THE UNIVERSITY OF NEWCASTLE UPON TYNE

DEPARTMENT OF CIVIL ENGINEERING

THE EROSION RESISTANCE OF COMPACTED CLAY FILL IN RELATION TO EMBANKMENT OVERTOPPING

Vol.1

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SYNOPSIS

Most texts on the subject of dam construction state flatly that under no circumstances should an earth dam be permitted to overtop, and that spillway capacity should be such as to ensure this is achieved.

The problems of flood forecasting are outlined and a survey of failures and accidents caused by overtopping presented. Information from this survey highlights weaknesses both in safety legislation and in operational procedures. These are presented, together with recommendations relevant to the design and construction of embankment dams.

The properties of clays which are relevant to the problem of erosion are investigated by literature survey and laboratory research and the results presented. The mechanism and degree of erosion are detailed in an attempt to quantify the amount of erosion likely to occur for any particular clay.

Recommendations are made with respect to areas where more research would be beneficial.

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CONTENTS

		Page
	SECTION I	
CHAPTER I	Introduction	1
1.1	Historical Dams	2
1.2	Earthfill Embankments	4
1.3	The Development of Safety Regulations for Dams	15
CHAPTER II	The Problems of Flood Forecasting	17
2.1	Introduction	17
2.2	Re-evaluation of Spillways	18
2.3	Flood Forecasting Methods	20
2.4	Lack of Data	22
2.5	Estimated and Observed Floods	22
2.6	Re-evaluation of Spillways	25
2.6.1	Institution of Civil Engineers' Recommendations	25
2.6.2	American Society of Civil Engineers' Recommendations	31
2.7	Safety Considerations	34
2.8	Conclusion	37
CHAPTER 3	Failures and Accidents	38
3.1	Introduction	38
3.2	Failures	38
3.3	Safety Regulations and Legislations	46
3.4	A Study of Failures	48
3.4.1	Small Dams	51
3.4.2	Engineering Supervision during Construction	56

3.4.3	Spillways/River Diversion during Construction	58
3.4.4	Spillways	58
3.4.5	Interruption of Work during Construction	59
3.4.6	Access	60
3.4.7	Mine Tailings/Chemical and Industrial Waste Lagoons	61
3.4.8	Awareness of Danger	63
3.4.9	Sabotage and Vandalism	64
3.4.10	Enlargement/Embankment Raising	65
3.4.11	Control Structures - Flashboards	65
3.4.12	Roller Gates	66
3.4.13	Power Supplies	67
3.4.14	Icing of Gates	67
3.4.15	Maintenance	69
	Valves and Gates	69
3.4.16	Settlement	70
3.4.17	Empty/Abandoned Reservoirs	72
3.4.18	Cascade Failures	72
3.4.19	Canals	75
3.4.20	Landslides	76
3.4.21	Slide/Failures	78
3.4.22	Damage caused in the Valley Downstream of the Dam	79
3.4.23	Corewalls - Steel/Concrete	80
3.4.24	Extreme Rainfall	83
3.5	General Considerations	84
3.6	Hazard Index and Safety	98
3.7	Warning Systems	100
3.8	Erosion Resistance	101
3.9	Flow through and over Rockfill	108
3.10	Conclusions	116

SECTION II

CHAPTER 4		Erosion of Clays - A Literature Review	117
4.1		Introduction	117
4.2		Erosion of Cohesive Soils	117
4.3		Properties of Cohesive Material	118
	4.3.1	Common Clay Minerals	120
	4.3.2	Isomorphous Substitution	123
	4.3.3	Plasticity Index and Activity	123
	4.3.4	Aggregation and Dispersion of Soil Particles	125
		4.3.4.1 Electrical Forces	125
	4.3.5	Interparticle Forces in Clay Soils and Clay Structures	129
	4.3.6	Factors Influencing the Behaviour of the Double Layer System	130
4.4		Non-Salt Flocculation	134
4.5		Nature of Water - Clay	134
4.6		Compaction	136
4.7		Shear Strength of Compacted Clay	138
	4.7.1	Forces between Particles	138
	4.7.2	General Strength Equation	139
	4.7.3	The Components of Shear Strength	139
4.8		Thixotropy of Clays	140
4.9		Erosion Studies	140
4.10		Conclusions	165
CHAPTER 5		Experimental Apparatus, Material and Programme	166
5.1		Introduction	166
5.2		Design Parameters for Apparatus	166
5.3		Principal Apparatus	170

	5.3.1	Closed Circuit Test Channel	170
	5.3.2	Upstream Reservoir	170
	5.3.3	Channel	174
	5.3.4	Downstream Reservoir	174
	5.3.5	Pump	176
5.4		Discharge Measurement	176
	5.4.1	Orifice Plate	176
	5.4.2	Turbine Flow-Meter	177
5.5		Water Depth Gauge	181
5.6		Pitot Tube	182
5.7		Shear Vane	182
5.8		Suspended Solid Concentration Measurement	184
5.9		Swelling Characteristic Apparatus	185
5.10		Recording/Photographic Apparatus	188
5.11		Clay Material	188
	5.11.1	Classification	188
	5.11.2	Grain Size Distribution	188
	5.11.3	Mineralogy	190
	5.11.4	Atterberg Limits	190
	5.11.5	Chemical Characteristics	194
	5.11.6	Soil Compaction Test	194
	5.11.7	Specific Gravity	194
	5.11.8	Shear Strength - Undrained Triaxial Tests	194
5.12		Modification of Soil Properties	196
	5.12.1	Sand Material	196
5.13		Development of an Experimental Programme	202
5.14		The Procedure of Work	202
	5.14.1	Preparation of Experiments	204

	5.14.2	Mixing	204
	5.14.3	Compaction	205
	5.14.4	Laying out the Model	208
5.15		Determination of Experimental Data	208
	5.15.1	Initial Soil Properties	208
	5.15.2	Test Procedure	210
5.16		Sample Calculation	214
		Notes from Series II, Test 53	
5.17		Conclusion	228
CHAPTER 6		Presentation and Discussion of Experimental Results	229
6.1		Introduction	229
6.2		Erosion	231
	6.2.1	Mechanism	231
	6.2.2	Movement of Particles	
	6.2.3	Removal of a Single Particle	
	6.2.4	Removal of a 'Floc'	239
	6.2.5	Mass Erosion	241
	6.2.6	Resulting form of the Bed	241
6.3		Formation of a Breach	246
6.4		Breaching of a Cored Dam	254
6.5		Effect of Temperature on Erosion	258
6.6		Effect of Suspended Solids on Erosion	258
6.7		Initiation of Erosion	259
6.8		Swelling	262
6.9		Effect of the Chemistry of the Eroding Water	270
6.10		Suspended Solids	272
	6.10.1	Unusual Forms of Suspended Solid v Time Graphs	283

6.11		Effect of Permeability on Erosion	283
6.12		Effect of Compaction and Moisture Content on Erosion	284
	6.12.1	Historical Development	284
	6.12.2	Experimental Results	285
6.13		Effect of Velocity of Flow on Erosion	290
6.14		Reduction in Shear Strength	290
6.15		Effect of Rainfall on Erosion	291
6.16		'Modified' Clay	298
	6.16.1	Cohesion and Moisutre Content	298
	6.16.2	Swelling/P.L./L.L.	298
6.17		Effect of Slope on Erosion	300
6.18		Erosion Rates	303
6.19		Models	313
6.20		Methods of Reducing Erosion Rates	313
6.21		Resistance to Erosion - Documented Evidence	314
6.22		Failure of a Dam - Sequence of Events	315
6.23		Dimensional Analysis	318
6.24		Conclusions	321
CHAPTER 7		Slope Protection	324
7.1		Introduction	324
7.2		Vegetation	324
7.3		Alternatives to Seeded Grass cover	328
	7.3.1	Protection methods in general	328
	7.3.2	Turf	329
	7.3.3	Reinforcements	329
	7.3.4	Graded Filters	330
	7.3.5	Maccaferri Gabions	331
	7.3.6	RENO Mattresses	331

	7.3.7	Asphaltic Concrete	332
	7.3.8	Concrete Facing	332
	7.3.9	Soil Cement	332
7.4		Suggested Protection Methods	335
	7.4.1	'Terram' (I.C.I. Fibres)	335
	7.4.2	'Paraweb' (I.C.I. Linear Composites Ltd.)	339
7.5		Indirect Methods of Slope Protection	345
	7.5.1	Fuse Plugs	347
7.6		Additional Protective Methods	352
7.7		Closing a Breach	354
7.8		Conclusions	354
CHAPTER 8		Hydraulic Considerations	356
8.1		Introduction	356
8.2		Hydraulic Considerations	356
	8.2.1	Peak Flows	356
	8.2.2	Actual Peak Outflows	357
8.3		Breach Dimensions	361
8.4		Erosion Retarding Layer	366
8.5		Discharge Coefficients for Breaches	367
8.6		Erosion Rates for Failure of Earthfill Dams	368
8.7		Design and Layout of Earth Emergency Spillways	370
8.8		Conclusions	374
CHAPTER 9		Conclusions and Recommendations	375
9.1		Introduction	375
9.2		Conclusions and Recommendations	375
	9.2.1	General	375

	9.2.2	Documentation	376
	9.2.3	Documentation of Failures	376
9.3		Safety Implications	377
	9.3.1	Embankment Behaviour	377
	9.3.2	Communication Network	377
	9.3.3	Interruption of Construction	377
	9.3.4	Access	378
	9.3.5	Crest	378
	9.3.5.1	Crest Widths	378
	9.3.6	Small Dams	378
	9.3.7	Control Equipment	380
	9.3.8	Vigilance	380
	9.3.9	Fuse Plugs	380
	9.3.10	Surface Features	381
	9.3.10.1	Berm Drains	381
	9.3.11	Waves	381
	9.3.12	Wavewall	381
	9.3.13	Rip-rap Protection to Downstream Face	381
	9.3.14	Erosion of the Different Zones within a Dam	383
	9.3.15	Warning Systems	383
	9.3.16	Industrial Lagoons	383
	9.3.17	Maintenance/Sabotage/Vandalism	384
	9.3.18	Grass	384
9.4		Erosion of Clay	385
	9.4.1	General	385
	9.4.2	Material Research	385
9.5		Recommendations for Further Research	389

LIST OF FIGURES

1.1 Construction Trends for Large Dams

1.2 Large Dams in the United Kingdom

1.3 Construction Trends for Large Dams in Great Britain - Earthfill

1.4 Construction Trends for Large Dams in Great Britain - Concrete Gravity

1.5 Construction Trends for Large Dams in Great Britain - Rockfill

2.1 Average Annual Cost for Differing Designs

3.1 Essential Mechanisms of Embankment Failure

3.2 Factors which may contribute to Overtopping

3.3 Examples of Types of Settlement which may lead to Overtopping

3.4 Number of Recorded Incidents versus Height of Dam (World Dams)

3.5 Longitudinal Section of Herrin Dam

3.6 Cumulative Distribution of a Sample of British Dams (Dams >22.7 t.c.m.)

3.6a Age Distribution of a Sample of British Dams

3.7 Year of Construction v. Number of Dams known to have required, Remedial Measures (British Dams)

3.8 Year of Remedial Measures v. Number of Dams (British Dams)

3.9 Year of Incident/Failure v. Number of Incidents/Failures (545 known Incidents/Failures throughout the World)

3.10 Year of Construction v. Number of Instances of Overtopping (376 Incidents throughout the World)

3.11 Age of Dam at Time of Incident/Failure v. Number of known Incidents/Failures (Incidents/Failures to 318 Dams throughout the World)

