UNIVERSITY OF NEWCASTLE UPON TYNE

DEPARTMENT OF CIVIL ENGINEERING

A STUDY IN THE ESTIMATION AND MEASUREMENT OF BED LOAD DISCHARGE

by

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A thesis submitted for the Degree of Doctor of Philosophy

January, 1968.

CONTAINS

PULLOUTS

ACKNOWLEDGEMENTS

The author wishes to record his thanks to the Natural Environment Research Council which made this study possible by the award of a studentship during the years 1964-1967.

Sincerethanks are offered to Professor W. Fisher Cassie for the provision of research facilities in the Department of Civil Engineering, and to Mr. P. Johnson for his supervision and assistance during the research project and the preparation of this thesis. Mr. E. Armstrong and his technical staff who built and maintained the equipment are thanked for their valuable assistance.

In addition, the author wishes to express his appreciation of the co-operation which he has received from within the University, from the Northumbrian River Authority, and from Lord Allendale, riparian owner of the Bywell reach of the River Tyne.

CONTENTS

FIGURES

NOTATION

a	$\sum i_b/d$ for a sediment mixture
	constant in the microphone equation 7.3.d.
A	cross-sectional area of flow
A_b	cross-sectional area of flow pertaining to the bed
A_b'	cross-sectional area of flow pertaining to the bed related to particle roughness
A_b''	cross-sectional area of flow pertaining to the bed related to bed shape resistance
A_t	total cross-sectional area of flow
A_w	cross-sectional area of flow pertaining to the banks
b	constant in the microphone equation 7.3.d.
c_s	velocity of sound in a sediment particle
C	bed load charge, i.e. ratio of bed load discharge by dry weight to water discharge by weight
C_s	concentration by weight of suspended sediment in fluid
d	particle diameter
d_a	arithmetic mean weight diameter of a sediment mixture
d_e	geometric mean of triaxial dimensions of a particle
d_g	geometric mean weight diameter of a sediment mixture
d_k	characteristic particle diameter used by Pantelupulos
d_m	effective diameter of a sediment mixture used by Meyer-Peter and Müller
d_n	nominal diameter of a particle
d_w	sedimentation diameter of a particle
d_r	arithmetic mean of triaxial dimensions of a particle
d_s	sieve diameter of a particle
d_1	major axis of a particle
d_2	intermediate axis of a particle
d_3	minor axis of a particle

$d_{5,10...}$	particle size than which 5,10...% by weight of a sediment mixture is finer
d_{50}	median diameter by weight of a sediment mixture
$_nd_{5,10...}$	particle size than which 5,10..% by number of a sediment mixture is finer
$_nd_{50}$	median diameter by number of a sediment mixture
dH	rate of rise of river stage
D	depth of flow, average depth of flow
D_b	average depth of flow in area pertaining to the bed
e	the exponential number
E	average vibrational energy emitted by inter-particle collision per unit time per unit area of bed
E_a	energy-surface level A.O.D. at section AB
E_e	energy-surface level A.O.D. at section EF
E_j	energy-surface level A.O.D. at section JK
f'''	a function in the regime slope equation 2.2.e.
$f_{1,..7}$	a function of
f_c	critical fluid force acting on a particle
F	denotes function of M_s, a, b
F_b	bed factor in regime equations
F_{bo}	zero bed factor in regime equations
F_s	side factor in regime equations
g	acceleration due to gravity
h_e	stage at cableway section EF above staff gauge zero (46.25 ft A.O.D.)
H	stage A.O.D.
H_a	stage A.O.D. at section AB
H_e	stage A.O.D. at section EF
H_j	stage A.O.D. at section JK

i_b	fraction by weight of bed material in a given size range
i_B	fraction by weight of bed load in a given size range
$i_B q_B$	bed load discharge in dry weight per unit width of a given size range
$i_B Q_B$	bed load discharge in dry weight of a given size range
k	coefficient in regime slope equation 2.2.e.
k_r	coefficient of particle friction with a plane bed
k_s	height of surface roughness projections
k_t	coefficient of roughness in Strickler formula
$k_{1,..6}$	constants in derivation of microphone equation 7.3.d.
$K_{1,..23}$	constants
l	coefficient in regime slope equation 2.2.e.
	time lag between laboratory microphone and sediment weighing device
m	coefficient in regime slope equation 2.2.e.
$m_{1,..7}$	exponents
M	uniformity modulus of a sediment mixture used by Kramer
M_s	average level of record of signal from microphone
$M\phi$	phi mean diameter of a sediment mixture
$Md\phi$	phi median diameter of a sediment mixture
n	coefficient of roughness in Manning formula
n_w	coefficient of roughness for the banks in Manning formula
o	ratio of longest to shortest diameter of a particle
p	factor indicating the proportion of the bed area taking fluid shear
p_i	factor indicating the proportion of the bed area occupied by a given size range taking fluid shear
P	wetted perimeter
P_b	wetted perimeter of bed

P_t	total wetted perimeter
P_w	wetted perimeter of banks
P_1, P_2	dimensionless variables used by Yalin
q	water discharge in volume per unit width
q_b	water discharge pertaining to the bed in volume per unit width
q_B	bed load discharge in dry weight per unit width
Q	water discharge in volume
Q_b	water discharge pertaining to the bed in volume
Q_w	water discharge pertaining to the banks in volume
Q_B	bed load discharge in dry weight
r	relative intensity of turbulence
$r(l)$	cross-correlation coefficient between laboratory microphone signal and bed load discharge for a time lag of l minutes
R	hydraulic radius
	residual, difference between observed and predicted value
R_b	hydraulic radius pertaining to the bed
R_b'	hydraulic radius pertaining to the bed related to particle roughness
R_b''	hydraulic radius pertaining to the bed related to bed shape resistance
R_t	total hydraulic radius
R_w	hydraulic radius pertaining to the banks
	weighted residual, percentage difference between observed and predicted value
Re_*	Reynolds number related to the particle, $u_* d/\nu$
S	slope
S_s	specific gravity of sediment
S_e	energy-surface slope

S_w	water-surface slope
$(SR)_m$	quantity obtained by solving Keulegan equation for RS with a known mean velocity
t	time taken for particle to move from microphone section to end of sediment bed
u_m	shear velocity equal to $\sqrt{g(SR)_m}$
u_*	shear velocity
u_*'	shear velocity related to particle roughness
u_*''	shear velocity related to bed shape resistance
U	instantaneous velocity of flow at grain level
$\bar{U}$	time average of velocity of flow at grain level
U_c	critical velocity of flow at grain level
U_g	instantaneous velocity of particle
$\bar{U}_g$	time average of velocity of particle
$\bar{U}_s$	time average of velocity of particle during a step
V	velocity of flow
V_b	velocity of flow in area pertaining to the bed
V_w	velocity of flow in area pertaining to the banks
V_y	velocity of flow at height y above bed
w	width of bed
	width of laboratory channel
W	water-surface width
x	parameter for transition from hydraulically smooth to hydraulically rough flow
X	characteristic particle size of sediment mixture
	function of M_s, a, b, used in regression analysis
y	height above bed
Y	pressure correction factor
	function of M_s, a, b, used in regression analysis
Z	function of M_s, a, b, used in regression analysis

α_φ	phi skewness measure of a sediment mixture
β	a logarithmic function, $\log_{10} 10.6$
β_x	a logarithmic function, $\log_{10} (10.6 \times x/k_s)$
β_φ	phi kurtosis measure of a sediment mixture
γ_f	specific weight of fluid
γ_s	specific weight of sediment
δ'	thickness of laminar sublayer for U_*'
δ_m	thickness of laminar sublayer for u_m
η_p	efficiency of bed load sampler on a fixed bed
η_s	efficiency of bed load sampler on a movable bed
θ	angle of repose of sediment particles
Θ_s	characteristic of sediment mixture used by Straub
λ	porosity of sediment, fraction of a volume not occupied by sediment particles
μ	dynamic viscosity of fluid
ν	kinematic viscosity of fluid
π	circular circumference-diameter ratio
ξ	hiding correction factor for a mixture of sediment particle sizes
ρ_f	mass density of fluid
ρ_s	mass density of sediment
σ	standard deviation of flow velocity fluctuations
σ_φ	phi standard deviation of a sediment mixture
τ_c	critical fluid shear stress for initiation of motion of a sediment particle
τ_o	fluid shear stress on solid boundary
τ_o'	fluid shear stress on bed related to particle roughness
τ_o''	fluid shear stress on bed related to bed shape resistance
τ_*	dimensionless shear parameter

φ	phi particle size notation ($-\log_2$ diameter in mm)
$\varphi_{5,10...}$	phi size than which 5, 10...% by weight of a sediment mixture is finer
Φ	dimensionless intensity of bed load transport function
Φ_*	dimensionless intensity of bed load transport function for a given size range
Ψ	dimensionless intensity of shear function
Ψ'	dimensionless intensity of shear function for a representative particle
Ψ_m	dimensionless intensity of shear function for a given size range in modified Einstein procedure
Ψ_*	dimensionless intensity of shear function for a given size range

Subscript c denotes critical or threshold conditions of movement of sediment in fluid.

SYNOPSIS

The subject of the thesis is the estimation and measurement of bed load discharge, i.e. the rate at which coarse sediment particles are transported in a flow of water, in particular in the River Tyne near Bywell.

The first part of the thesis deals with the estimation of bed load discharge using empirical or theoretical formulae. A description of the collection of the necessary hydraulic and sediment data for the Bywell reach is followed by an account of the determination of the relationship between bed load discharge and river stage by several of the formulae available at present. Estimates of the average annual bed load discharge in the River Tyne at Bywell are given.

In the second part of the thesis, concerning the direct measurement of bed load discharge, attempts to use a trap-type sampler from the Bywell cableway are described. An account of the development of an experimental laboratory channel for the investigation of an alternative technique, the detection of sediment movement by acoustic methods, is followed by a comparison of the observed and theoretical relationships between bed load discharge and the sound emitted by inter-particle collision of the moving sediment. An acoustic bed load detector for rivers is also described.

Finally, a summary of conclusions, including recommendations for further research, is given.

Section 1

1. General Introduction

1.1. The Sediment Problem

The ideal geological cycle can be considered to consist of the upheaval of a land mass, its erosion to a plain near to sea-level, and a subsequent upheaval; as part of this cycle the sediment process of erosion, transport and deposition has played, and still plays, a major role in the shaping of the surface of the earth. From the times of the ancient civilisations of Mesopotamia, China and Egypt to the modern highly-developed, industrialised world of today sediment has created multifarious economic, social, and scientific problems. Consideration of its importance in the fields of soil conservation, reservoir development, water supply, power development, irrigation, stream erosion, flood control, navigation etc. indicates that there is a vital need for further reliable information concerning all three phases of the process. This requires not only a deeper understanding of the complexities of sediment mechanics but the development of instruments and methods which will facilitate the collection and analysis of accurate field data.

In the British Isles the magnitude of the sediment problem is relatively small. Occurrences of intense rainfall causing severe erosion are extremely infrequent, while, even in the larger rivers, movements of substantial quantities of sediment are rare. Perhaps the greatest problems are created by the movement of sands and silts in estuaries where dredging is required to maintain a navigable channel (INGLIS and ALLEN, 1957 and GIBSON, 1933), although high concentrations of suspended sediment in rivers are of some concern to the water supply industry (INSTITUTION OF WATER ENGINEERS, 1961) and can cause considerable damage when deposited in inundated areas (UNITED NATIONS, 1953). Publications dealing directly with the movement of coarse sediments in the freshwater reaches of English rivers are few (CLAYTON, 1951). Nevertheless, there are problems facing river engineers today concerning bank erosion, shoaling, channel improvement, gravel extraction etc. which so far have been tackled only by methods based on experience and empiricism. It is evident that before a more sophisticated scientific approach can be attempted further precise systematic sediment data is needed.

1.2. Investigations on the River Tyne

Hydraulic investigations on the River Tyne have been restricted mainly to the improvement of harbour facilities and, more recently, the pollution aspects of the estuary. The earliest sediment measurements were made in the estuary by RICHARDSON (1937) in an attempt to compare the results of laboratory studies with observations in the river. SWAIN and NEWMAN (1952) carried out a hydrographic survey of part of the tidal reach of the river near Stella power station but the collection of sediment data was not included. A study of the estuarine hydraulics of the Tyne was conducted by ALLEN (1962), while HALL (1964) investigated the patterns of movement of sediment and produced a sedimdnt budget for the estuary based on an examination of dredging records. An account of the work of Allen and Hall is included in two reports published by KING'S COLLEGE, UNIVERSITY OF DURHAM (1960, 1961).

HALL (1964) also carried out the only previous sediment investigations on the freshwater part of the River Tyne. Assessments of upland catchment erosion were made from the results of a survey of the deposition in a reservoir on the upper reaches of a main tributary. Attempts were made to measure bank erosion directly and information was collected on the variation of bed material size over the whole length of the river from source to mouth. Determinations of the total solids discharge entering the estuary were made from measurements of suspended sediment and dissolved solids at Bywell, about eight miles above the tidal limit. The only estimates of bed load discharge were based on the evidence of local gravel-extracting companies.

1.3. Aims and Objectives of Present Research

One of the important topics of the investigations of HALL (1964) which, through lack of time, received little attention was the transport of sediment as bed load, i.e. the coarse material which moves on or near the river bed by rolling, sliding and saltating. It was decided, therefore, that the subject of the present research would be an investigation of bed load movement in the River Tyne near Bywell cableway gauging station. The main objectives of the investigation were to establish the applicability of existing methods of estimating bed load discharge under the conditions prevailing at Bywell, and, if necessary, to develop a suitable alternative method.

As an indirect approach to the problem it was decided to review the large number of bed load formulae which have been proposed during the past one hundred years and to use as many as were considered applicable to derive a relationship between bed load discharge and river discharge or stage. This part would involve a comprehensive survey of the hydraulic and sediment characteristics of the reach of the River Tyne near Bywell.

It was intended to check these estimates of bed load discharge by direct measurement at Bywell. Preliminary attempts using a sampler designed for use on the River Danube in Czechoslavakia encountered financial and personnel difficulties. It was decided, therefore, to develop a different type of instrument which could be suspended in the river from the cableway and would detect and record the sound of inter-particle collision on the river bed. Sixteen months after the start of the research programme an experimental sediment channel was obtained by the University of Newcastle upon Tyne for the Hydraulics Laboratories of the Department of Civil Engineering. It was then decided that part of the remaining available time would be devoted to an investigation of the relationships between sediment discharge and the magnitude and frequency of the sound emitted by inter-particle collision in the controlled conditions of the laboratory channel. This part of the research would include the development and installation of equipment necessary for the operation of the flume and for the measurement of the relevant quantities.

PART I

Estimation of Bed Load Discharge

Section 2

2. The Sediment Process and Bed Load Movement

2.1. The Sediment Process

The sediment process forms a major part of the geological cycle and involves three distinct phases; erosion, transport and deposition. Each phase is extremely complex and closely dependent upon the other phases. As a natural process it is delicately balanced and, as history proves, interference by man can cause appreciable short-term and long-term changes in the equilibrium of one or more of its phases.

2.1.1. Erosion

Erosion is the fragmentation of soils and rocks to produce sediment. Water is the most powerful erosive agent with temperature, wind, ice, gravity and human activities such as mining, construction etc. also acting as contributary factors.

Detachment of the soil particles by raindrop impact and their subsequent removal to drainage channels is known as sheet erosion; this type of erosion, which coincides with the overland flow phase of the hydrological cycle, is witnessed in the more barren moorland areas of both the North Tyne and South Tyne. In arid and semi regions where vegetation is sparse and rainfall intensities are considerably greater sheet erosion is a more serious problem. As overland flow concentrates into rills and gullies the increased energy of flow permits further erosion known as gully erosion.
Gully erosion develops into bank erosion and becomes an integral part of the transport phase of the sediment process. Much of the coarser material transported by the River Tyne is the product of bank erosion. A petographic analysis (see section 3.5.7) of a sample of the surface of the river bed near Bywell revealed a considerable number of stones from parent rocks in the Lake District and the Southern Uplands of Scotland. These particles must have originated in the glacial drift which forms the banks of the middle and upper reaches of the River Tyne and its tributaries.

The erodibility of the rainfall and the erosivity of the soil generally determine the quantity of sediment involved in the erosion phase. The total quantity of eroded material which completes the journey from source to a catchment outlet such as a reservoir, a confluence with a main river, or the sea, is known as the yield of that catchment. The annual yield rate, usually expressed in tons per square mile, depends on:-

1) Physical conditions such as catchment area, slope and drainage pattern.

2) Hydrologic conditions such as precipitation and runoff characteristics.

3) Geologic, pedologic and vegetative conditions of the catchment.

HALL (1964) determined that the volume of sediment which has accumulated in Catcleugh Reservoir on the upper Rede over a period of 54 years is in the order of 10 million cubic feet. Assuming a bulk density of 80 lb/ft^3 the annual sediment yield of the 15.4 square mile catchment can be calculated to be 420 $ton/mile^2$. Based on the measurements of suspended sediment at Bywell by Hall and the results of this investigation an estimate of 170 $ton/mile^2$ can be made for the River Tyne catchment to Bywell. The average surface slope and rainfall intensity are lower over the larger catchment, thereby accounting for the difference in yields.

2.1.2. Transport

Sediment moving in rivers can be classified in two ways:-

1) Origin of sediment. That part of the sediment load which is composed of particles sizes found in significant quantities in the bed of the river is known as the bed material load. The finer particles of the bed material load may move continually in suspension while larger particles slide, roll or saltate along the river bed;

the total quantity in motion depends entirely upon the transporting capacity of the flow. Superimposed upon the bed material load is the wash load consisting of fine particles which are found in the bed in only small amounts. It may be regarded as an additive to the river, which is picked up on the drainage basin and passes in suspension without participating in the formative process of the river system. The quantity of the wash load is thus controlled by its availability from the catchment.

2) Mode of transport of sediment. From the point of view of sediment transport theory and measurement this is the more convenient, and more often used, method of classification. There are two distinct modes of movement. Suspended load is material moving in suspension, kept up by the upward components of the turbulent currents or by colloidal forces. Bed load is material, usually coarse, moving on or near the bed. A subcommittee on sediment terminology of the AMERICAN GEOPHYSICAL UNION (1947) defined two other terms; contact load, material which rolls or slides along the bed in substantially continuous contact with the bed, and saltation load which bounces along the bed or is moved, directly or indirectly, by the impact of bouncing particles. Bed load can thus be considered to comprise contact load plus saltation load.

Suspended sediment consists of both suspended bed material load and wash load and, due to the variability of wash load, is not a unique function of river discharge. At the beginning of a storm more fine material is available from sheet and gully erosion and rainfall intensities are higher than towards the end. Consequently, as has been observed on several rivers (BENEDICT, 1957 and KENNEDY, 1964), including the River Tyne, peak suspended sediment discharge may precede peak water discharge. Time of year also affects suspended load since during summer months there are longer and drier intervals between storms and the supply of wash load is thus greater. The theory of the distribution and transport of suspended sediment is based on the assumption that the settling velocity of a particle is counteracted by turbulent exchange, the continual exchange of fluid between horizontal layers. Many workers have experimented in this field, notably HURST (1929), GRIFFITH (1938), VANONI (1941, 1944), DOBBINS (1943) and NAGY (1961), enabling close predictions of river conditions to be made. Further references and more detailed information are given by BROWN (1950), CHIEN (1954a) and the AMERICAN SOCIETY OF CIVIL ENGINEERS (1963). Measurement of the suspended load of rivers can be relatively easily effected by means of depth-integrating, point-integrating or instantaneous type samplers. These samplers and methods and analysis of sampling are described in reports 1, 3, 6, 7, 8 and 13 of the UNITED STATES INTER-AGENCY COMMITTEE ON WATER RESOURCES, SUBCOMMITTEE ON SEDIMENTATION. Using an instantaneous type sampler suspended at 0.6 depth at mid-channel at Bywell cableway gauging station on the River Tyne HALL (1964) produced a curve relating average suspended sediment discharge to water discharge. By combining this curve with a 5 year duration curve of mean daily flows he estimated the average annual suspended load at Bywell to be in the order of 130,000 tons.

The various theories, concepts and methods of measuring bed load transport are treated elsewhere in this thesis. On the River Tyne a figure of 20,000 ton/year has been given by two gravel firms as the average natural replacement of excavated material and this figure was accepted by HALL (1964) to approximate to the average annual bed load discharge at Bywell.

During every phase of geomorphological activity a river will try to obtain some form of equilibrium between sediment supply and sediment transport. The more important variables involved are:-

1) the independent variables of water discharge, sediment discharge and effective sediment size.

2) the dependent variables of river slope, mean depth to width ratio and the meander characteristic (KUIPER, 1965).

The whole process of obtaining equilibrium is obscured by several factors such as variability of river discharge and sediment supply and the change of bed configuration and channel alignment with different sediment transport intensities. Much research, observation and measurement has been conducted in an effort to obtain the laws of normal river behaviour (GILBERT, 1914, MACKIN, 1948, LEOPOLD and MADDOCK, 1953, LEOPOLD, WOLMAN and MILLER, 1964). This delicately balanced equilibrium of the transport phase of the sediment process is generally referred to as the "regime" of the fluid system; BLENCH (1957) considers the word "regime" to be analagous to climate. Over most of the length of the River Tyne the regime of the river is stable, except in the short reaches which have been affected by artificial interference.

2.1.3. Deposition

Deposition, the third phase of the sediment process occurs mainly when a river enters its estuary. In general, and especially in the River Tyne, the river carries into the estuary not only a sediment load, but sewage, floating debris and spillage from ship-loading wharves. Considerable quantities of sand and silt are often transported into the estuary from the sea; this occurs frequently in the River Tyne, especially during a flood tide or north-easterly gale. Fig. 2.1.a. which is based on reports published by KING'S COLLEGE, UNIVERSITY OF DURHAM (1960, 1961) shows the approximate

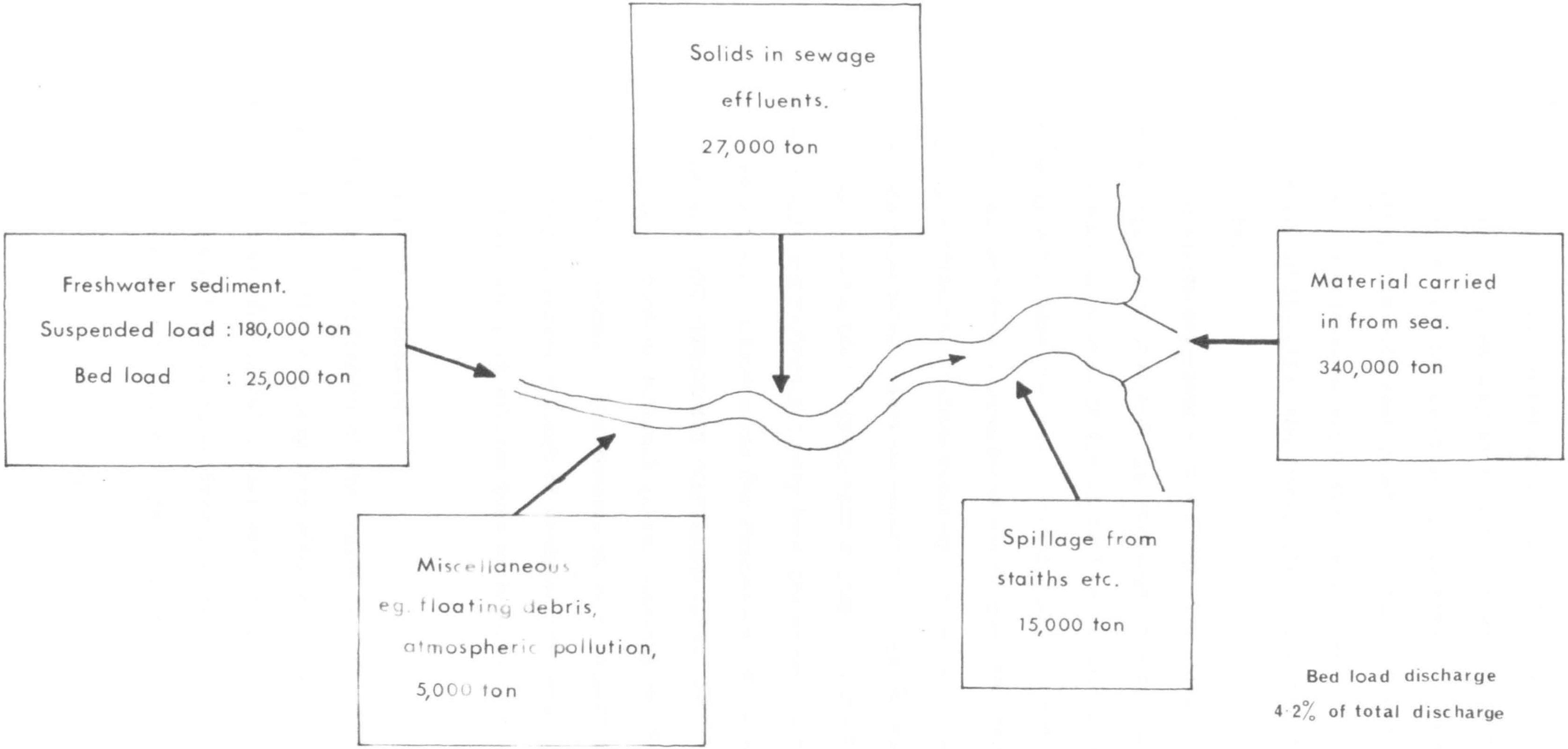


Fig. 2.1.a. ANNUAL SOLIDS DEPOSITION IN THE RIVER TYNE ESTUARY.

(Based on Bulletin No. 24 of the Dept. of Civil Engineering, King's College, Univ. of Durham.)

annual tonnage of solids entering the River Tyne estuary. Movement of sediment within an estuary is extremely complex (IPPEN, 1966), being affected by currents produced by the mixing of fresh water with salt water of a different density. These "density currents" can cause either erosion or deposition in the estuary, depending upon the state of the tide, the fresh water discharge and the shape of the estuary.

All the solids deposited in the River Tyne estuary (it can be seen from fig. 2.1.a. that bed load sediment constitutes only 4% of the total) must be removed if the river is to remain navigable by the shipping which uses the port. Until just over 100 years ago, when the Tyne Improvement Commission was formed, the River Tyne estuary was a tortuous, shallow waterway with shoals, sandbanks and even islands presenting a serious hazard to ships attempting to use the harbour. MACGREGGOR (1832), TAYLOR (1851), GUTHRIE (1880), SARGENT (1912) and HINDMARCH (1947) have described the prevailing conditions and the various plans for improvement which were proposed. Since then the TYNE IMPROVEMENT COMMISSION (1930, 1963) has removed many millions of tons of material in the clearing and deepening of a navigable river channel. Maintenance of this channel and the berths along the quays requires a fleet of dredgers and hopper barges which every year raise about 1½ million tons of material and deposit it far out to sea.

2.1.4. Artificial Interference

Artificial interference of the regime of a river can have appreciable short-term and long-term effects. In many areas of the United States, for example, increased erosion and hence increased sediment transport have resulted from the removal of large areas of vegetative cover (HUXLEY, 1945). The construction of reservoirs, bridges, weirs, river improvement schemes, flood control measures and gravel extractions all involve a disturbance of the delicate sediment equilibrium. Until recently the science of river engineering was based largely on individual experience and could predict only qualitatively the effect of engineering works. However, the recent publications of BLENCH (1966a), HENDERSON (1966) and THORN (1966) have enabled more quantitative approaches to be made.

A simple example of artificial interference is the construction of a weir, possibly for flow-gauging purposes. The backwater effect created upstream of the weir decreases the transporting capacity of the flow and the sediment load is deposited behind the weir. Downstream, the river is thus deprived of its normal sediment load and erodes both bed and banks to make up the deficiency. On the River Derwent at Rowlands Gill flood flows cause a large gravel shoal to form just upstream of a compound crump weir. The accumulation is removed at regular intervals by the Northumbrian River Authority. A model of the reach including the weir was built in the Department of Civil Engineering at the University of Newcastle upon Tyne and test results showed that errors in discharge estimation of up to 10% are possible if the presence of the shoal is disregarded.

Another interference from which the River Tyne has suffered considerably is the extraction of gravel from the river bed. Upstream of the workings the bed is lowered by increased velocities while further erosion occurs downstream due to the deficiency of sediment load in the river. River gravel is clean, well sorted, durable and readily obtained, offering an attractive proposition to gravel company operators. Until recently, however, practically uncontrolled extractions caused considerable damage, especially to the foundations of bridges situated upstream of the workings. Many thousands of pounds have been spent by local authorities, the Northumbrian River Authority and gravel companies on repairs to bridges at Ovingham, Hexham (fig. 2.1.b.), Haydon Bridge, Bardon Mill and Haltwhistle (figs. 2.1.c. and 2.1.d.) Extraction of gravel about one mile downstream of Bywell gauging station has altered the stage-discharge characteristics of the station and necessitated the sinking of a new float well. Gravel extractions are also detrimental to fishery interests by destroying the spawning grounds of migratory freshwater fish, and may be aesthetically undesirable (fig. 2.1.e.) As a result ROSS (1966), the Northumberland County Planning Officer, has recommended that no further licences for river gravel extractions should be granted; it was suggested that sufficient gravel could be obtained from glacial deposits.

Bank erosion in the River Tyne valley can be severe, resulting in the loss of valuable agricultural land. However, it has been realised by river engineers that if natural bank erosion is prevented at one place erosion will occur elsewhere due to the effort of the river to maintain its sediment equilibrium. Consequently, it is only where flood protection schemes or drainage outlet works are threatened that use is made of remedial measures such as pitching, stone-filled wire mesh crates, groynes and sheet-piling.

2.2. The Bed Load Transport Phase

During the past hundred years a vast amount of literature concerning the transport phase of the sediment process has been published in the English, French and German languages, and more recently the work of scientists and engineers in Russia and eastern European countries has become available. In the particular case of bed load transport a wealth of figures and statistics have been recorded, numerous contradictory statements made, and a plethora

Fig.2.1.b. Hexham Bridge on the River Tyne. Extensive sheet piling was found necessary to protect the bridge piers and adjacent banks against erosion caused by gravel extraction downstream.

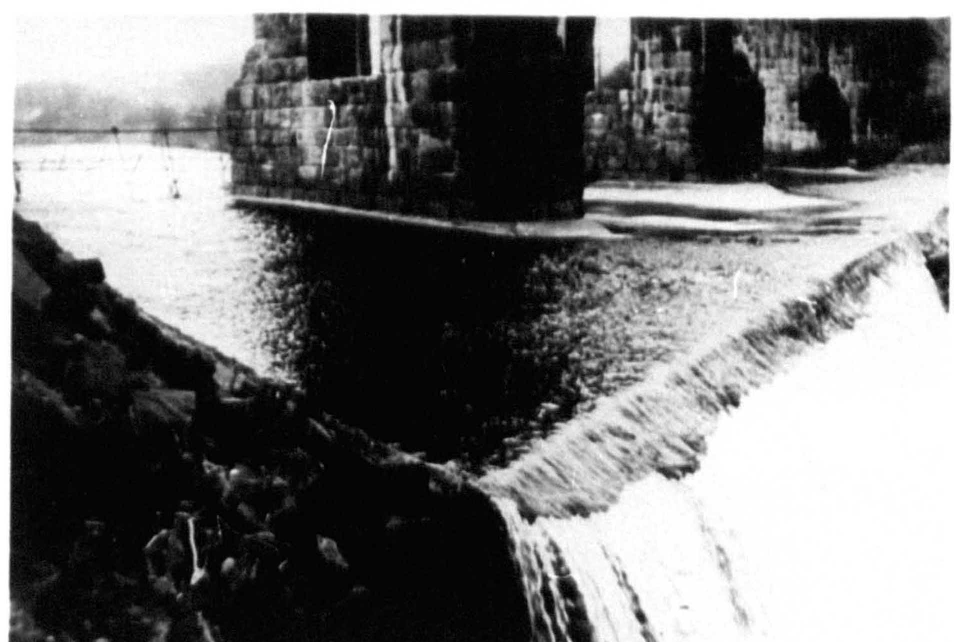


Fig.2.1.c. Haltwhistle Bridge on the River South Tyne. Gravel extraction one mile downstream caused severe undermining of the concrete apron between the bridge piers.

Fig.2.1.d. Haltwhistle Bridge on the River South Tyne. Gravel extraction one mile downstream has lowered the river bed nearly five feet.

Fig.2.1.e. Site of gravel extraction on the River South Tyne one mile downstream of Haltwhistle Bridge.

of theories developed. It is not intended to give here a detailed and comprehensive survey of this literature since this in itself would require several volumes. A general discussion only of bed load movement is given in this sub-section. Brief descriptions of several bed load theories and their application to the reach of the River Tyne at Bywell are given in section 4, following an account of the collection of the necessary sediment and hydraulic data in section 3. Much of the available literature on bed load movement and associated topics has been summarised in text books: LINSLEY, KOHLER and PAULHUS (1949), BROWN (1950), LELIAVSKY (1955), BLENCH (1957, 1966a), EINSTEIN (1964), HENDERSON (1966) and THORN (1966). Papers by CHIEN (1954a), the AMERICAN SOCIETY OF CIVIL ENGINEERS (1962, 1965, 1966) and BOGARDI (1965) include also substantial bibliography sections. For more detailed information on any work mentioned the original references should be consulted.

2.2.1. The Bed Load Concept

Due to the complexity of the movement of sediment in rivers a universal theory of bed load transport has yet to be formulated. The large number of variables involved, the problems of defining adequately some variables, and the complicated relationships between the variables have presented great difficulties. However, the rational approach of eliminating some of the variables and studying in detail selected parameters has produced valuable information. The effects of channel alignment, non-cohesive banks and variations in size and shape of the particles, for example, have been eliminated by conducting experiments with single-sized sediment in rectangular laboratory channels, where the variables of sediment discharge, water discharge, slope, depth and velocity may be controlled. The wide-ranging experiments and painstaking observations of GILBERT (1914) are classic in this respect.

By these means the laws governing the mechanism of entrainment and transport can be observed closely. As the velocity of flow of water over a bed of sediment is increased individual particles begin to move when a certain velocity is reached. This critical velocity is somewhat indefinite since the initial movement of the particles depends on the local turbulent fluctuations of velocity and the arrangement of the particles on the surface of the bed. When the velocity is sufficiently great it can be seen that the bed load, defined as that part of the sediment load which moves on or near the bed by rolling, sliding and saltating, moves within a thin layer, called the bed layer, only a few grain diameters thick. (For duned beds or for a wide range of particle sizes such as are found in gravel-paved rivers the bed layer concept becomes vague). KALINSKE (1942) has shown that the role of saltation in the fluid transport of sediment is of less importance than in the aeolian transport of sand (BAGNOLD, 1936).

It can be generalised that the frequency with which particles move from the bed and the velocity at which they travel depends mainly upon the velocity of flow near the bed.

2.2.2. Critical Conditions of Movement

When the hydrodynamic force acting on a sediment particle has reached a value such that, when it is increased, motion of the particle results, then critical or threshold conditions have been reached. Such quantities as depth, velocity and bed shear stress then have their critical or competent values. The problem of determining the critical conditions for the initiation of motion of sediment particles has received much attention, especially in the design of stable channels and canals through both cohesive and non-cohesive materials. Three main criteria have been used to define threshold conditions:- velocity, lift and shear stress.

Critical velocity, either the velocity near the bed or the mean velocity of flow, was the first to be considered. LELIAVSKY (1955) mentions that in 1753 Brahms formulated the well-known one sixth power law, i.e. the critical velocity is proportional to the one sixth root of the weight of the particle. This was restated by AIRY (1885), and several tables giving critical velocities for specific materials were produced by DU BUAT (1816), SCHOKLITSH (1914) et al. More recently, FORTIER and SCOBEY (1926), MAUVIS and LAUSHEY (1948), IPPEN and VERMA (1953), BOGARDI (1951) and NEILL and VAN DER GIESSEN (1966) are among those who have attempted to derive practicable relationships. The principal disadvantage of the velocity criterion is that the mean velocity of flow does not completely specify the scouring action of the water at the bed and the depth itself must be given. The difficulty of defining the bottom velocity of flow has also led to less consistent results than those obtained using bed shear stress as a criterion.

The use of the hydrodynamic lift as a criterion for the initiation of motion has received comparatively little attention. The theoretical considerations of JEFFREYS (1920) and EINSTEIN and EL SAMNI (1929), nevertheless, show that it can be of considerable importance. However, since in most cases lift depends upon the same variables as bed shear stress or drag, it is thus implicitly included in experiments involving observation of shear stress.

The criterion which has received the most attention is the bed shear stress, τ_o, or the critical bed shear stress, τ_c. The most important work in this connection was carried out by SCHIELDS (1936) who, by a rather devious method involving consideration of the forces acting on a sediment particle and the introduction of the Pradntl-Von Karman velocity distribution law, concluded that:-

$$\frac{\tau_c}{(\gamma_s - \gamma_f)d} = f_1\left(\frac{u_{*c}\, d}{\nu}\right) \quad \ldots\ldots\ldots\ldots \quad 2.2.a.$$

in which γ_s, γ_f are the specific weights of the sediment and fluid respectively, d is the particle diameter, f_1 means a function of, ν is the kinematic viscosity of the fluid and $u_{*c} = \sqrt{\tau_c / \rho_f}$ is the critical shear velocity; ρ_f is the fluid mass density. The left-hand side is termed the dimensionless critical shear stress, τ_{*c}, and the right-hand side the dimensionless critical Reynolds number of the particle, Re_{*c}. Equation 2.2.a. can thus be written:-

$$\tau_{*c} = f_1\,(Re_{*c}) \quad \ldots\ldots\ldots\ldots\ldots \quad 2.2.b.$$

BLENCH (1966b), however, points out that for the simple case considered (infinitely broad channel, uniform bed material) dimensional analysis would require the inclusion on the right-hand side of the above equation of two additional dimensionless variables, D/d and (S_s - 1), where D is the depth of flow and S_s is the specific gravity of the sediment.

Data from laboratory experiments by the UNITED STATES WATERWAYS EXPERIMENTAL STATION (1935), CHANG (1937), TISON (1948a, 1948b, 1953), PANTELUPULOS (1955, 1957, 1961), EGIAZAROFF (1957, 1959, 1965) and several Hungarian researchers (BOGARDI, 1965) have enabled a range of Re_* from 0.04 to 10,000 to be covered. Fig. 2.2.a. gives a plot of equation 2.2.b. as determined by this data. (It can be seen that bed configuration is also a function of τ_* and Re_*). For values of $Re_{*c} > 1000$ the value of τ_{*c} is constant at 0.055; equation 2.2.b. can then be reduced to:-

$$\tau_c = 0.055(\gamma_s - \gamma_f)d \quad \ldots\ldots\ldots\ldots\ldots \quad 2.2.c.$$

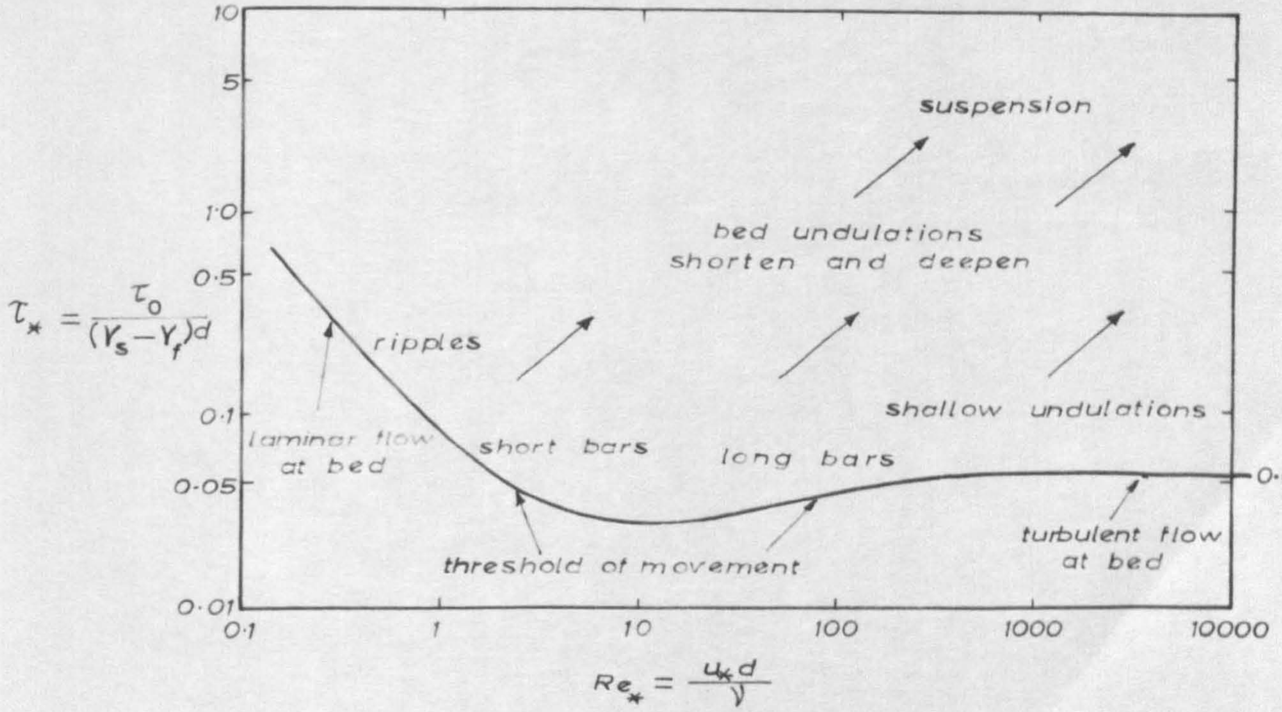


Fig.2.2.a. Schields entrainment function.

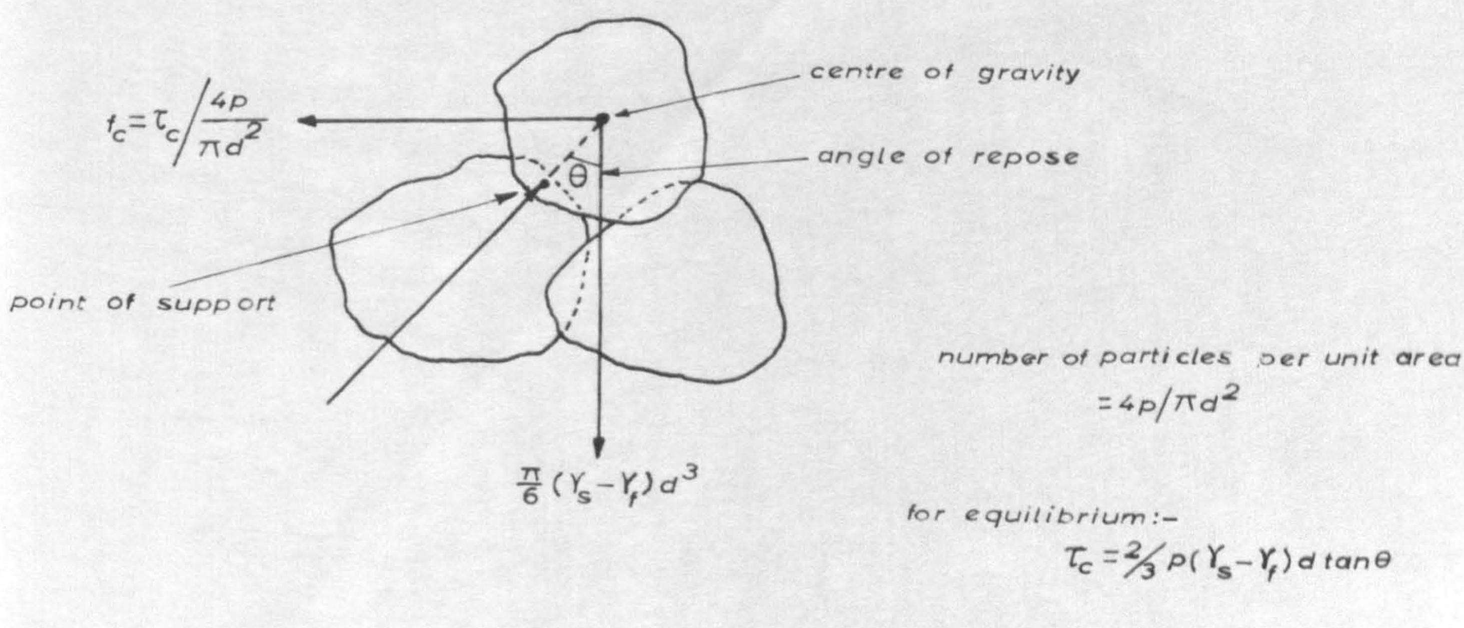


Fig.2.2.b. Forces acting on a particle at $Re_* > 3{\cdot}5$, (WHITE, 1940)

This denotes a hydraulically rough bed at which, in the case of water temperatures and specific gravities of sediment most frequently occurring in nature, particles larger than approximately 0.6 in, may be set in motion.

WHITE (1940), who conducted experiments with water flowing in specially designed nozzles, considered the forces acting on a particle on a horizontal bed. For equilibrium of the grain in turbulent flow, i.e. $Re_* > 3.5$, the critical fluid force acting on the grain (fig. 2.2.b.) is given by:-

$$f_c = \frac{\pi}{6} (\gamma_s - \gamma_f) d^3 \tan\theta$$

where θ is the angle of repose of the sediment particles. If p is the proportion of the bed taking fluid shear then the number of grains per unit area of bed is $4p/\pi d^2$ and the fluid force taken by each grain at critical conditions is:-

$$f_c = \tau_c / \frac{4p}{d^2}$$

giving $\tau_c = 2/3\ p\ (\gamma_s - \gamma_f) d \tan\theta$

Direct measurements of p and $\tan\theta$ for single-sized sands gave values of 0.35 and 1, respectively. White found that experimental results indicated it was necessary to introduce a factor of about ½ since turbulence in the region of the particle caused fluid forces to fluctuate about the measured force. The equation for the actual critical shear stress required to initiate motion is thus given by:-

$$\tau_c = 0.12 (\gamma_s - \gamma_f) d \quad \ldots\ldots\ldots\ldots \quad 2.2.d.$$

Experimental data from large scale turbulent flow such as occurs in rivers and laboratory channels show that the actual shear stress on the bed can be twice the measured mean value. Equation 2.2.d. therefore agrees well with the Schields diagram for $Re_* > 3.5$.

CAMP (1946) has shown how, by using $\tau_o = \gamma_f DS_e$, where S_e is the slope of the energy surface, and introducing the Darcy-Weisbach friction coefficient, it is possible to interpret equation 2.2.d. as a restatement of the Brahms one-sixth power law.

Due to the statistical nature of the entrainment process there is no true definition of the critical condition for the initiation of motion of a bed of sediment particles. Many subjective methods for describing movement near the threshold condition have been proposed; KRAMER (1934), for example, defined three intensities of movement:- weak, medium and general. However, some workers, including Schields, determined critical shear stress as the value of shear stress at zero bed load discharge obtained by extrapolation of the graph of sediment discharge against shear stress. This confusion undoubtably explains to a certain extent the variation in results of different workers in this field.

The results of SCHIELDS (1936), WHITE (1940) and others may predict the behaviour of uniform sediment particles forming a plane bed but their validity when applied to extremely non-spherical particles, mixtures of particle sizes and duned beds has not been proved.

2.2.3. Bed Load Discharge

The first attempt at the formulation of a rational theory of bed load transport was made by DU BOYS (1879) when he postulated that the bed load discharge should be a function of the difference between the bed shear stress and the critical shear stress of the sediment particles in the bed, i.e. a function of the excess tractive force. He assumed that the propulsive movement of the granular material varies gradually and uniformly from a maximum at the bed surface to zero at some depth below the surface. Basing his analysis on a concept of sliding layers of bed material kept in motion by the tractive force of the moving fluid Dy Boys proposed his now classical bed load formula:-

$$q_B = K_1 \tau_o(\tau_o - \tau_c)$$

in which q_B is the bed load discharge in weight per unit time per unit width, and K_1 is a constant depending upon the physical characteristics of the bed material.

Many bed load formulae are modifications of the basic Du Boys equation; several experimenters have used excess velocity or discharge raised to some power. Although observation of the movement of sediment in laboratory channels has shown that the over-simplified picture of sliding layers of sediment is definitely not true, good agreement with measured bed load discharges has been obtained with such formulae. JOHNSON (1938) shows statistically that they all fit experimental flume data equally well.

More recently, the problem has been approached on a more thorough theoretical basis. KALINSKE (1947), retaining the concept of critical tractive force, introduced the role of turbulent fluctuations and concluded:-

$$\frac{q_B}{u_* \gamma_s d} = f_2 \left(\frac{\tau_c}{\tau_o}\right)$$

in which the function f_2 involves the characteristics of the fluid turbulence around the grain.

The complex theory of EINSTEIN (1950) considered the probability of movement of individual particles to be a function of the hydrodynamic lift exerted on the particle. This lead to the functional relationship between two hydraulic parameters from which the bed load discharge could be calculated, i.e.

$$\Phi = f_3 (\Psi)$$

in which $\Phi = \frac{q_B}{\gamma_s}\left(\frac{\gamma_f}{\gamma_s - \gamma_f}\right)^{\frac{1}{2}}\left(\frac{1}{gd^3}\right)^{\frac{1}{2}}$, the dimensionless intensity of bed load transport function.

and $\Psi = \frac{(S_s - 1)d}{RS_e}$, the dimensionless intensity of shear function.

MEYER-PETER and MÜLLER (1948) derived a formula to fit the results of a large number of flume experiments, using the Froude law of similarity. In its simplest form it can be written:-

$$\frac{q^{2/3}S_e}{d} = K_2 + K_3 \frac{q_B{}^{2/3}}{d}$$

in which q is the fluid discharge in volume per unit width per unit time, and K_2, K_3 are constants. CHIEN (1954b) showed that not only could the above equation be modified to give a relationship between the Einstein parameters, Φ and Ψ, but it could be reduced to an excess tractive force equation of the Du Boys type.

From the above brief discussion it can be seen that all approaches to the problem of bed load transport are in fact related, the same dimensionless parameters occurring throughout. In the past decade considerable use has been made in sediment research of the technique of dimensional analysis by YALIN (1963) and GARDE and ALBERTSON (1961), inter alia. Using this technique it has been possible to combine a large number of variables into possibly significant dimensionless groups, many of which have been found to be similar to the parameters arrived at by a consideration of the physical processes of sediment transport.

2.2.4. Applicability of Bed Load Theories to Rivers

All "rational" bed load theories have been developed using the results of laboratory experiments. In some cases data were obtained from certain material under certain flow conditions and an empirical equation fitted (UNITED STATES WATERWAYS EXPERIMENTAL STATION, 1935, CHANG, 1937); other workers, in contrast, first evolved a theory of the mechanism of sediment transport and used this data to provide unpredicted coefficients which occurred in the analysis (KALINSKE, 1947, EINSTEIN, 1950). Strictly, therefore, these formulae are only applicable to the same materials in the same conditions, i.e. straight, smooth-sided laboratory channels with beds of uniform or well sorted sediments. The increasing use of dimensionless parameters has facilitated the application of these formulae to conditions such as those prevailing in river channels. However, while the range of values of many of these parameters are the same in river and laboratory, some parameters, especially those referring to channel geometry, are often different. The basic variables of depth and water discharge are usually greater in rivers. Extrapolation, therefore, becomes necessary.

River conditions differ in several other respects. Probably the most important differences are the existence of banks, the shape of the bed surface and the alignment of the river channel. Fully-developed dunes, bars and meanders (or a superimposed meandering thalweg such as that in the River Tyne at Bywell, described in section 3.3.3.) represent another type of resistance in addition to the roughness of the particles of which the bed is composed. Only the theories of EINSTEIN (1950) and MEYER-PETER and MÜLLER (1948) recognise the effects on sediment transport of the turbulence created by the two systems of roughness. Also, macroturbulence on the scale observed in rivers by MATTHES (1947) does not occur in laboratory flumes.

For gravel-paved rivers the problems of size and a large range of sizes of sediment arise. Few experimenters have used material other than sand, the largest size used in quantitative bed load laboratory experiments being 1.62 in by EGIAZAROFF (1959). Although SCHOKLITSCH (1934) and KALINSKE (1947) suggested the computation of bed load discharge for individual size ranges of a sediment mixture, and MEYER-PETER and MÜLLER (1948) used a single size to represent the mixtures used in their experiments, only EINSTEIN (1950) attempted to account for the mutual interference effects between particles of different sizes. On the River Tyne at Bywell accumulations of small particles can frequently be observed immediately downstream of large stones.

None of the theories allows for the effect of particle shape to be directly and quantitatively included. The surface of the beds of many rivers with underlying sedimentary rocks often contain a preponderance of disc-shaped particles; at Bywell about 60% of the bed surface particles are disc-shaped (see section 3.5.6.). With a large range of sizes this type of bed is particularly suited to the formation of a "pavement" having a critical shear stress for the initiation of movement considerably greater than that indicated by the equations of SCHIELDS (1936) or WHITE (1940).

Finally, the relationship between sediment motion and streamflow in natural watercourses is extremely complex. Not only does sediment discharge depend upon the hydrology, geology and pedology of the catchment but hydraulic characteristics such as depth, velocity, turbulence and shear, and hence the sediment transport characteristics are likely to vary non-uniformly both laterally and longitudinally within the river.

Where direct observation of bed load discharge in rivers has been possible for instance in the Danube in Hungary (KAROLYI, 1957), results have displayed considerable fluctuations, indicating that bed load movement in rivers is a non-steady process. The "rational" formulae, based on laboratory experiments in steady flow conditions, cannot then yield satisfactory predictions for rivers. It would seem that in order to further investigations of the relationship between actual and predicted sediment transport the development of an instrument for the continuous recording of bed load movement in both rivers and laboratory channels is necessary.

2.2.5. The Regime Approach

The concept of "regime", or the equilibrium state of rivers and canals, has been referred to in section 2.1.2. This concept and the practical necessity of excavating stable channels in alluvial material has led to the formulation of a set of empirical design rules known as the regime theory. Based on the observation of a number of Indian canals KENNEDY (1885) proposed the first quantitative rule that the "non-silting, non-scouring" velocity of a stable channel is proportional to some power of its depth. Further work by LINDLEY (1919), LACEY (1929, 1948) and many others led to the important concept of the three degrees of freedom of self-adjustment of regime channels, requiring three equations for the complete determination of the ultimate stable breadth, depth and slope of the channel. It is possible to consider that a fourth degree of freedom exists in rivers or neglected canals where meandering has been allowed to take place.

Recently, BLENCH (1955, 1957, 1961) introduced the bed load charge, C, the ratio of the bed load discharge by weight in air per unit time to the water discharge by weight per unit time. His equations are as follows:-

$$D = V^2/F_b$$

$$W = V^3/F_s$$

in which V is the mean velocity in the section, D is the depth and W is the width at mid-depth. F_b and F_s are termed the bed and side factors, respectively. The latest slope equation is given by BLENCH and QURESHI (1964) as:-

$$S = klm \left[\frac{F_{bo}^{11/12}}{\frac{3.63g}{\nu^{\frac{1}{4}}} \; W^{1/6} \; Q^{1/12}} \right] f'''(C) \quad . \; . \; . \; . \; . \; . \quad 2.2.e.$$

in which k, l, and m are coefficients to allow for meandering, the definition of the representative discharge, Q, and miscellaneous other effects (e.g. suspended sediment), respectively. F_{bo} is termed the zero bed-factor and is a function of d_{50}, the median particle diameter by weight of the bed material. Q is the formative discharge, usually taken in rivers as bankful discharge; exact definition of Q is rendered unnecessary by the exponent 1/12. The function, f'''(C), has been derived by BLENCH and ERB (1957) from the results of GILBERT (1914) and other workers as:-

$$f'''(C) = \frac{(1 + 0.12C)^{11/12}}{(1 + C/233)}$$

The theoretical lower limit of f'''(C) is thus unity.

The regime formulae have been developed with the immediate purpose of facilitating the design of stable channels in alluvial material; the "rational" approach has been formulated with the specific intention of predicting bed load discharge. However, it is possible that the slope equation of Blench could be used to determine the bed load discharge of a river in which both water and sediment discharges fluctuate widely. NIXON (1959) has been able to establish regime relationships for several English rivers of this type.

The most promising approach to a complete solution of bed load transport, since this is the part of the sediment load most closely associated with channel shape, is probably the ultimate combination of the "regime" and "rational" theories. In this connection the work of ACKERS (1964) on small streams in alluvium and the derivation of the regime equations using the principle of minimum energy degradation rate by BREBNER and WILSON (1967) could prove to be valuable contributions.

Section 3

3. The River Tyne at Bywell

It has been seen in section 2 that a number of hydraulic and sediment parameters are necessary for the estimation of bed load discharge in a river. This section describes the collection of this required data for the reach of the River Tyne near Bywell.

3.1. The River Tyne Catchment

The River Tyne drains an area of about 1,140 square miles of the counties of Northumberland, Durham and Cumberland in the north of England and a small part of the county of Roxburghshire in Scotland. The catchment, roughly triangular in shape, is bounded in the south and west by the main Pennine block and in the north by the Cheviot dome (fig. 3.1.a.). Both main tributaries, the River North Tyne and the River South Tyne respectively, are considerably varied in character throughout their lengths; in some parts, the rivers meander freely through large banks of gravel, locally termed haughs, in marked contrast to the fast-flowing, rocky reaches in other parts. From the confluence of the two rivers the lower River Tyne flows eastwards, past Bywell gauging station, for about thirty miles to enter the North Sea at Tynemouth. The tidal influence extends upstream as far as Wylam.

PEEL (1941) has demonstrated that the longitudinal profile of the River Tyne is of considerable interest. On the basis of many measurements, mainly on the North Tyne, he was able to produce the thalwegs of the River Tyne and its main tributaries, as shown in figs. 3.1.b. and 3.1.c. The profiles reveal features entirely consistent with topography except for a break, at which the River North Tyne suddenly steepens, just below Bellingham at a height of about 350 A.O.D. Peel suggested that this "knick-point" and similar breaks on other tributaries are the result of river rejuvenation. Assuming a semi-logarithmic equation for the

RIVER TYNE DRAINAGE AREA.

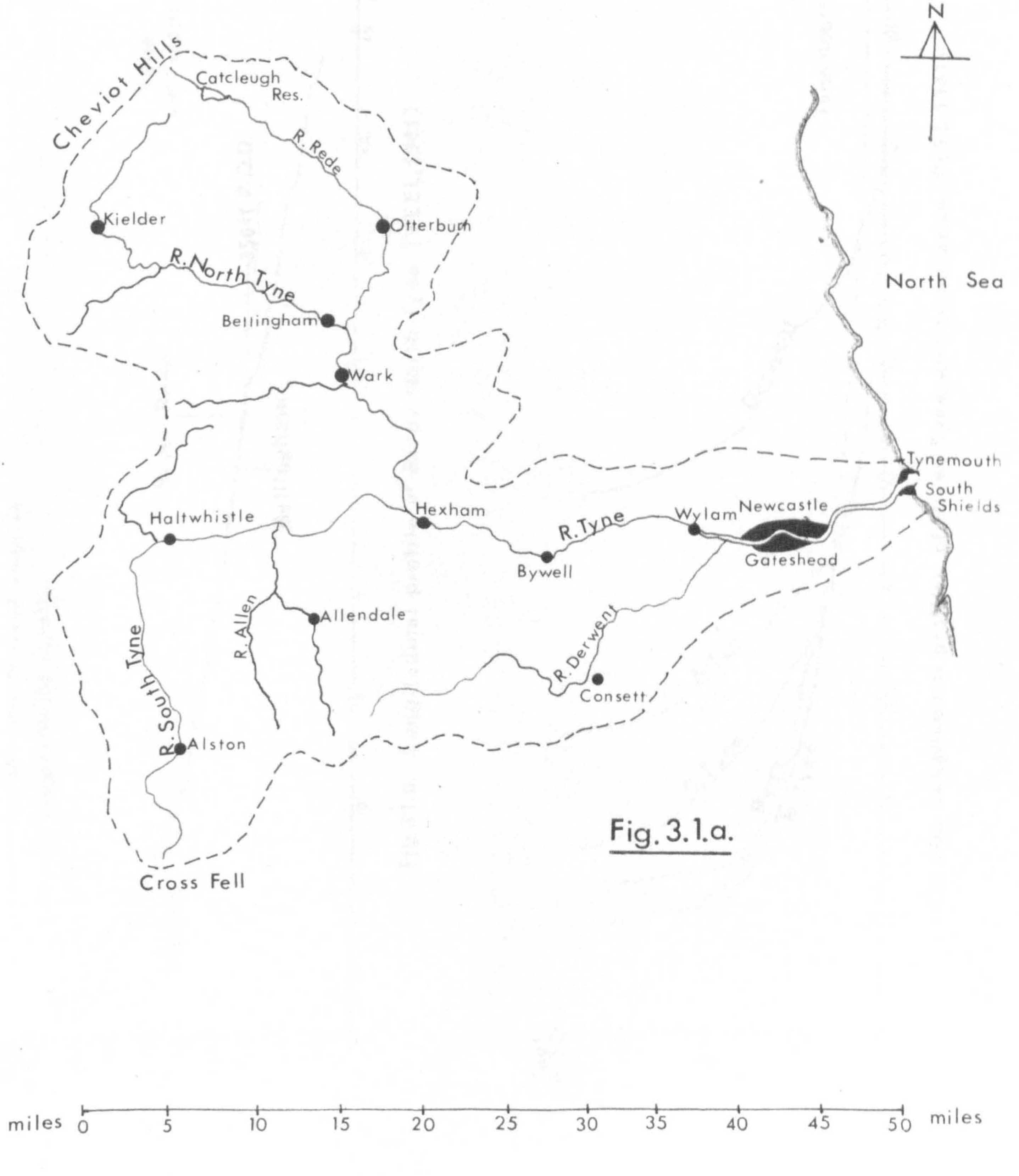


Fig. 3.1.a.

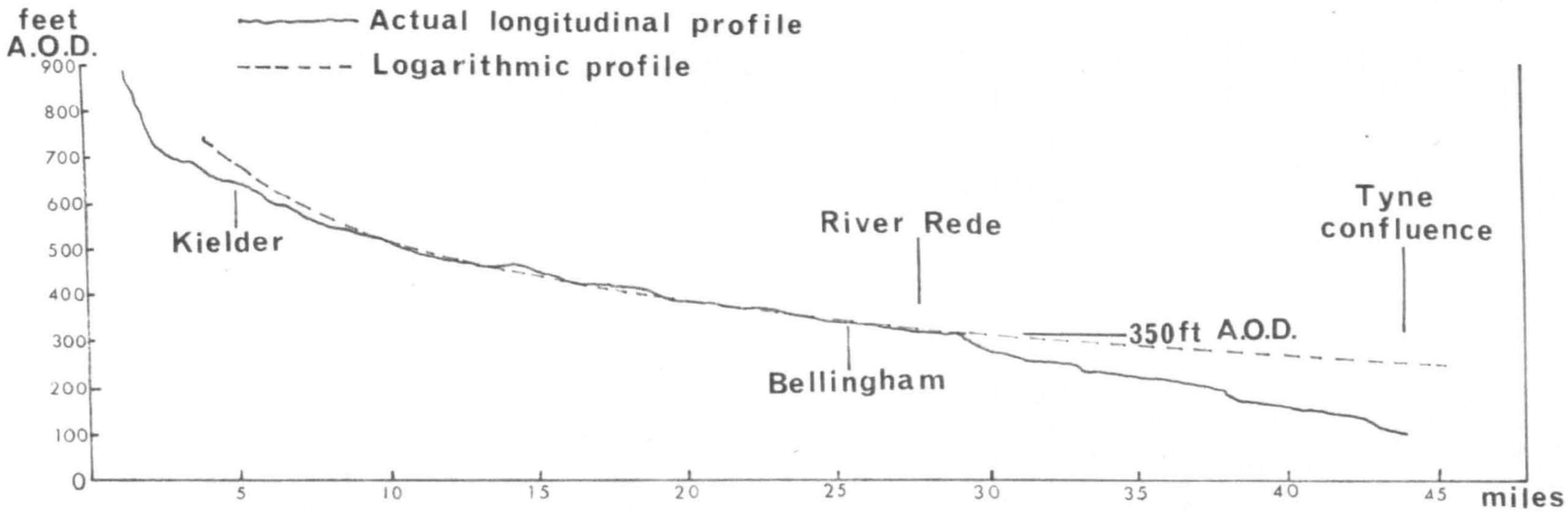


Fig.3.1.b. Longitudinal profile of River North Tyne (PEEL,1941)

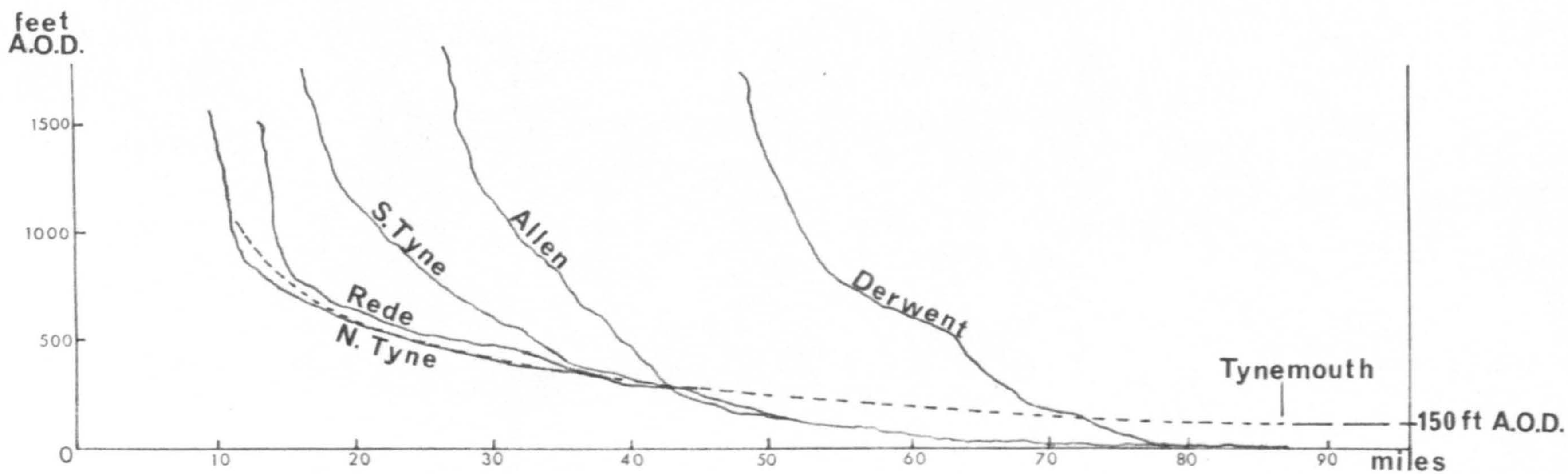


Fig.3.1.c. Longitudinal profiles of River Tyne and tributaries (PEEL,1941)

mature profile he showed that the central part of the River North Tyne is graded to a base level about 150 ft above the present level while the lower part is still in a stage of youth, being unadjusted and irregular.

The geology of Northern England has been comprehensively described by EASTWOOD (1963) and ROBSON (1965). The underlying rocks of the River Tyne catchment are mainly sandstones, limestones, shales and coal measures of the Devonian and Carboniferous period; the only igneous rocks are those of the Cheviot volcanic extrusion and a number of dykes and sills of quartz dolerite. Glacial drift, up to 200 ft. thick in places, has been deposited by ice which flowed over north-east England from Scandanavia and the Lake District during the period of maximum glaciation. The main mass of the drift is usually a tough dark deposit containing boulders, mostly of local origin; above this is a more sandy, brownish clay containing smaller boulders and pebbles of material foreign to the region (HICKLING et al, 1931).

Thin, acid soils in upland areas of the catchment support moorland vegetation, rough pasture and, especially in the North Tyne region, considerable expanses of forest; in the valleys and lowland areas loams and clays, mostly originating from glacial carboniferous till, provide large areas of good meadowland and permanent pasture (NORTH-EAST DEVELOPMENT ASSOCIATION, 1950 and NORTHUMBERLAND COUNTY COUNCIL, 1952).

The greater part of the River Tyne catchment is sheltered from the prevailing south-west winds by the mountain barriers of the Northern Pennines and the Cheviot Hills. Annual average rainfall varies from nearly 80 in. on the South Tyne area near Cross Fell to about 26 in. on Newcastle upon Tyne (METEOROLOGICAL OFFICE, 1961).

According to HALL (1964) runoff amounts to about 64% of rainfall on the catchment. Concentration of direct runoff on the River Tyne and its tributaries is often extremely rapid. During the present investigations the river level on the River Tyne at Bywell was observed to rise 9 feet in 1½ hours. This effect has caused the occurrence of serious flooding on several occasions, especially in 1771 (PALMER 1882) and more recently in 1954 (BLACK, 1957).

3.2. The Test Reach

The Bywell cableway gauging station (grid reference NZ/038.617) is situated on the River Tyne 1½ miles upstream of Bywell Bridge, Northumberland and 8 miles upstream of the tidal limit at Wylam (Fig. 3.2.a.). It was constructed in 1956 by the Northumberland and Tyneside River Board (now part of the Northumbrian River Authority) on land owned by Lord Allendale. The area of the River Tyne catchment to the station is 834 square miles.

The gauging station, which is easily accessible, consists of a cableway for current-meter gauging and a wooden hut located over a float well on the left bank of the river. A wire cable spans the distance between two stanchions, 12 feet high, situated on opposite banks about 30 feet above low water level. A small trolley mounted on this wire can be manoeuvred to any position across the river by means of a continuous cable which passes round one drum of a hand-operated winch unit in the hut. Another cable, for suspending an instrument from the trolley at any depth in the river, can be manipulated by means of a second drum on the winch. The suspension cable is of a coaxial type, with a single core insulated from the outer braiding, and is used for carrying electrical signals to or from the suspended instrument. The station is equipped with a Lea Recorder vertical-drum recorder

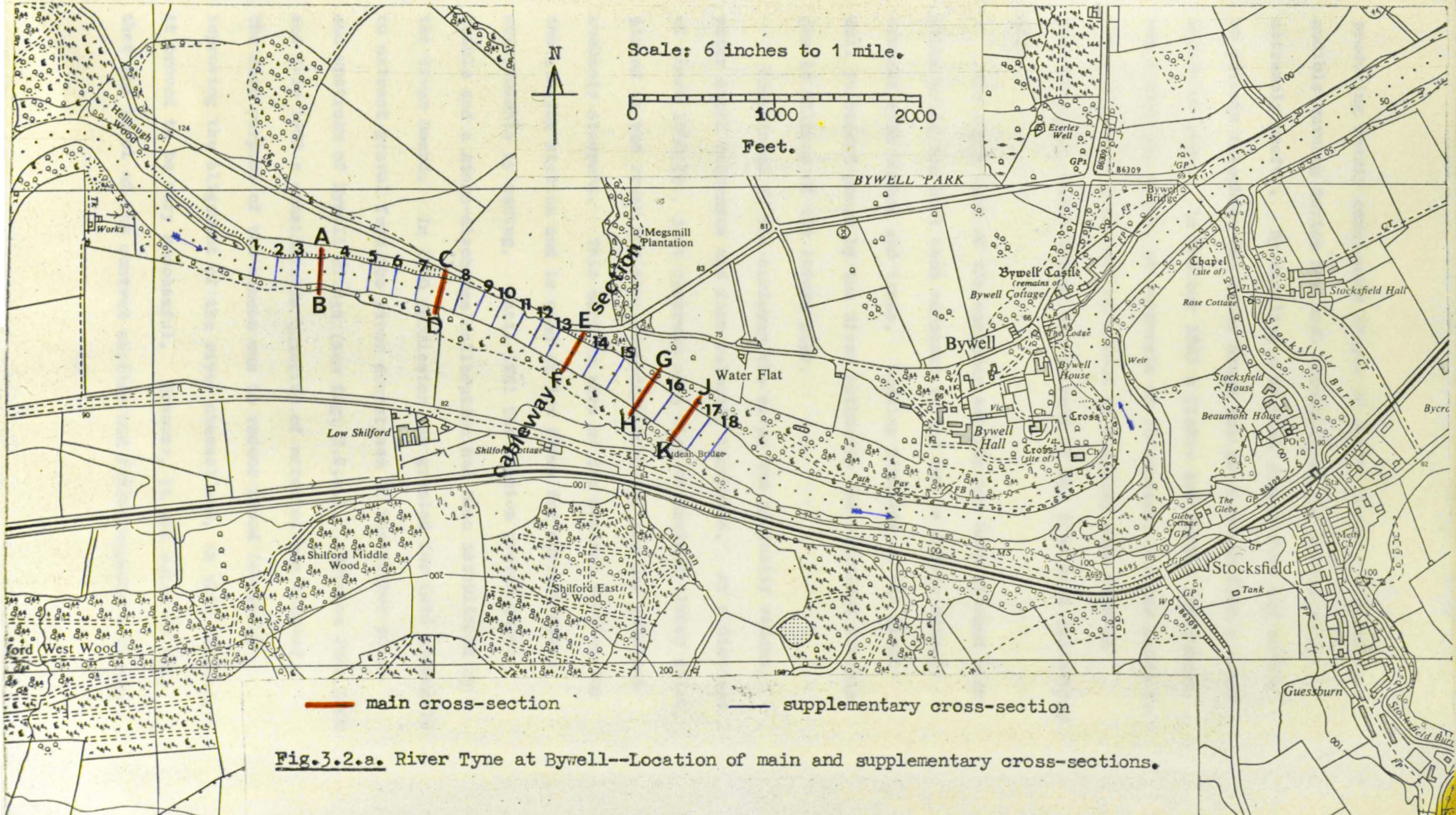


Fig.3.2.a. River Tyne at Bywell--Location of main and supplementary cross-sections.

producing weekly continuous charts of water level. A teletone audible warning device has been incorporated and proved to be extremely useful. By dialling a special number from any public or private telephone the river stage can be obtained from a coded series of pips. In October 1965 a Fisher and Porter instrument was installed, which, at intervals of 15 minutes, records stage to the nearest one hundredth of a foot in a binary code form on punched tape; water levels can be read at any time on a calibrated disc.

The right bank of the cableway section has been pitched with stone while the left bank adjacent to the hut is fairly densely covered with bushes and trees. Regular flushing of the float-well in recent years by the River Authority has minimised trouble due to silting of the intake pipe.

Downstream of the cableway the river bed gradually widens, water depth decreases and flow velocity increases. At a distance of about 1000 ft., just upstream of a line of ancient timber piles, placed in the river for some forgotten purpose, the bed gradient suddenly steepens. This section acts as the control section for the gauging station and is used at low flows for current-meter measurements by wading. Until 1961 the section proved quite stable and a stage-discharge calibration had been established by the River Board. In 1960 permission was granted for Lord Allendale to extract gravel from the river downstream of the timber piles and upstream of Bywell Bridge (see fig. 3.2.a.). Between June 1960 and July 1962 a considerable quantity of material was removed. The main purpose of the scheme was to reduce flood levels by improving the alignment of the river channel and, in this respect, it proved to be very successful. However, it was noted in 1961 that the level of the control section had fallen causing a change in the low-flow part of the stage-discharge curve; no change could be detected at high flows. In September 1964, by which time the river bed had regained stability, the float-well and intake were reconstructed and a new calibration of the station started. It should perhaps be noted that periodic fluctuations of the lower ends of the rating curves of cableway stations on the River Tyne and other Northumberland rivers have been observed where gravel extractions have not taken place.

EINSTEIN (1964) has emphasised the importance of the concept of the test reach which should adequately describe the overall characteristics of the river channel. One of the greatest difficulties in applying hydraulic equations, especially those concerning sediment transport, to a natural river is the basically irregular flow pattern in such a channel. Each cross-section is different from all other cross-sections. In the application of all but one of the methods mentioned in section 4 for computing bed load a knowledge of the average water-surface or energy-surface slope is necessary. Only one method requires a number of cross-sections in the reach to be averaged. The nature of the longitudinal profile of the River Tyne near Bywell made the determination of these average conditions even more difficult. Observation of the river channel showed that the profile consists of a series of deep slow-flowing "pools" separated by shallow "bars" which at low and medium flows form rapids. This undulating "pool-bar" configuration occurs frequently in gravel-bed rivers; LELIAVSKY (1955) describes how some Russian rivers have been found to be of this type. Bed irregularities have little effect on water-surface profiles at high discharges but are closely reflected in low or medium flow profiles. The cableway gauging station at Bywell is located in a "pool", while the "bar" downstream acts as the control section.

It was decided to survey several sections on the fairly straight reach of river which extends from rapids about ½ mile upstream of the cableway to just downstream of the control section The total length of the reach is about ¾ mile.

3.3. Survey of the Reach

In May 1965, permission to survey the river reach near Bywell was obtained from the Northumbrian River Authority and the riparian owner, Lord Allendale.

3.3.1. Main Cross-sections

After inspection of the reach it was decided to sound in detail five cross-sections AB, CD, EF (the cableway), GH and JK (fig. 3.2.a.). A peg and nail had already been located at the cableway on each bank; the other four cross-sections two upstream and two downstream of the cableway, were fixed by driving a peg and nail into the left bank at a suitable position. Equipment and personnel were organised such that all five cross-sections could be sounded in one day at a convenient low flow. Sounding was carried out from an eight foot long fibre-glass dinghy by means of a ten foot long pole graduated in tenths of a foot. At the cableway it was possible to use the pulley and traversing cable to locate the dinghy in the required position. The left bank peg, E, was taken as zero chainage and soundings made every 10 feet across the section. At the other four sections a 1000 feet line, marked at 5 feet intervals by coloured tags, was stretched across the river from the left bank and secured to a peg and nail driven into the right bank. The dinghy was then manoeuvred across the section and soundings made at 10 feet intervals.

At all sections the elevations of the left and right bank pegs above the water-surface were measured using an ordinary surveying level. Water-surface slope across each section was assumed to be zero and the level of the right bank peg and all soundings were related to the level of the left bank peg. The banks of each section were surveyed, relating all levels to the left bank peg.

It was then necessary to reduce the levels of the left bank pegs of each section to a common datum. During the construction of the gauging station the Northumberland and Tyneside River Board had used a temporary bench mark on the corner of the concrete plinth of the stanchion beside the gauging hut. Using this mark the left benk peg, E, at the cableway was established as a T.B.M. (66.91 ft. A.O.D.) for the survey. The elevations A.O.D. of the other left bank pegs A, C, G and J were determined by careful levelling from peg E. All cross-sections were then related to O.D. as shown in figs. 3.3.a., 3.3.b., 3.3.c., 3.3.d. and 3.3.e.

