HR Wallingford

CHANNEL PROTECTION
TURBULENCE DOWNSTREAM OF STRUCTURES

Interim Report

Report SR 265
June 1991

Address: **Hydraulics Research Ltd,** Wallingford, Oxfordshire OX10 8BA, United Kingdom.
Telephone: 0491 35381 International + 44 491 35381 Telex: 848552 HRSWAL G.
Facsimile: 0491 32233 International + 44 491 32233 Registered in England No. 1622174

This report describes work funded by the Department of the Environment under Research Contract No. PECD 7/6/163. It is published on behalf of the Department of the Environment, but any opinions expressed in this report are not necessarily those of the funding Department. The work was carried out in the Research Department of HR Wallingford headed by Dr W R White. The nominated project officers were Dr R Thorogood for DoE and Dr W R White for HR Wallingford.

ABSTRACT

A survey carried out by HR Wallingford for CIRIA had shown that, despite a considerable amount of past research on channel protection materials, and in particular on riprap, the available methods gave widely-varying predictions. An extensive literature review on riprap sizing formulae is presented in this interim report as well as some general notions on turbulence generated downstream of hydraulic structures. It was found that the existing guidelines do not apply to highly turbulent flows and that the nominal stone size given by the different equations can vary as much as four times. This refers both to normal turbulent flows ie. flows in natural, straight channels and to highly turbulent flows, ie. downstream of structures. In terms of weight the predictions vary by a factor of up to 64. Therefore any uncertainties may have major economic consequences.

The experimental set-up, and the data acquisition procedure are described in this report as well as the materials selected for the study and the preliminary tests already carried out. The next stages of the project which will include bed and bank revetment stability, performance of filters and alternative materials to riprap are also indicated in the report.

LIST OF SYMBOLS

A	Coefficient in Jansen's equation
B	Crest length
B_1	Coefficient in PIANC's equation
C	Coefficient in Maynord's equation
C_1	Coefficient in Izbash's equation
D	Nominal particle size
Dn	Size of the equivalent cube ($= \left(\frac{W}{g\rho s}\right)^{1/3}$)
D_s	Diameter of the equivalent sphere ($= \left(\frac{6\ W}{\pi\rho s g}\right)^{1/3}$)
D_x	Dimension of stone which exceeds dimension of x% of the stones by weight
Fr	Froude number of flow ($= U/(gy)^{0.5}$)
g	Acceleration due to gravity
H	Crest height
h	Height of a point (x,y,z) above a horizontal datum
k	Constant
k_s	Nikuradse's roughness height
p	Pressure
R	Correlation coefficient
r	Relative turbulence intensity in Pilarczyk's equation
s	Specific gravity of stone
t	Time
U	Mean flow velocity in the channel
u	Streamwise velocity component
V	Instantaneous velocity
V_b	Velocity near the bed
V_{cr}	Local flow velocity at the threshold of movement
v	Cross stream velocity component
W	Weight
W_x	Weight of stone greater than that of stones in x% of the mixture by weight
w	Vertical velocity component
y	Flow depth
y_o	Total flow depth
α	Bank slope
γ	Coefficient in Pilarczyk's equation
γ_r	High turbulence factor in Pilarczyk's equation
ν	Kinematic viscosity
ρ	Density of water
ρ_o	Angle in the equation of the Department of Transport of State of California
σ	Standard deviation
τ	Shear stress
ϕ	Internal friction angle of the stone
Ψ_s	Shields parameter ($= \frac{\tau}{\rho g\ (s-1)D}$)
Ω	Factor for reduced stability of stones on banks

$'$	Fluctuation around the mean
$\overline{x}$	Time-averaged value of quantity x
rms	Root mean square value

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$$

CONTENTS

FIGURES

1 INTRODUCTION

Flows downstream of structures such as gates, weirs and stilling basins can be highly turbulent and the velocity distributions very non-uniform. Channel protection is therefore normally required to prevent, or at least limit, the extent of scour produced by the flow on the river bed and banks. Riprap is one of the most widely used forms of flexible protection for natural and artificial watercourses. However, it was found from a survey carried out recently by HR Wallingford for CIRIA that, despite a considerable amount of past research, many of the available design methods give widely-varying predictions of stable stone sizes. Such uncertainties can have major economic consequences: a typical difference of 30% in predicted stone size can increase the weight of the stone by a factor of 2.2.

The objectives of this research project were:

1 to carry out experimental work on riprap stability in high turbulence flows downstream of hydraulic structures, and

2 to identify and compare possible low-cost alternatives to riprap for channel protection.

The ultimate aim of this study was to produce guidelines for the design of stable protection of channels. This could be achieved by developing rational design formulae that relate stable stone sizes to the local flow conditions and the degree of turbulence. Existing guidelines such as the ones given by the US Bureau of Reclamation are based on limited data and do not take quantitative account of highly turbulent flows. Current research being carried out by the US Army Corps of Engineers is concerned with the use of riprap for protection of stream banks against current attack; their work is applicable where uniform flow conditions in a channel determine the velocities around its perimeter.

It was also the purpose of this study to assess the effect that the grading of the material has on its resistance and to recommend suitable filters for riprap protection. Where suitable supplies of stone are not available, the use of riprap for channel protection can be prohibitive. Although several low-cost alternatives have been tried (eg semi-permeable groynes, baffles and concrete blocks), no rational criteria for their design have been developed so far. Basic research work was also needed in this area to determine their suitability and compare their performance with riprap.

2 TURBULENCE

2.1 General concepts

Turbulence can be described as a process where the energy of an 'orderly' steady flow is converted into the random kinetic energy of eddies of decreasing sizes down to the molecular level. At this level the energy is transferred in the form of heat (Yuen and Fraser, 1979). The fluid particles move in extremely irregular paths producing instantaneous changes in the velocity direction and intensity. Due to the random nature of turbulent flows it is usual to consider the instantaneous velocity V (and other quantities such as the pressure) as the sum of two terms:

$$V = \bar{V} + V' \tag{1}$$

where

$\overline{V}$ is the time-averaged velocity responsible for the transport of fluid particles and
V' represents turbulent fluctuations around the mean.

The turbulent fluctuations introduce considerable additional shear stresses by increasing the momentum exchange rate when compared with laminar flow. As the flow paths are so erratic in turbulent flows, the velocity components in the three orthogonal directions (u, v and w) can assume similar importance. These components figure in the Navier-Stokes (N-S) equations for turbulent flows (see, for example, Tennekes and Lumley, 1972) as can be seen in the following three-dimensional form of the N-S equation in the x direction:

$$\frac{\partial \bar{u}}{\partial t} + \bar{u}\frac{\partial \bar{u}}{\partial x} + \bar{v}\frac{\partial \bar{u}}{\partial y} + \bar{w}\frac{\partial \bar{u}}{\partial z} = -\frac{\partial}{\partial x}\left(\frac{p}{\rho} + gh\right) + \nu\nabla^2\bar{u} - \underbrace{\left[\frac{\partial \overline{u'^2}}{\partial x} + \frac{\partial \overline{u'v'}}{\partial y} + \frac{\partial \overline{u'w'}}{\partial z}\right]}_{\text{additional stress components due to turbulence}} \tag{2}$$

For the definition of these variables refer to the list of symbols at the beginning of this report.