3.12 Age of Dam v. Number of Incidents/Failures (Incidents/Failures to 121 British Dams)

3.13 Year of Incident/Failure v. Number of Incidents/Failures (Incidents/Failures to 121 British Dams)

3.14 Year of Construction v. Number of Incidents/Failures (Incidents/ Failures to 74 British Dams)

3.15 Year of Construction v. Number of Incidents/Failures (227 Failures/Incidents throughout the World)

4.1 Sheet Silicate Minerals

4.2 Consistency Limits

4.3 Cation Bond

4.4 Dipole-Cation-Dipole Arrangement

4.5 Water Dipole Linkage

4.6 Diffuse Double Layer Theories

4.7 The Effects of Change in System Properties on Diffuse Double Layer

4.8 Clay Structure

4.9 Flocculation and Dispersion of Kaolinite in Water

4.10 Effect of Compaction on Structure

4.11 Dunn's Jet Test Apparatus

4.12 Influence of Antecedent Water on the Erodibility of a 10% Natural Silt-Grenada Silt Loam Mixture, packed at 8% Water to a 1.4 Bulk Density - after Grissinger

4.13 Effect of Ageing on the Erodibility of a Grenada Silt Loam packed at 10% Water to a 1.4 Bulk Density - after Grissinger

4.14 Influence of Temperature on Erodibility of Grenada Silt Loam with Admixtures and tested at 10% Water - after Grissinger

4.15 Variation of Erosion Rates with Bottom Shear Stress - after Partheniades

5.1 Flow over a Dam

5.2 Front Elevation of Closed-Circuit Channel Apparatus

5.3 Plan of Closed-Circuit Apparatus

5.4 'Sample Section' of Channel

5.5 Swelling Characteristic Apparatus

5.6 Particle Size Distribution for Clay Material

5.7 Dry Density/Moisture Content Relationship for Clay Soil Sample

5.8 Relationship between Cohesion, Moisture Content and Bulk Density for Clay Soil

5.9 Graph of Liquid Limit v. Percentage of Sand by Weight in Mix

5.10 Graph of Plastic Limit v. Percentage of Sand by Weight in Mix

5.11 Dry Density/Moisture Content Relationship for the Clay Soil Sample with various Percentages of Sand by Weight in Mix.

5.12 Dam Profiles used in the Experimental Programme

5.13 Mould Assembly

5.14 Velocity Measurement Positions

5.15 Suspended Solid Filtering Apparatus

5.16 Sub-Sample Cutter

5.17 Graph of Sampling Time v. Sediment Concentration

5.18 Graph of Time v. Swelling Movement

5.19 Model and Erosion Process for Series II, Test 53

6.1 Erosion Susceptibility in Relation to Water Velocity and Particle Size

6.2 Surface and Mass Erosion

6.3 Influence of Particle Size on Flow Conditions Around a Particle

6.4 Saltation

6.5 Erosion due to Protruding Stones

6.6 Erosion due to Small Stones and Local Depressions

6.7 Slope Stability

6.8 Erosion of the Upstream Face by Channelling

6.9 Erosion of the Toe

6.10 'Step' Formation

6.11a Stages in the Development of a Breach
and
6.11b

6.12 Typical Cracking to Clay Core, Restrained on Three Sides when Subject to Overtopping

6.13 Graph ov Mean Velocity v. Suspended Solids Concentration

6.14 Graph of Time v. Swelling - 0% sand

6.15 Graph of Time v. Swelling - 5% sand

6.16 Graph of Time v. Swelling - 10% sand

6.17 Graph of Time v. Swelling - 12% sand

6.18 Graph of Time v. Swelling - 20% sand

6.19 Graph of Percentage of Clay v. Swelling

6.20 Graph of Percentage of Clay v. Expansion (Keer & Raczkowski)

6.21 Graph of Suspended Solid Concentration v. Time γ 1885-1890, v = 2.7

6.22 Graph of Suspended Solid Concentration v. Time γ 1885-1890, v = 2.86

6.23 Graph of Suspended Solid Concentration v. Time γ 1885-1890, v = 2.99

6.24 Graph of Suspended Solid Concentration v. Time γ 1940-1950, v = 2.97

6.25 Graph of Suspended Solid Concentration v. Time γ 1940-1950, v = 2.97

6.26 Graph of Suspended Solid Concentration v. Time γ 1940-1950, v = 2.7-3.1

6.27 Graph of Suspended Solid Concentration v. Time γ 1940-1950, v = 3.24

6.28 Typical Suspended Solid v Time Graph (after Arulanandan)

6.29 Typical Suspended Solid v Time Graph

6.30 Graph of Initial Moisture Content v. Final Moisture Content
γ_B = 1885-1890 kg/m^3, v = 29-30 m/s

6.31 Graph of Initial Moisture Content v. Final Moisture Content
γ_B = 1940-1945 kg/m^3, v = 29-30 m/s

6.32 Velocity Distributions for Flow in Channel
Test Series I, No.1, $\bar{v}$ = 2.86 m/s, d = 14.6 mm

6.33 Velocity Distribution for Flow in Channel
Test Series I, No.27, $\bar{v}$ = 3.13 m/s, d = 15.8 mm

6.34 Variation of Vane Shear with Depth

6.35 Variation of Vane Shear with Depth

6.36 Moisture Content Distributions

6.37 Relationship between Cohesion, Moisture Content and Bulk Density for 'Modified' Clay

6.38 Erosion Rates for Various Slopes

6.39 Erosion Rates for Various Slopes

6.40 Moisture Content v Erosion Rates γ_B = 1840-1850kg/m^3 v = 2.38-2.42 m/s

6.41 Moisture Content v Erosion Rates γ_B = 1840-1850kg/m^3 v = 2.51-2.74 m/s

6.42 Moisture Content v Erosion Rates γ_B = 1840-1850kg/m^3 v = 3.0-3.15 m/s

6.43 Moisture Content v Erosion Rates γ_B = 1920-1940kg/m^3 v = 2.4 m/s

6.44 Moisture Content v Erosion Rates γ_B = 1920-1950kg/m^3 v = 2.63 m/s

6.45 Moisture Content v Erosion Rates γ_B = 1920-1940kg/m^3 v = 3.04 m/s

7.1 Soil Cement Facing

7.2 Vallon Des Bimes Dam

7.3 'Terram' - Pore Size Distribution

7.4 'Terram' used to Improve Frictional Stability

7.4a Schematic Diagram showing the Construction of a continuous downstream protection layer.

7.5 Interlacing through Weft and Warp

7.6 Bolted joints with stiffener or Eyelets

7.7 Rods and Bar Jointing

7.8 Nailed Joints

7.9 Stapled Joints

7.10 Crest Anchorage for Protection Methods

7.11 Ideal locality for a Fuse Plug

7.11a Longitudinal Cross Section Showing Sectionalised Construction for Progressive Failure

7.12 Marland's Buried Louvre

8.1 Graph of Depth of Breach v Width of Breach

8.2 Graph of Heigh of Dam v Width of Breach

8.3 Floods from Dam Failures

8.4 'Rectangular' Hydrograph

8.5 'Triangular' Hydrograph

8.6 Layout of Earth Emergency Spillways

9.1 Section through Crest

9.2 Berm Drains

LIST OF TABLES

1.1 Cost Comparisons for Various Types of Dams

1.2 Classification of 'Large' Dams built during the last 73 years according to Type

1.3 Large Dam Construction Data for China for the Period 1949-1972

2.1 Observed and Design Floods of Selected Sites

2.2 ICE Flood Design Standard Recommendations

2.3 Classification of Hazard Potential

2.4 Reservoir Size Classification

2.5 Performance Standard Classification

3.1 Analysis of Failures

3.2 Analysis of Failures

3.3 Examples of Loss of Life caused by the Failure of 'Small' Dams

3.4 Examples of Cascade Failures

3.5 Observed Velocities of Flood Waves at Selected Sites

3.6 Observed Velocities of the Flood Wave from the St. Francis Dam

3.7 Rainfalls exceeding 125 millimetres (5 inches) for the Period 1863-1967

3.8 Main Features of the Armando de Salles Oliveira and Euclides da Cunha Schemes

4.1 Values of Specific Surface for Some Common Clay Minerals

4.2 Soil Properties used in Evaluating Erosion of Cohesive Beds by Some Researchers

5.1 Estimated Velocities achieved during Overtopping - Bernoulli's Equation

5.2 Estimated Velocities achieved during Overtopping - Manning's Formula

5.3 Mineralogical Analysis fo Clay Fraction

5.4 Atterberg Limits

5.5 Water Soluble Chemical Analysis

5.6 Sand Properties

6.1 Suspended Solid Tests

6.2 Chloride Contents of Eroding Fluid

6.3 pH Values of Eroding Fluid

6.4 Vane Shear Results

6.5 Erosion Rates for Various Slopes

7.1 Some Properties of 'Terram' Fibres

7.2 Properties of Paraweb and Terram Duplex Structure

8.1 Reported Dimensions of Breaks in Dams which have been Overtopped - 1

8.2 Reported Dimensions of Breaks in Dams which have been Overtopped - 2

8.3 Estimated Times to Empty Reservoirs

LIST OF PLATES

3.1 Euclides da Cunha Project - Aerial View

3.2 Euclides da Cunha Projrect - Aerial View taken from Downstream

3.3 Euclides da Cunha Projrect - Detail of the Erosion on the Downstream Slope

3.4 Euclides da Cunha Project - Erosion caused by Overtopping on the Downstream Slope

3.5 Armando de Salles Oliveira Project - Aerial View

3.6 Armando de Salles Oliveira Project - Erosion caused by the Overtopping on the Left Abutment

5.1 Closed-Circuit Test Channel

5.2 Calibration of Channel

5.3 Turbine Flow-Meter

5.4 Turbine-Meter

5.5 Flow-Meter Revolution Counter

5.6 Swelling Characteristic Apparatus

5.7 Time Lapse Photography Equipment

5.8 Clay Particles

5.8(a) Clay Particles

5.9 Compaction and Sample Forming Apparatus

5.10 Series II, Test 53; Model before test

5.11 Series II, Test 53; Initiation of Erosion just below the Crest - 2 hours after start of test

5.12 Series II, Test 53; Erosion at the Toe of the Model - 20 mins. after start of test

5.13 Series II, Test 53; Erosion at the Toe of the Model - 40 mins. after start of test

5.14 Series II, Test 53; Erosion Pattern - 2 hours after start

LIST OF PLATES

6.1 Series I, Test 12 - Resulting Form of Bed

6.2 Series II, Test 56 - Resulting Form of Bed - 2 hours after start of test

6.3 Step Features on the Downstream Face of a P.F.A. Model

SECTION I

CHAPTER ONE

INTRODUCTION

The increasing demand for reliable water supplies, for domestic and industrial use, for irrigation, and for power, has required man to construct reservoirs in an attempt to control and improve his society.

The dam was probably one of the earliest structures developed by the human race, and it has become one of the more versatile and important of the structural servants of mankind. The earliest builders were probably the beavers whose craftmanship has long been recognised and envied. Their dams may leak, but they are structurally sound. Man found it difficult to emulate the design:

> "I built it like beavers, with sticks and rocks to reinforce it, with grass and leaves to make it tight, but it still broke" - the bulldozer operator turned contractor stared at the 6 metre wide gap in the 2.5 metre high dam. He repeated this excuse to angry residents who had been startled by a sudden flood, washing out culverts and roads, damaging property

for half a mile down the creek. He built
the dam under contract to a fishing pond,
all he had to show for his labour was
several acres of soft clay and some dying fish.'[1]

Engineering design and supervision must be provided for structures.