Attempts were made to check the assumption of zero cross-sectional water-surface slope but sighting distances proved too great for the equipment used.

In order to detail further the bed and, later, to be able to apply the various bed load formulae it was decided to divide the "channel" total wetted perimeter, P_t, into "bed", that part of the perimeter composed of gravel etc. P_b, and "bank", that part of the perimeter composed of bushes, grass etc., P_w. That is:-

$$P_t = P_b + P_w$$

Fig.3.3.a. Cross-sectional profile of section AB

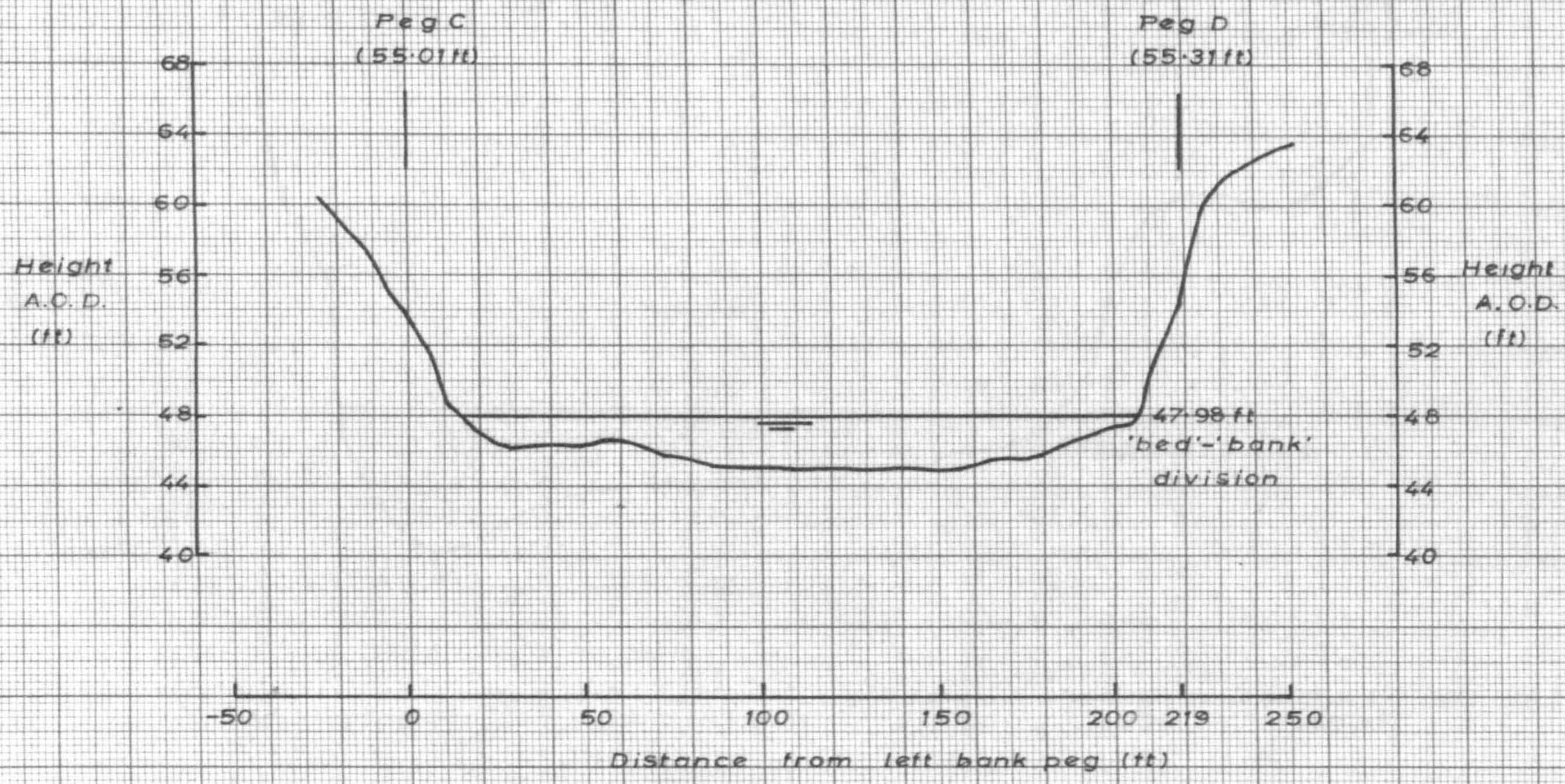
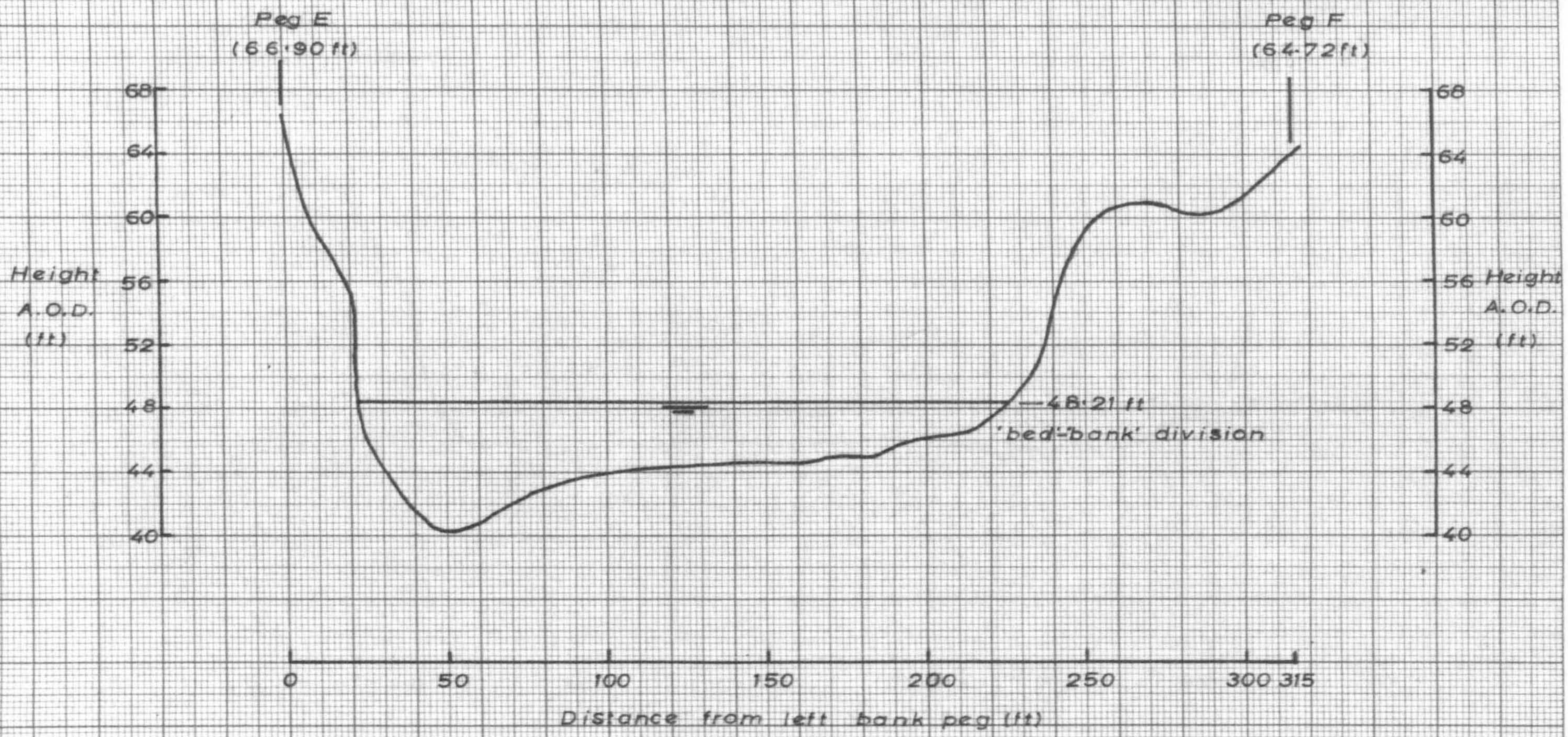


Fig.3.3.b. Cross-sectional profile of section CD



Staff gauge zero: 46·25 ft A.O.D.

Fig. 3.3.c. Cross-sectional profile of section EF (cableway section).

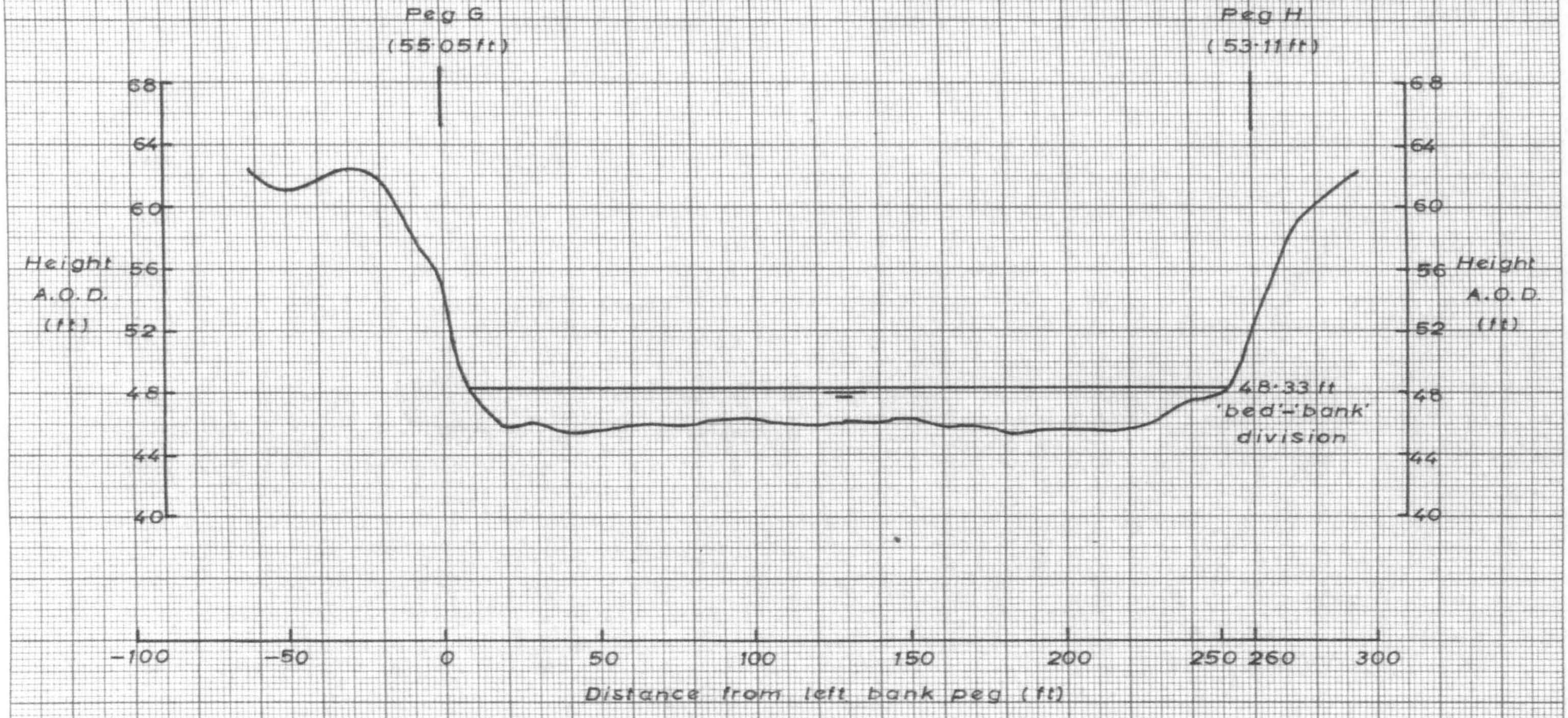


Fig.3.3d. Cross-sectional profile of section GH

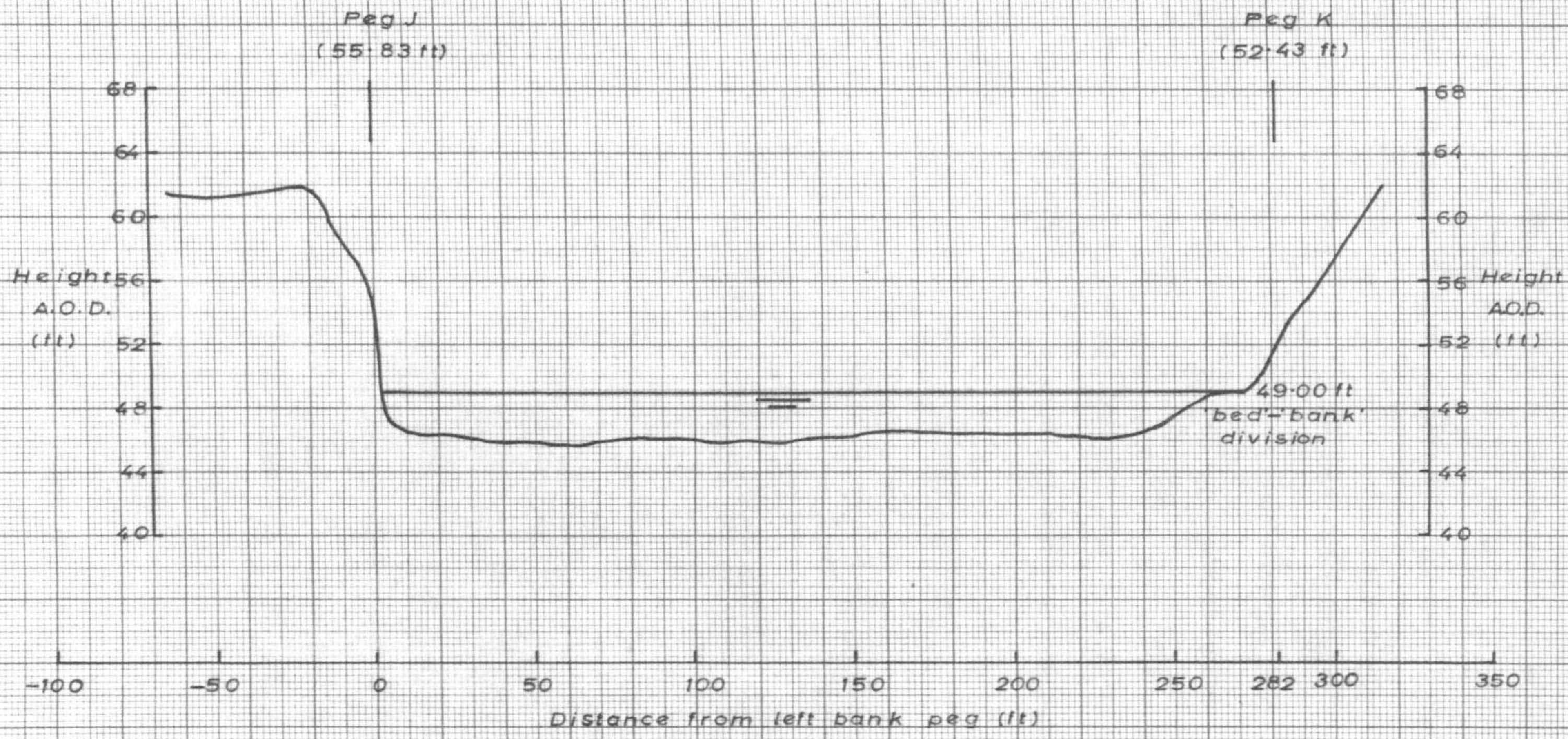


Fig. 3.3.e. Cross-sectional profile of section JK

Accordingly, field observations of the elevation and chainage of the bed-bank divisions were made at each section. The elevations of the bed-bank divisions were found to be almost equal on each side of each section and it was possible to ascribe to each section a particular water-level at which the "bed" was just submerged, i.e. at which P_t equalled P_b. This level is indicated in figs. 3.3.a. to 3.3.e. and is given in the following table with the corresponding water-surface width or "bed" width, the mean "bed" level and the wetted perimeter of the "bed", P_b. Mean "bed" level was obtained by dividing cross-sectional area by water-surface width and subtracting this mean depth from the water-surface level.

Table 3.3.a. "Bed" details at main cross-sections

Section	Water level (ft. A.O.D.)	"Bed" width (ft)	Mean "bed" level (ft. A.O.D.)	"Bed" wetted perimeter P_b (ft)
AB	47.71	141	43.81	149
CD	47.98	193	45.83	199
EF	48.21	203	44.29	211
GH	48.33	244	46.19	249
JK	49.00	270	46.50	272

Cross-sectional areas at one foot intervals of water level were measured by planimeter on sections drawn to a large distorted scale, 10 horizontal to 1 vertical. Wetted perimeters were measured using a map measurer. Figs. 3.3.f. and 3.3.g. show the variation of total cross-sectional area, A_t, and total wetted perimeter, P_t respectively, with water level, H, for all five sections.

A form of tacheometric traverse, using a theodolite and vertical levelling staff to measure distances and horizontal angles, was carried out to fix the positions of the main cross-sections relative to each other. Arbitrary co-ordinates were assigned

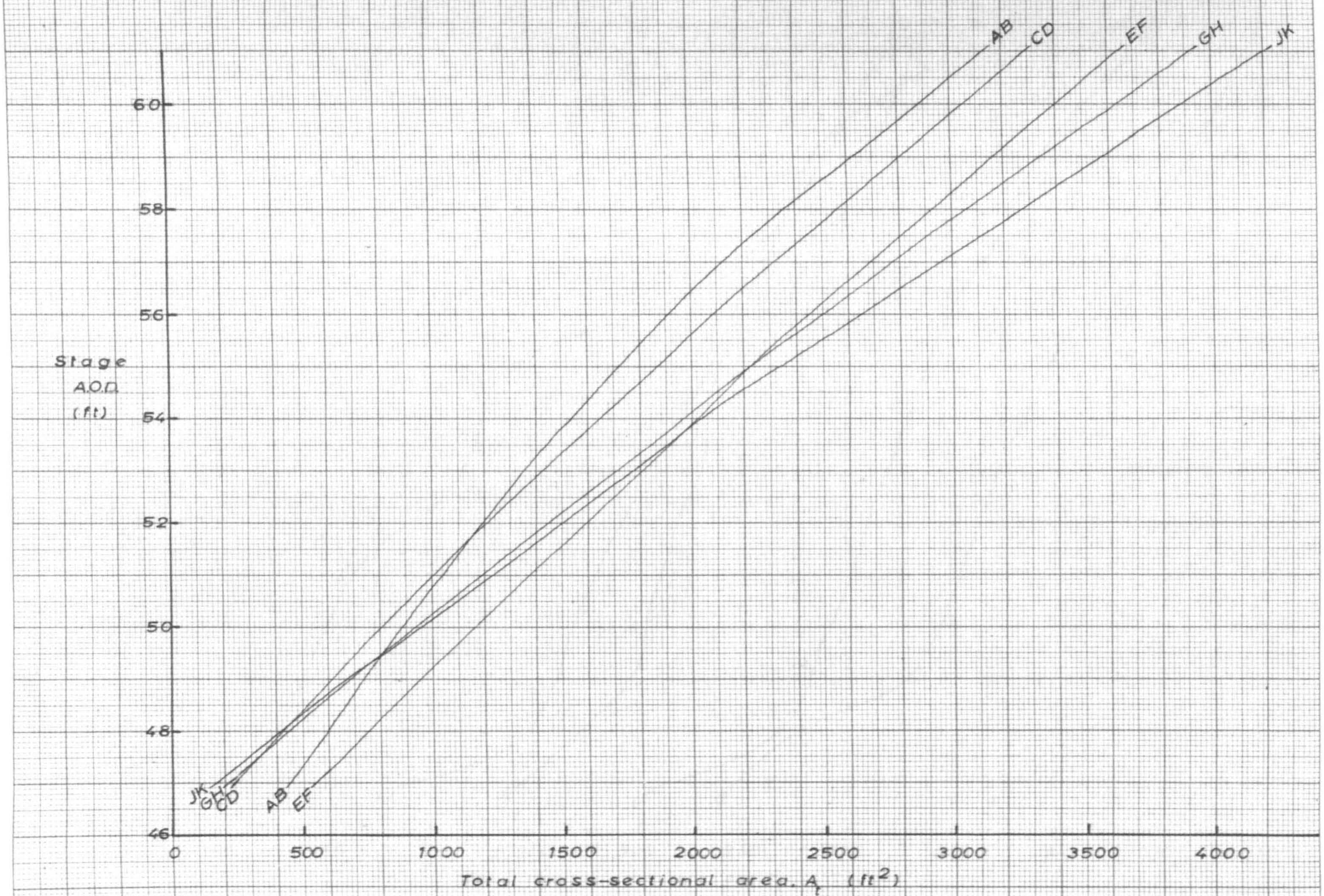


Fig. 3.3.1. Area-stage relationships of main cross-sections

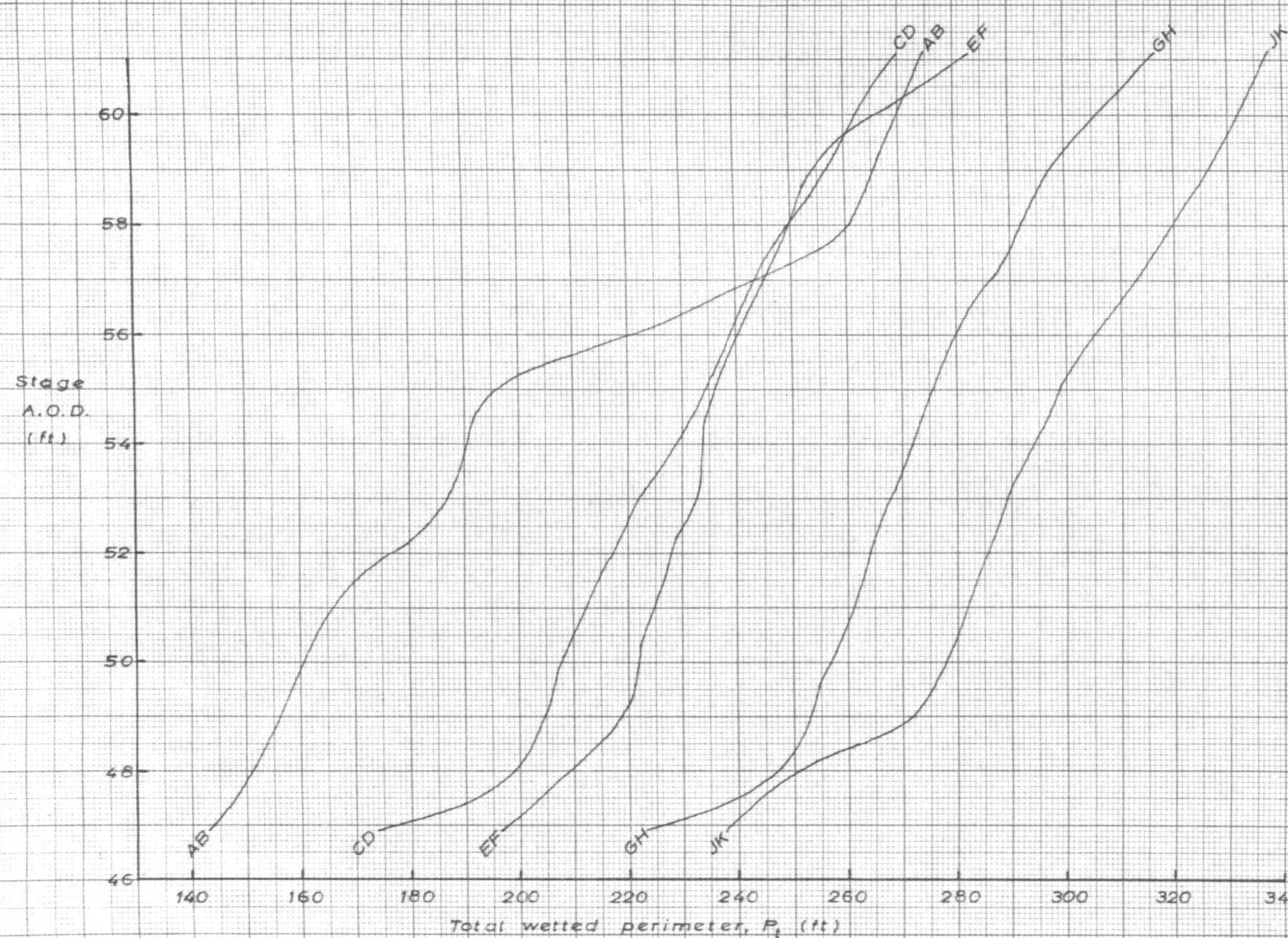


Fig.3.3.g. Wetted perimeter–stage relationships of main cross-sections.

to peg K and the co-ordinates of all other pegs calculated. As a check on the accuracy of the survey the pegs A and J were located on a 25 inches to the mile Ordnance Survey map of the area using fences etc. and the line AJ measured on the map as 2755 ft. When calculated from their co-ordinates the distance between A and J was 2761.4 ft.

The distance between the sections was taken to be the distance between the mid-points of the river "bed" at each section. These mid-points were easily calculated from the survey details and the distances determined as follows:-

Section	:	AB		CD		EF		GH		JK
Distance between sections (ft)	:		765.5		1128.2		535.3		321.8	

The total distance between sections AB and JK was 2750.8 ft or 0.520 miles.

3.3.2. Supplementary Cross-sections

From figs. 3.3.a to 3.3.e. and table 3.3.a. it can be seen that there is considerable variation in the shapes, "bed" widths, mean "bed" levels etc. of the five main cross-sections AB, CD, EF, GH, JK. To obtain a more complete picture of the river reach it was decided to supplement these sections with a number of intermediate sections. A further eighteen sections, about 150 feet apart, were located by pegs and nails driven into the left bank; 3 sections upstream of AB, 13 between AB and JK and 2 downstream of JK. Distances between sections were measured by tape along the left bank and proportionately adjusted such that the distances between the five main sections agreed with those determined in the tacheometric survey. Peg elevations were fixed by levelling from the T.B.M, peg E, at the cableway.

Inspection of the five main cross-sections showed that the mean "bed" level of each section could be obtained by the addition of 0.36 ft to the mean of the "bed" levels at distances across the "bed" of 1/5, 2/5, 3/5 and 4/5 of the "bed" width. In this way detailed surveys of the bed and banks of all eighteen sections could be avoided, and it was decided to sound at these four positions and at the mid-point of the "bed". The position and depth at the deepest point were also noted at each section. Sounding was effected in a manner similar to that used with the five main sections except that the line was secured to a steel rod temporarily driven into the right bank. At each section the water level was related to the corresponding peg height, the "bed" width noted and the location of the sounding positions determined. Mean "bed" levels and the "bed" levels at the deepest point and mid-point of the "bed" were then calculated.

3.3.3. Longitudinal Profiles

Details of the bed at all 23 cross-sections are given in fig. 3.3.h. (in pocket).

The plan view, drawn with a straight centre line to facilitate plotting of the profiles beneath it, clearly shows that the bed width increases gradually downstream over the length of the reach. The thalweg profile is uneven and tends to meander within the bed, the deepest parts occurring near the banks. It can be seen that the highest point on the thalweg of the "bar" forming the control section of the gauging station (or the outlet of the preceding "pool") is located at JK while the highest point of the mean "bed" level profile occurs further downstream.

An attempt was made to assign an average bed slope to the whole length of reach between sections 1 and 18, a distance of 3531 feet or 0.73 miles. The method of least squares was used to determine the best straight line through the mean "bed" levels of all 23 sections plotted against distance downstream of section 1. The slope of this line was calculated to be 0.171×10^{-3} upwards in a downstream direction. It is evident from this result that it is impossible to describe the river channel by its bed slope over a reach of this length. Possibly, the bed slope of a "pool-bar" type river could be taken as the average slope through the tops of two or more successive "bars", but the physical significance of this slope is doubtful when applied to a single cross-section in hydraulic calculations involving, for example, sediment transport or flood routing. If the slope of a river at a single cross-section must be described then the water-surface (or energy-surface) slope would have to be used; this, however, will vary considerably with stage, especially in "pool-bar" type rivers.

3.4. Hydraulic Characteristics of the Reach

In order to apply the bed load theories described in section 4 it was necessary to determine the relationship between stage, discharge and the water-surface and energy-surface slopes. The velocity distribution and tractive force distribution at the cableway section were also investigated.

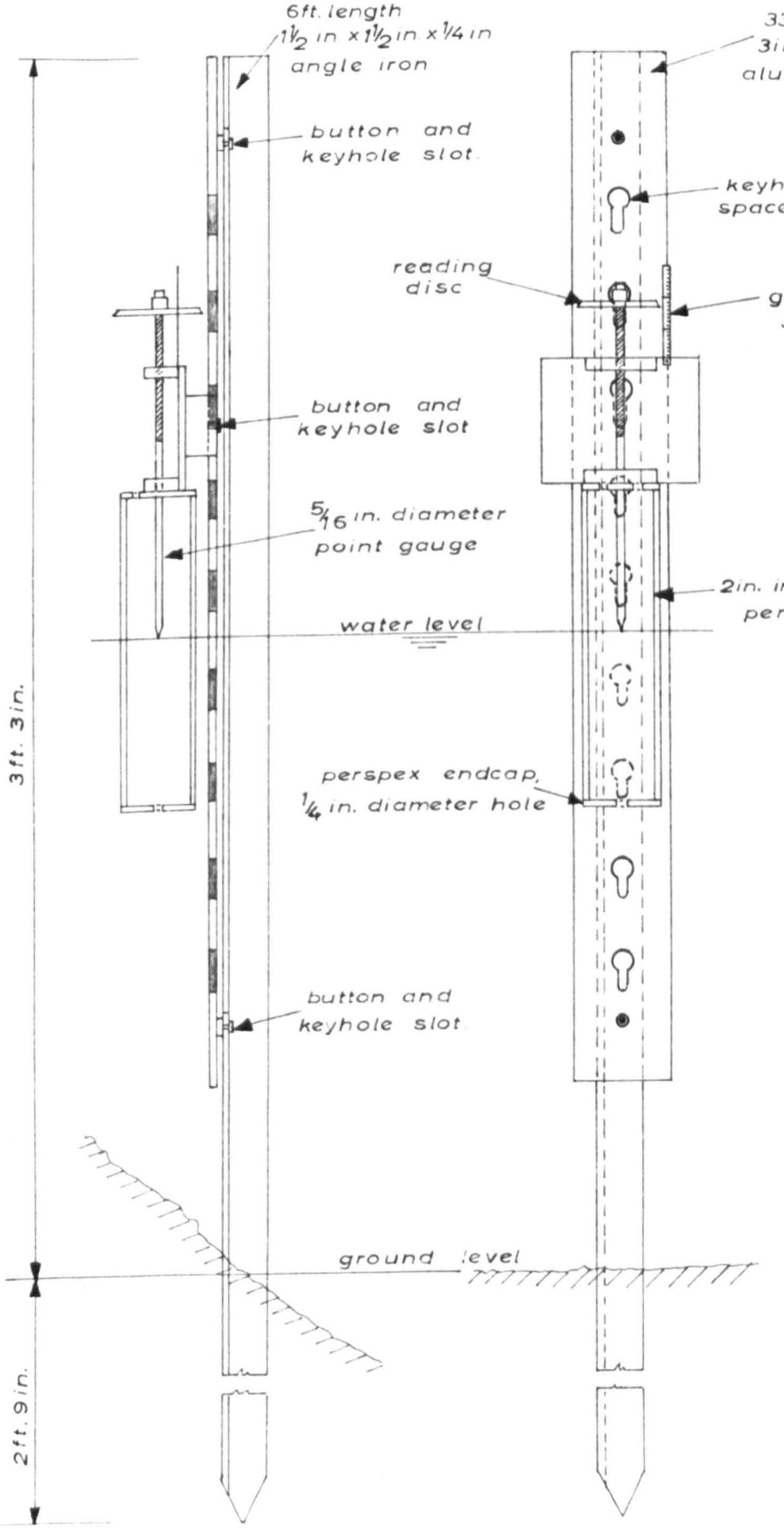
Measurement of water-surface slope within the reach involved the determination of water-surface levels at three sections AB, EF (the cableway section) and JK.

3.4.1. Measurement of Water-surface Level

Measurement of the water-surface levels at EF presented little difficulty. The Fisher and Porter punched tape recorder housed in the cableway hut indicated to one hundredth of a foot the river stage above staff gauge zero. This datum was determined by levelling from the T.B.M., peg E, to be 46.25 ft A.O.D. The height of water-surface A.O.D. could then be obtained easily from the reading of the recorder.

For the measurement of water-surface levels at sections AB and JK staff gauges located on the banks would have been insufficiently accurate, would have been required to be robust and elaborately anchored and might possibly have been aesthetically undesirable. To overcome these difficulties a modified point gauge arrangement as shown in fig. 3.4.a. was designed. The tip of the point gauge could be moved over a range of 3 inches inside a 2 inch internal diameter perspex tube. The tube was partially blanked off at the lower end by a perspex cap with a central ¼ inch diameter hole, thus considerably damping surface ripples. Air vents were provided at the upper end of the tube. The location of the point of the gauge was indicated by the position of a disc on the top end of the moving stainless steel rod relative to a fixed 3 inch long metal scale. By means of a button and keyhole slot arrangement the point gauge could be attached to one of nine, numbered positions, 3 inches apart, on a flat aluminium support. In the top position the level of the top of the aluminium support coincided with that of the top, or zero, of the 3 inch scale. The aluminium support itself could be fixed by two button and keyhole slots to any of several 6 foot lengths of rust-proof painted 1½ in x 1½ in x ¼ in angle iron driven into the river bank. The range of water level covered by each length of angle iron was thus 2¼ feet.

For ease of access to the water-surface the left bank was chosen for the location of the measuring points and eight numbered lengths of angle iron were positioned at the required height, allowing a small overlap, at both sections AB and JK. The height A.O.D. of the top of the aluminium support when fixed to each angle



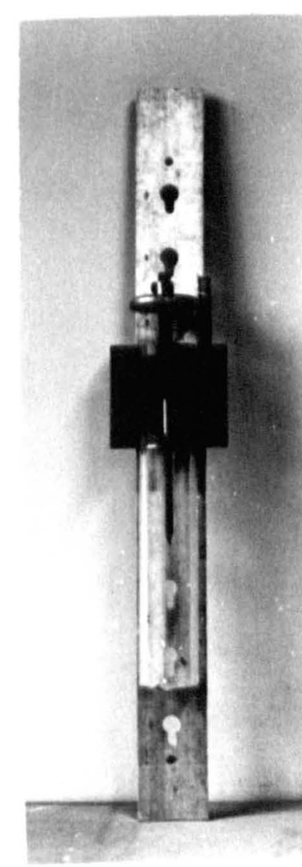


Fig. 4·3. a.

Point gauge for measurement of water-surface level.

was determined by levelling from the appropriate peg, A or J. (These levels were later checked after several high flows and found to be unchanged.) Knowing the length between point and disc it was then possible to draw up a table for each section giving the level A.O.D. of the point, when the reading on the 3 inch metal scale was zero, for any angle iron and any position of the gauge on the aluminium support. In this way the measurement of the water-surface level was reduced to the simple procedure of noting:-

1) the number of the angle iron,

2) the number of the position of the gauge on the aluminium support,

and 3) the reading of the disc against the 3 inch metal scale to the nearest 1/20 inch.

The accuracy of the levels determined by this method depends upon the accuracy of the levelling. At high flows macroturbulence near the banks caused surface levels to surge and it was found necessary to take the mean of three or four readings of the point gauge.

The River Tyne at Bywell is subject to rapid changes of stage and the following procedure was developed to enable the water levels at A, E and J to be measured by one person within as short a period of time as was possible. Advantage was taken of the Fisher and Porter instrument which recorded the level at E on punched tape at 0, 15, 30 and 45 minutes past each hour. A few minutes before the instrument was due to record, the water level at A was measured. The observer then moved as quickly as possible to J (a vehicle could be used for part of the distance) to measure the level there. The time between the two measurements was usually about 7 minutes during which time the level at the cableway section, EF, had been recorded. This level was noted together with the preceding and succeeding punched tape recordings which were used subsequently to determine the rate of rise or fall of stage.

Thirty-seven sets of readings were made (table 3.4.a) over a range of stage at E above staff gauge zero from 1.76 ft to 13.01 ft. Four longitudinal profiles are shown in fig. 3.4.b. Profile No. 1 was determined on the day that the five main cross-sections were sounded, during which time flow conditions remained steady. It included water levels measured at C and G, which can be seen to lie on straight lines AE and EJ, respectively. The water-surface profiles in the reach were therefore considered to consist of two straight lines whose slopes become more nearly equal at higher stages.

3.4.2. Computation of Water-surface and Energy-surface Slopes

To determine the energy-surface levels at each section, discharge values corresponding to each of the 37 sets of water level measurements were obtained from the stage-discharge curve of fig. 3.4.e. Mean velocities, V, at each of the sections AB, EF and JK were calculated using the area-stage curves of fig. 3.3.f. Assuming a Coriolis coefficient of unity, the velocity head, $V^2/2g$, was calculated and added to the water-surface level to give the energy-surface level (table 3.4.a.).

Surface slope at the cableway section, EF, was calculated as the geometric mean of the slopes from AB to EF and from EF to JK. Examination of a number of laboratory channel drawdown profiles, to which the reach profiles approximated, showed that the assumption of the geometric mean rather than the arithmetic of the upstream and downstream slopes should be more accurate. The geometric mean was in all cases less than the arithmetic mean; since the flatter slope extended over the greater length of the reach (fig. 3.4.b.) this was considered a justifiable bias.

Water-surface slopes, S_w, and energy-surface slopes, S_e, for the 37 profiles are given in table 3.4.a.

Table 3.4.a.

h_e = stage at section EF above staff gauge zero (46.25 ft. A.O.D.) (ft)

H_a = water-surface level at section AB (ft. A.O.D.)

H_e = water-surface level at section EF (ft. A.O.D.)

H_j = water-surface level at section JK (ft. A.O.D.)

S_w = water-surface slope at section EF

E_a = energy-surface level at section AB (ft. A.O.D.)

E_e = energy-surface level at section EF (ft. A.O.D.)

E_j = energy-surface level at section JK (ft. A.O.D.)

S_e = energy-surface slope at section EF

dH = rate of rise (+) or fall (-) of stage at section EF (ft/hr)

Distance between sections AB and EF = 1894 ft.

Distance between sections EF and JK = 857 ft.

Profile No.	h_e	Water-surface				Energy-surface				dH
		H_a	H_e	H_j	$S_w x10^3$	E_a	E_e	E_j	$S_e x10^3$	
1	2.38	48.80	48.63	47.99	0.259	48.83	48.65	48.06	0.256	0.000
2	6.62	53.88	52.87	51.68	0.860	54.39	53.24	52.28	0.825	-0.042
3	5.62	52.57	51.87	50.76	0.692	52.94	52.10	51.22	0.675	-0.100
4	11.93	59.88	58.24	57.15	1.049	61.50	59.77	58.60	1.117	-0.100
5	4.11	51.04	50.36	49.41	0.631	51.19	50.46	49.69	0.588	+0.860
6	4.83	51.73	51.08	50.10	0.626	51.96	50.43	50.43	0.599	+0.140
7	3.74	50.42	49.99	49.18	0.463	50.53	50.07	49.37	0.444	0.000
8	4.36	51.32	50.61	49.68	0.637	51.49	50.74	49.95	0.604	+0.520
9	1.76	48.11	48.01	47.52	0.174	48.12	48.02	47.55	0.170	0.000
10	1.87	48.28	48.12	47.66	0.212	48.29	48.14	47.70	0.201	+0.260

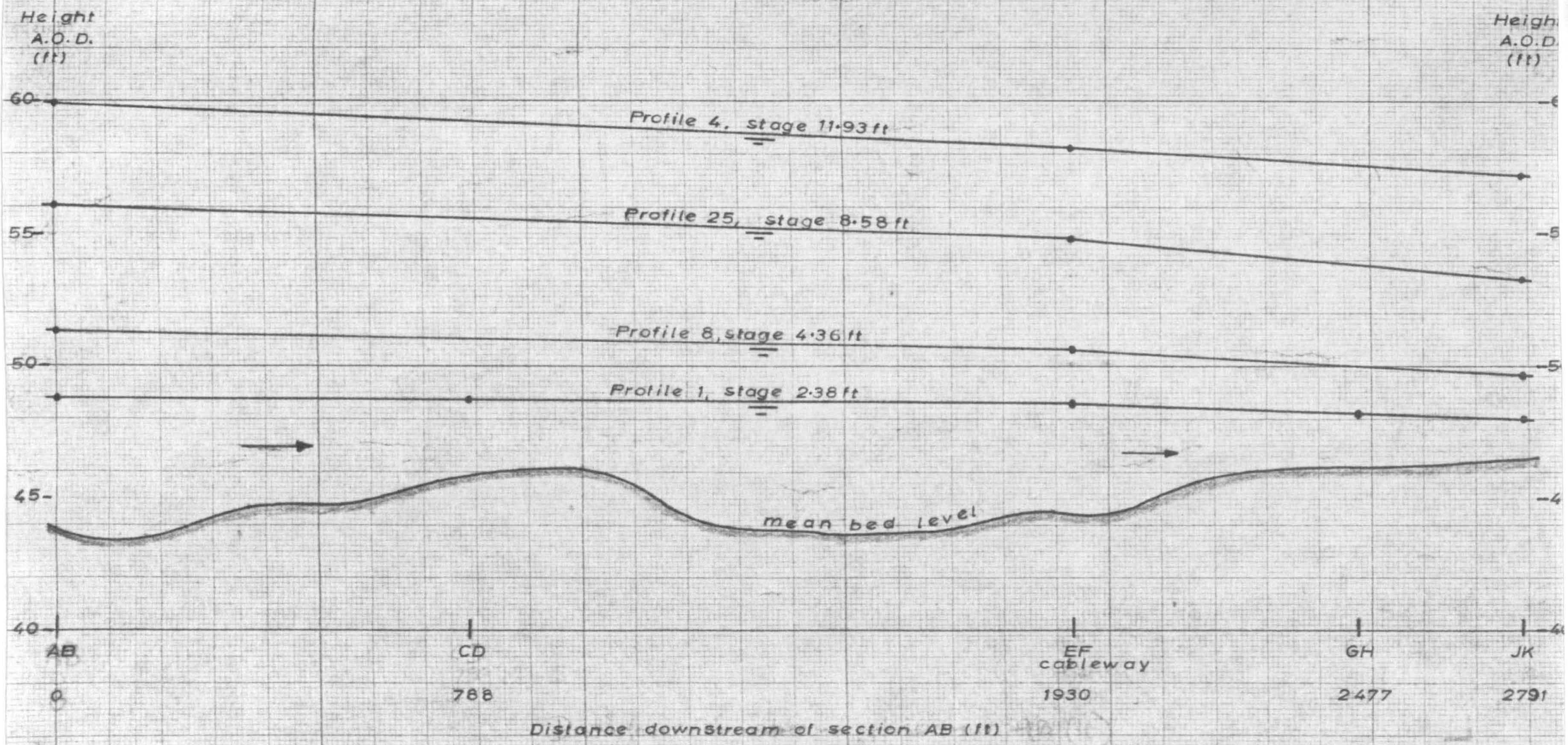


Fig. 3.4.b. Water surface profiles of River Tyne at Bywell.

Table 3.4.a. (cont.)

Pro-file No.	h_e	Water-surface				Energy-surface				dH
		H_a	H_e	H_j	$S_w x10^3$	E_a	E_e	E_j	$S_e x10^3$	
11	2.19	48.61	48.44	47.86	0.247	48.63	48.46	47.92	0.235	0.000
12	2.00	48.38	48.25	47.71	0.208	48.39	48.27	47.76	0.174	0.000
13	6.12	53.28	52.37	51.23	0.799	53.70	52.67	51.73	0.773	-0.160
14	5.93	52.90	52.18	51.04	0.711	53.32	52.45	51.53	0.702	-0.220
15	5.52	52.54	51.77	50.65	0.729	52.86	51.99	51.07	0.702	-0.320
16	2.83	49.36	49.08	48.43	0.335	49.41	49.11	48.54	0.324	-0.040
17	7.50	54.87	53.75	52.46	0.943	55.63	54.26	53.21	0.007	-0.180
18	7.16	54.50	53.41	52.07	0.949	55.12	53.87	52.80	0.908	-0.400
19	6.76	54.05	53.01	51.85	0.862	54.59	53.40	52.57	0.780	-0.440
20	6.42	53.63	52.67	51.51	0.828	54.11	53.01	52.09	0.790	-0.360
21	6.17	53.39	52.42	51.24	0.840	53.71	52.73	51.64	0.811	-0.420
22	3.25	49.86	49.50	48.77	0.402	49.93	49.55	48.99	0.362	-0.040
23	3.51	50.14	49.76	48.97	0.430	50.26	49.82	49.28	0.378	-0.120
24	7.55	54.94	53.80	52.55	0.937	55.61	54.32	53.31	0.896	+0.820
25	8.58	56.11	54.83	53.50	1.024	57.05	55.54	54.44	1.011	+0.420
26	6.89	54.22	53.14	51.84	0.930	54.78	53.55	52.51	0.888	-0.320
27	6.76	54.07	53.01	51.74	0.911	54.60	53.40	52.38	0.869	-0.220
28	6.66	53.94	52.91	51.63	0.902	54.46	53.29	52.27	0.853	-0.200
29	6.51	53.76	52.76	51.52	0.874	54.26	53.11	52.12	0.837	-0.180
30	6.43	53.64	52.68	51.46	0.850	54.12	53.02	52.03	0.819	-0.080
31	7.50	54.99	53.75	52.39	1.019	55.63	54.26	53.15	0.968	-0.480
32	5.93	53.06	52.18	51.02	0.793	53.46	52.45	51.52	0.761	-0.380
33	13.01	61.02	59.26	57.85	1.236	62.05	61.07	59.70	1.259	-1.060
34	7.18	54.24	53.43	52.15	0.800	54.90	53.89	52.86	0.800	+0.540
35	5.03	51.99	51.28	50.26	0.667	52.25	51.45	50.62	0.639	-0.180
36	9.01	56.69	55.26	53.93	1.082	57.68	56.05	54.94	1.056	+0.420
37	8.67	56.35	54.92	53.55	1.098	52.27	55.64	54.50	1.070	-0.680

3.4.3. Variation of Surface Slopes with River Stage

For correlation of surface slopes with river stage a standard multiple regression programme was used on the KDF9 computer of the University of Newcastle upon Tyne Computing Laboratory. By means of this programme trial correlations of several variables could be obtained rapidly together with the correlation coefficient and significance level of each variable.

After several trials it was found that the best correlations were obtained by plotting the surface slopes against the logarithm of the stage at E above a certain datum. The resulting equations were:-

$$S_w \times 10^3 = 1.504 \log_{10} (H_e - 45.55) - 0.449 \quad \text{Cor}^n.\ \text{coeff}^{nt}. = 0.983$$

$$S_e \times 10^3 = 1.869 \log_{10} (H_e - 44.35) - 0.918 \quad \text{Cor}^n.\ \text{coeff}^{nt}. = 0.990$$

These equations and the observed data are plotted in figs. 3.4.c and 3.4.d. respectively.

The rate of rise or fall of river stage, dH, was introduced into the regression programme as a third variable to determine whether any improvement of correlation could be obtained. The above equations were modified to:-

$$S_w \times 10^3 = 1.497 \log_{10} (H_e - 45.55) - 0.012dH - 0.444 \quad \text{Cor}^n.\ \text{coeff}^{nt}. = 0.983$$

$$S_e \times 10^3 = 1.865 \log_{10} (H_e - 44.35) - 0.004dH - 0.915 \quad \text{Cor}^n.\ \text{coeff}^{nt}. = 0.990$$

The t-value significance levels of the variable dH in these equations were only 35% and 15% respectively. It was concluded that, with the observed information, evidence of the effect of rate of rise or fall of river stage on water-surface or energy-surface slope could not be detected.

3.4.4. Stage-discharge Relationship

The Northumbrian River Authority have established a rating curve (fig. 3.4.e) for the cableway section from gaugings carried out since January, 1965, two years after the cessation of gravel extracting operations downstream of the cableway. The equation to the curve is:-

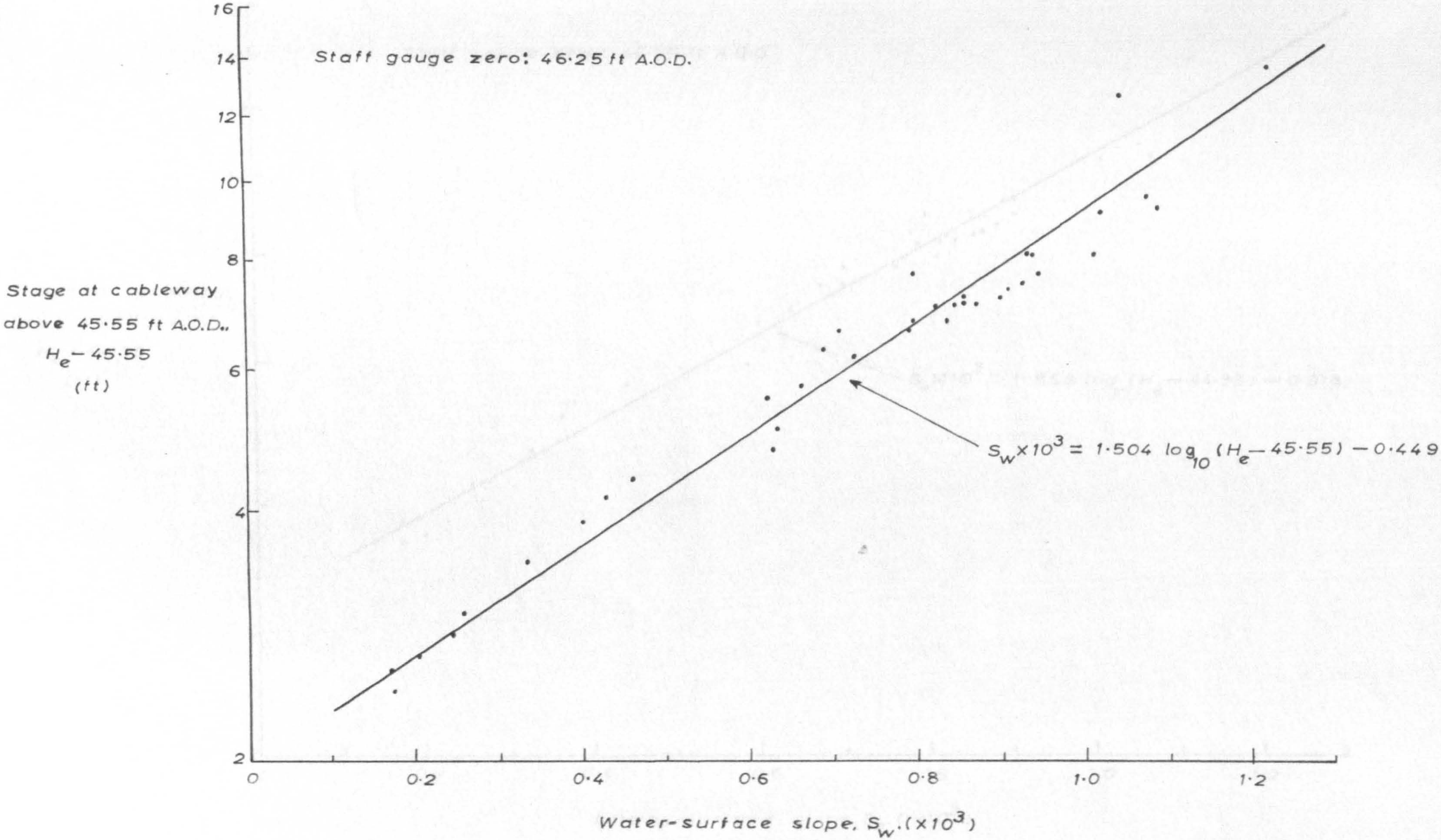


Fig.3.4.c. Relationship between stage and water-surface slope at Bywell.

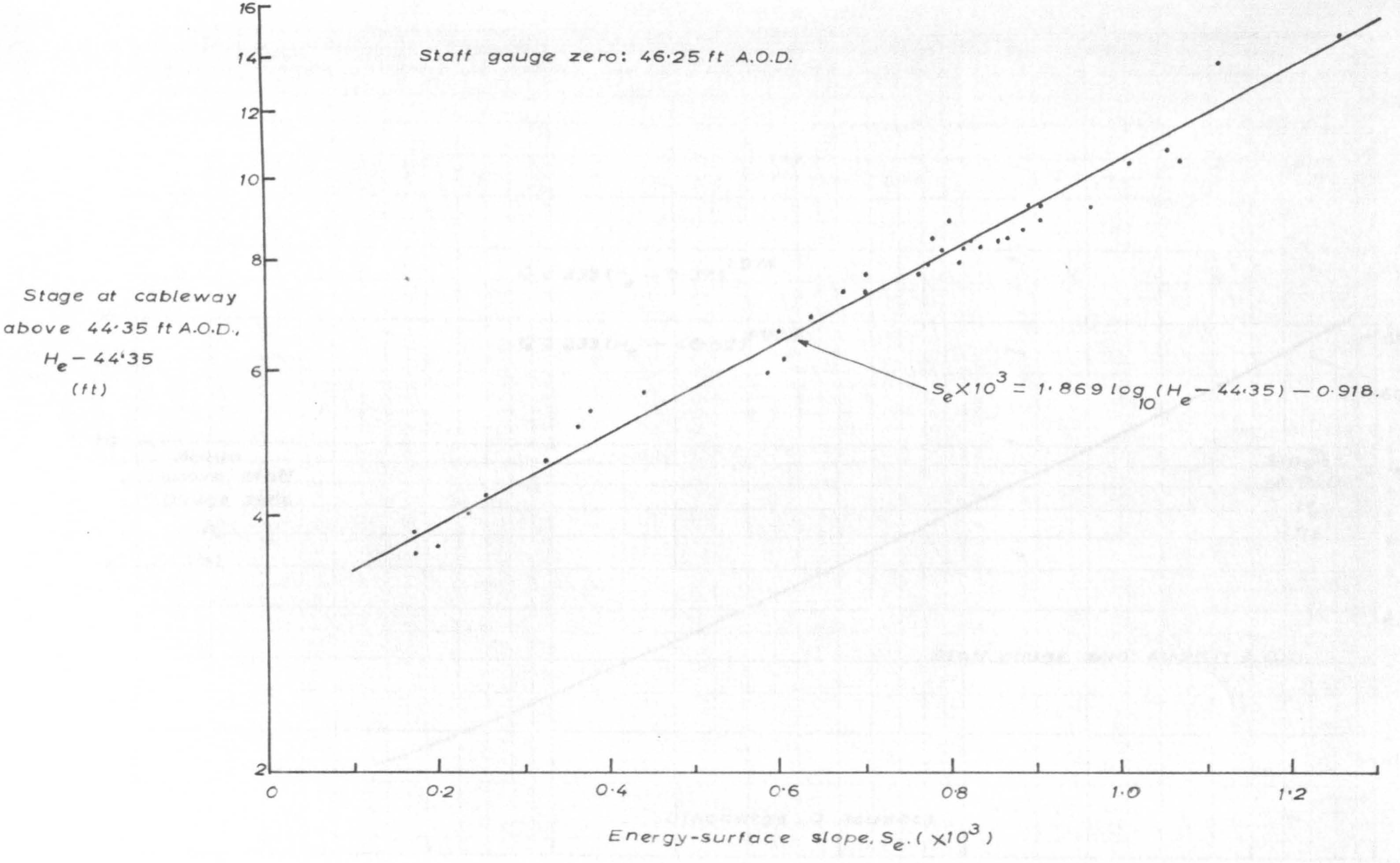


Fig. 3. 4. d. Relationship between stage and energy-surface slope at Bywell.

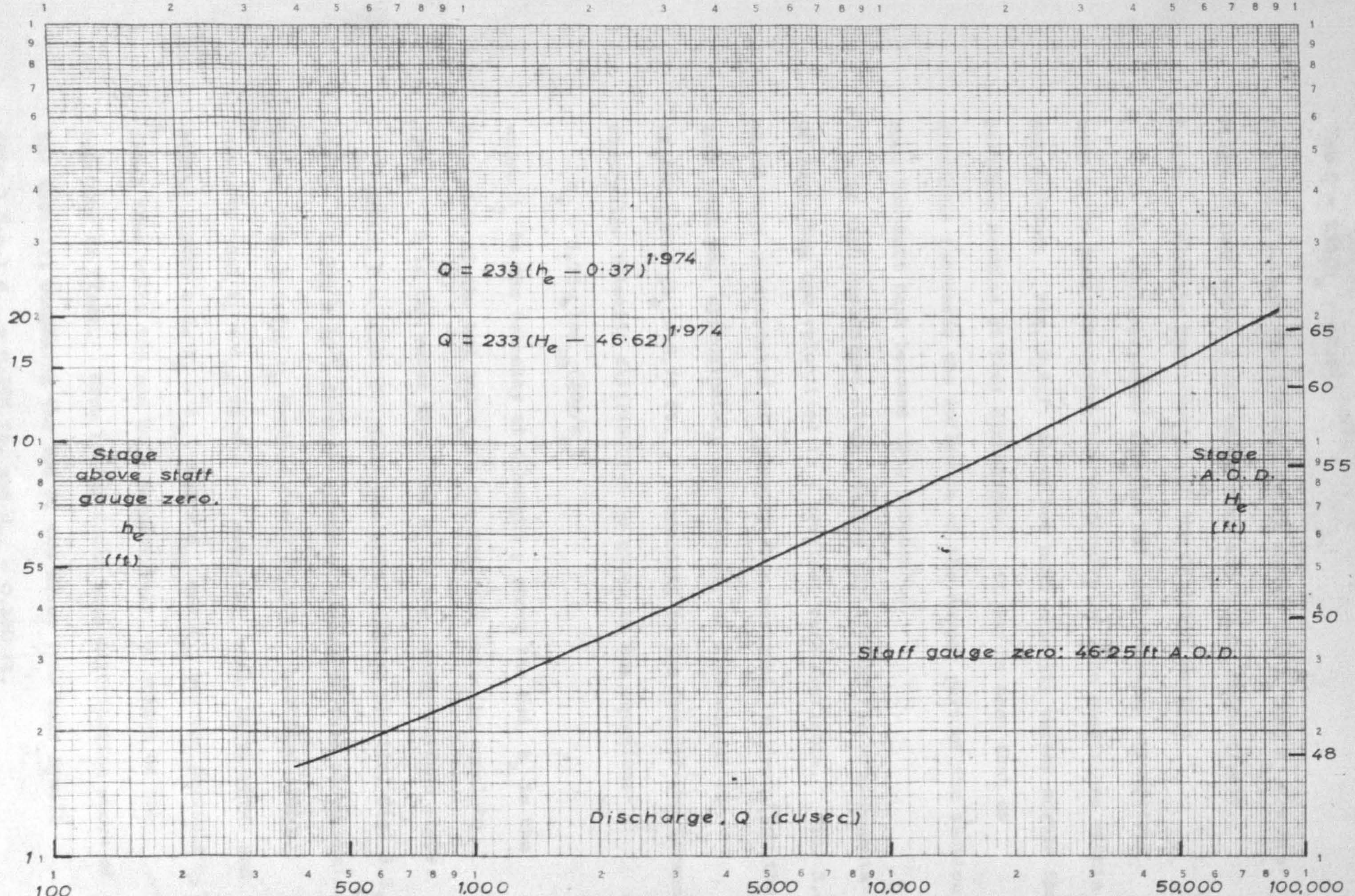


Fig. 3.4.e. Stage-discharge relationship for Bywell cableway section.

$$Q = 233(h_e - 0.37)^{1.974}$$

$$\text{or } Q = 233(H_e - 46.62)^{1.974}$$

in which Q is the river discharge in cusecs and h_e and H_e are the stage in feet above staff gauge zero and A.O.D. respectively.

3.4.5. Velocity Distribution at the Cableway Section

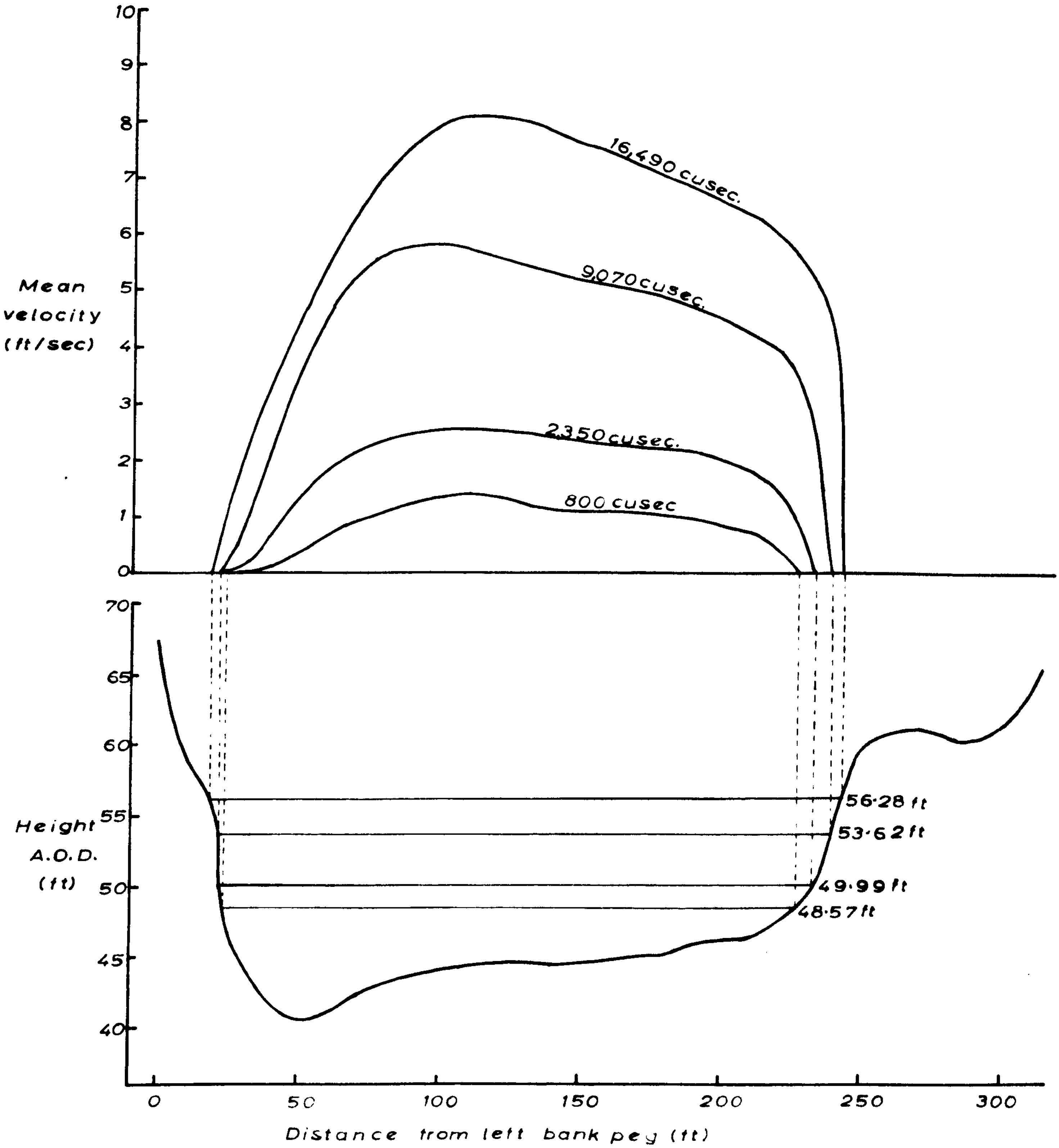
In the application of several bed load formulae assumptions have to be made concerning the uniformity of flow conditions within the section. Fig. 3.4.f. shows how mean velocity varies across the cableway section at four discharges. It can be seen that as discharge increases the influence of the deeper part of the section near the left bank becomes more noticeable.

At high discharges stage changes rapidly and few measurements of more than two velocities in a vertical were available. Fig. 3.4.g. shows four measurements of velocity in a vertical 130 feet from the left bank peg, approximately mid-channel, at a discharge of 12,200 cusec (h_e = 8.73 ft.). According to the Prandtl-Von Karman theory the velocity distribution is given by the equation:-

$$V_y = 5.75 u_* \log_{10}(30y/k_s)$$

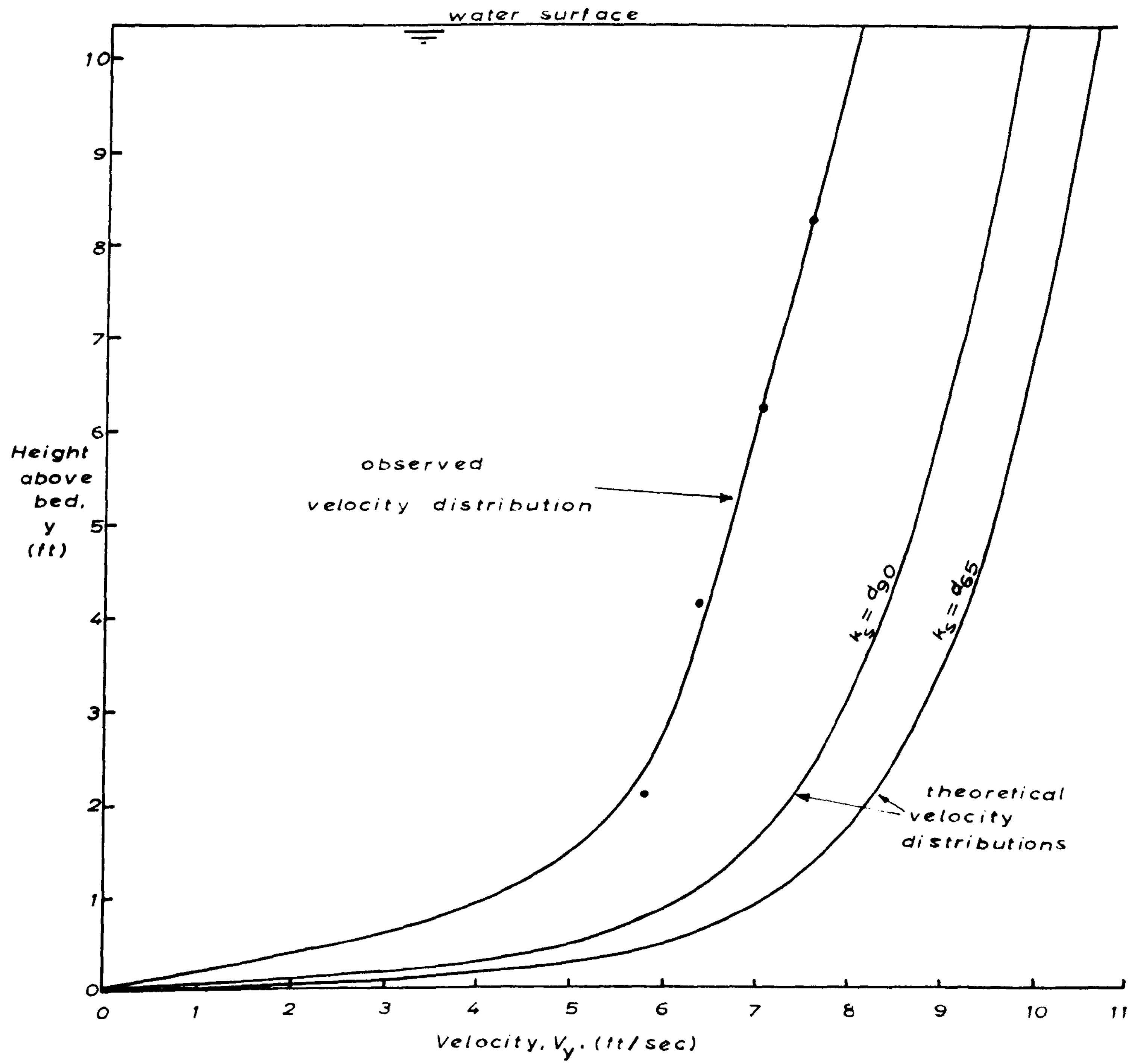
where V_y is the velocity at a height y above the bed, k_s is the roughness of the bed and u_* is the friction velocity equal to $\sqrt{gDS_e}$. D is the water depth and S_e is the energy-surface slope.

Taking D = 10.35 ft. from the cross-section profile, fig. 3.3.c., and $S_e = 1.000 \times 10^{-3}$ from the graph of energy-surface slope against stage, fig. 3.4.d., u_* becomes 0.577 ft/sec. EINSTEIN (1950) suggests that $k_s = d_{65}$ of the bed material, while MEYER-PETER and MÜLLER (1948) take $k_s = d_{90}$, where d_{65} and d_{90} are the particle sizes than which 65% and 90%, respectively, by weight of the sediment is finer. From the particle size distribution curve of the material composing the bed of the River Tyne near Bywell (fig.3.5.e.) d_{65} = 0.196 ft. and d_{90} = 0.350 ft.



Staff gauge zero: 46·25 ft A.O.D.

Fig. 3.4.f. Velocity distributions at Bywell cableway section



Location: 130 ft from left bank peg.

Stage: 8·73 ft above staff gauge zero (46·25 ft A.O.D.)

Discharge: 12,200 cusec.

Depth, D : 10·35 ft.

Energy-surface slope, S_e : $1{\cdot}000 \times 10^{-3}$

Prandtl–Von Karman Law:- $V_y = 5{\cdot}75 u_* \log_{10}(30y/k_s)$

where $u_* = \sqrt{gDS_e} = 0{\cdot}577$ ft/sec.

k_s is the roughness height, $d_{65} = 0{\cdot}196$ ft or $d_{90} = 0{\cdot}350$ ft.

Fig.3.4.g. Observed and theoretical vertical velocity distributions.

The resulting theoretical velocity distributions for each value of k_s are shown in fig. 3.4.g. The discrepancies between the theoretical and observed distributions indicate the extent to which river banks and bed configuration offer additional resistance to flow.

3.4.6. Tractive Force Distribution at the Cableway Section

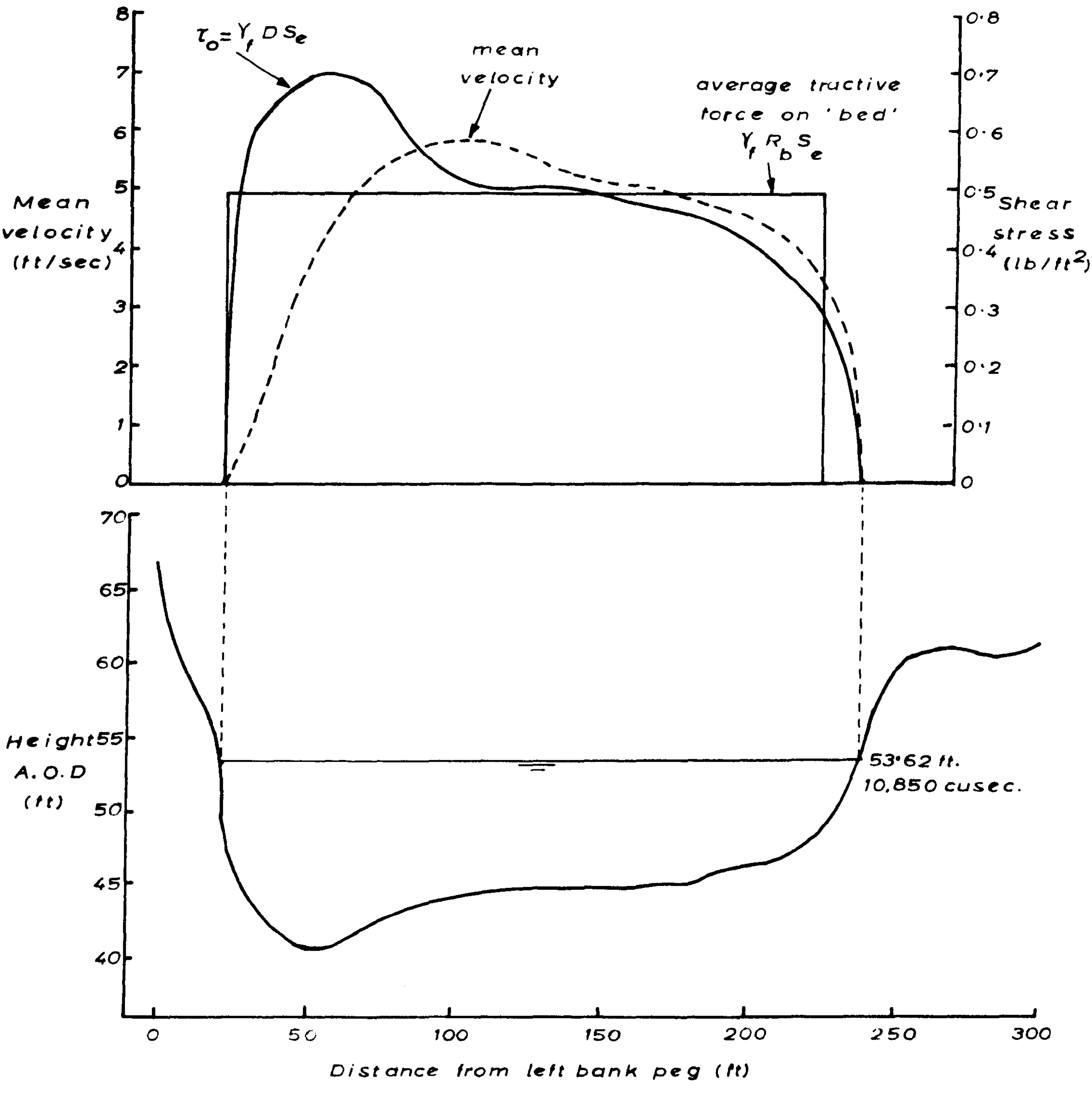
The average tractive force acting on the bed in a depth D of water with an energy-surface slope of S_e is given by $\tau_o = \gamma_f DS_e$ (CHOW, 1959) where γ_f is the specific weight of water. The tractive force acting on the bed in a given flow is thus proportional to the depth of flow above the bed. Fig. 3.4.h. shows the theoretical tractive force distribution across the section at a discharge of 10,850 cusec. The measured velocity distribution is also shown and illustrates that the actual tractive force distribution is modified by the shape of the cross-section.

3.5. Bed Material of the Reach

A detailed sediment sampling programme was necessary for the comprehensive description of the material which constitutes the bed of the River Tyne at Bywell. As will be described later in section 4 properties such as particle size distribution and specific gravity have a direct application in the determination of the critical conditions of movement, sediment discharge etc; other properties such as particle roundness, shape and petrography are mainly of interest to geologists and sedimentologists but have been included to provide a complete description.

3.5.1. Methods of Sampling and Analysing River Bed Material

The sampling and analysis of river bed sand presents little difficulty, since only small quantities are required to produce a representative sample. Several instruments, which may be grouped into three classes of drag-bucket or scoop, vertical pipe or cylinder,



Staff gauge zero: 46·25 ft A.O.D.

Fig. 3.4.h. Tractive force distribution at Bywell cableway section.

and clam shell, have been designed for this purpose and are described in Report 14 of the UNITED STATES FEDERAL INTER-AGENCY COMMITTEE ON WATER RESOURCES, SUBCOMMITTEE ON SEDIMENTATION. However, the wide range of size and shape and the variable areal distribution of particles in a gravel-bed river makes sampling considerably more difficult. It is probably for this reason that there exists comparatively little recorded information on such rivers.

There are basically four methods for sampling and analysing coarse sediment:-

1) Volumetric sampling and sieve analysis by weight
2) Areal sampling, measurement of a representative diameter, and frequency analysis by number
3) Areal sampling and sieve analysis by weight. A method used by LANE and CARLSON (1953) in which all stones exposed in an area of one square yard were collected and sieved as a bulk sample. The main disadvantages of this method are its inapplicability when patches of sand are present and the difficulty in finding a representative and reasonably small sampling area, particularly in very coarse material.
4) Photography of a small area of the bed either through a grid or with a measuring scale placed on the bed. Representative diameters of the surface particles can then be estimated from the photograph and a frequency analysis by number plotted.

Methods 1 and 2 were used in the sampling investigations at Bywell and are described in greater detail in sections 3.5.3 and 3.5.4.

All of these methods raise the problem of ascribing a representative diameter to a sediment particle. Despite attempts to standardise nomenclature by authorities such as the AMERICAN GEOPHYSICAL UNION (1947) there are several dimensions in common usage:-

d_1 : major axis

d_2 : intermediate axis — three mutually perpendicular axes

d_3 : minor axis

d_r : arithmetic mean of major, intermediate and minor axes

d_e : geometric mean of major, intermediate and minor axes

d_n : nominal diameter, i.e. the diameter of the sphere of the same volume as the sediment particle

d_s : sieve daimeter, i.e. the length of the side of the square sieve opening through which the particle will just pass.

d_w : sedimentation diameter, i.e. the diameter of the sphere of the same specific gravity and the same terminal uniform settling velocity as the sediment particle in the same sedimentation fluid.

For a perfectly spherical particle all eight diameters are, of course, equal. The nominal diameter has little significance in sediment transport but is sometimes useful in discussing the nature of sediment deposits. Silt and clay sizes are usually described by the sedimentation diameter which COLBY (1963) has shown to be easily related to the sieve diameter. Representative diameters involving any of the three axes of the particle are often used in frequency analysis by number and are discussed in more detail in section 3.4.4.

Sieve analysis is the most convenient and most commonly used method of treating most sediment samples; the grade scale given in table 3.5.a. is recommended by the AMERICAN SOCIETY OF CIVIL ENGINEERS (1962) and is based on sediment sizes described by the sieve diameter.

Table 3.5.a. Sediment grade scale

Class name	Particle size range	
	in	mm.
Very large boulders	160 - 80	4096 - 2048
Large boulders	80 - 40	2048 - 1024
Medium boulders	40 - 20	1024 - 512
Small boulders	20 - 10	512 - 256
Large cobbles	10 - 5	258 - 128
Small cobbles	5 - 2.5	128 - 64
Very coarse gravel	2.5 - 1.3	64 - 32
Coarse gravel	1.3 - 0.6	32 - 16
Medium gravel	0.6 - 0.3	16 - 8
Fine gravel	0.3 - 0.16	8 - 4
Very fine gravel	0.16 - 0.08	4 - 2
Very coarse sand		2.000 - 1.000
Coarse sand		1.000 - 0.500
Medium sand		0.500 - 0.250
Fine sand		0.250 - 0.125
Very fine sand		0.125 - 0.062
Coarse silt		0.062 - 0.031
Medium silt		0.031 - 0.016
Fine silt		0.016 - 0.008
Very fine silt		0.008 - 0.004
Coarse clay		0.004 - 0.002
Medium clay		0.0020 - 0.0010
Fine clay		0.0010 - 0.0005
Very fine clay		0.0005 - 0.00024

For purposes of size classification by means of the above table and in investigations concerning channel roughness and sediment transport it is common practice to describe a particle size distribution by a single diameter. The arithmetic mean, geometric mean, and median diameter are those most frequently used; these measures are applied to frequency distributions by weight and number in sections 3.5.3. and 3.5.4. For most natural sediments these diameters are not equal. This has led to a certain amount of confusion since in many papers and reports the "mean"diameter of a particular sediment is given without stating exactly which diameter has been measured.

