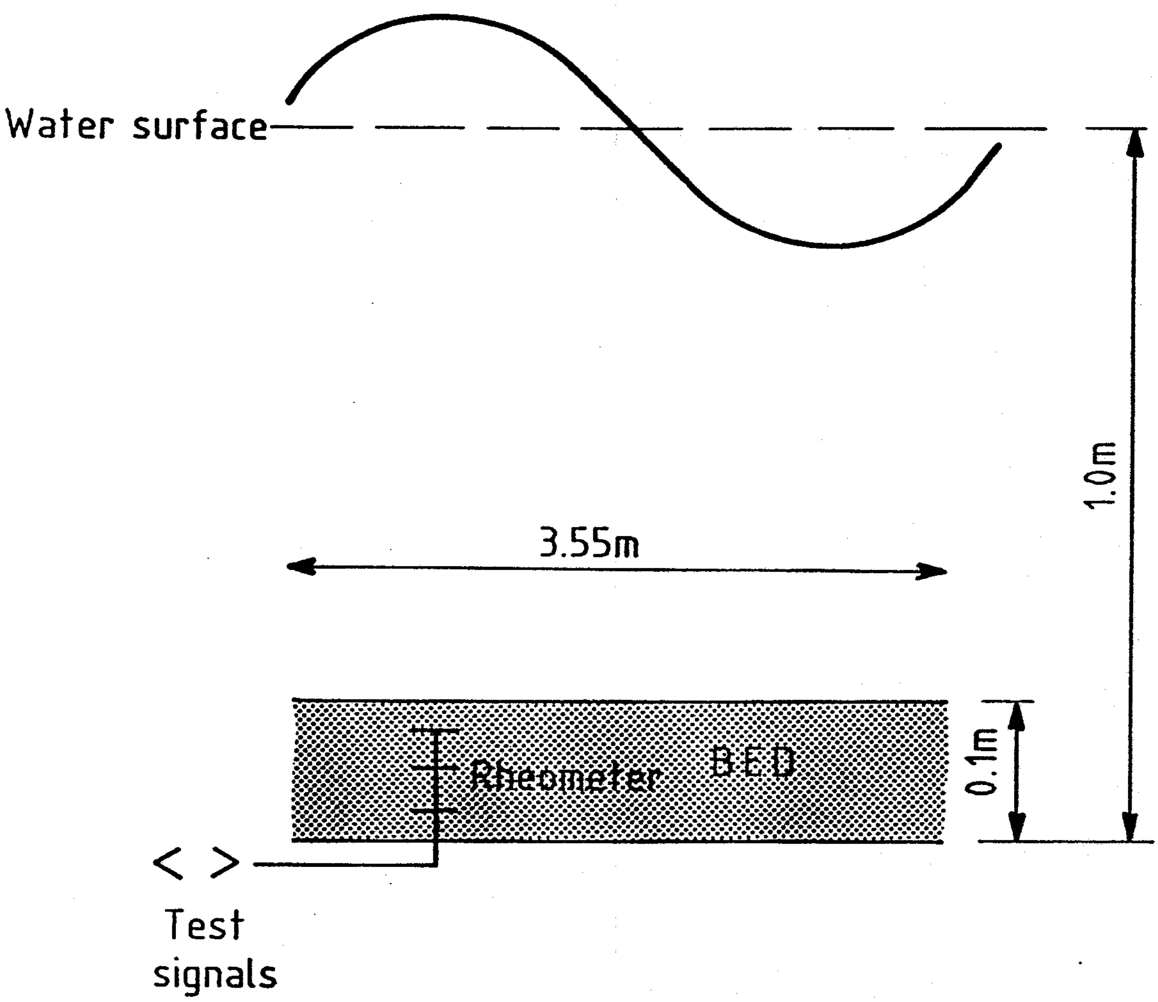


FIGURE (2)