For the study of turbulent flows it is obviously important to assess the role of turbulent velocity fluctuations in relation to the time-averaged velocity, since these fluctuations can be larger than the average value. This can be done by determining the turbulence intensities, defined as:

$$(\overline{u'^2})^{1/2}/\bar{u}\ ,\ (\overline{v'^2})^{1/2}/\bar{u}\ \text{ and }\ (\overline{w'^2})^{1/2}/\bar{u}$$

in the x, y and z direction, respectively. The numerators of these ratios give the standard deviation from the mean and are commonly known as the rms values. In fact, according to the definition, the standard deviation is given by:

$$\sigma = \left[\lim_{T \to \infty} \int_0^T (V-\bar{V})^2 \, dt \right]^{1/2} = (\overline{V'^2})^{1/2} \qquad (3)$$

A correlation coefficient (R) between the x and y directions can be defined as $R = \dfrac{\overline{u'v'}}{(\overline{u'^2}\ \overline{v'^2})^{1/2}}$. Hence, for a specified value of R it is possible to determine $\overline{u'v'}$ provided that the rms values are known. These values can be obtained experimentally.

Turbulent flows are common in most engineering problems and, in particular downstream of hydraulic structures, where high velocity and pressure fluctuations usually impose considerable stress on the channel bed and banks. The type and extent of the protection required mainly depends on the level of turbulence and will be discussed in the next sections.

2.2 Turbulence produced in hydraulic jumps

Considerable research has been carried out to characterise turbulence downstream of structures such as weirs, sluice gates and spillways, where a hydraulic jump is formed to establish the transition from a supercritical to a subcritical flow (see Fig 1). The majority of these studies has been orientated towards the measurement and analysis of pressure forces, induced by the turbulent flow, on concrete slabs of stilling basins. This has been done in order to predict and, if possible, prevent the occurrence of damage in joints of slabs, excessive vibrations and cavitation erosion.

The highly turbulent nature of hydraulic jumps, which is in fact responsible for the dissipation of a considerable part of the energy of the supercritical flow, has been pointed out by Rouse et al (1958), Campbell (1966) and Narayanan (1978), amongst others. Whilst Rouse et al investigated the characteristics of expanding flow in the hydraulic jump, Campbell's study focussed on the protection required for river beds and banks submitted to various levels of turbulence. The levels of turbulence considered were the levels expected downstream of culverts, of small stilling basins and in channels. Small stilling basins were defined as having a length three times the theoretical tailwater depth d_2 or greater, and a design depth equal to d_2. 'Small turbulent basins' was the name given to basins with lengths smaller or equal to 2.5 times d_2 and a tailwater depth less than d_2. The stone weight and equivalent diameters necessary to protect the river bed are given on a chart. This author stresses, however, that these criteria are not suitable for large energy dissipation which should be studied in physical models. Narayanan (1978) analysed pressure fluctuations beneath submerged jumps to determine their rms values and their frequency distribution. More recently, a two dimensional numerical model, developed by McCorquodale and Khalifa (1983) to predict the internal structure of the hydraulic jump, demonstrated once again the importance of turbulent pressures on the configuration of such jumps. Neglecting turbulent pressure fluctuations was found to affect the geometric features of high Froude number jumps.

3 CHANNEL PROTECTION - PREVIOUS STUDIES

3.1 Initiation of particle movement

The initiation of particle movement can be taken as the beginning of the failure process of a river protection revetment. A shear stress is exerted on the bottom of the channel as a result of the water current action and determines the slope of the vertical velocity profile along the depth of the flow. Lift and drag forces are therefore present in this process. In turbulent flows the magnitude, direction and point of application of these forces are random quantities, fluctuating around their mean values. Even the laminar sublayer, normally considered to be dominated by viscosity, is affected by high energy eddies coming from the main turbulent flow. These generate 3-D high- and low-speed velocity bursts in the laminar sublayer (Raudkivi, 1990). On the other hand, the main flow is also influenced by the burst of low momentum fluid coming from the sublayer. This contributes to a local deceleration of the flow and generates more eddies. Rock protection revetments can start to move not only due to the shear force produced by the primary water current but also to the impulse drag exerted by a passing eddy or to a local decrease in pressure which generates uplift forces.

A number of factors can influence the initiation of particle motion, some of them due to the geotechnical characteristics of the rock, some to the layout of the revetment and others to the hydraulic features of the flow. The first group may include the size, the specific weight, the surface roughness, the gradation and the porosity of the rockfill. The particle shape, defined by a suitable shape factor, may also be included in this group. Some tests have shown that flatter stones have a lower threshold velocity than standard quarry stone. However, tests performed at the Delft Hydraulics Laboratory, The Netherlands, with coarse particles showed no direct relationship between shape and threshold velocity for particles with the same nominal size (Pilarczyk, 1984). The effect of the gradation seems to be pronounced only for wide particle gradations : the finer particles are eroded first by the flow thus leaving a layer of coarser grains which prevents further scour. Associated with the gradation is the range of porosities that can be achieved for a particular rockfill. It seems obvious that the higher the degree of compaction (ie the lower the porosity) the higher is the rock stability. However, no systematic studies are known to have been carried out on this topic. One possible reason for this is that river protection downstream of hydraulic structures is normally done by dumping riprap on the river bed, so no mechanical compaction takes place.

3.2 Riprap design formulae

Since riprap is undoubtedly the most common material used as river bed protection, several guidelines on grading have been developed over the years based on experience as well as on common sense. These design criteria normally refer to the gradation in terms of stone weight rather than its dimension, to the thickness of the riprap blanket and to the ratio between the maximum and minimum dimension of each block. An example of the lower and upper limits for grading riprap is given by Hemphill and Bramley (1989):

$$W_{100}/W_{50} = 2 \text{ to } 5$$

$$W_{85}/W_{50} = 1.7 \text{ to } 3.3$$

$$W_{15}/W_{50} = 0.1 \text{ to } 0.4 \quad (4)$$

$$W_{85}/W_{15} = 4 \text{ to } 12$$

where W_x is the weight of the stone that is greater than that of x% of the stones by weight. Angular shaped stones are preferred to round stones because of increased stability, and the maximum dimension of each particle should not exceed three times the minimum dimension. Regarding the thickness of the riprap blanket, it can be taken to be at least 1 to 1.5 times the maximum diameter of the largest stones or twice the average diameter (Keown et al, 1977).