3.5.2. Location of Sampling Positions

Several factors dictated the locations at which the river bed material at Bywell could be sampled. Only those accumulations of gravel exposed at low flows were sampled, since no method for taking bulk samples from underwater was available. Examination of the reach showed that, in order to obtain sufficient samples, gravel deposits upstream and downstream of the test reach would have to be considered. It was necessary that the sampling positions were easily accessible and that the accumulations were natural deposits. During the extraction of gravel downstream of the test reach an artificial roadway was constructed along the left bank of the river near Bywell Hall. Sampling of these accumulations would thus have been erroneous. The gravel near the right bank became progressively coarser downstream of section JK, eventually including boulders of 4 to 5 feet diameter. There are several explanations for the presence of these boulders; KRUMBEIN and LIEBLEIN (1956) have shown by the theory of extreme values that they can form part of the normal gravel population of the river. However, it was considered that a sample of this extremely coarse deposit would not be representative of the major part of the bed material of the river reach.

With these points in mind six sampling positions were selected as shown in fig. 3.5.a. The deposits at positions 1 and 2 are shown in figs. 3.5.b. and 3.5.c.

3.5.3. Bulk Sampling

Few references to the bulk sampling of coarse river bed material could be found in the available literature. JONES (1959) describes how nine samples of about 4 cwt. each were taken from a 10 mile reach of a New Zealand river and KELLERHALS (1967) mentions that samples of

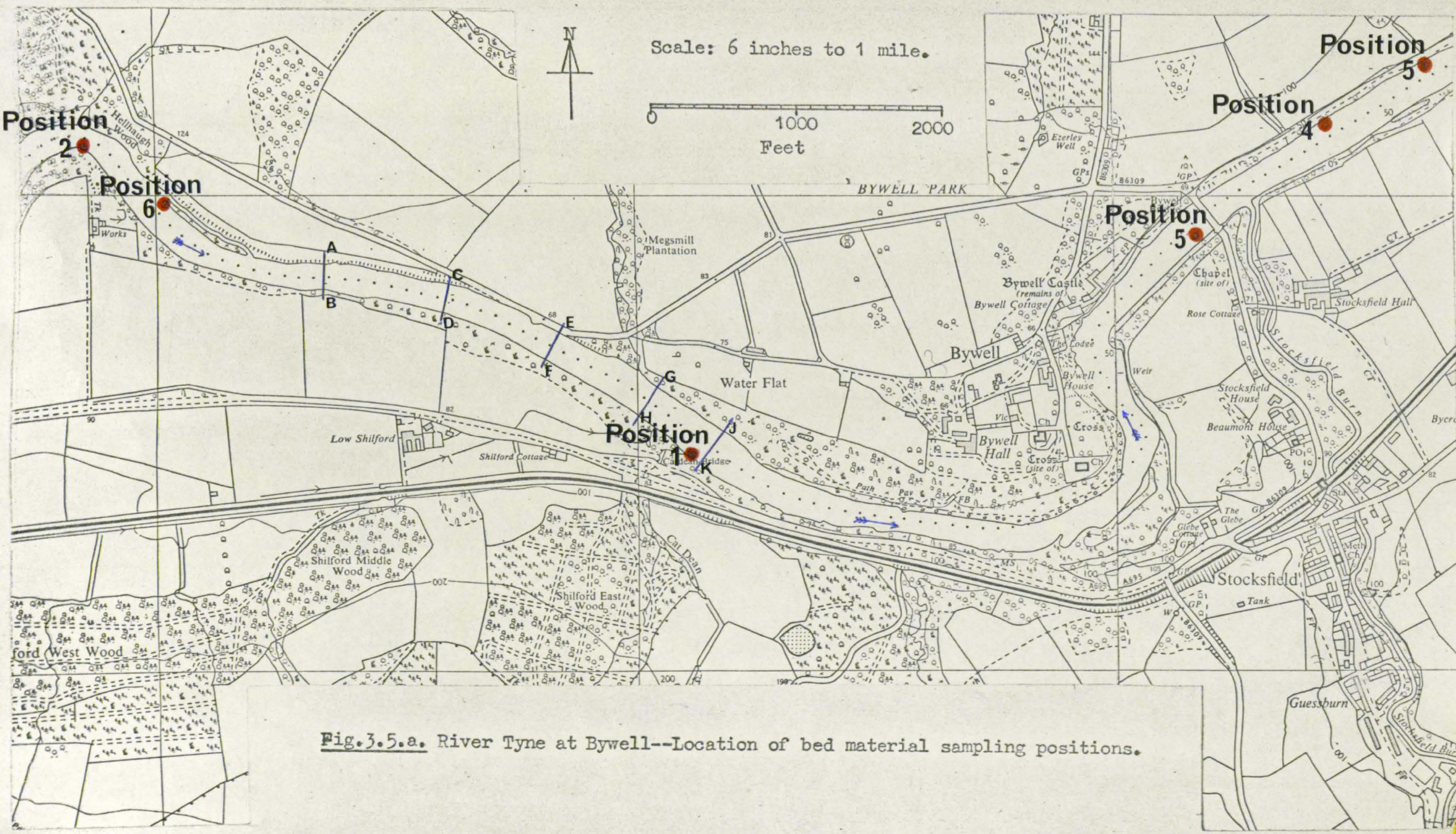


Fig.3.5.a. River Tyne at Bywell--Location of bed material sampling positions.

Fig.3.5.b. Bed material at sampling position 1.

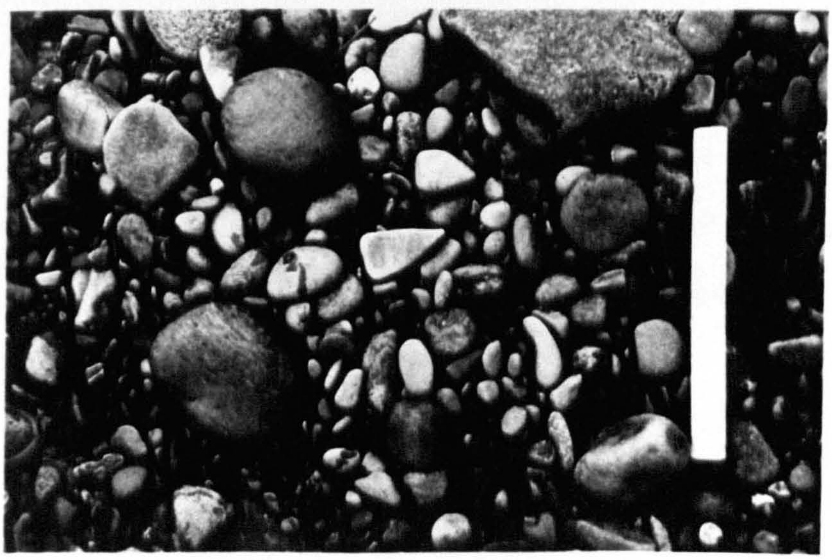


Fig.3.5.c. Bed material at sampling position 2.

1 cubic metre each and totalling 90 tons were taken from the Hasli-Aare River in Switzerland. Extrapolation of a graph in the BRITISH STANDARDS INSTITUTION (1960) publication dealing with concrete aggregates showed that when the maximum size present in substantial proportions is 4 inches then the minimum weight to be taken for sieving should be 220 lb. In order to be representative of the sediment being transported the depth to which the sample is taken should be equal to the depth of the moving layer of sediment. An arbitrary depth of about twice the maximum size present, i.e. about 1 foot, was considered sufficient in this respect for the bed material at Bywell.

The sampling procedure adopted at Bywell utilised four metal bins, 14 in. diameter and 30 in. deep. At each sampling location a 2 ft. square area of the gravel surface was randomly chosen and the four bins half filled with gravel; in this way each bin weighed about ½ cwt. and could be carried by one person. The total sample, therefore, weighed approximately 220 lb. and the depth of the resulting hole was about 1 ft. Each sample was taken to the Materials Testing Laboratory of the Department of Civil Engineering where it was spread on a clean concrete floor, allowed to dry and then sieved. Sizes up to ¾ in. were mechanically shaken, between ¾ in. and 3 in. manually sieved and for sizes greater than 3 in. each stone was passed through a specially constructed square opening.

The weights and particle size distributions of all six samples are given in table 3.5.b. and plotted on logarthmic-probability paper in fig. 3.5.d. The six samples were combined to produce the composite sample, no. 7, details of which are given in table 3.5.b. and fig. 3.5.e.

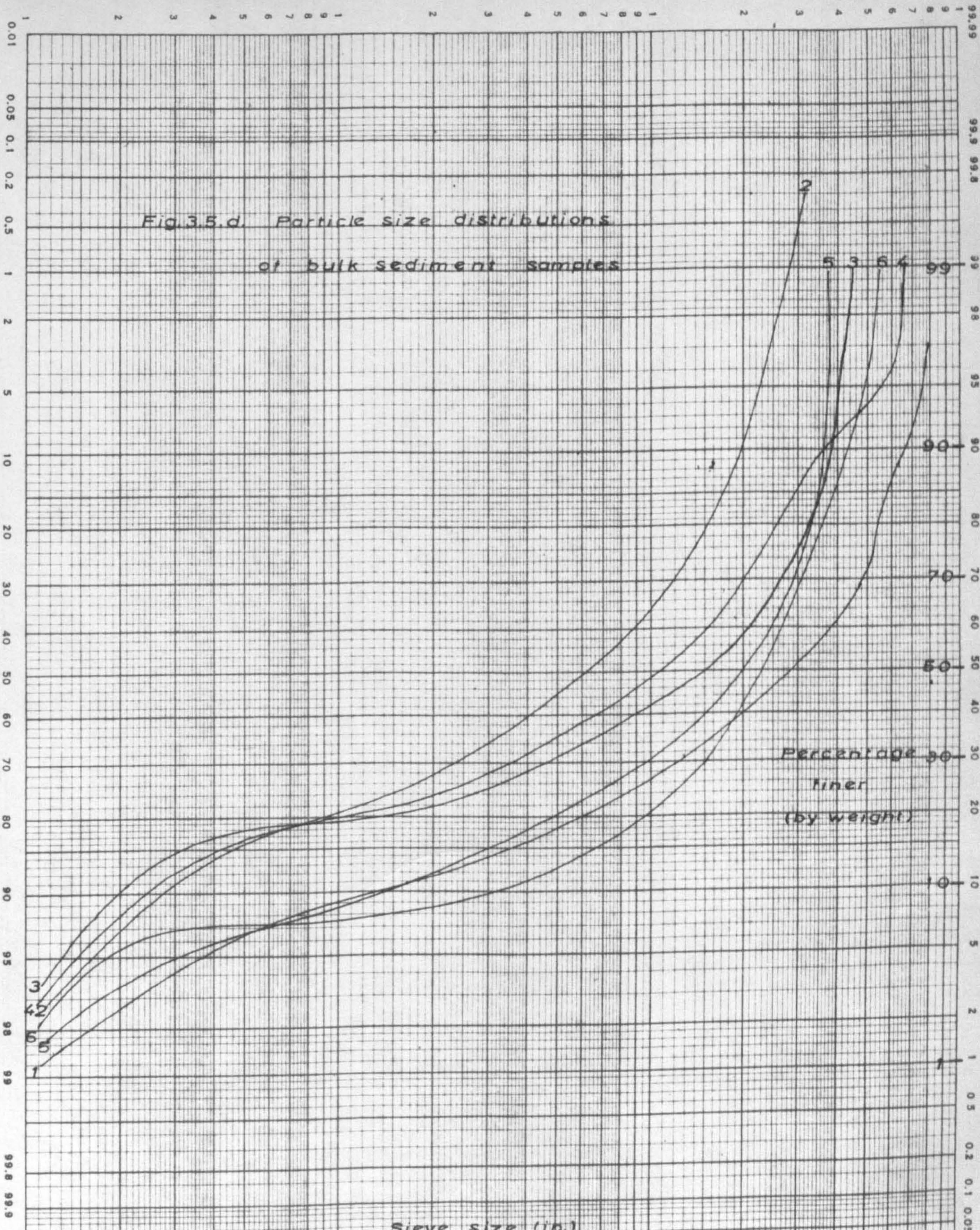


Fig.3.5.d. Particle size distributions of bulk sediment samples

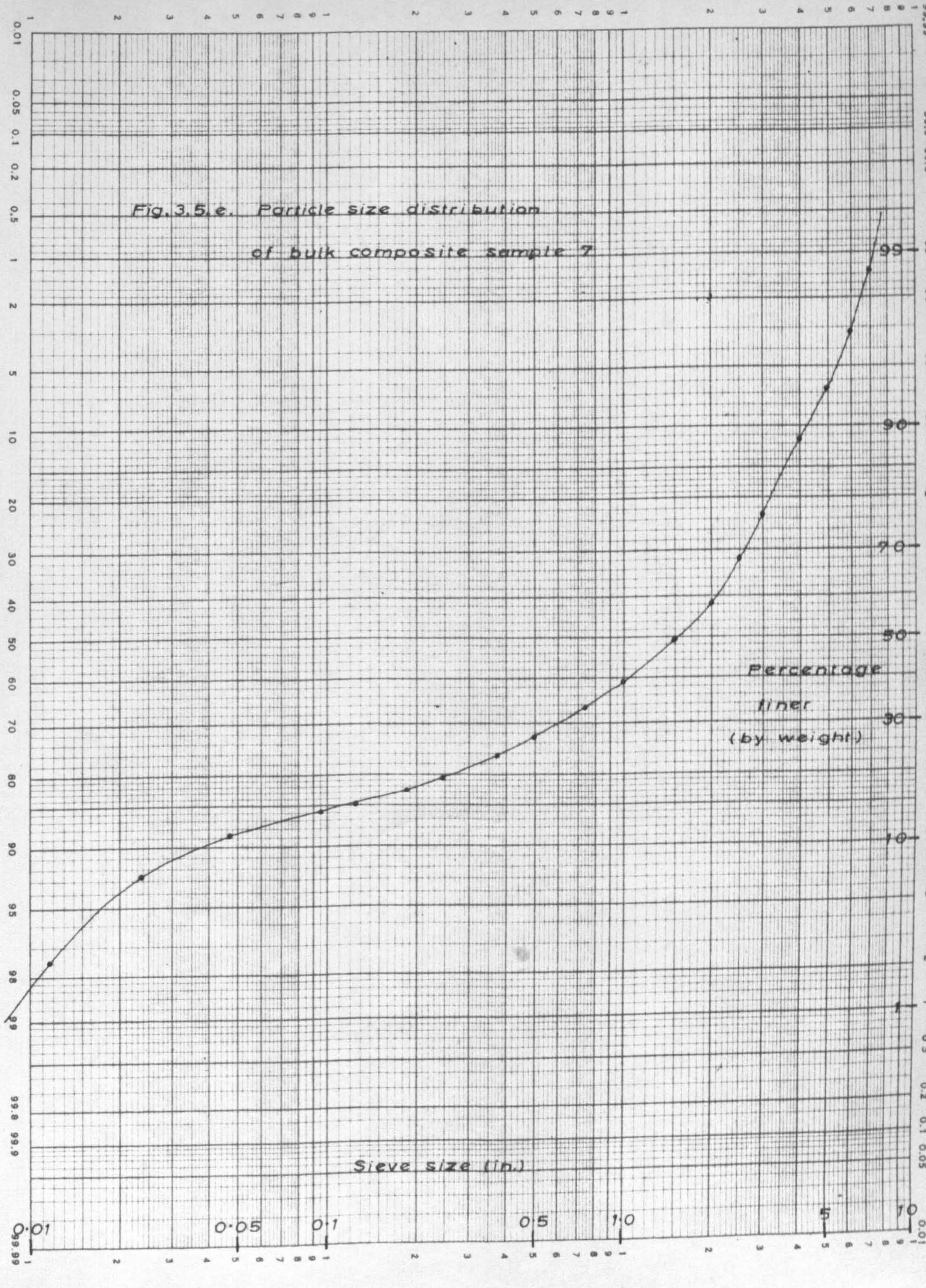


Fig. 3.5.e. Particle size distribution of bulk composite sample 7

Table 3.5.b. Sieve analysis of bulk samples

Sieve Opening (in)	% finer by weight						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6	Composite Sample 7
8	100.0	100.0	100.0	100.0	100.0	100.0	100.0
7	91.6	100.0	100.0	100.0	100.0	100.0	98.6
6	86.6	100.0	100.0	95.3	100.0	100.0	97.0
5	71.1	100.0	100.0	95.3	100.0	94.9	93.5
4	61.0	100.0	91.9	91.6	100.0	87.7	88.4
3	53.1	99.6	16.1	86.0	73.2	67.7	76.3
2.5	45.8	97.0	65.6	76.9	62.5	54.5	67.4
2.0	40.7	89.7	57.3	69.0	50.9	41.2	58.6
1.5	33.7	79.9	49.7	59.6	40.6	29.1	49.2
1.0	26.4	64.1	41.3	48.9	29.9	20.3	38.9
0.75	22.7	55.5	36.7	43.1	25.7	16.5	33.8
0.50	18.4	44.7	31.0	35.7	20.3	12.8	27.5
0.375	15.7	38.0	27.6	31.0	17.1	10.9	23.7
0.250	12.9	30.9	24.0	26.2	13.6	9.2	19.8
0.188	11.3	26.8	21.7	23.1	11.5	8.5	17.4
0.125	9.5	22.1	19.9	20.4	9.4	7.9	15.2
0.0945	8.6	19.9	19.1	19.2	8.4	7.7	14.1
0.0472	6.1	15.2	17.5	15.9	6.4	7.3	11.7
0.0236	3.2	8.5	12.5	9.6	4.4	6.2	7.6
0.0118	1.3	2.7	3.7	2.9	1.8	2.6	2.5
0.0059	0.5	0.8	0.7	0.8	0.8	0.8	0.7
Sample Weight (lb)	250.0	263.3	263.5	266.6	187.4	275.7	1504.5

The 65% and 35% finer by weight sizes of the gravel extracted at Bywell between 1960 and 1962 were given by the company concerned as 1.50 in and 0.19 in, respectively.

From the size classification table 3.5.a. and the particle size distribution curve of the composite sample, fig. 3.5.e, it can be seen that about 30% by weight of the bed material consists of cobbles and about 50% by weight is in the medium coarse to very coarse gravel range.

In order to describe adequately the particle size distributions in a quantitative manner it was found necessary to consider measures used in the field of sedimentology. It is recommended practice in this field (KRUMBEIN, 1934 and INMAN, 1952) to describe particle size in φ (phi) notation, where $\varphi = - \log_2$ diameter in millimetres. This allows the use of arithmetic rather than logarithmic size scales for the plotting of particle size distributions, greatly facilitates the calculation of descriptive parameters, and with many natural sediments produces a straight line. Tables are available (PAGE, 1955) for the rapid conversion of particle and sieve sizes to phi-notation, and vice versa.

FOLK (1966) has recently reviewed the parameters often used to describe sediments. Some of the measures were used directly in the application of bed load formulae to the reach (section 4), while others, such as sorting and skewness measures, were included because they are characteristics closely associated with sediment movement and deposition. The following measures were calculated for the size distribution of all samples, including the composite sample, and are given in table 3.5.c.

d_a : Arithmetic mean diameter, which describes the centre of gravity of the distribution when size is plotted to an arithmetic scale. It was obtained by the method of moments using the mean sizes of 10% ranges.

d_g : Geometric mean diameter, corresponding to the centre of gravity of the distribution when size is plotted to a logarithmic scale, or in phi-notation. It is equal to the phi mean diameter, $M\varphi$, expressed in inches McCAMMON (1962) gives:-

$$M\varphi = (\varphi_5 + \varphi_{15} + \ldots\ldots\ldots\ldots + \varphi_{85} + \varphi_{95})/10$$

where φ_5, φ_{15}, etc. are the phi sizes than which 5, 15 etc. % by weight of the sediment is finer.

d_{50} : Median diameter. The size than which 50% by weight of the sediment is finer. When expressed in phi notation it is termed the phi median diameter, $Md\,\varphi$.

$\sigma\varphi$: Geometric, or logarthmic, phi standard deviation, and is a measure of the spread or sorting of the distribution. McCAMMON (1962) gives:-

$$\sigma\varphi = (\varphi_3 + \varphi_{10} + \varphi_{20} + \varphi_{30} - \varphi_{97} - \varphi_{90} - \varphi_{80} - \varphi_{70})/9.1$$

$\alpha\varphi$: Phi skewness measure, which describes the extent of departure of the distribution from the log-normal distribution

$$\alpha\varphi = (M\varphi - Md\varphi)/\sigma\varphi$$

$\alpha\varphi = 0$ for a symmetrical distribution

$0 < \alpha\varphi < 1$ for a distribution skewed towards fine sizes

$-1 < \alpha\varphi < 0$ for a distribution skewed towards coarse sizes

$\beta\varphi$: Phi kurtosis measure, which represents the peakedness of the distribution.

$$\beta\varphi = \left[\tfrac{1}{2}(\varphi_5 - \varphi_{95}) - \sigma\varphi\right] / \sigma\varphi$$

$\beta\varphi = 0$ for a normal distribution

$\beta\varphi > 0.65$ for a distribution less peaked than normal

$\beta\varphi < 0.65$ for a distribution more peaked than normal.

Table 3.5.c. Descriptive measures of bulk samples

Sample number	d_a (in)	d_g (in)	d_{50} (in)	$\sigma\varphi$	$\alpha\varphi$	$\beta\varphi$
1	3.15	1.63	2.82	2.10	0.38	0.79
2	0.60	0.39	0.64	2.21	0.33	0.59
3	1.75	0.63	1.50	2.60	0.48	0.63
4	1.56	0.60	1.08	2.55	0.34	0.64
5	1.93	1.13	1.99	1.98	0.41	0.76
6	2.38	1.24	2.35	1.94	0.47	1.40
7	1.97	0.88	1.55	2.37	0.36	0.74

From the above table it can be seen that, while d_a and d_{50} correspond fairly closely, d_g is always smaller; the need for specifying exactly which mean diameter has been measured is evident. All distributions are skewed towards the fine sizes and most are slightly less peaked than normal. BLENCH (1952) found that nearly all river bed sands exhibit a log-normal distribution. As is shown by the S-shaped curves of figs. 3.5.d. and 3.5.e. and the values of in table 3.5.c. this is not so for the bed material at Bywell. It is probable that at high flows fine material in the bed will go into suspension and it seems likely, therefore, that the bed material may then approximate to a log-normal distribution. With phi standard deviations varying from 1.94 to 2.80 the sediments would be described in sedimentological terms as "poorly sorted" (FOLK, 1966).

3.5.4. Areal Sampling

A procedure for sampling coarse river bed material involving the collection of a random sample of particles from the surface of the bed has been proposed by WOLMAN (1954). As will be seen later, areal sampling methods based on this procedure require less effort and equipment and produce samples more amenable to shape and roundness analysis than bulk sampling methods. It was decided, therefore, to supplement the six bulk samples described in section 3.5.3. with areal samples taken from the same locations.

Several methods can be used to obtain the areal sample and it was decided to test each method at sampling positions 1 and 2 to determine which produced the most consistent results in the most convenient way. The methods tested were as follows:-

1. 3 ft. line transect at right angles to the direction of flow.
2. 5 ft. line transect at right angles to the direction of flow.
3. 7 ft. line transect at right angles to the direction of flow.

4. 3 ft. line transect parallel to the direction of flow.

5. 5 ft. line transect parallel to the direction of flow.

6. 7 ft. line transect parallel to the direction of flow.

In these methods a length of string, marked the necessary length, was stretched in the appropriate direction from a randomly chosen point on the bed. All the stones exposed on the surface of the bed and over which the string passed were collected.

7. 9 in. x 9 in. quadrat.

8. 12 in. x 12 in. quadrat.

9. 18 in. x 18 in. quadrat.

In these methods a square of the appropriate size was marked out around a randomly chosen point on the bed. All stones exposed on the surface within the square were collected.

10. Paced grid

This was the method used by WOLMAN (1954). A grid was paced out such that the number of stones taken from the intersections of the grid totalled a predetermined number; for the purposes of testing the methods 40 stones were collected. Subjectivity in sampling was suppressed by refraining from looking at the bed as each pace was made and by taking the required stone from under the tip of the toe of the boot.

Small particle sizes cannot be measured in the field so an arbitrary minimum size of $\frac{1}{4}$ in. intermediate diameter was assumed. Three samples were taken by each of the ten methods at positions 1 and 2. Each sample was weighed in air and in water, and the number of stones counted. It was then a simple calculation to determine the nominal diameter of the mean volume stone for each sample. This measure was of little practical significance but served as a means of comparing sampling methods. The results for positions 1 and 2 is given in table 3.5.d.

Table 3.5.d. Comparison of areal sampling methods

Method number	Section 1 Number of particles	Nom. dia. of mean volume particle (in)	Average of three samples (in)	Section 2 Number of particles	Nom. dia. of mean volume particle (in)	Average of three samples (in)
1	17	3.41	3.20	19	1.75	1.58
	26	2.88		22	1.68	
	21	3.32		22	1.30	
2	28	3.42	3.76	29	1.65	1.61
	26	3.97		30	1.54	
	27	3.89		29	1.64	
3	34	4.20	3.73	43	1.68	1.74
	45	3.21		43	1.88	
	29	3.79		44	1.67	
4	14	3.52	3.95	20	1.55	1.57
	14	4.08		19	1.43	
	16	4.26		19	1.74	
5	27	3.28	3.23	25	1.70	1.57
	25	3.17		32	1.51	
	24	3.26		34	1.50	
6	29	3.38	3.39	48	1.65	1.78
	28	3.28		40	1.86	
	31	3.51		42	1.84	
7	20	2.91	2.92	33	1.35	1.42
	17	2.83		30	1.44	
	14	3.03		36	1.47	
8	20	3.07	3.28	57	1.38	1.47
	22	3.66		50	1.41	
	35	3.10		45	1.57	
9	42	3.32	3.30	98	1.47	1.62
	37	3.46		81	1.64	
	60	3.11		82	1.75	
10	40	3.55	3.52	40	1.72	1.74
	40	3.34		40	1.63	
	40	3.67		40	1.88	

The main disadvantage of both the line transect and quadrat methods was the difficulty in determining which stones were exposed on the surface of the bed. In the latter method, especially, many stones were taken from just below the surface resulting in the inclusion of a number of small stones; this is reflected in the results of methods 7, 8 and 9 in table 3.5.d. No difference could be detected in taking line transect samples at right angles to or parallel with the direction of flow. The paced grid method shows less variation than the other methods and has the advantage of greater area of coverage and constant sample size. It was decided, therefore, to use the grid method, selecting 50 stones at each of the sampling positions to give a composite sample of 300 stones.

If the sediment particle is spherical, or nearly spherical, then each of the diameters listed in section 3.5.1 can be considered equal. Since the bed material at Bywell contained many non-spherical particles it was first necessary to determine which diameter was the most consistent measure of stone size. It was decided to measure the nominal diameter (d_n), sieve diameter (d_s), and the major (d_1), intermediate (d_2) and minor (d_3) axes of 75 stones whose nominal diameters ranged from 1.15 to 7.19 in. The arithmetic mean (d_r) and geometric mean (d_e) of the triaxial dimensions of each stone were also calculated. From this test it was found that on average d_2 was 2.12% larger than d_n, while d_r, d_e and d_s were 2.47, 6.40 and 8.19%, respectively, smaller than d_n. BLENCH and QURESHI (1964) and GRANT (1959) have used d_2 as the stone size. In the sample of 75 stones, however, the percentage differences of d_2 from d_n displayed more scatter than those of d_r from d_n. The arithmetic mean of the triaxial dimensions, d_r, would thus appear to be a more consistent measure of size than the intermediate axis, d_2, alone.

Accordingly, the major, intermediate and minor axes of each stone of the six areal samples were measured and the arithmetic mean of these dimensions taken as particle size. The weight of each stone was also recorded. Particle size distributions by number frequency are given in table 3.5.f. and figs. 3.5.f. and 3.5.g.

Table 3.5.f. Particle size analysis of areal samples

d_r (in)	% finer by number						
	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6	Composite Sample 7
7	100	100	100	100	100	100	100
6	92	100	100	100	100	100	99
5	84	100	96	100	98	94	95
4	72	100	84	92	90	90	88
3	50	96	68	72	68	56	68
$2\frac{1}{2}$	36	90	54	62	44	36	54
2	24	80	42	50	30	24	42
$1\frac{1}{2}$	4	52	24	28	14	16	23
1	0	4	8	0	0	0	2
$\frac{3}{4}$	0	0	0	0	0	0	0

3.5.5. Comparison of Bulk and Areal Sampling Methods

Bulk sampling is a means of obtaining a particle size distribution by weight of the sediment composing the river bed to a certain depth; samples obtained by areal methods are analysed on a number frequency basis and represent the areal distribution of sediment sizes on the surface of the river bed.

The main disadvantage of the bulk method when applied to coarse sediments is that each sample must weight several hundred pounds in order to be representative. Considerable effort and equipment, often unavailable, are required to collect, dry, sieve and weigh each sample. Areal methods enable many samples to be taken by one person with simple equipment in a comparatively short time, all measurements being recorded in the field. Each areal sample is representative of a far larger area than a bulk sample and can be

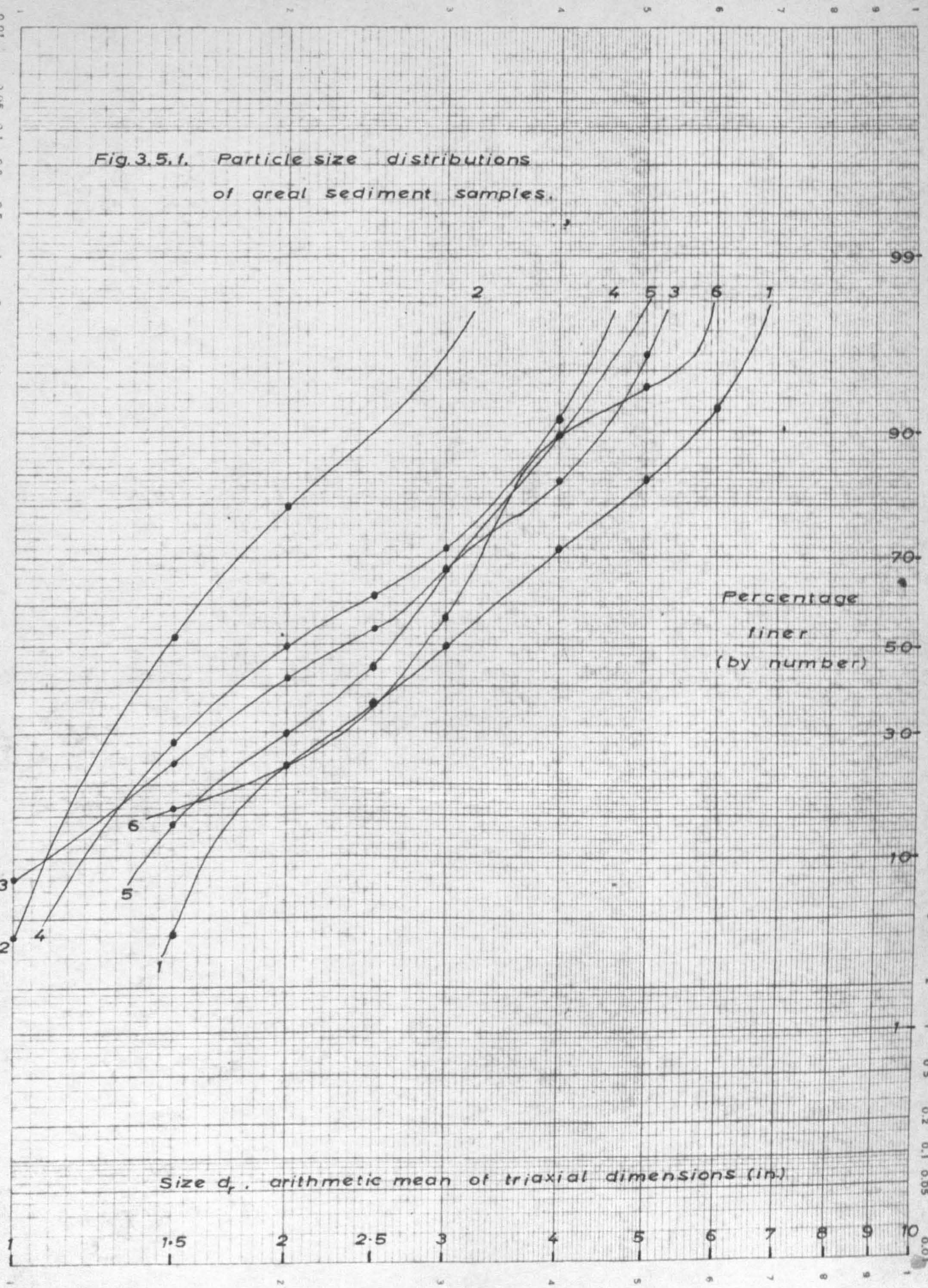


Fig. 3.5.1. Particle size distributions
of areal sediment samples.
Percentage
finer
(by number)
Size d_r, arithmetic mean of triaxial dimensions (in.)
1
1·5
2
2·5
3
4
5
6
7
8
9
10
99
90
70
50
30
10
1
2
3
4
5
6
7

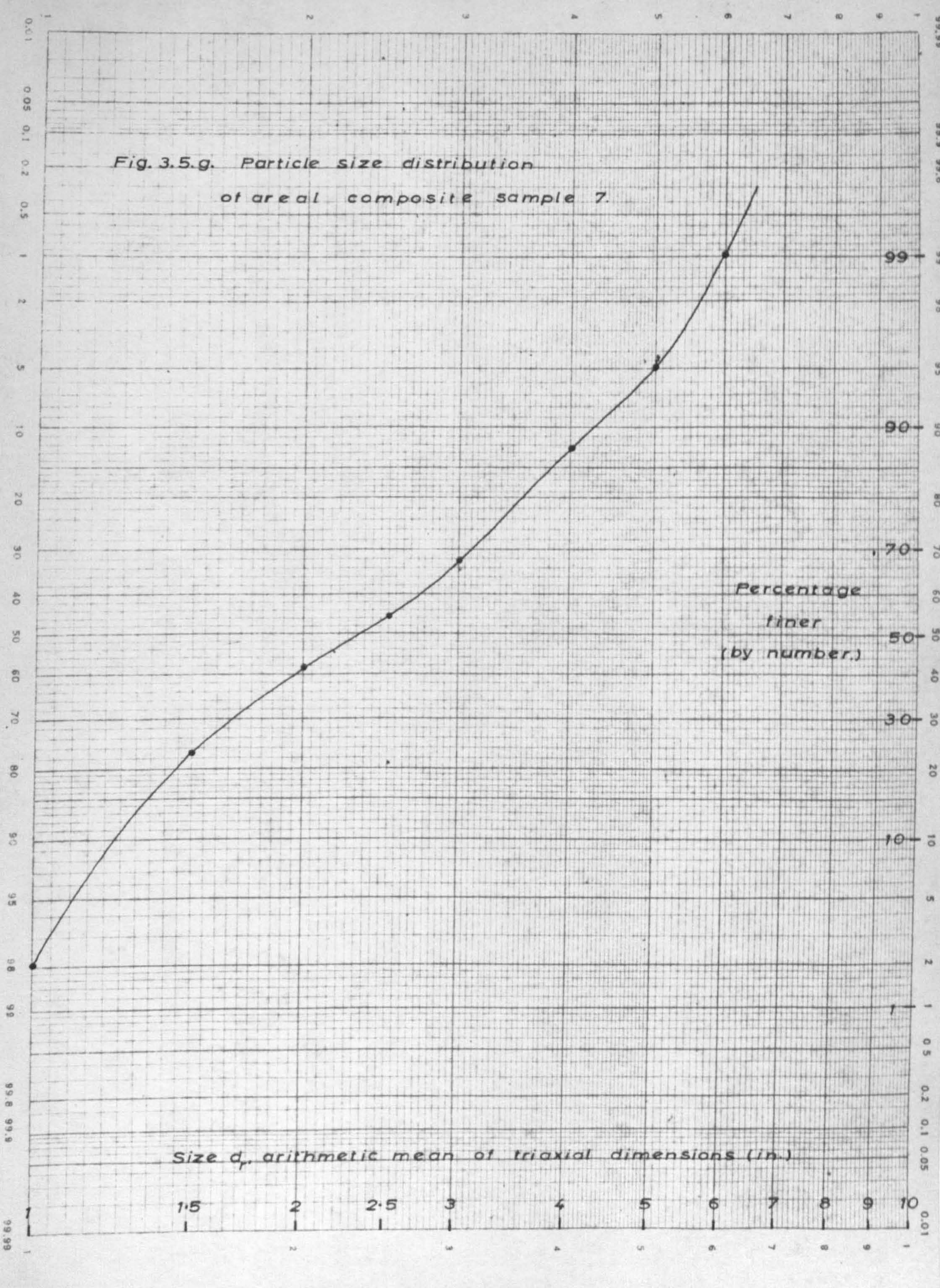


Fig. 3.5.g. Particle size distribution
of areal composite sample 7.
Percentage
finer
(by number.)
99
90
70
50
30
10
1
Size d_r, arithmetic mean of triaxial dimensions (in.)
1
1·5
2
2·5
3
4
5
6
7
8
9
10

obtained from underwater if necessary. Probably the major limitation of the areal method is that it has a lower limit of particle size which can be sampled. The absence of fines, however, results in a good approximation to a log-normal size distribution (fig. 3.5.g.).

In the comparison of the size distributions of the bulk and areal samples it should be remembered that they have been obtained by quite different procedures. The bulk samples have been sieved and weighed, while the average of the trixial dimensions of the particles of the areal samples have been measured and plotted on a number of frequency basis. Table 3.5.g. shows that the median diameter of the areal samples, ${}_n d_{50}$, is always larger than the median diameter of the bulk samples, d_{50}.

Table 3.5.g. Comparison of median diameters of bulk and areal samples

Sample number	${}_n d_{50}$ (in)	d_{50} (in)	${}_n d_{50}/d_{50}$
1	3.00	2.82	1.01
2	1.45	0.64	2.27
3	2.34	1.50	1.56
4	2.00	1.08	1.85
5	2.66	1.99	1.34
6	2.81	2.35	1.20
7	2.34	1.55	1.51

Areal samples could possibly be of considerable use in the estimation of the bed roughness coefficient for use in hydraulic formulae such as the Manning formula. EINSTEIN (1950) suggested that the effective roughness size of a sediment mixture should be d_{65}, the size than which 65% by weight of the sediment is finer; MEYER-PETER and MÜLLER (1948), in contrast, proposed d_{90} as the representative gravel size for roughness estimation. Figs. 3.5.h. and 3.5.i. show ${}_n d_{50}$ plotted against d_{65} and d_{90} respectively.

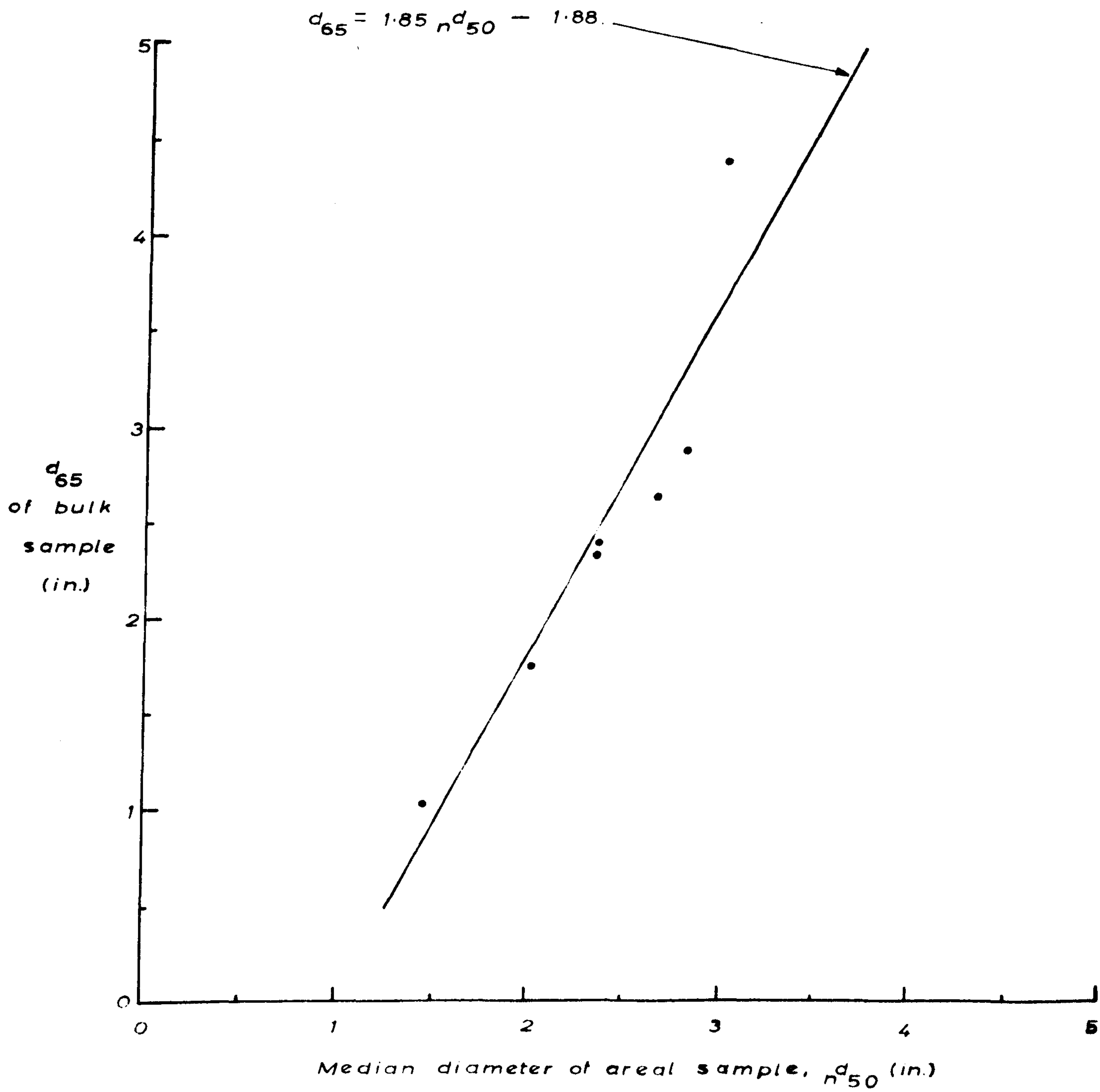


Fig. 3.5.h. Relationship between ${}_{n}d_{50}$ and d_{65}.

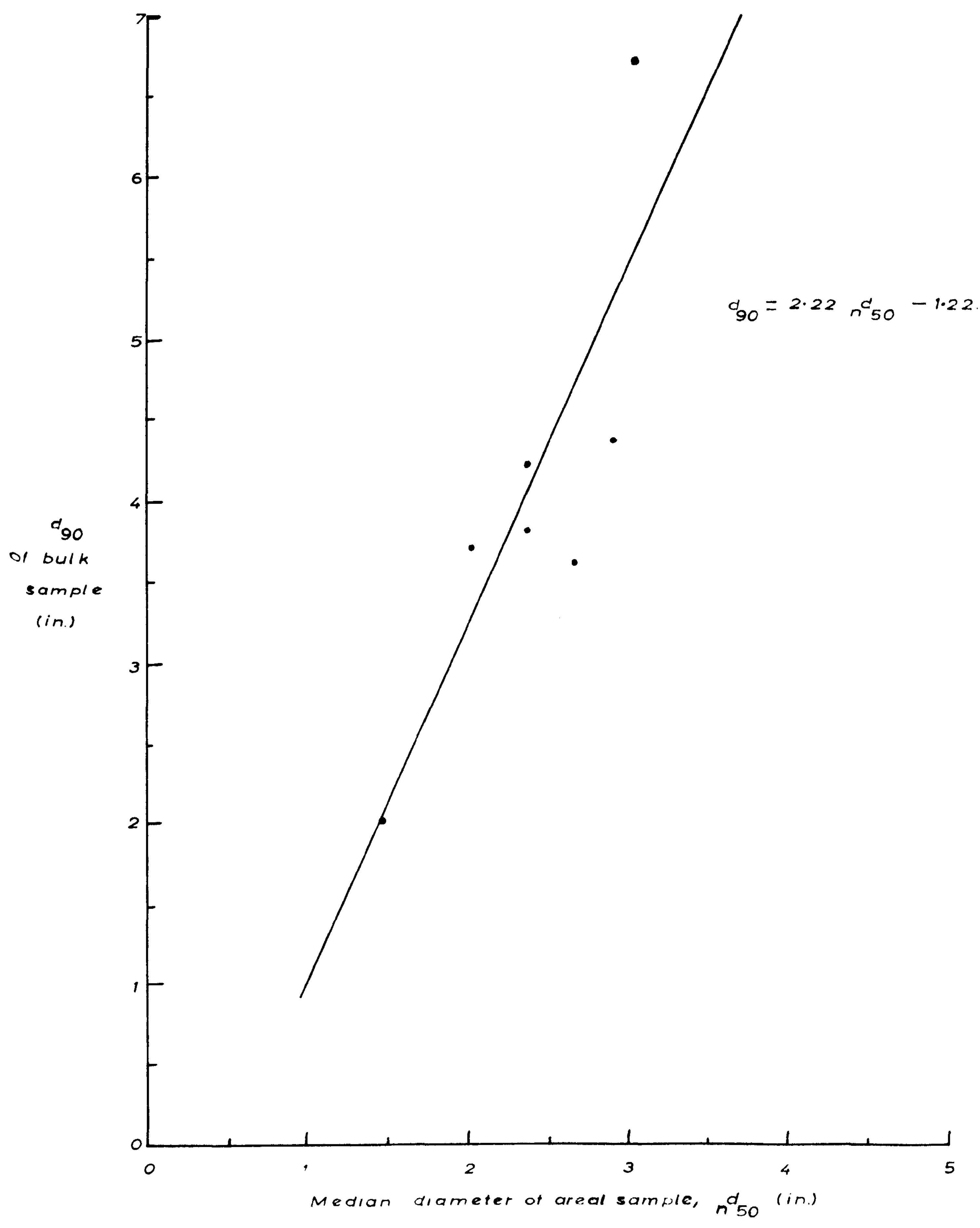


Fig. 3.5.i. Relationship between $_nd_{50}$ and d_{90}

While some relationships are indicated, the number of samples and range of size data are insufficient to enable definite conclusions to be drawn. It would seem, however, that areal samples could be used to determine the representative roughness size of a sediment mixture. The following table shows how ${}_n d_{50}$ could possibly be taken to equal d_{65}, the roughness size suggested by Einstein.

Table 3.5.h. Possible use of areal samples for roughness size estimation

Sample number	Areal sample median dia. ${}_n d_{50}$ (in)	% by weight of bulk sample finer than ${}_n d_{50}$
1	3.00	52.0
2	1.45	78.0
3	2.34	63.0
4	2.00	69.0
5	2.66	65.0
6	2.81	63.0
7	2.34	65.0

There is, in fact, no need for extreme accuracy in defining the representative roughness size of a gravel surface, since it is known that grain roughness on a plane bed (represented by the Manning coefficient, for instance) is proportional to the sixth root of this size. Hence, if the gravel size is overestimated by 100% then the roughness coefficient itself will have been overestimated by only 10.5%.

3.4.6. Particle Shape and Roundness

Although there are no provisions for the inclusion of measures of particle shape and roundness in any bed load formulae they are most probably important factors in the consideration of bed formation, initiation of movement and sediment transport (BLENCH, 1966b). It was decided, therefore, to record these characteristics for each particle of the six areal samples taken from the surface of the river bed at Bywell. Few references were found in civil engineering literature on the measurement of these properties (BRITISH STANDARDS INSTITUTION, 1960, and MACKAY, 1965) and it was necessary once again to refer to measures devised by sedimentologists.

The word "shape" describes the form of the particle without reference to the sharpness of its edges. To measure this characteristic it was decided to use the KRUMBEIN (1941) approximation to the WADDELL (1935) definition of sphericity as the cube root of the ratio of the volume of the circumscribing sphere. This is approximated by Krumbein as $\sqrt[3]{(d_2 d_3)/d_1^2}$, where d_1, d_2, and d_3 are the major, intermediate, and minor axes, respectively, of the particle. Table 3.5.i. gives the distribution of sphericities and average sphericities of each areal sample and of the composite sample, number 7.

Table 3.5.i. Distribution of particle sphericities in areal samples

Sample Number	Sphericity range							Average Sphericity
	0.30–0.39	0.40–0.49	0.50–0.59	0.60–0.69	0.70–0.79	0.80–0.89	0.90–0.99	
1	-	5	14	19	9	3	-	0.63
2	1	-	12	22	12	3	-	0.66
3	-	1	5	22	19	3	-	0.68
4	-	2	11	20	13	2	2	0.66
5	-	4	14	16	14	2	-	0.63
6	-	2	13	22	7	6	-	0.64
7	1	14	69	121	74	19	2	0.65

In conjunction with the sphericity measure each particle was classified according to the four types proposed by ZINGG (1935) as follows:-

Class	d_2/d_1	d_3/d_2
Disc	$\geqslant 2/3$	$< 2/3$
Sphere	$\geqslant 2/3$	$\geqslant 2/3$
Blade	$< 2/3$	$< 2/3$
Rod	$< 2/3$	$\geqslant 2/3$

Table 3.5.j. gives the distribution of shapes in each of the samples

Table 3.5.j. Shape classification of areal samples

Sample number	Zingg shape class			
	Disc	Sphere	Blade	Rod
1	27	4	12	7
2	34	5	6	5
3	33	9	5	3
4	31	10	5	4
5	29	4	14	3
6	31	8	7	4
7	185	40	49	26

The word "roundness" describes the sharpness or radius of curvature of the edge of the particle and reflects the abrasion resulting from the transport of the particle along the river bed. POWERS (1953) proposed a new roundness scale for sedimentary particles but the visual method of KRUMBEIN (1941) has proved to be the most convenient for many workers in the field. Krumbein defined roundness as the ratio of the average radius of curvature of the edges and corners of the image of the particle to the radius of the inscribed circle and produced a chart (fig. 3.5.j.) showing calculated values of various particle outlines. Each particle of the areal samples was classified by means of comparison with the chart; table 3.5.k. gives the distribution of roundness within each sample and that of the composite sample.

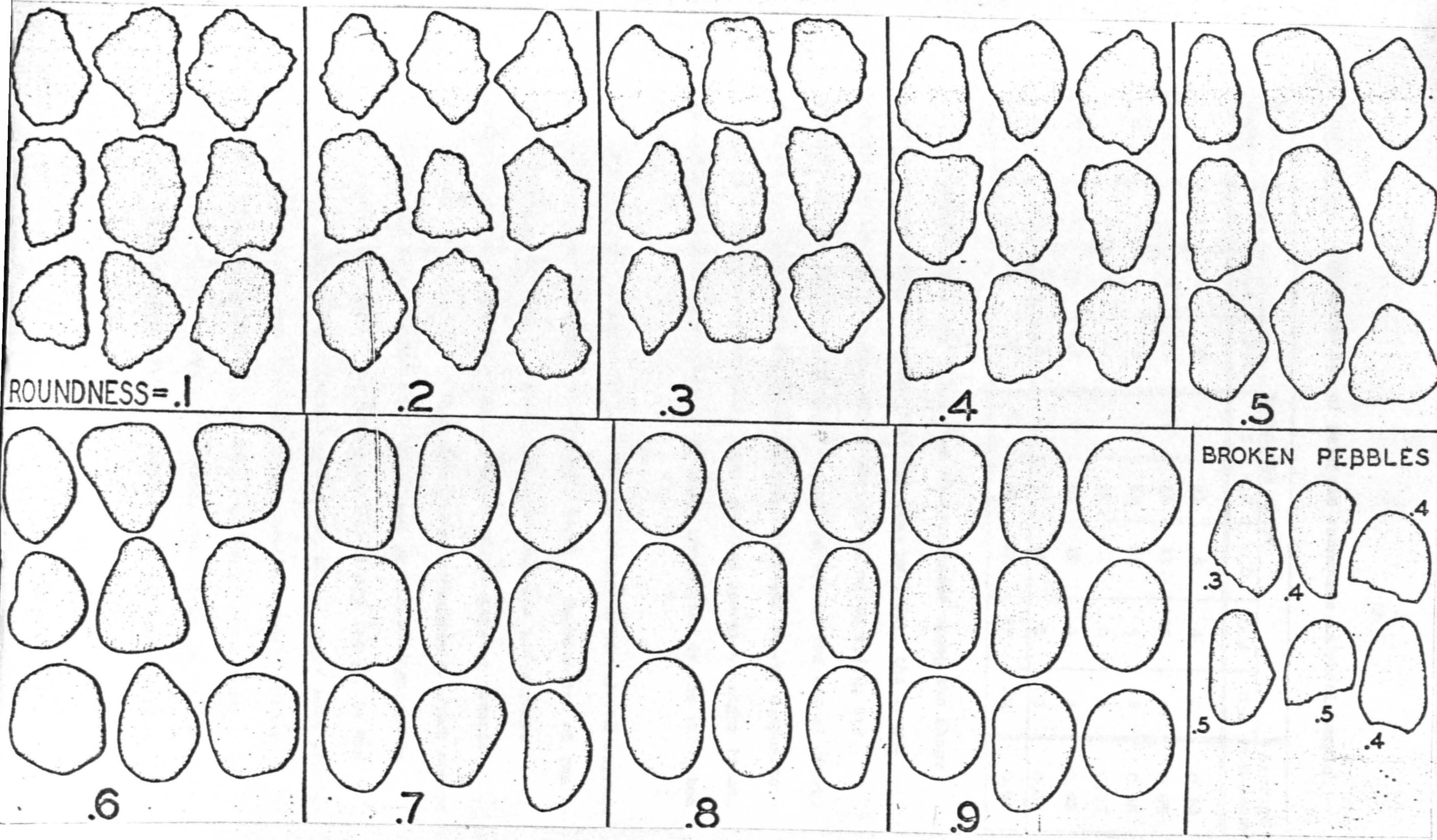


Fig. 3.5.j. Chart for visual estimation of particle roundness. (KRUMBEIN, 1941)

Table 3.5.k. Distribution of particle roundness in areal samples

Sample number	Roundness class								Average Roundness
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	
1	1	2	5	9	21	6	4	2	0.58
2	-	-	2	4	16	11	13	4	0.68
3	-	-	3	9	10	21	4	3	0.65
4	-	-	-	7	17	16	5	5	0.67
5	-	-	5	8	15	15	4	3	0.63
6	-	-	2	8	17	15	5	3	0.64
7	1	2	17	45	96	84	35	20	0.64

In tables 3.5.i and 3.5.j. the distributions among the shape classes of all samples follow much the same pattern; the larger differences in the roundness distributions of table 3.5.k. are probably due to subjectivity involved in the use of the visual chart. Plots of the sphericity and roundness distributions of the composite sample on arithmetic-probability paper yielded almost straight lines, indicating a normal distribution of these properties in the river bed material.

Shape plays an important part in the formation of the bed surface in gravel-bed rivers (LANE and CARLSON, 1954). Examination of the reach of the River Tyne at Bywell revealed that the bed surface particles were arranged with their flatter faces sloping upwards in a downstream direction. This imbricated arrangement, which occurs in most rivers of predominately disc-shaped particles (from table 3.5.j., 61% of the surface particles at Bywell are discs), is the result of the tendency of these particles to assume their most stable position under the forces exerted by the flow of water. Finer particles are usually washed away from the top of the larger, flatter particles and accumulate between, underneath, or in the eddy immediately downstream of them. This action leads to a form of armouring or protection of the bed surface; such "bed pavements" have been observed in rivers in Russia (LELIAVSKY, 1955) and Canada (KELLERHALS, 1967).

A gravel bed of flat-shaped particles is therefore able to resist much higher shear stresses than a bed of uniform-sized spherical particles. The use of a tractive force formula such as equation 2.2.a (SCHIELDS, 1936) on the river reach at Bywell is thus likely to underestimate the critical shear stress required for the initiation of movement for two reasons:- i) the coarser particles of the bed tend to occupy the surface layer and ii) no provision is indluded in the formula for the relatively high stability of disc-shaped particles in an imbricated position. The influence of shape on the susceptibility to movement of sediment particles at Bywell was clearly demonstrated in the following simple observation carried out at sampling position 1 (fig. 3.5.a). Ten stones were selected from as near mid-channel as possible, their triaxial dimensions measured and then they were replaced on the bed with their orientation marked by yellow paint. The arithmetic mean of the triaxial dimensions varied from 3.1 to 8.4 inches. After the passage of a short flood of about 17,000 cusec it was noted that, while the smaller disc-shaped particles had remained stationary, most of the larger stones had moved at least some distance; two of the more spherical particles, about 5 inches in diameter, had travelled over 100 feet.

The influence of shape on the rate of transport of sediment as bed load is not so immediately evident. It is possible that particles moving as contact load, i.e. in more or less continuous contact with the bed, tend to maintain their most stable orientations, with the result that flat particles travel more slowly than spherical particles of the same size. Disc-shaped particles, however, have a lower settling velocity than spheres so that flat, saltating, particles probably travel further before being deposited. Saltation load is relatively unimportant in fluvial sediment transport (KALINSKE, 1942) and hence it may be expected that bed load formulae applied to the Bywell reach will overestimate bed load discharge (assuming that the formulae will predict accurately bed load discharge for spherical particles of the same size).

3.5.7. Petrographic Analysis

A petrographic analysis of each of the areal samples was carried out; table 3.5.1 gives the results.

Table 3.5.1. Petrographic analysis of areal samples

Rock type	Sample number							
	1	2	3	4	5	6	7	%
Sandstone	33	36	24	28	21	27	169	56.3
Limestone	2	-	-	3	2	-	7	2.3
Greywacke	4	3	9	6	15	15	52	1[illegible].3
Ganister	-	1	-	-	-	-	1	0.3
Chert	-	1	-	-	-	-	1	0.3
Sinter	-	1	-	-	-	-	1	0.3
Ironstone	-	1	1	2	-	-	4	1.3
Quartzite	-	2	-	-	-	-	2	0.7
Volcanic ash	2	3	-	2	3	-	10	3.3
Basalt	1	1	-	1	2	-	5	1.7
Dolerite	1	-	2	2	-	3	8	2.7
Gabbro	-	-	1	-	-	-	1	0.3
Andesite	6	1	11	5	6	5	34	11.3
Rhyolite	-	-	1	-	-	-	1	0.3
Felsite	-	-	-	-	1	-	1	0.3
Syenite	-	-	-	1	-	-	1	0.3
Granite	1	-	-	-	-	-	1	0.3
Phyllite	-	-	1	-	-	-	1	0.3

As would be expected the sedimentary rock, sandstone, forms a major part of the bed population; its presence is reflected in the preponderance of particles in the Zingg classification of disc-shaped (table 3.5.j.). The presence of a considerable number of andesite and greywacke particles which must have originated from parent rocks in the Lake District and the Southern Uplands of Scotland indicates that much of the coarse bed material of the River Tyne at Bywell has been eroded from the glacial drift which forms the banks of the middle and upper reaches of the River Tyne. It is not possible to distinguish between particles from the Rivers North and South Tyne. The sparsity of limestone in the bed indicates a high solution load.

3.5.8. Specific Gravity

The specific gravity of the bed material, required in section 4 for the application of bed load formulae to the reach, was obtained by weighing a sample in air and in water according to the method described by the BRITISH STANDARDS INSTITUTION (1960). It was found that the specific gravity of the sediment, $S_s = 2.60$, hence the specific weight, $\gamma_s = 162.3$ lb/ft^3.

The specific weight of a fluid-sediment mixture is equal to:-

$$\frac{\gamma_f \;\; \gamma_s}{\gamma_s - C_s \, (\gamma_s - \gamma_f)}$$

where γ_f, γ_s are the specific weights of the fluid and sediment, respectively and C_s is the concentration by weight of sediment in the fluid. Assuming the specific weight of water to equal 62.42 lb/ft^3 and the maximum concentration of suspended sediment to be 1500 p.p.m. then the maximum value of the specific weight of the water-sediment mixture can be calculated to be 62.47 lb/ft^3. The difference between the specific weight of the clear water and that of the mixture is less than ½%; it was decided, therefore, that the specific weight of the river water at Bywell would be assumed equal to 62.42 lb/ft^3.

3.6. Conclusions

The longitudinal bed profile of the River Tyne at Bywell exhibits the undulatory form characteristic of most rivers with gravel-paved beds. This "pool-bar" configuration, while providing convenient control sections for cableway gauging stations, greatly complicated the selection of a suitably straight and uniform test reach. Twenty three cross-sections in the ¾ mile reach which included the cableway section at Bywell displayed considerable variations in shape and it was found impossible to determine an average bed slope over the length of the reach. Soundings showed that a meandering thalweg is superimposed on the relatively straight length of river. Examination of the horizontal and vertical velocity distributions at the cableway section showed that this complex bed configuration offers resistance to flow in addition to that of the particle roughness of the bed and affects the distribution of tractive force on the bed.

The instrument and procedure developed for the measurement of the water surface slope by a single observer proved successful although a recommended improvement would be the simultaneous measurement of water level at each end of the reach by two observers. At low flows the water-surface and energy-surface profiles above the cableway are fairly flat, becoming steeper downstream of the cableway; at higher stages the upstream and downstream slopes become more nearly equal. The geometric mean of the two surface slopes was taken to represent the actual slope at the cableway section and was found to vary from 0.2×10^{-3} at low flows to 1.2×10^{-3} at just below bankful stage. Good linear correlations were obtained between the logarithm of stage above a given datum and the water-surface and energy-surface slopes. No improvement in the correlation could be detected by the introduction of the rate of change of stage into the regression analysis.

There are, at present, no standardised methods for the sampling and analysis of coarse river bed material; recent papers by CAMPBELL and CADDIE (1964) in New Zealand and NEILL and GALAY (1967) in Canada have emphasised the need for systematic collection of river data. The two basic methods of sampling, bulk sampling and areal sampling, were both carried out on the River Tyne at Bywell. Six bulk samples, with a total weight of 2/3 ton, were collected and sieved; about 75% by weight of the bed material can be classified according to the AMERICAN SOCIETY OF CIVIL ENGINEERS (1962) grade scale as cobbles and coarse gravel. The particle size distribution curves are not log-normal, being skewed towards the fine sizes. A certain amount of confusion has been caused in sediment transport studies by the large number of measures used to describe a size distribution. In order to facilitate comparison all of the following properties have been calculated for each bulk sample collected at Bywell:- arithmetic mean diameter, geometric mean diameter, median diameter and measures of sorting, skewness and kurtosis.

Three methods of obtaining areal samples of the bed surface material were tested; the most convenient was found to be the paced grid method suggested by WOLMAN (1954). Particle sizes were defined by the arithmetic mean of the triaxial dimensions and plotted on a number frequency basis.

For the description of particle shape the sphericity measure of KRUMBEIN (1941) and the ZINGG (1935) classification were used, since only the triaxial dimensions of the particles were required. Shape probably plays an important role in bed formation, initiation of motion, and sediment transport, although there is no provision for the quantitative inclusion of this property in any existing bed load formulae; it is most likely of particular importance at

Bywell where over 60% of the bed surface particles can be classified as disc-shaped. Roundness was found to be most easily measured using the visual chart produced by KRUMBEIN (1941).

Comparison of the particle size distributions of the bulk samples with those of the areal samples indicates that the latter could possibly be used to obtain estimates of the representative roughness size of the gravel bed.

Areal sampling was found to have several advantages over bulk sampling:-

1) The equipment and personnel required are minimal; bulk sampling of coarse material requires heavy equipment for collection, drying and sieving.
2) The method is quick and simple, enabling several samples to be collected and analysed in a considerably shorter time.
3) Samples are representative of larger areas (underwater, if necessary).
4) All measurements are made in the field.
5) Descriptions of particle shape and roundness are facilitated.

A petrographic analysis of the bed surface particles at Bywell showed that, while predominantly sandstone, much of the material originated from outside the Tyne Catchment, and must therefore be the result of erosion of the glacial drift which forms the banks of the River Tyne for most of its length.

Section 4

4. Application of Bed Load Theories to the River Tyne at Bywell

The general concept of bed load movement has been discussed in section 2 and the collection of data required for the application of bed load theories to the Bywell test reach has been detailed in section 3. This section describes the application of the bed load theories to the River Tyne at Bywell to determine the bed load rating curve, i.e. the relationship between bed load discharge and river stage (or river discharge). The resulting rating curves are used to obtain estimates according to each theory of the average annual bed load discharge of the River Tyne at Bywell.

4.1. Determination of Bed Load Rating Curve

Almost thirty theories and formulae for the prediction of bed load discharge were found in the available literature. Only nine of these theories, including the modified Einstein method, could possibly be considered suitable for application to the River Tyne at Bywell; nevertheless, considerable extrapolation and estimation were required by some of the methods due to the sediment and stream flow conditions at Bywell.

Each method was applied to determine the bed load discharge at the cableway section at 1 foot intervals of stage from 48 ft. to 61 ft. A.O.D.

4.1.1. Preliminary Considerations

To avoid unnecessary repetition the essential data required by all the methods are noted in this section.

River temperature at Bywell was observed to vary from $0^{\circ}C$ to $20^{\circ}C$. Study of recorded river temperatures in the north of England (HERSCHY, 1965 and SURFACE WATER SURVEY, 1966) showed that a mean temperature of about $8^{\circ}C$ could be assumed. Hence:-

Specific weight of river water, γ_f = 62.42 lb/ft^3 (section 3.5.8.)

Mass density of river water, ρ_f = 1.94 slug/ft^3

Viscosity of river water at $8^{o}C$, $\mu = 2.843 \times 10^{-5}$ slug/ft. sec.

Kenematic viscosity of river water

at $8^{o}C$, $\nu = 1.465 \times 10^{-5}$ ft^2/sec.

Details of the river bed material are obtained from section 3.5 as follows:-

Specific gravity of bed material,	$S_s = 2.60$
Mass density of bed material,	$\rho_s = 5.04$ slug/ft^2
Specific weight of bed material,	$\gamma_s = 162.3$ lb/ft^3
Arithmetic mean weight diameter,	$d_a = 1.97$ in $= 0.164$ ft.
Geometric mean weight diameter,	$d_g = 0.88$ in $= 0.0733$ ft.
Median weight diameter,	$d_{50} = 1.55$ in $= 0.129$ ft.

All the methods which were used, except the Einstein method, were concerned solely with the cableway section, EF. Table 4.1.a. summarises the physical and hydraulic properties of the section at 1 ft. intervals of stage. The water-surface and energy-surface slopes have been calculated from the equations determined in section 3.4.3:-

$$S_w \times 10^3 = 1.504 \log_{10} (H_e - 45.55) - 0.449$$

$$S_e \times 10^3 = 1.869 \log_{10} (H_e - 44.35) - 0.918$$

where H_e is the stage in feet A.O.D.

The total cross-sectional area of flow, A_t, the water-surface width, W, and the total wetted perimeter, P_t, have been determined in section 3.3. Mean depth of flow, $D = A_t/W$ and the total hydraulic radius, $R_t = A_t/P_t$. The total water discharge through the section, Q, is given by the equation (section 3.4.4):-

$$Q = 233 (H_e - 46.62)^{1.974}$$

and mean velocity in the section, $V = Q/A_t$.

TABLE 4.1.a.

A_w (ft^2)	P_w (ft)	R_w (ft)	V_w (ft/sec)	Q_w (cusec)	Q_b (cusec)	V_b (ft/sec)	D_b (ft)	P_b (ft)	R_b (ft)	u_* (ft/sec)	τ_o (lb/ft^2)
0	0	0	0	0	440	0.58	3.76	209	3.61	0.124	0.0298
4	8	0.50	0.43	1.7	1289	1.35	4.71	211	4.54	0.219	0.0935
11	11	1.00	0.82	9	2570	2.21	5.71	211	5.50	0.294	0.168
21	14	1.50	1.21	25	4276	3.14	6.71	211	6.46	0.359	0.250
32	17	1.88	1.53	49	6405	4.09	7.71	211	7.42	0.419	0.340
45	20	2.25	1.76	79	8959	5.06	8.71	211	8.38	0.474	0.436
60	23	2.61	2.14	128	11920	6.05	9.71	211	9.34	0.526	0.538
77	25	3.08	2.49	192	15290	7.03	10.71	211	10.30	0.576	0.645
96	29	3.31	2.70	259	19080	8.02	11.71	211	11.27	0.625	0.757
119	34	3.50	2.94	350	23280	9.02	12.71	211	12.23	0.671	0.872
144	39	3.69	3.07	442	27880	10.01	13.71	211	13.19	0.715	0.991
175	43	4.07	3.36	588	32860	11.01	14.71	211	14.15	0.758	1.115
211	55	3.84	3.30	696	38300	12.01	15.71	211	15.11	0.800	1.240
260	70	3.71	3.29	855	44100	13.00	16.71	211	16.08	0.841	1.371

Table 4.1.a. Physical and hydraulic properties of cableway section, EF. For explanation of notation and determination of values see text.

H_e (ft A.O.D)	h_e (ft)	S_e $(x10^3)$	S_w $(x10^3)$	A_t (ft^2)	W (ft)	D (ft)	P_t (ft)	R_t (ft)	Q (cusec)	V (ft/sec)	A_b (ft^2)
48	1.75	0.133	0.136	755.0	201	3.76	209	3.61	439.9	0.583	755
49	2.75	0.330	0.360	960.7	209	4.60	219	4.39	1291	1.34	957
50	3.75	0.488	0.526	1117	212	5.52	222	5.27	2579	2.20	1160
51	4.75	0.620	0.658	1384	214	6.46	225	6.1[illegible]	4301	3.10	1363
52	5.75	0.734	0.768	1598	215	7.43	228	7.01	6454	4.03	1566
53	6.75	0.833	0.862	1814	217	8.36	233	7.78	9038	4.98	1769
54	7.75	0.922	0.945	2032	219	9.28	234	8.68	12050	5.93	1972
55	8.75	1.002	1.018	2252	221	10.19	236	9.54	15480	6.88	2175
56	9.75	1.075	1.083	2474	224	11.04	240	10.31	19340	7.82	2378
57	10.75	1.142	1.143	2699	227	11.89	245	11.02	23630	8.75	2581
58	11.75	1.203	1.198	2928	230	12.73	250	11.71	28210	9.67	2784
59	12.75	1.261	1.248	3161	236	13.39	254	12.44	33440	10.58	2986
60	13.75	1.314	1.297	3401	245	13.88	266	12.79	39000	11.47	3190
61	14.75	1.365	1.339	3653	260	14.05	281	13.00	44950	12.30	3393

In section 3.3 the total cross-sectional wetted perimeter, P_t, was divided into "bed", that part of the perimeter composed of gravel etc., P_b, and "bank", that part of the perimeter composed of bushes, grass etc., P_w. Hence $P_t = P_b + P_w$. Bed load computations were made only for the "bed" and in an attempt to eliminate the effect of the banks it was decided to assign part of the cross-sectional area of flow to the "bed" and part to the "banks". This was effected as shown in fig. 4.1.a.; the area of flow pertaining to the "bed", A_b, is bounded by the "bed", the water-surface and two verticals from the "bed-bank" division to the water-surface. The area of flow pertaining to the banks is given by $A_w = A_t - A_b$. From table 3.2.a. it can be seen that the level of the "bed-bank" division at section EF is 48.21 ft A.O.D., the width of the "bed", $w = 203$ ft and the wetted perimeter of the "bed", $P_b = 211$ ft. The hydraulic radius of the "bed", $R_b = A_b/P_b$ and the hydraulic radius of the "banks", $R_w = A_w/P_w$. Assuming a Manning coefficient for the "banks", $n_w = 0.040$, the mean velocity in the area pertaining to the "banks" is given by:-

$$V_w = \frac{1.486}{n_w} R_w^{2/3} S_e^{\frac{1}{2}}$$

and the discharge in the area pertaining to the "banks", $Q_w = A_w V_w$. The discharge in the area pertaining to the "bed", $Q_b = Q - Q_w$ and the mean velocity in the area pertaining to the "bed", $V_b = Q_b/A_b$. The mean depth of flow in the area pertaining to the "bed", $D_b = A_b/w$. The average tractive force on the bed, $\tau_o = \gamma_f R_b S_e$ and the shear velocity $u_* = \sqrt{\tau_o/\rho_f} = \sqrt{gR_b S_e}$.

4.1.2. Schields Method

SCHIELDS (1936) used the results of his own experiments and those of GILBERT (1914) on uniform materials of diameter 1.56 to 2.47 mm and specific gravity 1.06 to 4.25 to produce the dimensionless equation:-

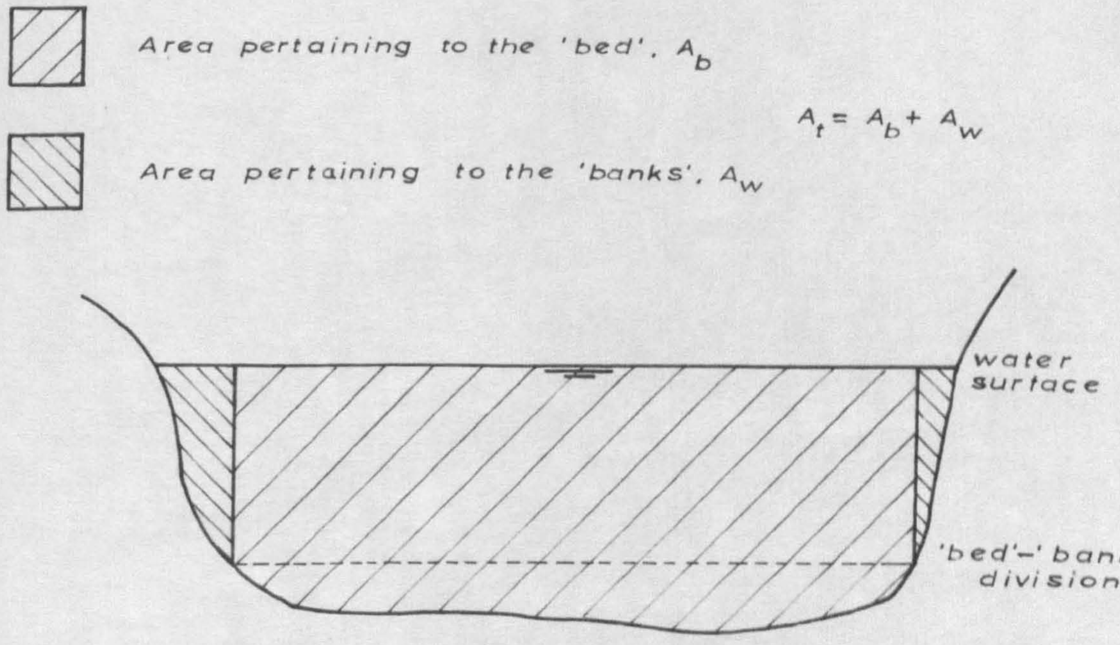


Fig. 4.1. a. Division of cross-sectional area of flow.

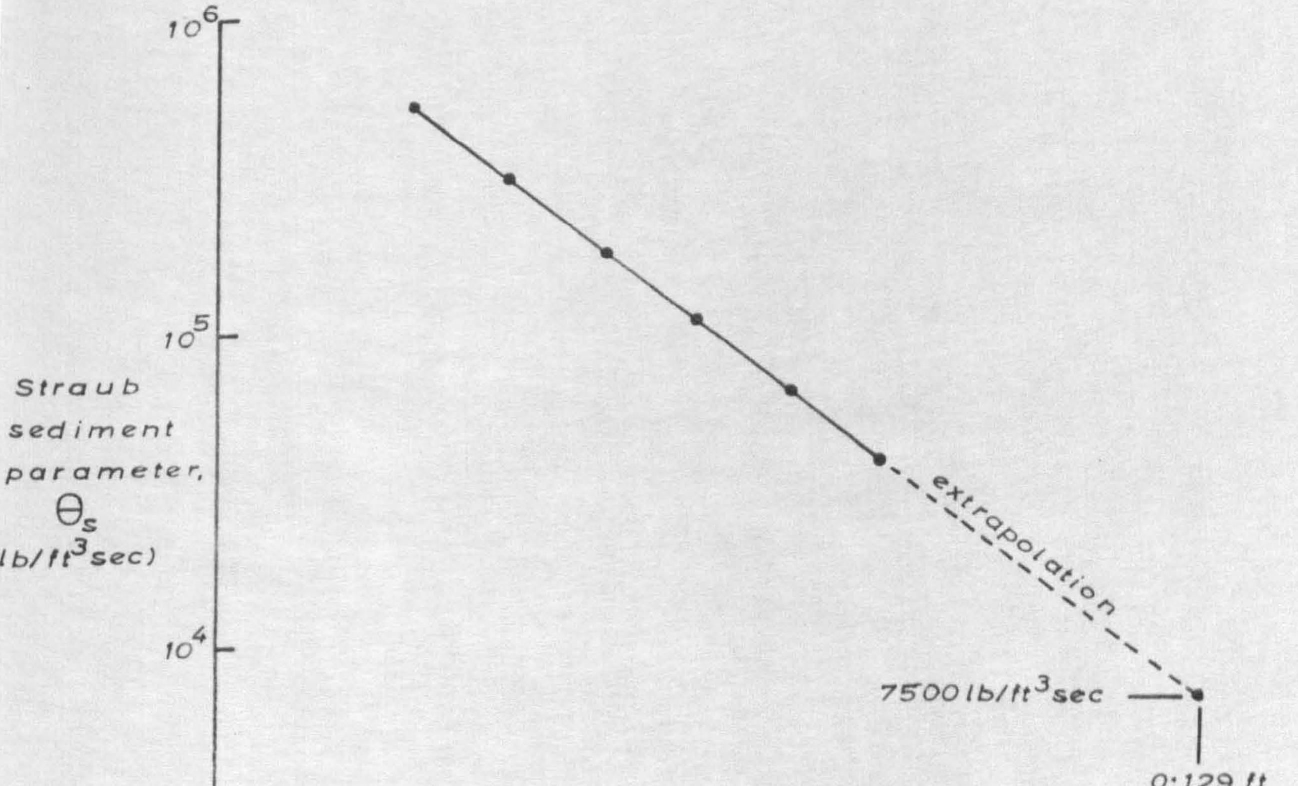


Fig. 4.1. b. Determination of Straub sediment parameter.

$$\frac{q_B}{q_b S_e \gamma_f} = 10(\tau_* - \tau_{*c}) = \frac{10(\tau_o - \tau_c)}{\gamma_f(S_s - 1)d} \quad \ldots\ldots\ldots \quad 4.1.a.$$

where q_B is the bed load discharge by weight in air per unit width per unit time, q_b is the water discharge pertaining to the "bed" by volume per unit width per unit time, and d is the sediment diameter. From the results of the MEYER-PETER and MÜLLER (1948) experiments it appears that for sediment mixtures the representative diameter, d_m, (see section 4.1.8) should be used in the Schields critical tractive force equation for $R_{e*} > 10^3$:-

$$\tau_c = 0.055(\gamma_s - \gamma_f)d$$

For the bed material at Bywell $d_m = 0.151$ ft and using $R_{e*} = \frac{u_* d_m}{\nu}$ it is found that R_{e*} varies from 1.27×10^3 at $H_e = 48$ ft to 8.66×10^3 at $H_e = 61$ ft. Hence $\tau_c = 0.055(\gamma_s - \gamma_f)d_m = 0.829$ lb/ft^2. Equation 4.1.a. can be rewritten:-

$$Q_B = \frac{10Q_b S_e}{(S_s - 1)d_m} (\tau_o - \tau_c)$$

in which Q_B is the bed load discharge by weight in air per unit time and Q_b is the water discharge pertaining to the "bed" in volume per unit time. Values of Q_B were calculated for 1 ft intervals of stage and the results plotted in fig. 4.1.m.