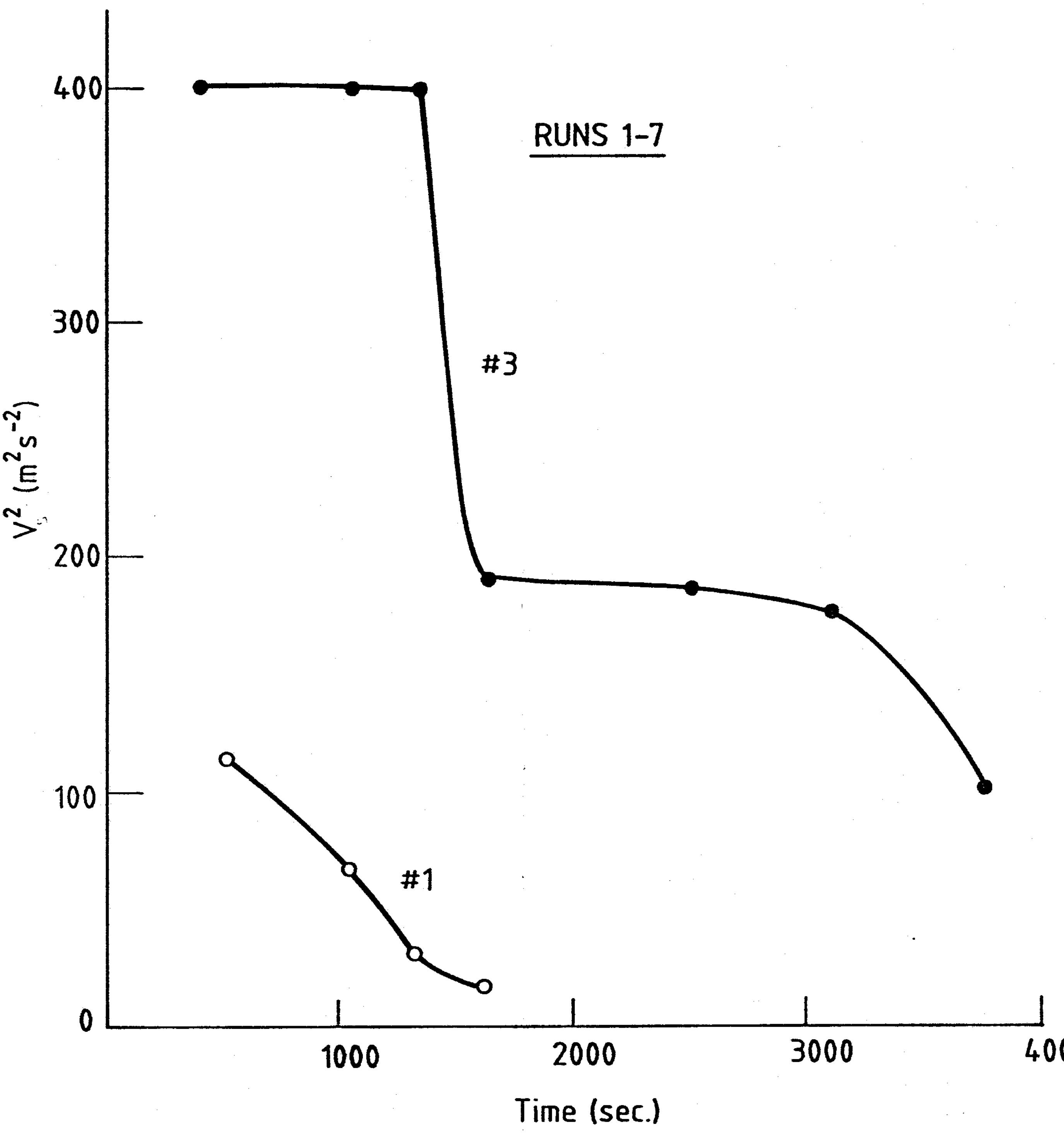

RUNS 1-7
#3
#1
400
300
200
100
0
1000
2000
3000
400
$V_s^2$ $(m^2s^{-2})$
Time (sec.)