Raudkivi (1990) suggests the following simple relationship* as a first approach to sizing riprap protection on horizontal beds:

$$D_s = 0.0413\ V_b^2 \quad (5)$$

where

V_b = velocity near the bed, and
D_s = diameter of the equivalent sphere of specific gravity 2.65.

This author also developed a relationship combining the Manning-Strickler formulae with the Shields threshold criteria for unidirectional flow with the Shields parameter equal to 0.04 and specific gravity of stones of 2.65:

* All equations given in this report are in SI units unless otherwise stated (eg the dimensions of the stone diameter are in metres and the flow velocity in m/s).

$$D = 4.5 \times 10^{-3}\ U^3/y_o^{0.5} \tag{6}$$

where

D = nominal size of the stone
y_o = flow depth, and
U = mean flow velocity

Peterka (1964) combining existing equations, laboratory results and prototype observations produced a curve for sizing riprap downstream of stilling basins. The curve is said to give a good estimate of the size of most of the stones in a well graded mixture. His results can assume the following mathematical form (with a correlation coefficient of 0.99999):

$$D = 0.0376\ V_b^2 \tag{7}$$

where

D = stone diameter, and
V_b = bottom velocity

Peterka points out, however, that the curve is only tentative and therefore liable to modification resulting from further tests or more extensive field observations.

For the design of bank riprap subjected to currents moving parallel to the banks, Searcy (1967) recommends the use of two charts adapted from the Hydraulic Design Criteria, US Corps of Engineers (see Fig 2). One of the charts allows the conversion of the average velocity in the channel into the velocity at stone level. This velocity is entered in the second chart which will then give the equivalent spherical diameter (or weight) of stone for various bank slopes. This trial and error method suggests to use 0.4 of the total depth when the flow depth is greater than 10ft. The resulting stone size is then considered to be stable not only at the toe of the bank but also closer to the water surface. However, the transition between these two different procedures is not absolutely clear.

The Department of Transportation of the State of California (1970) recommends the use of the following expression for the design of rock armour in slopes under current attack (note that this equation is in ft-s units):

$$W = \frac{0.00002\, V_b^6\, s\, \mathrm{cosec}^3\, (\rho_0-\alpha)}{(s-1)^3} \qquad (8)$$

where

W = minimum weight in lb of outside stone for no damage

V_b = stream velocity in ft/s to which the bank is exposed

s = specific gravity of stone

ρ_0 = 70° for randomly placed rubble

α = face slope

Where no accurate velocity data are available V_b can be taken as 2/3 of the average stream velocity for parallel flow tangential to bank; for impinging flow against curved banks V_b can be taken as 4/3 of the average stream velocity.

An experimental study of riprap stability in decelerating flow was carried out by Maynord (1978) using stone sizes with D_{50} between 7.9 and 11.3mm, a bottom slope of 0.008 and various bank slopes. The following relationship was obtained:

$$D_{50}/y_o = CFr^3 \tag{9}$$

where

y_o = water depth

Fr = Froude number of flow = $U/(gy_o)^{0.5}$

U = mean channel velocity

g = acceleration due to gravity

C = coefficient dependent on the channel geometry (straight or curved) and on location of riprap (bottom or slope). Different factors of safety can also be included in this coefficient.

For straight channels and bottom riprap, incipient motion conditions led to C = 0.22. Maynord pointed out that in decelerating flows intense and irregular vorticity is generated which can resemble the turbulence downstream of a hydraulic structure. Hence, the values of C refer to relatively high levels of turbulence. However, it should be noted that the experimental procedure used by this author only produced the additional turbulence associated with expansion in decelerating flows.

Based on studies of river closure by transverse dumping of rock, Izbash and Khaldre (1970) developed a relationship which can be used not only for 'normal' turbulence flows but also for flow downstream of hydraulic structures such as culverts. The diameter of the equivalent spheres $D_{s50} = \left(\frac{6\ W_{50}}{\pi\ \rho\ s}\right)^{1/3}$ can be found using:

$$D_{s50} = C_1 \frac{V_b^2}{g(s-1)\Omega} \tag{10}$$

where

V_b = local flow velocity at the threshold of movement

s = specific weight of stone

C_1 = coefficient variable with the level of turbulence

= 0.35 low turbulence (ie normal river flow)

= 0.68 no fully developed turbulent boundary layer (ie higher turbulence levels)

Ω = factor that allows for the reduced stability of particles on a sloping bank

$$\Omega = \left(1 - \frac{\sin^2\alpha}{\sin^2\phi}\right)^{1/2}$$

where α is the bank slope, and

ϕ is the internal friction angle of the stone.

A similar equation for riprap sizing is suggested by Jansen et al (1979), also taking into account the level of turbulence in the flow but this time in terms of the mean flow velocity U:

$$D_s > \frac{A}{(s-1)} \quad \frac{U^2}{2g} \quad \frac{1}{1 - \dfrac{\sin^2\alpha}{\sin^2\phi}} \qquad (11)$$

Where D_s is the diameter of spherical particles and all the other symbols have the same meaning as in Izbash's formula. Based on investigations carried out by the US Bureau of Reclamation, Jansen et al recommend the following values for A:

A = 0.2 minor turbulence

A = 0.5-0.7 normal turbulence

A = 1.4 major turbulence

Using the Shields critical velocity approach, Pilarczyk (in PIANC, 1987) produced a formula which also takes into account the level of turbulence. This formula, however, was developed only for turbulence levels as high as the ones generated by bends:

$$\frac{Dn_{50}}{y_o} = \left[\frac{U}{B_1 [g(s-1)\Omega \Psi_s y_o]^{0.5}} \right]^{2.5} \tag{12}$$

where

Dn_{50} = size of equivalent cubes (>1mm ; non-cohesive)
U = critical flow velocity
Ω = as defined before ($\Omega = (1 - \frac{\sin^2\alpha}{\sin^2\phi})^{0.5}$)
s = specific weight of stone
y_o = depth of flow at the toe of the banks
Ψ_s = Shields parameter
- Ψ_s = 0.03 no movement
- Ψ_s = 0.04 start of instability
- Ψ_s = 0.06 movement

B_1 = coefficient dependent on the turbulence level in the channel
- B_1 = 8-10 minor turbulence (eg uniform flow, smooth bed, laboratory flumes)
- B_1 = 7-8 normal turbulence of rivers and channels
- B_1 = 5-6 major turbulence (eg outer bends, local disturbances)

The grain diameter Dn_{50} is defined as $\left(\frac{W_{50}}{\rho s}\right)^{1/3}$ where W_{50} represents the weight of the stone that is greater than that of 50% of the stones by weight; ρ is the fluid specific gravity and s the stone specific gravity. Blocks of any type with dimensions greater than the one given by the above equation would withstand currents up to approximately 4m/s. Stability of these blocks would not be guaranteed in areas of high turbulence where uplift forces may occur. Pilarczyk (1984) also recommends a general stability formula, valid for stones with specific gravity between 2.6 and 2.7:

$$D = \gamma \frac{V_{cr}^{3}}{y_o^{0.5}} \tag{13}$$

where

D = equivalent diameter of the average weight of stones W_{50}

V_{cr} = critical velocity (believed to be equivalent to the mean velocity U)

y_o = water depth

γ = numerical coefficient

- γ = 0.005 - horizontal bottom with no bed roughness discontinuity and uniform flow (limited stone transport)
- γ = 0.010 - bottom protection for limited stone transport, construction phases of a dam or sill with B/H > 5 (where B is crest length and H crest height)
- γ = 0.015 - bottom protection for absolute rest of stone or a sill with B/H < 5.