No references concerning the actual application of this formula were found but its dimensionless homogeneity and the inclusion of specific gravity are to be noted.

4.1.3. Straub Method

STRAUB (1939) proposed a formula for the determination of bed load discharge based directly on the Du Boys theory. For granular materials in the range of fine sands and gravels:-

$$q_B = \frac{\Theta_s}{\gamma_f^2} \tau_o (\tau_o - \tau_c) \quad \ldots\ldots\ldots\ldots\ldots \quad 4.2.b.$$

where Θ_s is a sediment parameter. Combining the results of his own experiments on uniform materials with those of GILBERT (1914) et al he produced a table giving values of τ_c for use in the above equation for particle sizes from 0.125 to 4.0 mm. Comparison of these values with those given by the Schields critical tractive force equation showed that the representative size of a mixture would have to be somewhat smaller than the Meyer-Peter and Müller representative diameter, d_m. Accordingly, the median diameter, d_{50}, was used, giving τ_c = 0.109 lb/ft^2. Straub also produced a graph giving values of Θ_s for sizes from 0.125 mm (0.00041 ft) to 4 mm (0.0131 ft); fig. 4.1.b. shows the extent of extrapolation required to obtain Θ_s = 7500 lb/ft^3 sec for d_{50} = 0.129 ft. Equation 4.1.b. was used to calculate $Q_B = wq_B$ and the results plotted in fig. 4.1.m.

HUBBELL and MATEJKA (1959) applied this method to a sand-bed river in Nebraska, United States and found that it gave bed load discharges of nearly twice the total load measured at a specially constructed turbulence flume in which the whole sediment load of the river could be put into suspension and measured.

4.1.4. Egiazaroff Method

Combining a theory based on the elements of dimensional analysis with the results of experiments by GILBERT (1914), the UNITED STATES WATERWAYS EXPERIMENTAL STATION (1935) and several Russian workers on sediment sizes ranging from 0.21 mm to 40.5 mm, EGIAZAROFF (1957, 1959) established the following relationship for fully developed turbulence:-

$$\frac{q_B}{q_b \gamma_f S_o^{\frac{1}{2}}} = 0.0225 \frac{(\tau_o - \tau_c)}{(S_s - 1)\ \tau_c} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad 4.1.c.$$

The experimental data showed a certain amount of scatter which Egiazaroff attributed to non-homogeneity of supposedly uniform sediments, the influence of particle shape, and the impossibility of defining the critical conditions of movement with precision.

The Schields criterion of critical tractive force was used in the development of the equation so it was decided to calculate τ_c for the bed sediment at Bywell from $\tau_c = 0.055\ \gamma_f(S_s-1)d_m$, in which $d_m = 0.151$ ft. Hence $\tau_c = 0.829$ lb/ft^2. Rewriting equation 4.1.c.:-

$$Q_B = 0.0225\ \frac{\gamma_f\ S_e^{\frac{1}{2}} Q_b}{(S_s - 1)}\ \frac{(\tau_o - \tau_c)}{\tau_c}$$

Substitution of the appropriate values gave the bed load rating curve shown in fig. 4.1.m.

In later investigations EGIAZAROFF (1965) extended the thoory to include all ranges of flow, i.e. turbulent, transitional and laminar, as well as for hydraulically smooth and rough boundaries. This involved tho inclusion on the right-hand side of equation 4.1.c. of a coefficient dependent upon the velocity distribution near the bed.

4.1.5.. Yalin Method

Combining a mathematical consideration of the saltation paths of sediment particles in water with the results of dimensional analysis YALIN (1963) derived an expression for the bed load discharge produced by a steady, turbulent flow over a plane bed composed of grains of equal size and shape. Experimental coefficients were obtained using the results of experiments by GILBERT (1914) and MEYER-PETER and MÜLLER (1948) with sizes ranging from 0.32 to 28.6 mm. The equation can be written:-

$$q_B = 0.635\ \gamma_f d\ u_* P_1 \left[1 - \frac{1}{P_1 P_2} \log_e(1 + P_1 P_2)\right] \quad . \quad . \quad . \quad . \quad 4.1.d.$$

in which $P_1 = \left[\frac{\tau_o}{\tau_c} - 1\right]$ and $P_2 = (2.45\ \tau_{*c}^{\frac{1}{2}}) / S_s^{2/5}$

The application of the formula to a river bed with a large range of sizes is questionable; indeed, the role of saltation in the fluid transport of sediment particles is considered by KALINSKE (1942) and EINSTEIN (1941) to be of negligible importance. As suggested by NORDIN and BEVERAGE (1964) the median diameter $d_{50} = 0.129$ ft, has been used to calculate τ_c from the Schields equation. The dimensionless critical shear parameter, $\tau_{*c} = 0.055$, giving $P_2 = 0.393$. Substitution of the appropriate values in equation 4.1.d. and multiplication by the bed width, w, produced the rating curve shown in fig. 4.1.m.

4.1.6. Schoklitsch Method

SHULITS (1935) has discussed comprehensively a formula for the prediction of bed load discharge proposed by Schoklitsch in Germany in 1934. The two basic assumptions are that the bed material will begin to move at some critical value of water discharge and that the bed load discharge is a function of the work done by that part of the tractive force in excess of that required to overcome the resistance of the wetted perimeter. The necessary experimental coefficients were derived by Schoklitsch from the results of flume measurements by GILBERT (1914) and additional data of his own. Although the sediments used were uniform-sized (0.31 to 4.9 mm), Schoklitsch suggested that the formula could be used to compute the discharge of individual size ranges of a mixture, the total load being the summation of the individual discharges. Hence if d, in feet, is the geometric mean of a size range and i_b is the proportion by weight of that size range in the bed material, then the bed load discharge of that size range, $i_B Q_B$ in lb/sec, where i_B is the proportion by weight of that size range in the bed load, is given by:-

$$i_B Q_B = \frac{25.03}{d^{\frac{1}{2}}} \; i_b S_e^{3/2} (Q_b - Q_{bc}) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad 4.1.e.$$

where Q_{bc} is the critical value of water discharge in cusecs pertaining to the bed and is given by:-

$$Q_{bc} = 0.0638 \; wd/S_e^{4/3} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad 4.1.f.$$

in which w is the width of the bed, in feet. The total bed load discharge, $Q_B = \sum i_B Q_B$.

Since the cumulative particle size analysis distribution of the bed material (fig. 3.5.e.) is open-ended it was decided to compute the bed load discharge of the particle sizes between d_{99} = 7.20 in and d_1 = 0.0084 in. Table 4.1.b. gives the size ranges into which the bed material was divided.

Table 4.1.b. Bed material size ranges

Size range (in)	Geometric mean of size range (in)	Geometric mean of size range (ft)	i_b ($x10^2$)
< 0.0084			1.0
0.0084 - 0.020	0.0103	0.000862	5.2
0.020 - 0.040	0.0283	0.00236	4.6
0.040 - 0.080	0.0566	0.00472	3.0
0.080 - 0.160	0.113	0.00944	2.6
0.160 - 0.320	0.226	0.0188	5.4
0.320 - 0.640	0.452	0.0376	10.0
0.640 - 0.904	0.761	0.0634	3.2
0.904 - 1.280	1.073	0.0894	10.0
1.280 - 1.808	1.521	0.1267	10.0
1.808 - 2.560	2.146	0.1783	14.0
2.560 - 3.618	3.042	0.2534	16.5
3.618 - 5.120	4.292	0.3576	8.8
5.120 - 7.200	6.071	0.5060	4.7
7.200 <			1.0

Since S_e varied with Q_b for the Bywell reach it was necessary to solve equation 4.1.f by graphical means to obtain Q_{bc} for a given value of d. Using equation 4.1.e. the individual discharges of each size range were then calculated and summated as shown in table 4.1.c. and plotted in fig. 4.1.c.

This method of application to sediment mixtures neglects the hiding effect of small particles in the laminar sublayer and behind larger particles, an influence which could be significant in gravel bed rivers with a large range of sizes. It also assumes an unlimited

TABLE 4.1.c.

range of sediment size, d (ft)						$Q_B = \sum i_B Q_B$ (lb/sec)	Q_B using d_{40} (lb/sec)
0.0894	0.1267	0.1788	0.2534	0.3576	0.5060		
-	-	-	-	-	-	-	-
-	-	-	-	-	-	0.14	-
-	-	-	-	-	-	1.25	-
-	-	-	-	-	-	4.01	-
-	-	-	-	-	-	9.14	-
-	-	-	-	-	-	17.54	-
-	-	-	-	-	-	29.59	-
0.74	-	-	-	-	-	46.16	7.75
1.94	0.76	-	-	-	-	68.12	19.93
3.48	1.97	0.31	-	-	-	95.46	35.55
5.37	3.48	2.62	0.54	-	-	130.84	54.63
7.63	5.31	4.66	2.41	-	-	171.91	77.54
10.29	7.47	7.13	4.69	0.81	-	220.85	104.39
13.34	9.96	9.94	7.36	1.93	-	276.94	135.28

Table 4.1.c. Bed load discharge computations by the Schoklitsch method

H_e (ft A.O.D)	$i_B Q_B$ (lb/sec) for each						
	0.000862	0.00236	0.00472	0.00944	0.0188	0.0376	0.0634
48	-	-	-	-	-	-	-
49	0.14	-	-	-	-	-	-
50	0.85	0.32	0.08	-	-	-	-
51	2.39	1.08	0.40	0.14	-	-	-
52	4.95	2.39	0.97	0.46	0.37	-	-
53	8.70	4.35	1.85	0.97	1.06	0.61	-
54	13.81	7.03	3.06	1.68	2.05	1.78	0.18
55	20.37	10.49	4.64	2.62	3.37	3.39	0.54
56	28.57	14.83	6.61	3.81	5.07	5.50	1.03
57	38.46	20.08	9.01	5.25	7.15	8.81	1.64
58	50.03	26.24	11.83	6.96	9.61	11.78	2.38
59	63.59	23.45	15.15	8.97	12.52	14.95	3.27
60	79.12	41.72	18.94	11.27	15.87	19.24	4.30
61	96.72	51.10	23.24	13.89	19.69	24.29	5.48

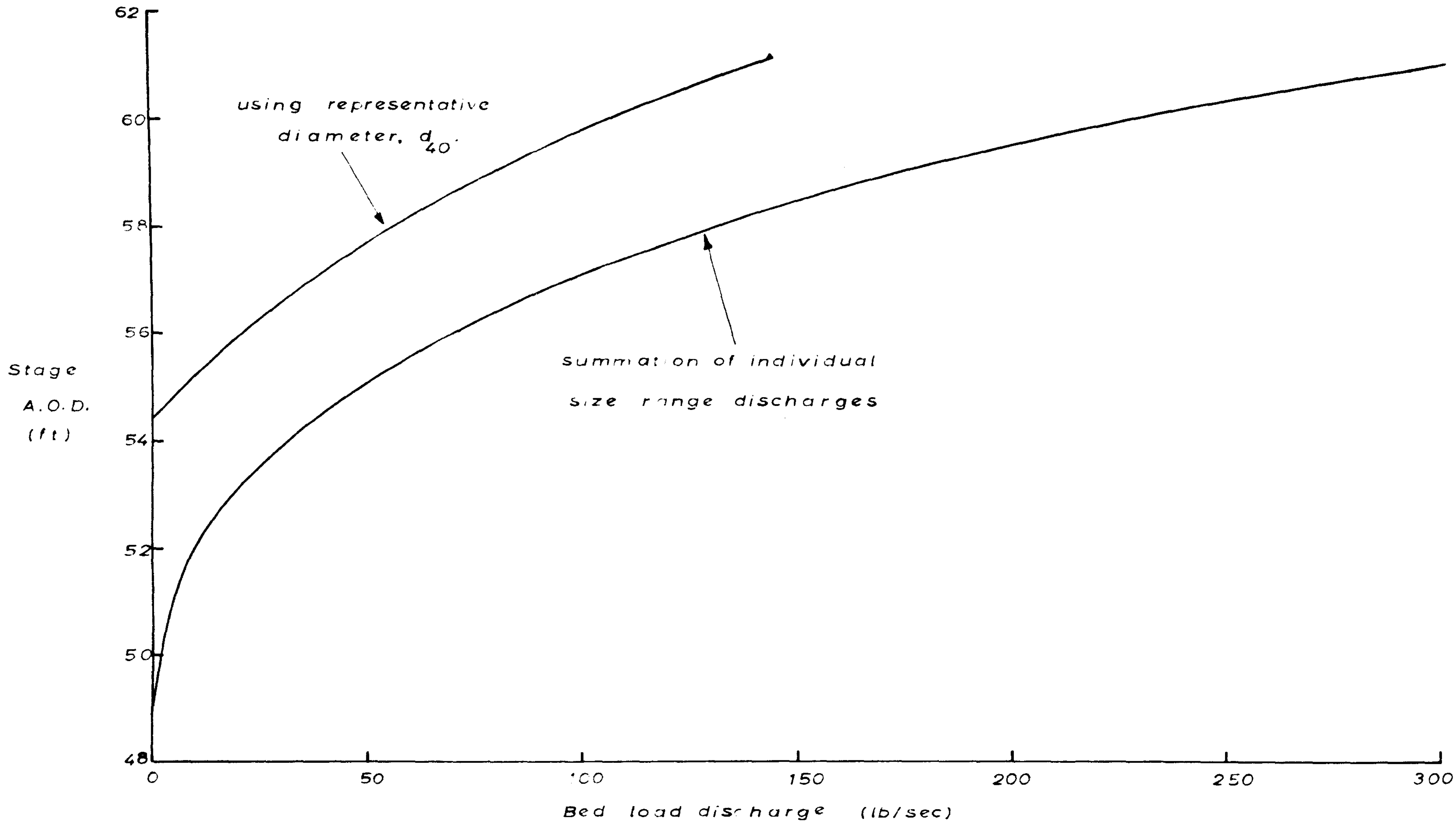


Fig. 4.1.c. Bed load rating curves by the Schoklitsch method.

supply of fine material from the bed. According to SHULITS (1935) and KRESSER and LASZLOFFY (1964) the results of bed load measurements on the Rivers Danube and Terek agree reasonably well with the Schoklitsch formula when d_{40} is used as the representative diameter of a mixture. From fig. 3.5.e. d_{40} = 1.05 in = 0.0875 ft for the Bywell bed material, and this value was used in equations 4.1.e. and 4.1.f. as shown in table 4.1.c. and plotted in fig. 4.1.c.

Since the method as applied to mixtures neglects some important factors it was considered that bed load discharges computed using a single representative diameter were more reliable. Therefore, the bed load rating curve obtained using d_{40} has been plotted in fig. 4.1.m. for comparison with the results of other bed load formulae.

4.1.7. Meyer-Peter and Müller Method

MEYER-PETER and MÜLLER (1948) described the development of an empirical bed load formula which can be shown to obey the Froudian law of similarity. The results of preliminary laboratory experiments with uniform materials of natural specific gravity were found to agree with the following equation:-

$$\frac{q^{2/3} S_e}{d} = K_2 + K_3 \frac{q_B^{2/3}}{d} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad 4.1.g.$$

in which K_2 and K_3 are constants. Further tests were carried out with mixtures and, although it was found possible to represent a sediment mixture by a single diameter, $d_m = \sum d i_b$, satisfactory agreement with equation 4.1.g. could not be obtained. A series of special tests concerning the commencement of motion indicated that for most cases the value of $\gamma_f D S_e / d_m$ was a constant at critical conditions, i.e. $\tau_c \propto d_m$, which confirmed the Schields equation for fully developed turbulence. Since commencement of movement was dependent upon a limiting shear stress Meyer-Peter and Müller concluded that the bed load discharge should also be a function of shear stress. Accordingly, equation 4.1.g. was altered to:-

$$\frac{\gamma_f D S_e}{d_m} = K_4 + K_5 \frac{q_B^{2/3}}{d_m} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad 4.1.h.$$

in which K_4 and K_5 are constants.

A large number of tests with mixtures of various sizes were carried out but it was found that bed load discharges occurring with ripples, dunes or shoals gave very poor agreement. It was evident that the shear stress on the bed was taken by two distinct resistances:-

1) Form resistance due to the unevenness of the shape of the bed.
2) Grain resistance due to friction on the individual particles of the bed.

Meyer-Peter and Müller therefore divided the total energy slope into two corresponding partial slopes and by means of the Strickler formula showed that the pure frictional slope was equal to $\left(\frac{k_t}{k_r}\right)^2 S_o$, where k_t is the coefficient of total roughness in the Strickler equation, $V = k_t D^{2/3} S_e^{\frac{1}{2}}$, and k_r is the coefficient of particle friction with a plane bed. Since only the energy dissipated by grain resistance is effective in the transport of bed load, S_e in equation 4.1.h. should be replaced by $\left(\frac{k_t}{k_r}\right)^2 S_e$. However, evaluation of test results and other considerations indicated that the exponent of $\left(\frac{k_t}{k_r}\right)$ should be adjusted to 3/2, with equation 4.1.h. rewritten as:-

$$\gamma_f \left(\frac{Q_b}{Q}\right)\left(\frac{k_t}{k_r}\right)^{3/2} \frac{D_b S_e}{d_m} = K_6 + K_7 \frac{q_B^{2/3}}{d_m}$$

in which K_6 and K_7 are constants and Q, Q_b and D_b have been introduced to account for the effect of bank friction in open channel flow.

Further experiments with materials of different densities enabled a final dimensionless form of the equation to be proposed:-

$$\left(\frac{\gamma_f}{\gamma_s - \gamma_f}\right)\left(\frac{Q_b}{Q}\right)\left(\frac{k_t}{k_r}\right)^{3/2}\frac{D_b S_e}{d_m} = 0.047 + \frac{0.25}{d_m}\left(\frac{\gamma_f}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma_f}{\gamma_f}\right)^{2/3}\left(\frac{q_B}{\gamma_s - \gamma_f}\right)^{2/3} \quad . \quad 4.1.i.$$

Flume experiments on which the formula was based covered the following ranges:-

Particle size	:	0.016 to 1.15 in	Specific gravity	:	0.25 to 3.20
Depth of flow	:	0.033 to 3.80 ft	Slope	:	0.4×10^{-3} to 20×10^{-3}

At zero bed load discharge on a flat bed in a flow without bank friction equation 4.1.i reduces to:-

$$\gamma_f D S_e = \tau_c \approx 0.05\ (\gamma_s - \gamma_f) d_m$$

which agrees closely with the Schields equation for the critical tractive force in fully turbulent flow. It can be concluded then that the representative diameter $d_m = \Sigma\, d\, i_b$ should be used for sediment mixtures; its value depends on the whole particle size distribution.

Equation 4.1.i was applied to the Bywell test reach as shown in table 4.1.d. The required values of Q, Q_b, D_b and S_e are listed in table 4.1.a. From table 4.1.b. the representative diameter of the bed material, $d_m = \Sigma\, d i_b$, was calculated to be 0.151 ft. The coefficient of roughness in the Strickler formula was calculated at each stage from:-

$$k_t = \frac{V_b}{R_b^{2/3} S_e^{\frac{1}{2}}}$$

and has the units of $ft^{1/3}$/sec. From test results Meyer-Peter and Müller noted that the coarse particles of a mixture are most effective in determining the grain roughness of a sediment mixture. They suggested that the coefficient of particle friction with a smooth bed, $k_r = 38.64/d_{90}^{1/6}$, where d_{90} is expressed in feet and k_r has the units $ft^{1/3}/sec$. From the particle size distribution curve of the bed material, fig. 3.5.e., $d_{90} \doteq 0.350$ ft. giving $k_r = 46.03\ ft^{1/3}/sec$. Total bed load discharge, $Q_B = wq_B$, was computed at one foot intervals of stage and plotted in fig. 4.1.m.

Table 4.1.d. Bed load discharge computations by the Meyer-Peter and Müller method

H_e (ft. A.O.D.)	$\frac{Q_b}{Q}$	k_t ($ft^{1/3}/sec$)	$\left(\frac{k_t}{k_r}\right)^{3/2}$	S_e ($x10^3$)	D_b (ft)	q_B (lb/sec.ft)	Q_B (lb/sec)
48	1.000	21.61	0.321	0.133	3.76	-	-
49	0.998	27.09	0.451	0.330	4.71	-	-
50	0.997	32.10	0.582	0.483	5.71	-	-
51	0.994	36.36	0.702	0.620	6.71	-	-
52	0.992	39.67	0.800	0.734	7.71	-	-
53	0.991	42.66	0.892	0.833	8.71	-	-
54	0.898	44.92	0.964	0.992	9.71	-	-
55	0.988	46.91	1.029	1.002	10.71	-	-
56	0.987	48.71	1.088	1.075	11.71	0.1766	35.8
57	0.985	50.26	1.140	1.142	12.71	0.6150	124.8
58	0.985	51.74	1.192	1.203	13.71	1.269	257.6
59	0.983	53.02	1.236	1.261	14.71	2.092	424.7
60	0.982	54.17	1.277	1.314	15.71	3.095	628.7
61	0.981	55.26	1.315	1.365	16.71	4.299	872.7

The Meyer-Peter and Müller formula, with slight modifications, has been found to describe closely bed load discharges in some sand-bed rivers in Holland (TOPS, WEMELSFELDER and VOLKER, 1959), Nigeria (NEDECO, 1959) and the United States of America (HUBBELL and MATEJKA, 1959). GEMAEHLING, GINOCCHIO and CHABERT (1957) reported that the total quantities of coarse material (median diamater about 2 in) transported during several floods in the central portion of the River Rhone agreed well with those computed by the formula. BAUER (1965) also found good agreement with bed load movements in the River Danube with sediment of representative diameter of 0.75 in.

4.1.8. Kalinske Method

KALINSKE (1947) accepted two basic concepts in the development of a bed load transport formula. The first concept is that there is some minimum fluid force which will cause a sediment particle to move, and the second concept is that the shear stress on the particle may not be constant but will fluctuate due to turbulence about a mean value. For an estimate of the critical shear stress required to start movement Kalinske accepted the conclusion of WHITE (1940) (see section 2.2.2.) that:-

$$\tau_c = 0.12\ (\gamma_s - \gamma_f)d \quad . \quad . \quad . \quad 4.i.k.$$

From an analysis of data on pebble movements the velocity of a single particle at any instant was assumed to be equal to $U_g = U - U_c$, where U is the instantaneous fluid velocity at grain level and U_c is the critical fluid velocity. As shown in section 2.2.2. the number of particles per unit area of bed is $4p/\pi d^2$, where p is the proportion of the bed taking fluid shear. Hence the bed load discharge in dry weight per unit width of bed is given by:-

$$q_B = 2/3\ p\ \gamma_s d\bar{U}_g \quad . \quad . \quad . \quad 4.1.1.$$

where $\bar{U}_g$ is the mean particle velocity. If velocity fluctuations due to turbulence are in accordance with the normal error law then $\bar{U}_g$ can be obtained by integrating the error function for $U_g = U - U_c$ between the critical fluid velocity, U_c, and infinity:-

$$\bar{U}_g = \int_{U_c}^{\infty} (U - U_c) \frac{1}{\sigma\sqrt{2\pi}} \cdot e^{\frac{-(U - \bar{U})^2}{2\sigma^2}}\ dU$$

in which $\bar{U}$ is the mean fluid velocity at grain level and $\sigma = \sqrt{\overline{(U - \bar{U})^2}}$ is the standard deviation of the velocity fluctuations. After division by $\bar{U}$ it can be shown that $\bar{U}_g/\bar{U}$ is a function of the relative intensity of turbulence $r = \sigma/\bar{U}$ and the ratio of U_c to $\bar{U}$. Since shear stress varies with the square of velocity, then $U_c/\bar{U}$ can be written as τ_c/τ_o and:-

$$(\bar{U}_g/\bar{U}) = f_5(\tau_c/\tau_o) \quad . . . . \quad 4.1.1.$$

where $f_5(\tau_c/\tau_o)$ is a function which varies with r. Kalinske stated that for turbulent flow near the bed r had been found to equal $\frac{1}{4}$ and he described equation 4.1.1. by the graph shown in fig. 4.1.d.

According to Kalinske $\bar{U}$ is approximately equal to $11u_*$ which enables equation 4.1.k. to be written:-

$$q_B = 7.3\ p\ \gamma_s\ d\ u_*\ f_5\ (\tau_c/\tau_o) \quad . . . . \quad 4.1.m.$$

If the formula is to be applied to a mixture then the bed load discharge per unit width of a given size range is:-

$$i_B q_B = 7.3\ p_i\ \gamma_s d\ u_* f_5(\tau_c/\tau_o) \quad . . . . \quad 4.1.n.$$

in which p_i is the proportion of the bed area covered by grains in a size range and is given by pi_b/ad, where d is the geometric mean of the size range, i_b is the proportion by weight of the size range in the bed material, p is the proportion of the bed taking fluid shear (= 0.35 according to White) and $a = \Sigma(i_b/d)$.

Using the data of table 4.1.b. a was calculated to be equal to 98.64 enabling p_i to be calculated for each size range. Equation 4.1.j. was used to calculate τ_c for each size range and values of $f_5(\tau_c/\tau_o)$ were taken from fig. 4.1.d. for each value of τ_o. Bed load discharges of each size range over the whole width of section, $i_B Q_B = w i_B q_B$, were calculated from equation 4.1.n. and are given in table 4.1.e. with the summated discharges, $Q_B = \Sigma\ i_B Q_B$.

As with the Schoklitsch method, the application of the Kalinske method to mixtures neglects the mutual interference effects of particles of different sizes and assumes an unlimited supply of the finer sizes.

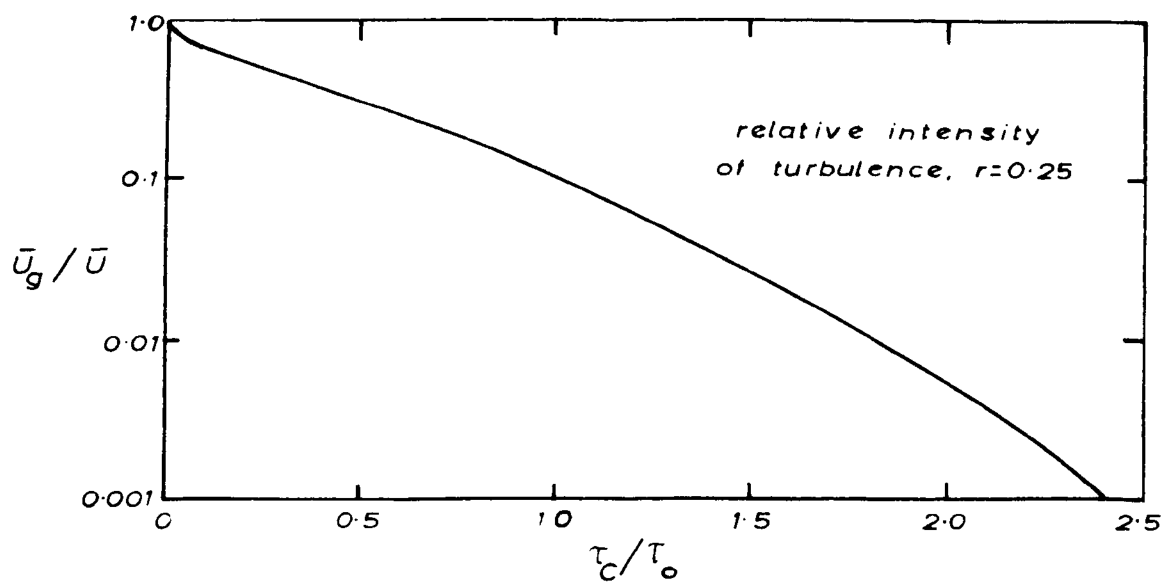


Fig. 4.1. d. Ratio of critical shear to bed shear after Kalinske.

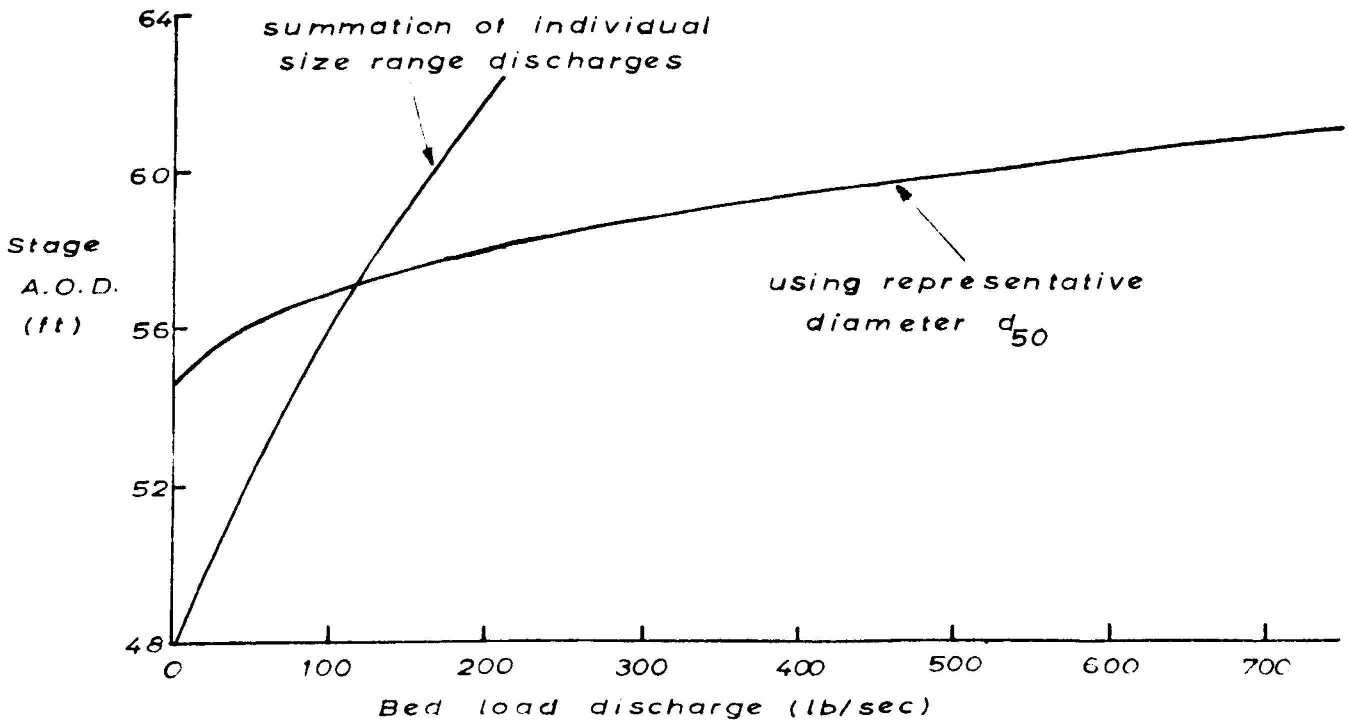


Fig. 4.1. e. Bed load rating curves by the Kalinske method.

Kalinske suggested that the median diameter could be used as a representative diameter. Bed load discharges were therefore computed using equation 4.1.m. with $p = 0.35$ and $d = d_{50} = 0.129$ ft. Total bed load discharges across the section are given in table 4.1.e. and plotted in fig. 4.1.e. for comparison with the summated discharges of the individual size ranges.

For the same reasons as those given in the discussion of the Schoklitsch method the bed load rating curve obtained using the median diameter of the bed material is considered more reliable and is plotted in fig. 4.1.m.

The principal merits of the Kalinske method are that it introduces the concept of fluid turbulence statistically and that it does not utilise the empirical results of any bed load experiments. However, the validity of certain assumptions such as $p = 0.35$, $U_g = U - U_c$, and $\bar{U} = 11u_*$ is uncertain. According to ELZERMAN and FRIJLINK (1951) measurements of bed load transport in Holland indicate that the constant 7.3 in equation 4.1.m. may in fact be as low as 5.

4.1.9. Einstein Method

EINSTEIN (1941, 1950) presented a complex method for the computation of the bed material discharge of an alluvial channel, i.e. the summation of the bed load discharge and suspended bed material discharge. This section is concerned solely with the part of the theory concerning bed load.

The method was developed for application to an average cross-section of a reach of river in which several cross-sections have been surveyed. It was necessary, therefore, before making any sediment computations to determine the physical and hydraulic properties of the representative cross-section of the Bywell reach, using the five main cross-sections AB, CD, EF, GH and JK. Each cross-section has been described by the two curves of stage against total cross-sectional area (fig. 3.3.f.) and stage against total wetted perimeter (fig. 3.3.g.). The corresponding curves for the representative section were obtained by sliding the curves of each section along an average slope into the plane of the cableway section, EF. Since the water-surface slope of the Bywell reach varies from 0.136×10^{-3} at $H_e = 48$ ft A.O.D. to 1.339×10^{-3} at $H_e = 61$ ft A.O.D. it was decided to use an average slope of 0.75×10^{-3}. The curves thus obtained were averaged directly to give the variation of A_t and P_t with H_e for the representative section (figs. 4.1.f. and 4.1.g., respectively). Since the plane of the cableway section, EF, was used as a common datum, H_e was taken to denote stage A.O.D. at the average section. Values of A_t, P_t and total hydraulic radius, $R_t = A_t/P_t$ are given for several values of H_e in table 4.1.f. together with water discharge, Q, and energy-surface slope, S_e, taken directly from table 4.1.a. (Preliminary calculations indicated that bed load movement was unlikely to occur below $H_e = 55$ ft., and for this reason only 2 ft. intervals of stage have been considered below this level).

The elimination of the influence of bank friction was effected in a manner similar to that described in section 4.1.1.; it was assumed that the level of the "bed-bank" division on the representative cross-section was 48.00 ft. A.O.D. and that the slope of the banks was 1 vertical to 3 horizontal. Calculated properties are given in table 4.1.f.

An important concept in the Einstein method is that the resistance to flow over a sediment bed is composed of two distinct types:-

1) Resistance due to the shape of the bed; the part of the flow energy which corresponds to shape resistance is transformed into turbulence at some distance from the bed

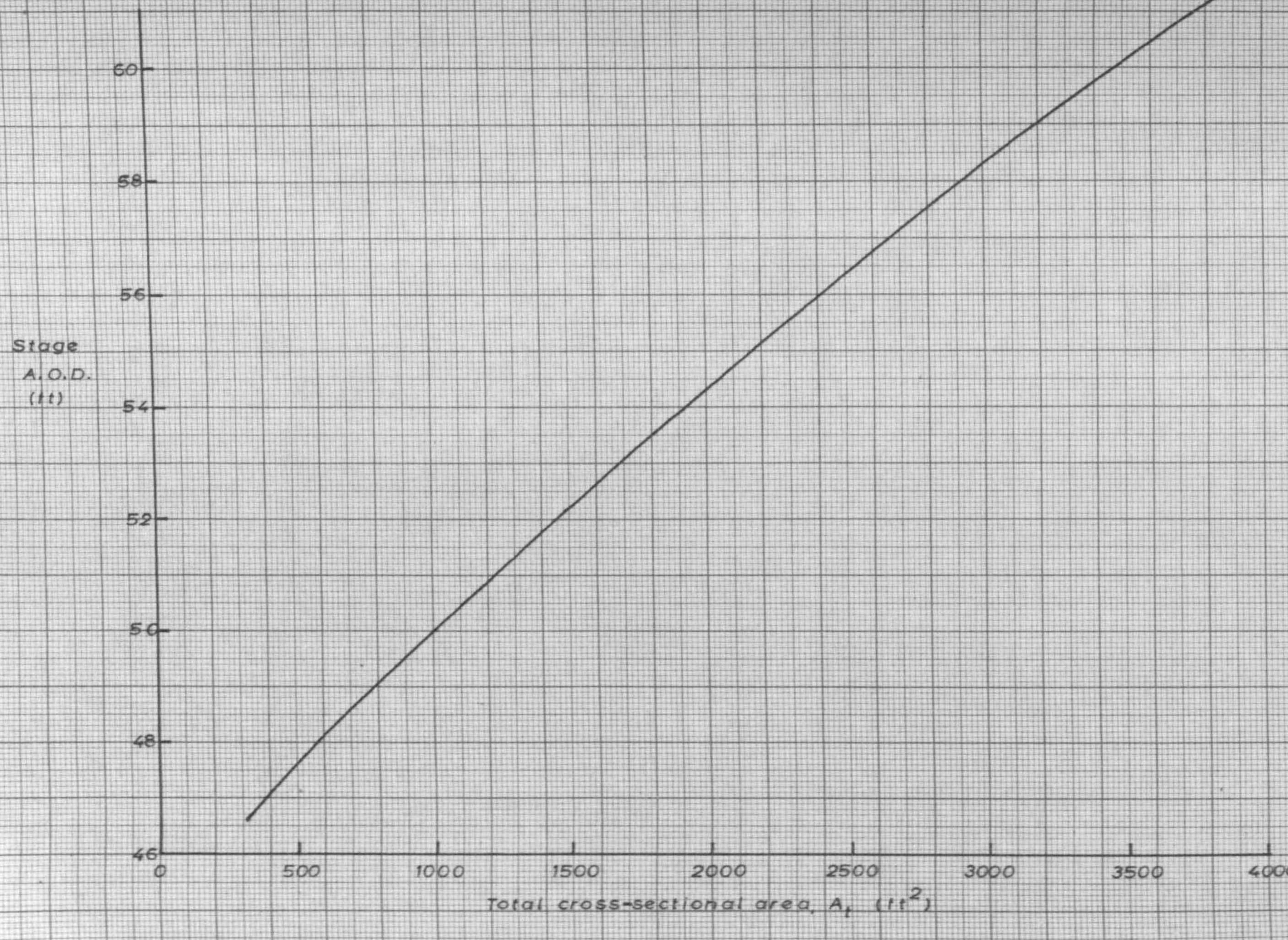


Fig. 4.1.1. Area – stage relationship for representative cross-section.

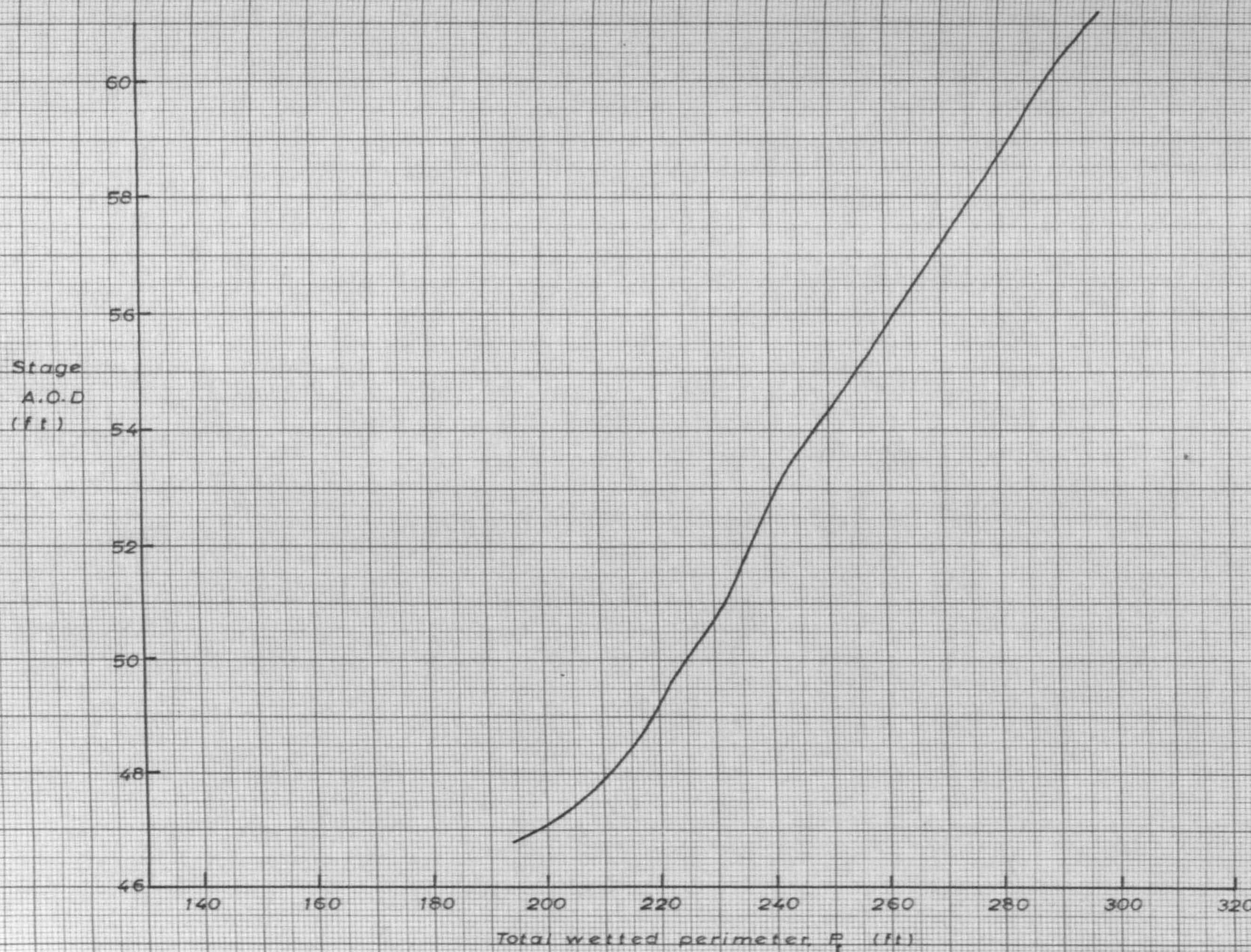


Fig. 4.1.g. Wetted perimeter—stage relationship for representative cross-section.

Table 4.1.f. Physical and hydraulic properties of representative section

H_e (ft A.O.D)	A_t (ft^2)	P_t (ft)	R_t (ft)	S_e ($x10^3$)	Q (cusec)	A_w (ft^2)	P_b (ft)	P_w (ft)	R_w (ft)	V_w (ft/sec)	Q_w (cusec)	Q_b (cusec)	A_b (ft^2)	V_b (ft/sec)
48	580	211	2.75	0.133	440	0	211	0	0	0	0	440	580	0.76
50	1010	225	4.49	0.448	2580	12	211	14	0.86	0.74	9	2570	998	2.58
52	1460	237	6.16	0.734	6450	48	211	26	1.85	1.52	73	6380	1412	4.52
54	1930	248	7.78	0.922	12050	108	211	37	2.92	2.30	248	11800	1922	6.48
55	2160	255	8.47	1.002	15480	147	211	44	3.34	2.62	385	15480	2013	7.50
56	2410	262	9.20	1.075	19340	192	211	51	3.77	2.95	566	18774	2218	8.46
57	2670	269	9.93	1.142	23630	243	211	58	4.19	3.26	792	23630	2427	9.41
58	2930	276	10.62	1.203	28310	300	211	65	4.62	3.57	1071	27239	2630	10.36
59	3190	282	11.31	1.261	33440	363	211	71	5.11	3.91	1419	33440	2827	11.33
60	3470	288	12.05	1.314	39000	432	211	77	5.61	4.25	1836	37164	3038	12.23
61	3750	296	12.67	1.365	44950	507	211	85	5.96	4.51	2287	44950	3243	13.16

and hence does not contribute significantly to the bed load transport of sediment particles.

2) Resistance due to the bed particles; the flow energy which is dissipated by turbulence in the immediate vicinity of the particles has a large effect on bed load movement.

MEYER-PETER and MÜLLER (1948) also noted this distinction (see section 4.1.6.) and assigned a part of the energy-surface slope to each resistance. Einstein, however, extended the principle of the distribution of total cross-sectional area of flow between "bed" (A_b) and "bank" (A_w) and divided A_b into A_b' and A_b'', areas corresponding to grain resistance and shape resistance, respectively. Both types of resistance are distributed over the entire "bed" surface and hence act along the same perimeter, P_b. In this way, S_e is held constant in the drag expression, $\tau_o = \gamma_f R_b S_e$, and $R_b' = A_b'/P_b$ becomes the hydraulic radius with respect to the particles and $R_b'' = A_b''/P_b$ becomes the hydraulic radius with respect to channel shape, such that:-

$$R_b = R_b' + R_b''$$

$$\text{giving} \quad \left.\begin{aligned} \tau_o' &= \gamma_f R_b' S_e \qquad & u_*' &= \sqrt{g R_b' S_e} \\ \tau_o'' &= \gamma_f R_b'' S_e \qquad & u_*'' &= \sqrt{g R_b'' S_e} \\ \tau_o &= \tau_o' + \tau_o'' \qquad & u_*^2 &= u_*'^2 + u_*''^2 \end{aligned}\right] \quad . \; . \; 4.1.o$$

According to Einstein the mean velocity in a vertical is given by the Keulegan Equation:-

$$\frac{V_b}{u_*'} = 5.75 \log_{10}\left(12.27 \frac{R_b' x}{k_s}\right) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; 4.1.p.$$

where k_s is the representative roughness diameter of the bed and considered by Einstein to equal d_{65}. The value of the corrective parameter x is dependent upon whether the flow is hydraulically rough, transitional or hydraulically smooth; it is a function of k_s/δ' as shown in fig. 4.1.h., where δ' is the thickness of the laminar sublayer calculated from:-

$$\delta' = 11.6\ \nu/u'_* \quad . . . . 4.1.q.$$

The derivation of the final Einstein bed load equation for sediment mixtures is lengthy and complex, and will be described only briefly here. By equating the number of particles being eroded from unit area of the bed in unit time to the number being deposited in unit area per unit time Einstein was able to express the probability that a given sediment particle would move as a function of the bed load discharge and physical properties of the fluid and sediment. Laboratory experiments indicated that this probability depended upon the size, shape and weight of the particle and upon the flow pattern near the bed. After postulating that the probability was a function of the ratio of the instantaneous hydrodynamic lift exerted by the flow to the submerged weight of the particle, Einstein concluded that the bed load equation representing the general relationship between bed load discharge, flow conditions and bed material is given by:-

$$\Phi_* = f_7\left(\Psi_*\right)$$

where Φ_* = the dimensionless intensity of bed load transport function.

$$= \frac{i_B}{i_b}\left[\frac{q_B}{\gamma_s}\left(\frac{\gamma_s}{\gamma_s - \gamma_f}\right)^{\frac{1}{2}}\left(\frac{1}{gd^3}\right)^{\frac{1}{2}}\right] \quad . . . . 4.1.r.$$

and Ψ_* = the dimensionless intensity of flow function

$$= \xi Y(\beta^2/\beta_x^{\ 2})\ \Psi \quad . . . . 4.1.s.$$

where $\Psi = \dfrac{(S_s -)d}{R'_b S_e}$, the intensity of shear on a single particle

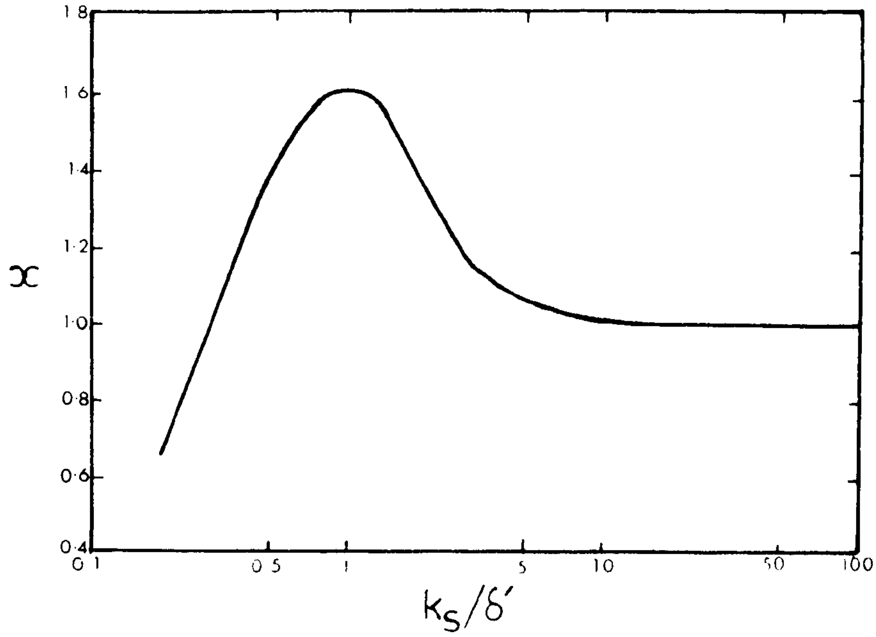


Fig. 4.1.h. Surface drag correction factor.

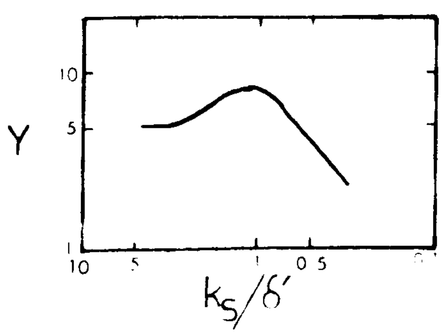


Fig. 4.1.i. Lift force correction factor.

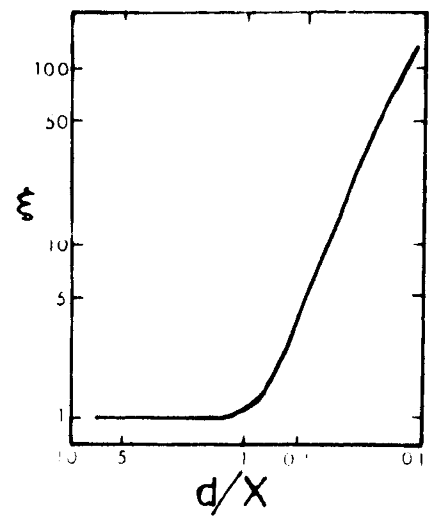


Fig. 4.1.j. Hiding correction factor.

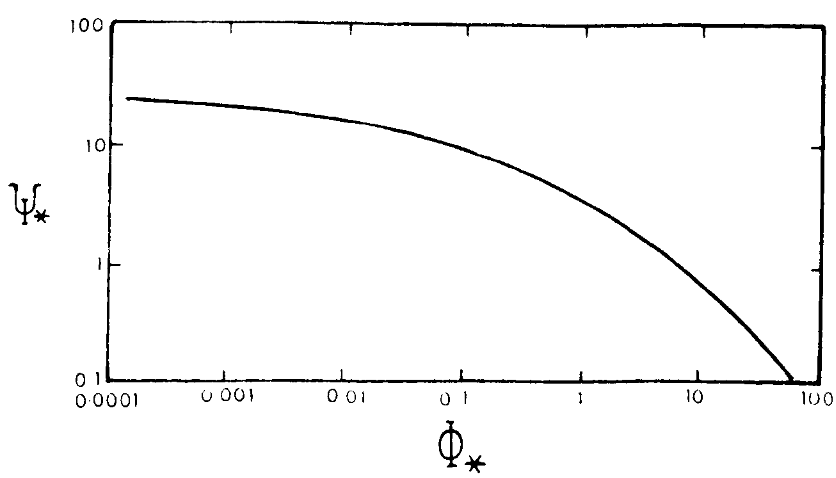


Fig. 4.1.k. Einstein bed load function.

$\beta = \log_{10} (10.6)$ . . . . 4.1.t.

$\beta_x = \log_{10} (10.6Xx/k_s)$ . . . . 4.1.u.

X = a reference particle size for a given bed. Laboratory experiments have shown that:-

$X = 0.77\ k_s/x$ if $k_s/x > 1.80\ \delta'$

$X = 1.39\ \delta'$ if $k_s/x < 1.80\ \delta'$

Y = a lift force correction factor necessary for mixtures with different roughness conditions. It is given as a function of k_s/δ' in fig. 4.1.i.

ξ = a correction factor for the effect of small particles which tend to hide between larger particles or in the laminar sublayer. It is given as a function of d/x in fig. 4.1.j. Recent research (EINSTEIN, 1964) indicates that, for values of d/x smaller than that at which $u_*'d/\nu = 3$, the factor remains constant.

Using the results of his own flume experiments and those of GILBERT (1914) on uniform materials between 0.315 and 28.6 mm Einstein produced a curve of the relationship, $\Phi_* = f_7\left(\Psi_*\right)$ as shown in fig. 4.1.k.

Before applying the bed load equation to the Bywell test reach further hydraulic calculations were made for the representative cross-section. In his paper EINSTEIN (1950) assumed that a stage-discharge curve for the test reach would not be available and proposed that it should be obtained by the following procedure. It was suggested that for most rivers the value of V_b/u_*'', which can be considered to be a function of the coefficient of resistance due to bed shape, should be a function of the flow intensity function for a representative particle size, $\Psi' = (S_s - 1)d_{35}/R_b'S_e$. The curve shown in fig. 4.1.l. is the results of investigations on several rivers in the United States of America with values of d_{35}

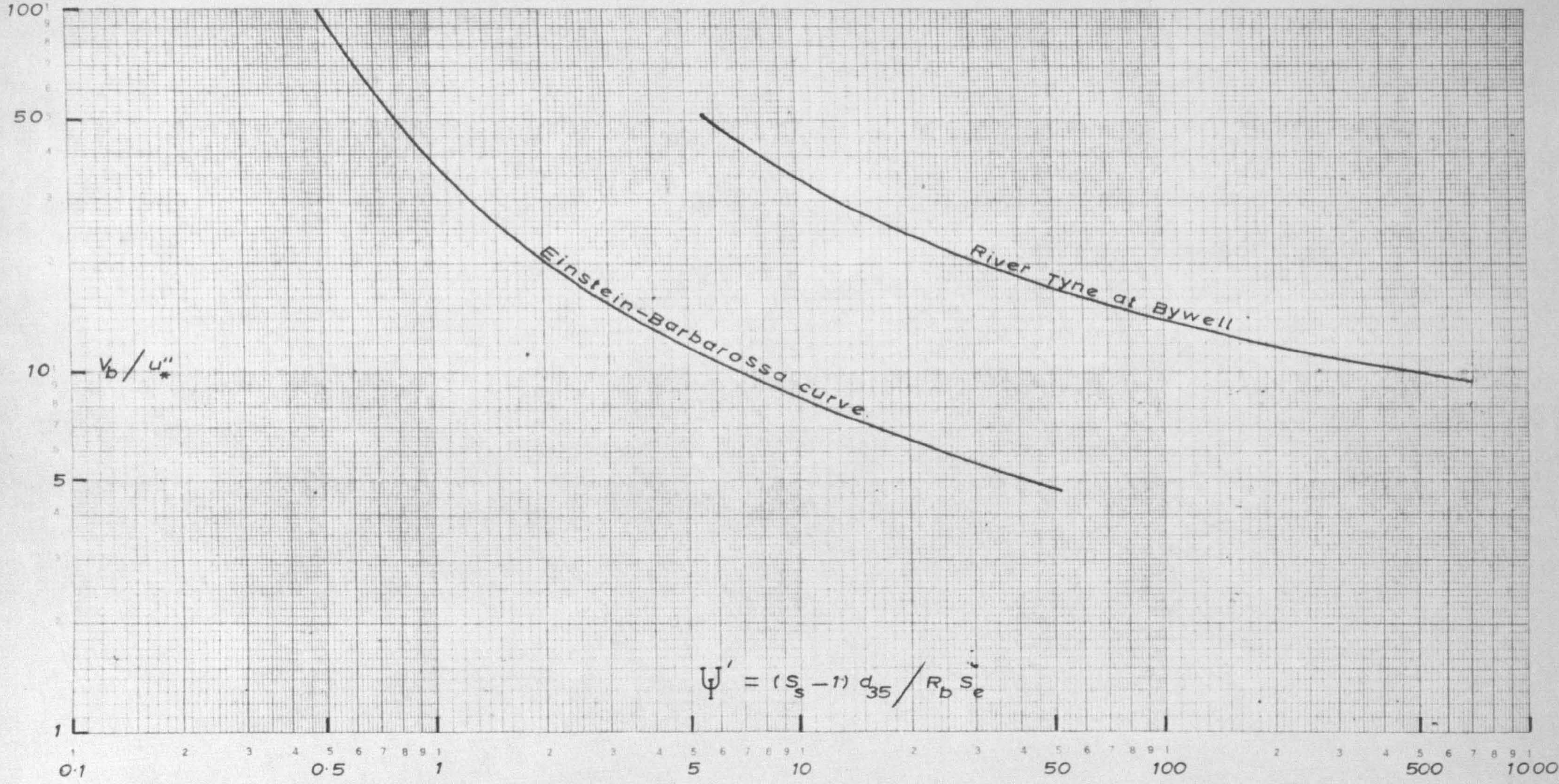


Fig. 4.1.1. Determination of bed shape resistance.

between 0.00052 ft and 0.0033 ft; the establishment of this curve, which purports to give the normal amount of shape resistance encountered in rivers, has been described in detail by EINSTEIN and BARBAROSSA (1951). Einstein recommended that an average value of S_e should be determined for the reach and, with an assumed value of R_b', equation 4.1.p. should be used to calculate V_b. From fig. 4.1.1. u_*'' could then be calculated and by using the relationship of equation 4.1.o. the value of R_b, and hence stage, corresponding to $Q_b = A_b V_b$ could be obtained.

Although a stage-discharge curve was available for the Bywell reach an attempt was made to follow the complete Einstein method. However, it was found that values of R_b'' obtained by the above procedure were extremely large and it was decided to compute the relationship between V_b/u_*'' and Ψ' using the data of table 4.1.f. The results of these computations are given in table 4.1.g., together with other hydraulic calculations required for the computation of bed load discharge. Assuming hydraulically rough flow, i.e. $x = 1$, and with $k_s = d_{65} = 0.196$ ft, equation 4.1.p. was used to obtain R_b', knowing V_b and S_e for each value of stage. Equations 4.1.o. and 4.1.q. enabled u_*', δ' and k_s/δ' to be calculated and since $k_s/\delta' > 10$ for all values of stage the assumption of $x = 1$ proved correct. Using equation 4.1.o. V_b/u_*'' was calculated and with $d_{35} = 0.0683$ ft, $\Psi' = (S_s - 1)d_{35}/R_b'S_e$. These calculations enabled the curve for the River Tyne at Bywell to be added to fig. 4.1.1.; the deviation from the Einstein and Barbarossa curve would seem to indicate that the importance of channel shape resistance as part of total resistance to flow is considerably less in gravel-bed rivers. BROOKS (1955) et al also found that observed shape resistance in both laboratory flumes and rivers differed from the recommended curve.

The remaining hydraulic calculations in table 4.1.g. were necessary before direct computations of bed load discharge could be made. Since $k_s/x > 1.80\delta'$ then $X = 0.77\ k_s/x = 0.77\ d_{65}$. The value of Y was obtained from fig. 4.1.i. and $(\beta/\beta_x)^2$ obtained from equations 4.1.t. and 4.1.u.

Bed load discharges were computed for each of the thirteen size ranges given in table 4.1.b., taking d equal to the geometric mean size of the range. For each size ξ was taken from fig. 4.1.j, and Ψ_* calculated from equation 4.1.s. Using the $\Phi_* - \Psi_*$ curve of fig. 4.1.k., Φ_* was obtained and the bed load discharge per unit width for that size range was calculated from equation 4.1.r., rewritten as:-

$$i_B q_B = i_b\ \gamma_s g^{\frac{1}{2}} d^{3/2} (S_s - 1)^{\frac{1}{2}}\ \Phi_*$$

The total transport rates for each size range over the whole section $i_B Q_B = w i_B q_B$ were calculated and summated to give $Q_B = \Sigma\ i_B Q_B$ (table 4.1.h.). The curve of total bed load discharge of all sizes is plotted against stage in fig. 4.1.m.

Although the concept of bed load movement embodied in the Einstein theory is more realistic than that of Du Boys or Yalin, the dependency of the method upon strictly empirical relationships such as figs. 4.1.i. and 4.1.j. and the definition of X tends to detract from its validity. In particular, the evaluation of the hiding factor, ξ, for a very large range of sizes is extremely uncertain; the results of table 4.1.h. show that particles smaller than about ½ inch are stated by the theory to remain stationary even at high tractive forces. It would seem, then, that the method is likely to predict only minimum rates of bed load transport. Another limitation of the method as proposed by Einstein is indicated by comparison of the shape resistance curve for the River

Table 4.1.g. Hydraulic calculations for the Einstein method

H_e (ft A.O.D)	R'_b (ft)	u'_* (ft/sec)	δ' (ft x 10^3)	k_s/δ'	R''_b (ft)	u''_* (ft/sec)	V_b/u''_*	Ψ'	X	Y	β_x	$\left(\beta/\beta_x\right)^2$
48	1.19	0.071	2.38	82.3	1.56	0.081	9.37	691	0.1509	0.54	0.91?	1.263
50	2.68	0.205	0.839	234	2.05	0.179	14.4	83.6	"	"	"	"
52	4.45	0.324	0.524	374	2.24	0.230	19.7	33.5	"	"	"	"
54	6.35	0.434	0.391	501	2.29	0.261	24.8	18.7	"	"	"	"
55	7.45	0.490	0.347	564	2.09	0.259	29.0	14.6	"	"	"	"
56	8.48	0.542	0.313	626	2.03	0.264	31.9	12.0	"	"	"	"
57	9.51	0.591	0.287	683	1.99	0.270	34.8	10.1	"	"	"	"
58	10.60	0.641	0.265	739	1.86	0.268	38.7	8.6	"	"	"	"
59	11.73	0.690	0.246	797	1.66	0.259	43.7	7.4	"	"	"	"
60	12.75	0.734	0.231	848	1.65	0.264	46.3	6.5	"	"	"	"
61	13.94	0.783	0.217	903	1.43	0.250	52.7	5.7	"	"	"	"

Table 4.1.h. Computation of bed load discharge by the Einstein method

H_e (ft A.O.D)	$i_B Q_B$ (lb/sec) for each size range of sediment, d (ft)													$Q_B = \Sigma\ i_B Q_B$ (lb/sec)
	0.000862	0.00236	0.00472	0.00944	0.0188	0.0376	0.0634	0.0894	0.1267	0.1788	0.2534	0.3576	0.5060	
48	-	-	-	-	-	-	-	-	-	-	-	-	-	-
50	-	-	-	-	-	-	-	-	-	-	-	-	-	-
52	-	-	-	-	-	-	-	-	-	-	-	-	-	-
54	-	-	-	-	-	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	0.40	-	-	-	-	0.40
56	-	-	-	-	-	-	-	-	2.83	1.99	-	-	-	4.82
57	-	-	-	-	-	-	-	0.80	10.17	9.55	-	-	-	20.52
58	-	-	-	-	-	-	-	3.15	21.48	26.53	7.65	-	-	58.81
59	-	-	-	-	-	-	-	7.37	41.82	55.71	24.27	-	-	129.17
60	-	-	-	-	-	-	0.06	13.39	68.95	92.85	52.78	3.11	-	231.14
61	-	-	-	-	-	-	0.13	18.75	90.43	124.69	79.16	7.07	-	320.23

Tyne at Bywell with that given by Einstein and Barbarossa (fig.4.1.1.). If a stage-discharge curve is available the evaluation of channel shape resistance is still dependent upon an accurate determination of the energy-surface slope of the reach. COLBY and HEMBREE (1955) and HUBELL and MATEJKA (1959) reported that total bed material discharges computed by the Einstein method for two sand-bed rivers in the United States compared poorly with suspended sediment loads measured at specially constructed turbulent sections at which all sediment was brought into suspension. The particle sizes transported were also found to be considerably different from predicted sizes, a result which has been confirmed by STALL, RUPANI and KANDASWAMY (1958).

4.1.10. Modified Einstein Method

Studies of total sediment load measured in a contracted, natural rock section of a sand-bed river in the United States of America have led to the development by COLBY and HEMBREE (1955) of a modified Einstein procedure for the computation of total bed material discharge. There are four modifications to the original procedure for calculating bed load discharge:-

1) Computations are made for a single cross-section at which the channel shape and stage-discharge curve are known. For application of the method to the Bywell reach the cableway section, EF, was used.

2) The mean velocity equation 4.1.p. is modified to:-

$$\frac{V_b}{u_m} = 5.75 \log_{10}(12.27 \frac{xD_b}{k_s}). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 4.1.v.$$

in which u_m is a shear velocity equal to $\sqrt{g(SR)_m}$, $(SR)_m$ being the quantity obtained by solving equation 4.1.v. for SR with a known value of V_b. The mean depth, D_b, has replaced R_b'.

3) Both the dimensionless intensity of shear function for a single particle, Ψ_*, and the function for a representative particle, Ψ', have been replaced by Ψ_m, which is computed for each size range, d, from:-

$$\left.\begin{aligned} \Psi_m &= 0.4(S_s - 1)\frac{d}{(SR)_m} \quad \text{if } d > 2.5d_{35} \\ \text{or } \Psi_m &= (S_s - 1)\frac{d_{35}}{(SR)_m} \quad \text{if } d < 2.5d_{35} \end{aligned}\right] \quad . \quad . \quad 4.1.w.$$

4) The dimensionless intensity of bed load transport function, Φ_*, determined from fig. 4.1.k. using $\Psi_m = \Psi_*$, is divided by 2 "to make computed sediment discharges agree better with measured total sediment discharges". The bed load discharge per unit width for each size range is thus given by:-

$$i_B q_B = i_b \gamma_s g^{\frac{1}{2}} d^{3/2} (S_s - 1)\ (\Phi_*/2) \quad . \quad . \quad . \quad 4.1.x.$$

The hydraulic calculations necessary before computing bed load discharges are given in table 4.1.i. Values of V_b and D_b were obtained from table 4.1.h. and, assuming hydraulically rough flow, i.e. x = 1, equation 4.1.v. was solved for u_m. The thickness of the laminar sublayer, $\delta_m = 11.6\nu/u_m$. Since $k_s/\delta_m > 10$ at all stages ($k_s = d_{65}$), then according to fig. 4.1.h. the assumption of x = 1 was correct. The value of $(SR)_m$ was given by $U_m = \sqrt{g(SR)_m}$.

Bed load computations were made for the thirteen sizes ranges of table 4.1.h., using fig. 4.1.k. and equations 4.1.w. and 4.1.x. Discharge over the whole section for each size range, $i_B Q_B = w i_B q_B$, and the summated discharge, $Q_B = \Sigma\, i_B Q_B$, are given in table 4.1.j. The resulting bed load rating curve of the cableway section is plotted in fig. 4.1.m.

Table 4.1.j. Computation of bed load discharge by the modified Einstein method

H_e (ft A.O.D)	$i_B Q_B$ (lb/sec) for each size range of sediment d (ft)													$Q_B = \Sigma i_B Q_B$ (lb/sec)
	0.000862	0.00236	0.00472	0.00944	0.0188	0.0376	0.0634	0.0894	0.1267	0.1738	0.2354	0.3576	0.5060	
48	-	-	-	-	-	-	-	-	-	-	-	-	-	-
50	-	-	-	-	-	-	-	-	-	-	-	-	-	-
52	-	-	-	-	-	-	-	-	-	-	-	-	-	-
54	-	-	-	-	-	-	-	-	-	-	-	-	-	-
55	-	-	-	-	-	-	-	-	-	-	-	-	-	-
56	-	-	-	-	0.01	0.04	0.03	0.15	0.26	-	-	-	-	0.49
57	-	-	-	0.01	0.05	0.30	0.21	1.10	1.85	2.94	-	-	-	6.10
58	-	0.01	0.01	0.03	0.18	0.97	0.68	3.55	5.98	10.47	1.12	-	-	23.99
59	-	0.02	0.03	0.07	0.42	2.20	1.54	8.06	13.59	25.54	6.35	-	-	57.80
60	0.01	0.03	0.05	0.13	0.77	4.05	2.83	14.83	24.99	48.52	18.80	0.86	-	115.86
61	0.01	0.05	0.09	0.22	1.27	6.69	4.68	24.50	41.30	84.26	38.10	3.68	-	204.79

Table 4.1.i. Hydraulic calculations for modified Einstein method

H_e (ft)	V_b (ft/sec)	D_b (ft)	u_m (ft/sec)	δ_m (ftx10^3)	k_s/δ_m	$(RS)_m$ (ftx10^3)
48	0.58	3.76	0.042	4.045	48.14	0.055
50	2.21	5.71	0.151	1.125	174	0.708
52	4.09	7.71	0.265	0.641	306	2.18
54	6.05	9.71	0.378	0.449	436	4.44
55	7.03	10.71	0.432	0.393	498	5.79
56	8.02	11.71	0.487	0.349	561	7.36
57	9.02	12.71	0.540	0.317	618	9.04
58	10.01	13.71	0.593	0.287	682	10.90
59	11.01	14.71	0.646	0.203	745	12.95
60	12.01	15.71	0.698	0.243	807	15.10
61	12.00	16.71	0.748	0.227	863	17.38

The major attraction of the part of the modified Einstein procedure concerned with bed load discharge is that the difficulty of accurate measurement of energy-surface slope is avoided; the data required is limited to the size distribution and specific gravity of the bed material and the variation with stage of mean velocity, depth and width at a single cross-section. A further advantage is the simplicity of the necessary computations, only three equations and one graph being required. The important effect of the hiding of small particles is still included, although it can be seen from table 4.1.j. that a slightly wider range of particle sizes has been calculated to be in motion. However, the modified procedure has been developed for the computation of total sediment discharges in sand-bed rivers and the validity of modifications 3 and 4 for coarse gravel-bed rivers such as the River Tyne is uncertain. SCHROEDER and HEMBREE (1956) found good agreement between measured and computed sediment discharges in several sand-bed rivers in the United States of America.

Fig. 4.1.m.

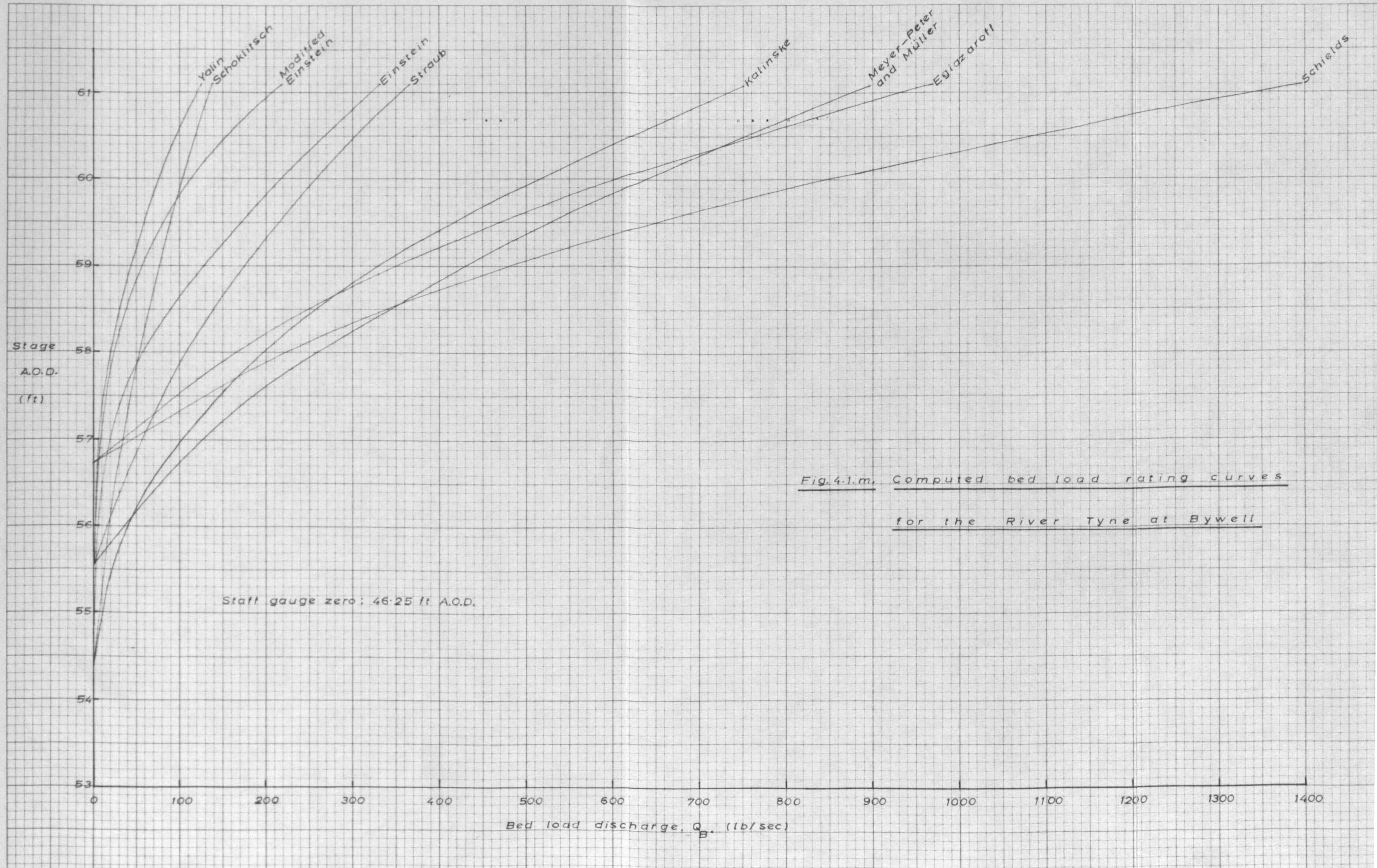


Fig. 4.1.m. Computed bed load rating curves for the River Tyne at Bywell

4.2. Regime Theory

Although the regime theory has been developed with the immediate purpose of facilitating the design of stable channels in alluvial material it is possible that the slope equation given by BLENCH and QURESHI (1964) (equation 2.2.e.) could be used to determine the bed load discharge of a river, such as the River Tyne, in which both water and sediment discharges fluctuate widely. An attempt was made, therefore, to calculate the bed load discharge at the Bywell cableway section at a stage of 61 ft A.O.D. using the regime theory.

Equation 2.2.e. can be rewritten:-

$$f'''(C) = \frac{\left(\frac{3.63g}{\sqrt{\nu}}\right) S W^{1/6} Q^{1/12}}{klm\ F_{bo}^{1/12}} \quad . \ . \ . \ . \ . \ . \ . \ 4.2.a.$$

At the cableway section, EF, Q = 44,950 cusec at the near bankfull stage, H_e = 61 ft A.O.D. From the graph of surface-energy slope against stage (fig. 3.4.d.) $S_e = S = 1.365 \times 10^{-3}$. The product klm was arbitrarily assumed equal to 1.3. From the cross-sectional profile (fig. 3.3.c.) W was found to be 220 ft. The particle size distribution curve of the bed material (fig. 3.5.e.) gave d_{50} = 0.129 ft, for which fig. 4.2.a. (BLENCH and QURESHI, 1964) gave F_{60} = 4.50. Substitution of the appropriate values in equation 4.2.a. yielded f'''(C) = 0.947, which indicates zero bed load movement.

It would appear, therefore, that the regime theory is not yet sufficiently developed for the unsteady conditions of gravel-paved rivers.

4.3. Other Bed Load Theories

In addition to the nine bed load theories applied to the River Tyne at Bywell many more were found in the available literature. Although they were not used for bed load computations at Bywell, mainly because various coefficients (K) and exponents (m) could not be evaluated, they are mentioned briefly in this section.

From the results of a series of tests on various materials GILBERT (1914) proposed several bed load formulae, one of which stated that for a particular material under given conditions:-

$$q_B = K_7(q - q_c)$$

CHANG (1937) conducted laboratory experiments on three different materials and concluded that:-

$$q_B = K_8 \frac{n}{\tau_c^{2}} \tau_o(\tau_o - \tau_c)$$

in which n is the Manning roughness coefficient and τ_c is given by:-

$$\tau_c = 0.0175 \left[(S_s - 1)d \; o^{1/3}\right]^{m_1}$$

where o is the ratio of the longest to the shortest diameter of a particle. Chang also mentioned formulae attributed to:-

Chyn: $q_B = K_9 V^{m_2} (\tau_o - \tau_c)$

Fabre: $q_B = K_{10} S_e^{3/2} (q - q_c)$

where $q_c = K_{11}(S_s - 1)^{7/9} d/S_e^{3/2}$

Casey: $q_B = K_{12} S_e^{m_3} (q - q_c)$

where $q_c = K_{13} d^{3/4}/S_e^{5/4}$

Nakayama: $q_B = (K_{14} + K_{15} S_e) q (V^2 - V_c^2)$

MACDOUGALL (1933) proposed:-

$$q_B = K_{16} S_e^{m_4} (q - q_c)$$

The UNITED STATES WATERWAYS EXPERIMENTAL STATION (1935) tested nine sands in laboratory flumes and concluded:-

$$q_B = \frac{1}{n}\left[\frac{(\tau_o - \tau_c)}{k_{17}\gamma_f}\right]^{m_5}$$

in which $\tau_c = 0.0132\sqrt{\gamma_f(S_s - 1)d/M}$, where M is the KRAMER (1934) uniformity modulus of a mixture.