1.1 Historical Dams

What is probably the first record of a reservoir is found upon a Babylonian tablet, dating back to about 4000 B.C.[2] Reference is made to the effect that Eannudu, the reigning monarch of the time, constructed several canals, and near to one of these he made a small reservoir or tank; "A basin, 3600 gur of water" (about 130 cubic metres). Alternatively, reference is often made to Sadd-el-Kafara, or "Dam of the Pagans", as being the oldest known dam, constructed some time between 2950 B.C. and 2750 B.C., it is situated some 32 kilometres south of Cairo.[3]

Sadd-el-Kafara was essentially a masonry structure, consisting of two rubble masonry walls, 24 metres thick, spaced 36.0 metres apart; the space being filled with gravel from the bed of the wadi. The resulting structure had a crest length of 106 metres, a maximum height of 11 metres, a crest width of 61 metres, a base length of 81 metres and a base thickness of some 84 metres. The dam has been estimated as having contained 106,000 tonnes of material. The remains of the dam were discovered by the German archaeologist, G. Schweinfurth in 1885. Doubt exists as to whether the dam was built with a spillway, but it is certain that if a spillway was provided its capacity was inadequate.

Evidence exists to suggest that, over the catchment of 186.4 square kilometres, no runoff would occur unless each fall of rain exceeded 8 millimetres. Analysis shows that, in three years out of four, it is likely that a rainfall of more than 10 millimetres in one day will occur; and that once in every four years an individual fall of more than 20 millimetres could be expected.

From the remains of the dam the capacity of the reservoir has been estimated as 56500 cubic metres, and it is clear that a rainfall of 20 millimetres would have been just enough to fill the empty reservoir basin. A total fall in excess of 20 millimetres within a short period would certainly have overtopped the dam. This, probably the first historical failure, may well have deterred Egyptian engineers from similar undertakings for many years.

A further early dam is believed to have been built on the River Nile at Kosheish in Egypt, where a 15 metre high masonry structure was built in about 2900 B.C.. Egypt was one of the forerunners in dam construction because rainfall was scanty, with high summer temperatures. Other dams for irrigation existed from about 1000 B.C. within central Asia (now part of the USSR).

In the Far East, an earth dam 18 kilometres long and 21 metres high, and containing 13×10^6 cubic metres of earth, was built in Ceylon as early as 500 B.C., an impressive engineering feat for any age. Many similar dams exist today.[4]

Some dams in India are at least 1600 years old and remain in service. More than 20000 dams have been located in Madras state alone. The majority of these dams are of homogeneous construction, with

trapezoidal cross-section, and although failures occur at intervals due to inadequate spillway capacity, many are in a good state of repair.

The construction of dams remains a combination of science and art, albeit with a much increased emphasis on science as was previously the case. The properties of materials used in dam construction can now be evaluated, as can the behaviour of the completed structure. As might be expected, embankment dams were, and remain, the most common type of structure, although remains of masonry-concrete dams are said to have been found in the Anieni Valley, above Subiaco in Italy, where it has been proposed that the middle dam of three, constructed by Nero for recreational purposes, was a 'concrete' dam faced with masonry.[3]

1.2 Earthfill Embankments

The development of embankment design has resulted in ever increasing heights and a decrease in crest width.

Knowledge and experience have resulted in new design features such as internal cores, reduction in computed layer thickness, 'new' materials such as rockfill, and hydraulic fill dams, although the latter are now declining in popularity.

Embankment dams continue to be built in the largest numbers, primarily because of their inherent advantages. The main advantage of the earth embankment is that local material may be used for construction and therefore not involve the high cost of importing material. This, unfortunately, leads to economic pressure to use local material whether suitable or not. Earth embankments are less demanding with respect to foundation requirements, important when one considers that most of the

more suitable sites for dams have been used. Embankment dams can easily be raised, facilitating two stage construction programmes if desired. Flexibility of design and construction is an additional advantage.

Embankments are usually more economical to construct than other types of dams, primarily because the unit cost of earthfill and rockfill is considerably less than the unit cost of cement, but this is partially offset by the volume of material used in an earth dam being considerably larger than that used in a concrete dam. In addition, rapid development in the capacity of earthmoving machinery and efficiency of compaction equipment has kept the unit cost of earthfill material essentially the same, in fact, as is shown in Table 1.1, there is a tendency for the unit cost of earthfill and rockfill to decrease in absolute terms. The figures in Table 1.1 have been amended to compensate for inflation and therefore enable a direct comparison to be made, although consideration must be given to the dam's location.[5] It is also evident that a discontinuous construction process and the high cost of skilled labour is causing a rapid rise in the cost of concrete dams.

Approximately 60 per cent of the world's 'large' dams are earthfill structures. The construction trend of 'large' dams for selected countries during the period 1900 to 1973 is shown in Table 1.2, showing the dramatic decline in the popularity of the gravity dam.[6] It can be seen from these statistics that, generally, the rate of earthfill/rockfill dam construction is relatively constant, and in many instances actually increasing. In contrast, the rate of concrete dam construction is declining. This is particularly marked in the case of the United States of America, France, Canada and Italy.

TABLE 1.1: Cost Comparisons for Various Types of Dam

Constr. Date	Name	Original Constr. Price £/m^3	Amended Constr. Price (1978) £/m^3	Type
	'Average' values for HEP Reservoirs Scotland	£4-£5	£9.4	E+R
1965	Balderhead	£1.3	£2.8	E
1954	Shira-Lower West	£4.5	£10.0	E
1960	Breaclaigh	£3.3	£7.2	R
1976	Kielder	-	£6.0	E
1954	Shira-Lower East	£16.4	£39	CG
1962	Nant	£9.8	£20	CG
1966	N.A.	£10	£29	B
1978	Airy Holm	-	£16.5	CG+E
1976	Castlehill	-	£31.6	AG

E - Earthfill, R - Rockfill, CG - Concrete Gravity, B - Buttress,

AG - Arch Gravity

HEP - Dams belonging to North of Scotland Hydro-Electric Board

Country	Period	Numbers				Percentage of Total			
		Earthfill Rockfill	Concrete Gravity	Buttress	Arch and Multi-Arch	Earthfill Rockfill	Concrete Gravity	Buttress	Arch and Multi-Arch
Great Britain	1900-1909	39	71	0	0	78	22	0	0
	10- 19	14	6	0	0	70	30	0	0
	20- 29	14	4	0	1	74	21	0	5
	30- 39	24	8	0	5	65	22	0	13
	40- 49	10	5	1	0	63	31	6	0
	50- 59	24	31	8	1	38	48	12	2
	60- 69	28	24	5	3	47	40	8	5
	70- 73	7	2	4	0	54	15	31	0
Canada	1900-1909	3	4	0	0	43	57	0	0
	10- 19	7	13	7	0	26	48	26	0
	20- 29	12	45	4	2	19	71	6	0
	30- 39	7	25	1	0	21	76	3	0
	40- 49	20	24	2	0	42	50	4	4
	50- 59	88	47	2	2	64	34	2	0
	60- 69	78	31	3	0	68	27	3	2
	70- 73	38	3	0	0	93	3	0	0
France	1900-1909	3	13	0	0	19	81	0	0
	10- 19	0	8	0	0	0	100	0	0
	20- 29	1	24	0	3	0	86	0	11
	30- 39	4	20	0	8	12	63	0	25
	40- 49	4	13	1	15	12	39	4	45
	50- 59	9	12	1	43	13	18	3	66
	60- 69	35	10	5	18	51	15	7	27
	70- 73	17	1	0	6	71	4	0	25

TABLE 1.2: Classification of 'Large' Dams built during the last 73 years according to type

Country	Period	Numbers				Percentage of Total			
		Earthfill Rockfill	Concrete Gravity	Buttress	Arch and Multi-Arch	Earthfill Rockfill	Concrete Gravity	Buttress	Arch and Multi-Arch
USA	1900-1909	104	46	6	5	65	29	4	2
	10- 19	157	87	26	29	53	29	9	9
	20- 29	204	86	18	62	55	23	5	17
	30- 39	263	51	13	41	71	14	4	10
	40- 49	245	45	4	16	79	15	1	5
	50- 59	750	89	0	20	87	10	0	3
	60- 69	1746	53	1	27	96	3	0	1
	70- 73	558	19	0	3	96	3	0	1
Japan	1900-1909	29	4	0	0	90	10	0	0
	10- 19	87	9	0	0	91	9	0	0
	20- 29	124	40	4	1	73	24	2	1
	30- 39	149	58	3	1	71	27	1	<1
	40- 49	112	51	0	0	69	31	0	0
	50- 59	124	152	2	8	43	53	1	3
	60- 69	138	162	8	34	40	47	3	10
	70- 73	41	31	1	2	55	41	1	3
Italy	1900-1909	0	5	0	3	0	63	0	37
	10- 19	5	7	0	3	33	47	0	20
	20- 29	12	41	0	23	16	54	0	30
	30- 39	3	41	0	6	6	62	0	12
	40- 49	0	22	0	4	0	85	0	15
	50- 59	15	35	1	3	27	65	2	6
	60- 69	18	14	0	0	56	44	0	0
	70- 73	21	12	0	0	64	36	0	0

TABLE 1.2 (continued)

Only in Spain is it found that three times as many concrete as earth dams are being constructed, whereas in America there are one hundred times as many earthfill dams as concrete dams, and twenty times in India.

Also shown in Table 1.2 are the numbers of each type of dam constructed. It can be seen that particular countries, through influence of topography and climate, 'favour' different types of construction, but that generally the earthfill/rockfill embankment construction prevails. Construction trends for selected countries are shown in Figure 1.1.

The popularity of earth dams in China is reflected in the construction figures between 1949 and 1972, tabulated in Table 1.3. Factors involved here are the use of local material and cheap mass labour.[7]

Historically, the demand for water for the canal system in Great Britain led to a large number of small reservoirs and the first few large dams being built at the end of the eighteenth century. With the first Industrial Revolution reservoir construction expanded rapidly.

The number of large dams in Great Britain, according to the World Register[6] is shown in Figure 1.2. Superimposed on this diagram is the proportion of dams which are of earthfill construction, approximately 73 per cent of British large dams. This relates closely to the figure of 70 per cent of the world's large dams being of earthfill and rock-fill construction. It can be seen that the rate of large dam construction in Great Britain ranged from one per decade in 1800-1809 to fourteen per decade in 1940-1949, with a peak of forty-six in the