This method suggests that the value of the critical velocity be reduced by a factor γ_r to account for the high turbulence such as that generated in hydraulic jumps. This factor is given by:

$$\gamma_r = \frac{1.45}{1+3r}$$

where

r represents the relative turbulence intensity and can take the values r = 0.15 for uniform flow over a rough bed

r = 0.3 to 0.35 immediately downstream of stilling basins.

A precise definition of the relative turbulence intensity r is not given by Pilarczyk but it can be seen that a value of r of 15% results in $\gamma_r = 1$. Values of r above 15% correspond to turbulent conditions superimposed on the "normal" turbulence of natural streams. It seems reasonable to assume that a value of r equal to 0.15 corresponds to an rms of the velocity fluctuation of 15% of the mean.

The relationship between the bottom velocity and the mean velocity for a rough turbulent flow can be obtained by the following equation (Rouse, 1950):

$$V_b/U = \frac{1}{0.68 \log (y_o/k_s) + 0.71} \tag{14}$$

where

V_b = velocity near the bed
U = mean flow velocity
y_o = water depth
k_s = Nikuradse's roughness height

Uncertainty normally arises when trying to estimate the value of k_s in terms of a suitable particle size in the above equation. Pilarczyk (in Closure of Tidal Basins, 1984) suggests k_s = 1 to 2 D_{50} for uniform size and k_s = 1 to 2 D_{90} for non-uniform graded sediment. This is supported by Raudiki (1967) who stresses that the value of k_s varies considerably with the actual type/state of the mobile bed. Armouring can occur on natural beds of well-graded material thus increasing the roughness value.

Another relationship between V and U is given by the Waterways Experiment Station - WES - (in Ramos, 1990):

$$V_b/U = \frac{0.71}{0.68 \log (y_o/D_s) + 0.71} \qquad (15)$$

where

D_s = size of the equivalent sphere ($D_s = (\frac{6W}{\pi \rho s})^{1/3}$) and all the other symbols have the same meaning as in equation (14).

Assuming that $k_s = D_s$, it can be seen that this equation differs from equation (14) by a factor of 0.71. However, no apparent justification was found for this discrepancy. Having been derived from the early work on pipe resistance carried out by Nikuradse and by Prandtl, equation 14 seems therefore to be more reliable.

It can be seen from the literature review that most relationships give the nominal stone size, D, proportional to V to the power 2 to 3 (V either being the mean flow velocity or the critical velocity at stone level). The equations where $D \alpha V^2$ are in accordance with Brahms incipient motion formula which gives the critical velocity as $V_{cr} = kW^{1/6}$, where k is an empirical constant and W is the particle weight (see, for example, Raudkivi, 1967).

Since $W \alpha D^3$ it follows that $D \alpha V_{cr}^2$. For high velocity flow conditions it is apparent that inadequate velocity estimates can greatly affect the size of riprap required to protect channels downstream of structures. Furthermore, all the relationships presented that take into account the influence of turbulence levels, only define these levels qualitatively. Hence considerable subjective judgement is involved in the process.

Two graphs have been produced relating the Froude number (Fr) to the ratio between the stone size and the total water depth (D/y_o) - Figs 3 and 4. They allow a comparison of the different equations presented in the literature review. Since the Froude number is usually defined using the mean flow velocity, U, the equations where the critical velocity is given in terms of the velocity near the bed, V_b, had to be modified. Equation (14) was therefore adopted for the relationship between V_b and U. As mentioned earlier, it is not certain which nominal stone size should be used for the roughness height k_s. For the present comparison it was decided to take $k_s = D_{50}$. It must be stressed, however, that the value of k_s has a marked effect on the ratio V_b/U. For example, considering the range of $D/y_o = 0.01$ to 0.1, a value of $k_s = 2D_{50}$ would correspond to an increase of 11 to 17% in the ratio V_b/U, when compared to $k_s = D_{50}$. The procedure using equation (14) was followed for the equations proposed by Izbash, Raudkivi (equation (5)) Peterka and the chart proposed by Searcy.

The first graph (Fig 3) refers to equations obtained under normal turbulence conditions: equations (5), (6), (10), (11), (12), (13) and Searcy's work. The second graph (Fig 4) refers to equations obtained under high turbulence conditions: equations (7), (9), (10), (11), (12) and (13). It should be noted once again that Maynord's equation was derived for turbulence generated by decelerating flow and not for turbulence downstream of structures. Similarly, Pilarczyk's equation (1987) refers only to turbulence generated by bends and Peterka's equation is, as the author himself warns, only tentative. As for Izbash's equation, it can be argued that the coefficient for high turbulence was obtained for isolated stones placed on top of a triangular shaped rockfill structure. This situation somewhat differs from that of a rockfill bed placed downstream of a stilling basin, for example. The fact that most of the formulas in Figure 4 do not apply to highly turbulent flows only emphasises the need for research in this area.

The comparison of the two graphs shows that, as expected, bigger stone sizes are required to protect against higher levels of turbulence. This is apparent from the shift of the curves to the left in Figure 4, ie lower Froude numbers for the same stone size. The widely varying predictions of the stone size given by the different equations can also be seen in Figures 3 and 4. For example, for a mean velocity $U = 1.88$ m/s and a water depth $y_o = 1$m, the nominal stone size can vary as much as four times, from 0.021m to 0.076m under normal turbulence or from 0.046m to 0.180m under high turbulence. In terms of weight, the predictions vary by a factor of up to 64.

4 EXPERIMENTAL SET-UP

4.1 Test rig

The tests were carried out in an existing 2.4m wide by 28m long flume fitted with three pumps having a total capacity of 0.5m³/s. In order to obtain a wider range of velocities and tailwater depths it was decided to reduce the width of the channel from 2.4m to 1.21m. An adjustable sluice gate was designed and installed in the flume to produce a hydraulic jump with associated turbulence upstream of the test section. The tailwater depths were controlled by means of a flap gate and a valve at the downstream end of the flume. Model materials representing different sizes of riprap were placed in a 2.60m long test section.