O'BRIEN (1936) analysed material dredged from the Columbia River and suggested:-

$$q_B = K_{18}(V/R^{1/3})^{m_6}$$

SHULITS (1956) mentioned that Haywood developed the following:-

$$q_B = K_{19}d^{m_7}(S_e q^{2/3}/d^{1/3} - K_{20})$$

The regional report from Japan in the proceedings of the INTERNATIONAL ASSOCIATION FOR HYDRAULIC RESEARCH (1959) stated that Tsubaki analysed data from the experiments of GILBERT (1914) to obtain:-

$$q_B = 25\sqrt{(S_s - 1)\gamma_f d^3}\,\tau_*^{4/3}(\tau_* - 0.8\,\tau_{*c})\,(k_s/d_{65})^{-0.44}$$

PANTELUPULOS (1955, 1957, 1961) considered the importance of turbulence and the velocity distribution near the bed:-

$$q_B = 2/3\ \gamma_s p\ d_k \overline{U}_g$$

where d_k is a characteristic grain size.

BOGARDI (1965) attributes the following formula to Levi:-

$$q_B = K_{20}(V/\sqrt{gd})^3\ (d/R)^{\frac{1}{4}}d(V - V_c)$$

Bogardi also mentions the experiments of Pedroli with bed load movement over a smooth inerodible bed giving:-

$$q_B = 14.5\,\frac{\tau_o^{8/5}g^{3/5}d^{1/5}}{\gamma_s^{3/5}\,\nu^{1/5}} - 23.2\,\gamma_s\,\nu$$

ROTTNER (1959) applied the technique of dimensional analysis to the results of several experimenters to obtain:-

$$q_B = \gamma_s \left[g R^3 (S_s - 1) \right]^{\frac{1}{2}} \left[\frac{\left(K_{21} (d/R)^{2/3} + K_{22}\right) V}{(S_s - 1)^{\frac{1}{2}} R^{\frac{1}{2}}} - K_{23} (d/R)^{2/3} \right]^3$$

LAURSEN (1958) introduced the ratio of the shear velocity to the fall velocity of the bed particles in an attempt to predict the total sediment load of rivers.

Probably the most sophisticated, though as yet insufficiently developed, approach is due to BAGNOLD (1954, 1957). Although the concept of bed load movement postulated by Bagnold is one in which the whole bed is "live" and moves as a thick grain-fluid mixture the variables deduced are the same as the Φ and Ψ of EINSTEIN (1950).

4.4. Average Annual Bed Load Discharge

In order to calculate the average annual bed load discharge at Bywell according to each of the nine methods of section 4.1. it was first necessary to establish the distribution of river discharge (or stage) above 12,050 cusec (54 ft. A.O.D., 7.75 ft. above staff gauge zero) over as long a period of time as possible. The Lea Recorder instrument at Bywell provided a continuous record of water level for the ten year period 1956/66 but the time scale of the weekly charts was too small to enable an accurate determination of the duration of high flows to be made. Records were available, however, for the year 1965/66 from the Fisher and Porter instrument which recorded the stage above staff gauge zero on punched tape in a binary code form at intervals of 15 minutes. It was thus possible to plot the hydrograph of each flood greater than 12,050 cusec during 1965/66, determine the duration of flow above any given discharge and summate for the whole year to produce the cumulative flow frequency curve (curve 1 of fig. 4.4.a.). Records

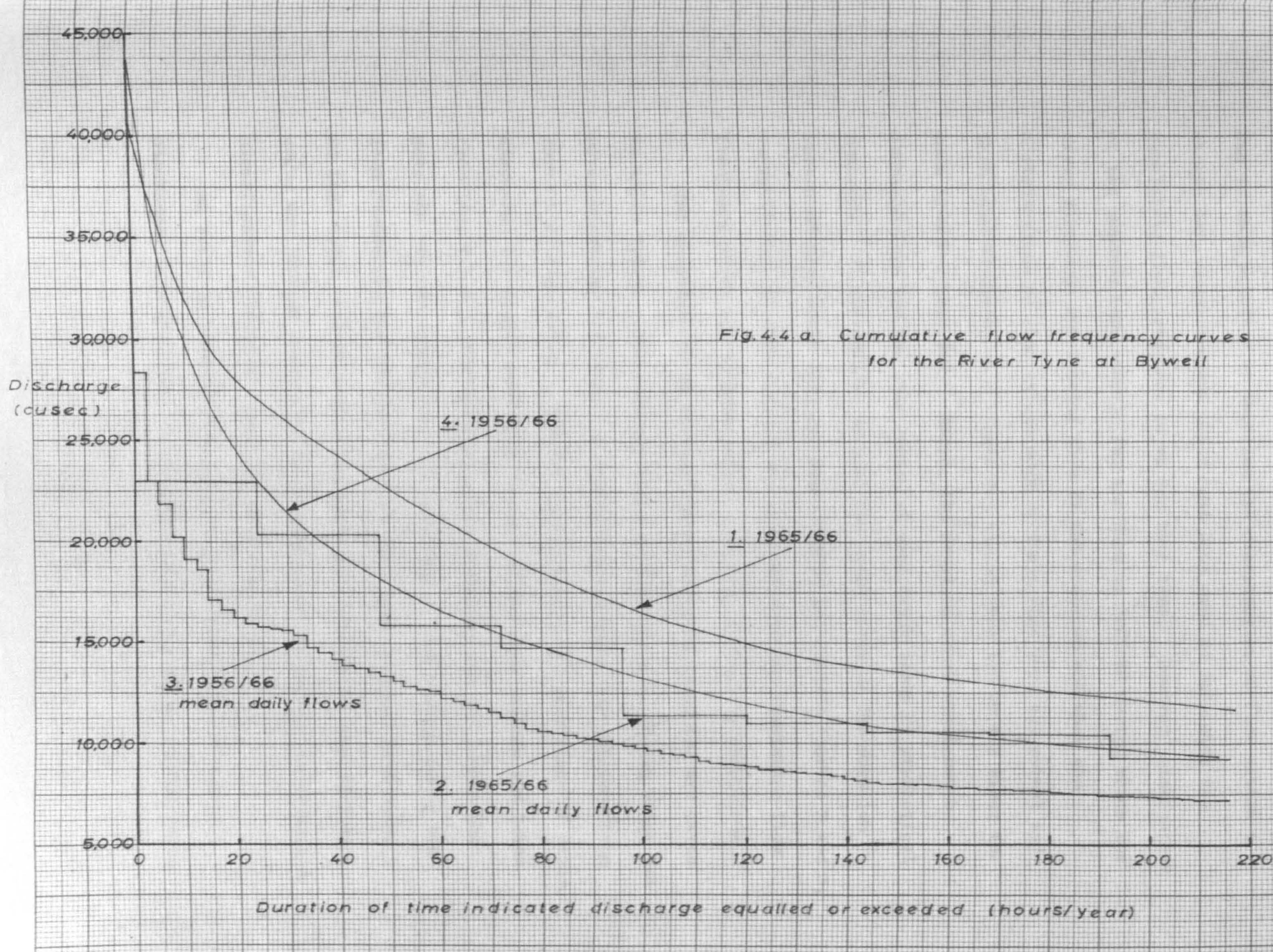


Fig. 4.4 a. Cumulative flow frequency curves
for the River Tyne at Bywell
Discharge
(cusec)
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
20
40
60
80
100
120
140
160
180
200
220
Duration of time indicated discharge equalled or exceeded (hours/year)
1. 1965/66
2. 1965/66
mean daily flows
3. 1956/66
mean daily flows
4. 1956/66

of mean daily flow were available for the ten year period 1965/66 and, had it been possible to describe the distribution of flow within a single day, these records could have been used to determine the cumulative flow frequency curve for that time. However, examination of the punched tape records for the days in 1965/66 during which the river discharge was greater than 12,050 cusec showed that the distributions of flow within each of these days were not normal and that peak discharge in a single day varied from 1.2 to 4.3 times the mean flow for that day. Hence, the required curve could not be obtained from the ten year record of mean daily flows by statistical methods. It was decided to plot the upper parts of the duration curves of mean daily flow for both 1965/66 and 1953/66 (curves 2 and 3, respectively, in fig. 4.4.a.) and then estimate the cumulative flow frequency curve for 1956/66 (curve 4 in fig. 4.4.a.) by comparison.

The calculations of average annual bed load discharge are given in table 4.4.a. The average length of time per year during which the river discharge (or stage) is within a given range was obtained from curve 4 of fig. 4.4.a. The bed load discharge in that range of river discharge (or stage) according to each of the nine methods was taken from fig. 4.1.m. Multiplication and summation for all ranges then gave the average annual bed load discharge. The corresponding bed load discharges for the year 1965/66 have been computed from curve 1 of fig. 4.4.a. and show that the estimated cumulative flow frequency curve for 1956/66 would have to be substantially altered to produce a significantly different total bed load discharge.

As would be expected the calculated average annual bed load discharges vary greatly; if the Meyer-Peter and Müller method is to

Kalinske		Einstein		Mod. Einstein	
ton/hr	ton	ton/hr	ton	ton/hr	ton
4.83	142.48				
18.51	379.45				
33.81	625.48	2.41	44.58		
51.52	695.52	5.95	80.32		
80.50	805.00	10.46	104.60	3.05	30.50
134.46	1176.91	22.54	191.59	7.56	64.26
204.41	1226.46	45.08	270.48	12.88	77.28
280.14	1260.63	77.28	348.66	25.76	115.92
365.47	1279.14	115.92	405.72	46.69	163.41
470.12	1527.89	167.44	544.18	75.67	245.92
590.87	1181.74	244.72	489.44	109.48	218.96
724.50	1267.87	326.83	571.95	156.17	273.29
909.65	1364.47	404.11	606.16	220.57	330.85
090.77	545.39	479.78	239.89	293.02	146.51
	13478.43		3897.57		1684.90
13,480		3,900		1,690	
22,210		6,260		2,530	

TABLE 4.4.a.

Egiazaroff		Yalin		Schoklitsch		Meyer-Peter and Müller	
ton/hr	ton	ton/hr	ton	ton/hr	ton	ton/hr	ton
				8.37	171.58		
				17.71	327.63	2.41	44.58
				27.37	369.49	35.42	478.17
		3.54	35.40	37.03	370.30	85.33	853.30
12.88	109.48	6.76	57.46	49.91	424.23	157.78	1341.13
101.43	605.58	13.68	82.08	64.40	386.40	251.16	1506.96
204.47	920.11	25.76	115.92	80.50	362.25	354.20	1593.90
326.83	1143.90	41.86	146.51	96.60	338.10	474.95	1662.32
473.34	1538.35	53.13	172.67	115.92	376.74	605.36	1967.42
652.05	1304.10	85.33	170.66	135.24	270.48	759.92	1519.84
843.47	1484.82	110.09	192.65	157.78	276.11	922.53	1614.42
1054.55	1581.82	141.68	212.52	180.32	270.48	1107.68	1661.52
1324.22	662.11	174.68	87.34	205.27	102.64	1308.12	654.06
	9353.27		1273.21		4046.43		14897.62
9,350		1,270		4,050		14,900	
15,060		2,040		6,710		25,090	

Table 4.4.a. Computation of annual average bed load discharge of the River Tyne at Bywell (1956/66)

Staff gauge zero : 46.25 ft A.O.D.

Range of discharge (cusec)	Range of stage (ft A.O.D.)	Duration (hours/year)	Schields ton/hr	Schields ton	Straub ton/hr	Straub ton
12,050-13,710	54.0-54.5	29.50				
13,710-15,480	54.5-55.0	20.50				
15,480-17,360	55.0-55.5	18.50				
17,360-19,340	55.5-56.0	13.50			7.24	97.74
19,340-21,430	56.0-56.5	10.00			37.03	370.30
21,430-23,600	56.5-57.0	8.50	17.71	150.53	70.84	602.14
23,600-25,930	57.0-57.5	6.00	96.60	579.60	109.48	656.88
25,930-28,310	57.5-58.0	4.50	275.31	1238.89	152.95	688.27
28,310-30,830	58.0-58.5	3.50	439.53	1538.35	199.64	698.74
30,830-33,440	58.5-59.0	3.25	656.88	2134.86	252.77	821.50
33,440-36,160	59.0-59.5	2.00	914.48	1828.96	316.56	631.12
36,160-38,910	59.5-60.0	1.75	1151.15	2014.51	381.57	667.74
38,910-41,940	60.0-60.5	1.50	1576.19	2364.28	452.41	678.61
41,940-44,930	60.5-61.0	0.50	2376.19	1188.10	533.71	266.86
				13031.08		6179.90
Average annual bed load discharge (ton)			13,030		6,180	
Bed load discharge 1965/6 (ton)			20,220		10,510	

be considered the most reliable (section 4.1.11) then a figure of 15,000 ton could be accepted tentatively. According to LANE and BORLAND (1951) it has been suggested by Maddock that for gravel rivers with suspended sediment concentrations of less than 7500 p.p.m then the average annual bed load discharge amounts to between 5% and 12% of the suspended sediment discharge. HALL (1964) has given 130,000 ton as the average annual suspended sediment discharge at Bywell; taking an average of 8½% gives an annual bed load discharge of about 11,000 ton. Since Hall used the duration curve of mean daily flows for the five slighly drier years 1956/61 it is likely that a more accurate estimate of the average annual suspended load would be somewhat greater than 130,000 ton. Comparison with other rivers is difficult due to the scarcity of field measurements of bed load discharge in gravel-bed rivers. However, KRESSER and LASZLOFFY (1964) have been able to state that the River Iech, with a catchment area to its confluence with the River Danube equal to that of the River Tyne at Bywell, has an annual bed load discharge of about 15,000 ton (11% of the suspended sediment discharge) of material up to 4 inches in diameter. The foregoing figures appear to corroborate the results of the Meyer-Peter and Müller method.

4.5. Conclusions

The main conclusions drawn from the application of a number of bed load theories to the River Tyne near Bywell can be discussed in relation to the nature of the bed material and the hydraulic characteristics of the reach.

1. Bed material

Most bed load formulae utilise the concept of a critical tractive force or critical bed shear stress for the initiation of motion of the sediment particles. This has caused a certain amount of confusion, mainly due to the difficulty of defining precisely these critical conditions. Some researchers have taken it to mean conditions at weak, medium, or general movement of the bed particles, while others have used the conditions prevailing at zero bed load discharge, obtained by extrapolation of experimental data. For a single particle size the equation or diagram (fig. 2.2.a) of Schields, who accepted the latter definition, can be used to obtain the critical stress pertaining to that size; for a sediment mixture such as that at Bywell, however, the concept of critical conditions of movement is rather vague, since a single size must then be selected to represent the range of sizes. At zero bed load discharge the MEYER-PETER and MÜLLER (1948) formula reduces to a form agreeing closely with the Schields equation for fully developed turbulent flow. It might seem, therefore, that the Meyer-Peter and Müller representative diameter, d_m, should be used in the Schields equation to obtain the critical tractive force of a sediment mixture. Account must be taken, however, of the natural accumulation of larger particles on the surface of the bed of most gravel rivers. Also, as described in section 3.5.6., particle shape can play an important part in the susceptibility to movement of the sediment. At Bywell the bed surface particles are predominantly disc-shaped (table 3.5.j.) and rest on the bed in such a way as to increase their resistance to movement. Hence, if it is required to estimate the stage or water discharge at which movement of a coarse-grained, disc-shaped bed material will commence a particle size slightly larger than the representative diameter, d_m, (for the bed material at Bywell, possibly d_{55} or d_{60} of the composite bulk sample) should be used in the Schields equation.

For the computation of discharge rates by some bed load formulae it was necessary to select a single size, such as the mediam diameter, d_{50}, or d_{40}, to represent the sediment mixture. It would seem that the Meyer-Peter and Müller representative diameter, d_m, is more appropriate in this respect since it has been found by experiment to describe sediment mixtures in movement and its calculation involves the whole size distribution curve.

Some bed load theories recognise the dubiousness of assigning a single particle size to a large range of sizes, and attempt to give discharge rates for individual size ranges of a sediment mixture. Comparison of tables 4.1.c. (Schoklitsch method) and 4.1.e. (Kalinske method) with tables 4.1.h. (Einstein method) and 4.1.j. (modified Einstein procedure) illustrates clearly that neglect of the mutual interference between particles of different sizes leads to extremely large predicted loads of fine material. The Einstein and modified Einstein methods include a correction factor for this important influence, but for their application to the River Tyne it was necessary to extend the validity of this factor to the large size and range of sizes prevailing at Bywell.

All nine bed load formulae applied to the Bywell reach were developed using empirical constants or functions obtained by observation of the behaviour of certain sediments, usually of uniform size, in certain flows along laboratory channels. The size and range of sizes of the bed material at Bywell were found to be greater than those of any of the material used in these experiments and in this respect extrapolation, sometimes considerable, was necessary.

2. Hydraulic characteristics

The rational theories consider bed load movement to be a steady, uniform flow process and use shear or stream power (product of shear and velocity) as a measure of the transporting ability of the flow. Except for the modified Einstein procedure all these theories require the accurate determination of energy-surface slope for the evaluation of these measures. As mentioned in section 3.6., flow conditions in most natural rivers, especially those with the "pool-bar" configuration of the River Tyne, are rarely uniform or steady. Cross-sectional shapes vary considerably (figs. 3.3.a. to 3.3.e.). Water-surface slopes are generally low and may be affected by wind or transverse currents. Measurement of slope over a long reach may overcome some difficulties, but energy-surface gradients thus obtained may not be representative of the single cross-section at which bed load discharge is to be estimated. Thus, measurements of bed shear using energy-surface slope are unlikely to be accurate, especially in gravel-bed rivers.

Accurate estimates of slope are of particular importance in the application of the MEYER-PETER and MÜLLER (1948) and EINSTEIN (1950) methods, which divide total bed shear into that due to particle friction (effective shear contributing to the movement of the sediment) and that due to bed configuration. With the Meyer-Peter and Müller equation effective bed shear is obtained by multiplying total bed shear by the reduction factor $(k_t/k_r)^{3/2}$; evaluation of k_t requires the use of the Strickler uniform flow equation. In the Einstein method the Keulegan equation is used to divide total hydraulic radius into R' and R''. (Einstein proposed that in certain cases the curve of EINSTEIN and BARBAROSSA (1951), shown in fig. 4.1.1., should be used to effect this division; data from the Bywell reach, however, indicated that the curve may not be applicable to all types of rivers).

It can be seen that the necessity for accurate measurement of energy-surface slope is the main stumbling block of nearly all bed load formulae. In the development of the modified Einstein procedure COLBY and HEMBREE (1955) overcame this difficulty by using the solution of the Keulegan equation for $(RS)_m$ for a known mean velocity as a measure of effective bed shear. More recently COLBY (1964) suggested that mean velocity and depth together are a sufficiently accurate, and certainly more convenient, measure of the transporting ability of flow in a sand-bed river.

From the above considerations on the nature of the bed material and hydraulic characteristics of the River Tyne at Bywell it can be seen that methods available at present for the estimation of bed load discharge are likely to give only approximate results. For computation of discharge rates of coarse-grained disc-shaped sediment there is clearly a need, not necessarily for a new formula, but possibly for the development of existing methods. In the same way as the Einstein procedure has been modified to give bed load discharges in sand-bed rivers, so it may be possible to extend the procedure, or more likely the Meyer-Peter and Müller equation, for application to gravel-bed rivers. This further development must deal with the three principal drawbacks of present methods:-

1. Accurate evaluation of energy-surface slope. It may be possible that where effective bed shear is to be used as a measure of the transporting ability of the flow, then current meter observations of velocity near to the bed could be used to obtain shear directly proportional to

the square of this velocity. Alternatively, mean velocity in the total depth may be measured and a relationship such as the Keulegan equation used to determine effective bed shear. In this way, a complete programme of current meter observations could yield the variation of shear both across the section and with river stage.

2. The influence of particle shape. More information is needed on the shear stresses required to initiate movement of the coarse disc-shaped particles observed to form a natural pavement on the surface of most gravel-bed rivers. The effect of particle shape on transport rates also requires attention.

3. Mutual interference between particles of different sizes. If the new method is to estimate bed load discharge rates of individual size fractions then more must be known about the "hiding" of smaller particles in the laminar sublayer and between larger particles, especially for the large ranges of sizes occurring in rivers such as the River Tyne.

The bed load rating curves for Bywell computed by nine of the methods available at present (sections 4.1.2. to 4.1.10) are shown in fig. 4.1.m. At a near bankfull stage of 61 ft A.O.D. calculated bed load discharges vary over a very wide range from 120 lb/sec by the Yalin method to 1350 lb/sec by the Schields method. Calculated values of critical stage at which bed load movement begins can be seen to lie between 54 ft and 57 ft A.O.D., or 7.75 ft and 10.75 ft above staff gauge zero. The discrepancies between the rating curves are most likely due to the various degrees of extrapolation of parameters such as particle size, water depth etc. and also to the different concepts of bed load movement and the pertinent assumptions embodied in each method.

If one of the methods in sections 4.1.2. to 4.1.10 is to be considered the most reliable then the MEYER-PETER and MÜLLER (1948) method has several points in its favour. It is based on a theoretical foundation, the Froude law of similarity, and utilises a wide range of experimental hydraulic and sediment data. This data includes the results of experiments on sediment mixtures. The formula is relatively simple to use, and has been found to describe the movement of bed load in certain European gravel-bed rivers. For these reasons the Meyer-Peter and Müller method is considered to yield the most reliable estimate of the bed load rating curve for the River Tyne at Bywell. However, considerations of the influence of particle shape (section 3.5.6.) indicates that the method may somewhat overestimate discharge rates of the predominantly disc-shaped sediment at Bywell. Also, due to the natural paving of coarse material on the bed surface and the resistance to entrainment of the disc-shaped particles, critical stage is probably slightly higher than that given by the rating curve, i.e. at about 56 ft A.O.D. (9.75 ft above staff gauge zero).

Using the Meyer-Peter and Müller rating curve with a flow frequency curve for the ten year period, 1956/66, the average annual bed load discharge of the River Tyne was calculated to be about 15,000 ton, approximately 10% of the average annual suspended sediment discharge. The results of other workers on similar rivers corroborate these figures. Experience gained in this research programme indicates that if any future investigation should require estimation of only the annual average bed load discharge of a particular river, then a figure of 10% of the average annual suspended sediment discharge (as suggested by LANE and BORLAND, 1951) will be as accurate as, and more easily obtained than, the result of application of any bed load formulae to the river.

PART II. Measurement of Bed Load Discharge

Section 5

5. Measurement of Bed Load Discharge in Rivers

5.1. Methods of Measuring Bed Load Discharge

Although the measurement of suspended sediment discharge in rivers can be relatively easily effected there exists at present no single apparatus or procedure which has been accepted as being completely adequate for the determination of bed load discharge over the wide range of sediment and hydraulic conditions occurring in nature. The majority of literature in this field has been published in the German, Russian and East European languages and, although some translations are available at various establishments in the United States of America, only Reports 2, 8 and 14 of the UNITED STATES INTER-AGENCY COMMITTEE ON WATER RESOURCES - SUBCOMMITTEE ON SEDIMENTATION and HUBBELL (1964) have reviewed existing techniques and apparatus at length in English. A large number of methods for measuring bed load discharge have been devised and are classified here as:- samplers or traps, river structures, tracer techniques, dune movement, and miscellaneous possible methods. They are discussed in sections 5.1.1. to 5.1.5. with particular reference to their applicability in the range of conditions prevailing in the River Tyne at Bywell.

5.1.1. Bed Load Samplers

The simplest and most direct method of measuring bed load discharge is to place some kind of trap or sampler on the river bed and weigh the quantity of sediment collected in a given time. Ideally, the bed load sampler should:-

1) not alter the distribution of water velocity and bed load discharge in the vicinity of the sampler.
2) collect the largest and smallest particles in motion.
3) sample a definite width of bed, trapping all the material which passes through that width.

4) be stable on the river bed

5) be correctly orientated both horizontally and vertically on the river bed

6) be designed such that the height of the opening is at least twice the maximum particle size, and its width about 150 times the average particle diameter (NOVAK, 1957).

7) have a leading edge which conforms to the shape of the river bed.

All the above criteria cannot be completely satisfied and each sampler should be calibrated to determine its sampling efficiency, i.e. the ratio, expressed as a percentage, of the weight of bed load collected during a given sampling time to the weight of bed load that would have passed through the sampler width had it not been there. Generally, the efficiency can most easily be determined by laboratory flume tests; however, even under the controlled conditions in a flume, two main difficulties arise. Firstly, due to the size of most bed load samplers, scale models must be used to avoid alteration of the flow conditions, thus producing the problems of similarity inherent in most sediment model studies. Secondly, the unsteady temporal distribution (JOHNSON, 1939) and the lateral distribution due to side effects of bed load movement in laboratory channels render determination of actual bed load discharge through the sampler width exceedingly difficult. According to HUBBELL (1964) calibrations of certain samplers have been carried out in flumes with fixed beds by Ehrenberger in 1932 and in flumes with movable beds by Einstein in 1937. The latter method more closely reflects conditions in natural streams. NOVAK (1957) conducted a comprehensive series of tests on several samplers using both methods.

The accuracy of the total bed load discharge in a river as determined by a particular sampler is affected by three factors:-

1) The estimate of the efficiency of the sampler. Apart from difficulties involved in the calibration of the sampler under the controlled conditions of a laboratory flume, the efficiency of the sampler may vary with any or all of the following:- water velocity, depth of flow, particle size, magnitude of bed load discharge, degree of filling, and bed configuration. Since all of these factors vary in natural streams, estimates of efficiency should therefore be considered to be highly variable and uncertain.

2) The spatial and temporal distribution of sediment transport in the river. Not only must a sufficient number of samples be taken across the section to observe the lateral distribution of bed load discharge but each sample must be taken over a length of time sufficient to account for the oscillatory or unsteady character of bed load movement. From measurements made with bed load samplers Ehrenberger (HUBBELL, 1964) concluded that variations in the River Danube and Inn could be characterised by constant periods of oscillation of 18 minutes and 7 minutes, respectively. KAROLYI (1957) also observed similar fluctuations on rivers in Hungary and postulated that, while these variations may have been to some extent due to the inaccuracy of the bed load samplers, the fluctuations were the result of pulsations of water discharge and velocity due to helicoidal currents, beds and the reflection of large water masses from various obstacles such as islands, banks etc. It can be seen, therefore, that maximum accuracy would be achieved by sampling at a few verticals over a long period of time; on rivers with rapidly changing stage this may prove to be impossible.

3) Methods of suspension of the sampler in the river. Because of the hydraulic resistance of most samplers an elaborate system of suspension and retention cables may be necessary. As the sampler is lowered into layers of progressively decreasing velocity it achieves an upstream motion due to its own weight and the elasticity of the cables. In this way the sampler, on reaching the bed, may scoop up bed material not in movement. MEYER-PETER (1937) has suggested that a form of rigid suspension would minimise this effect. In addition, when the sampler is on the bed, fluctuating drag forces may cause the sampler to oscillate and possibly scoop sediment from the bed.

In general all bed load samplers can be classified according to one or more of the following types:- box or basket, pan or tray, and pressure-difference.

Box or basket type samplers operate by retaining sediment that is deposited by a reduction of the flow velocity in a box which is open at the front and top, or in a basket which is screened by mesh on all sides except the front. According to Report 2 of the UNITED STATES INTER-AGENCY COMMITTEE ON WATER RESOURCES - SUBCOMMITTEE ON SEDIMENTATION one of the earliest devices was used by Davis in 1898 in the Nicaragua Canal and LABAYE (1948) mentions a trap used by Schaffernak in 1908. In the 1930s several more sophisticated basket type traps using rudders to improve stability were developed for use on European rivers by Mühlhofer, Ehrenberger, Nesper (fig. 5.1.a.) and the Swiss Federal Authority. All these samplers were of a similar design, the latter three having leading edges of loosely woven iron rings that conformed to the shape of the bed. Experiments by Ehrenberger and and Einstein showed that their sampling efficiencies varied from 30% to 80%, figures which were later verified by NOVAK (1957). The use of the Ehrenberger sampler to measure coarse bed load discharge on the

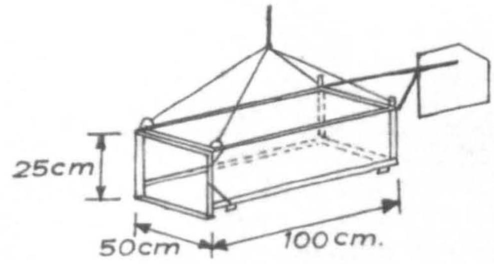


Fig.5.1.a. Nesper box type sampler.

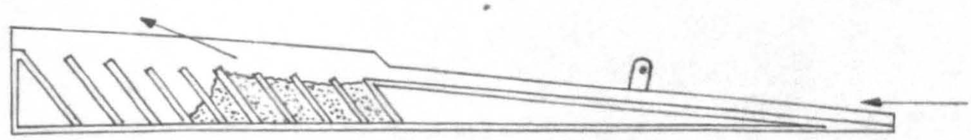


Fig.5.1.b. Polyakov pan type sampler.

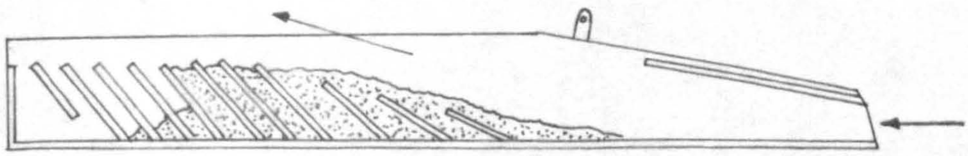


Fig.5.1.c. SRIH pressure difference type sampler.

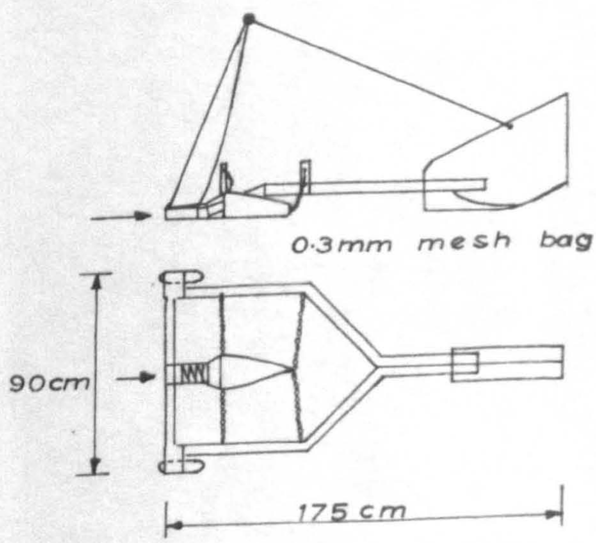


Fig.5.1.d. Arnhem pressure difference type sampler

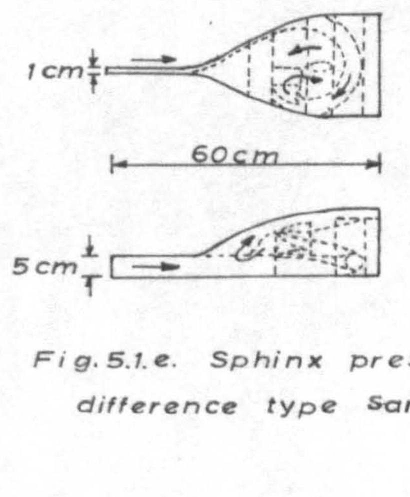


Fig.5.1.e. Sphinx pressure difference type Sampler.

River Danube has been referred to by KRESSER and LASZLOFFY (1964) and TSCHOCHNER (1964), while BOGARDI (1951) described a wire-mesh basket type sampler used on the River Tisza in Hungary. Because of their large capacity these types of samplers are more suitable for the measurement of the discharge of coarse bed load, but they cause considerable disturbance of flow and are subject to selectivity of sampled particle size.

Pan or tray type samplers, which have been used principally in Russia, are usually wedge-shaped in longitudinal section with the pointed end of the wedge facing upstream. They operate by retaining the sediment particles which roll or slide up an entrance ramp into a transverse slot or series of slots. HUBBELL (1964) described the samplers designed by Losiebsky and Polyakov (fig. 5.1.b.) which have been shown by Shamov to give efficiencies as low as 38% and 46%, respectively, mainly due to the accumulation of sediment in front of the adverse slope of the entrance ramp. This type of sampler is only suitable for low velocities in smooth, sand-bed rivers.

In the operation of ordinary basket and pan type samplers, the resistance to flow of the sampler causes an undesirable disturbance of the water and sediment regime at the sampler entrance. The pressure-difference type sampler has been designed so that entrance velocity and stream velocity are maintained equal by constructing the sampler walls so that they diverge towards the rear, thus creating a pressure drop at the sampler exit. Report 10 of the UNITED STATES INTER-AGENCY COMMITTEE ON WATER RESOURCES - SUBCOMMITTEE ON SEDIMENTATION described two traps based on the pan or tray type designed by the Scientific Research Institute for Hydrotehnics (fig. 5.1.c.) in Russia and by the United States Corps of Engineers, Little Rock. The Arnhem bed load sampler (fig. 5.1.d.) described by SHAANK (1937), has been developed to collect material in the size range 0.15 to 5 mm in a fine mesh bag flexible attached to a rigid rectangular entrance. According to MEYER-PETER (1937) the efficiency of the sampler is about 70%. Its use on the Dutch part of the River Rhine has been refered to by ELZERMAN and FRIJLINK (1951) and TOPS, WEMELSFELDER and VOLKER (1959) and on the River Niger by NEDECO (1959). Another pressure-difference type trap, called the Sphinx sampler (fig. 5.1.e.), has been devloped in Holland by VINCKERS, BIJKER and SCHIJF (1953) for grain sizes smaller than 0.4 mm. In this sampler flow enters through a rectangular nozzle that gradually becomes circular, passes through a series of settling chambers and then out a wide exit at the rear. BOGARDI (1951) described how Karolyi modified the wire-mesh sampler used in Hungary into a pressure-difference type sampler with a horizontal, curved dividing wall about midway between top and bottom of the rear of the sampler. NOVAK(1957) found from laboratory tests that this sampler, which was used to measure coarse sand and gravel discharges, has an efficienty of only 45% After an exhaustive series of laboratory tests on scale models of several bed load samplers at the Vyzkumny Ustav Vodohospodarsky (V.U.V), Prague, NOVAK (1957, 1959) developed a new pressure-difference type trap, known as the V.U.V. sampler, for collecting bed load in the size range 1 to 100 mm and in water velocities of up to 3 metre/sec. The efficiency of this sampler, which is described in greater detail in Section 5.2, was determined by model tests to be 70%. HUBBELL (1964) mentions two traps of a similar type designed by Uppal and Gupta for use in Indian Canals but no information concerning their efficiencies is available.

5.1.2. River Structures

According to Report 2 of the UNITED STATES INTER-AGENCY COMMITTEE ON WATER RESOURCES - SUBCOMMITTEE ON SEDIMENTATION Mühlhoffer in 1933 trapped bed load in a series of boxes placed in the river bed with their tops flush with the bed surface in order to determine the particle size distribution of the material moving along the river bed. Other investigators have extended this idea to the determination of bed load discharge by measuring the time required for a slot, pit or trench of known volume to fill. The most elaborate installation, which has been described by JOHNSON and DOBSON (1940), was constructed by the United States Soil Conservation Service on the Enoree River, South Carolina. The entire 100 feet width of the river was concreted and divided by piers into 14 channels, each having a grated slot which could be opened and closed. Bed load which dropped into the slots was pumped out through a pipe beneath the concrete floor to a hopper on the bank for weighing. Measurements of suspended load were also made and in this way a continuous record of total sediment discharge could be made (EINSTEIN, ANDERSON and JOHNSON, 1940). EINSTEIN (1944) described a semi-portable slot type structure used by the United States Soil Conservation Service in Mountain Creek, South Carolina, operating on the same principle of withdrawal of the trapped sediment and measurement by weighing. HUBBELL (1964) proposed a similar pit sampler but its use is probably restricted to sand-bed streams of low velocity.

It is possible that the devices constructed for the exclusion of sediment from irrigation, power and municipal canals could be considered for the measurement of bed load discharge. ROBINSON (1960) improved upon the vortex tube sand trap devised by Parshall which was observed to be very effective in the removal of coarse material up to the size of cobbles. The instrument consisted of a tube with a slot along the top placed in the stream bed at an angle of 45° to the direction of flow; a vortex motion was thus induced inside the tube, washing the trapped sediment out at the downstream end. Efficiencies of between 80% and 100% are possible with these slot or pit samples.

Another type of structure, working on a different principle, is the turbulence flume constructed on the Middle Loup River, near Dunning, Nebraska (SERR, 1951). A specially designed series of baffles were installed in the river to induce turbulence sufficient to bring the total sediment load of the sand-bed river into suspension. Conventional methods were used to measure the suspended material and these observations were compared with suspended sediment measurements at an upstream section to give the bed load discharge (SCHROEDER and HEMBREE, 1956).

The major disadvantage of all such structures is the excessive cost of both construction and operation.

5.1.3. Tracer Techniques

During the last decade several investigations have been carried out along the coast and in estuaries using tracers to determine the direction of movement of sand and silt in marine and tidal currents. The various methods by which sediment particles can be labelled have been described by KIDSON and CARR (1962). The HYDRAULICS RESEARCH STATION (1961), ZENKOVITCH (1960) and REID and JOLLIFFE (1961), using fluorescent tracers such as rhodamine , primuline, anthracene and limogene, have obtained useful qualitative results. At present, however, no direct measurements of bed load discharge in rivers have been made with fluorescent tracers.

Similar coastal and estuarine investigations have been conducted with radioactive tracers in Japan (INOSE and SHIRAISHI, 1956), France (GERMAIN, FOREST and JAFFRY, 1957), Great Britain (UNITED KINGDOM ATOMIC ENERGY AUTHORITY, 1957) and Holland (ARLMAN, SVASEK and VERKERK, 1960). In France estimates of total bed load discharges occurring during several floods on the River Rhone near Lyons were made by RAMETTE and HEUZAL (1962). At the Hydraulics Research Station, Wallingford, England, research has been carried out (DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH, 1960 to 1965) on the use of radioactive tracers for the determination of sand transport rates in a 5 ft wide, 350 ft long laboratory flume. Two techniques, similar to those used in dilution techniques of water discharge measurement, have been developed. In the space-integration method (CRICKMORE and LEAN, 1962a) the bed load discharge was deduced from observations of the activity distribution downstream of the point of injection of the irradiated sand particles. The time-integration method (CRICKMORE and LEAN, 1962b) involves the determination of the time variation of activity at a fixed point downstream. These methods have been applied to the sand-bed River Idle, in Nottinghamshire, but the accuracy of the results cannot be evaluated. Similar experiments have been carried out in the United States of America by HUBBELL and SAYRE (1964) who, assuming a Lagrangian probability concept of sediment transport, derived distribution functions of concentration and discharge similar to those of EINSTEIN (1950).

Results of the British and American research indicate that tracer techniques are feasible in laboratory flumes and sand-bed rivers. Their major restrictions are that measurement must be made over an extended period of time during which conditions must remain steady, and that a skilled team of operators is required.

5.1.4. Observation of Dune Movement

The possibility of determining bed load discharge from the dimensions and speed of movement of dunes and ripples in channels was realised over seventy years ago (HUBBELL, 1964). However, it is only recently that technical advances in portable electronic equipment have enabled rapid and reliable determinations of bed configuration over large areas of the bed of a channel to be made. SIMONS, RICHARDSON and NORDIN (1965) proposed a bed load equation based on the mean forward velocity and mean height of ripples and dunes. Good agreement with measured sediment discharges was obtained when this equation was applied to the observations from a sonic depth recorder in laboratory flume experiments with four uniform sands from 0.19 to 0.93 mm. Although dune movement is a promising line of research, the proposed equation requires several assumptions concerning uniformity of sediment and flow conditions and is only applicable to dune and ripple forming sediments.

5.1.5. Other Possible Methods

Several other possible methods which have been investigated are mentioned by HUBBELL (1964). Kennedy and Mundorf independently proposed bed load samplers for collecting small particles that are composed wholly or partly of magnetic minerals. However, since the specific gravity of such particles can be as high as 5.2 they are unlikely to be representative of the behaviour of the majority of particles in a river bed.

Taniguchi developed a method for computing total sediment load using a tiltmeter, consisting of a Zöllner pendulum suspension and recording equipment, which measures variations in ground tilt due to the passage of different weights of water and sediment through a channel. Several important assumptions are necessary and it seems likely that the tiltmeter method could only be applied to rivers with large variations in sediment discharge and which flow through unconsolidated materials.

A device using ultrasonic waves has been developed by Smoltczyk for use on beds of fine sand. It consists of an open-ended rectangular tube with a transducer and reflector housed in opposite sides of the tube; the transducer transmits and receives the reflected high-frequency sound waves enabling the amount of acoustic energy absorbed by the sediment water complex to be calculated. Several important, unverified assumptions are necessary to relate the absorbed energy to bed load discharge.

Some attempts have been made to photograph bed load movement but a number of technical and practical difficulties were encountered. It appears that the use of photography would be restricted to streams of low suspended sediment concentrations and transporting bed load particles greater than ¼ inch.

Several attempts have been made to detect sediment movement by recording the sound emitted by coarse bed load particles. This promising possibility is discussed in greater detail in sections 7 and 8.

5.2. Use of V.U.V. Bed Load Sampler on the River Tyne at Bywell

It can be seen from the literature review of section 5.1. that the portable trap and permanent structure are the only techniques of bed load discharge measurement applicable and sufficiently developed for the flow and sediment conditions of the River Tyne at Bywell. However, the construction and operation of a permanent structure was impossible due to a lack of finance and personnel. The development of an entirely new sampler was also precluded, for two main reasons. Adequate, precise knowledge of the size, distribution and rates of movement of bed load at Bywell would first have been necessary for the design and testing of a new sampler, and existing laboratory facilities were inadequate for the determination of the stability, performance and efficiency of a new sampler. Correspondence with the Hydraulics

Research Station, Wallingford, England and the Federal Inter-Agency Sedimentation Project, Minneapolis, U.S.A. indicated that no research on samplers of coarse bed load discharge was being conducted in Great Britain or the United States of America. It was decided, therefore, to try to use the V.U.V. sampler designed by NOVAK (1957, 1959) at the Hydraulics Research Institute, Prague, for the following reasons:-

1) The V.U.V. trap was designed to sample particles of up to 4 inches diameter in flow velocities of up to 12 ft/sec, conditions similar to those expected at Bywell. Examination of the Northumberland and Tyneside River Board gauging records showed a maximum recorded velocity just below the surface of 13 ft/sec and it was considered from inspection of gravel shoals in the Bywell reach that the quantity of material in motion greater than 4 inches diameter would not be significantly large.

2) The trap had been subjected to an exhaustive series of laboratory and field tests.

3) The trap has a relatively high sampling efficiency of 70%

5.2.1. The V.U.V. Bed Load Sampler

The V.U.V. bed load sampler, which was developed by NOVAK (1957) from the Karolyi pressure-difference sampler, is 2.8 metres long, has a 50 cm x 20 cm opening and can collect a sample of about 30 Kg (fig. 5.2.a., in pocket). The principal features of the instrument are the horizontal curved dividing wall which separates the lower sediment-retaining part of the sampler from the upper direct flow-through part, and the rear door which was designed to improve the stability of the sampler while covering the 5.5 cm high slit at the rear of the lower part. During lowering and raising, the tension of the suspension cables maintains the rear door in a closed position, the water flowing through the sampler and out the wire mesh in the top rear part of the sampler. When the trap is located on the river bed the suspension lines slacken and the flow of water causes the rear door to open until the top of the door rests on the rudder. The shape of the sampler and the position of the wire mesh were designed such that with the rear door open, sufficient pressure drop is created at the exit to overcome the flow resistance of the sampler. Experiments showed that the hydraulic efficiency (the ratio, expressed as a percentage, of the water discharge through the sampler to the product of the undisturbed mean velocity of flow at the entrance and the area of the entrance) of the sampler is 100%.

Novak carried out comprehensive studies on the V.U.V. sampler and four other types of sampler in laboratory flumes of 60, 100 and 250 cm width and with sampler models of scale ratios of 1:1, 1:2, 1:4 and 1:8. Determinations of sampler efficiencies were made with the samplers placed on fixed beds (Ehrenberger method) and on movable beds (Einstein method), with particle size mixtures and individual size fractions ranging from 0.1 to 100 mm, and with mean water velocities from 0.6 to 2.2 metres/sec. The model scales, particle size ranges, and velocity ranges were selected in each test so as to obtain approximate similarity with the flume size. Sampling efficiencies of the V.U.V. sampler were calculated from the ratio, expressed as a percentage, of the weight of sediment retained by the sampler to the weight of sediment which would have passed through the sampler width, for fixed beds (η_p) and for movable beds (η_s) as shown in table 5.2.a.

From the results of laboratory tests Novak also concluded that the efficiency of the V.U.V. sampler increases only slightly, if at all, with particle size and water velocity. It was found that the sampler retained particle size distributions that agreed, in general, with the size distributions of the actual bed load, and that the efficiency is independent of the degree of filling so long as it does not exceed 30% of the sampler capacity.

Table 5.2.a. Sampling efficiencies of model V.U.V. samplers

Scale ratio (model to prototype)	Ratio of flume width to sampler width in indicated flume			Sampling efficiencies in indicated flume						Recommended overall efficiency
				250 cm		100 cm		60 cm		
	250 cm	100 cm	60 cm	η_p	η_s	η_p	η_s	η_p	η_s	
1:1	5	-	-	82	52	-	-	-	-	70
1:2	10	4	-	70	64	75	40	-	-	
1:4	-	8	4.8	-	-	93	61	72	58	
1:8	-	16	-	-	-	83	55	-	-	

Details of the V.U.V. bed load trap were obtained from Dr. Novak and a prototype sampler constructed from 16 s.w.g. brass. Particular care was taken to ensure the correct dimensions, especially those of the horizontal curved partition. A second hinged door was provided at the rear of the lower part to enable the sampler to be emptied of collected material. For ease of transport the sampler was constructed such that it could be dismantled into box and rudder. The weights of these two parts were 86 lb and 60 lb, respectively, giving a total weight of 146 lb. (A test of the suspension cable of the Bywell cableway showed that it had a tensional breaking load of 0.77 ton, i.e. ten times the weight of the sampler). Five $\frac{1}{8}$ in diameter cables were attached to the sampler by small brass shackles and their lengths adjusted so that the sampler could be suspended from a single point at an angle of about 20° to the horizontal with the entrance uppermost. This ensured that, on immersion of the sampler, the rudder would enter the water first and correctly orientate the sampler to face upstream.

5.2.2. Sampling Attempts

It was decided initially to observe the stability of the sampler when suspended by a single cable from the Bywell cableway.

The first tests were made at a low stage of about 2 ft above staff gauge zero (600 cusec) and it was found that, despite very slow flow velocities, there was considerable resistance during raising and lowering, and that correct orientation of the sampler on the bed could not be guaranteed. Two further trials were made at stages of 7.8 ft (12,000 cusec) and 6.0 ft (7,100 cusec). Several attempts were made on each occasion to place the sampler on the bed at a distance of 50 ft from the left bank where surface velocities would be about 8 ft/sec. When the trap was let down on to the water surface the rudder entered first and, as expected, the trap turned to point upstream. However, the pressure of the flow of water on the underneath of the sampler caused it to move downstream, but remain on the water surface, as the suspension cable was paid out. At about 15 ft downstream of the cableway the trap entered the water causing a sudden increase of load on the suspension cable. As the sampler sank to the bed it became extremely unstable and correct orientation on the bed could not be ensured. (It should be recorded that no material was collected by the sampler on the river bed at the 7.8 ft river stage).

It was obvious from these preliminary trials that either a rigid suspension or a system of suspension and retention cables would be necessary to ensure both the accuracy and safety of the sampling procedure.

5.2.3. Possible Sampling Methods

The use of a rigid suspension of the bed load sampler would obviate the need for an elaborate system of cables and also improve the accuracy of the instrument (section 5.1.1.). Some preliminary calculations were made for a twin-hulled boat moored to the cableway as shown in fig. 5.2.b.

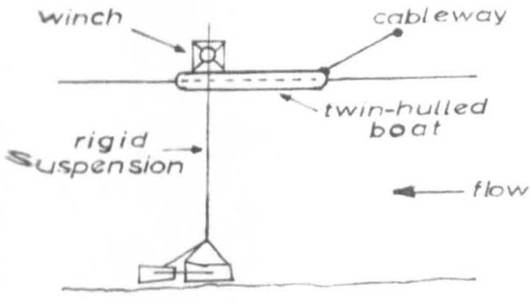


Fig.5.2.b.

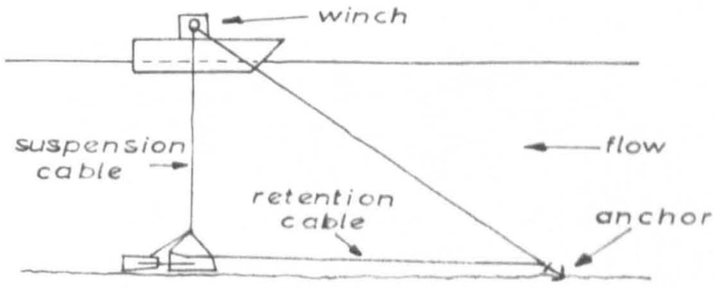


Fig.5, 2. c.

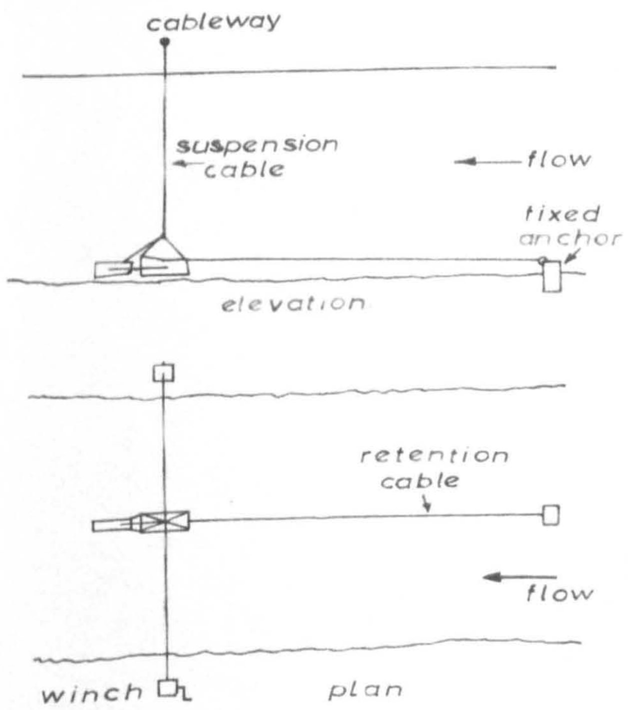


Fig. 5. 2. d.

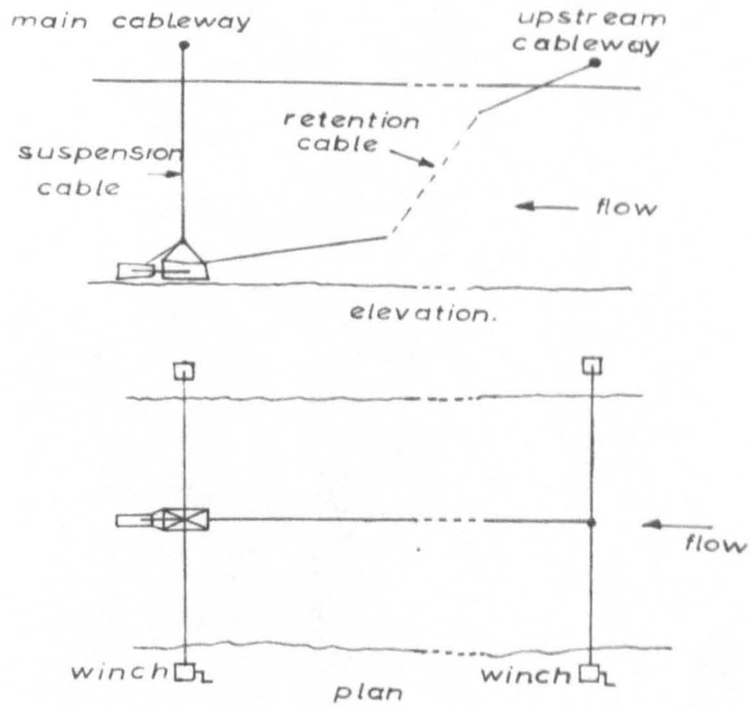


Fig. 5.2. e.

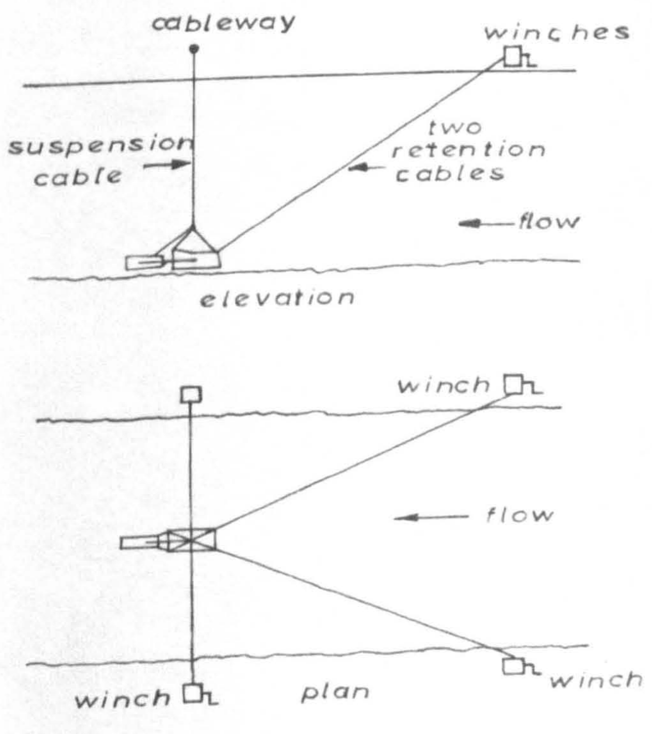


Fig.5.2.f.

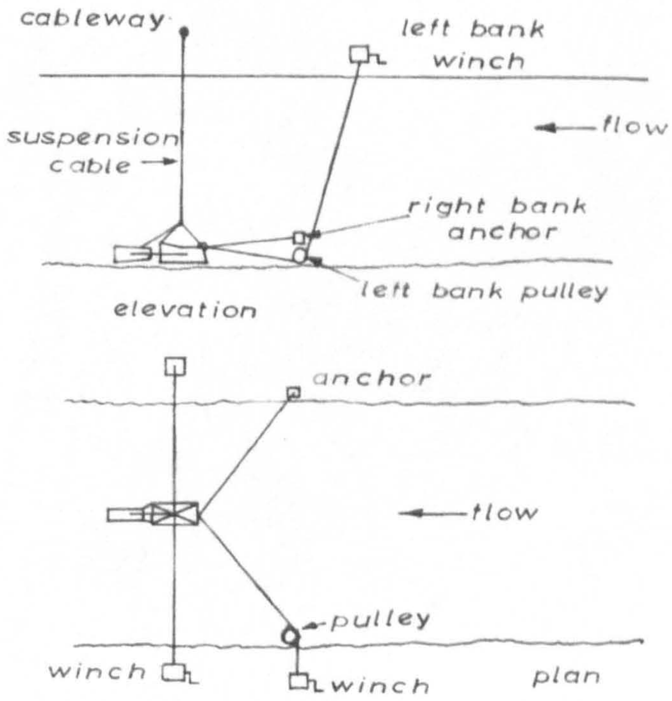


Fig. 5.2. g.

Suspension arrangements for bed load sampler

However, on tentative enquiries to the Northumbrian River Authority the scheme was rejected as the cableway was not designed to take lateral loading.

Fig. 5.2.c. shows the method of suspension used by Novak for measurement on the River Danube in Czechoslovakia. There were two main objections to using this type of flexible suspension at Bywell:-

1) The material of the bed was too large to ensure that an anchor would remain fixed when dropped on to the bed.

2) It would be impossible to manoeuvre sufficiently in high velocities.

An alternative method, similar to that used on the Danube, is shown in fig. 5.2.d. where the retention cable is anchored permanently to the bed some distance upstream. The major disadvantage is that there would be only one position of the sampler at which the retention cable would be parallel to the direction of flow. With a 1000 ft long retention cable the sampler would be inclined at an angle of 9° to the direction of flow when placed near the bank.

The retention cable could be attached to a travelling pulley on a second cableway upstream of the gauging station as shown in fig. 5.2.e., enabling sampling to be carried out at any point across the section. The distance between the cableways would have to be considerable since the retention cable would be inclined to lift the leading edge of the sampler off the bed.

Another possibility would be the use of two retention cables attached to winches at the top of each bank (fig. 5.2.f.). Using the winches it would be possible to maintain the sampler vertically below the cableway.

The system shown in fig. 5.2.g. could also be used. With this system the retention cable would be anchored near low water level on the right bank, passed through a ring on the front of the sampler, round a capstan or pulley near low water level on the left bank and wound on to a winch at the top of the bank.

The Northumbrian River Authority were approached with the possibility of using a suspension system similar to either fig. 5.2.f. or fig. 5.2.g., although it was realised that these methods would require at least three operators and could be costly. However, permission could not be granted and attempts to make direct measurements of bed load discharge were therefore abandoned.

5.3. Conclusions

The most accurate method of measuring bed load discharge is the slot or pit type structure which traps all of the sediment moving along the whole width of the river bed. By means of withdrawal by pumping and then weighing of the collected material a continuous record of bed load movement can be obtained. However, the cost of construction and operation of such structures will usually be prohibitive, especially in larger rivers.

Since all portable bed load traps affect the ambient flow and sediment regime by their presence on the bed, each trap must be calibrated to ascertain its sampling efficiency under given conditions. This can only be effected in laboratory flumes with sampler models, thereby introducing similarity problems and detracting from the reliability of the determined efficiencies. In addition, the efficiency of any sampler is likely to vary with water velocity, particle size, bed load discharge, degree of filling and bed configuration, and since these factors can vary considerably in a single river, the true efficiency of the sampler must be rather uncertain. Due to the oscillatory, or unsteady, nature of bed load movement a single short-term measurement is not necessarily representative of the mean bed load discharge; to determine the total bed load discharge in a river each point in the section should be measured for a considerable time.

For the measurement of the discharge rates of fine sand and sand the Sphinx and Arnhem samplers are probably the most suitable although precise information concerning their efficiencies is not available. The V.U.V. pressure-difference type sampler, which was selected for use on the River Tyne at Bywell, is the most advanced sampler of coarse bed load for the following reasons:-

1) It has been subjected to a comprehensive series of laboratory and field tests to determine its stability, performance and efficiency.

2) It has a relatively high efficiency of 70%, which varies little, if at all, with particle size and water velocity.

3) It samples sediment sizes from 1 to 100 mm and generally gives a good representation of the size distribution of the bed load.

4) It can be used in water velocities up to 4 metre/sec.

The techniques involving readioactive tracers and dune tracking by sonic recorders are promising, but are not yet sufficiently developed. Several other methods such as a magnetic sampler, tiltmeter, ultrasonic device and underwater photography have been used with limited success.

Preliminary experiments with the V.U.V. bed load sampler suspended by a single cable at the Bywell cableway gauging station indicated that a system of suspension and retention cables would be necessary to ensure correct orientation and stability, and hence accuracy and safety. Several suggested alternative arrangements were rejected by the Northumbrian River Authority and all attempts to measure bed load discharge at Bywell had to be abandoned. These investigations have shown that the measurement of bed load discharge in rivers under any conditions is, at present, likely to be extremely costly and that the use of a bed load trap at a cableway gauging station would require at least three winches and a team of three operators.

Section 6

6. Development of Apparatus for Laboratory Investigation of the Acoustic Detection of Bed Load Movement

6.1. Aims of Laboratory Experiments

After the attempts to make direct measurements of bed load discharge using the V.U.V. sampler at Bywell had been abandoned, it was decided to consider further a possible means of detecting coarse bed load movement which hitherto had been investigated by only a few research workers. The method, mentioned briefly in section 5.1.5., involves the recording of some measure of the intensity of sound emitted by moving bed load particles either during collison with an object placed on the river bed or during inter-particle collision. Since most of the other methods of measuring bed load discharge described in section 5.1. have been found to be either unsuitable or impractical in the sediment and flow conditions of the River Tyne at Bywell, it was decided that the method of acoustic detection warranted further investigation. Plans were therefore made for the design and construction of a suitable instrument which could be used at the cableway gauging station near Bywell. The development and use of this instrument is described in section 8; a survey of existing literature on the acoustic detection of sediment movement is also included in section 8 since all previous work in this field has been concerned solely with river observations.

Soon after the decision to develop an acoustic bed load detector the Hydraulics Laboratory of the Civil Engineering Department of the University of Newcastle upon Tyne received from [redacted] some equipment intended for use as an experimental sediment channel. It was then decided that, at the same time as field investigations were being carried out, a series of controlled laboratory experiments would be conducted to evaluate the potential of the method as a means of not only detection but accurate measurement of bed load discharge. The aim of the laboratory experiments was, therefore, to investigate the relationship between bed load discharge in the laboratory channel and the intensity and frequency of the sound emitted by the moving bed material.

The sediment channel, in the condition received from the manufacturers, could not be used immediately for the planned experiments. Considerable modifications had first to be carried out, and the equipment for the measurement of sediment feed rate, sediment discharge, water discharge, etc. had to be developed. The design, construction and testing of the necessary apparatus proved to be a considerable task, occupying much of the available time, and is described in this section; the experiments, their analysis and the resulting conclusions are described in section 7.

6.2. Description of the Sediment Channel

The 40 ft. long, 18 in. wide, tilting, glass-sided sediment channel with a self-contained circulatory water system was located in the basement Hydraulics Laboratory in the Stephenson Building of the University of Newcastle upon Tyne.

6.2.1. General Description

The general arrangement of the sediment channel and the associated equipment is shown in figs. 6.2.a. and 6.2.b.

The water supply for the system was stored in a 10 ft x 6 ft x 3 ft deep galvanised steel sump tank; it was found necessary to raise the sides of the tank near the channel outlet and to extend the channel outlet chute to avoid splashing. Approximately 60 lb of sodium nitrite were added to the 1250 gallons of water in the system to give a 0.5% by weight solution for protection against corrosion.

Water could be extracted from the tank by a 10 in axial flow pump, running at 1470 r.p.m. and powered by a 20 h.p., 3 phase, a.c. motor. (The 12 h.p. motor supplied by the manufacturers was

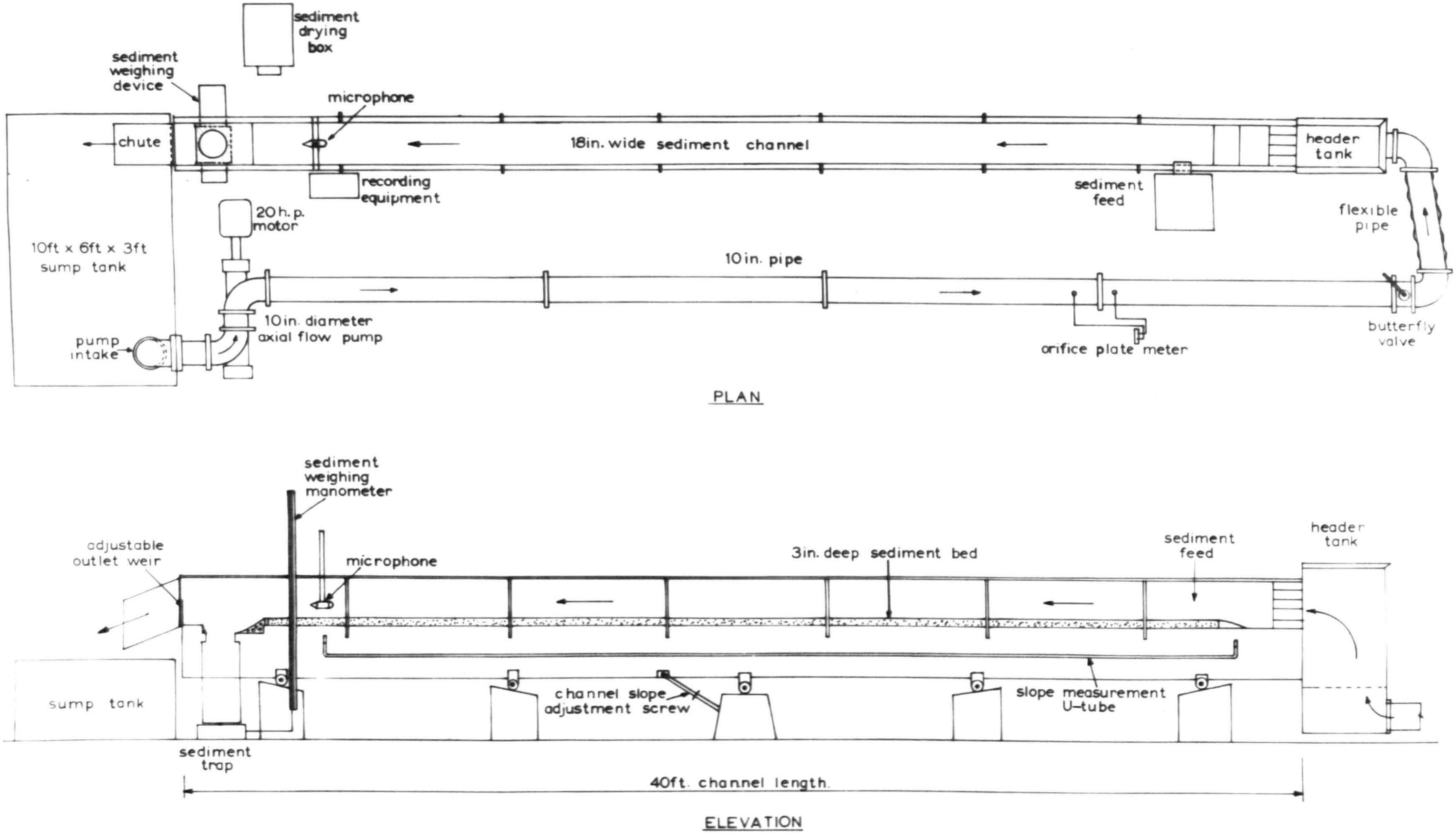


Fig. 6.2.a. Laboratory sediment channel.

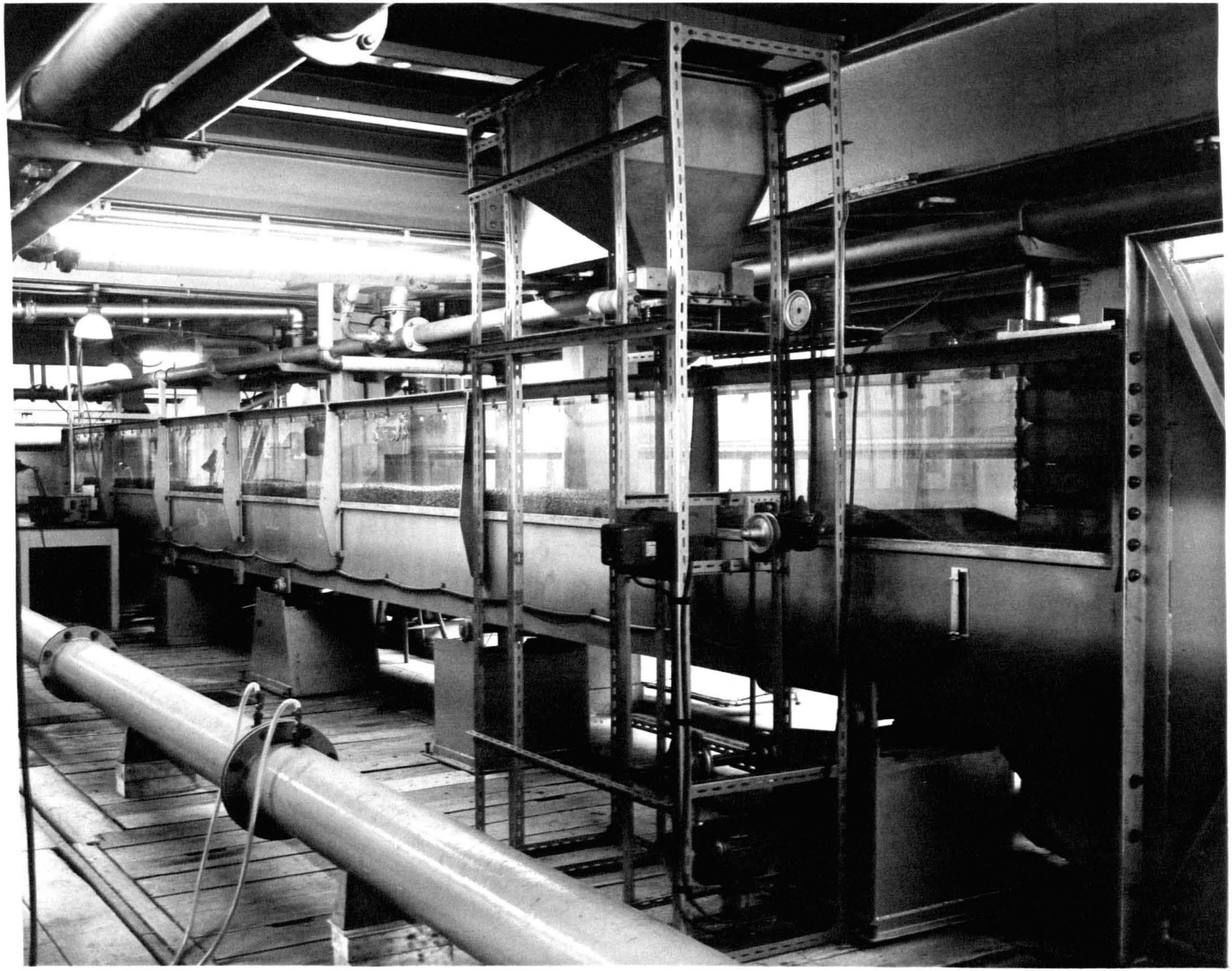


Fig. 6.2.b. General view of laboratory channel from upstream end.

found to be overloaded when pumping low water discharges, and took 5 months to be replaced). It was also found necessary to replace the length of flexible pipe connecting the suction side of the pump to the sump tank by a specially fabricated length of mild steel pipe. To avoid the entrainment of air at the pump intake a 90° bend had to be fitted inside the tank leaving a 6 in gap between the intake and the bottom of the sump tank.

The water was pumped through four 10 ft straight lengths of 10 in diameter mild steel pipe, and thence through a 10 in diameter butterfly valve by means of which the water discharge could be controlled. Since the whole channel, including the header tank, was designed to move both horizontally and vertically during tilting it was necessary that some flexible connection be provided between the butterfly valve and the header tank. However, the 4 ft length of bellows-type tubing supplied by the manufacturers extended and pulsated violently at high discharges and had to be contained in a specially constructed cradle of ½ in diameter steel rods.

Water entered the base of the header tank which was 6 ft deep, 3 ft long and 18 in wide and contained a mesh screen located immediately above the inlet. Since the header tank and the channel were of the same width and since no curved vertical transition was provided, the state of the flow entering the channel was extremely turbulent with superimposed waves and surges. Some improvement was obtained by placing a wire basket containing a bank of 12 in long, 3 in internal diameter fired clay field drains just inside the channel.

The flume itself was rectangular in cross-section with glass sides and a steel base, having approximate dimensions of:- length, 40 ft; width, 18 in; height, 21 in. It was supported by a frame of 5/16 in steel plate, 21 in deep, with several stiffening cross-members. Five pairs of 4 in diameter wheels, attached to the side of the channel support frame, rested on five equally spaced fabricated steel pedestals. The centre pedestal was bolted to the floor and by means of a large diameter turn-buckle screw between the pedestal and the underside of the channel it was possible to move the whole channel over the length of a pedestal. The parts of the pedestals on which the wheels rested were sloped such that movement of the channel could produce forward (positive) or adverse (negative) slopes of up to 1 in 240.

At the downstream end of the channel a 2 ft long by 4 in deep depression of the bed fitted with three 2 in diameter gas connections on each side was provided by the manufacturers as a sediment trap. For measurement of the rates of gravel movement obtainable in the flume it was necessary to modify the arrangement as described in section 6.2.6.

The $\frac{3}{8}$ in thick aluminium adjustable outlet weir was held in position by a number of Allen cap head screws whose location close to the outlet chute made manipulation of the Allen key difficult. A further inconvenience was that the weir height could not be adjusted without the flow of water being shut off.

6.2.2. Measurement of Water Discharge

Although the measurement of water discharge was not required for the intended experiments, it was considered, however, to be an essential part of the setting up of the sediment channel.

The general layout of the channel was unsatisfactory for the accurate measurement of flow rate for the following reasons. Arrangements for volumetric measurement the most direct and accurate method, were precluded by lack of space in the laboratory. It would also have been impossible to use the outlet weir as a sharp-edged rectangular weir since during experiments flow conditions immediately upstream would have been disturbed by the sediment trap. An alternative method would have been the installation of an electro-magnetic flowmeter in the 10 in diameter return pipe, but the cost of the meter and recorder was considered disproportionately high.

According to the BRITISH STANDARDS INSTITUTION (1964a) publication dealing with pipe flow measurement there was insufficient length of pipe downstream of the double bend and axial flow pump to reduce vorticial flow disturbances to a level at which an orifice plate meter could be used. It was decided, however, to insert an orifice place at a distance 30 ft downstream of the pump outlet and to attempt to reduce swirl by the installation of flow straighteners in the pipe (fig. 6.2.c.). Using the approximate formulae given by the BRITISH STANDARDS INSTITUTION (1964a) a sharp-edged orifice plate meter with D and D/2 pressure tappings was designed for a maximum discharge of 4.50 cusec and a pressure difference of 60 in of water. The required orifice size was calculated to be 7¾ in (irrecoverable head loss of about 2 ft). A suitably sized orifice plate was machined in the Civil Engineering Department workshop and installed with pressure tappings leading to a differential pressure water manometer constructed to measure head differences of up to 5 ft of water (fig. 6.2.d.). Large pressure fluctuations were greatly reduced by the insertion of a cross-shaped flow straightener made of ½ in steel plate (fig. 6.2.e.) 20 ft upstream of the orifice plate.

The accurate formulae of the BRITISH STANDARDS INSTITUTION (1964a) were used to calculate the calibration curve shown in fig. 6.2.f. As a check on this calibration a sharp edge was machined on the crest of the channel outlet weir, the sediment trap was covered and the channel set horizontal. (The sediment bed was not then in place). A point

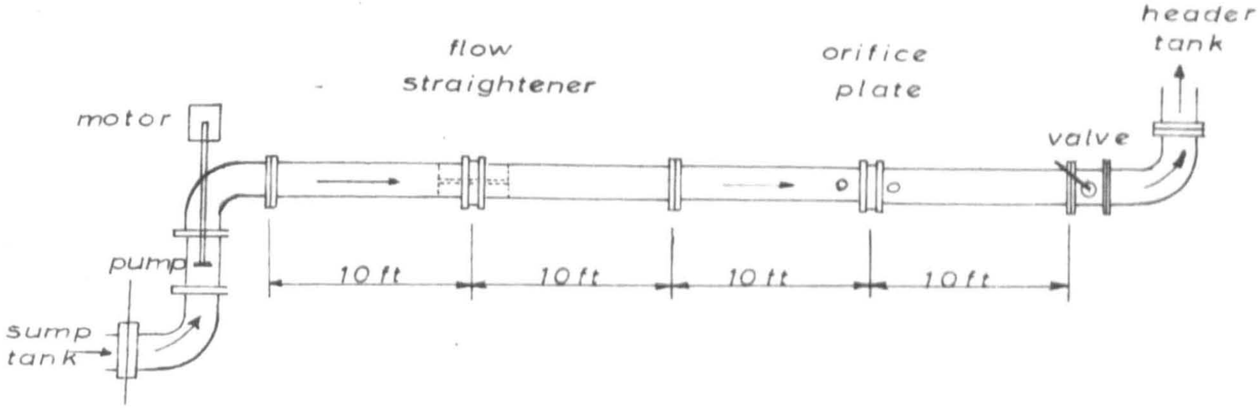


Fig. 6.2.c. Location of orifice plate and flow straightener

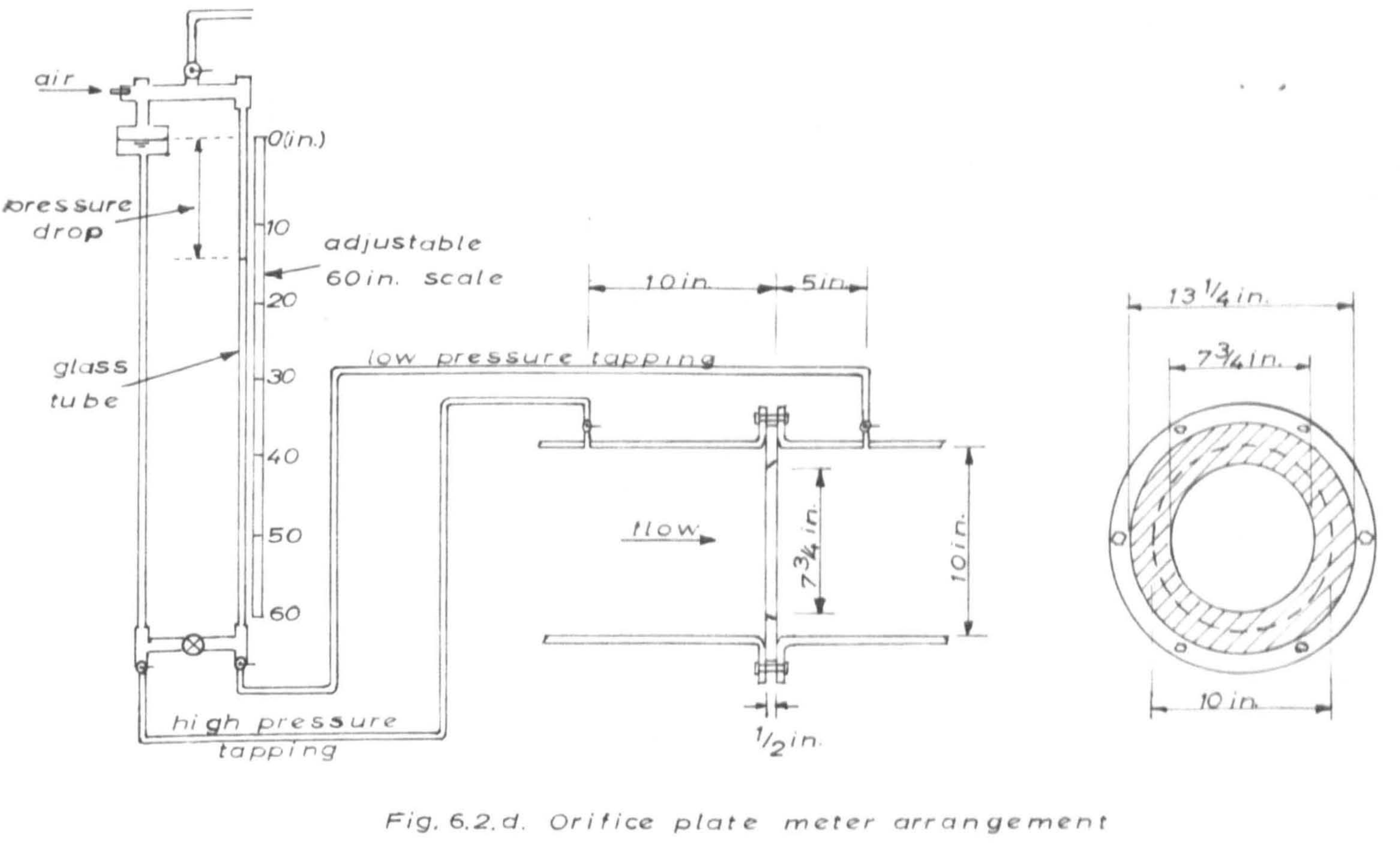


Fig. 6.2.d. Orifice plate meter arrangement

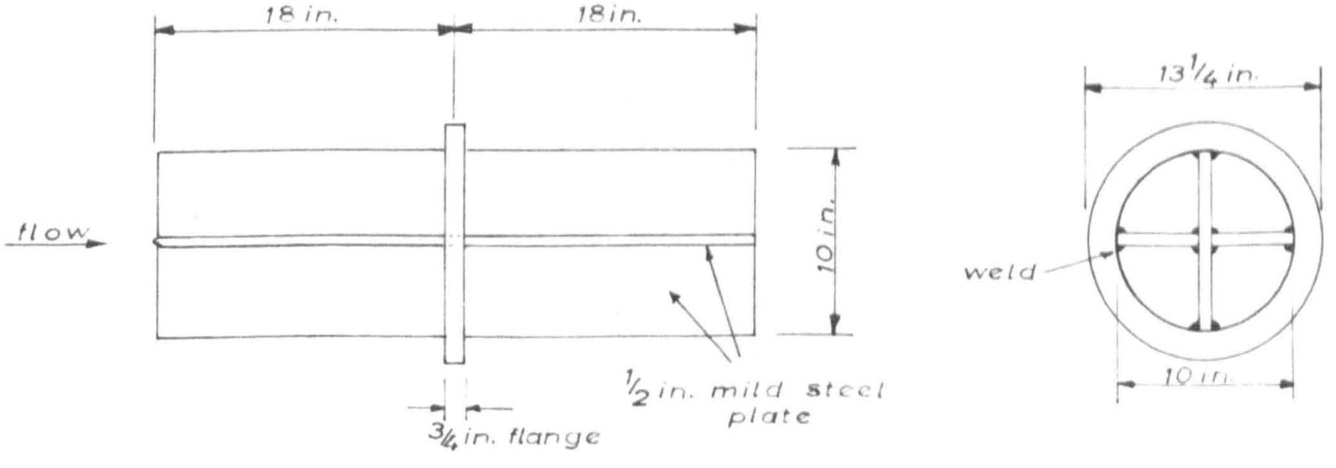


Fig. 6.2.e. Flow straightener.