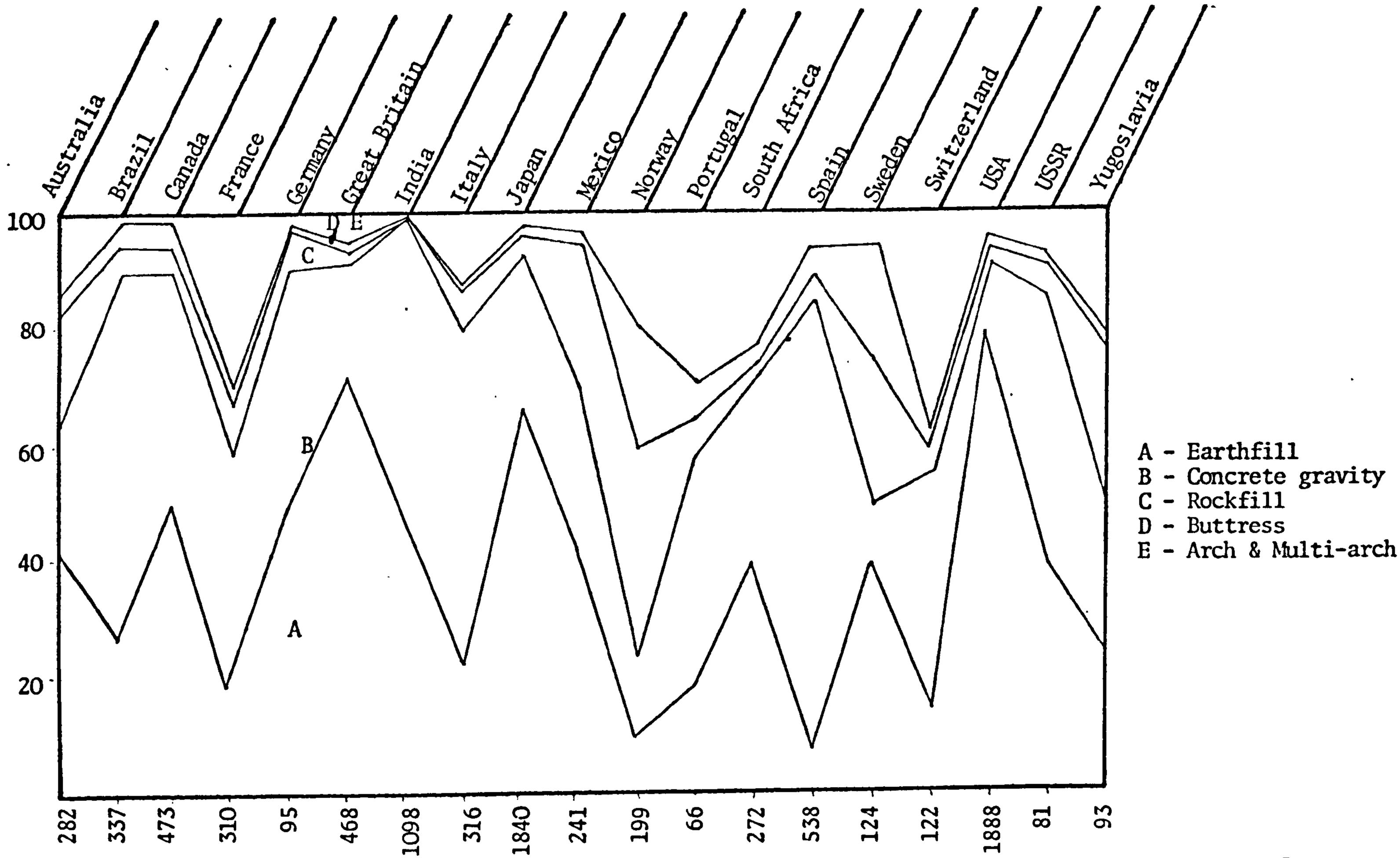
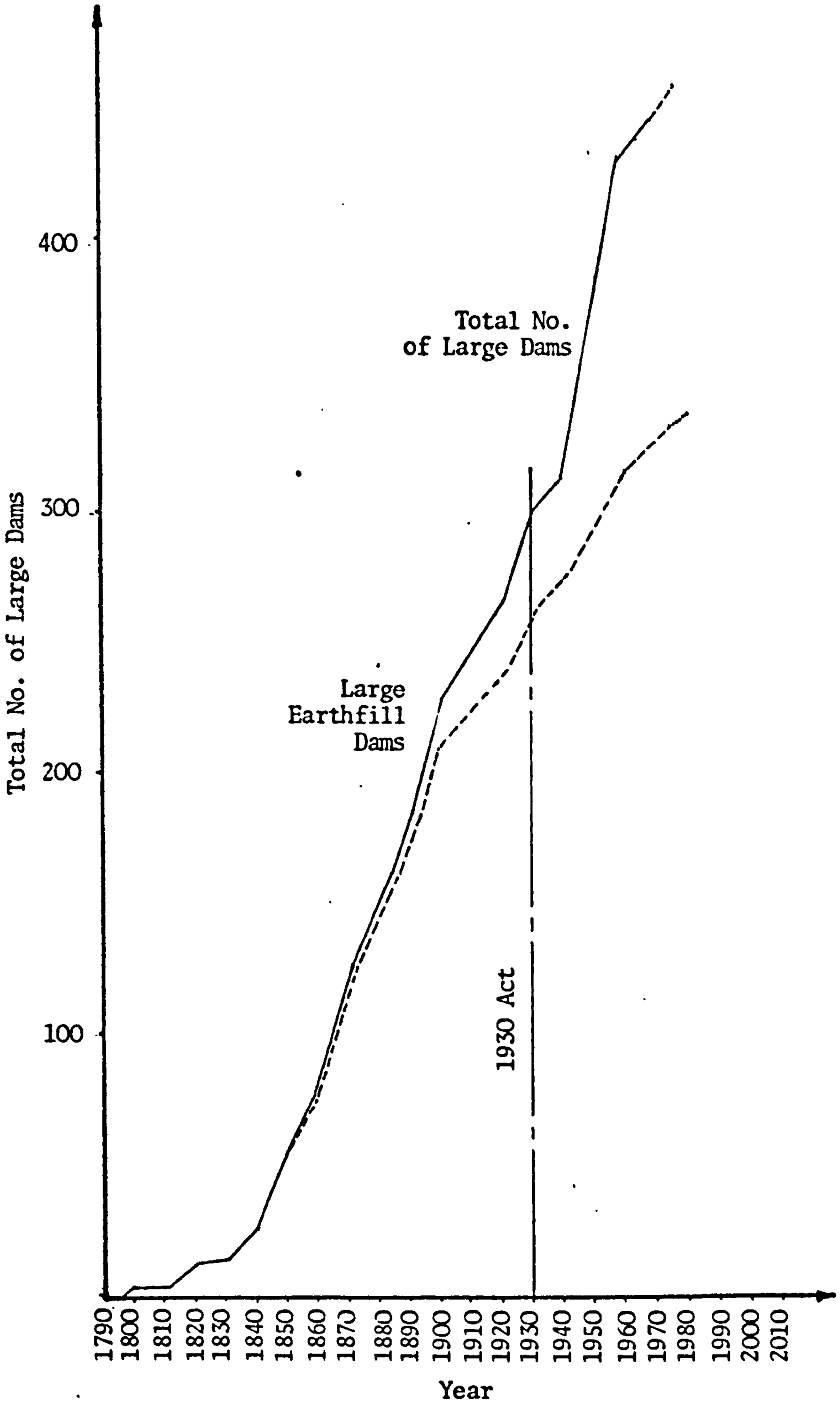


FIGURE 1.1: Construction Trends for Large Dams (compiled from 1973 WRD)

TABLE 1.3: Large Dam Construction Data for China for the Period 1949-1972[7]

		Number
By Type	Earthfill	11877
	Rockfill	129
	Masonry	438
	Concrete	73
By Height	15-30 m	11321
	30-60 m	1150
	>60 m	46
By Capacity	Large >100 x 10^6 m^3	254
	Medium 10 x 10^{-6} - 100 x 10^6	1307
	Small	10956
Total		12517

FIGURE 1.2: Large Dams in the United Kingdom



decade 1870-1880. It is interesting to note that until 1841 all the large dams were of an earthfill construction.

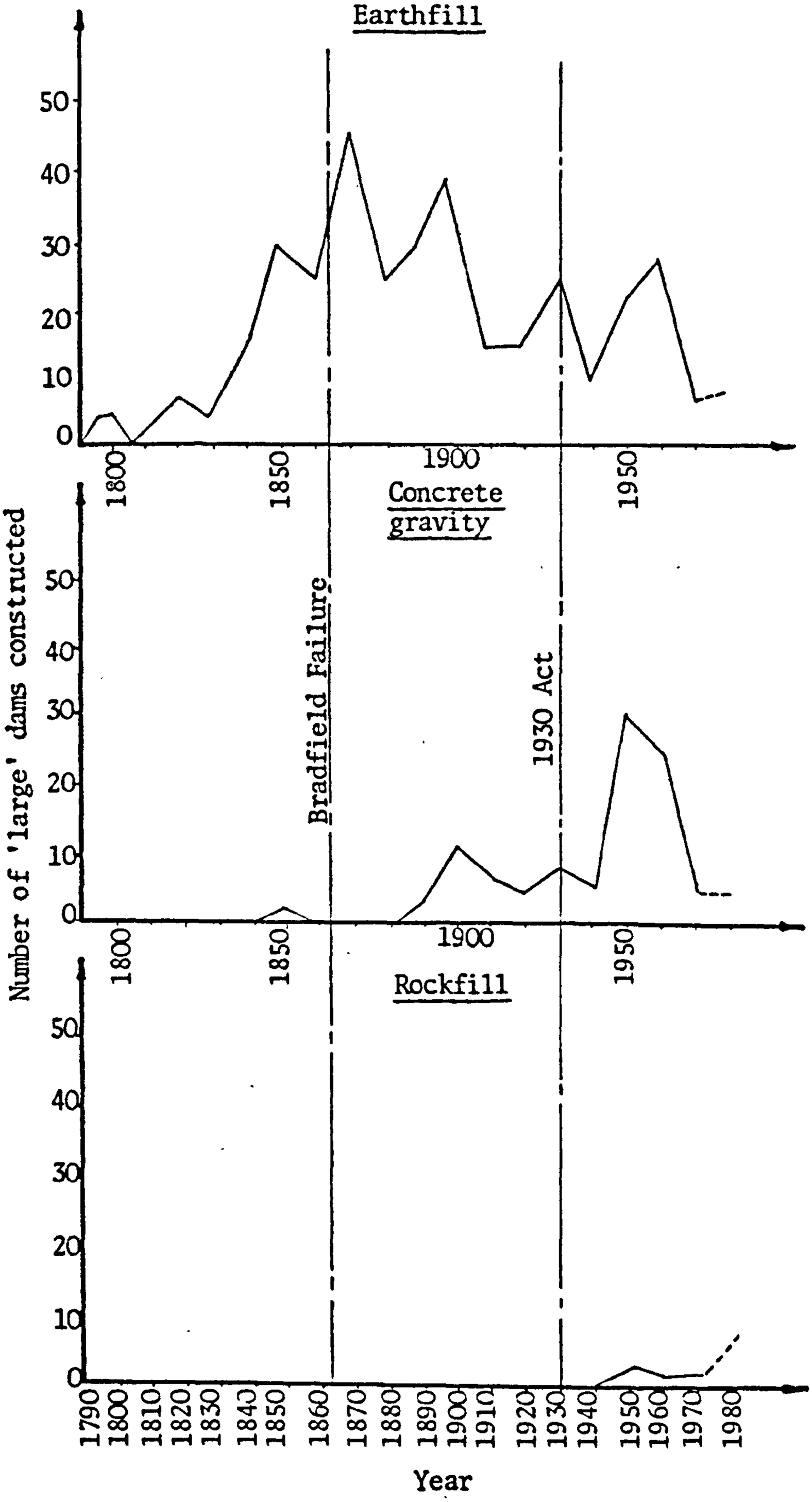
The construction trends for the various types of 'large' dams in Great Britain for the period 1794 to 1973 are shown in Figures 1.3-1.5.

Industrial demand, and the increasing demand for an adequate public water supply led to an accelerating rate of growth, both in size and number,of dams in Great Britain from about 1840.

Great Britain has been constructing large dams since the early eighteenth century - it is believed the first large dam built in this country was March Haigh, near Huddersfield (1794).[8] The present total of large dams listed for Great Britain in the ICOLD Register, i.e. those exceeding 15 metres in height, or between 10 and 15 metres if the volume of water stored is in excess of 100000 cubic metres, is 464, of which over 40 per cent date back to the nineteenth century or earlier, and at least 65 per cent were constructed prior to World War II. Britain is not, by world standards, a country of big dams or large reservoirs, but it is an interesting thought that in terms of numbers Britain still ranks high in the world lists and leads in Europe.