The transition between the smooth surface flume bed and the test section was achieved by a 1.74m long reach of stone fixed with glue to wooden boards placed on the flume bed. The purpose of the fixed stone reach was twofold: firstly, to act as a transition between a smooth and a rough surface; and secondly, to prevent excessive scour produced by unrealistically high turbulence levels upstream of the test section. Otherwise the formation of scour holes and bars would most probably affect the levels of turbulence in the test section. The upstream end of the fixed stone reach was placed 1.14m downstream of the sluice gate and stone sizes varied from test to test but were always smaller than, or equal to, the sizes that were being tested. The layout of the flume is shown in Figure 5.

4.2 Instrumentation

Discharges were measured by a Crump weir downstream of the flume which was calibrated at the beginning of the tests. A simple scale and a micrometer screw point gauge were installed upstream of the sluice gate and downstream of the test section, respectively, to measure water levels in the flume. The accuracy of the point gauge is approximately 0.00003m.

Point values of instantaneous flow velocity in the test section were measured by a three-component ultrasonic Minilab current meter. The meter calibration was checked independently against a Braystoke current meter which was also used to measure mean velocities just above the flume bed, upstream of the sluice gate. Preliminary tests with the ultrasonic current meter showed that it required regular monitoring of the offset signals at zero flow velocity conditions. This can be accounted for by the sensitivity of this type of equipment and therefore the probe's offsets were recorded regularly during the tests.

4.3 Model materials

Various sizes of stone were selected for the tests and their grading curves and specific gravity were obtained. Preliminary tests were carried out with a round stone having D_{50}= 7.7mm. Three different angular stones with D_{50} between 4.6 and 11.8mm were selected for the first set of tests. The positioning of various layers of gravel will also be considered to assess the effectiveness of filters and to provide guidelines for filter design. Tests will also be carried out with round stone having the same D_{50} as the selected gravel to compare their performance under turbulent conditions.

In the first stage of this study the materials were placed in rectangular cross-sections, on a horizontal bed. A second stage will deal with trapezoidal channels, ie riprap stability on banks, and sloping beds.

4.4 Data acquisition

The point velocity measurements from the three-component current meter were logged automatically into a Compaq Deskpro 286e micro-computer fitted with a differential analogue input board (AIP-24). The data acquisition board was used to convert voltage signals into digital signals read by the computer. This 24 channel board was also equipped with three filters to reduce interference by high frequency noise.

Records of 4096 point velocity measurements for each of the three directions (main stream, across the flume and vertical) were collected at a frequency of 12.5Hz and processed using a program developed at HR Wallingford. As mentioned before, records were regularly taken of the offset signals at zero velocity, ie the ultrasonic probe was removed from the test section and placed in still water. These 1024 point records were also taken at a frequency of 12.5Hz.

5 PRELIMINARY TESTS

A number of preliminary tests was carried out prior to the study not only to test the equipment but mainly to obtain an indication of the levels of turbulence expected in this study. Tests were therefore run over a smooth bed and over round stone with D_{50}= 7.7mm for different values of discharge. Velocity measurements were taken at various depths above the bed and turbulence intensities defined as $\overline{(V'^2)}^{1/2}/\bar{u}$ (where V represents any velocity component and u is the streamwise velocity component) were determined. Turbulence intensities of the order of 6% in the stream direction at bed level were obtained for discharges of around $0.075m^3/s$ over a smooth bed, whereas values of the order of 12% were obtained for flow over a rough bed. These tests were performed with naturally developed turbulence, ie the sluice gate was fully open thus not affecting the flow in the flume. These flow conditions will hereafter be referred to as normal turbulence and will provide a basis for comparison with turbulence downstream of structures.

6 ACKNOWLEDGEMENTS

This study was carried out by Ms M Escarameia and Mr I Willoughby in Mr R W P May's section of the Research Department headed by Dr W R White.

7 REFERENCES

CAMPBELL, F B (1966). Hydraulic Design of Rock Riprap. Miscellaneous Paper No 2-777. U S Army Engineer WES, USA.

Department of Transportation of the State of California (1970). Bank and Shore Protection in California Highway Practice. Caltrans, USA.

HEMPHILL, R W and BRAMLEY, M E (1989). Protection of River and Canal Banks. CIRIA, Butterworths, London.

IZBASH, S V and KHALDRE, Kh Yu (1970). Hydraulics of River Channel Closure. Butterworths, London

JANSEN, P Ph, BENDEGOM, L van, BERG, J van den, VRIES M de and ZANEN, A (1979). Principles of River Engineering. The non-tidal alluvial river. Pitman, London.

KEOWN, M P, OSWALT, N R, PERRY, E B, DARDEAU Jr, E A (1977). Literature Survey and Preliminary Evaluation of Streambank Protection Methods. Technical report H-77-9, US Army Engineer WES, USA.

MAYNORD, S T (1978). Practical Riprap Design. Miscellaneous Paper H-78-7. US Army Engineer WES, USA.

McCORQUODALE, J A and KHALIFA, A (1983). Internal Flow in Hydraulic Jumps. Journal of Hydraulic Engineering, Proc ASCE, Vol 109, No 5, pp684-701.

NARAYANAN, R (1978). Pressure Fluctuations Beneath Submerged Jumps. Journal of the Hydr. Div., Proc ASCE, Vol 104, No 7, pp1331-1342.

PIANC (1987). Guidelines for the Design and Construction of Flexible Revetments Incorporating Geotextiles for Inland Waterways. Supplement to bulletin No 57, Belgium.

PETERKA, A J (1964). Hydraulic Design of Stilling Basins and Energy Dissipators. US Department of the Interior, Bureau of Reclamation, Engineering Monograph No 25.

PILARCZYK, K W (1984) in The Closure of Tidal Basins. Closing of estuaries, tidal inlets and dike breaches. Deft University Press, The Netherlands.

RAMOS, C M (1990). Dimensionamento de Pontes (in Portuguese) ITH30, LNEC, Lisbon.

RAUDKIVI, A J (1967). Loose Boundary Hydraulics (1st Edition). Pergamon Press, Oxford.

RAUDKIVI, A J (1990). Loose Boundary Hydraulics (3rd Edition). Pergamon Press, Oxford.

ROUSE, H (ed), (1950). Engineering Hydraulics. Proc of the Fourth Hydraulics Conference. Iowa Institute of Hydraulic Research, June 1949. John Wiley & Sons, Inc., New York.