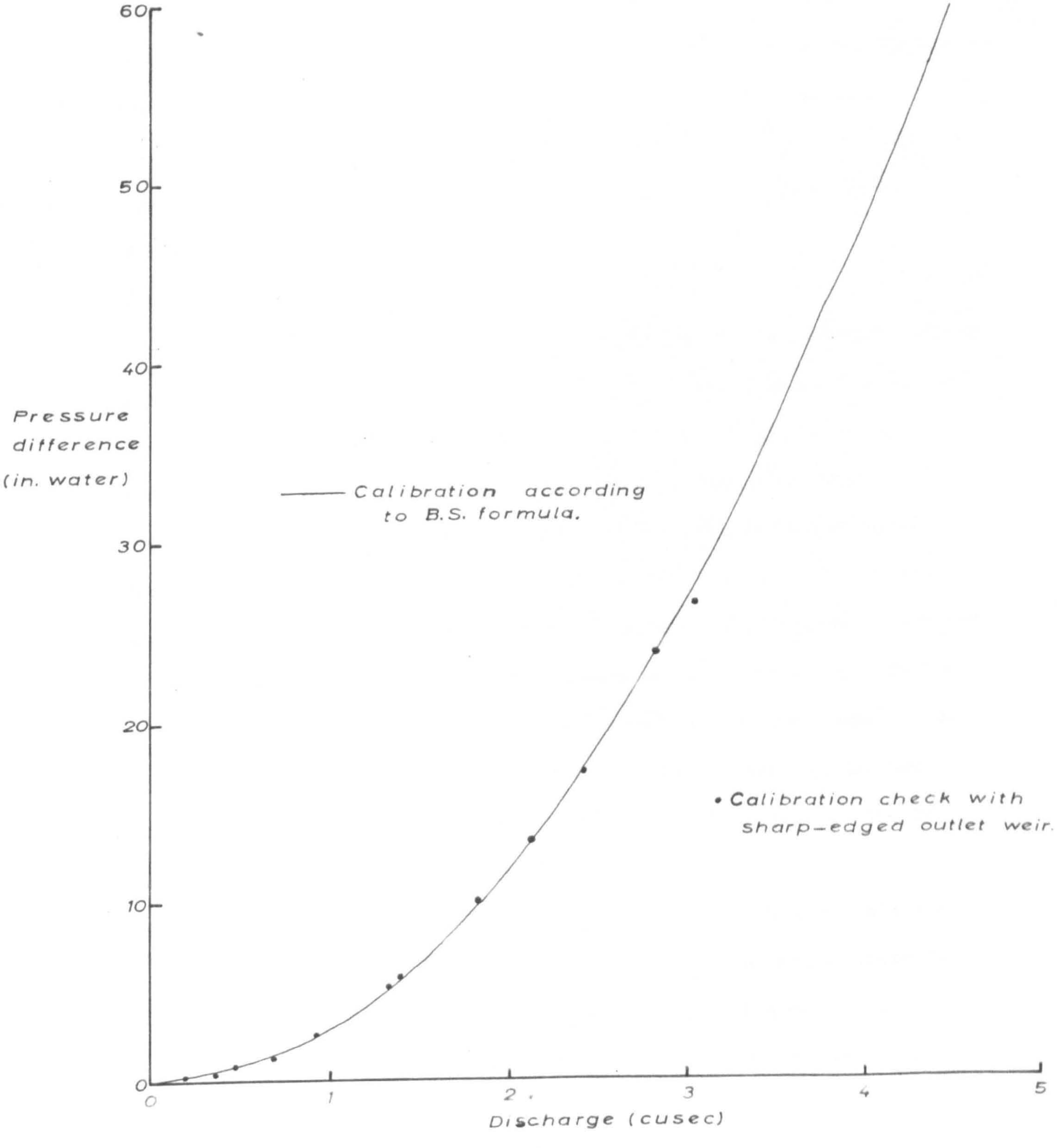


Fig. 6.2.f. Calibration of orifice plate meter

gauge located upstream was zeroed on the weir crest using a precise level and the head on the weir measured at several water discharges. Discharges were calculated according to the BRITISH STANDARDS INSTITUTION (1964b) publication on flow measurement in open channels and found to agree well with the orifice plate meter calibration curve (fig. 6.2.f.). At discharges above 2.75 cusec the ratio of head to weir height exceeded the limit stipulated by the British Standard.

6.2.3. Measurement of Channel Slope

The slope of the channel was measured by means of a form of U-tube fixed to the side of the channel. Two 6 in lengths of ½ in internal diameter perspex tube were attached in a vertical position to the channel sides at a distance apart of 400 in. The U-tube was completed by a length of green plastic hose bracketed to the channel and filled with water lightly coloured by fluorescein. The channel was set horizontal using a precise level and two 6 in scales, graduated in 1/20 in., were located beside the perspex tubes and fixed with the 3 in graduations at the same level as the menisci in the tubes. For any inclination of the channel the rise or fall of the channel bed over a length of 400 in. was thus equal to the difference of the two scale readings.

Time was not available for the provision of rails for a sliding instrument carriage or piezometer tappings to enable measurements of depth and water-surface slope to be made. For sediment studies involving the assessment of bed shear stress suitable equipment of this kind would be essential.

6.2.4. Sediment

One ton of rounded, uniform-sized river gravel (fig. 6.2.g.) was obtained from Hoveringham Gravel Co., Hoveringham, Nottingham for use

Scale: full size

Median diameter	: 0.197 in. (5.00 mm)
Geometric mean diameter	: 0.198 in. (5.03 mm)
Geometric standard deviation	: 1.30
Specific gravity	: 2.62
Specific weight	: 163.5 lb/ft^3
Bulk density	: 92.5 lb/ft^3

Fig. 6.2.g. Gravel used in laboratory investigations

in the experiments. Sieve analysis produced the particle size distribution curve of fig. 6.2.h. from which the following properties were obtained:-

Median diameter, d_{50} = 5.00 mm = 0.197 in.

Geometric mean diameter, d_g = 5.03 mm = 0.198 in.

Specific gravity, specific weight and loose bulk density were determined according to the BRITISH STANDARDS INSTITUTION (1960) as follows:-

Specific gravity, S_s = 2.62

Specific weight, γ_s = 163.5 lb/ft^3

Loose bulk density = 92.5 lb/ft^3

The gravel was composed of a mixture of particles of limestone, sandstone, quartz etc.

About ½ ton of sediment was placed in the channel to provide a 3 in. deep bed over a length of 35 ft and contained by end pieces of sheet brass. The remainder of the sediment was stored either in the hopper of the feed device or in bins in the ground floor Hydraulics Laboratory.

6.2.5. Sediment Feed Device

In an attempt to determine the best method of feeding gravel at a fixed rate into the channel several universities and research establishments were approached. The literature of previous laboratory workers in sediment research and the catalogues of several chemical firms were also consulted. It was found that most feed devices could be classified in the following manner:-

1) Material contained in a hopper is fed axially to a rotating disc beneath the hopper and extracted by means of an adjustable deflector blade. The hopper itself can be vibrated or the drive shaft of the rotating disc can pass through the material in the hopper.

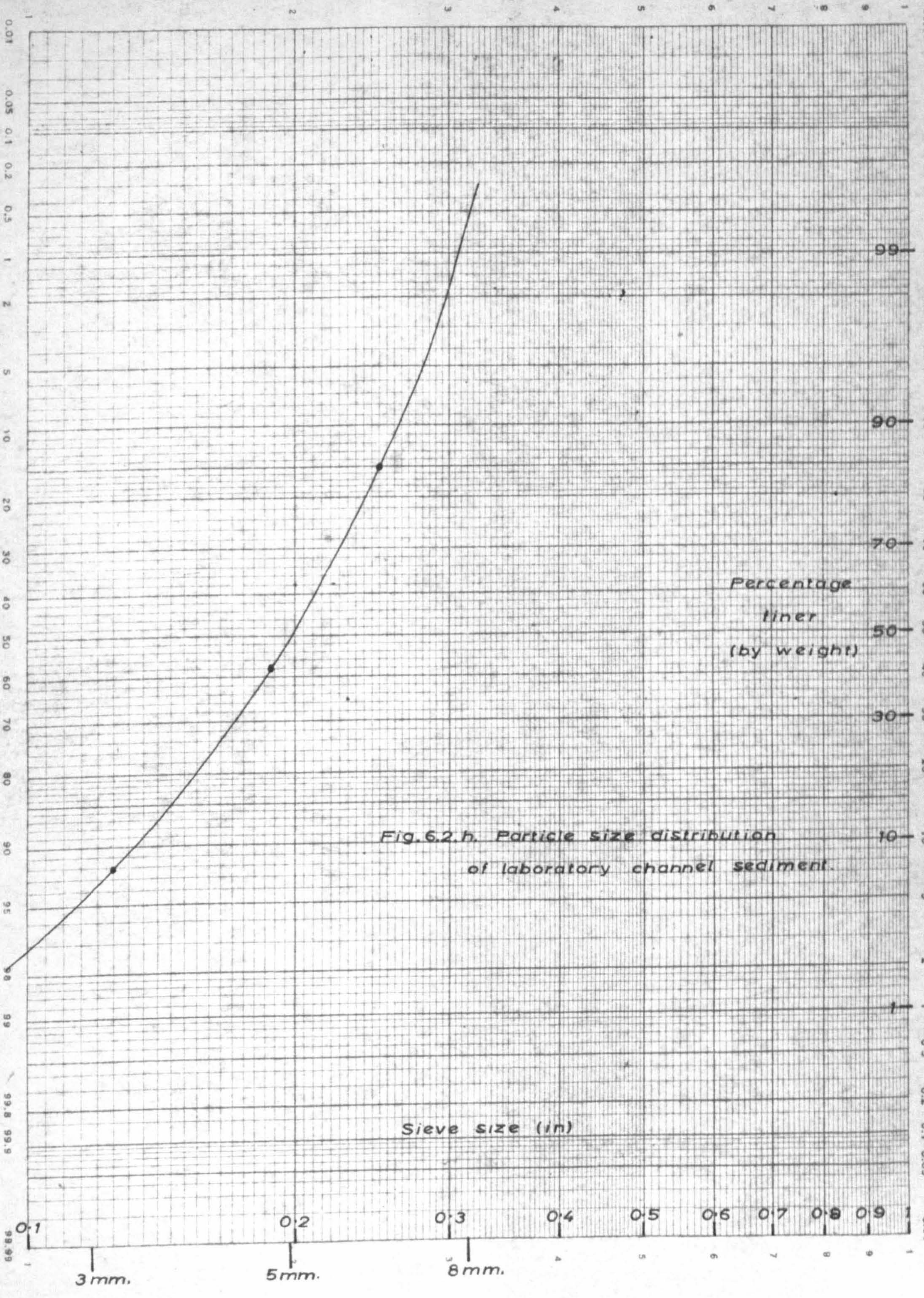


Fig.6.2.h. Particle size distribution of laboratory channel sediment.

2) Material is fed from a hopper, sometimes vibrated, into one end of a horizontal shaft in which a motor driven screw rotates. The material is discharged from the other end of the shaft at a rate dependent upon the speed of rotation of the screw. The Vibra Screw Feeder manufactured by Simon Handling Engineers, Stockport, Cheshire is of this type and has been found by the Hydraulics Research Station, Wallingford to be ideal for the feeding of sand and ground anthracite into hydraulic models (DEDOW, 1965).

3) Material is fed from a hopper on to a moving belt passing beneath the hopper. The feed rate can be controlled by the speed of the belt and the height of an adjustable gate on the hopper.

4) Material is held in a tray or shallow box which is inclined at a small angle. The tray is tapped or vibrated causing the material to pass under an adjustable gate at the lower end of the tray.

All types except 3 are usually unsuitable for material coarser than 2 or 3mm and the principal disadvantage of commercially available machines is the difficulty of obtaining sufficiently low feed rates.

Enquiries were first made to Simon Handling Engineers who claimed that their Live Bin Belt Feeder would be able to feed 5 mm gravel at rates between 10 and 600 lb/hour. The price quoted was £573. It was learnt soon after that a local firm, United Analysts, East Boldon, Co. Durham used a form of belt feeder for taking and dividing samples of crushed coal and other solid materials in a flowable condition. Arrangements were made to borrow one of their feeders so that tests could be carried out using the 5 mm gravel. The feeder consisted of a small hopper (½ ft^3 capacity) located above a 6 in. wide plastic composition belt and fitted with a 3 in. wide gate of adjustable height. A spring-loaded metal plate beneath the belt maintained contact between the hopper base and the belt. The belt was driven by a small electric motor through a worm reduction gear; by changing the size of the pulley on either the gear or the motor it was possible to determine the feed rates produced by different belt speeds and gate openings. It was also found that at small gate openings the gravel in the hopper tended to depress the spring-loaded plate and become jammed between the belt and hopper base.

On the basis of the results of these tests and in consultation with United Analysts a larger, continuously variable belt feeder was designed and constructed in the United Analysts workshops. The final price of the device was £203, an obvious improvement on the price of commercially available machines. The feeder is detailed in fig. 6.2.i. and can be seen in the general view of the channel, fig. 6.2.b. It consisted of a 6 ft^3 capacity hopper located above a 4 ft long, 3 in. wide continuous plastic composition belt which passed round two 3 in. diameter aluminium pulleys. Although the alignment of the rear pulley could be finely adjusted to prevent lateral movement of the belt over the pulley sides it was found necessary to provide guide runners on each side of the belt immediately before it ran on to both pulleys. In the original design it was intended that a series of 2 in. diameter rollers placed beneath the belt could be adjusted in height to maintain the belt in close contact with the hopper base. However, it was found that small particles still tended to become jammed between the hopper and the belt; this problem was overcome by providing a short, rubber skirt around the inside of the hopper base and by placing a smooth, thin, steel sheet between the belt and the rollers. The hopper was constructed with a 2 in. wide opening and an adjustable

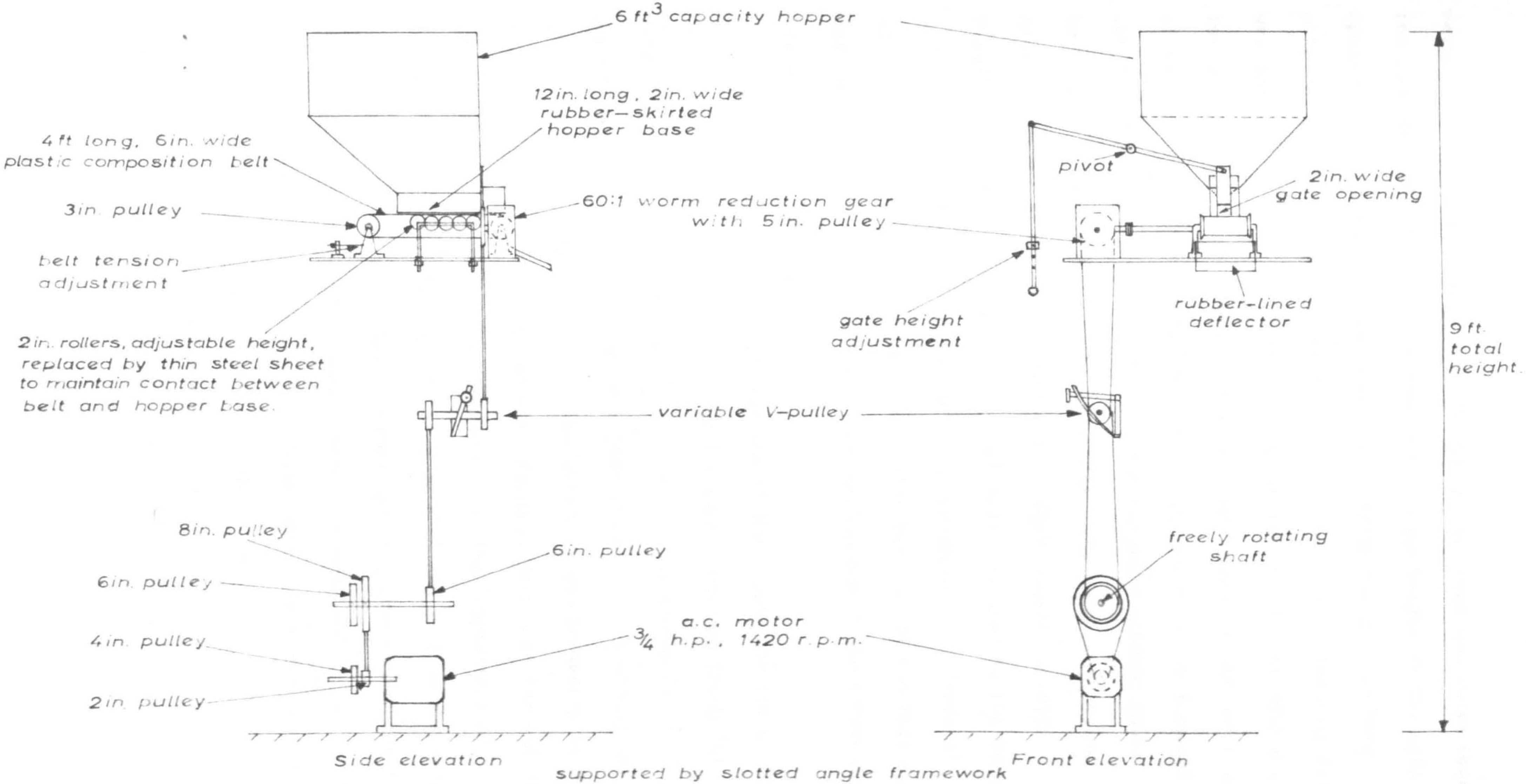


Fig.6·2·i. Sediment feed device.

vertical gate. Experience with the small belt feed had shown that the feed rate was particularly sensitive to the height of the gate opening; it was decided, therefore, to arrange the gate to have only two fixed openings of nominally $\frac{1}{2}$ in. and 1 in. The belt drive was provided by a $\frac{3}{4}$ h.p., 3 phase, a.c. motor, running at 1420 r.p.m. Two pulleys, 2 in. and 4 in. in diameter were fixed to the drive shaft of the motor. A further two pulleys, 8 in. and 6 in. in diameter, on one end of a freely rotating shaft above the motor enabled speed reductions of 8:2 and 6:4 to be selected. The other end of the freely rotating shaft was connected to a continuously variable V-pulley and thence through a 60:1 worm reduction gear to the front belt pulley. The whole arrangement was supported by a frame of slotted angle and located such that gravel falling off the belt was deflected by a rubber-lined chute into the centre of the upstream end of the channel.

Lack of clearance between the top of the hopper and the ceiling necessitated the provision of a small trapdoor through which the hopper could be loaded in the metal floor of the ground floor Hydraulics Laboratory. Since the feed device would probably not give consistent feed rates with damp gravel it was necessary to dry the material trapped in the sediment weighing device at the end of the channel. Wet gravel was placed in the top compartment of a wooden box with a horizontal division of wire and sacking and warm air from a 2 kilowatt fan heater blown into the lower compartment. By this means about 4ft^3 of gravel could be surface-dried in about 6 hours. The dry gravel was then transported upstairs and stored in bins above the hopper of the gravel feed device.

The device was calibrated using dry gravel for several pulley combinations and gate openings as shown in fig. 6.2.j; belt speeds

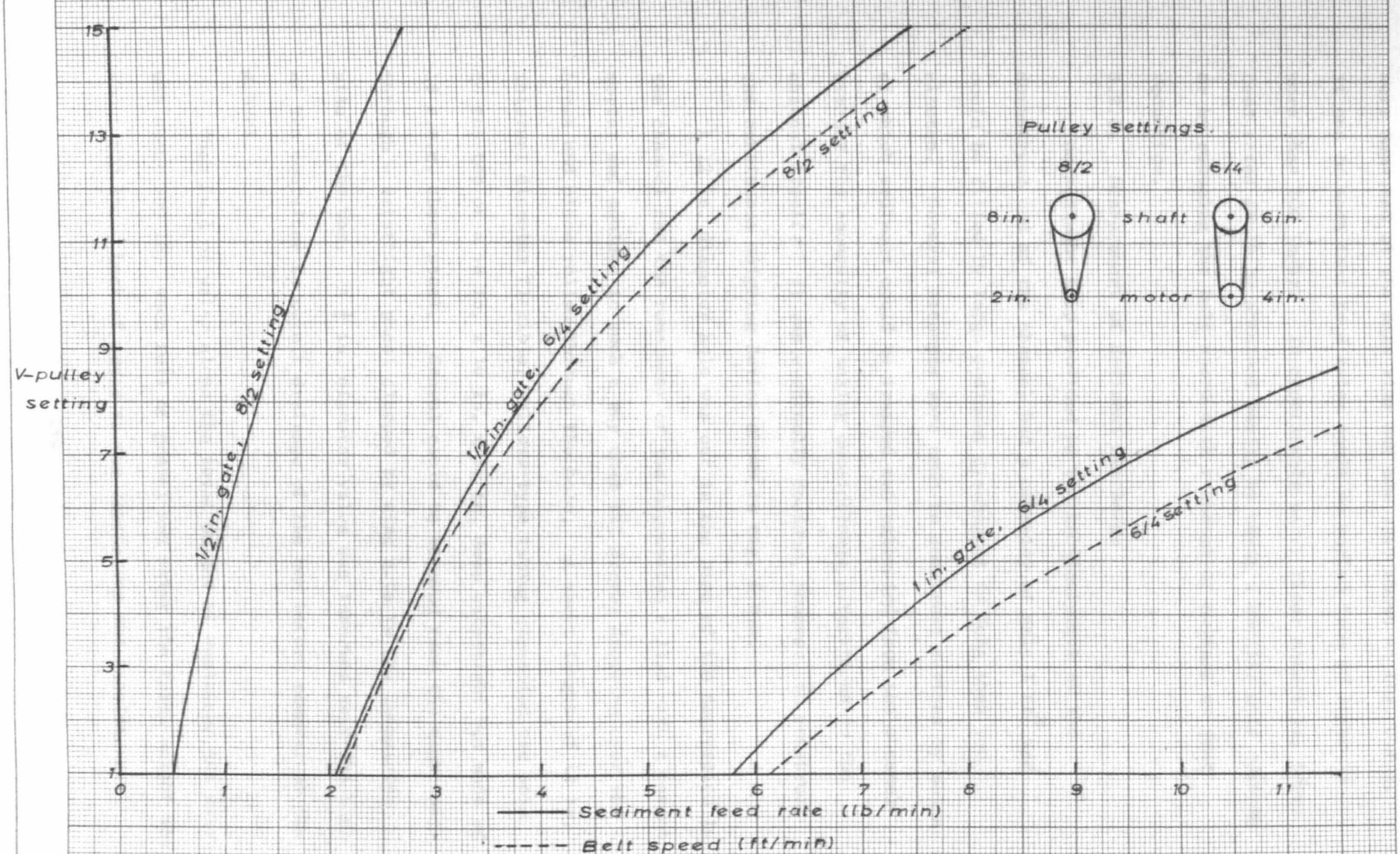


Fig. 6.2.j. Calibration of sediment feed device.

were measured at the same time and are also shown in fig. 6.2.j. It can be seen that feed rates could be varied from 30 lb/hr to more than 700 lb/hr. In experiments using finer materials, lower feed rates could be obtained easily by using a smaller gate opening or by reducing the belt speed with a change of pulleys or gear reduction.

6.2.6. Measurement of Sediment Discharge

The sediment trap provided by the manufacturers at the downstream end of the channel had a capacity of only 1 ft^3 and could be filled by large sediment discharges in less than ten minutes. Several attempts were made to construct devices by means of which sediment could be trapped and extracted from the channel without the necessity of making considerable alterations to the channel itself. It was eventually decided, however, to construct the continuous weighing device shown in fig. 6.2.k.

A 12 in. diameter hole was cut in the base of the depression in the channel and the remainder of the depression filled with a cement mortar and moulded in such a way that all gravel dropping over the brass end piece rolled through the hole. A short length of 12 in. internal diameter flexible, bellows-type tubing connected the hole to the top of a 16 in. x 16 in. x 30 in. deep watertight box made of 16 s.w.g. sheet steel. Gravel could be removed from the container through a watertight panel clamped by wing nuts to the downstream side of the box. A ½ in. diameter valve was inserted near the base of the container to permit drainage of the box before removal of the gravel.

When full of water alone the total weight of container and water was about 300 lb; when full of gravel and water the total weight was about 600 lb. Lack of space in the laboratory and insufficient

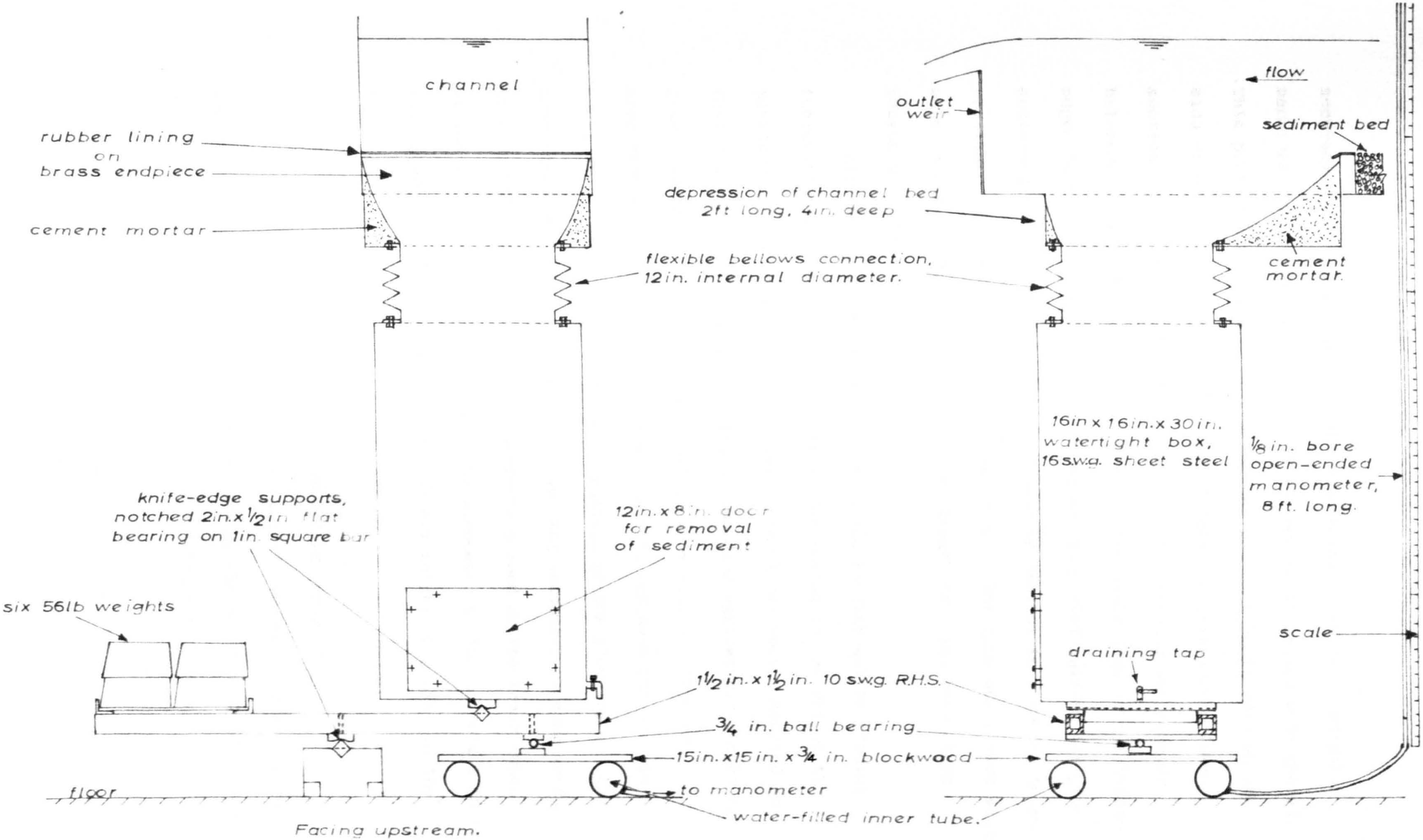


Fig. 6.2.k. Sediment weighing device.

accuracy precluded the use of most commercially available weighing machines for continuous weighing of the box as it filled with gravel. This problem was overcome by resting the box on a knife-edge on one side of a lever made of two lengths of 1½in. rectangular-hollow-section bar. The weight of the water plus container was counter-balanced by placing six 56 lb weights on the other side of the knife-edged fulcrum. Any increase in load due to gravel falling into the container was transferred by a ball-bearing contact to a wooden board resting on a motor scooter tyre inner tube. The tube was filled with water and connected to a vertical 8 ft length of ⅛ in. bore glass tubing attached to a graduated scale.

Calibration of the weighing device was carried out by placing increments of weight in the container and noting the height of the meniscus in the open-ended manometer. The loads were converted so that a direct calibration of manometer reading against dry weight of gravel in the container was obtained (fig. 6.2.1.). (During experiments an expanded graph of fig. 6.2.1. was used for greater accuracy). Due to the turbulent condition of the flow immediately downstream of the end of the gravel bed the manometer reading tended to fluctuate slightly with the fluctuating load on the inner tube. It was possible, however, to read the manometer to the nearest 1/16 in. i.e. approximately to the nearest ½ lb dry weight of gravel in the container.

6.3. Microphone and Recording Equipment

In section 6.1 it was mentioned that the movement of coarse bed load particles can be detected by recording the sound emitted by either collision of the moving particles with an object placed on the sediment bed or inter-particle collision. The former technique suffers from the same disadvantage as the use of a bed load trap, viz. it causes a

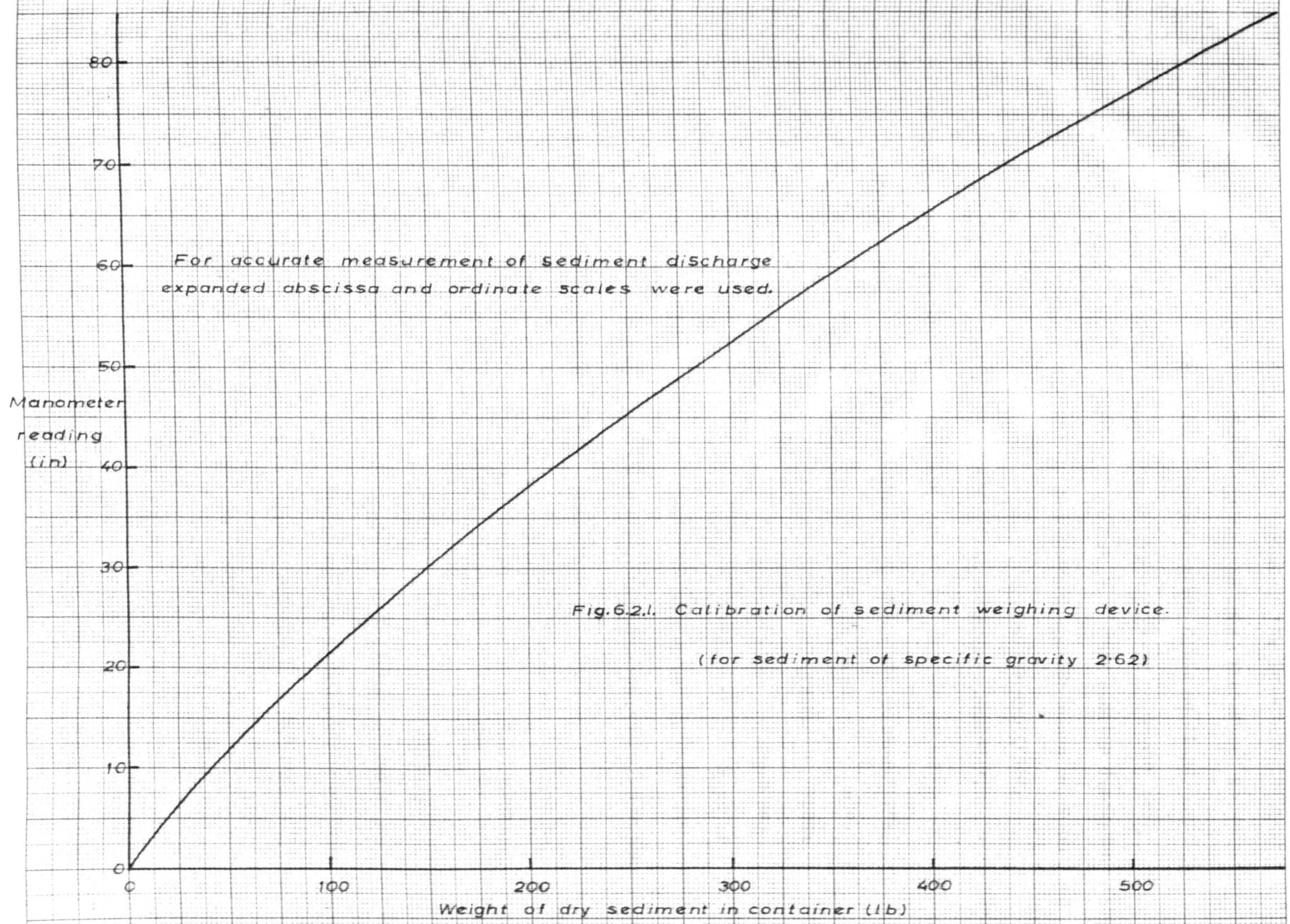


Fig.6.2.1. Calibration of sediment weighing device.

(for sediment of specific gravity 2·62)

disturbance of the sediment and flow regime in the immediate vicinity of the object placed on the bed. It was decided, therefore, to construct a small, streamlined microphone which could be fixed rigidly above the gravel bed of the laboratory flume to detect the sound of undistrubed inter-particle collison. This section describes the microphone constructed for the experiments and the means by which the electrical signal picked up by the microphone was amplified, filtered (when necessary), rectified and continuously recorded.

6.3.1. Microphone

Descriptions of the various types of microphones in existence, e.g. crystal, hot-wire, condenser, ribbon, moving-coil, carbon etc. can be found in most standard text books on sound (RICHARDSON, 1953a and 1953b, STEPHENS and BATE, 1950). After reference to these and other text books and after consultation with members of the staff of the Physics Department of the University of Newcastle upon Tyne it was decided that a crystal microphone was the type most suited for the intended laboratory experiments.

The crystal type of microphone utilises the phenomenon known as piezoelectricity. Certain crystalline materials (and some recently developed ceramic materials) undergo electric polarization when subjected to a mechanical stress, the polarization being directly proportional to the stress, and depending for its sign upon the direction of the stress. This is known as the piezoelectric effect. The phenomenon is reversible and, in the converse piezoelectric effect, when a piezoelectric material is electrically polarised it becomes strained by an amount directly proportional to the strength of the polarising field (RICHARDSON, 1957). The latter effect is used in the production of sonic and ultrasonic transducers; the former effect is utilised in the detection of sound pressure waves by recording the electric polarization, or potential difference, produced in the material by the pressure waves.

The most important natural piezoelectric materials are quartz, Rochelle salt and ammonium dihydrogen phosphate (KELLY, 1954). In recent years, however, many ceramic substances which exhibit the phenomenon have been manufactured (CRAWFORD, 1961).

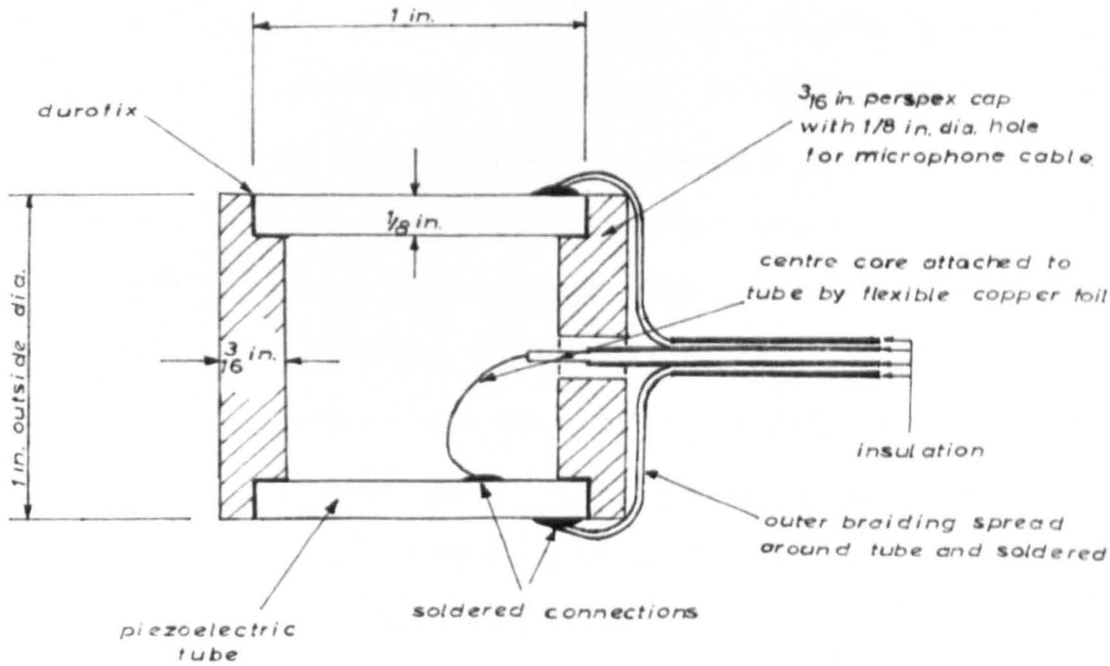
On the advice of the Physics Department of the University of Newcastle upon Tyne it was considered that a tubular ceramic element made up as shown in fig. 6.3.a. would be suitable for the detection of underwater sound in the audio-frequency range (15-20,000 Hz). It consisted of a modified lead zirconate titanate polycrystalline ceramic (PZT4, plated and polarised) manufactured by Brush Clevite Co. Ltd., Hythe, Southampton, moulded in the shape of a tube with the following dimensions:- length, 1 in.; outside diameter, 1 in.; thickness $\frac{1}{8}$ in. Perspex endcaps were fitted (ensuring water-tightness but not mechanically stressing the tube) such that incident sound pressure waves flexurally stressed the tube and caused an electrical potential difference to be set up between the inner and outer walls. One endcap was therefore provided with a small hole through which the centre core of a length of microphone cable could be connected to the inside of the tube by flexible copper foil. The outer insulated braiding of the cable was spread around the outside of the tube, tied by thread and soldered. To ensure adequate connection the whole tube was painted with conductive silver paint and then covered by a coat of waterproofing polyurethane. Two further coats of each paint were also added.

The principal advantages of this type of microphone are:-

1) The more important physical characteristics are stable over a wide range of temperature and do not change with time.

Modified lead zirconate titanate ceramic PZT–4

Brush Clevite part no.16125, plated and polarised.



Coated with:-

conductive paint (silver paint)

waterproofing paint (polyurethane)

Fig. 6.3.a. Piezoelectric crystal.

2) It has a high mechano-electrical coupling factor, i.e. a large electrical output from a small mechanical force.

3) In a tubular form the crystal is non-directional, thus having little frequency discrimination.

4) It is possible to soft solder electrical connections without destruction of the piezoelectric properties.

5) It is robust and reliable.

Since the microphone was to be suspended in flowing water it was necessary to house the detecting element in a streamlined body as shown in fig. 6.3.b. The solid hemispherical nose and conical tail sections were machined from glued $\frac{3}{8}$ in. sheets of perspex. It had been intended to make the centre portion also of perspex but it was found difficult to prevent the development of hairline cracks at the connection between the tubular brass suspension rod and the perspex; it was decided eventually to use a length of thin-walled brass tube for the centre portion. Obviously, it was desirable that minimum sound energy would be lost by reflection in the passage of the sound pressure waves from the water to the detecting element. The quantity of sound energy transmitted through the boundary of one medium with another is a function of the characteristic acoustic impedances of the two media i.e. a function of the product of density and velocity of sound in each medium. Since greater sound energy transmission occurs through the boundary of media with more equal acoustic impedance it was decided that the cavity in the streamlined body should be filled with a liquid such as castor oil. (Table 6.3.a. gives the velocity of sound in, and the density and acoustic impedance of, various materials). It would have been possible to fill the cavity with water but since castor oil is electrically non-conductive the possibility of short-circuiting due to leakage was thus prevented.

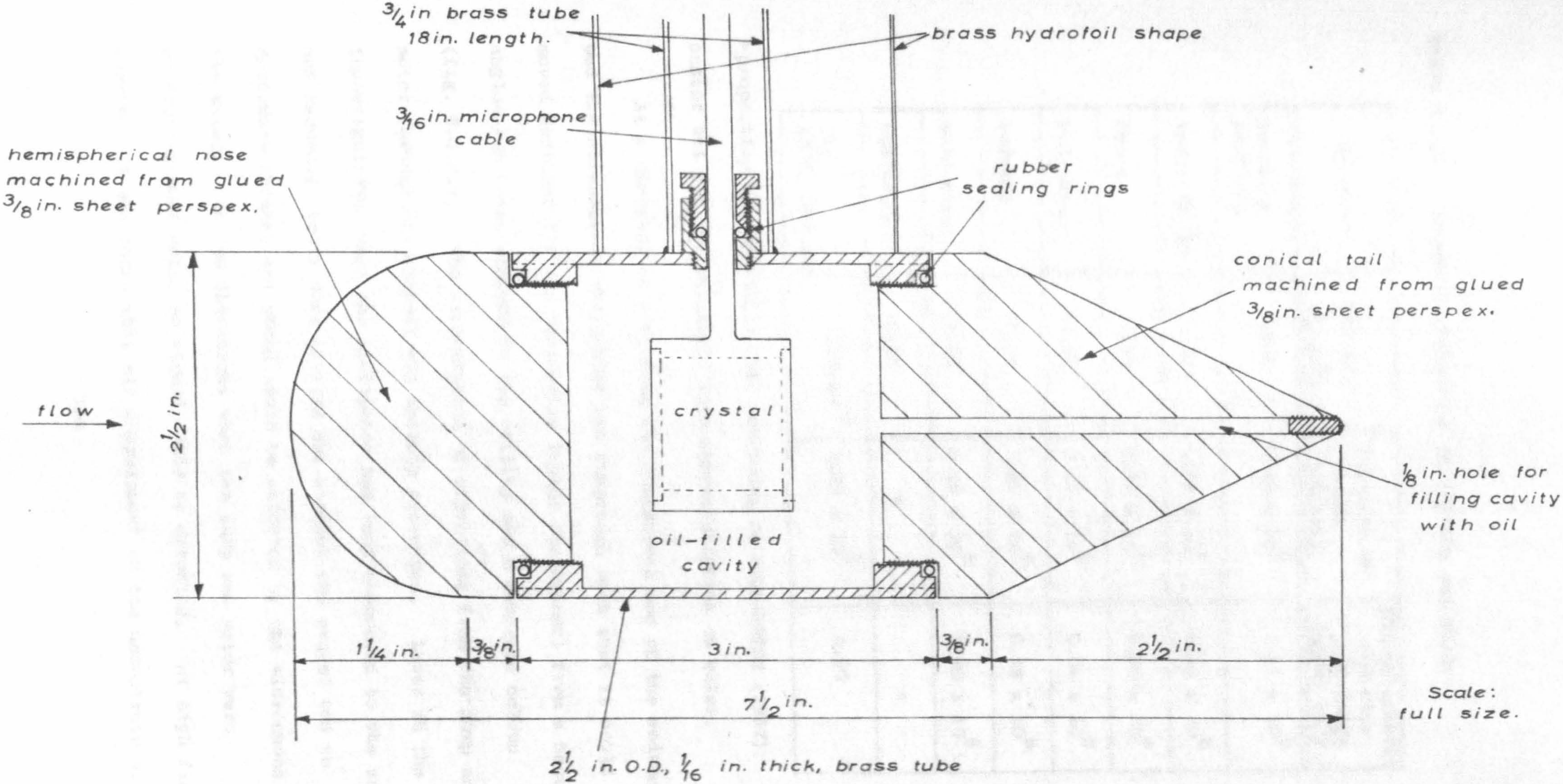


Fig. 6.3.b. Microphone for detection of sediment movement in laboratory channel.

Table 6.3.a. Acoustic properties of various materials

Material	Density (g/cm^3)	Velocity of sound (cm/sec)	Characteristic acoustic impedance ($g/sec\ cm^2$)
Sediment particle	2.62	4.22×10^5	1.11×10^6
Water (15^oC)	1.00	1.44×10^5	0.14×10^6
Brass	8.40	3.40×10^5	2.90×10^6
Polythene	0.92	0.43×10^5	0.04×10^6
Perspex	1.19	1.62×10^5	0.19×10^6
Mild steel	7.70	5.05×10^5	3.90×10^6
Castor oil	0.96	*	*
Air (0^oC, 760 mm)	1.29×10^{-3}	0.33×10^5	4.30

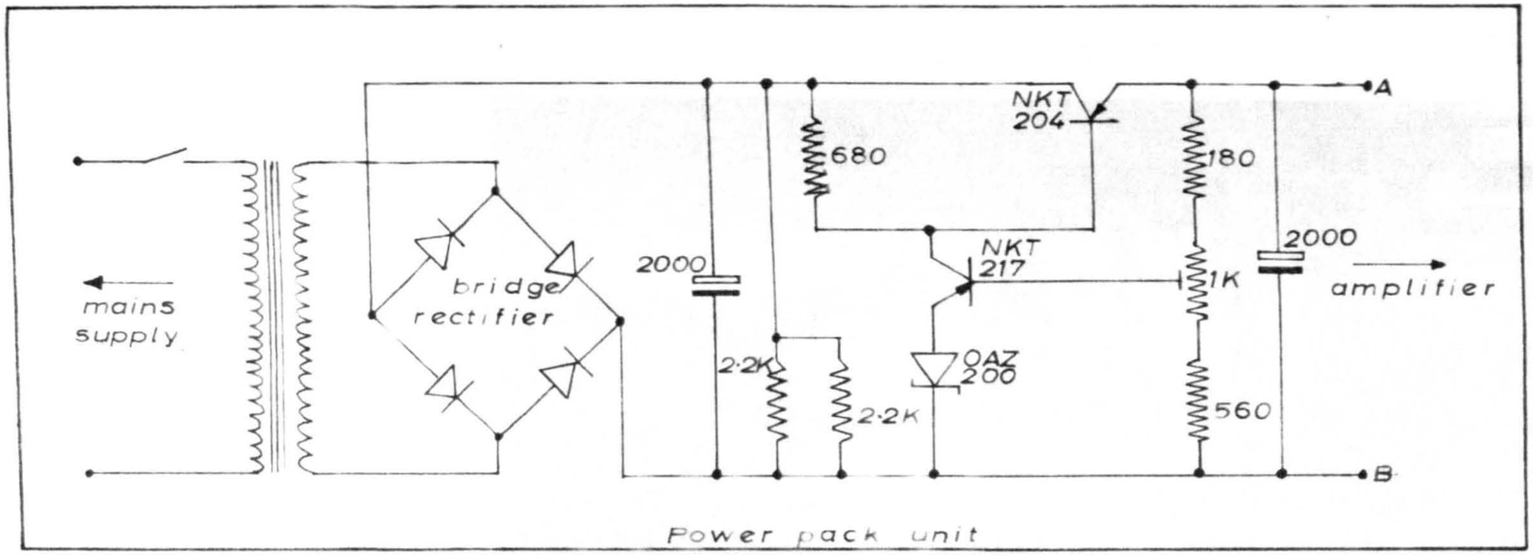
*properties not available, but according to RICHARDSON (1957) castor oil has approximately same characteristics as water.

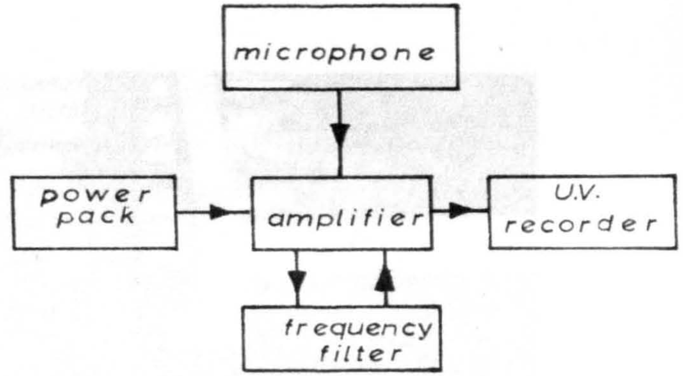
At a distance of 2 ft from the downstream end of the sediment bed the streamlined microphone was suspended such that it could be moved horizontally and vertically inside the channel from a heavy angle-iron frame secured to the ceiling and a concrete column (fig. 6.3.d.). The transmission of vibrations from the pump and motor through the channel was thereby prevented. Later in the investigations, when the microphone had been connected to the amplifier and recorder, tests were carried out without the gravel bed to determine whether any sound would be detected by the microphone in the water. At low discharges when the pump and motor were generating most noise no signal could be detected. At high flows, however, it was found that air entrainment on the downstream side of the circular brass suspension tube produced a small signal; this noise was eliminated by placing a length of thin, brass, hydrofoil-shaped tube around the circular tube. It was concluded that any sound detected by the microphone with the gravel bed in place would be caused by the movement of the sediment particles.

6.3.2. Amplifier

In order to obtain an estimate of the probable strength of the output signal of the microphone and the required amplification it was necessary to carry out some preliminary experiments. At this stage in the research programme the laboratory sediment channel was not completely operative and the sound emitted by moving sediment had to be produced by allowing sediment particles to move under gravity down an inclined trough immersed in a tank of water. In the channel the sediment particles would be moved along an almost level bed under the action of fluid shear forces). The apparatus consisted of a wooden hopper containing ½ in. concrete aggregate located above a 1 ft wide wooden trough on which a layer of aggregate had been glued. The microphone was suspended above the trough and by opening a sliding gate on the hopper and allowing gravel to roll down the slope it was possible to decide upon the amplification required to give a measurable signal.

The circuit diagram of the amplifier and output circuit used in the laboratory experiments is given in fig. 6.3.c.; the equipment itself is shown in fig. 6.3.d. A power pack unit was constructed which converted normal 240 volts, 50 Hz, a.c. mains supply to a 14 volt d.c. supply. This voltage was applied across a resistance and Zener diode to provide a ripple-free 10 volt d.c. supply, independent of mains variations, to the three-stage transistorised





All resistances in ohms.
All capacitances in microfarads.

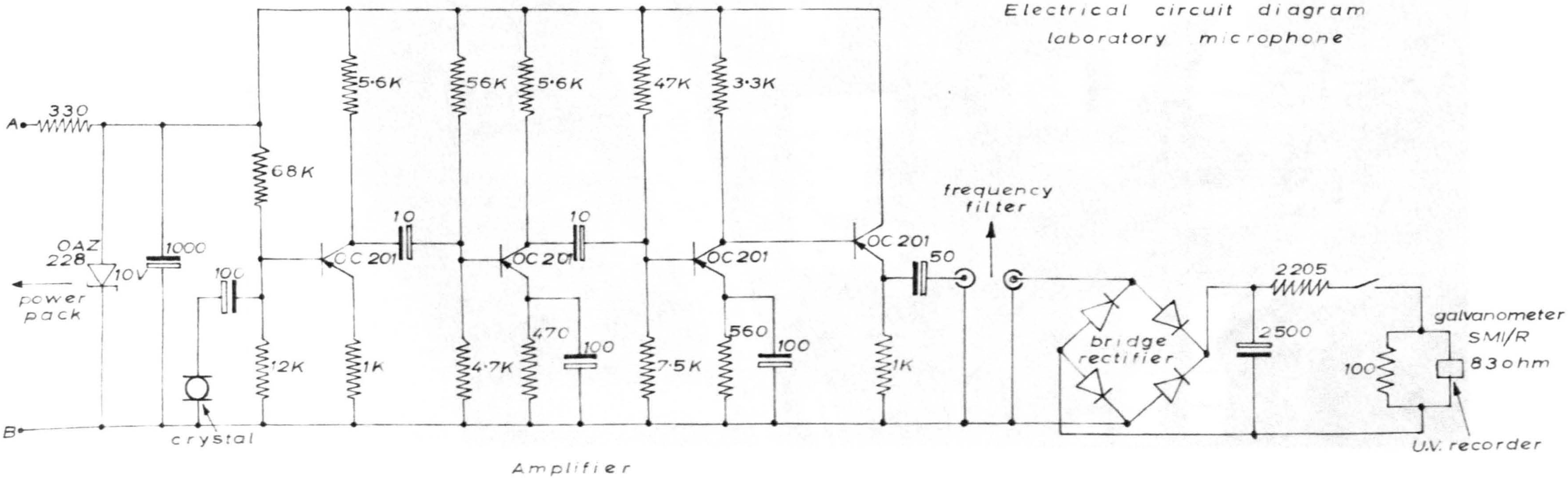


Fig. 6.3.c.

Electrical circuit diagram laboratory microphone

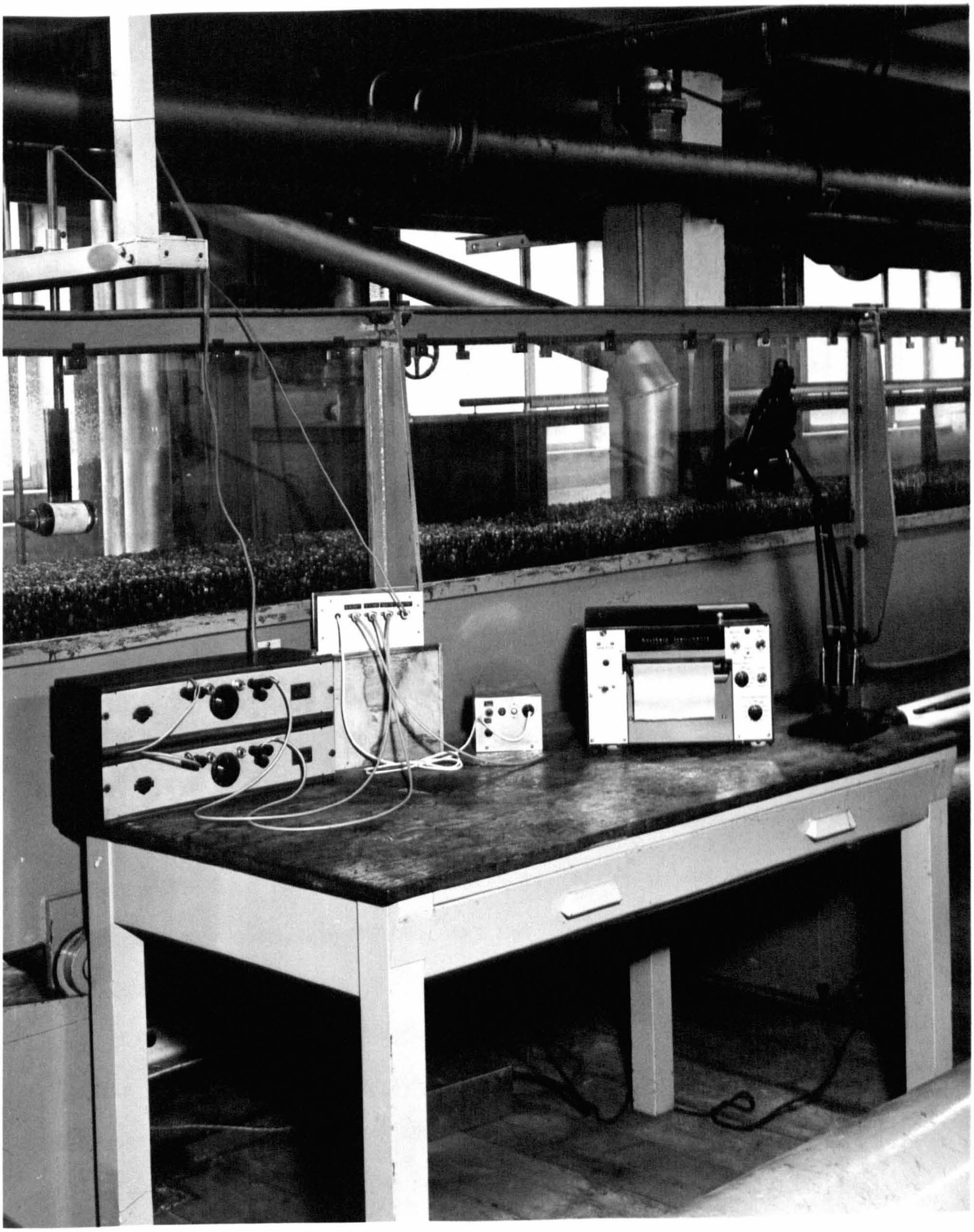


Fig. 6.3.d. Laboratory microphone and recording equipment.

amplifier. An emitter-follower was inserted to match the high impedance amplifier output to the low impedance output circuit.

An Audio-Frequency Signal Generator (Type H, Model 1B) manufactured by Advance Electronics Ltd., Hainault, Essex was used to investigate the frequency response of the amplifier. Using this instrument it was possible to replace the microphone by an a.c. sine wave input signal of 1 mv. r.m.s. value at any frequency over the range 15 to 50,000 Hz, and to observe the output current on the recorder (fig. 6.3.e.). It can be seen that the amplifer gain is constant over almost the whole audio-frequency range (3 decibel range of 140 to 20,000 Hz). The plateau current of 0.263 ma through the recorder required an a.c. output from the emitted-follower of 1.21 volts r.m.s., i.e. the voltage gain was 1.21×10^3. Linearity of gain at 1500 Hz for a range of input of 0 to 1.5 mv is shown in fig. 6.3.f.

6.3.3. Frequency Filter

It was intended to investigate the frequency spectrum of the sound emitted by the moving sediment particles in the channel. Provision was made, therefore, for the inclusion, when necessary, of a frequency filter between the emitter-follower and bridge rectifier. For this purpose, two variable cut-off high and low pass filters (Mullard types GFF 001/02 and GFF 001/01) could be connected in series enabling various frequency band widths to be selected.

6.3.4. Output Circuit and Recorder

The a.c. output signal from the amplifier and emitter-follower was passed through a bridge rectifier and a smoothing circuit containing a direct recording Ultra-Violet Oscillograph (Series M.1300) manufactured by Southern Instruments Ltd., Camberley, Surrey. This instrument was capable of simultaneous recording of up to ten channels

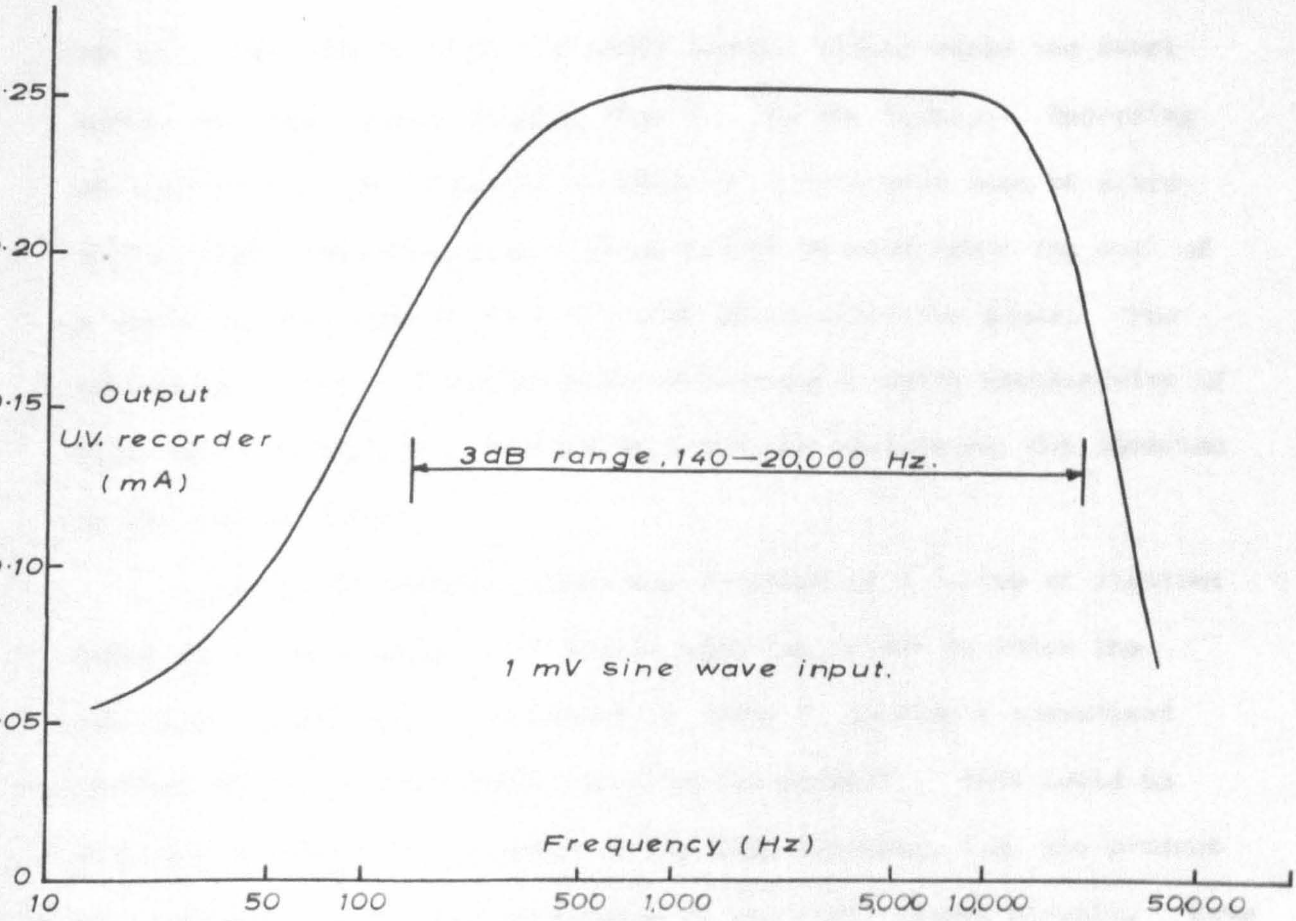


Fig. 6.3.e. Frequency response of laboratory amplifier.

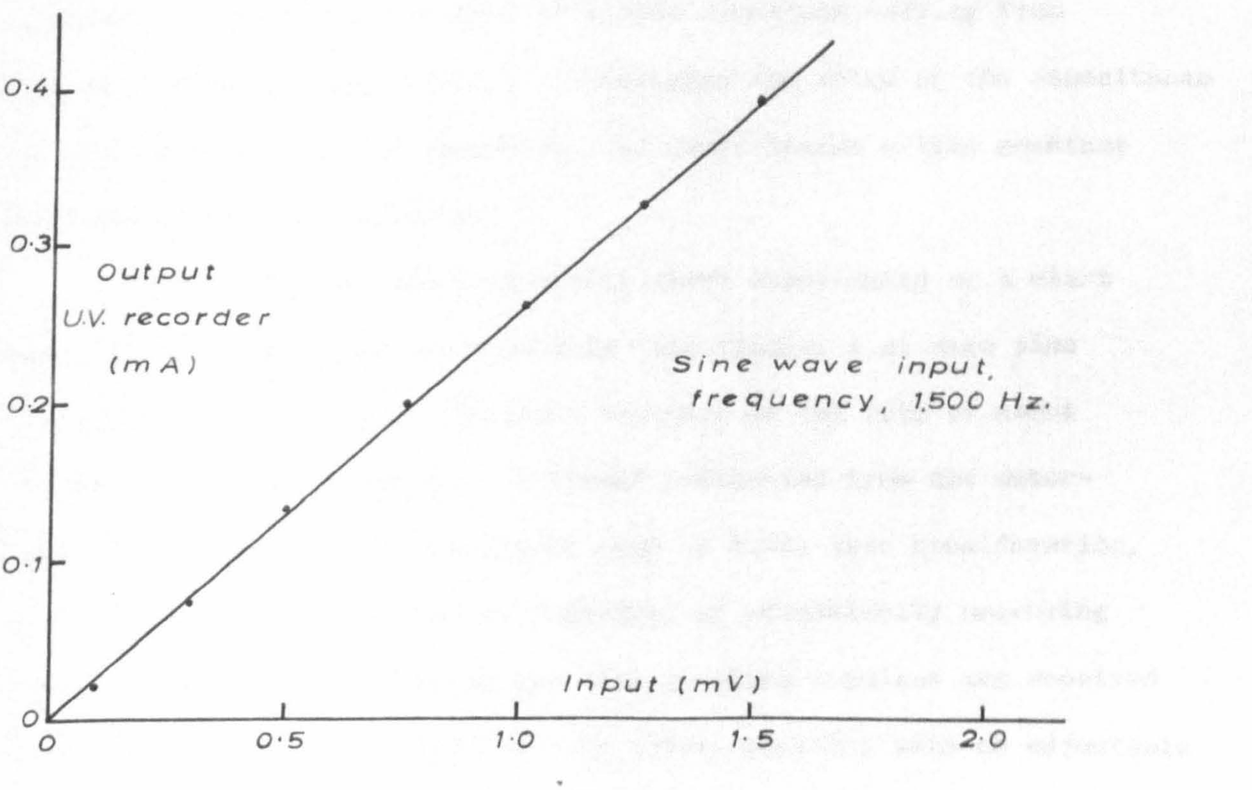


Fig. 6.3.f. Linear gain of laboratory amplifier.

of d.c. information, with reference traces, timing marks and event marks, at paper speeds ranging from 0.15 to 100 in/sec. Recording of each channel was effected by means of a zero mass beam of ultra-violet light reflected from a plane mirror mounted above the coil of a small galvanometer on to 6 in. wide photo-sensitive paper. For recording of the microphone signal galvanometer SMI/R (sensitivity of 0.01 mA/cm deflection), shunted by a 100 ohm resistance, was inserted in the oscillograph.

Since the microphone signal was composed of a series of distinct impulses it was necessary to decide upon the extent to which the rectifier output should be damped in order to provide a convenient measure of the average sound level in the channel. This could be effected by altering the value of the time constant, i.e. the product of capacitance and total resistance in the final output circuit. Some preliminary experiments were therefore carried out with the microphone fixed in the sediment channel over a high discharge of gravel. Recorder trances were obtained with time constants varying from 0.5 to 7 seconds (fig. 6.3.g.) by changing the value of the capacitance in the circuit. After examination of these traces a time constant of 5.63 seconds was selected.

A trace was also obtained during these experiments on a chart speed of 50 in/sec with no smoothing capacitance, i.e. zero time constant (fig. 6.3.h.). Distinct impulses at the rate of about 500 per second are evident. Although reflection from the water-surface and from the channel sides must be taken into consideration, the alternative possibility is indicated of acoustically measuring gravel movements by recording the rate at which impulses are received by the microphone. An impulse rate meter, possibly with an adjustable threshold setting, could be used for this purpose.

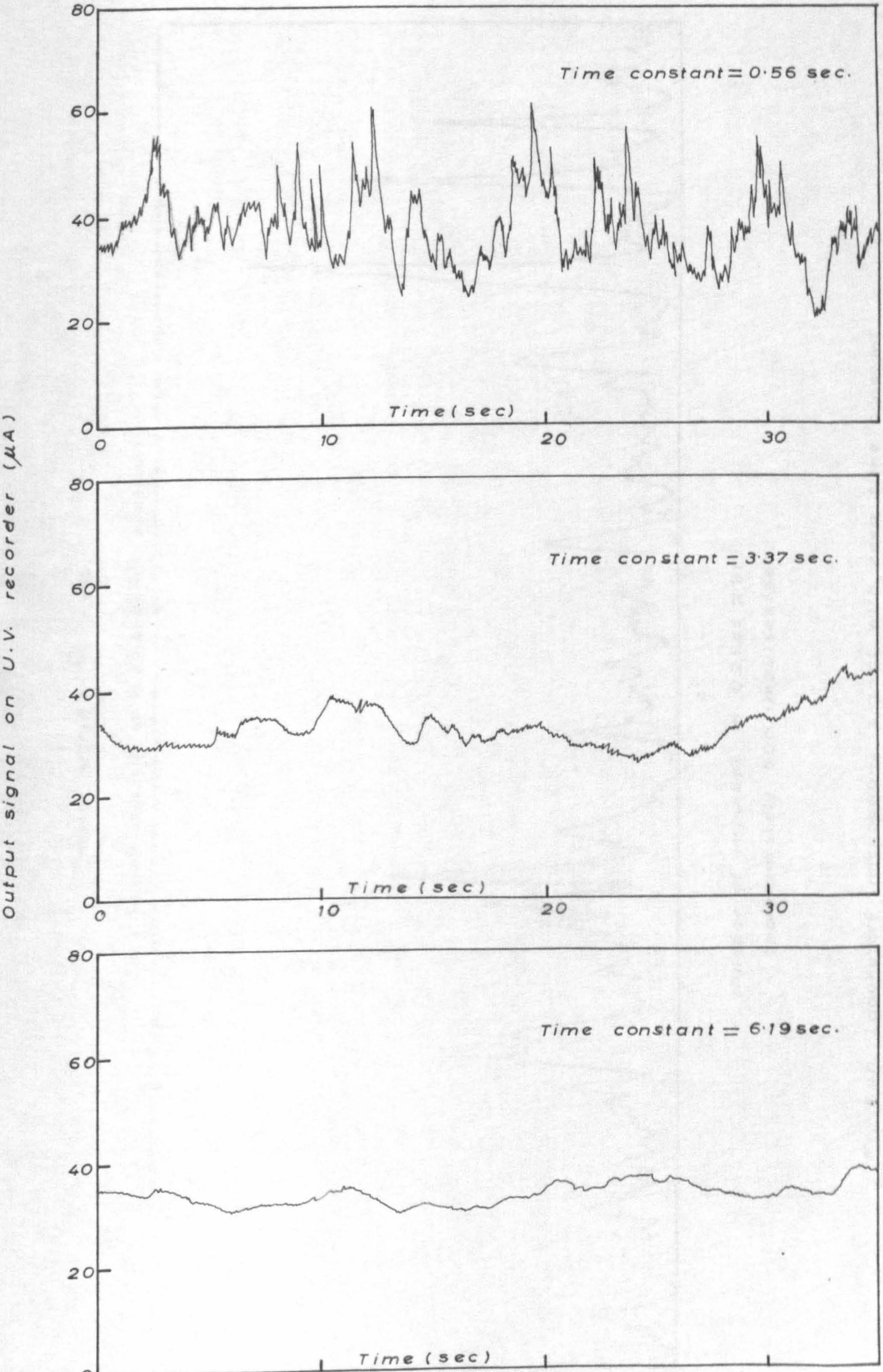


Fig. 6.3.g. Effect of time constant on laboratory microphone signal.

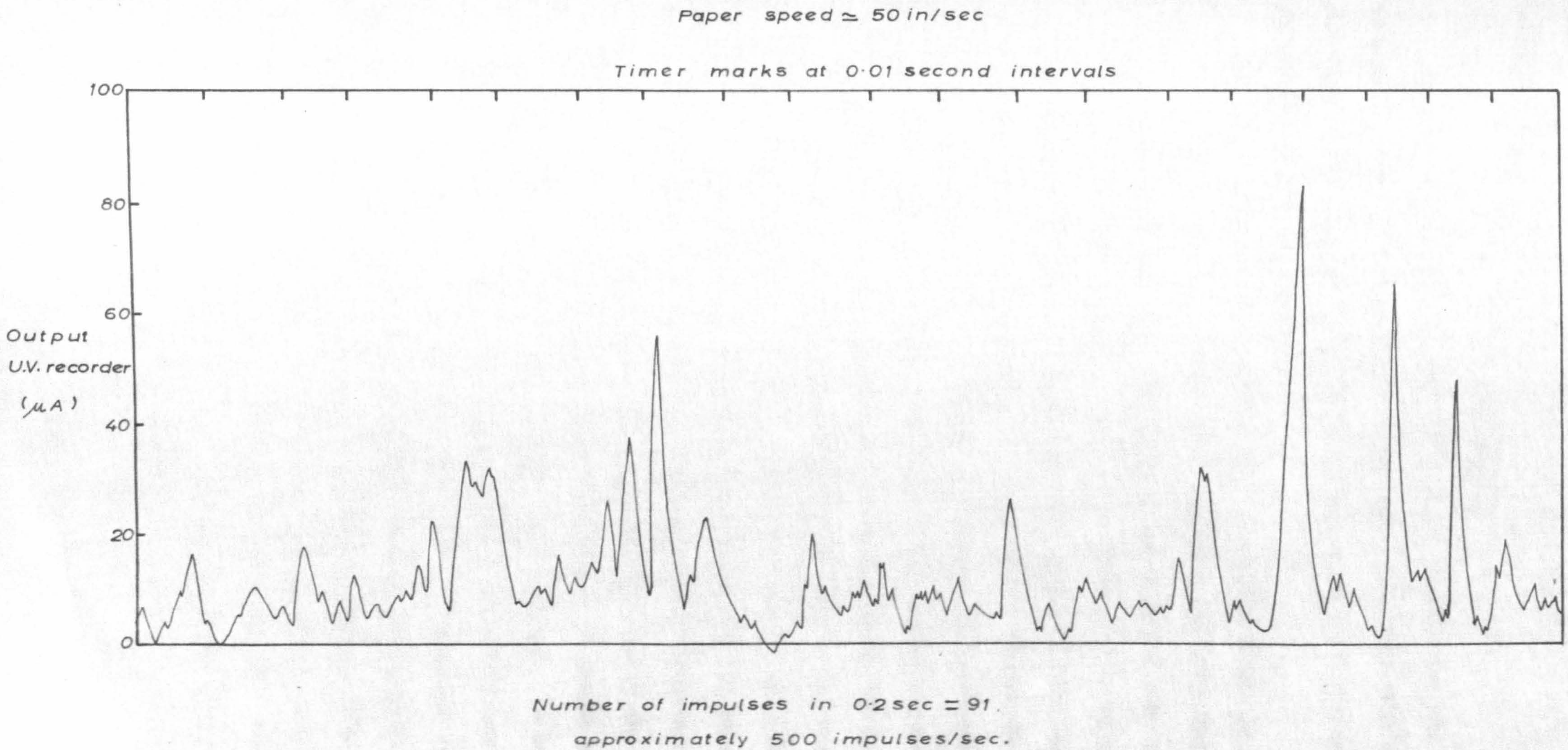


Fig. 6.3.h. Laboratory microphone signal with zero time constant.

6.4. Conclusions

The experience gained in the development of apparatus for the laboratory investigation of sediment transport indicated that the following aspects should receive particular attention in the design of a laboratory sediment channel:-

1) Measurement of water discharge. Provision should be made either for the installation of equipment for the volumetric measurement of discharge, which is possibly the most accurate method of measurement, or for a suitable location at which a pressure difference device such as an orifice plate meter may be used. With the layout of the sediment channel used in the present laboratory investigations an electromagnetic flow meter would have been desirable (but disproportionately expensive, since measurement of water discharge was not essential for the intended experiments) due to the vertical flow conditions existing in the return pipe. The accuracy of the orifice plate meter which was installed to measure discharge could possibly be increased by the insertion of a second cross-shaped flow straightener upstream of the meter.
2) Header tank. Provision should be made for a smooth and gradual transition from header tank to channel. A considerable length of channel can be wasted due to the turbulence generated by a sudden transition and the consequent waves and surges superimposed upon the flow in the channel.
3) Measurement of slope. Not only should channel slope be measured but also, especially in experiments involving the estimation of bed shear stress, the water-surface slope. This would involve the provision of a series of piezometric tappings or an instrument carriage sliding on rails fixed above the complete channel length.

4) Sediment feed. A number of methods exist for the injection of sediment into the upstream end of the channel. For the present investigations a sediment feed device was developed which was capable of feeding 5mm gravel at rates from 30 lb/hr to more than 700 lb/hr. It operated on a hopper and continuous moving belt principal, the feed rate being controlled by an adjustable gate on the hopper and a continuously variable V-pulley system.