The World Register gives very little indication as to the total number of dams in a particular country. While the Register lists 4725 large dams in the United States, it is known that some 40000 dams exist. Similarly, the Register lists 725 large dams for India when at least some 43000 of varying sizes exist. The most reliable information on reservoirs in the United Kingdom indicates a total of approximately 5000 reservoirs in existence as of 1971, although this figure does not include reservoirs in Northern Ireland.[9]

FIGURE 1.3-1.5: Construction Trends for Large Dams in Great Britain



In Britain, as knowledge of new materials increased small numbers of other types of dams were constructed; Abbeystead (1881 - masonry), Loch Katrine (1859 - concrete). It is also true that it is only in recent years that the satisfactory design criteria for rockfill/earth-fill have been, and continue to be, established.

Knowledge of soil mechanics, pressure grouting, instrumentation, heavy earth moving plant and compacting plant have all improved. The importance and existence of uplift pressures was only realised at the end of the nineteenth century, grouting procedures have been established in the last 40 years while foundation drainage is of more recent origin. As a result design criteria have changed radically over the years, and even now there remain many assumptions, approximations and unknowns, resulting in problems in attempting to evaluate the safety of old dams.

1.3 The Development of Safety Regulations for Dams

The first legislation relating to dam safety in this country appeared, indirectly, within the "Waterworks Clauses Act" (1863)[10] This Act made provision that if a person considered a reservoir to be in a dangerous condition and complained to two justices, then an inspection was carried out and, if necessary, the justices could make an order to lower the water in the reservoir. It is believed that this provision of the Act was never called into effect.

The series of the Eigiau and Coedty dams on November 2nd, 1925, finally led to the passing of specific legislation on safety, the Reservoirs (Safety Provisions) Act (1930).[11]

Following disasters overseas, and consequent repercussions within the profession, the Institution of Civil Engineers (I.C.E.) constituted a committee to assess the situation and make recommendations. Some of the recommendations for modifying the 1930 Act, reported in 1966[12] were adopted, others were not. In due course a new Reservoirs Act (1975)[13] received Royal Assent. This Act, however, will only be brought into force upon a date to be prescribed by the Secretary of State for the Environment, and at time of writing the 1930 Act applies.

CHAPTER TWO

THE PROBLEMS OF FLOOD FORECASTING

2.1 Introduction

The chapter reviews current flood forecasting methods and, by example, shows the need for re-evaluation of the adequacy of some spillway capacities.

In addition the development of flood estimation recommendations by the Institution of Civil Engineers and by the American Society of Civil Engineers is outlined.

With the large number of earth and rockfill dams requiring spillways for the safe passage of flood flows, and with many concrete dams not designed for extreme overspill conditions, more thought and study is being given to the investigation of run-off characteristics and to flood estimation, including floods of theoretical maximum intensity. The repercussions of such developments in terms of spillway sizing can be very significant, and can introduce non-technical considerations of risk philosophy.

Even now, experience in estimating flood potentials shows that no statements of absolute certainty can be made about future flood events. There must always remain some finite degree of risk that the design will be inadequate.

2.2 Re-evaluation of Spillways

It is only in the last 25 years that the quality of available data, and the advances in meteorology and hydrology have permitted re-evaluation of spillway capacities on a rational basis as opposed to empirical rules. Many agencies are therefore reviewing the adequacy of their dams, particularly older, i.e. pre 1950, structures.

The need to do this has been indicated by figures released by several enforcement agencies. In the state of Maine in the United States of America, of more than 1100 dams inspected in 1975, only some 200 attained an adequate standard for spillway capacity. A study of repairs to dams in California has indicated that 60 per cent of repairs have resulted from defective spillways.[14]

In 1966, it was reported[15] that the state of Missouri had 1300 reservoirs possessing a surface area 20000 square metres or larger. Many of these reservoirs were built with the intention of getting the greatest storage capacity at the lowest cost and even designed and constructed without qualified professional advice. Since 1958 Missouri has experienced 17 failures, mostly due to overtopping and generally affecting dams in the 6 metre to 9 metre height range. It was also reported that 39 further dams were considered to be in danger of failure, including a 21 metre high dam with a spillway only 0.6 metres below the crest which was blocked with debris, and a second dam at risk in another part of the state, which could create a cascade failure.[15]

Many early failures were due to an absence of reliable rainfall and runoff data and the benefits of analytical analyses coupled with a failure to recognise the dangers of unprecedented precipitation.

Records of runoff and rainfall data that did exist at the time of construction were often gained over a record period falling far short of those required for meaningful analysis, even by the techniques then available.

Many of the elderly dams in the United States and in this country were built to pass a comparatively small 'design flood' of perhaps 1 in 50 years. Since many of the dams in the USA were built between 1910 and 1920 and much earlier in this country, this has frequently been exceeded. Spillways of many of the dams do not meet today's standards primarily because the meteorological and hydrological records on which their design was based was inadequate. Re-appraisal of the adequacy of such spillways is in many cases taking a high priority in the maintenance and safety programmes of the larger authorities.

It is an understandable, but nevertheless disturbing, fact that many dams in this country have required remedial measures to increase freeboard to an adequate level. Of one hundred embankments studied by Banks[16] approximately 50 per cent were deficient in spillway capacity, while another 10 per cent were deficient in other respects.

The United States Bureau of Reclamation (U.S.B.R.) have been re-evaluating many of their dams since about 1974. Of 54 studied, 10 were found to require no increase in spillway capacity, but the remaining 44 were found to require an increase in capacity ranging from 10 per cent to a maximum of 300 per cent with an average of 90 per cent.[17] The U.S.B.R. then adopted the position that since they had used the most favourable, i.e. least severe, methods of analysis, it was reasonable to assume that their study was fairly representative on

a national scale. They also indicated that attention should particularly be directed to pre-1950 dams.

South Africa has also initiated a re-evaluation programme and over some seventy dam sites found that the probable maximum flood (PMF) varied from 1.8 to 11.0 times the 1 in 100 year flood, with an average of 6.0 times the 1 in 100 year flood generally used in their designs.[14]

2.3 Flood Forecasting Methods

In general the procedures for fixing spillway design floods can be divided into two categories:

1. Deterministic
2. Probabilistic

Methods currently used for flood forecasting and for predicting spillway design floods are:

1. Unit Hydrograph: A method found to be particularly good for catchments with an area between 100 square kilometres and 5000 square kilometres.

2. Routed Hydrograph: A method used for catchments with an area in excess of 5000 square kilometres and involving the use of hydrographs derived from sub-catchments, which are then lag-routed to the dam.

3. Statistical Analysis: This method utilises the mean annual precipitation and statistical techniques to calculate PMF's.

4. Regionalisation of the Moments of Frequency Distribution Functions: Flood frequency estimates are obtained by the combination of any available observed data with relationships developed between catchment characteristics and the moments of the assumed frequency distribution.

5. Experience Diagram: Maximum recorded floods are plotted against size of catchment. Creager envelopes are also drawn on the diagram for various Creager ratings. This provides a method whereby recorded maximum floods can be compared with computed values.

Since 1940 the U.S.B.R. have used the unit hydrograph method for flood estimation. Prior to that, an estimate of the required spillway capacity was obtained by applying a 'judgement' factor to an envelope curve of peak discharges which have occurred in the same general vicinity, in watersheds with similar run-off characteristics. With few exceptions, the U.S.B.R. has found that the unit hydrograph method gives larger estimates of floods. Since approximately 100 large U.S.B.R. dams were built without the use of the unit hydrograph method prior to 1950, these are now being re-appraised, as indicated earlier.

Flood design and risk, both in construction and in the operation of reservoirs, is a compromise between design, economy and responsibility. A purely hydrometeorological approach is, however, deficient from the economic viewpoint. The flood design value obtained from such analyses is frequently of such a magnitude that if used for spillway design sizing, the resultant design might be quite uneconomic. This has resulted in a tendency to over-emphasise the part played by experience and judgement in attempting to modify the design value to economically and structurally acceptable limits.

2.4 Lack of Data

Lack of data, especially in those under-developed countries where no data may be available, results in the situation where an engineer is still expected to make an accurate prediction of extreme flood, which in the extremes leads to either uneconomic overdesign or unacceptable implied risk. Any calculation of this type is a balance between safety and economy, and even when data is available the best that can be achieved is to minimise the possibility of a failure to an acceptably remote value; the risk cannot be eliminated. Hydrographic information is never conclusive, but is always a function of cumulative probabilities.

2.5 Estimated and Observed Floods

The magnitude of anticipated floods even after exhaustive study of available data has frequently proved to fall far short of subsequent occurrences. It could be argued that any flood which exceeds the capacity of a spillway system could not be expected on the sole basis of the existing records and probabilistic theory, and in many developing countries an estimate of design flood is based on incomplete data of past floods. However, a flood described as having a statistical return period of 1 in 10000 years could arrive next year and the year after and not return for a further 30000 years.

Values for design floods and observed floods at several schemes are shown in Table 2.1.

It is interesting to note that the flood experienced at the Rincon de Bonete scheme has been estimated as having a return period of 1 in

Country	Scheme	Year of Flood	Design Flood (m^3/sec)	Observed Flood (m^3/sec)	Ratio $\frac{\text{Observed Flood}}{\text{Design Flood}}$
Algeria	El Habra	1927	900	5000	5.6
Italy	Molare	1935	800	2200	2.8
USA	Bartlett (Verde River)	1938	1728	3398	2.0
India	Kaddam	1957	7000	15064	2.2
Uruguay	Rinconde Bonete	1959	9500	16800	1.8
Brazil	Oros	1960	2000	2250	1.1
India	Kharagpur	1961	560	868	1.6
South Africa	Nahoon	1970	700	1875	2.7
South Africa	Hluhluwe	u/c	1590	2950	1.9

TABLE 2.1: Observed and Design Floods at Selected Sites

500000 years.

Many 'flashy' rivers present additional problems when attempting to establish design floods. Data for the Djerdap project, near the Iron Gate, on the River Danube, was collected over 50 years and gave the following results:[17]

Minimum Flow (m^3/sec)	Maximum Flow (m^3/sec)	Mean Flow (m^3/sec)	Ratio of Mean Flow to Maximum Flow
1040	10600	5520	1.9

The corresponding results for the Pasco de los Torres scheme on the Rio Negro, Uruguay, gauged over a period of 29 years are:

Minimum Flow (m^3/sec)	Maximum Flow (m^3/sec)	Mean Flow (m^3/sec)	Ratio of Mean Flow to Maximum Flow
1	5460	490	11.1

The Kaddam scheme in India was designed for a maximum flood of 2500 m^3/sec. During construction a flood of 3556 m^3/sec was experienced and the design was consequently uprated to 7000 m^3/sec. In the event, the dam failed when a flood of 15064 m^3/sec was experienced soon after completion.[17]

Avon dam, Sydney, Australia, a gravity structure built in 1927, was designed to pass a flood of 765 m^3/sec. The design was re-assessed after a 24 hour storm resulted in a flood of 1130 m^3/sec during construction.[18]

The hydrological safety of earth dams bears no relation to elapsed time. It observes the laws of probability. However, the scale of damage downstream, caused by overtopping, is increasing as societies develop, and the margin of safety must be acceptable in terms of life and assets downstream.

2.6 Re-evaluation of Spillways

The Institution of Civil Engineers Floods Working Party,[19] and the American Society of Civil Engineers (A.S.C.E.) Committee on the Re-evaluation of the Adequacy of Spillways of Existing Dams[20] have studied the problem of choice of design floods. Both groups recommended that flood protection for reservoirs whose failure would endanger human life downstream should be assessed according to the number of lives at risk.