ROUSE, H, SIAO, TT and NAGARATNAM, R (1958). Turbulence Characteristics of the Hydraulic Jump. Journal of the Hydr. Div., Proc ASCE, Vol 84, No 1, pp1528-1 to 1528-30.

SEARCY, J K (1967). Use of riprap for bank protection. US Department of Transportation, Bureau of Public Roads, Washington DC, USA.

TENNEKES, H and LUMLEY, J L (1972). A First Course in Turbulence. The MIT Press, Cambridge, USA.

YUEN, C and FRASER, D (1979). Digital Spectral Analysis. Pitman Publishing Limited, London.

FIGURES.

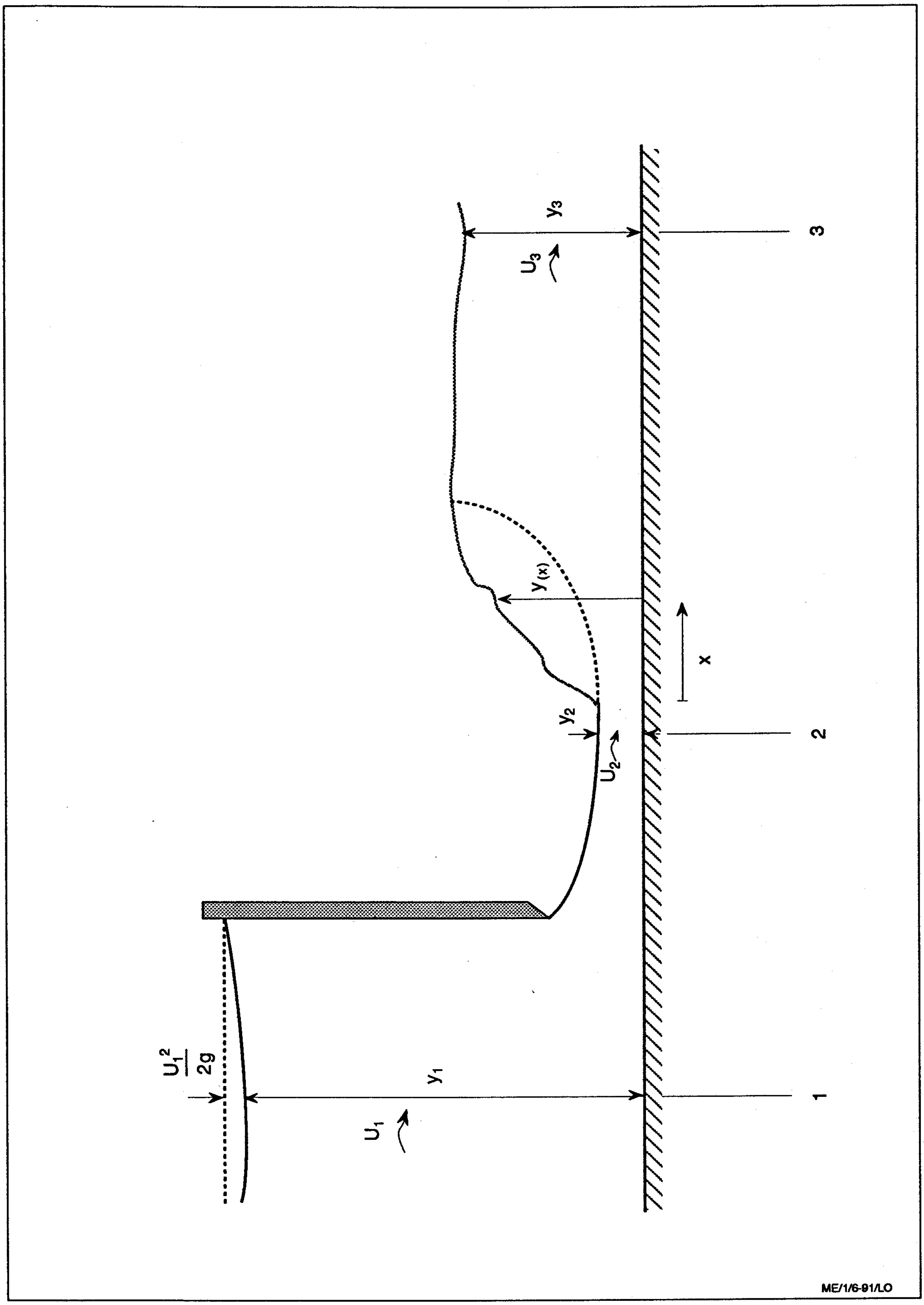

Fig 1   Schematic diagram of a hydraulic jump generated by a sluice-gate

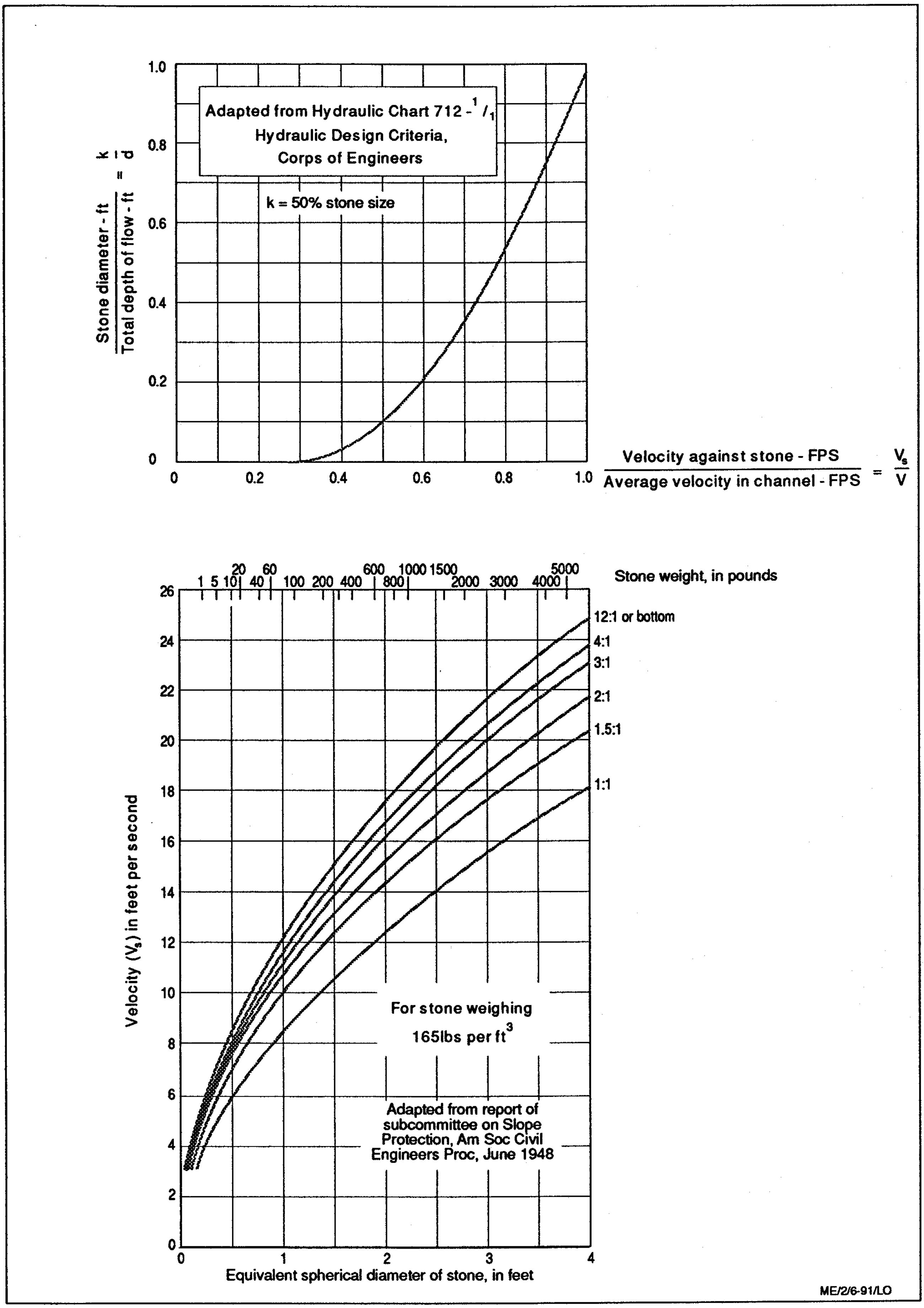

Fig 2 Charts for sizing stone protection on banks under parallel current attack (from Searcy, 1967)

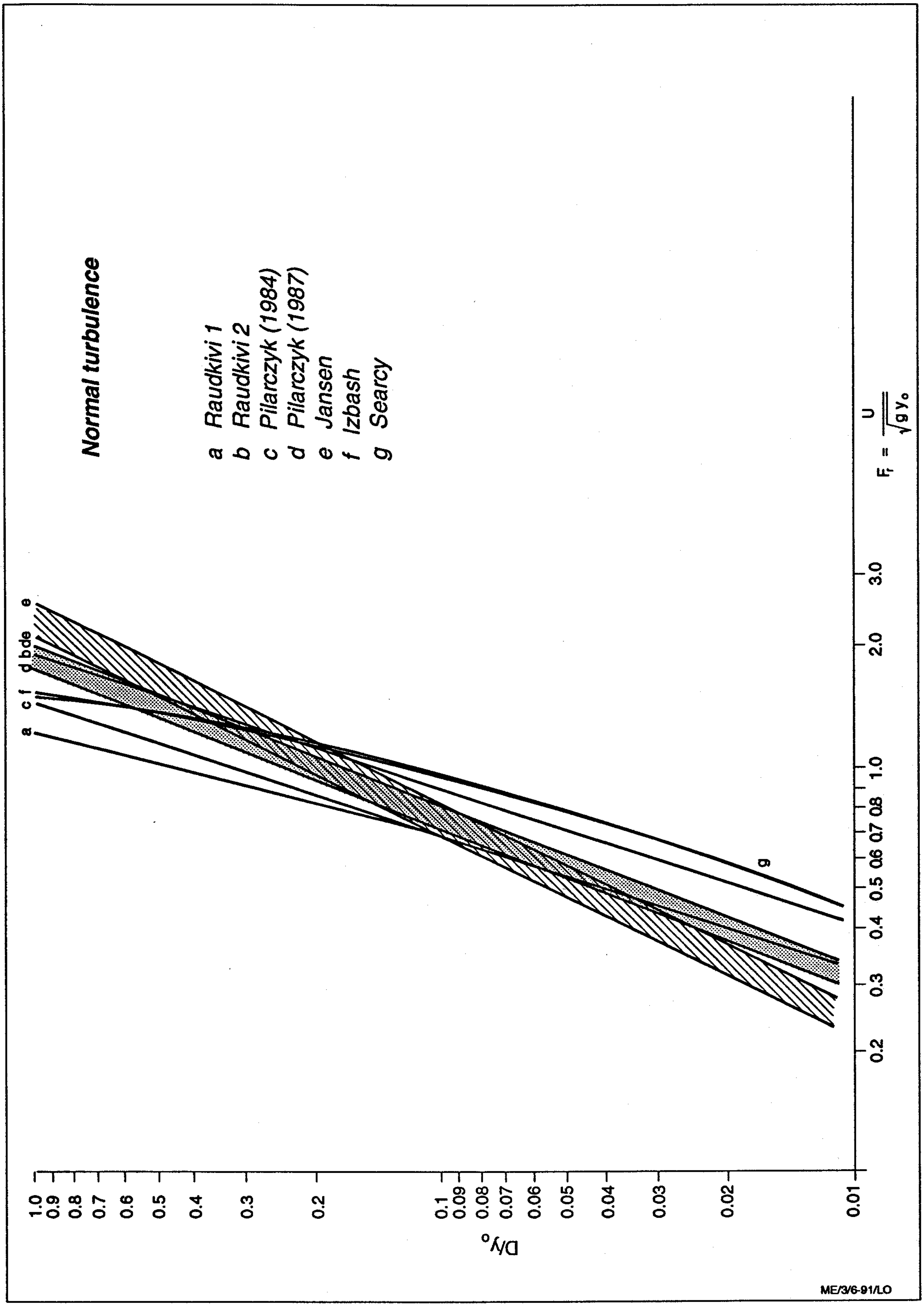

Fig 3 Comparison of different equations relating D/y_o to the Froude Number: normal turbulence