5) Measurement of sediment discharge. The installation of a suitable device for the measurement of sediment discharge involved considerable modification of the original channel design. A 4½ ft^3 capacity watertight container was connected by flexible tubing to the base of the downstream end of the channel such that bed load was discharged continuously into the container. The weight of the container was transferred through a counter-balanced lever to a water-filled motor scooter tyre inner tube connected to a vertical open-ended manometer. The manometer was calibrated to give directly the dry weight of gravel in the container, and by this means a continuous record of sediment discharge could be obtained. The weight of dry gravel in the container could be determined to the nearest ½ lb.

For the detection of the sound emitted by inter-particle collision of moving sediment a piezoelectric ceramic microphone housed in an oil-filled streamlined body suspended above the sediment bed was designed and constructed. After some preliminary experiments it was concluded that a three-stage transistorised amplifier and emitter-follower (voltage gain of 1.21×10^3, 3 dB frequency range of 140-20,000 Hz) was sufficient to obtain a measurable signal. The amplifer output was connected to a bridge rectifier and a smoothing circuit (time constant 5.63 sec) containing a direct recording ultra-violet oscillograph.

Some experiments with zero time constant in the output circuit showed that impulses were received by the microphone when suspended over moving sediment at the rate of about 500 impulses per second. This suggested an alternative possibility of acoustic measurement of bed load discharge using an impulse rate meter.

Further experiments with the microphone suspended in the channel from a rigid angle-iron frame attached to the ceiling indicated that any sound detected by the microphone would be due solely to the movement of sediment particles in the channel.

Section 7

7. Laboratory Investigation of the Acoustic Detection of Bed Load Movement

The principal aim of the laboratory investigations was to determine experimentally the relationship between average bed load discharge and the average signal recorded by the microphone. It was considered, however, that certain aspects of the acoustic detection of bed load movement in the laboratory channel should be investigated before carrying out the main series of experiments. Section 7.1., therefore, briefly discusses a number of preliminary experiments concerning the location of the microphone above the bed, the area of sensitivity of the microphone, the influence of the channel sides and the frequency spectrum of inter-particle collison sound emitted by the channel sediment. The main series of calibration experiments is then described in section 7.2., followed by a theoretical derivation of the relationship between bed load discharge and microphone signal in section 7.3. Section 7.4. includes a discussion of the experimental and theoretical relationships, and section 7.5. summarises the conclusions reached during the laboratory investigations.

7.1. Preliminary Experiments

7.1.1. Location of Microphone above the Sediment Bed

The height of the microphone above the sediment bed could be expected to influence the results of the main experiments in two ways:-

1) Location of the microphone close to the sediment bed would produce an increase in the velocity of flow near the surface of the bed. The resulting local disturbance of the sediment regime immediately below the microphone would not be representative of the general conditions of sediment transport in the remainder of the channel bed. Since the microphone was most sensitive to inter-particle collision sound emitted by this part of the bed it was

considered necessary to avoid such disturbance of the flow velocity near the bed. Experiments were carried out, before the sediment bed was placed in the channel, with the microphone fixed at various heights above the metal base of the channel. Flow velocities near the bed were measured using a miniature velocity probe, the axis of the propeller of the probe being located at a height of 0.33 in. above the bed. It was found that maximum disturbance of the flow velocity occurred at a point 3 in. downstream of the hemispherical perspex nose of the microphone. Measurements of bed velocity were made at this point for several water discharges. Fig. 7.1.a. shows the increase in flow velocity as the microphone was lowered towards the bed at two water discharges. Although turbulent velocity fluctuations made the detection of small increases in bottom velocity difficult it was concluded that, with the microphone located at least 3 in. from the bed, velocities near the bed surface would be unaffected.

2) The height of the microphone above the bed could possibly affect the magnitude of the signal detected by the microphone. However, if the moving sediment bed is assumed to act as a large source of closely-spaced spherical wavefronts, each produced by an inter-particle collision, then it can be concluded that the average distance of the microphone from a wavefront source would not be changed significantly by a relatively large change in the vertical distance of the microphone from the bed. Alternatively, since the impulse sources are not only

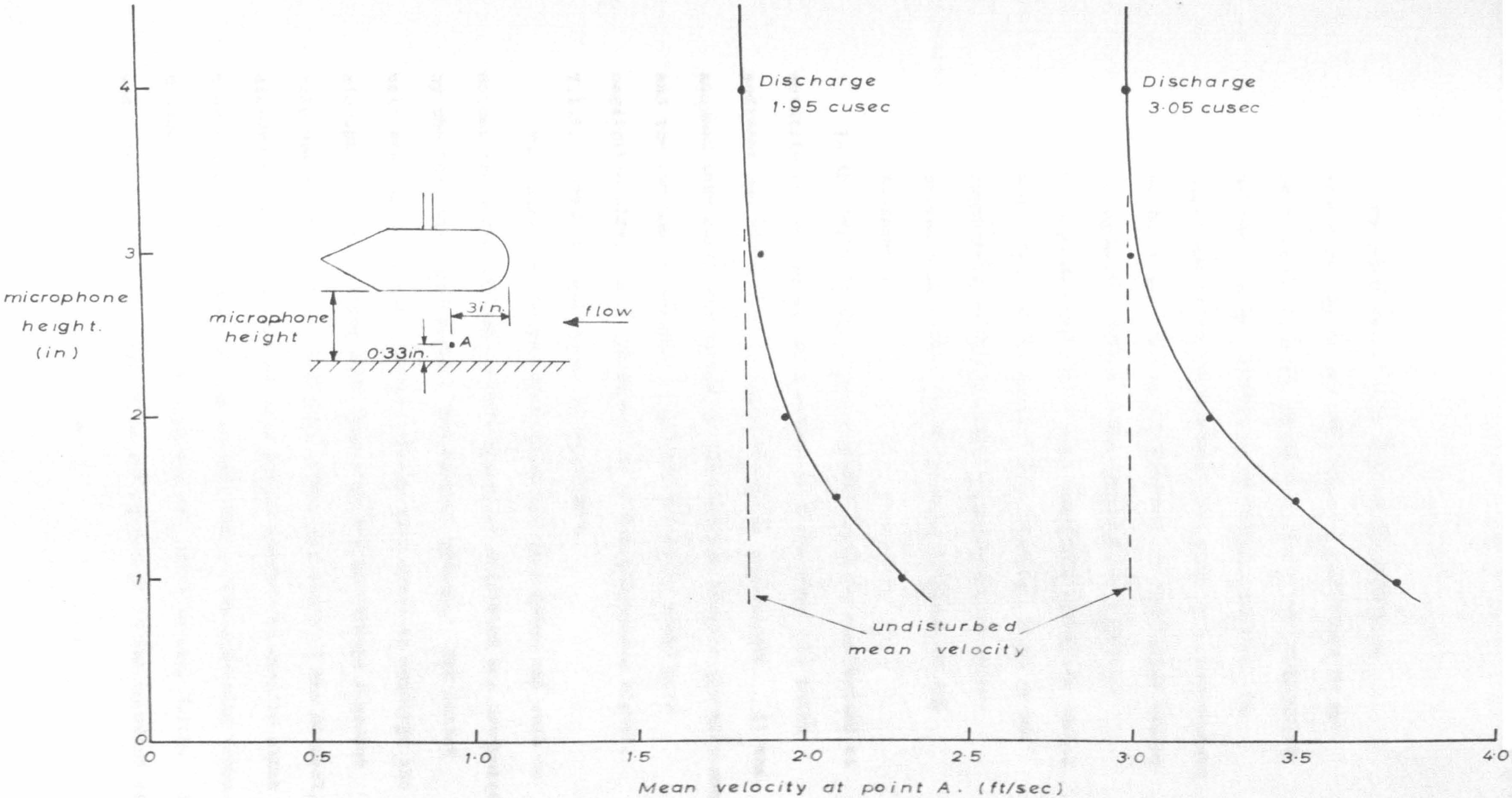


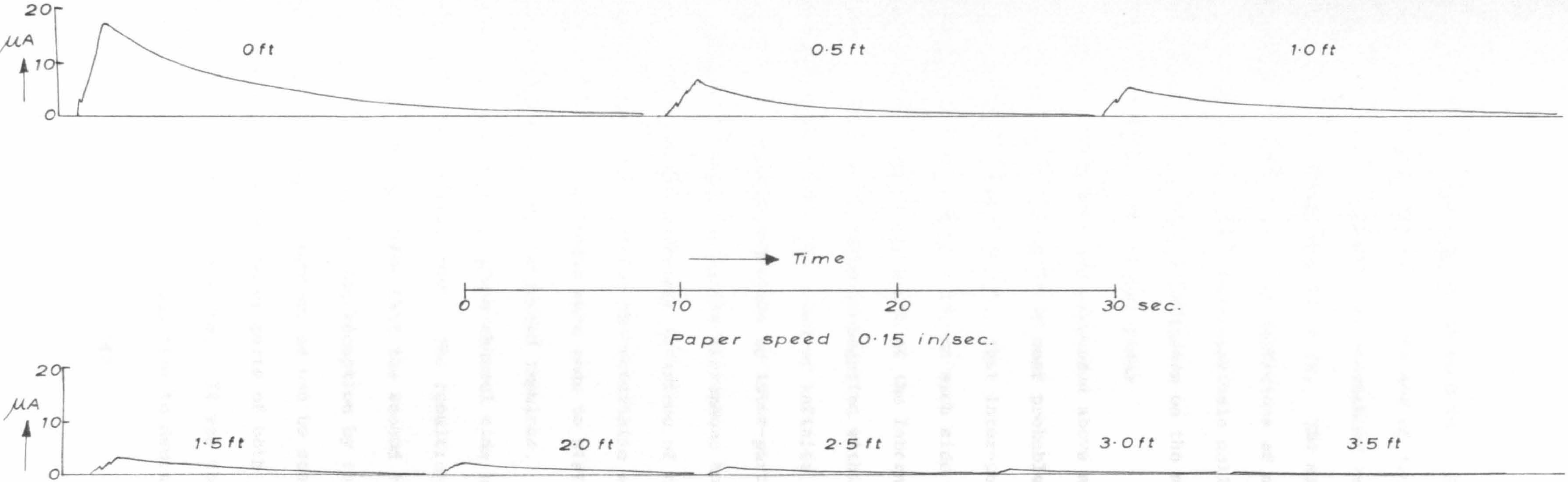
Fig. 7.1.a. Effect of microphone height on flow velocity near bed.

closely-spaced but also frequently occurring (section 6.3.4), the bed could be considered to act as a source of plane waves such that, with negligible attenuation by turbulence and viscous damping, the magnitude of the microphone signal would be independent of the height of the microphone. A controlled series of tests to confirm these conclusions, with the microphone fixed at several positions above the moving bed, could not be carried out, however, since it was impossible to ensure exact reproduction of inter-particle collision sound for each position of the microphone.

In the light of the above considerations it was decided to maintain the microphone at a height of 3 in. above the plane sediment bed throughout the main series of experiments. It was assumed that small variations in the distance between the microphone and the bed due to changes in bed configuration would have negligible effect upon the magnitude of the microphone signal.

7.1.2. Area of Sensitivity of Microphone

The length of channel over which the microphone was able to detect the sound emitted by inter-particle collision was estimated by the following approximate, but direct, method. The outlet weir was raised and the channel filled with water to submerge the microphone. At various distances from the microphone a wooden rule was drawn lightly but firmly across the width of the channel, disturbing the bed particles in a manner similar to that in which they are disturbed when moving as bed load. The resulting traces produced by the ultra-violet recorder are shown in fig. 7.1.b. It was concluded that the microphone was sensitive to the movement of



Recorder traces produced by movement of bed particles at various distances from microphone.

Fig. 7.1.b. Determination of area of sensitivity of laboratory microphone.

gravel at distances of up to $2\frac{1}{2}$ to 3 ft. Since the microphone was located 2 ft. upstream of the end of the sediment bed the length of channel to which the microphone responded was thus estimated to be approximately 5 ft. The microphone signal was therefore dependent upon the conditions of sediment movement over an area of about $7\frac{1}{2}$ ft^2, inter-particle collisions nearer the microphone having greater influence on the magnitude of the signal.

7.1.3. Influence of Channel Sides

If the microphone was suspended above an infinitely wide movement of gravel it would be most probable, on the basis of the experiments of section 7.1.2., that inter-particle collisions at distances of up to $2\frac{1}{2}$ to 3 ft. on each side of the microphone would be detected. Since the width of the laboratory channel was only $1\frac{1}{2}$ ft. conditions of sound propagation within the channel were thus different from those over a bed of infinite width. In the laboratory, impulses generated by inter-particle collision within the channel probably reach the microphone both directly and by reflection from the glass-air interface of the channel sides (due to the dissimilarity of the characteristic acoustic impedances of glass and air). Attempts were made to distinguish experimentally between the direct and reflected impulses. The streamlined microphone body and the glass channel side were in turn given sharp taps with a metal rod. The resulting traces are shown in fig. 7.1.c. It was hoped that the second trace would reveal a slower decay rate due to the reception by the microphone of reflected impulses. However, as can be seen from fig. 7.1.c., a logarithmic plot of the decay parts of both traces revealed no significant difference in slope. It was thus not possible with the apparatus available at the time to demonstrate the existence

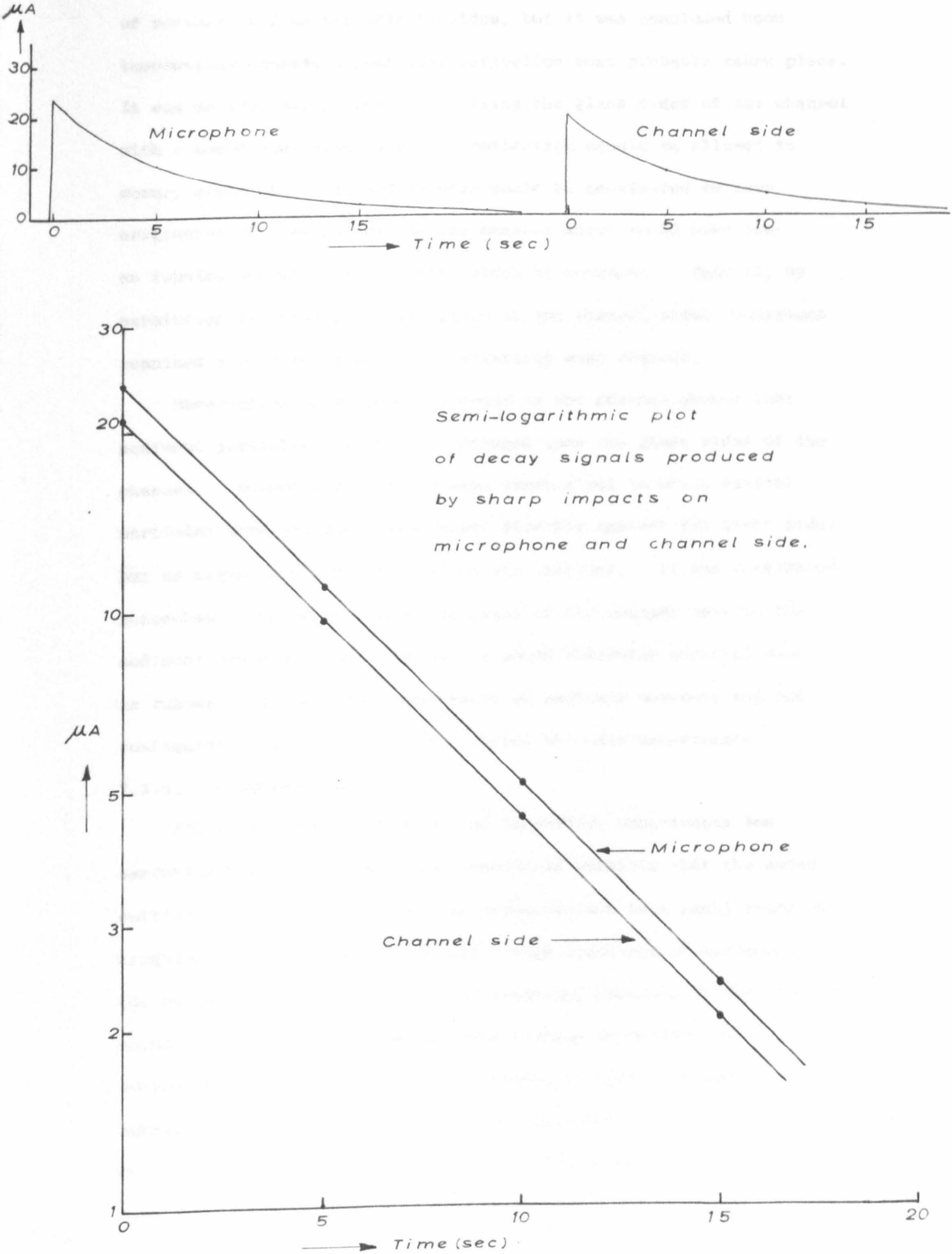


Fig. 7.1.c. Comparison of decay signals.

of reflection from the channel sides, but it was concluded upon theoretical considerations that reflection most probably takes place. It was decided that, instead of lining the glass sides of the channel with a sound absorbing material, reflection should be allowed to occur, since the reflected impulse could be considered to have originated at a point outside the channel which would have been an impulse source in an infinite width of movement. That is, by permitting reflection to take place at the channel sides conditions remained more like those in an infinitely wide channel.

Observation of sediment movement in the channel showed that sediment particles only rarely impinged upon the glass sides of the channel. Nevertheless, tests were carried out in which several particles were projected underwater directly against the glass side, but no signal could be obtained on the recorder. It was considered unnecessary, therefore, that the sides of the channel near to the sediment bed should be lined with a sound absorbing material such as rubber. In this way observation of particle movement and bed configuration was still possible during the main experiments.

7.1.4. Frequency Spectrum

Since the sediment used in the laboratory experiments was essentially single-sized it was considered possible that the sound emitted by the particles might be characterised by a small range of frequency. It was decided to set a high discharge of sediment in the channel and to investigate the frequency spectrum of the resulting sound. Using the high and low pass filters (described in section 6.3.3.) in series it was possible to select frequency bandwidths with centre frequencies at approximately ½ octave intervals (ratio at bandwidth to centre frequency, 1:3.25). The main difficulty in these experiments was the maintenance of a sediment discharge which produced a signal sufficiently large and steady to enable the various bandwidths to be accurately sampled.

Fig. 7.1.d. shows a plot of filtered signal per Hz bandwidth against centre frequency. It can be seen that the frequency spectrum extends over a wide range, almost the whole audio-frequency range, and that there is no particular characteristic peak frequency. It was decided, therefore, not to filter any selected ranges of frequency during the main experiments but to sample the total signal within the frequency cut-off range of the amplifier (3dB range, 140 - 20,000 Hz).

7.2. Calibration Experiments and Analysis

This section describes the main series of calibration experiments and the analysis of results. The principal aim of the experiments was to determine the relationship between average bed load discharge and the average signal recorded by the microphone in the laboratory flume.

7.2.1. Experimental Procedure

The main laboratory investigations comprised a number of experimental runs, each conducted at a constant water discharge; during each run a continuous record was kept of sediment discharge, sediment feed rate and microphone signal.

Throughout the whole series of experiments the channel was set at the maximum slope of 1 in 240. Before each run the surface of the sediment bed was moulded to a parallel slope by a wooden template, and the microphone fixed at a height of 3 in. above the plane sediment bed. At the start of each run a steady water discharge was set and measured, and approximate depths of flow noted at upstream and downstream sections of the channel; the height of the

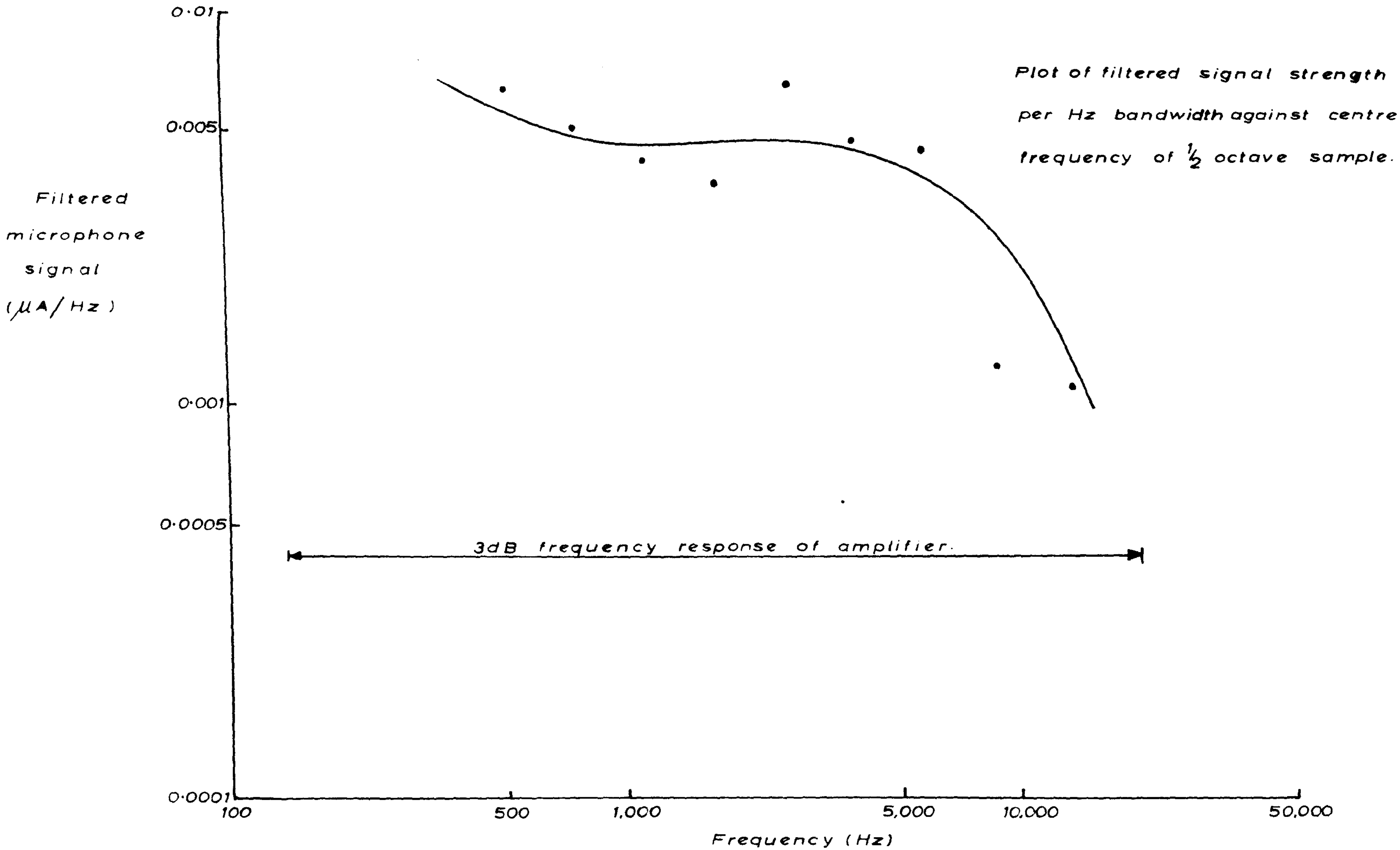


Fig 7.1.d. Frequency spectrum of inter-particle collision sound in laboratory channel.

outlet weir was then adjusted until the bed-surface and water-surface profiles were approximately parallel.

The establishment of equilibrium water-sediment conditions was not required for the experiments. Reading of the manometer of the sediment weighing device and recording of the microphone signal were therefore commenced simultaneously within several minutes of the setting of a steady water discharge.

The sediment discharge weighing manometer was read at intervals of one minute to the nearest 1/16 in., i.e. to the nearest ½ lb of dry gravel in the container. This enabled the average bed load discharge over a two minute period to be plotted against time during the experimental run. An example of the continuous record kept in this way is shown in fig. 7.2.a.

Examination of the continuous plot of sediment discharge with time during the experiment indicated the rate at which sediment should be injected into the upstream end of the channel to maintain a constant volume of sediment in the channel bed. The sediment feed device was then set and adjusted to the appropriate feed rate, as shown in fig. 7.2.a.

The paper speed on the ultra-violet recorder was set at the slowest speed of approximately 0.15 in/sec and the timing device adjusted to mark the paper at intervals of 10 sec. An example of the type of trace recorded during a run is shown in fig. 7.2.b.
At the end of each run the average microphone signal during intervals of one minute, calculated by averaging the instantaneous signal magnitudes at 6 second intervals, was plotted against time for comparison with the sediment discharge during the run (fig.7.2.a.)

Water temperature in the sump tank was measured at the start and finish of each run, and was generally found to rise about 1^{o}C during

Fig. 7.2.a.

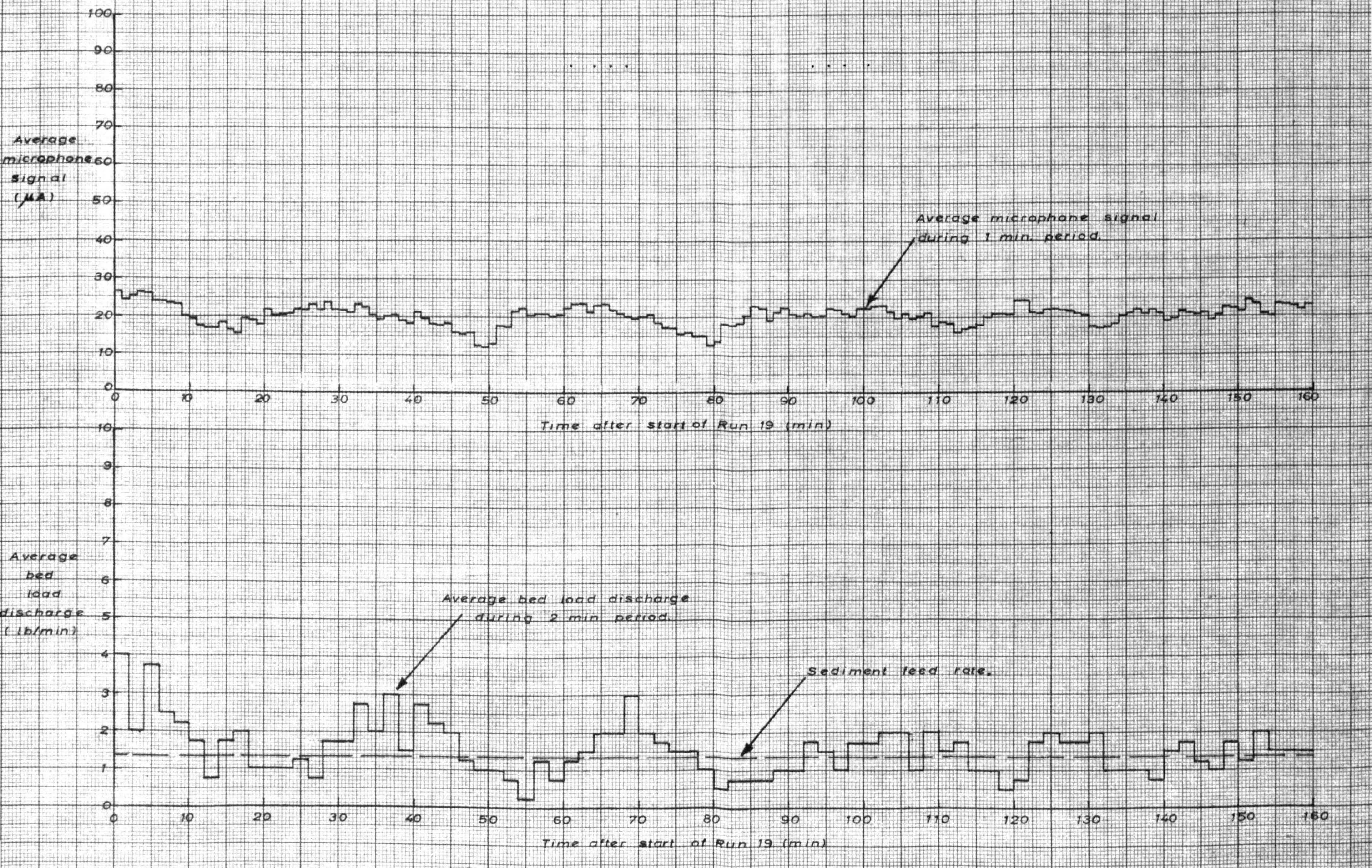


Fig. 72.a. Continuous record of microphone signal and bed load discharge in laboratory channel.

Fig. 7.2.b.

Paper speed ≃ 0·15 in/sec.

Timer marks at 10 sec intervals.

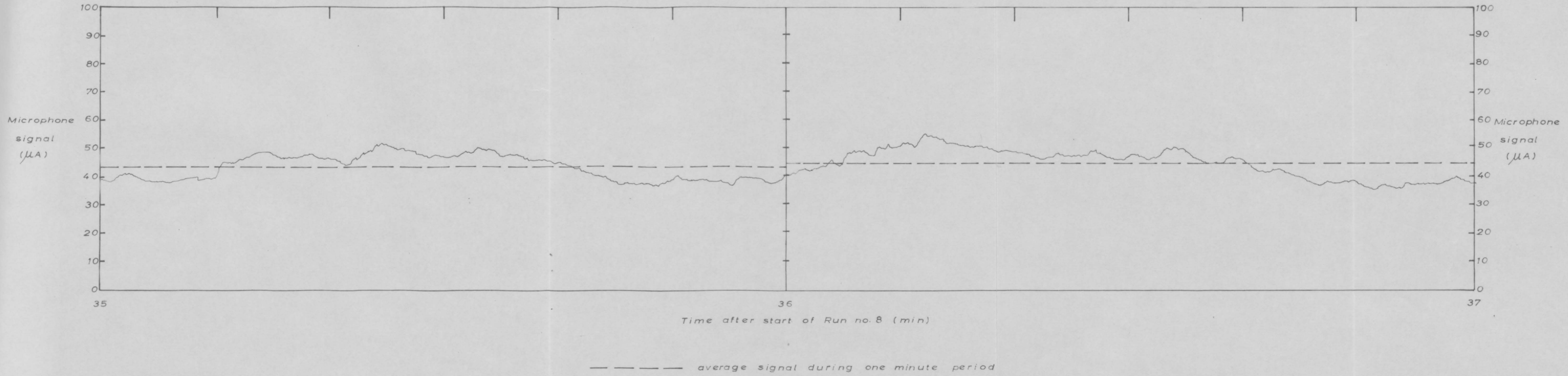


Fig. 7.2. b. Sample trace of microphone signal produced by inter-particle collision of sediment moving in laboratory channel.

a run. Over the whole series of experiments water temperature remained in the range 16.0-19.5°C.

Each experimental run was continued until the container of the sediment weighing device was filled; the total time required for a single run varied from 40 minutes to 4 hours depending upon the average sediment discharge. A total of 24 runs was conducted covering the full range of sediment discharges possible in the laboratory channel.

7.2.2. Time Lag between Microphone and Weighing Device

Since the microphone was located a distance 2 ft upstream of the end of the sediment bed it was evident that a certain time lag in the recording of sediment discharge should exist between the microphone and the sediment weighing device. In order to compare, or develop a relationship between the average microphone signal and the the average bed load discharge over a short period of time it was therefore necessary to determine the magnitude of this time lag.

The magnitude of the lag would be expected to be some function of the average bed load discharge. Hence, if a curve of sediment discharge against time lag could be established it would be possible to obtain the correct average microphone signal corresponding to an average bed load discharge by lagging the microphone record by the appropriate time interval. Consideration of fig. 7.2.a., which shows the variation with time of microphone signal and bed load discharge during run 19, suggested a possible means of obtaining the required relationship. It can be seen that both the microphone signal and the bed load discharge exhibit a cyclic pattern of variation, with the peaks and troughs of the latter lagging behind the former. (Similar unexplained periodic fluctuations of sediment discharge in laboratory channels were noted by Einstein (JOHNSON, 1938) and RATHBUN and GUY (1967)).

If $Q_B(i)$ is the average bed load discharge in the ith minute and $M_s(i-l)$ is the average microphone signal in the $(i-l)$th minute, where l minutes is the lag of the weighing device behind the microphone, then it is possible to calculate a cross-correlation, or product moment correlation coefficient for a lag of l min, r(l), defined as the ratio of the covariance of $Q_B(i)$ and $M_s(i-l)$ to the geometric mean of their variances.

Putting:-

$$Q_B(i) = x$$

$$\text{and } M_s(i-l) = y$$

then the cross-correlation coefficient can be calculated from:-

$$r(l) = \frac{\Sigma\, xy - \dfrac{(\Sigma\, x\, \Sigma y)}{n}}{\sqrt{\left[\Sigma x^2 - \dfrac{(\Sigma x)^2}{n}\right]\left[\Sigma\, y^2 - \dfrac{(\Sigma\, y)^2}{n}\right]}}$$

where Σ signifies summation over a selected range of i, and n is the number of one minute intervals in that range. (Since the cross-correlation coefficient is in fact identical to the correlation coefficient between two variables this technique implies a linear relationship between microphone signal and bed load discharge over the range of variation of each within their cyclic fluctuations).

In run 19 (fig. 7.2.a.), for example, a range of 100 min, comprising three complete cycles, from i = 21 to i = 120 was selected. The value of r(l) was calculated for 1 min increments of l from 0 to 12 min and plotted against l as shown in fig. 7.2.c. From this graph a maximum cross-correlation coefficient was taken to occur at a lag time of 6 min, corresponding to an average bed load discharge, Q_B, of 1.45 lb/min, during the selected time range.

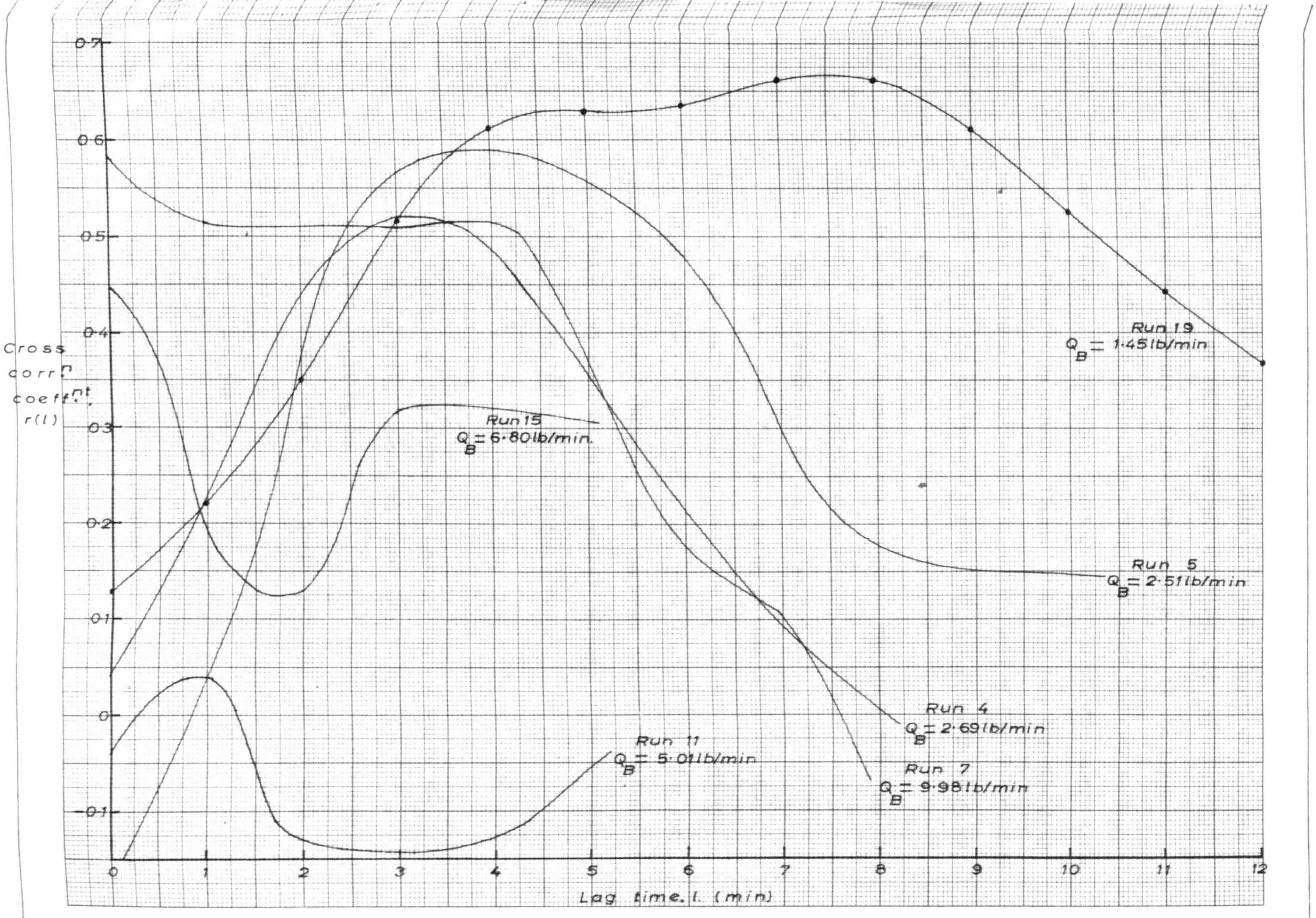


Fig. 7·2·c. Relationship between lag time and cross correlation of microphone signal and bed load discharge.

The analysis was repeated using five other experimental runs, covering a range of average bed load discharge, and the resulting curves of r(l) against l plotted in fig. 7.2.c. At the higher sediment discharges smaller time ranges had to be selected with less evident cyclic patterns; maximum cross-correlation coefficients were thus found to be somewhat lower. The time lags corresponding to the six maximum cross-correlation coefficients of fig. 7.2.c. were plotted against the appropriate average bed load discharge as shown in fig. 7.2.d.

In an attempt to confirm the results of the above technique an approximate theoretical relationship between the time lag and average bed load discharge was derived as follows.

The time lag, l min, was assumed to be made up of the time taken for a sediment particle to move from the microphone section to the end of the sediment bed, t min, plus the time taken for the particle to fall over the edge, down the slope into the container of the weighing device, and on to the sediment already in the container. This was estimated to be about 0.2 min i.e.

$$l = t + 0.2$$

$$\therefore t = l - 0.2$$

The distance of the microphone section from the end of the sediment bed was 2 ft. Hence the mean particle velocity,

$$\bar{U}_G = 2/(l - 0.2) \text{ ft/min}$$

The bed load discharge in dry weight per unit time (lb/min) was given by:-

$$Q_B = \gamma_s (1 - \lambda) wz \bar{U}g$$

where γ_s is the specific weight of the sediment particles

λ is the porosity of the sediment bed, i.e. the fraction of the volume not occupied by particles.

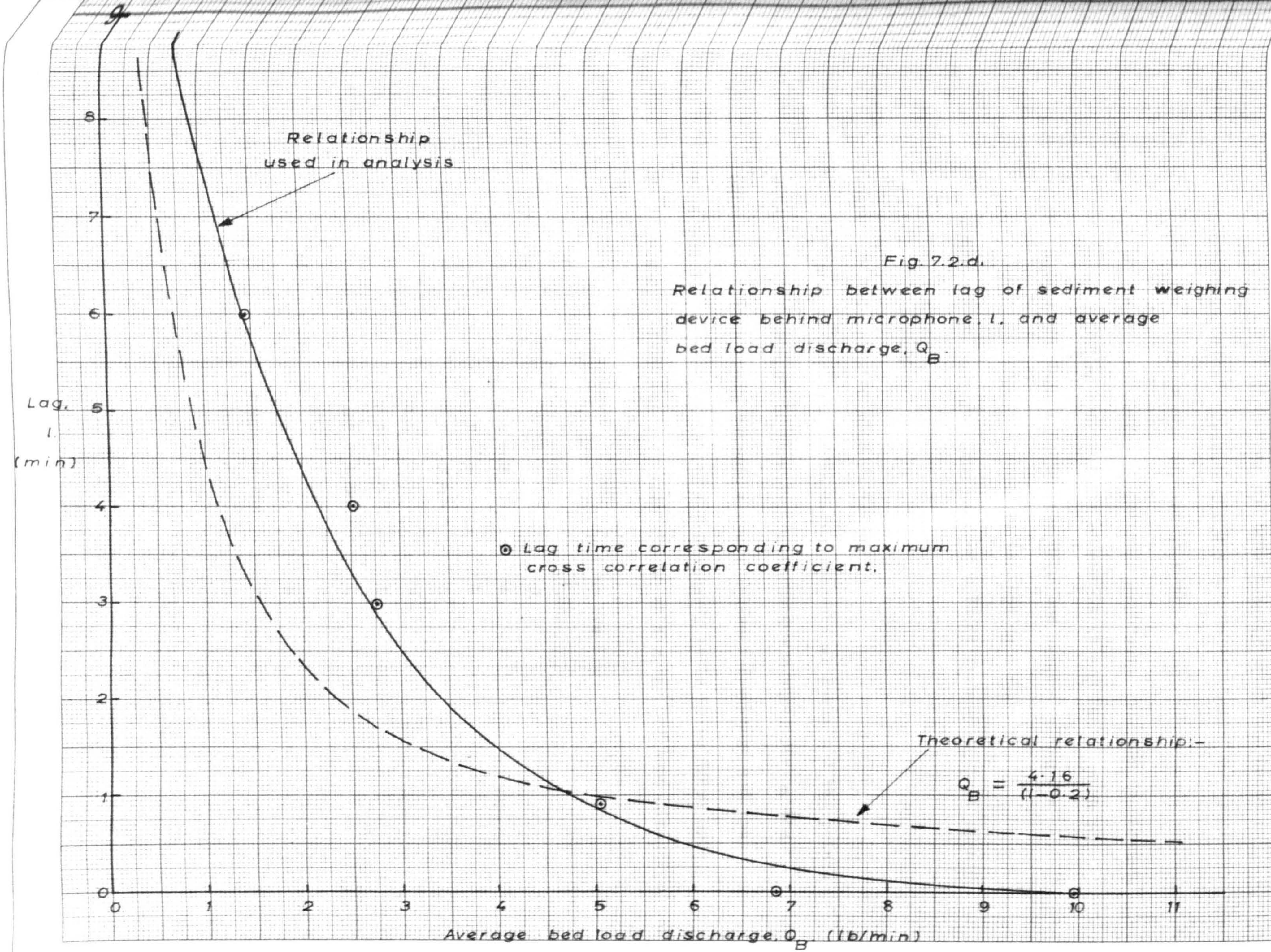


Fig. 7.2.d.
Relationship between lag of sediment weighing device behind microphone, l, and average bed load discharge, Q_B.

$\gamma_s(1 - \lambda)$ is therefore the bulk density which was found to be 92.5 lb/ft^3 (section 6.2.4.)

w is the width of the channel (1.5 ft)

z is the average depth of particle movement. At low discharges z may only be 2 or 3 grain diameters ($\frac{1}{2}$ - $\frac{3}{4}$ in.); at high discharges when a noticeable dune bed configuration was observed to occur z could be as high as 3 in.

Assuming an average value of z of 0.15 ft (nearly 1$\frac{3}{4}$ in.) then:-

$$Q_B = \frac{4.16}{(1 - 0.2)}$$

which has been plotted in fig. 7.2.d.

After consideration of both the theoretical curve and the points obtained using the cross-correlation technique a curve giving the estimated relationship between average sediment discharge and the time lag was assumed, enabling the following table to be drawn up:-

Table 7.2.a. Time lag corresponding to an average bed load discharge

Range of bed load discharge (lb/min)	Time lag (min)
$	0
4 - 6	1
3 - 4	2
2.5 - 3.0	3
2.0 - 2.5	4
1.5 - 2.0	5
1.25 - 1.50	6
1.00 - 1.25	7
0.75 - 1.00	8
0.50 - 0.75	9
0.25 - 0.50	10
0 - 0.25	11

7.2.3. Abstraction of Data from Experimental Results

Having established the time lag between the weighing device and the microphone for any given range of sediment discharge it was then possible to abstract from the continuous records obtained during the experimental runs the average bed load discharge over any selected length of time together with the corresponding average microphone signal over the same length of time lagged by the appropriate amount. At high sediment discharges the sediment weighing device could be filled in less than 40 minutes so that averages over periods of time greater than this could not be obtained. It was decided therefore to take average values of bed load discharge and microphone signal over lengths of time of 10, 20, 30 and 40 minutes.

The data obtained in this manner are plotted in figs. 7.2.e., 7.2.f., 7.2.g. and 7.2.h.

7.3. Theoretical Relationship between Microphone Signal and Bed Load Discharge

In order to fit a curve to the experimental data plotted in figs. 7.2.e. to 7.2.h. a tentative theoretical relationship was derived between the microphone output signal, M_s, and the bed load discharge in dry weight per unit time per unit width, q_B, of a sediment of single size, d, on a plane bed of infinite width.

Two basic assumptions concerning the mode of movement of the sediment particles as bed load were made:-

1) Each particle moves in a series of jumps or steps, the probability of its being moved from the bed being dependent upon the shape, size and weight of the particle and upon the flow conditions near the bed. The average distance travelled by the particle is a constant for any

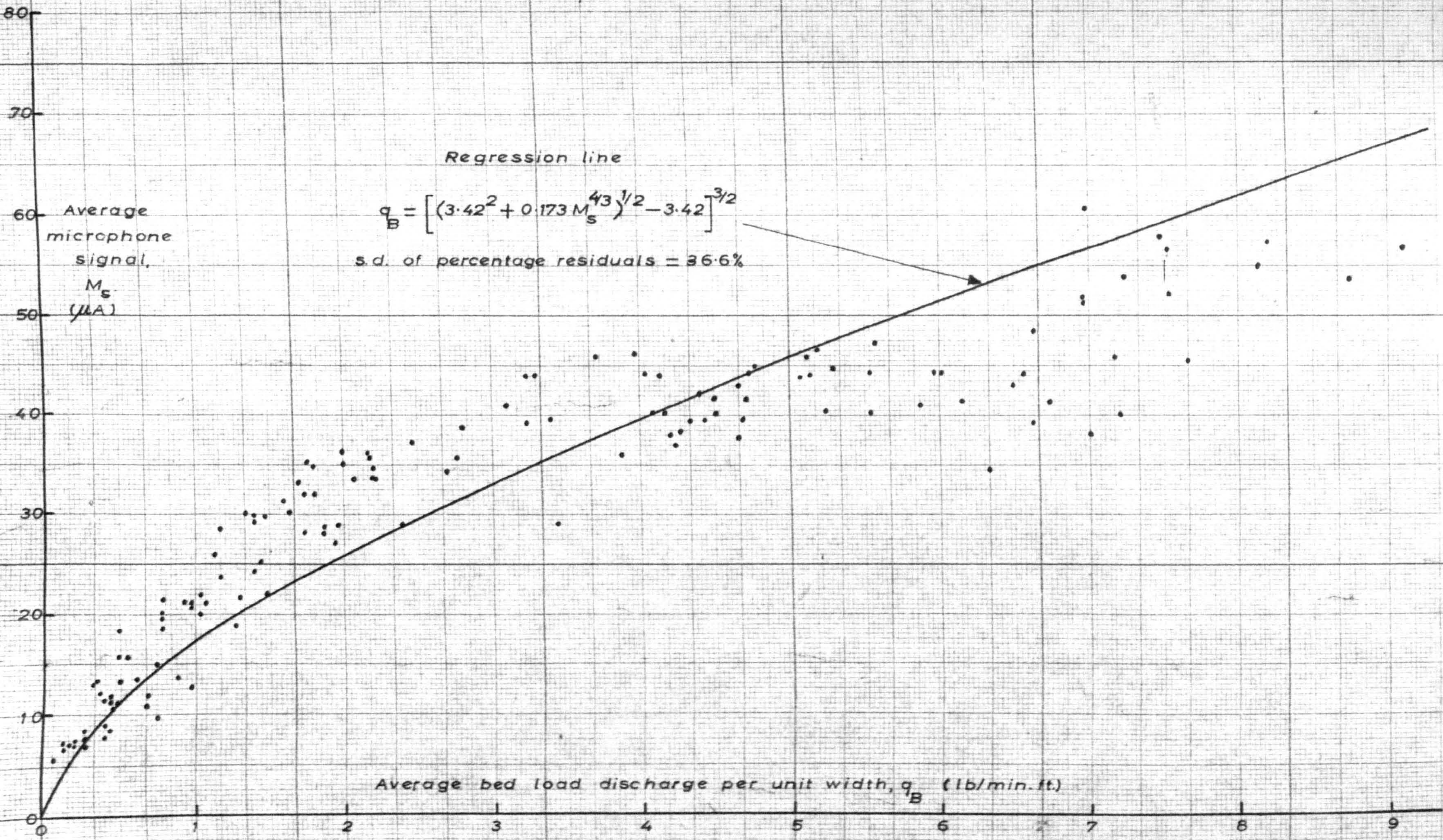


Fig.7.2.e. Relationship between average microphone signal and average bed load discharge over 10 minute period.

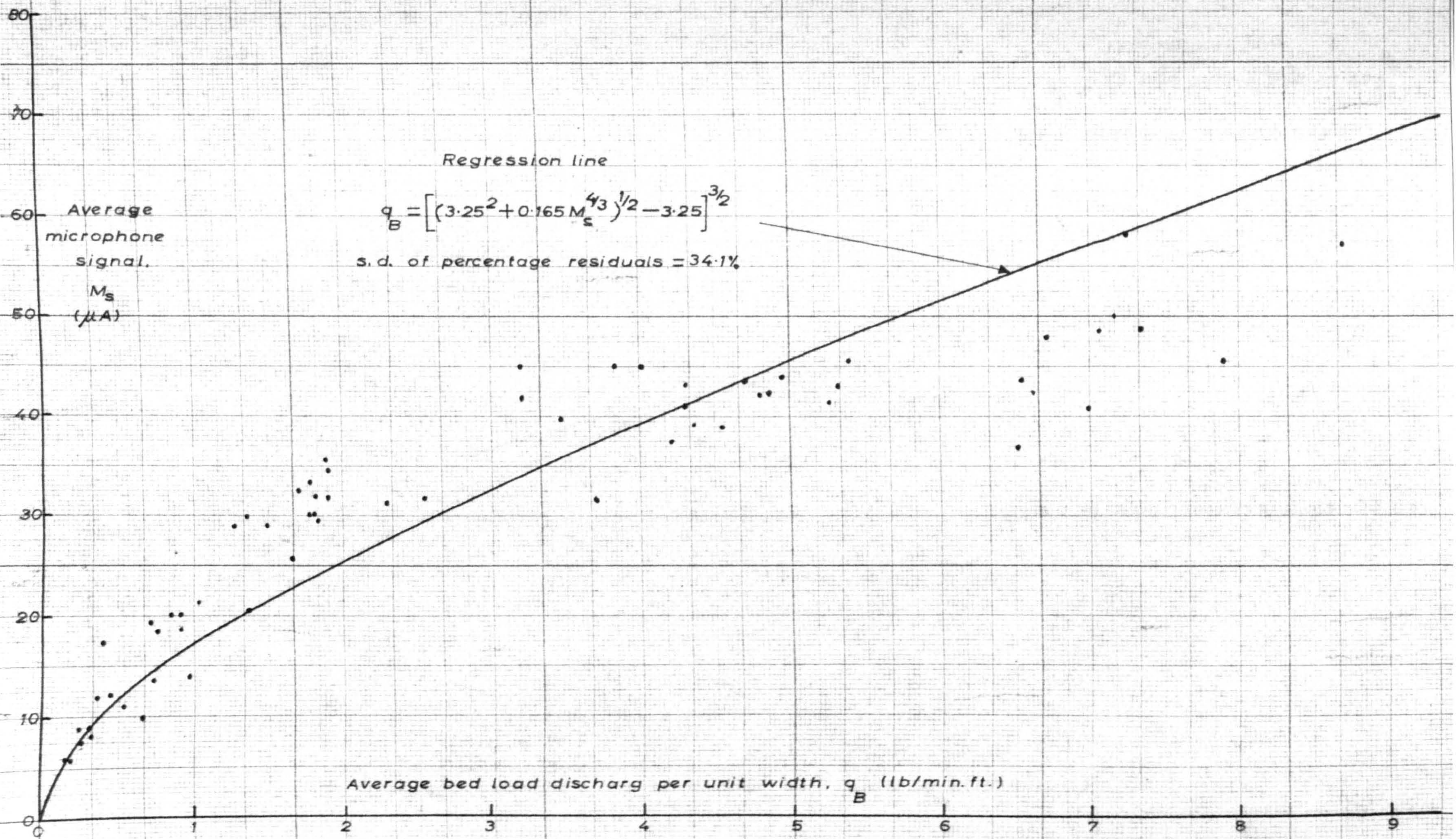


Fig. 7.2.f. Relationship between average microphone signal and average bed load discharge over 20 minute period.

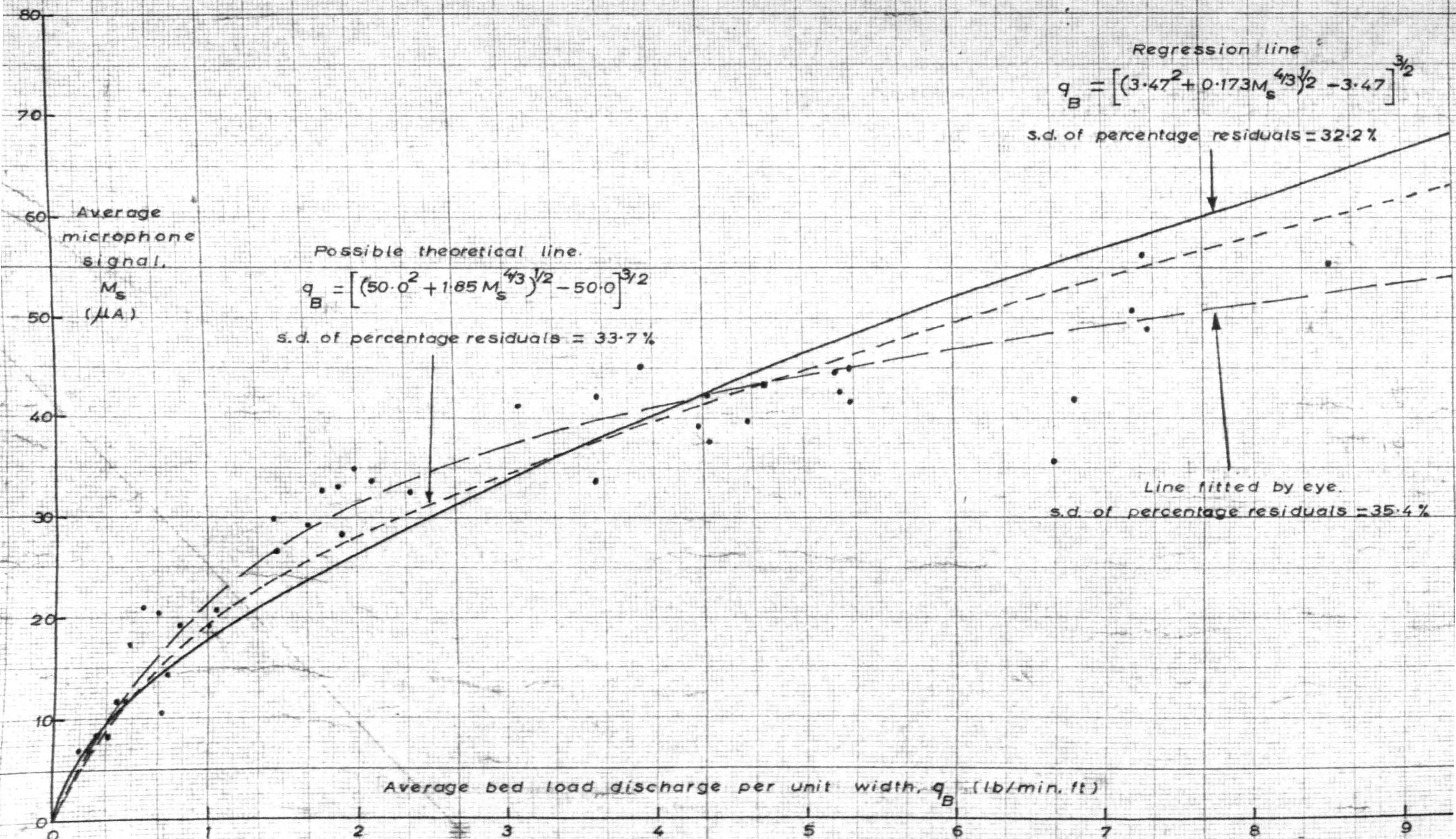


Fig. 7.2g. Relationship between average microphone signal and average bed load discharge over 30 minute period.

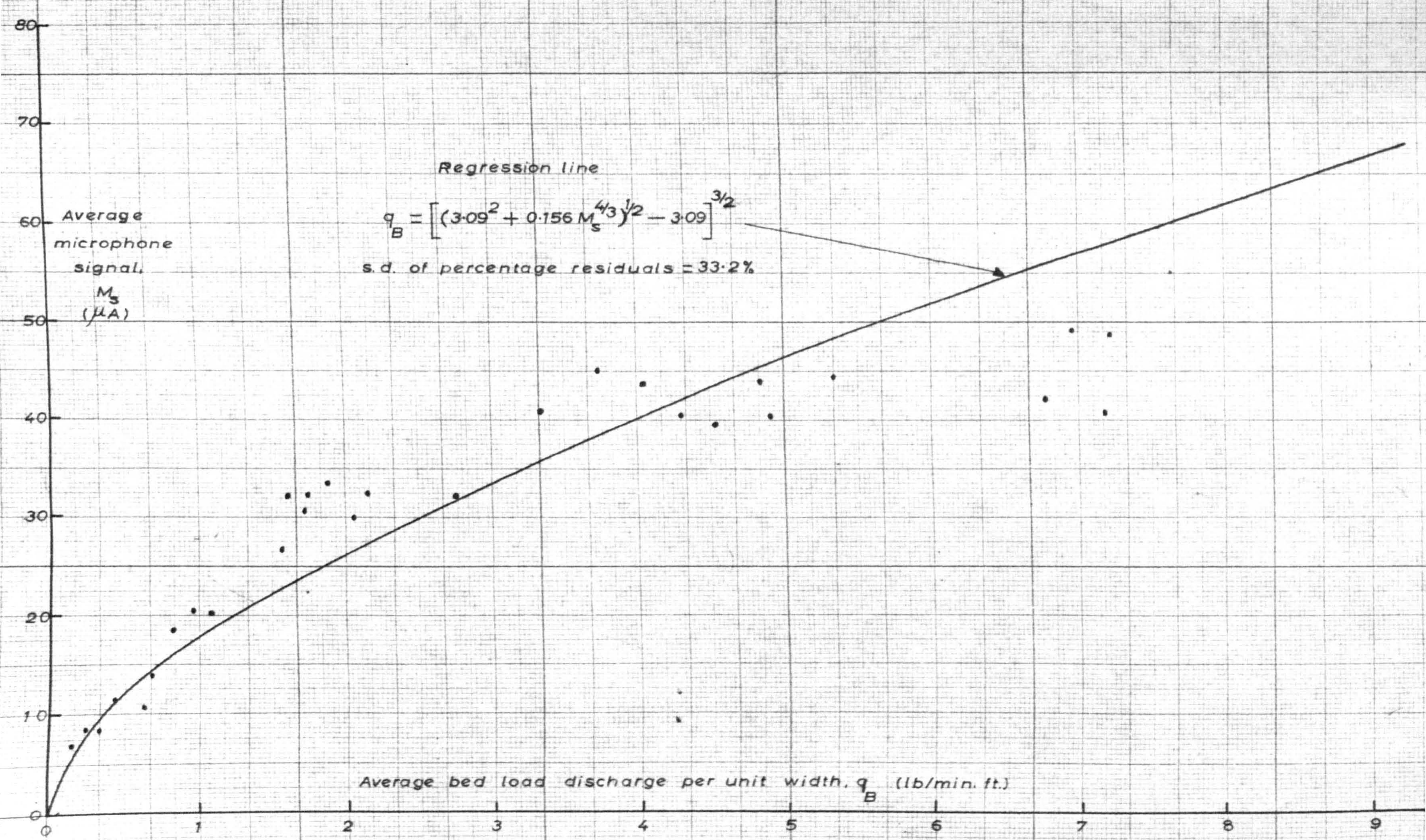


Fig. 7.2.h. Relationship between average microphone signal and average bed load discharge over 40 minute period.

particle and is independent of the flow conditions, the rate of transport and the bed composition, i.e. the average length of a particle step is equal to $k_1 d$, where k_1 is a dimensionless constant.

2) The average velocity of a particle during a step, $\bar{U}_s$, is equal to the mean flow velocity near the bed, $\bar{U}$.

After observation of the movement of sediment particles in a number of laboratory flumes EINSTEIN (1950) used both the above assumptions in the development of his bed load theory (section 4.1.9.).

At any particular cross-section, of unit width, in a bed load discharge of q_B all particles are performing a step length of $k_1 d$ and will therefore be deposited at some distance from zero to $k_1 d$ downstream of the section. The area of deposition is thus equal to $k_1 d$. The volume of each particle is $\pi d^3/6$ and its specific weight is γ_s. The number of particles deposited in unit area per unit time is therefore given by:-

$$\frac{q_B}{\frac{\pi}{6} d^3 \gamma_s k_1 d} = \frac{6 q_B}{\pi k_1 \gamma_s d^4}$$

According to the Hertz law of impact (GOLDSMITH, 1960), which is applicable to elastic bodies moving at low velocities, the ratio of the vibrational energy emitted by the collision of a moving sphere, velocity $\bar{U}_s$, with a stationary sphere to the initial kinetic energy of the moving sphere is equal to $\bar{U}_s/50c_s$, where c_s is the velocity of sound in the sphere. Assuming that this law can be applied to the impact, or deposition, of a particle on the bed, the vibrational energy emitted by a single deposition is given by:-

$$\left[\frac{\bar{U}_s}{50 c_s}\right] \frac{1}{2} \left[\frac{\frac{\pi}{6} \gamma_s d^3}{g}\right] \bar{U}_s^{\,2} = \left[\frac{\pi \gamma_s d^3}{600 g\, c_s}\right] \bar{U}_s^{\,3}$$

The average vibrational energy emitted per unit time per unit area by inter-particle collision of the bed particles is given by:-

E = number of impacts per unit area per unit time
x vibrational energy emitted by a single impact

$$= \left[\frac{6q_B}{\pi k_1 \gamma_s d^4}\right]\left[\frac{\pi \gamma_s d^3}{600g\ c_s}\bar{U}_s^{\,3}\right]$$

$$= \frac{q_B}{100k_1 g\ c_s d}\bar{U}_s^{\,3}$$

It has been assumed that the average velocity of a particle during a step, $\bar{U}_s$, is equal to the mean velocity of flow near the bed, $\bar{U}$. Boundary layer theory indicates the $\bar{U}$ is proportional to the shear velocity, u_*, (e.g. the Prandtl-Von Karman velocity distribution equation given in section 3.4.5.) an assumption made by KALINSKE (1947) in the development of his bed load theory (section 4.1.8.). Hence:-

$$\bar{U}_s = \bar{U} = k_2 \sqrt{\tau_o/\rho_f}$$

where τ_o is the fluid shear stress on the bed particles

ρ_f is the mass density of the transporting fluid

k_2 is a dimensionless constant

The vibrational energy emitted by the bed particles per unit area per unit time can then be expressed as:-

$$E = \frac{k_2^3\ q_B}{100k_1 g\ \rho_f^{3/2} c_s d}\ \tau_o^{3/2} \quad \text{. . . . . 7.3.a.}$$

MEYER-PETER and MÜLLER (1948) produced a bed load formula, borne out by experiment to be applicable principally to coarse material (section 4.1.7.), i.e. to sediment sizes which emit substantial inter-particle collision sound. For a plane bed of infinite width the formula can be written:-

$$\tau_o = 0.047(\gamma_s - \gamma_f)d + 0.25\left(\frac{\gamma_f}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma_f}{\gamma_f}\right)^{2/3} q_B^{2/3}$$

where γ_f is the specific weight of the transporting fluid.

Substituting for τ_o in equation 7.3.a.:-

$$E = \left[\frac{k_2^{3}}{100k_1 g\,\rho_f^{3/2} c_s}\right]\frac{q_B}{d}\left[0.047(\gamma_s - \gamma_f)d + 0.25\left(\frac{\gamma_f}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma_f}{\gamma_f}\right)^{2/3} q_B^{2/3}\right]^{3/2} \quad \ldots\ldots\ 7.3.b.$$

Since the number and rate of impacts on unit area of the bed is high the moving sediment particles can be assumed to approximate to a source of continuous, plane, sound waves with an energy flux given by the above equation 7.3.b.

Before reaching the piezoelectric crystal the energy of these sound waves must be transmitted from the sediment particles to the fluid and then through the fluid, a thin brass partition and the oil-filled microphone cavity. It can be shown (STEPHENS and BATE, 1950) that the ratio of transmitted acoustic energy to incident energy at the junction of two materials is $4Z_1Z_2/(Z_1 + Z_2)^2$, where Z_1 and Z_2 are the characteristic acoustic impedances of the materials, i.e. the product of density and velocity of sound in each material. The energy flux of the sound pressure waves received by the piezoelectric crystal is therefore equal to k_3E where the dimensionless constant k_3 is a function of the characteristic acoustic impedances of the sediment particles, fluid and materials of the microphone housing.

The energy flux of the sound pressure waves can be shown to be proportional to the square of the amplitude of the pressure fluctuations (RICHARDSON, 1953a). In the piezoelectric crystal the amplitude of the induced electrical potential difference is proportional to this pressure amplitude. The induced voltage or current is amplified linearly (fig. 6.3.f.) and the output measured as a current, M_s, by a continuous recorder. The acoustic energy flux received by the piezoelectric crystal is therefore proportional to the square of the amplified recorder output, i.e.

$$M_s^2 = k_4 k_3 E$$

where the constant k_4 is a function of the mechano-electrical characteristics of the piezoelectric crystal, amplifier and recorder output circuit

Substituting for E from equation 7.3.b. gives:-

$$M_s^2 = \left[\frac{k_4 k_3 k_2^3}{100 k_1 g \rho_f^{3/2} c_s}\right] \frac{q_B}{d} \left[0.047(\gamma_s - \gamma_f)d + 0.25\left(\frac{\gamma_f}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma_f}{\gamma_f}\right)^{2/3} q_B^{2/3}\right]^{3/2}$$

For a given microphone suspended over a bed of most natural sediment particles the above equation can be reduced to:-

$$M_s^2 = k_5 \frac{q_B}{d}\left[k_6 d + q_B^{2/3}\right]^{3/2}$$

Expressing q_B explicitly as a function of M_s and d:-

$$q_B = \left[\left(\frac{k_6^2}{4} d^2 + \frac{d^{2/3}}{k_5^{4/3}} M_s^{4/3}\right)^{\frac{1}{2}} - \frac{k_6}{2} d\right]^{3/2} \quad \text{. . . . . 7.3.c.}$$

For a given sediment size the relationship between q_B and M_s can be written:-

$$q_B = \left[\left(a^2 + b M_s^{4/3}\right)^{\frac{1}{2}} - a\right]^{3/2} \quad \text{. . . . . 7.3.d.}$$

in which the constants a and b are functions of:-

1. The ratio of average particle step length to particle diameter
2. The ratio of mean flow velocity near the bed to shear velocity

3. The velocity of sound in, and the size and specific weight of, the sediment particles
4. The velocity of sound in, and the specific weight of, the transporting fluid
5. The characteristic acoustic impedances of the materials of the microphone housing
6. The mechano-electrical characteristics of the piezoelectric crystal, amplifier and recorder output circuit.

7.4. Discussion of Observed and Theoretical Relationships

Determination of the theoretical values of the constants a and b in the microphone equation 7.3.d. would have required a number of hydraulic and acoustic experiments for which neither suitable apparatus nor time was available. It was possible, however, to calculate the best values of a and b in the equation such that curves could be drawn through the points plotted in figs. 7.2.e. to 7.2.h. for the prediction of average bed load discharge, q_B, from a knowledge of the average microphone signal, M_s.

Evaluation of the constants a and b was effected by a least squares curvilinear regression of q_B, the dependent variable, on M_s, the independent variable. Partial differentiation with respect to a and b of the weighted residuals given by equation 7.3.d. yielded simultaneous normal equations which were not directly soluble for a and b. The regression had to be carried out by iterative procedure for which a computer programme was developed for use on the KDF 9 computer of the University of Newcastle upon Tyne. A description of the method of regression analysis, a copy of the computer programme and samples of the input and output data are given in the Appendix. The analysis was carried out on the data from each of the graphs of the 10, 20, 30 and 40 minute averages.

From the computer output data it was possible to obtain the values of a and b giving the least sum of squares of percentage residuals and the value of the standard deviation of the percentage residuals. These are given in table 7.4.a.

Table 7.4.a. Results of curvilinear regression analysis

Period of averages (min)	Number of points	a	b	s.d of percentage residuals
10	141	3.42	0.173	36.6 %
20	63	3.24	0.165	34.1 %
30	42	3.47	0.173	32.2 %
40	29	3.09	0.156	33.2 %

The curves obtained using the regressed values of a and b in equation 7.3.d. are given in the appropriate graphs of figs. 7.2.e. to 7.2.h. A measure of the accuracy of the prediction of bed load discharge from the microphone signal using these curves can be given by the 65% confidence lines, approximately equal to $\pm$ 1 s.d. of the percentage residuals.

It can be seen that the regression curves, while giving the least sum of squares of percentage residuals (and hence the least s.d. of the percentage errors of prediction) do not appear to be a good fit of the plotted points, consistently overestimating bed load discharge in one part of the graph and underestimating in the other. It was possible to select values of a and b in equation 7.3.d. giving curves which appeared to fit the data better. For example, with the 30 minute averages of fig. 7.2.g. putting a = 50 and b = 1.85 yielded curve 2, which gives a s.d. of percentage residuals of 33.7%, slightly greater than that of the regression curve. Curve 3 of fig. 7.2.g. was drawn through the plotted points by eye to no particular mathematical function and, although seeming to be an even better fit than curves 1 and 2, gives a greater s.d. of percentage residuals of 35.4%.

Attempts were made to determine whether the plotted data fitted more closely to a power relationship of the type:-

$$q_B = a\ M_s^{\ b}$$

or a polynomial such as:-

$$q_B = a\ M_s + b\ M_s^{\ 2}$$

Both regressions were carried out using a modification of the iterative regression programme (see Appendix) and produced curves similar to the regressed theoretical equation, but with larger s.ds of the percentage residuals. The value of the exponent b in the power relationship was calculated to be 1.72; a plot of q_B against M_s on logarithmically scaled graph paper appeared to fit well to a straight line with a slope of 2. This would seem to suggest that the bed load discharge might be correlated well with the square of the microphone output signal, i.e. with power output of the amplifier. Such a technique, involving continuous recording of the amplified a.c. output on a milliwatt-meter, would dispense with the need for a rectifier and arbitrarily selected time constant in the output smoothing circuit. It might also result in an increase in the possible accuracy of prediction.

Consideration of the curves of fig. 7.2.g. indicated that the actual relationship between bed load discharge and microphone signal appeared to deviate from the theoretical relationship derived in section 7.3. This could be due to the neglect in the theory of certain physical conditions in the laboratory channel and also the assumption of some "constants" which in reality vary with bed load discharge. The major difference between the conditions of the theory and those in the laboratory is the constricted width of the channel bed. In the discussion on the influence of the channel sides in section 7.1.3., however, it was concluded that, due to reflection from the sides, the channel might still approximate to an infinitely wide bed. Nevertheless, this side effect and reflection from the water surface must be expected to cause some discrepancy. The ratio of particle step length to particle diameter, k_1, was assumed in the derivation of the theory not to vary with bed load discharge. In fact, it is possible that at higher sediment discharges the average particle step length becomes somewhat greater; since the microphone signal, i.e. inter-particle collision sound, depends upon the actual deposition or impact of particles on the bed then an increase in bed load discharge would result in a proportionately smaller increase in microphone signal. A further result of high sediment discharges was observed to be the formation of a noticeable bed configuration, consisting of intermittently appearing dunes of approximate wavelength 4 ft and crest to trough amplitude of up to nearly 5 in. The destruction of the plane bed (together with the existence of side friction) would require the inclusion on the left hand side of the Meyer-Peter and Müller formula of a factor of less than unity, giving in equation 7.3.b. a smaller energy output from the bed, i.e. a smaller microphone signal for a given bed load discharge. The probable increase in k_1 and the production of a bed configuration both provide possible explanation for the lower microphone signals observed at high level bed load discharges.

Due to the above effects at high sediment discharges the microphone signal becomes relatively insensitive to large increases in bed load discharge above approximately 3 lb/min.ft.width. This demonstrates one of the major disadvantages of the present acoustic technique since it then becomes difficult to make accurate predictions of bed load discharge from a knowledge of the microphone signal.

Some of the scatter evident in the experimental observations plotted in figs. 7.2.e. to 7.2.h. can probably be attributed to the fact that the two methods of measurement, the microphone and the weighing device, did not refer to identical physical phenomena. While the microphone signal was dependent upon the general conditions of movement, or, more precisely, deposition within a length of channel of about 5 ft (section 7.1.2.), the weighing device recorded the variation with time of the bed load discharge at one particular cross-section in that length. The mode of movement of the bed particles within that length was by no means uniform. At low flows, areas of the bed surface were observed to move in "gusts" in a random manner; at high flows a distinct intermittent dune movement of the bed became evident. Since such spatial distribution of movement occurs within a length of channel then it would be possible that a bed load discharge at one particular cross-section could be associated with different microphone signals from that length. However, since the passage of a dune past a cross-section was rarely observed to take more than 4 to 5 minutes, scatter due to this effect should be minimised by taking average values over longer periods.

Scatter can also be attributed to the fact that the plotted points are the averages of instantaneous values which have varied over a considerably wide range. This refers not to the long term periodic fluctuations obtained in some experimental runs e.g. run 19, fig. 7.2.a (table 7.4.a. shows that an increase in the time over which the averages are taken does not reduce scatter to a significant extent), but to the irregular variations superimposed upon these fluctuations. The average microphone signal and the average bed load discharge over a period of time can therefore lie within a relatively wide range of each variable, resulting in a statistical scatter of the observations, especially in the less sensitive range of the curve.

Since the microphone signal is dependent upon the rate of deposition of particles on the bed it was considered possible that accretion, essentially a process of particle deposition, might produce a greater microphone signal than scour, a process of entrainment, for the same instantaneous bed load discharge. That is, less sound would be emitted during a rising (or scouring) bed load discharge than during a falling (accreting) discharge. Possibly also, experimental points lying above the mean curve might represent predominantly accreting periods of time (a greater M_s for a given q_B) and similarly those below might represent predominantly scouring periods. An attempt was made to confirm this hypothesis by describing each point as scour or accretion depending upon whether, for more than half the time, the instantaneous bed load discharge was greater or less, respectively, than the average bed load discharge over that period. No conclusive evidence, however, could be found in the distribution of the scour and accretion points. On a smaller time scale attempts were made to determine whether, during the movement of well-formed dunes past the microphone, the occurrence of deposition on the downstream face of a dune immediately below the microphone resulted in an increase in the recorded signal. No evidence of this effect was observed either, probably because the signal was influenced to a large extent by particle movement in other parts of the channel.

It is still possible, however, that while some of the scatter of the experimental observations may be statistical in nature a further parameter, perhaps involving scour or accretion, may be required to completely describe each observation.

7.5. Conclusions

Consideration of the results of the preliminary laboratory experiments enabled the following conclusions to be drawn:-

1. To avoid disturbance of the velocity of flow near the bed the microphone should be fixed at a height of 3 in. above the plane sediment bed.

2. Since the moving bed particles approximated closely to a source of plane sound pressure waves small variations in the height of the microphone above the bed have little effect on the strength of the microphone signal.

3. The microphone was able to detect inter-particle collision at distances of up to 2½ to 3 feet; it was therefore sensitive to the general conditions of sediment movement in a length of channel of about 5 ft. Since the width of the sediment channel was only 1½ ft the microphone signal would most likely be influenced by the existence of the channel sides. However, by permitting reflection of sound from the glass sides, the conditions were probably maintained more like those in an infinitely wide channel.

5. The sound emitted by the inter-particle collision of the uniform-sized sediment extended over most of the audio-frequency range, with no characteristic peak frequency.

It was decided to sample all frequencies within the amplifier cut-off range (140–20,000 H_z) throughout the remainder of the experiments.

A total of 24 experimental runs were conducted with a constant water discharge and continuous recording of bed load discharge and microphone signal. From the results of these experiments it was possible to obtain graphs of average bed load discharge, q_B, against average microphone signal, M_s, over periods of 10, 20, 30 and 40 minutes.

An approximate theoretical derivation indicated that the form of the relationship between q_B and M_s should be:-

$$q_B = \left[\left(a^2 + bM_s^{4/3}\right)^{\frac{1}{2}} - a\right]^{3/2}$$

where a and b are undetermined constants depending upon the physical, acoustic and hydraulic properties of the sediment, water and microphone. Curvilinear regression analysis produced curves of the theoretical function giving the least squares residuals (for the 30 min. period, 65% confidence limits of $\pm$ 32.2%), but not appearing to be a good fit to the plotted data. An apparently closer fit was obtained by selecting different values of a and b in the theoretical equation, but resulted in 65% confidence limits of $\pm$35.4%. No improvement in curve-fitting was obtained by regression analysis according to power or polynomial relationships.

Deviation of the observed relationship from the theory was probably due to:-

1. The influence of the restricted channel width.
2. An increase in particle step length with increasing bed load discharge.

3. The formation of a distinct bed configuration at high bed load discharges.

The latter two effects cause a relatively small increase in microphone signal for a large increase in bed load discharge. The major disadvantage of the present technique is the resulting difficulty in making accurate predictions of bed load discharge from a knowledge of the microphone signal.

Scatter of the observed data was probably due to the following:-

1. The two measurements did not refer to identical physical phenomenon. The microphone responded to the general conditions of movement within a length of channel, while the sediment weighing device recorded the bed load discharge at one particular cross-section in that length.

2. Each point was the average of a number of instantaneous values which varied over a wide range. Over a period of time the average microphone signal and average bed load discharge could therefore lie within a relatively wide range of each variable, resulting in a statistical scatter of the observations, especially in the less sensitive range of the curve.

3. The microphone signal was dependent upon the rate of deposition, not entrainment, of particles on the bed of the channel. It is possible that a further parameter, probably involving accretion (deposition) or scour (entrainment) is necessary to account for the observed scatter.