2.6.1 Institution of Civil Engineers' Recommendations

A study of floods in the United Kingdom and Northern Ireland has been reported in the National Environmental Research Council's (NERC) Flood Studies Report (1975)[21] and provides the basis for improved flood estimates, particularly in ungauged catchments. This Report supersedes the empirical enveloping curves of flood intensity used in the Institution of Civil Engineers' 1933 Report "Floods in Relation to Reservoir Practice"[22], and its reprint in 1960, and made a complete revision of these reports imperative.

The 1933 paper dealt only with reservoirs in upland areas and giving design standards and all necessary tables and figures required for design.

The NERC Flood Studies Report is based on the principle that for any catchment there is a maximum flood which will not be exceeded, and that its magnitude can be estimated. The Report proposes methods for determining the estimated maximum flow (EMF) and lesser flows, Q(T), with a return period T.

The Reservoir Flood Standards discussion paper[23] gives a set of reservoir safety standards, which are supplemented by the publication 'Floods and Reservoir Safety - an Engineering Guide'.[24]

The Institution of Civil Engineers recommended that maximum protection would be required where lives in a community would be in danger, and in this case the dam would be required to pass the PMF (Possible Maximum Flood). If the endangered lives are not in a community, that is only isolated properties occupy the downstream flood plain, the design flood drops to one half the PMF, or the one in 10000 year flood, whichever is larger. To avoid overdesign, and costly remedial measures, where there is little risk to life, two more reservoir conditions permit reductions of the design flood to one in 1000 years (or 0.3 PMF) and one in 150 years (0.2 PMF) depending on the degree of damage that could be caused by breaching. Relaxation of the general standard is permitted if the inspecting engineer considers an occasional, brief overtopping to be tolerable, but it is thought that this will only be applied to concrete and masonry dams and possibly to well protected (i.e. paved) embankments. The Institution of Civil Engineers' recommendations are shown in Table 2.2.[25]

Another option available to the inspecting engineer for certain dams (categories 'B' and 'C', Table 2.2) is a least cost analysis. This approach was also used by the ASCE committee. An economic analysis

Category	Dam Design Flood Inflow			Concurrent Wind Speed and Minimum Wave Surcharge	Initial Reservoir Condition
	General Standard	Minimum Standard if Rare Overtopping is Tolerable	Alternative Standard if Economic Study is Warranted		
A. Reservoirs where a breach will endanger lives in a community	Probable Maximum Flood (PMF)	0.5 PMF or 10000 yr flood (take larger)	Not applicable	Winter: Max. hourly wind once in 10 years. Summer: As category C, D. Run-up 0.6 m.	Spilling long-term Average daily inflow. (see footnote)
B. Reservoirs where a breach (i) may endanger lives not in a community (ii) will result in extensive damage	0.5 PMF or 1000 yr flood (take larger)	0.3 PMF or 1000 yr flood (take larger)	Flood with probability that minimises spillway plus damage costs. Not to be less than minimum standard but may exceed general standard.		Just full
C. Reservoirs where a breach (i) will cause negligible risk to life (ii) will cause limited damage	0.3 PMF or 1000 yr flood (take larger)	0.2 PMF of 150 yr flood (take larger)		Average annual max. hourly wind. Run-up 0.4 m.	No spill (see footnote)
D. Special cases where no loss of life can be foreseen as a result of a breach and very limited additional flood damage will be caused	0.2 PMF or 150 yr flood	Not applicable	Not applicable	Average annual max. hourly wind. Run-up 0.3 m.	Spilling long-term Average daily inflow.

Footnote: Where reservoir control procedures require, and discharge capacities permit, operation at or below specific levels defined throughout the year, these may be adopted, providing they are specified in the certificates or reports for the dam.

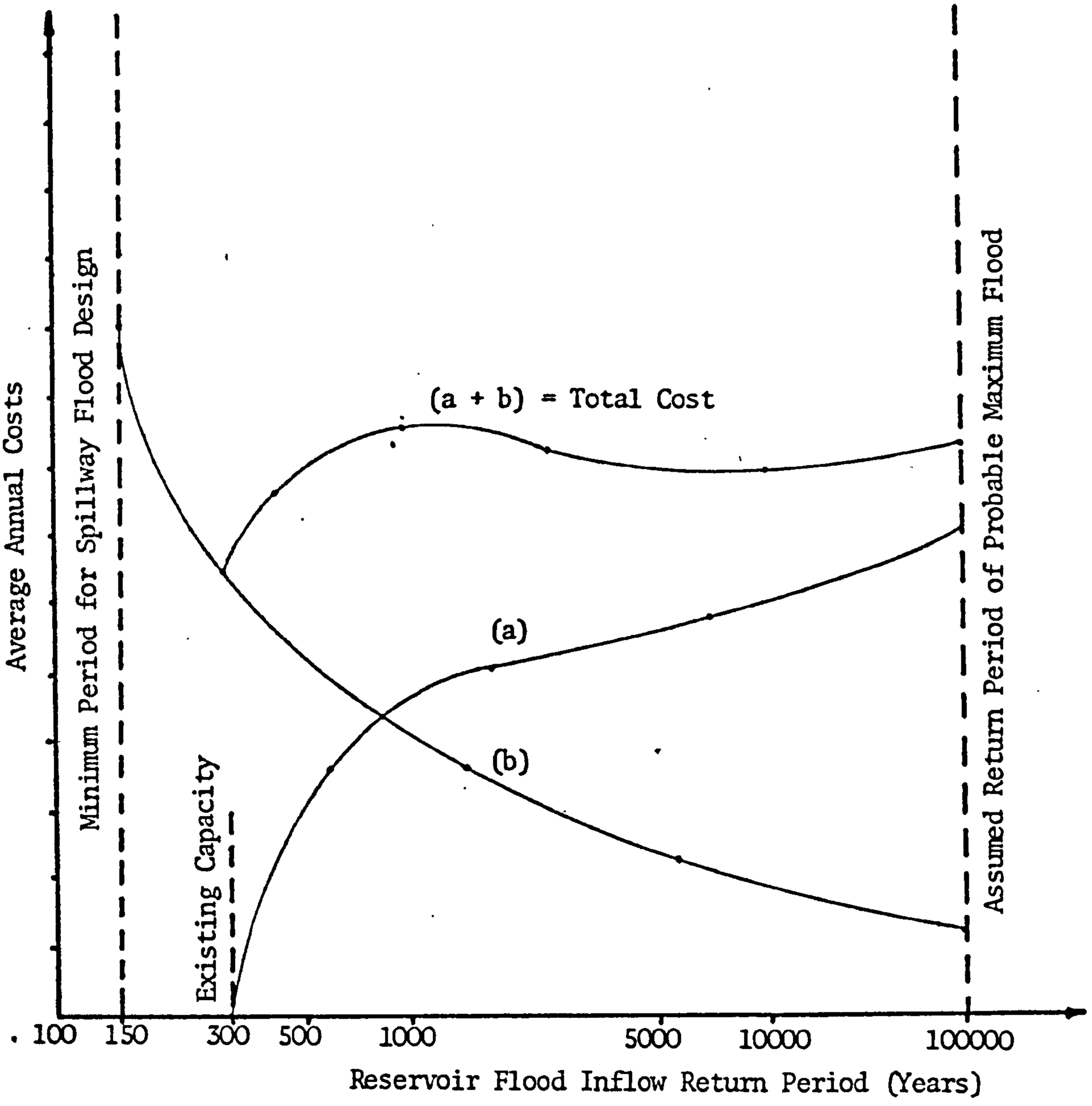
TABLE 2.2: I.C.E. Flood Design Standard Recommendations[25]

linking the cost of improving a dam's spillway with the damage to be expected from floods greater than the new capacity. As a result, there is an attempt to adopt an evaluated level of risk at which the total of all social costs will be a minimum. The social costs, in this context, are composed of two entities: the cost to provide safety and the cost which measures the magnitude of risk from lack of safety. A typical graph produced by an economic analysis is shown in Figure 2.1.[19]

The Institution of Civil Engineers' publication 'Floods and Reservoir Safety: an Engineering Guide'[24] is intended to assist those individuals who bear the personal responsibility that comes from being appointed to the statutory panel of engineers qualified to design and inspect impounding reservoirs. An attempt is made to define standards on a broad basis to cover all major points of principle and to give sufficient guidance to avoid excessive discrepancies in the conclusions reached by differing engineers when reviewing the safety of an existing dam.

Reservoir flood protection standards proposed include recommendations for assumptions to be made for the three main factors to be considered when defining the initial reservoir level, the flood inflow and concurrent wind speed. Reservoir flood and wave standards are tabulated and guidelines are given which enable a dam to be categorised according to the risk involved and a dam design flood inflow chosen.

The Guide identifies the main steps in any unit hydrograph method whether precisely following the Flood Studies Report approach[21] or otherwise and indicates the differences between estimating a flood of



(a) = Modification Costs = average annual costs of capital works to increase existing spillway capacity and/or dam freeboard

(b) = Risk Costs = annual flood damage cost caused by floods exceeding the design flood

FIGURE 2.1: Average Annual Costs for Differing Designs[19]*

given probability or the Probable Maximum Flood by the Flood Studies Report.

In addition a method for the rapid assessment of the ability of an existing dam to cope with the combination of flood, waves and initial reservoir level is presented.

Comments on the decisions and assumptions that are required at the outset of a routing calculation or when reviewing initial results are made.

Wave surcharge and dam freeboard are discussed and a method outlined for estimating wave surcharge allowance, i.e. the theoretical wave freeboard sufficient to prevent overtopping reaching quantities that could threaten the dam crest.

Finally recommendations for the risks taken over limited periods, such as during construction periods, are made.

The Working Party on Floods and Reservoir Safety have recommended that after ten years' experience has been gained by panel engineers in using the guide its contents should be reviewed and, if deemed necessary, it should then be revised.

2.6.2 American Society of Civil Engineers' Recommendations

The ASCE Committee recommended similar measures but based on reservoir size, a hazard rating, and the possible loss of life involved. These requirements are shown in Tables 2.3, 2.4 and 2.5.[26] The Committee have suggested arbitrarily setting the PMF equal to the 10000 year flood.

In this country and in the United States the potential loss of life that would result from a breach caused by overtopping is frequently such that the 'calculated risk' approach cannot be accepted, and the PMF flood is adopted for design.

Even with these recommendations, there is always some possibility that the design will prove inadequate.

The losses and lost values for the dam and its associated structures can be calculated and may include, especially in the case of a hydro-electric power scheme, the cost of loss of service during the flood and also while under reconstruction. In addition, damage occurring downstream can be evaluated once the downstream flood wave characteristics have been defined by the use of transient flow techniques, i.e. Di Marchi's Formula?[27]To put a financial figure on possible loss of life and injury is extremely difficult and moves into the realms of 'social' and 'political' values.

When the spillway capacity of a dam has been re-evaluated, which is usually done by routing the new inflow through the reservoir on the basis of the most advantageous operating procedures and allowing maximum acceptable encroachment on the dam's freeboard with regard to loading conditions, and the spillway still proves to be inadequate,

Size	Loss of Life	Economic Loss
Low	No loss of life	Minimal economic loss
Significant	Few	Appreciable economic loss
High	Greater than few	Excessive economic losses

TABLE 2.3: Classification of Hazard Potential[26]

Size	Storage (acre feet)	Height (feet)
Small	<1000	<40
Intermediate	1000-50000	40-100
Large	<50000	>100

TABLE 2.4: Reservoir Size Classification[26]

Size	Hazard Rating	Design Flood
Small	Low	50-100 yr flood
	Significant	100-½ PMF
	High	½ PMF-PMF
Intermediate	Low	100-½ PMF
	Significant	½ PMF
	High	PMF
Large	Low	½ PMF-PMF
	Significant	PMF
	High	PMF

TABLE 2.5: Performance Standard Classification[26]

there are several possible alternatives to alleviate this situation. These include:

1. Revision of operating procedures, including reduction of reservoir utilisation.
2. Structural modification - outlet, scour, spillway, fuse-plug, dam.
3. Construction of an upstream dam to reduce the maximum possible flood (MPF).
4. Abandonment of the dam.