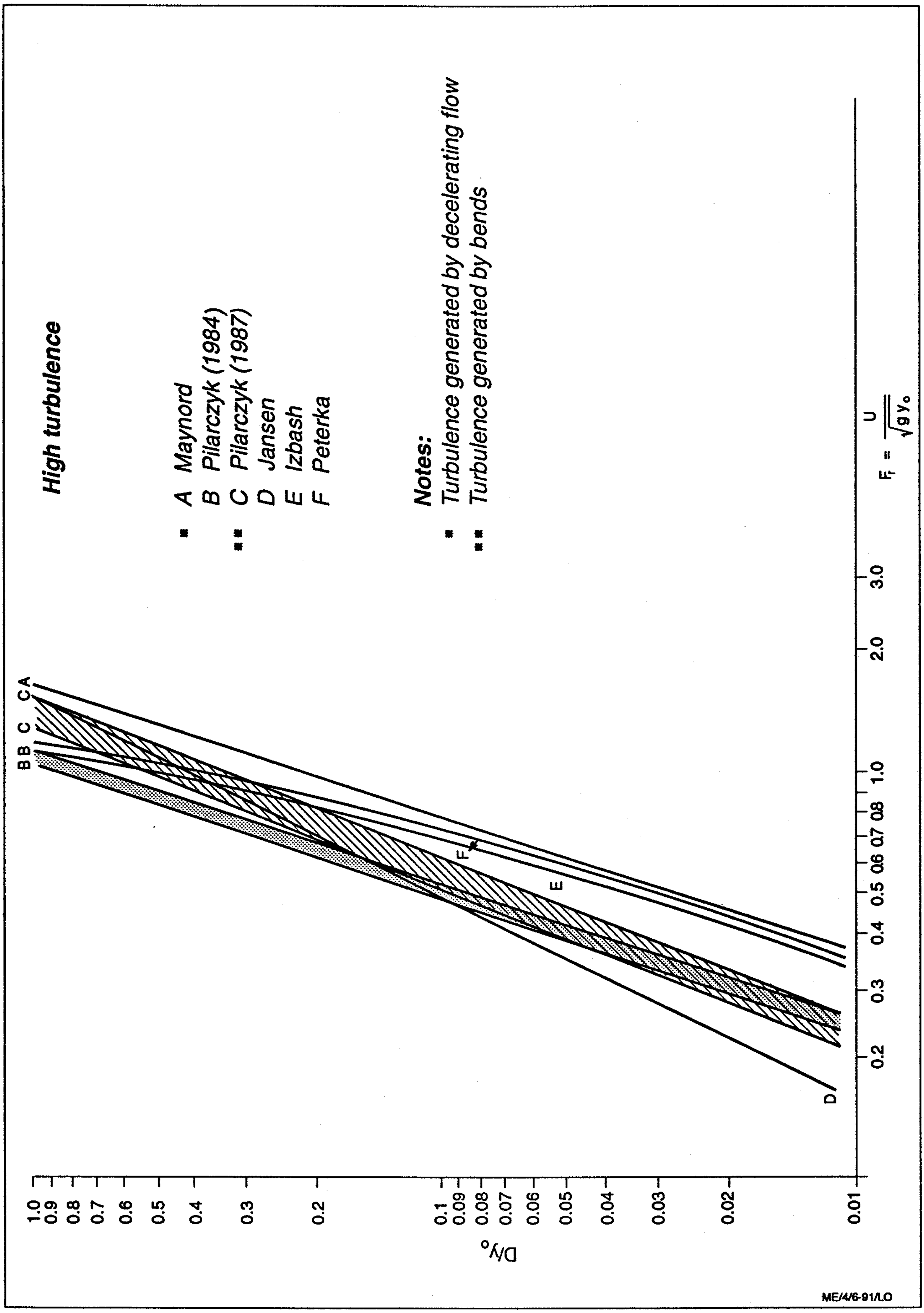

Fig 4 Comparison of different equations relating D/y_o to the Froude Number: high turbulence

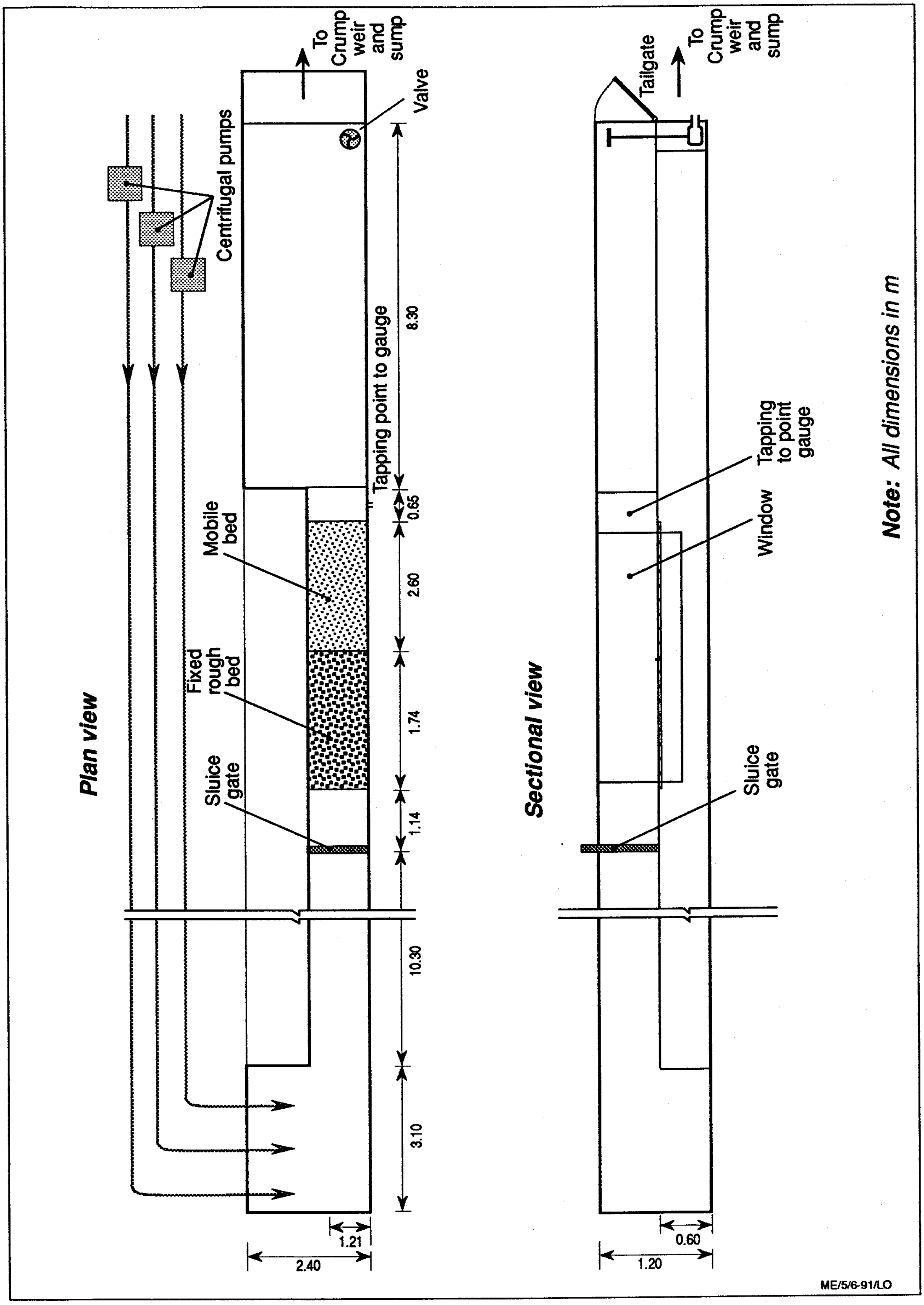

Fig 5 General layout of test rig