The results of these laboratory studies of the acoustic detection of bed load movement indicate that with the present technique measurement of bed load discharge in laboratory channels can be made with 65% confidence limits of about $\pm 35\%$. Possible improvement in the accuracy could be obtained by either recording the rate of occurrence of inter-particle collisions on an impulse rate meter or by continuously recording the a.c. power output of the microphone amplifier. Any further laboratory studies should be carried out in a wide channel to eliminate the influence of the channel sides, or the microphone should be made directional, thereby restricting the area of bed to which it is sensitive.

For use with different sediments the microphone would have to be calibrated for the particle size of each sediment. It is possible that the theoretical equation 7.3.c., which includes particle size in the empirical constants, could be used to extrapolate to slightly different sizes.

Section 8

8. Acoustic Detection of Bed Load Movement in Rivers

Sections 6 and 7 have described the experiments carried out on the acoustic detection of bed load movement in the controlled conditions of the laboratory sediment channel. Contemporaneous with these investigations an instrument was being designed for the detection of inter-particle collision sound when suspended above the bed of the River Tyne from the Bywell cableway. With this instrument it was intended to investigate the temporal and spatial distributions of sediment noise at the cableway section and to compare relative noise intensities at various river stages with the estimates of bed load discharge computed in section 4. A review of the available information on previous work in this field of bed load discharge measurement is first given, followed by a description of the design and testing of the microphone used on the River Tyne. Opportunities for testing preliminary designs of the instrument were, however relatively few and little progress could be made in extending the acoustic technique from the laboratory to the river.

8.1. Previous Investigations

Microphones for the detection of underwater sounds have been used for many years for a variety of purposes; detailed designs of several hydrophones, as they are called, can be found in standard text books on sound (RICHARDSON, 1953). Their principal applications are usually to be found in marine and coastal investigations. General studies of ambient noise propagated through the oceans and seas have been carried out by many countries throughout the world (WENZ, 1962), while more specific applications are common in the fields of oceanography, geophysics, navigation (STEPHENS and BATE, 1950), zoology (the Department of Zoology of the University of Newcastle upon Tyne utilised a specially constructed hydrophone during a study of the behaviour patterns of lobsters off the North-east coast of England), ultrasonics (RICHARDSON, 1957) and, recently, fisheries (TUCKER, 1967).

The detection of sediment movement in rivers by acoustic methods is not a recent development. According to LABAYE (1948) Muhlhofer, as early as 1931, was able to listen to the sound of gravel movement in the River Inn in Austria by placing a box containing a microphone on the bed. Soon after, in 1936, Reitz used some kind of microphone placed just below the water surface to record the sound of sediment disturbed by the formation and passage of large-scale boils (HUBBELL, 1964). In 1942 a hydrophonic detector was constructed at Grenoble, France and later modified by BRADEAU (1951) at the Service des Etudes et Recherches Hydrauliques, d'Electricitie de France. The instrument consisted of a flat plate installed on the stream bed and provided with a microphone to pick up the sound of gravel and coarse sand sliding over, or colliding with, the plate. HUBBELL (1964) described an instrument developed by Juniet in 1952 called l'Arenaphone, which consisted of a fork-shaped rod attached to a transducer. The assembly was supported on a tripod such that the rod was inserted a short distance into the river bed. Vibrations caused by sediment particles impinging upon the forked rod were amplified and transmitted to headphones or tape recorder.

CARLSON and MILLER (1956), and KAROLYI (1957), have stressed the need for a continuous method of recording bed load discharge, such as the acoustic technique, which could be used to determine the commencement and cessation of sediment movement, the effective width of movement, and the temporal and spatial distribution of movement. Such an instrument could also be used to determine the number and location of sampling verticals to be used with trap-type bed load samples. More recently, BEDEUS and IVICSICS (1963) in Hungary made further progress with the acoustic technique by developing a microphone which could be suspended at some height over the river bed. By virtue of its position the instrument did not, therefore, influence the bed load movement and recorded only the sound emitted by inter-particle collision. This sound was carried from the large crystal microphone, housed in a weighted, streamlined body, through the suspension cable to a small boat where it was amplified and its intensity registered on an ammeter.

It was learned (by personal communication) that some research was carried out on this topic by Türk at the Technische Hochschule, Karlsruhe, Germany, but no published information is available. The author has also learned recently (December, 1967) that Plessey Electronics Ltd., Marine Systems Division, are conducting investigations into the acoustic detection of underwater sediment. However, their work is at present concerned solely with the detection of fine particles in suspension.

Although it is not directly associated with the present research, it is interesting, perhaps, to note the phenomenon of "singing" observed to occur with certain natural seashore sands. Experiments have been carried out in the University of Newcastle upon Tyne by BROWN, CAMPBELL, ROBSON and THOMAS (1963) on the high frequency sounds emitted by these sands when subjected to the sudden impact of a large weight, e.g. the impact of the human foot when walking. A similar phenomenon, known as "the booming sands of the Kalahari", was also studied by BAGNOLD (1966).

8.2. Development of an Acoustic Bed Load Detector for use at Bywell Cableway Gauging Station

As mentioned earlier, opportunities for testing preliminary designs of a bed load detector under conditions of sediment movement at the Bywell cableway gauging station occurred relatively infrequently. It can be seen from calculations made in section 4 of this thesis (e.g. fig. 4.1.m) that bed load movement is unlikely to occur at river stages below 54 ft. A.O.D. (7.75 ft. above staff gauge zero) i.e. at discharges below about 12,000 cusec. According to the flood frequency curve given by HALL (1964) this flow is exceeded on average only six times per year. Further examination of discharge records over the ten year period 1956/66 shows that almost three-quarters of the flood discharges greater than 12,000 cusec occurred during the hours of darkness. For practical reasons, and in the interest of the safety of both equipment and personnel, work at Bywell had to be restricted to daylight hours.

Thus, in the time available for this research, only limited field experience with the acoustic detector could be gained, and it proved impossible to establish a final, tested design. An account of the various stages of development of a suitable instrument is given first, followed by a description of the latest design of the microphone and associated electrical recording equipment.

8.2.1. Design and Development of the Bed Load Detector

Ideally, the application of the laboratory acoustic technique to the river should involve a minimum of change in the conditions of operation of the instrument. It would seem, therefore, that the microphone should be located at a fixed height above the sediment bed at some point in the cableway cross-section. The installation and maintenance of the microphone, with some form of support frame, in the river bed would have required personnel skilled in underwater operations; such help was not at the time available. Moreover, it was intended to investigate not only temporal, but spatial, variations of bed load movement in the cross-section.

Basically, therefore, the acoustic detector was to consist of a microphone which would be suspended above the river bed at any point in the cross-section and which would transmit the sound emitted by inter-particle collision through the suspension cable to a recorder on the bank. The arrangement was similar to that in the laboratory, but with several unavoidable differences. The absence of a rigid suspension of the detector introduced problems of stability, especially in the considerably greater flow velocities occurring in the river (velocities of up to 13 ft/sec have been recorded at Bywell). The existence of a long length of coaxial suspension cable, the electrical resistance of which varied with the distance of the detector across the section, necessitated that the amplifier be located close to the microphone; in this way it was possible to minimise loss of signal and acquisition of extraneous noise during the transmission of the sound to the recorder. Since mains supply of electricity was not available at the cableway the microphone amplifier had to be battery-powered; similarly the recording instrument had to be either mechanically or battery-operated.

It was decided to house a small crystal microphone in an oil-filled cavity in the underside of a streamlined metal body; the 60 lb weight used by HALL (1964) to stabilise a suspended sediment sampler in high flows was considered suitable. A three-stage, transistorised, battery-powered amplifier was contained inside a watertight brass cylinder attached to a vertical hanger bar above the streamlined body. The cylinder was enclosed by a sheet brass faring ro reduce resistance to flow. To minimise excessive drainage of the batteries a mercury switch was installed in the cylinder such that the amplifer was powered only when the instrument was in an upright position. The amplified signal was transmitted through the coaxial suspension cable either to head-phones or to a portable Everett Edgcumbe moving-coil recorder, with mechanically operated chart drive (full scale deflection 1mA, paper speed 1 in/min). The instrument at this early design stage is shown in fig. 8.2.a.

The instrument was tested in moderately high flows at the cableway section and several improvements made to the streamlining of the instrument. As the results of laboratory investigations became available further modifications were also carried out on the electrical equipment. Later, a battery-powered, portable, potentiometric voltmeter, an improvement on the old moving-coil recorder, became available, and provision was also made for inclusion of a frequency filter, if required.

No suitably large flows for testing the instrument occurred at Bywell for a period of over ten months, until continuous, heavy rainfall on both the North Tyne and South Tyne catchments produced the highest flood in the lower reaches of the River Tyne since 1954. Peak stage at Bywell reached just over 18 ft above staff gauge zero (64.25 ft A.O.D.), corresponding to an estimated peak discharge of about 68,000 cusec. Attempts to immerse the bed load detector at this discharge resulted in the instrument being thrown completely clear of the water; the stability problem had obviously been greatly underestimated. Even at a stage of 14.5 ft (44,000 cusec),

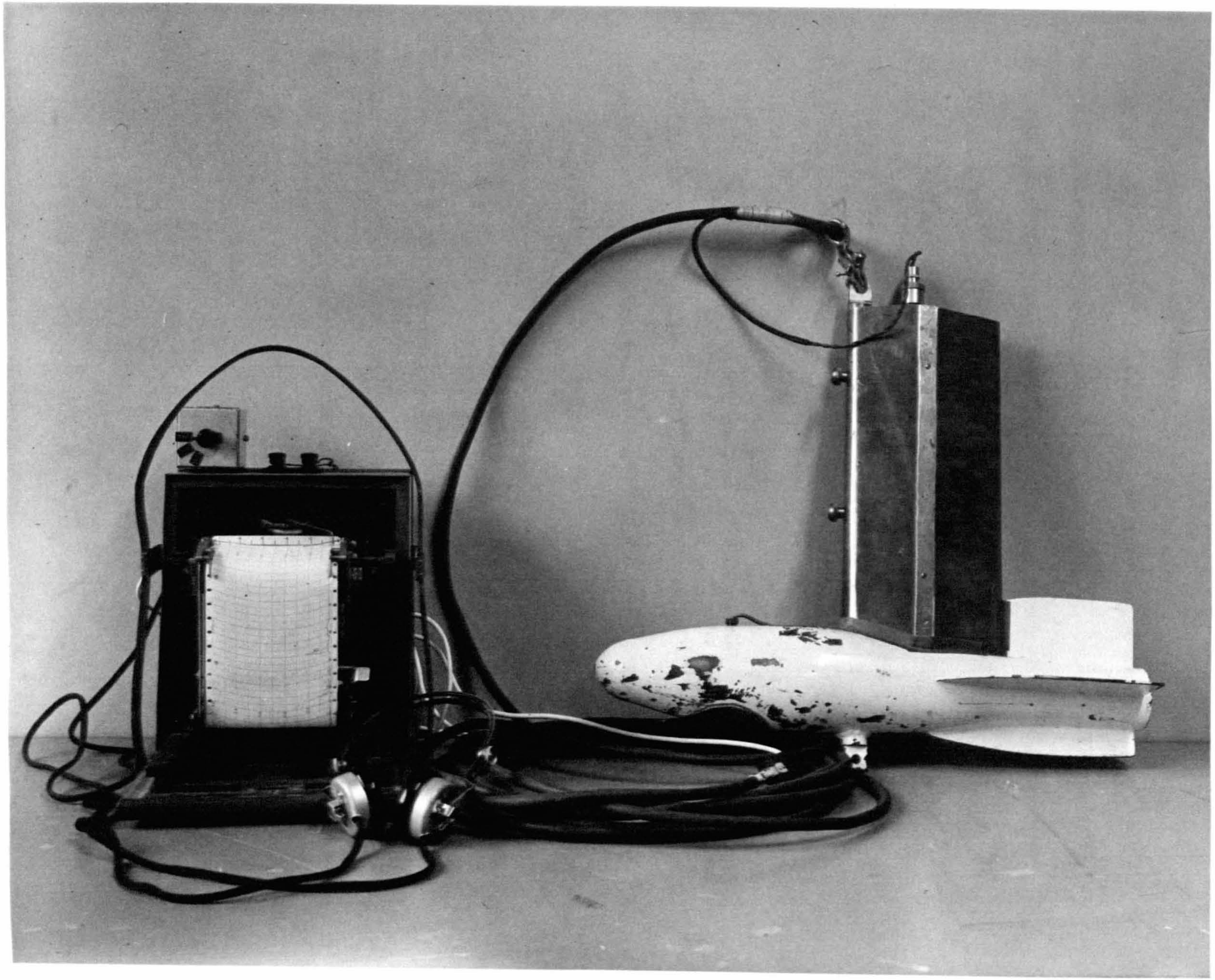


Fig. 8.2.a. Early design of acoustic bed load detector.

when it became possible to maintain the instrument below the water-surface, the mercury switch was observed to open and close rapidly, indicating that correct location and orientation of the detector could still not be ensured.

In the light of this experience the instrument was modified considerably and several further improvements made. Unfortunately, no more suitably high flows have occurred since the latest design, shown in fig. 8.2.b., was completed and further testing of the detector has not been possible. A complete description of the microphone and recording equipment at the present stage of development is given below.

8.2.2. Bed Load Detector

A diagram of the latest design of the acoustic bed load detector is given in fig. 8.2.c. The detecting element is identical to that used in the laboratory microphone, i.e. a modified lead zirconate titanate polycrystalline ceramic tube (PZT 4, plated and polarised), manufactured by Brush Clevite Ltd., Hythe, Southampton. It is made up as shown in fig. 6.3.a. and housed in a 2½ in. diameter cavity in the underside of a streamlined mild steel weight. The cavity is filled with castor oil (of characteristic acoustic impedance similar to that of water, thereby minimising sound energy transmission losses) and sealed with a sheet of 1/16 in thick polythene secured in position by a thin brass ring screwed into the metal weight. The underside of the streamlined body is built up with araldite to eliminate noise generated by the flow of water past protuberances near the microphone cavity.

The signal from the microphone is carried by a twin-core coaxial cable to the amplifier which is housed in a 10 in. long, 2 in. internal diameter, length of pipe welded to the rear of the streamlined body.

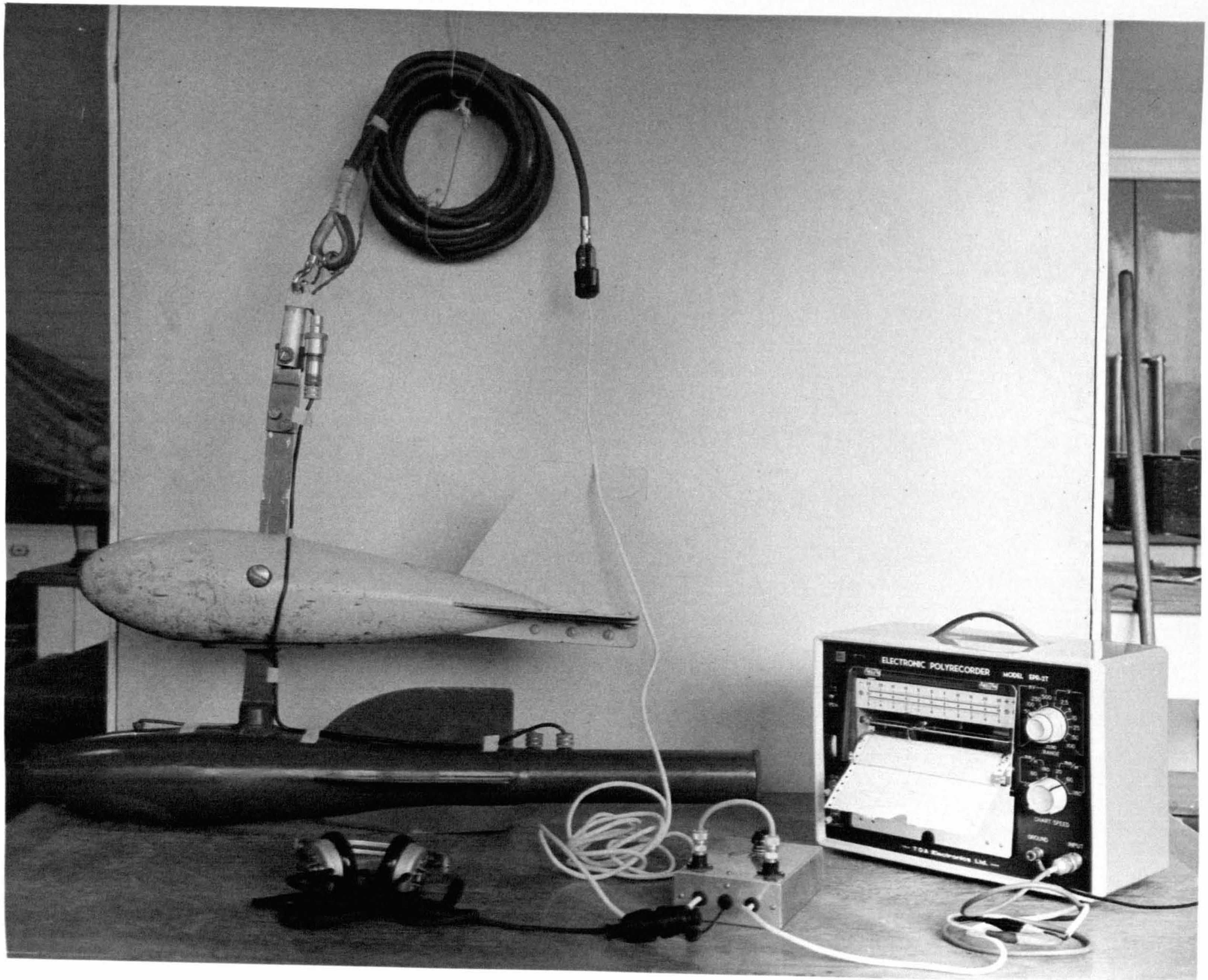


Fig. 8.2.b. Latest design of acoustic bed load detector.

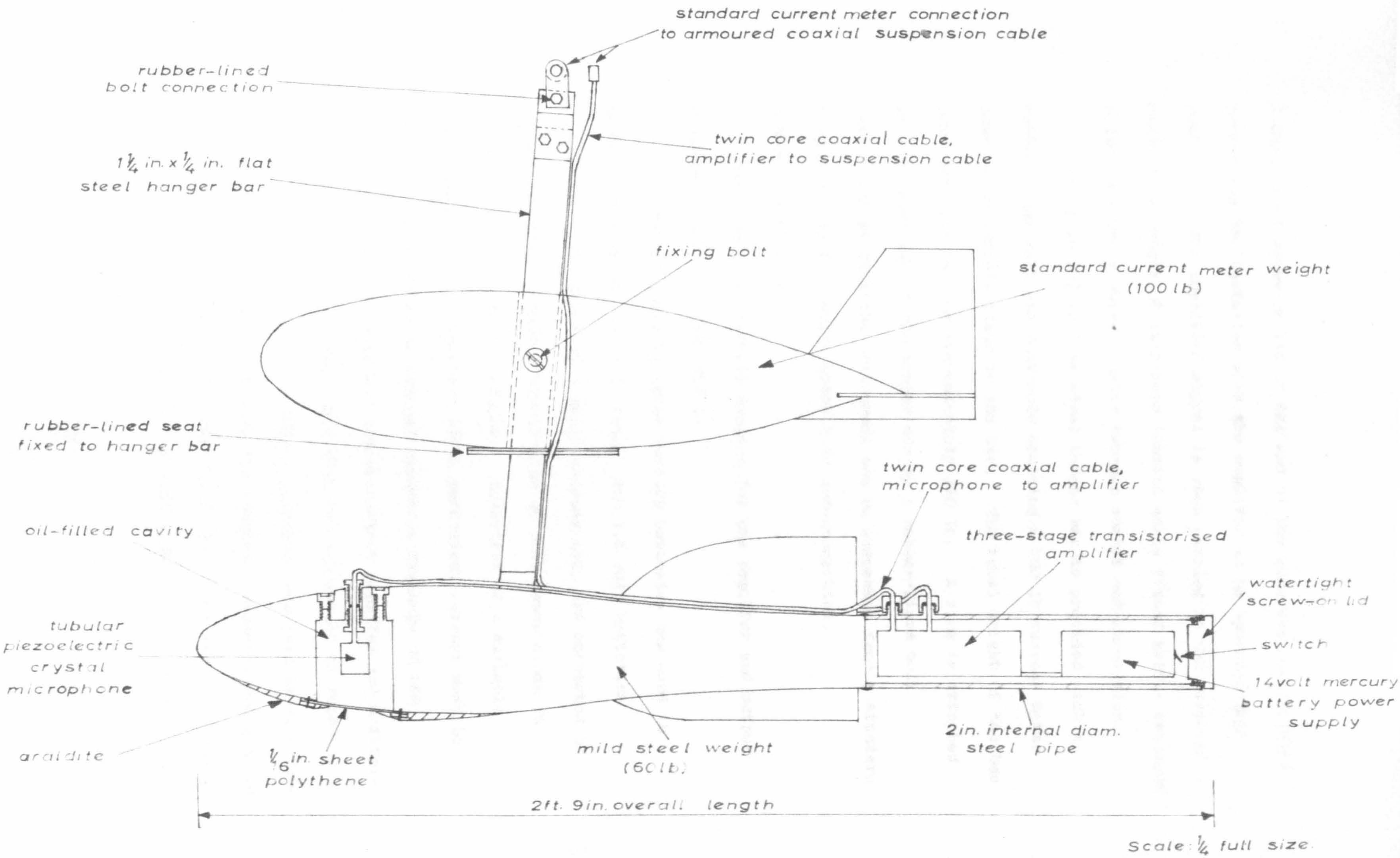


Fig. 8.2.c. Acoustic bed load detector.

A watertight screw-on lid on the end of the pipe enables a switch connecting the batteries with the amplifer to be operated when required The amplified signal is then carried to the suspension cable by a length of twin-core coaxial cable fitted with a standard Hilger and Watts current meter two-pin socket end-connection.

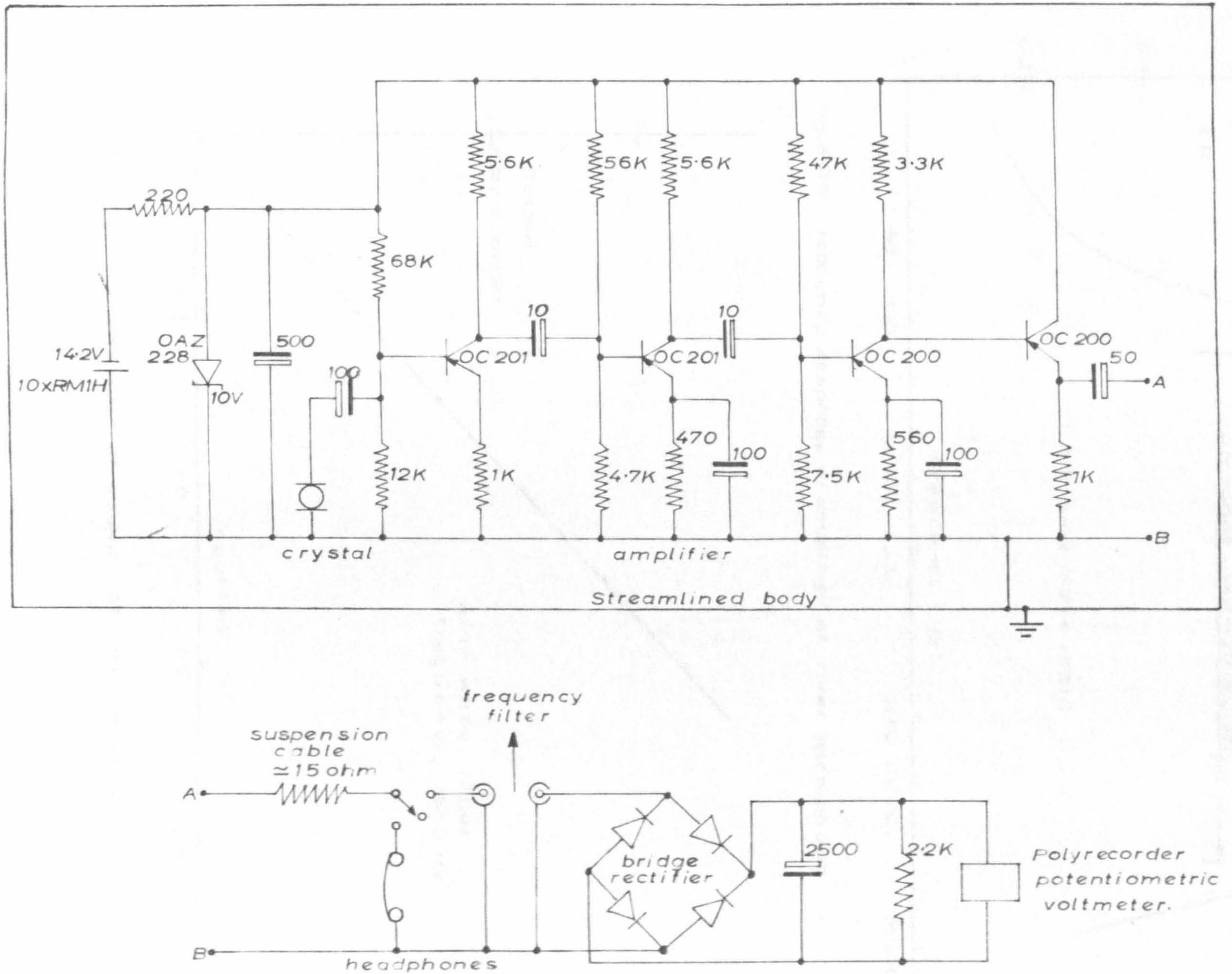
The 1¼ in x ¼ in flat steel hanger bar is provided with a rubber-lined seat and bolt-hole enabling a 100 lb current meter bomb to be rigidly fixed to the bar. The total weight of the bed load detector is thus approximately 160 lb. A ring is attached to the upper end of the hanger bar by a rubber-lined bolt connection so that the instrument can be suspended from a standard Hilger and Watts current meter hook end-connection.

8.2.3. Amplifier

The electrical circuit diagram for the amplifer and output circuit is shown in fig. 8.2.d.

In order to conserve space mercury batteries are used in place of the normal dry cell type; ten 1.4 volt batteries (type RM1H), contained in a small perspex box, are connected by an on-off switch across a stabilising 10 volt Zener diode in series with a 220 ohm resistance. Selection of a suitable resistance value is important since sufficient current must be taken to drive the diode without excessive drainage of the batteries. The three-stage, transistorised amplifer and emitter-follower is similar to that used with the laboratory microphone.

An audio-frequency a.c. signal generator was used as described in section 6.3.2. to investigate the frequency response and gain of the amplifier. Figs. 8.3.e. and 8.3.f. show that the amplifier has a linear r.m.s. voltage gain of 1.21×10^3 over a 3dB frequency range of 140-20,000 Hz.



Output circuit.

All resistances in ohms.
All capacitances in microfarads

Fig. 8.2.d.

Electrical circuit diagram river microphone.

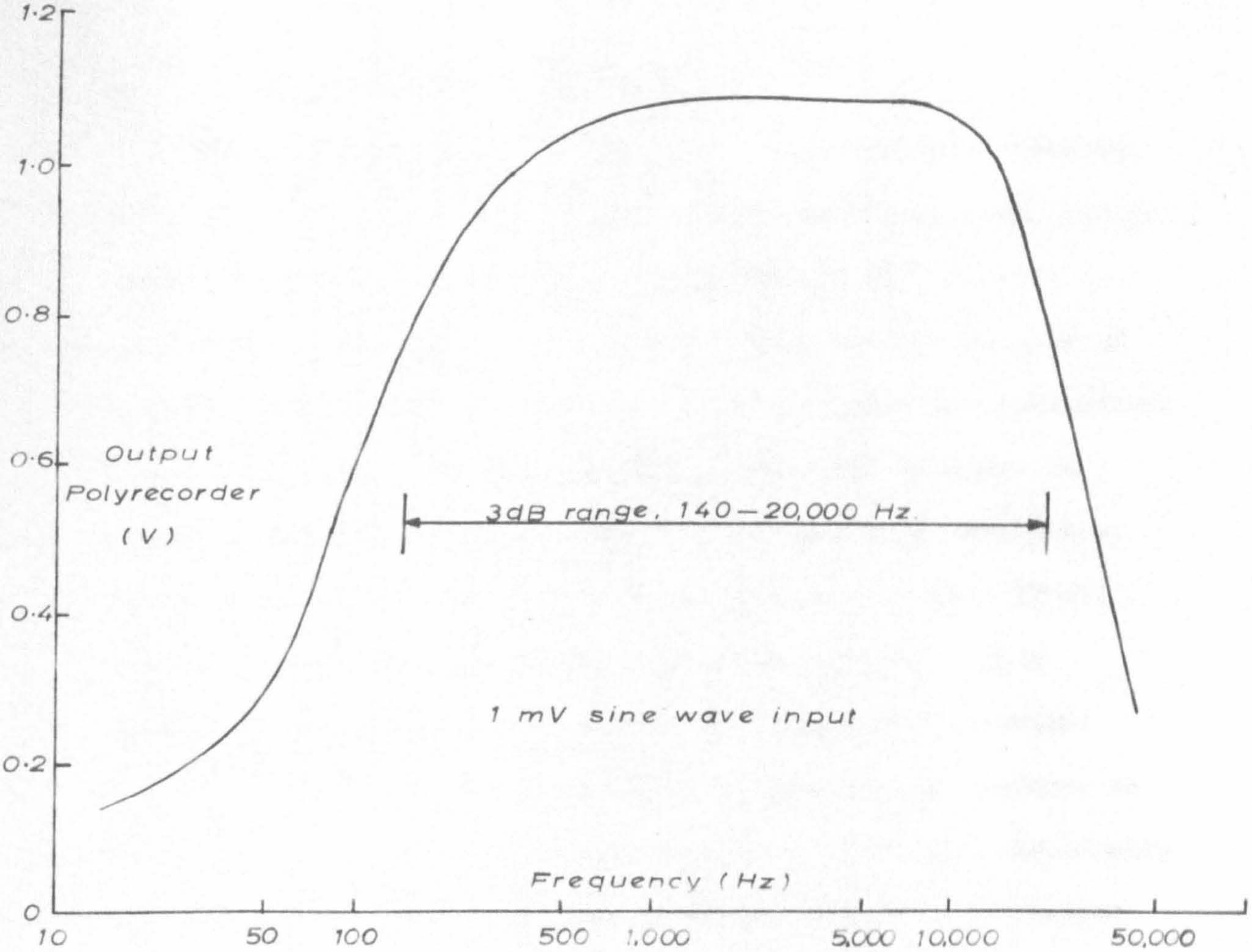


Fig. 8.2e. Frequency response of amplifier of river microphone.

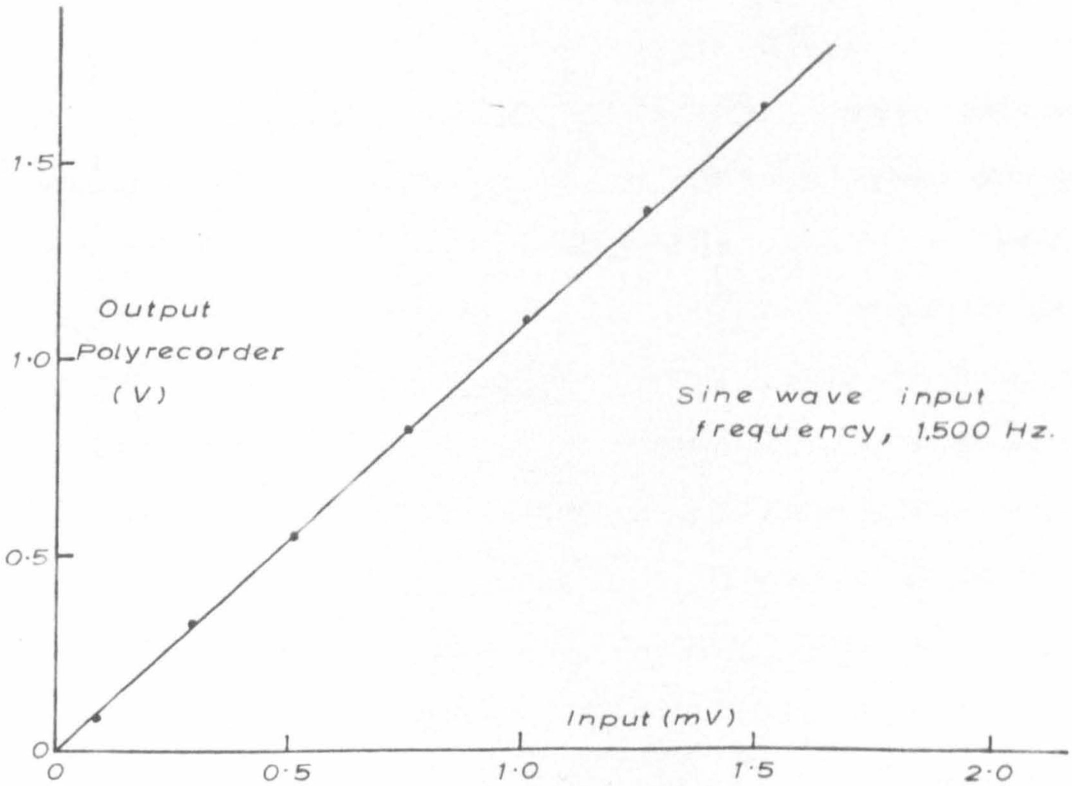


Fig. 8.2f. Linear gain of amplifir of river microphone.

8.2.4. Output Circuit and Recorder

The a.c. output from the amplifier is carried to a three-way switch on the bank by the armoured coaxial suspension cable (inner insulated core 2.5 ohm/100 ft., outer braiding 5 ohm/100 ft.); no insulation is provided on the outer braiding so that one line of the microphone signal is earthed. The signal can be passed direct to headphones, or through a frequency filter and rectifier to a smoothing circuit containing a battery-powered, high impendance, potentiometric voltmeter (Electronic Polyrecorder, Model EPR-2T, manufactured by Toa Electronics Ltd., Tokyo, Japan). This instrument is capable of continuous recording of d.c. voltages from 0.1 mV to 100V with chart speeds ranging from 20 mm/hour to 180 mm/min. On the basis of the results of laboratory experiments (section 6.3.4.) capacitance and resistance values in the output smoothing circuit were selected to give a time constant of 5.5 sec.

8.3. Use of Acoustic Bed Load Dectector at Bywell Cableway Gauging Station

Due to the difficulty of maintaining the acoustic bed load detector correctly orientated beneath the water surface during high flows little information can be extracted from the results obtained to date. However, at river stages up to 5.80 ft above staff gauge zero (54.05 ft A.O.D.), at which flow velocities of 7 ft/sec occur, no sound could be detected by the instrument (at the time in a design stage similar to that shown in fig. 8.2.a., but with an output circuit and recorder similar to the latest design).

During the 68,000 cusec flood mentioned previously it was eventually possible to maintain the instrument beneath the water surface when the river stage had subsided to 14.5 ft above staff gauge zero. The microphone was highly unstable, however, and it is evident from the recorder trace shown in fig. 8.3.a. that movement of the microphone within the water was causing the mercury switch to open and close rapidly. The sound heard in the headphones consisted mainly of high frequency "hissing" noises with lower frequency impulse sounds. It would appear that noise is generated not only by the flow of water past the streamlined body but by the production of large scale turbulence within the river. According to ARABADZHI (1967) the formation and collapse of large air bubbles in normal stream flow generates sound waves with a frequency spectrum extending over the range 40 to 8,000 Hz, almost the whole audio-frequency range; the possibility of filtering out inter-particle collision sound therefore seems doubtful since laboratory experiments have shown that this sound also extends over the audio-frequency range (section 7.1.4.).

The existence of the turbulence generated sound was shown by moving the microphone into still water close to the left bank. In this position an almost steady signal of about 0.1 volts was recorded. A possible further difficulty was then observed when the microphone was immersed in still water near the right bank at a distance of about 230 feet from the winch. The signal observed at this position was about 0.5 volt, suggesting that noise was being picked up in the extended length of suspension cable.

8.4. Conclusions

An instrument has been designed for the detection of bed load movement when suspended above the bed of the River Tyne from the cableway at Bywell gauging station. It consists of a piezoelectric crystal microphone housed in a 60 lb streamlined, metal body to which a standard current meter 100 lb weight can be attached. A three-stage, transistorised, battery-powered amplifier with a voltage

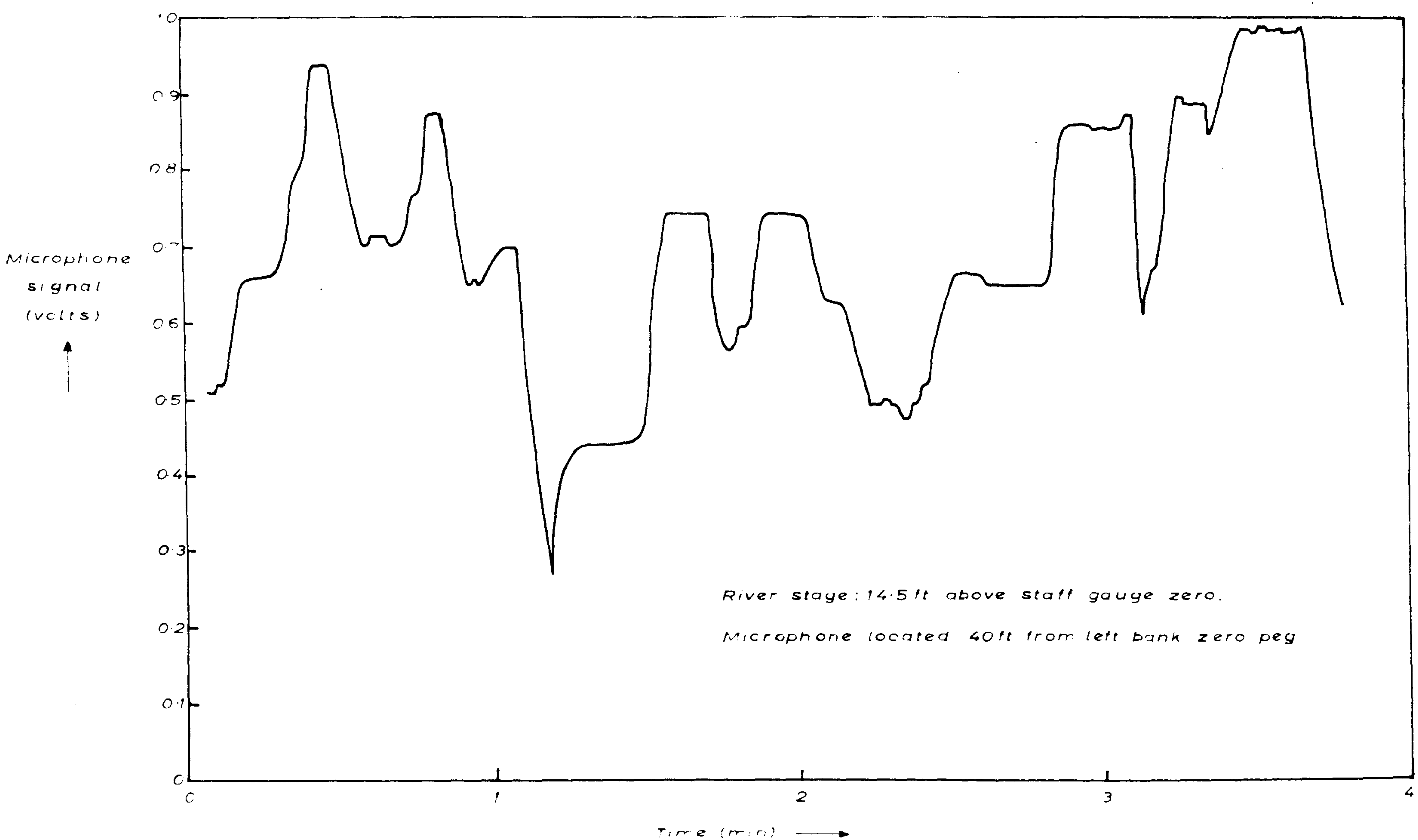


Fig. 8.3.1 Trace of signal from acoustic bed load detector in River Tyne at Bywell

gain of 1.21×10^3 over a 3dB frequency range of 140 to 20,000 Hz is located at the rear of the streamlined body, and transmits the sound of inter-particle collision through the suspension cable to either headphones or a continuously recording potentiometric voltmeter (time constant 5.5 sec).

Infrequent occurrences of sufficiently high flows in the River Tyne during the time available for this research did not permit a final, tested design to be established. However, experience has shown that the application of the acoustic technique to rivers is complicated by a number of difficulties. The principal difficulty is the maintenance of the bed load detector at a fixed position above the sediment bed when suspended from the cableway in high flow velocities. The latest design of the microphone weighs a total of 160 lb, but it may still be necessary to make use of retention cables to ensure correct orientation and stability. Problems of expense and manpower might then arise.

No noise could be detected at river stages up to 5.80 ft above staff gauge zero (6,600 cusec), when flow velocities of up to 7 ft/sec occur; at higher discharges, however, a distinct "hissing" sound was picked up by the microphone. This noise is most likely produced by both the flow of water past the streamlined body and large scale turbulence within the river; the noise spectrum most likely extends over a wide frequency range.

Experiments also indicated that some extraneous noise may be picked up in the long length of coaxial suspension cable through which the amplified microphone signal is transmitted to the recorder.

In attempts to extend the acoustic technique of bed load measurement to rivers conditions should ideally be maintained as similar as possible to those in the laboratory. Hence, even if the above difficulties of stability and extraneous noise can be eliminated (possibily by means of a rigid suspension and frequency filters, respectively) then the microphone must be calibrated in the laboratory with the appropriate size of sediment. Further difficulty may then arise in obtaining bed load discharges in a laboratory flume as large as those occurring in the river.

Section 9

9. Summary of Conclusions

The principal objectives of this research programme, viz. a study of the estimation and measurement of bed load discharge in the River Tyne at Bywell, have been discussed in section 1.3. The conclusions of sections 3 and 4, dealing with estimation of bed load discharge, are summarised in section 9.1., and those of sections 5, 6, 7 and 8, dealing with the measurement of bed load discharge are summarised in section 9.2. Recommendations for further research, section 9.3. complete the section.

9.1. Estimation of Bed Load Discharge

For the estimation of bed load discharge by the rational bed load theories the selection of a reach of river in which conditions approximate as closely as possible to uniform flow was found to be important; in "pool-bar" rivers, such as the River Tyne, this presents some difficulty. Measured water-surface and energy-surface slopes at Bywell were found to vary linearly with the logarithm of river stage (figs. 3.4.c., 3.4.d.)

Bulk sampling and sieve analysis of 2/3 ton of bed material at Bywell yielded the particle size distribution curve of fig. 3.5.e. Investigations indicated that certain information can be obtained by the quicker, more convenient, method of areal sampling, as suggested by WOLMAN (1954). Areal samples of the bed material at Bywell were found to be best described for shape by the KRUMBEIN (1941) sphericity measure and ZINGG (1935) classification (tables 3.5.i. and 3.5.j.), and for roundness by the KRUMBEIN (1941) visual chart method (table 3.5.k.) A petrographic analysis (table 3.5.1.) showed that the majority of the bed material at Bywell is sandstone, with an appreciable number of bed particles originating from parent rocks outside the River Tyne catchment.

Of the many bed load theories found in the available literature only nine were considered to be possibly applicable to the flow and sediment conditions at Bywell; extrapolation sometimes considerable, was necessary, nevertheless, in the application of all these methods. The major defects of most bed load formulae available at present were found to be the necessity for accurate measurement of energy-surface slope, neglect of the mutual interference between particles of different sizes, and neglect of the influence of particle shape.

Fig. 4.1.m. shows the bed load rating curves computed by the nine methods applied to the River Tyne at Bywell. Estimates of bed load discharge at near bankfull stage (61 ft A.O.D., 14.75 ft above staff gauge zero) vary from 120 lb/sec to 1350 lb/sec. Predictions of the critical stage at which bed load movement commences range from 54 ft to 57 ft A.O.D. (7.75 to 10.75 ft above staff gauge zero).

The bed load curve computed by the MEYER-PETER and MÜLLER (1948) formula is considered to be the most reliable. However, due to a natural paving of coarse material on the bed surface and the resistance to entrainment of the predominantly disc-shaped bed particles, bed load discharge most probably commences at a slightly higher stage than that indicated by the rating curve, i.e. at about 56 ft A.O.D. (9.75 ft above staff gauge zero).

Use of the Meyer-Peter and Müller rating curve with the flow frequency curve for the ten year period, 1956/66, indicated an average annual bed load discharge at Bywell of about 15,000 ton, approximately 10% of the average annual suspended sediment discharge. The estimation of average annual bed load discharge by assuming a certain percentage of the average annual suspended sediment discharge, as recommended by LANE and BORLAND (1951), is considered to be as accurate as, and more easily obtained than, that given by the application of bed load formulae.

The regime approach to sediment transport appears to be insufficiently developed for the determination of bed load discharge in coarse gravel-bed rivers. One most promising solution to the sediment problem, however, seems to lie in the ultimate combination of the regime and rational approaches.

9.2. Measurement of Bed Load Discharge

The second part of the research programme proved unsuccessful in achieving its immediate objective, the confirmation by direct measurement of the estimated bed load discharge in the River Tyne at Bywell. Some progress was made, however, in the development of a possible method of continuous measurement of bed load discharge in gravel rivers by an acoustic technique.

A survey of available literature (section 5.1.) showed that the most accurate method of measurement of bed load discharge is the slot or pit type structure; high construction and maintenance costs are usually prohibitive.

The most accurate and reliable bed load sampler for the flow and sediment conditions at Bywell was considered to be the V.U.V. pressure-difference sampler designed by NOVAK (1959). However, use of the sampler at a cableway gauging station requires a complex system of retention and suspension cables to ensure correct orientation and stability. Such an arrangement is likely to be expensive and require at least three winches and a team of three winch operators (section 5.2.).

The accuracy of bed load traps is usually low, due to their effect on the surrounding flow and sediment regime, the necessity for laboratory calibration, the variability of their efficiencies with parameters such as particle size, flow velocity etc. and the oscillatory, or unsteady, nature of bed load movement.

In the design of a laboratory sediment channel (section 6.2.) particular attention should be given to the measurement of water discharge, design of channel inlet conditions, and measurement of water-surface and energy-surface slopes. It was found that feed rates of 5 mm. gravel varying continuously from 30 lb/hr to 700 lb/hr could be obtained by a hopper and belt arrangement. Measurement of sediment discharge in the channel could be made by continuous weighing of a watertight container, connected by flexible tubing to the downstream end of the channel.

Preliminary experiments (section 6.3.) indicated that inter-particle collision sound in the laboratory sediment channel could be conveniently recorded by a piezoelectric crystal microphone and three-stage, transistorised amplifier (voltage gain 1.21×10^3, 3dB frequency range 140-20,000 Hz). The amplified, rectified signal was fed to an output smoothing circuit (time constant 5.63 sec) containing a continuously recording Ultra-Violet oscillograph.

The frequency spectrum of the inter-particle collision sound emitted by the single-sized 5 mm gravel in the laboratory channel was found to extend over the whole audio-frequency range (section 7.1.4.). Measurement of microphone signal was therefore made over the cut-off frequency range of the amplifier.

Theoretical considerations (section 7.3.) indicate that the relationship between bed load discharge of a given sediment per unit width, q_B, and microphone signal, M_s, expressed as a current or voltage, is given by:-

$$q_B = \left[(a^2 + b\,M_s^{4/3})^{\frac{1}{2}} - a \right]^{3/2}$$

where a and b are constants dependent upon the physical and electrical properties of the sediment, fluid, microphone and recording equipment.

Analysis of experimental results indicated that average bed load discharge in the laboratory channel over periods of 10, 20, 30 and 40 minutes could be predicted from a knowledge of the microphone signal with an accuracy of $\pm$35% at the 65% confidence level. Scatter of the observational results might possibly be reduced by the introduction of a third parameter involving scour or accretion.

The principal disadvantage of the present acoustic technique is the insensitivity of the microphone signal to relatively large changes in bed load discharge.

Development of an acoustic bed load detector for use at Bywell cableway gauging station was hampered by a lack of opportunities for the testing of preliminary designs. The latest design (section 8.2.) consists of a piezoelectric crystal microphone housed in a streamlined, 60 lb. weight connected to a battery-powered, three-stage, transistorised, amplifier (voltage gain 1.21 x 10^3, 3dB frequency range 140-20,000 Hz) located at the rear of the streamlined body. The amplified signal is transmitted through the coaxial suspension cable to headphones or a smoothing circuit (time constant 6.5 seconds) containing a battery-powered, continuously recording, potentiometric voltmeter. The detector is designed to enable a standard current meter 100 lb weight to be attached.

Experience indicated that the principal difficulties involved in the use of acoustic detector from a cableway are the maintenance of correct orientation and stability, noise generated by flow past the microphone and in large-scale turbulence (probably extending over a wide frequency spectrum) and extraneous noise picked up in the extended length of suspension cable.

For use in rivers the acoustic bed load detector must be calibrated in a wide laboratory channel over the appropriate size of sediment.

9.3. Recommendations for Further Research

Most river engineering problems, including the estimation of bed load discharge usually require some knowledge of the material forming the bed of the river. Experience at Bywell has shown that further investigations into the sampling, analysis, and description of coarse sediments are required. Comparison of the results of bulk and areal sampling methods, for instance, indicates that the latter method might possibly be used to obtain certain information more quickly and more conveniently. There appears, in fact, to be a need for the systematic collection of river data, as described recently by CAMPBELL and CADDIE (1964) and NEILL and GALAY (1967).

For estimation by formula of the bed load discharge of coarse sediment there is a need, not necessarily for a new formula but for the modification of an existing method, possibly that of MEYER-PETER and MÜLLER (1948). The new development should enable stimates of effective bed shear stress to be made without the necessity for accurate measurement of energy-surface slope, and should include also the influence of particle shape and the mutual interference of particles of different sizes. This possibility is discussed more fully in section 4.5.

Laboratory experiments have shown that the acoustic technique is potentially capable of continuous measurement of bed load discharge. With the aid of expert knowledge and more sophisticated methods, the accuracy of the technique could no doubt be improved. Possible alternatives to the technique used in the present research are the recording of the power output of the microphone and the measurement of the rate at which impulses produced by inter-particle impacts are received. Future laboratory experiments should be carried out in wide channels to eliminate side effects; construction of a directional microphone may similarly improve the method.

More field experience is required. The most promising results would probably be obtained by the location of a microphone, carefully designed such that flow past it produces no noise, at a fixed height above the river bed. In this way information could be obtained on the magnitude and frequency spectrum of the sound emitted by inter-particle collision, and of other noises within the river.

Appendix

Regression Analysis of Laboratory Experimental Data

APPENDIX

In section 7.4. it was required to obtain the values of the constants a and b to give a least squares fit to the experimental data. Since partial differentiation with respect to a and b of the residual given by the theoretical equation 7.3.d. resulted in normal equations not directly soluble for a and b, the following iterative procedure for the curvilinear regression analysis was carried out.

It was required to fit an equation of the type:-

$$q_B = f(M_s, a, b) \quad \ldots \ldots \ldots \ldots \ldots \quad A-1$$

where q_B is the dependent variable

M_s is the independent variable

a, b, are undetermined constants

If a_o, b_o are the best fit values of the undetermined constants a, b, and q_B is the observed value corresponding to the observed M_s, then the residual (the difference between observed and predicted q_B) is equal to:-

$$R = q_B - f(M_s, a_o, b_o)$$

Let a, b be first approximation of a_o, b_o such that Δa, Δb, are small corrections required to give:-

$$a_o = a + \Delta a \qquad\qquad b_o = b + \Delta b$$

$$\therefore R = q_B - f(M_s, a + \Delta a, b + \Delta b)$$

Expanding the right hand side of the above equation by Taylor's theorem for a function of several variables:-

$$R = q_B - f(M_s, a, b) - \frac{\partial[f(M_s, a, b)]}{\partial a}\Delta a - \frac{\partial[f(M_s, a, b)]}{\partial b}\Delta b$$

+ higher powers and products of Δa, Δb.

Denoting $f(M_s, a, b)$ by F, then:-

$$R = (q_B - F) - \frac{\partial F}{\partial a}\Delta a - \frac{\partial F}{\partial b}\Delta b$$

Since greater scatter was evident at higher values of q_B the residuals were weighted by multiplying by the inverse of q_B, i.e. by $1/F$. The regression thereby involved the minimisation of the sum of squares of the percentage residuals. The weighted residual is given by:-

$$R_w = \frac{(q_B - F)}{F} - \frac{1}{F}\frac{\partial F}{\partial a}\Delta a - \frac{1}{F}\frac{\partial F}{\partial b}\Delta b$$

$$= Z - X\Delta a - Y\Delta b$$

$$\text{where } Z = \frac{(q_B - F)}{F},\ X = \frac{1}{F}\frac{\partial F}{\partial a},\ Y = \frac{1}{F}\frac{\partial F}{\partial b}, \quad \ldots\ldots\ldots \quad \text{A-2}$$

The sum of squares of the weighted residuals is thus given by:-

$$\sum R_w^2 = \sum (Z - X\Delta a - Y\Delta b)^2$$

Partially differentiating with respect to Δa and Δb, equating to zero and solving for Δa and Δb gives:-

$$\Delta a = \frac{\Sigma XZ\,\Sigma Y^2 - \Sigma YZ\,\Sigma XY}{\Sigma Y^2\,\Sigma X^2 - (\Sigma XY)^2} \quad \ldots\ldots\ldots \quad \text{A-3}$$

$$\Delta b = \frac{\Sigma YZ\,\Sigma X^2 - \Sigma XZ\,\Sigma XY}{\Sigma Y^2\,\Sigma X^2 - (\Sigma XY)^2} \quad \ldots\ldots\ldots \quad \text{A-4}$$

The procedure is then repeated using new values of a, b equal to $a + \Delta a$, $b + \Delta b$ until:-

absolute values of $\frac{\Delta a}{a + \Delta a}$, $\frac{\Delta b}{b + \Delta b}$ < 0.005 i.e. ½%

The theoretical microphone equation 7.3.d. which it was required to fit to the experimental data was:-

$$q_B = \left[(a^2 + bM_s^{4/3})^{\frac{1}{2}} - a\right]^{3/2}$$

Therefore, in the determination of Δa, Δb by equations A-3, A-4:-

$$\left.\begin{aligned} Z &= \left[\frac{q_B}{\left[(a^2 + bM_s^{4/3})^{\frac{1}{2}} - a \right]^{3/2}} - 1 \right] \\ X &= \frac{{}^3/_2 \left[\dfrac{a}{(a^2 + bM_s^{4/3})^{\frac{1}{2}}} - 1 \right]}{\left[(a^2 + bM_s^{4/3})^{\frac{1}{2}} - a \right]} \\ Y &= \frac{{}^3/_2 \left[\dfrac{M_s^{4/3}}{2(a^2 + bM_s^{4/3})^{\frac{1}{2}}} \right]}{\left[(a^2 + bM_s^{4/3})^{\frac{1}{2}} - a \right]} \end{aligned} \right] \quad \ldots \text{A-5}$$

A computer programme was developed for use on the KDF 9 computer of the University of Newcastle upon Tyne. The programme used for the regression of the 10 minute averages of q_B on M_s is included in this appendix; the number of observational points was 141 and the initial assumed values of a and b were 7.0 and 0.31, respectively.

The required input data and the computer output are also included. The latter gives the final values of a, b and $\sum Z^2$. Since $Z = (q_B - F)/F$ the standard deviation of the percentage residuals, or percentage errors, is thus given by $100\sqrt{\sum Z^2/n-1}$. The 65% confidence limits for the prediction of q_B from M_s are approximately ± 1 s.d. of the percentage residuals.

The programme can be used with little modification for regression according to any function of the type given in equation A-1, e.g.

$$q_B = aM_s^{\,b}$$

$$q_B = aM_s + bM_s^{\,2}$$

The only alterations necessary, apart from the number of points and the initial assumed values of a and b, are the lines for the calculation of Z, X and Y (equations A-2 and A-5).

Iterative curvilinear regression programme for KDF9 computer.

```
CV09*MICROPHONE*EQN→

begin   library A0,A6;
        open(20);open(30);
begin   real a,dela,b,delb,
        sigXZ,sigXY,sigYZ,
        sigXsq,sigYsq,sigZsq;
        integer i,p;
        array qB,Ms,LZ,LX,LY[1:141];

        for i := 1 step 1 until 141 do
        qB[i]:= read(20);
        for i := 1 step 1 until 141 do
        Ms[i]:= read(20);

        a:= 7.0;
        b:= 0.31;
        p:= 0;

LOOP:   writetext(30,[[p]]);
        sigXY:=sigXZ:=sigYZ:=sigYsq:=sigXsq:=sigZsq:=0;
        p := p+1;

        for i := 1step 1 until 141 do
        begin

        LZ[i]:= qB[i]/((a↑2 + b×Ms[i]↑1.333)↑0.5 - a)↑1.5 - 1;
        LX[i]:= 1.5×(a/((a↑2 + b×Ms[i]↑1.333)↑0.5) - 1)/
                      ((a↑2 + b×Ms[i]↑1.333)↑0.5 - a);
        LY[i]:= 1.5×(Ms[i]↑1.333/2/((a↑2 + b×Ms[i]↑1.333)↑0.5))/
                      ((a↑2 + b×Ms[i]↑1.333)↑0.5 - a);

        sigXZ:= sigXZ + LX[i]×LZ[i];
        sigXY:= sigXY + LX[i]×LY[i];
        sigYZ:= sigYZ + LY[i]×LZ[i];

        sigXsq:= sigXsq + LX[i]×LX[i];
        sigYsq:= sigYsq + LY[i]×LY[i];
        sigZsq:= sigZsq + LZ[i]×LZ[i];

        end;

        dela:=  (sigXZ×sigYsq - sigYZ×sigXY)/
                (sigYsq×sigXsq - sigXY×sigXY);
        delb:=  (sigYZ×sigXsq - sigXZ×sigXY)/
                (sigYsq×sigXsq - sigXY×sigXY);

        writetext(30,[[2c]iteration*counter*=*]);
                write(30,format([nddcc]),p);
        writetext(30,[sigZsq*=*]);
                write(30,format([+ndd.ddddcc]),sigZsq);
```

```
writetext(30,[a*=*]);
        write(30,format([+ndddd.dddddc]),a);
writetext(30,[b*=*]);
        write(30,format([+ndddd.dddddcc]),b);
writetext(30,[delta*a*=*]);
        write(30,format([+ndddd.dddddc]),dela);
writetext(30,[delta*b*=*]);
        write(30,format([+ndddd.dddddcc]),delb);

if abs(dela/(dela + a)) > 0.005
and abs(delb/(delb + b)) > 0.005 then
begin
writetext(30,[delta*a*divided*by*a+delta*a,
*and*delta*b*divided*by*b+delta*b*[c]
are*both*greater*than*1/2*percent.[2c]
a*+*delta*a*=]);
        write(30,format([+ndddd.dddddc]),a+dela);
writetext(30,[b*+*delta*b*=]);
        write(30,format([+ndddd.dddddcc]),b+delb);

a := a + dela;
b := b + delb;
goto LOOP;
end else

begin
writetext(30,[one*or*both*of*them*within*1/2*percent[2c]
a*+*delta*a*=]);
        write(30,format([+ndddd.dddddc]),a+dela);
writetext(30,[b*+*delta*b*=*]);
        write(30,format([+ndddd.dddddc]),b+delb);
end;

end;    close(20);close(30);
end→
```

Input data.

```
0.07;0.13;0.13;0.20;0.20;0.17;0.20;0.27;0.27;0.27;
0.30;0.40;0.30;0.40;0.43;3.24;1.80;1.80;2.71;2.40;
1.17;1.40;2.00;1.77;2.00;2.20;1.64;1.33;1.40;1.77;
2.07;1.47;2.20;1.70;1.74;1.60;2.47;2.78;1.87;1.74;
1.97;1.94;1.40;1.13;1.87;8.77;7.03;6.36;6.66;6.76;
7.23;6.53;6.59;7.33;7.00;7.26;6.16;7.70;7.00;7.00;
7.20;3.41;3.88;3.55;1.44;2.17;4.15;2.81;3.11;3.33;
5.29;4.35;3.24;3.31;3.71;3.98;4.05;6.00;5.79;4.72;
4.52;4.67;4.15;4.52;4.40;4.18;4.44;4.27;5.98;5.07;
4.63;4.20;4.23;5.23;5.54;5.10;5.17;5.54;5.57;5.13;
4.73;4.79;4.67;0.33;0.50;0.97;0.77;1.47;1.23;0.53;
1.30;1.03;0.50;0.93;1.17;0.77;1.07;0.77;0.97;1.03;
0.77;0.67;0.77;0.67;0.43;0.36;0.43;0.46;0.43;0.40;
8.14;8.24;9.10;1.23;0.73;0.87;0.60;0.50;7.54;6.97;
7.47;
```

10 minute average q_B

```
 5.4; 6.6; 6.9; 6.8; 6.9; 6.8; 7.1; 6.8; 7.7; 8.4;
 8.0; 7.7; 8.0; 9.1; 8.5;39.5;35.0;32.1;34.5;29.0;
28.8;29.9;35.3;35.4;36.5;34.0;30.3;30.3;29.8;35.6;
33.8;29.9;34.9;33.4;32.2;31.5;37.5;35.9;28.3;28.5;
29.1;27.1;24.5;26.1;28.7;53.9;38.3;34.6;39.2;41.6;
40.1;43.2;44.4;48.8;51.7;54.3;41.8;45.9;51.6;51.3;
46.2;39.8;36.2;28.9;25.6;36.9;40.5;39.0;41.3;43.2;
44.9;39.8;44.6;44.8;46.3;46.5;44.5;44.6;41.1;41.9;
40.5;38.0;44.6;42.1;42.4;40.4;39.8;38.8;44.8;44.6;
40.5;37.8;36.4;40.5;40.5;46.1;46.9;44.5;47.7;44.3;
44.3;45.3;43.3;16.4;18.5;21.3;18.8;22.1;19.4;15.9;
21.8;20.0;15.8;21.3;23.9;21.8;21.6;20.4;21.0;22.0;
19.8;10.6; 9.6;11.5;11.0;12.2;11.9;11.2;11.6;11.6;
55.3;57.7;57.1;12.6;15.1;13.6;13.2;13.2;52.6;61.3;
58.3;
→
```

10 minute average M_s

Regression programme output.

```
ITERATION COUNTER =   1

SIGZSQ =  +22.04616

A =       +7.00000
B =       +0.31000

DELTA A =      -6.11348
DELTA B =      -0.23276

DELTA A DIVIDED BY A+DELTA A, AND DELTA B DIVIDED BY B+DELTA B
ARE BOTH GREATER THAN 1/2 PERCENT.

A + DELTA A =    +0.88652
B + DELTA B =    +0.07724



ITERATION COUNTER =   2

SIGZSQ =  +20.41553

A =       +0.88652
B =       +0.07724

DELTA A =      +0.91905
DELTA B =      +0.03495

DELTA A DIVIDED BY A+DELTA A, AND DELTA B DIVIDED BY B+DELTA B
ARE BOTH GREATER THAN 1/2 PERCENT.

A + DELTA A =    +1.80557
B + DELTA B =    +0.11220



ITERATION COUNTER =   3

SIGZSQ =  +17.79875

A =       +1.80557
B =       +0.11220

DELTA A =      +0.92587
DELTA B =      +0.03478

DEL,A A DIVIDED BY A+DELTA A, AND DELTA B DIVIDED BY B+DELTA B
ARE BOTH GREATER THAN 1/2 PERCENT.

A + DELTA A =    +2.73143
B + DELTA B =    +0.14698
```

```
ITERATION COUNTER =   4

SIGZSQ =  +18.06950

A =       +2.73143
B =       +0.14698

DELTA A =      +0.54171
DELTA B =      +0.02030

DELTA A DIVIDED BY A+DELTA A, AND DELTA B DIVIDED BY B+DELTA B
ARE BOTH GREATER THAN 1/2 PERCENT.

A + DELTA A =    +3.27315
B + DELTA B =    +0.16728


ITERATION COUNTER =   5

SIGZSQ =  +18.63427

A =       +3.27315
B =       +0.16728

DELTA A =      +0.13243
DELTA B =      +0.00502

DELTA A DIVIDED BY A+DELTA A, AND DELTA B DIVIDED BY B+DELTA B
ARE BOTH GREATER THAN 1/2 PERCENT.

A + DELTA A =    +3.40557
B + DELTA B =    +0.17230


ITERATION COUNTER =   6

SIGZSQ =  +18.77420

A =       +3.40557
B =       +0.17230

DELTA A =      +0.01186
DELTA B =      +0.00046

ONE OR BOTH OF THEM WITHIN 1/2 PERCENT

A + DELTA A =    +3.41743
B + DELTA B =     +0.17276

RAN/EL/001M17S/001M23S
```

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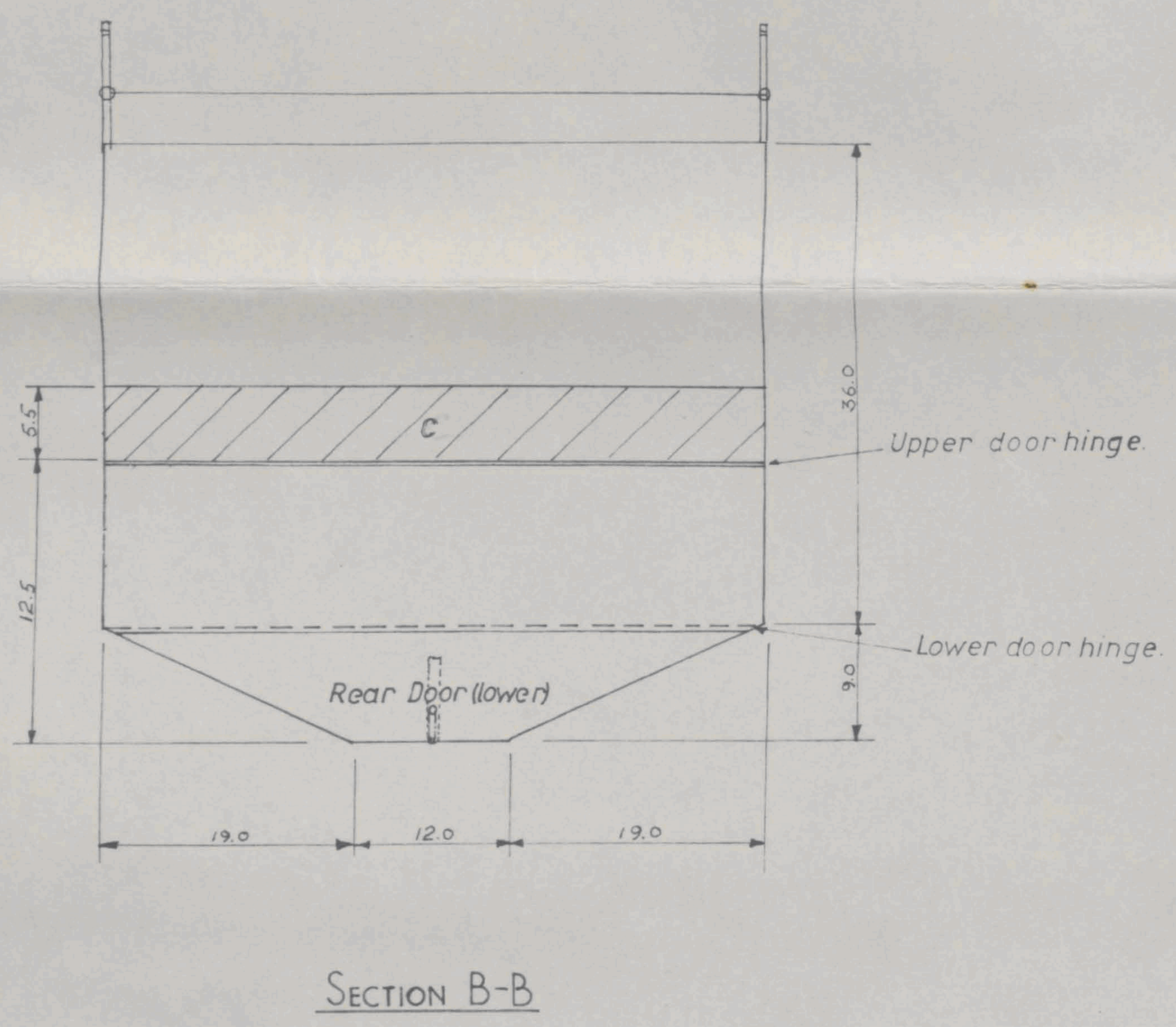
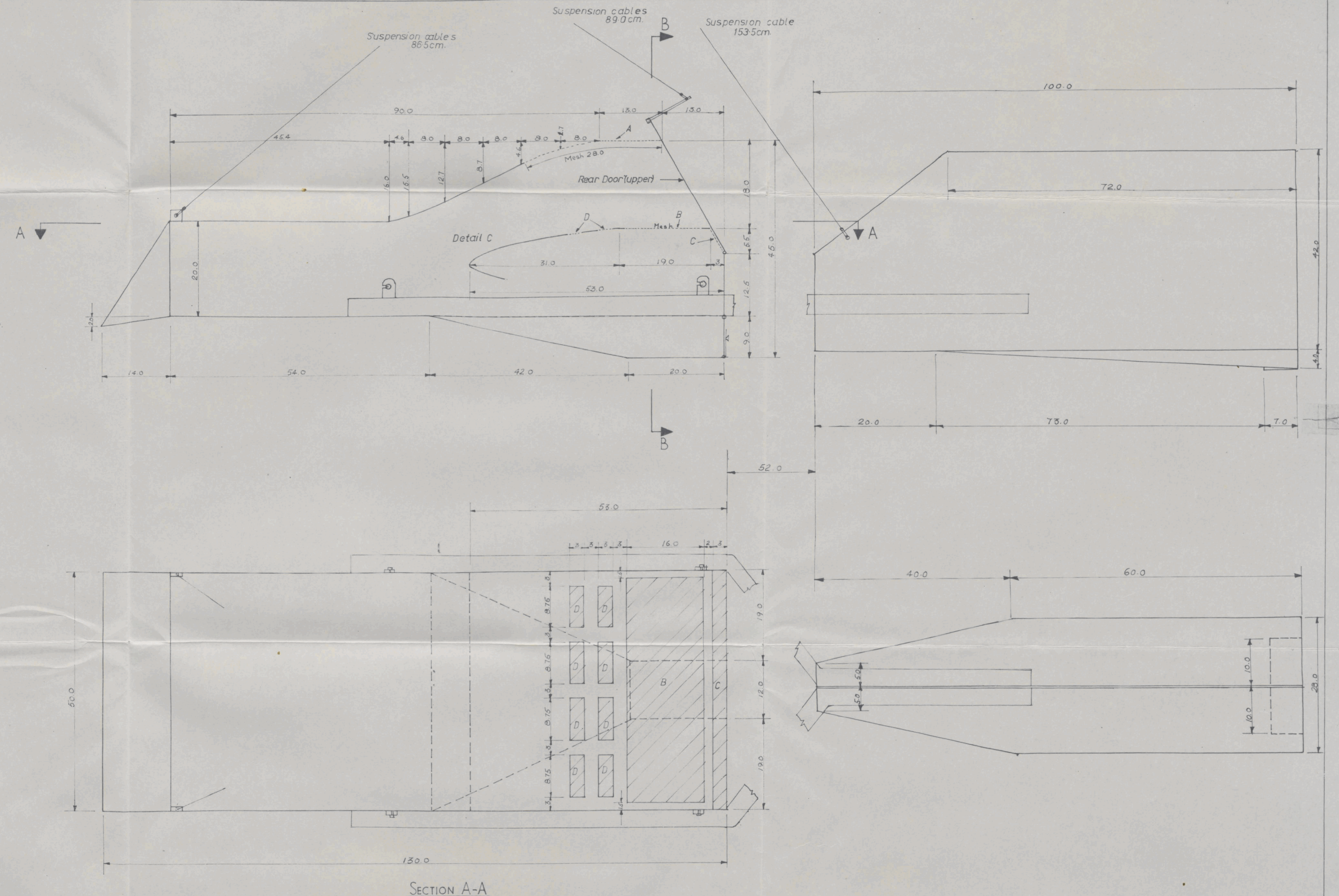
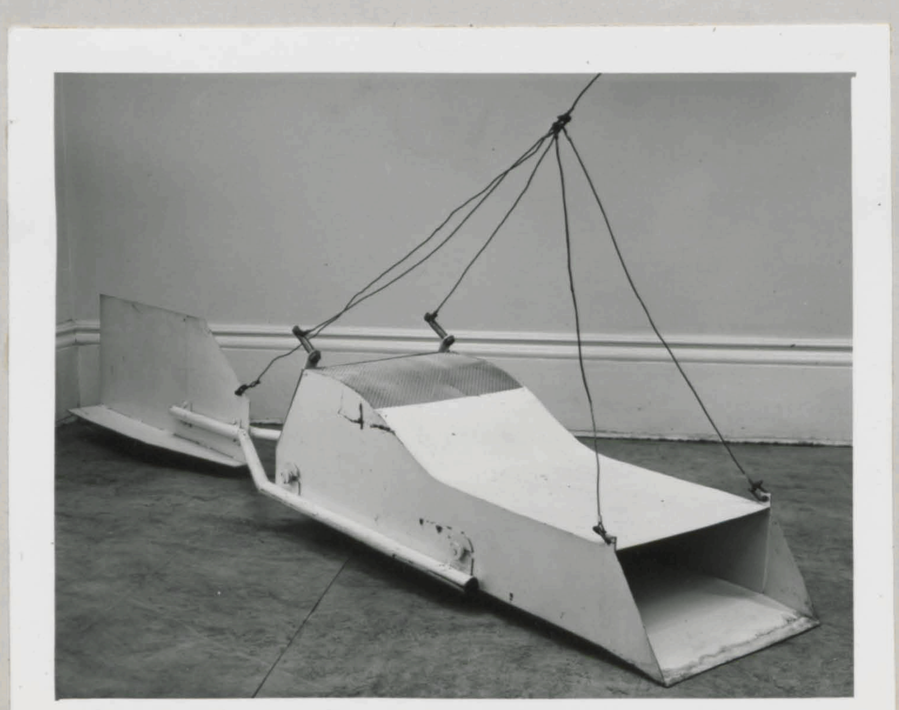
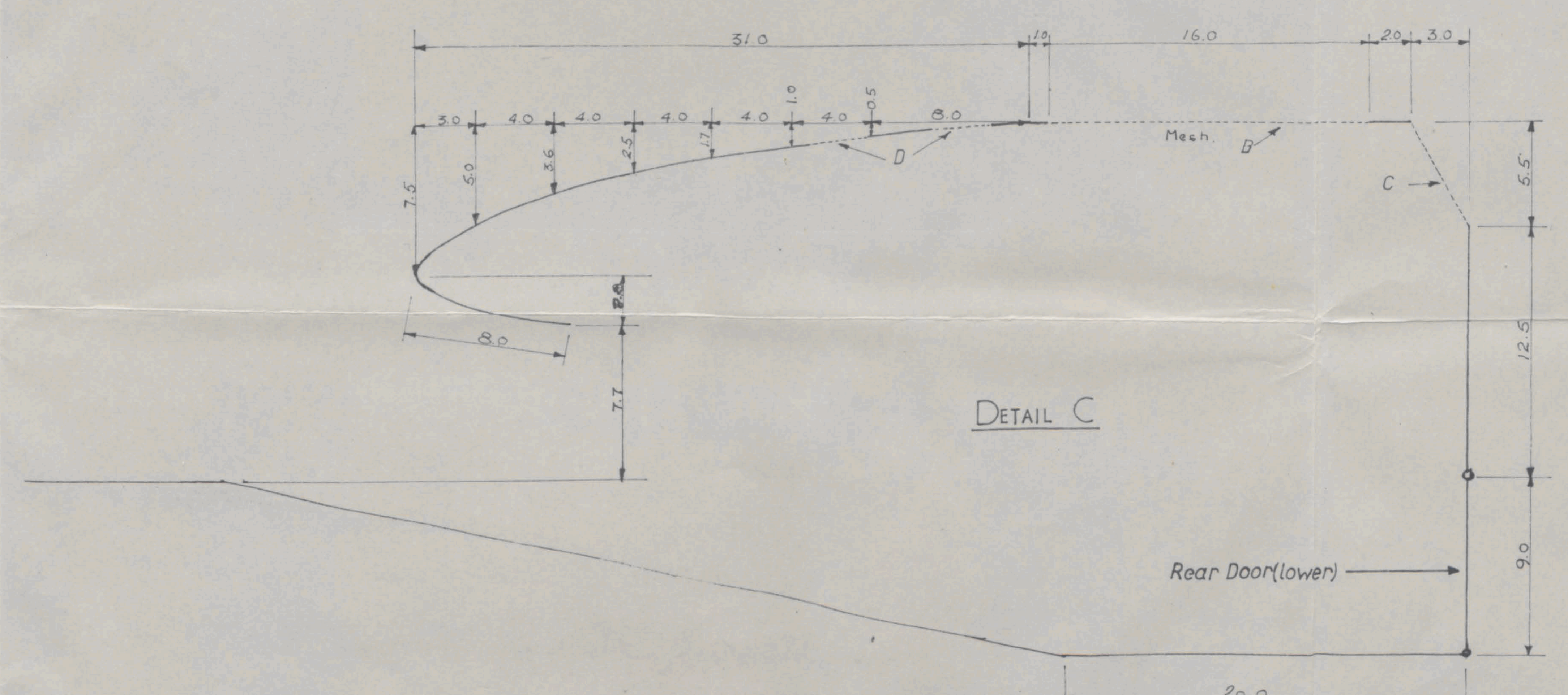




1. Trap and fin made of 15 gauge brass.
2. Connection between trap and fin made by 1½ in. O.D. steel tubing such that sampler can be dismantled.
3. Five suspension cables of ⅛ in. dia. with eyes at both ends to connect to brass shackles.
4. Mesh of 1/16 in. dia. wire with 3/16 in. square openings.

Part.	Size (cm)	No.
A	28.0 X 50.0	1
B	16.0 X 47.0	1
C	6.3 X 50.0	1
D	8.75 X 13.0	8





Scales:—
Main drawings: 1cm. represents 5cm.
Detail C: 1 cm. represents 10 cm.
All dimensions in centimetres.

Fig. 5.2.a. V.U.V. BED LOAD TRAP.

Fig. 3.3.h. RIVER TYNE AT BYWELL — Longitudinal profiles.