Revisions to the operating procedure may include one or more of the following:

1. Revision of schedules for opening spillway gates, or the removal of stop-logs, to reduce the maximum water level, or to give maximum spillway discharge.
2. Improvement of the inflow forecasting criteria, and the establishment of operating procedures to reduce reservoir storage during periods when a flood is likely to occur.
3. Improve facilities for standby power and communications and arrangements for competent staff to be in attendance during adverse and flood conditions.

Minor structural modifications may include one or more of the following:[14]

1. Restoration of the crest and therefore freeboard.

2. Extension of parapet walls when and where they are not continuous; especially near mitres since, generally, the compaction may not be so adequate, and erosion may occur.

3. Replacement or extension of all or parts of the spillway wall.

4. Improvement of the spillway stilling basin.

5. Modification to spillway gates.

6. Refilling joints in parapet walls.

Any remedial measures suggested should be studied with care since some recommendations may in fact promote local damage from various causes. As an example, increasing the height of a wave wall, or replacing the wall, can lead to local slipping on the crest due to extra superload. The wave wall can also divert run-off to the down-stream slope of the dam and to the mitres, instead of into the reservoir, causing local erosion damage. A largely vertical wave wall on the crest of an embankment can cause wind-driven water, i.e. spray, to be thrown up into the wind, onto the downstream slope of the embankment.

2.7 Safety Considerations

As stated earlier, a flood with a nominal return period of 1 in 10000 years may occur next year at any time and the year after, and then not for a further 30000 years. It is debatable whether it is possible to properly comprehend return periods of the order of tens of thousands of years. It may be argued that it takes a mathematician to fully appreciate what is meant by, defining statistically, a chance of 1 in 100000, and it may be difficult, if asked, to explain the

logic of statistical hydrology applied to spillway design in terms that can be understood by the layman.

Society is not entirely rational, and judgements are therefore not made on a totally rational basis. Griffiths and Berry[28] point out that whereas river flooding is viewed as a natural phenomenon, the failure of a man-made structure such as a dam is viewed in a very different light. It is not 'socially' or 'politically' acceptable for a dam to fail, and it seems that the public associate and tacitly expect differing standards of safety in differing aspects of life. It is accepted, for instance, that highways are not designed to eliminate loss of life, and a 1 in 100 chance of being killed on the road is judged tolerable by society. Bass[29] reports, from the Registrar General's returns, that the odds against suffering other forms of violent death are (male and female together):

Homicide	1500:1 against
Falls	110:1 against
Roads	85:1 against
Any violent death	27:1 against

It is also accepted that in the case of homicide investigations very large amounts of money will be used to carry out an exhaustive police investigation in attempts to bring to trial and subsequently to maintain one convicted criminal.

The 'fail-safe' concept is associated with nuclear power stations, nuclear weapons, and aircraft, and it may be argued that this terminology creates a false sense of security - nothing is 'fail-safe'.

It is evidently impossible to offer complete protection in an environment which knows no limits to combination of events.

The 'Maximum Credible Incident' (MCI) concept applied to nuclear installations is an attempt to quantify the worst possible combination of events. Any design involves some risk, and our task becomes one of selecting the criteria on which our judgement of acceptability is to be based.

The 'safe life' concept is applied to components in many structures, including aeroplanes, and it can be argued that this approach may be relevant to the safety of dams.

It is generally recognised that the cost of providing complete protection against loss of life is unacceptable, and in economic analyses it has become customary to attempt to value such loss in monetary terms.

It was Griffiths' and Berry's[28] opinion, as it is the Author's, that in the present legal climate and with public opinion regarding dam disasters as it currently is, where loss of life is foreseeable the dam and spillway together must be able to pass the estimated maximum flood without total failure. Griffiths and Berry also suggest that the use of cost benefit analysis is not appropriate to the determination of the optimum size of spillway since it is possible to obtain almost any result depending on the initial assumption. A 'least cost' method of analysis is therefore to be recommended.

2.8 Conclusion

The problem of flood forecasting is complex, and choice of 'design flood' a very controversial issue. This will continue to be so in the foreseeable future.

CHAPTER THREE

FAILURES AND ACCIDENTS

3.1 Introduction

In striving to optimise the use and control of water resources through the construction and operation of dams and associated structures, the potential menace of these works if improperly designed, constructed, operated and/or maintained, must be recognised. Dams represent a major potential hazard, as in few other types of engineering works are the consequences of total failure likely to prove so severe.

Dams will continue to fail, and while failures represent only a fraction of the total stock of dams throughout the world, the damage resulting from the failure of even a low dam can be disastrous and costly. An engineering failure of this kind is a serious matter, involving not only the loss of all or part of the structure and storage, but possibly property, and may cause injury and death, as examples have so aptly illustrated, with appalling loss of life and damage to property.

3.2 Failures

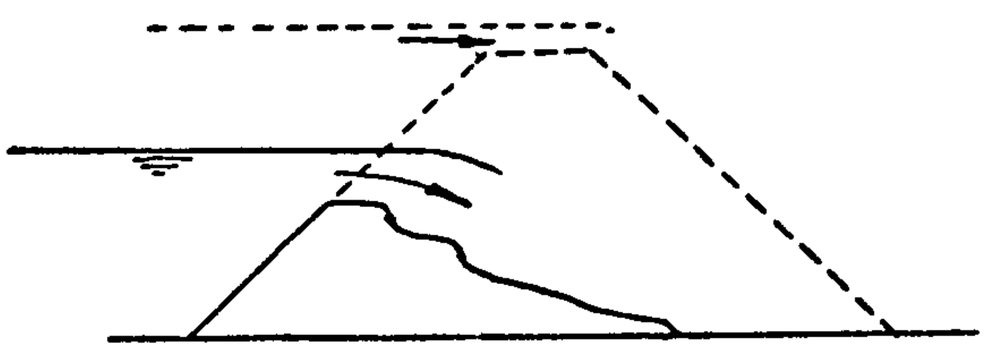
The failure of an engineering structure, unless attended by loss of life or property, does not usually attract the attention of the layman. When it involves loss of life as well as property it becomes

of great concern to the general public. Every dam, regardless of its size, represents to some degree a potential threat to those living downstream.

The building of dams is as old as the history of mankind. The demand for water for all purposes requires larger reservoirs and higher dams to be built in progressively less suitable locations. In addition, the engineer's endeavour to construct a safe structure will be in conflict with his efforts to exploit materials economically and to the best advantage.

A specific cause of failure is often difficult to isolate and evaluate, and the reason assigned must inevitably depend to some extent on the judgement of the engineer. Dams can fail in many ways. Essential mechanisms include overtopping due to inadequate spillway capacity, excessive settlement, embankment slips, seepage, piping and many others. Sketches illustrating some of the ways in which embankments can fail are shown in Figure 3.1.

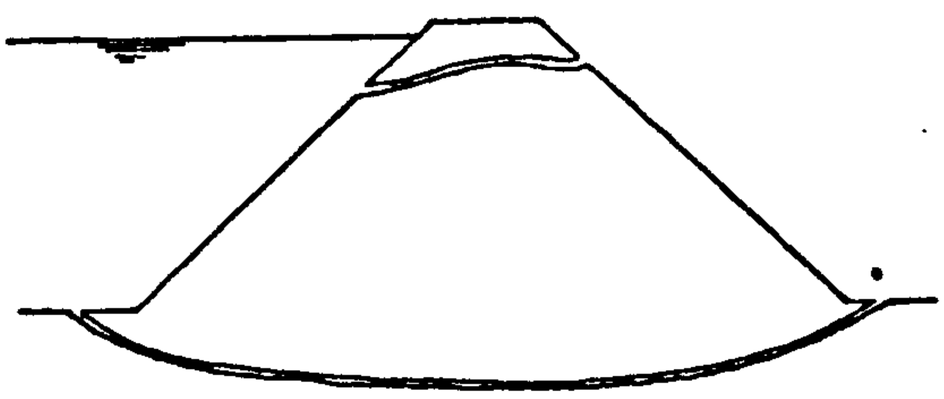
Overtopping can result from one of several defects in the structure and its design. Defects may include inadequate spillway capacity, lack of maintenance for control equipment installed on or in the dam, settlement caused by seepage or by insufficient compaction, extremes of rainfall, embankment slips, incorrect operation of the dam, and even vandalism, as shown in Figure 3.2. Within each of these subsidiary or contributory causes of overtopping there may be a multitide of other such factors which may initiate the failure process. Figure 3.3 illustrates some of the ways in which settlement may lead to overtopping and eventual failure.