FIGURE (3)

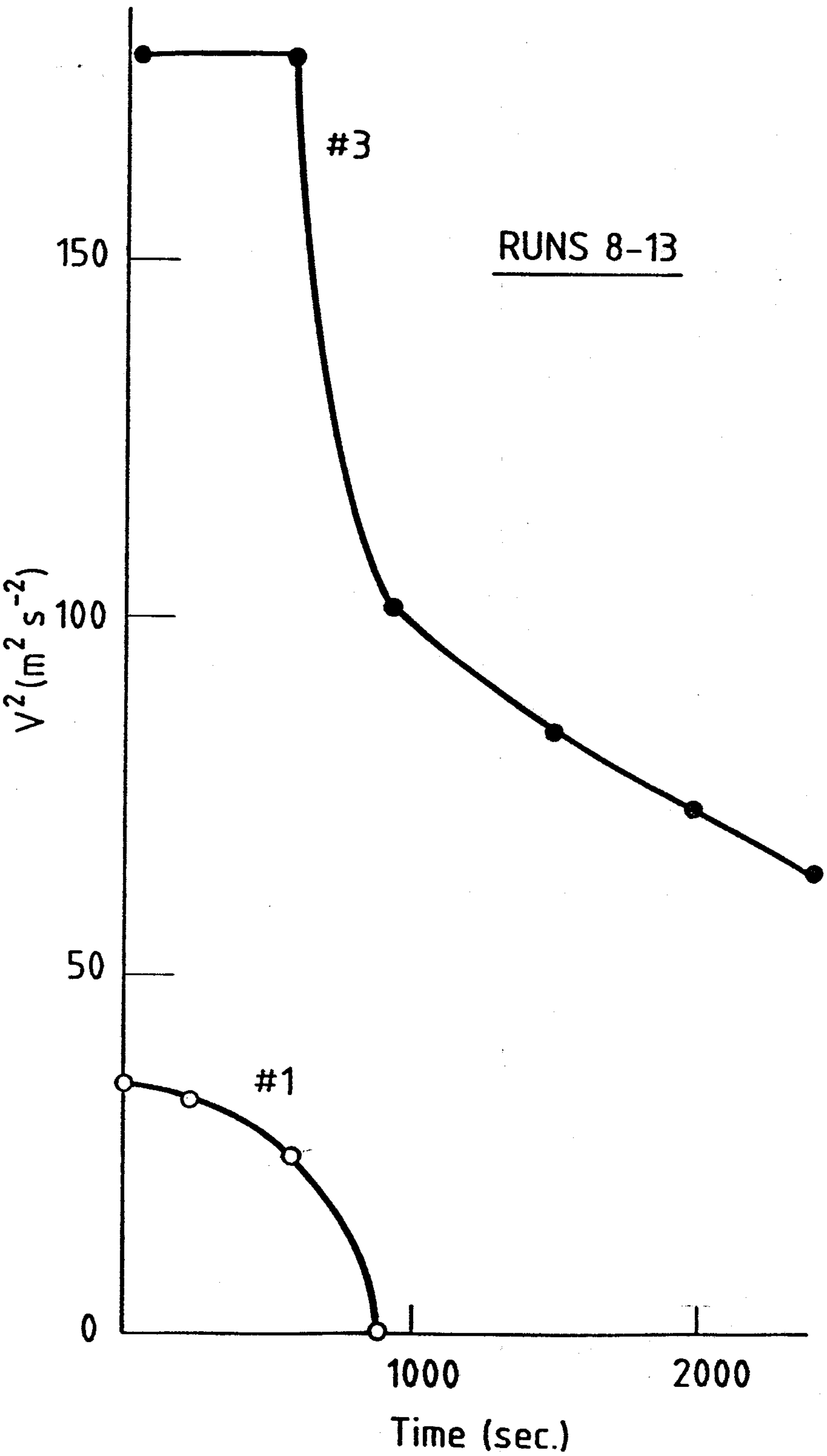
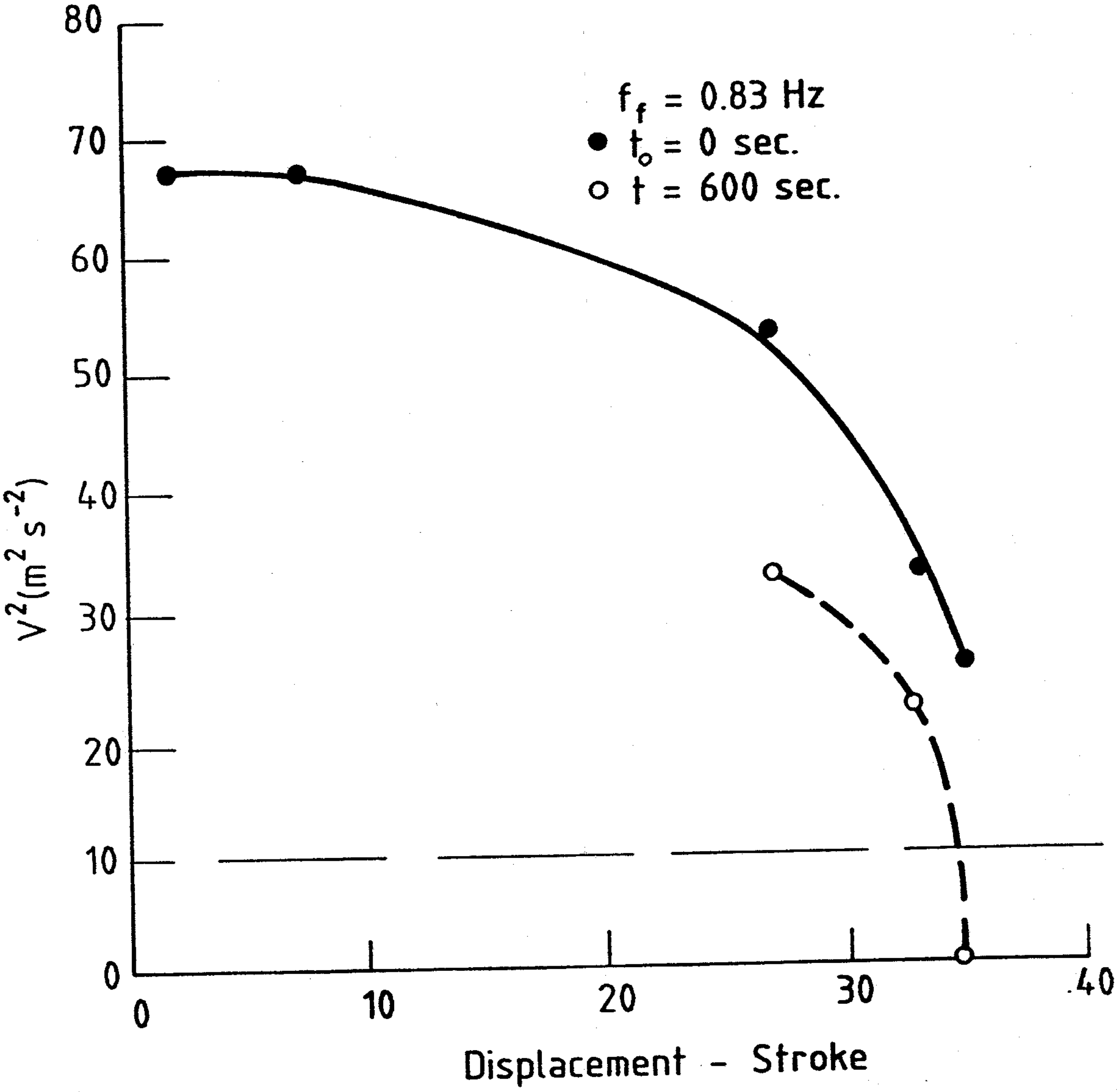
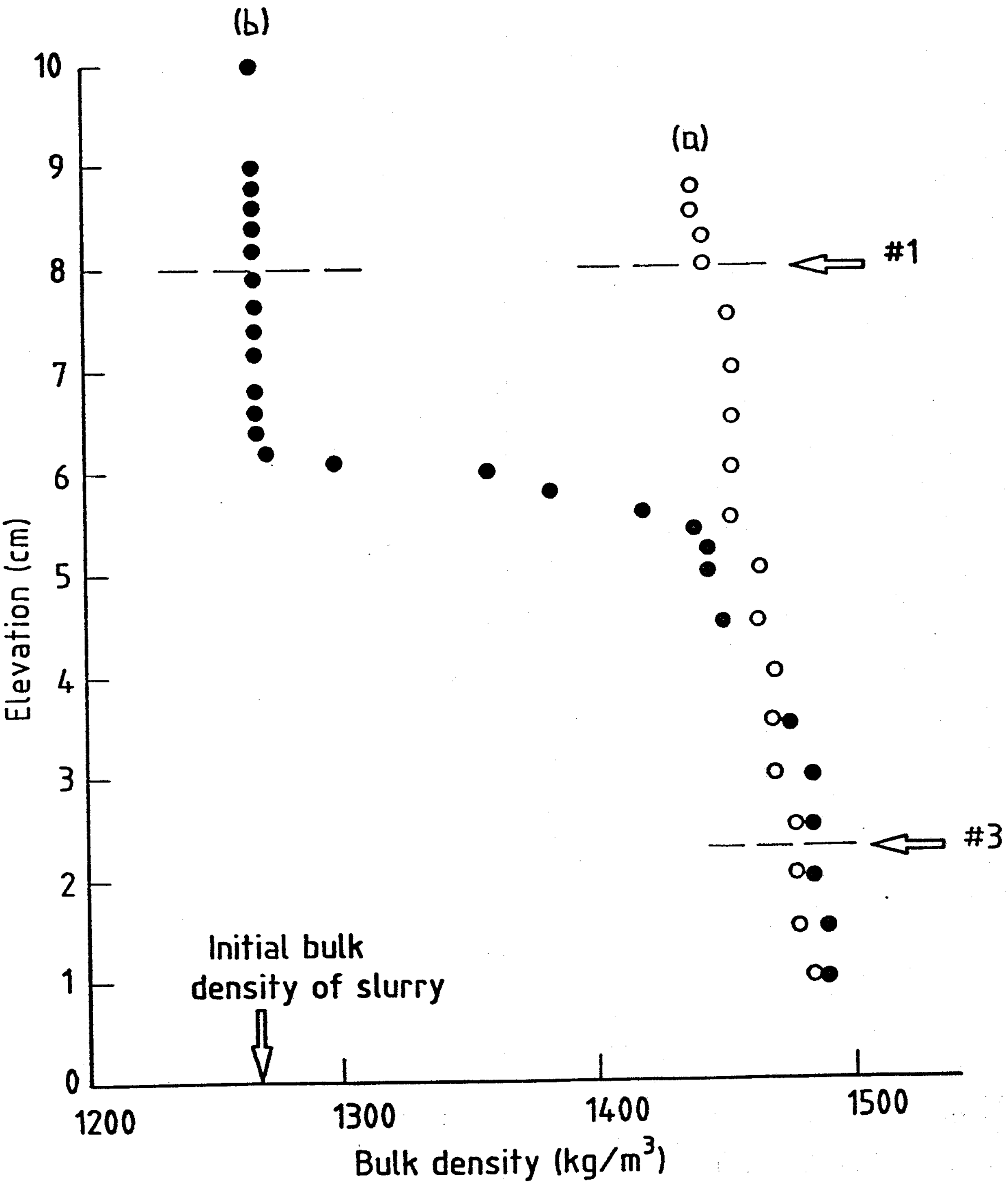


Fig. 4



80
70
60
50
40
30
20
10
0
$V^2 (m^2 s^{-2})$
$f_f$ = 0.83 Hz
$t_o$ = 0 sec.
t = 600 sec.
0
10
20
30
40
Displacement - Stroke

FIGURE (5)


(b)
(a)
#1
#3
Initial bulk
density of slurry
Elevation (cm)
Bulk density (kg/m³)
10
9
8
7
6
5
4
3
2
1
0
1200
1300
1400
1500