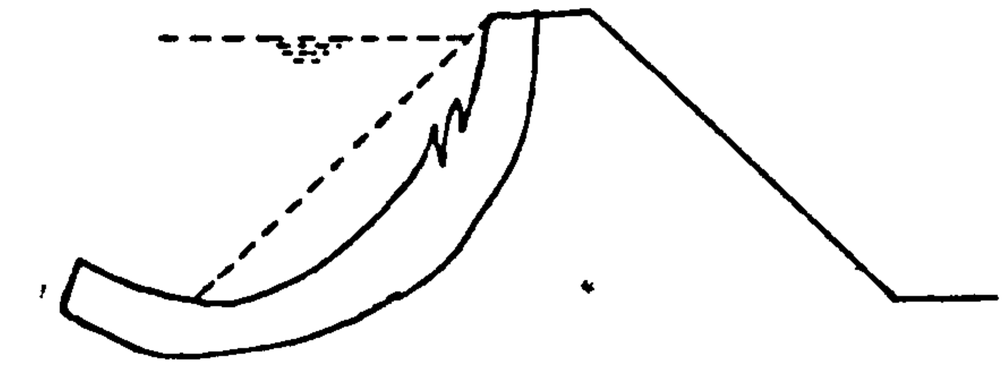
overtopping



foundation settlement



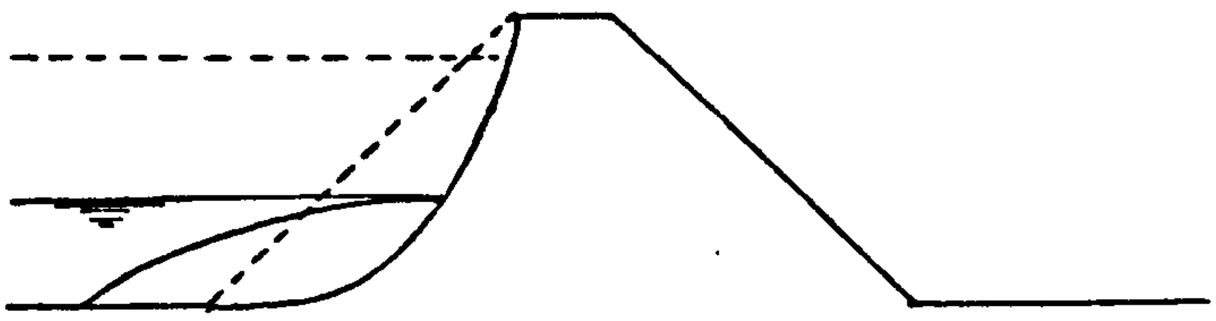
seepage/piping through dam/foundation



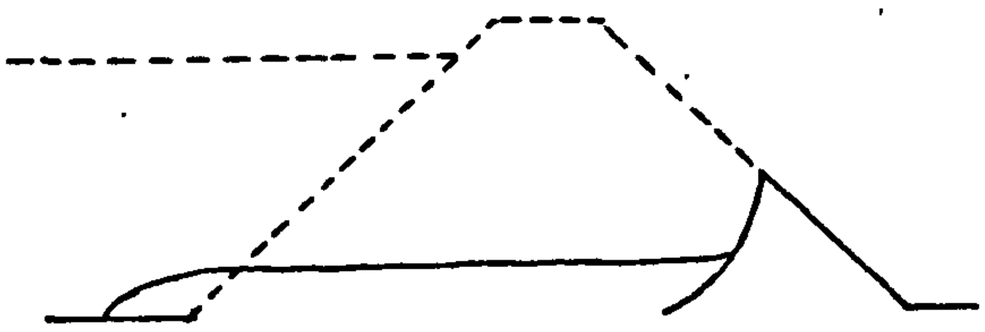
slide - weak foundation



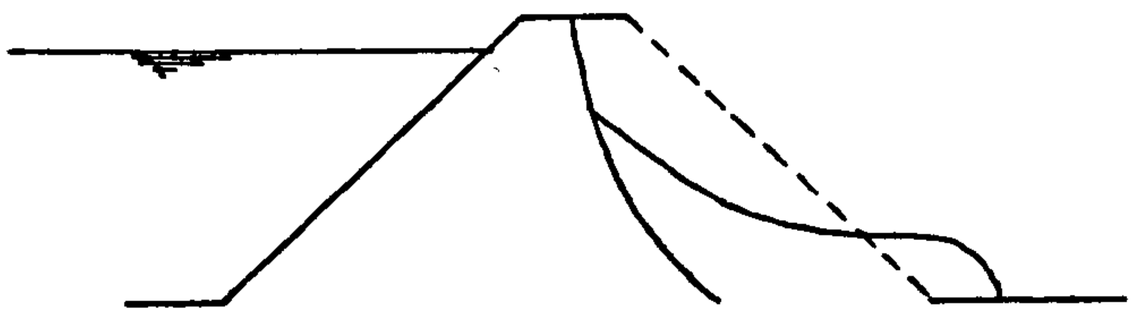
dessication and shrinkage



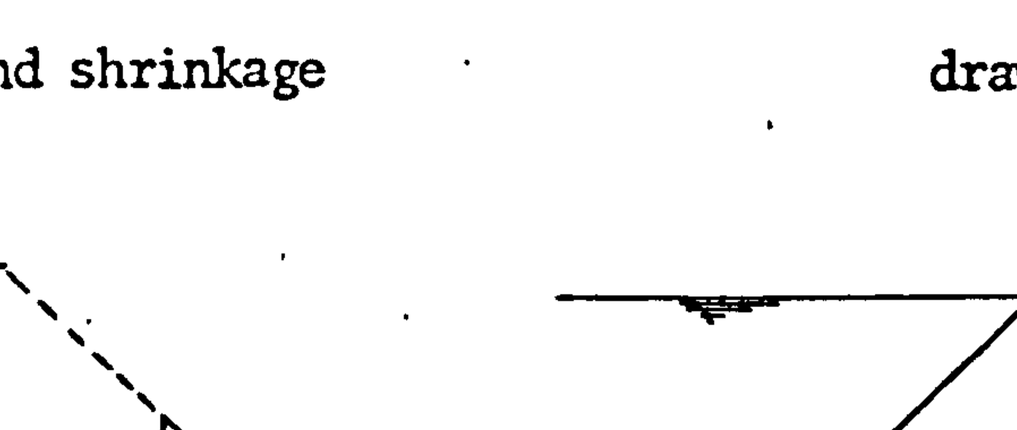
drawdown slide



flow slide in sand/silt



weak embankment - d/s slide



structural inadequacy - overturning/
movement downstream

FIGURE 3.1: Essential Mechanisms of Embankment Failure

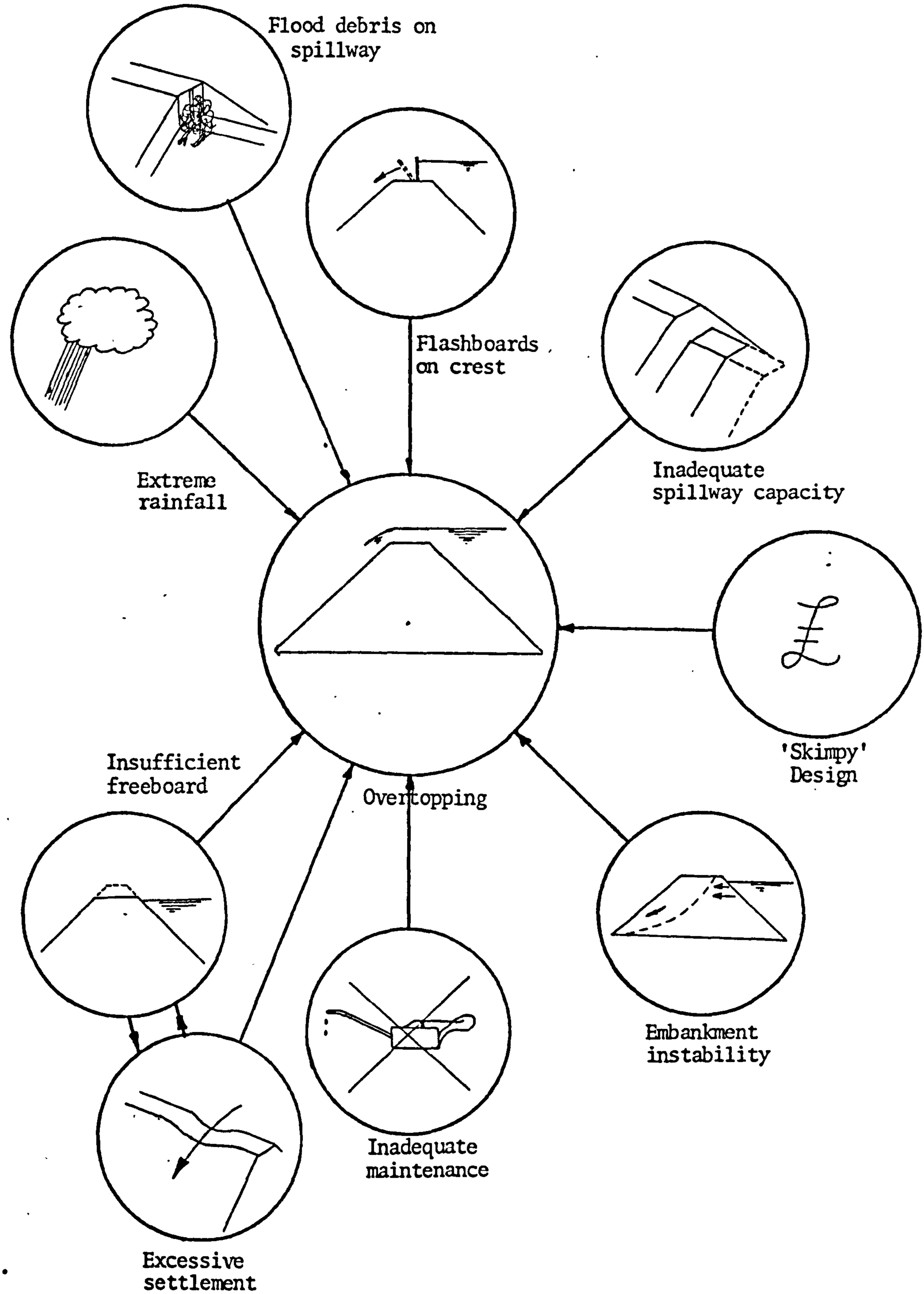
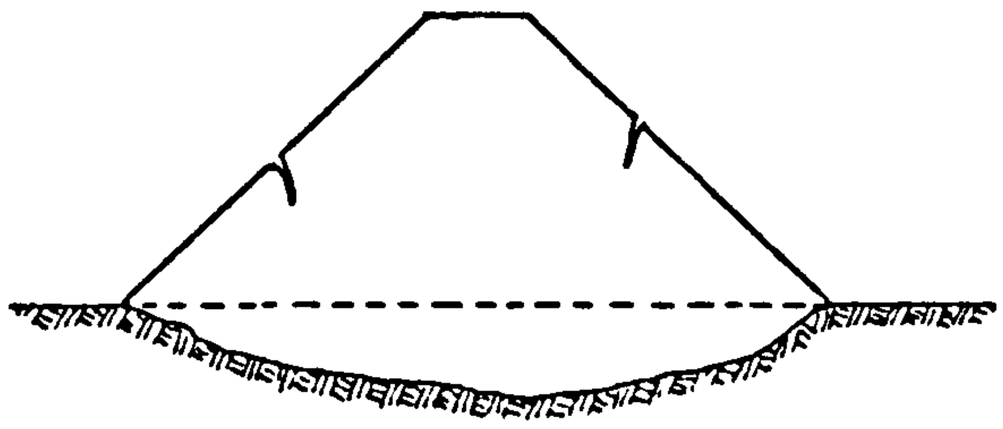
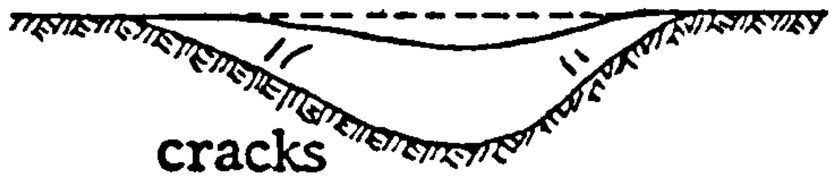


FIGURE 3.2: Factors which may contribute to Overtopping



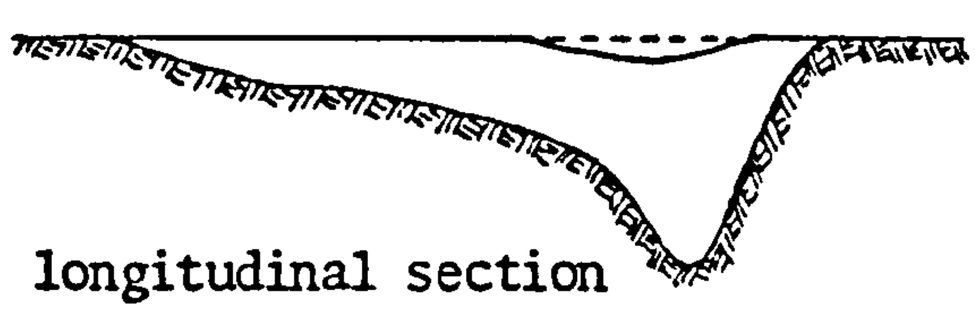
cross-section



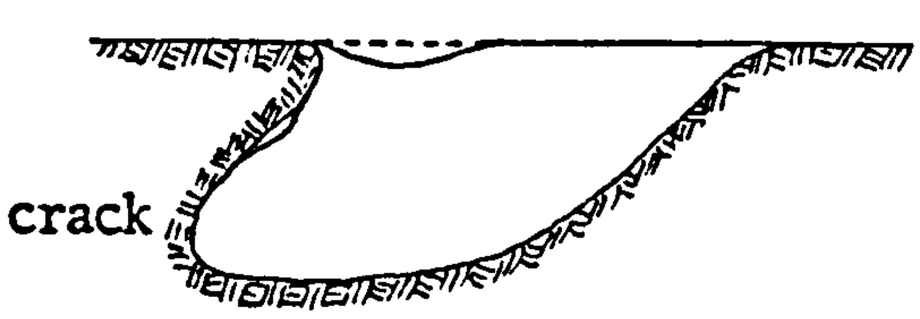
cracks

longitudinal section

settlement



longitudinal section



crack

longitudinal section

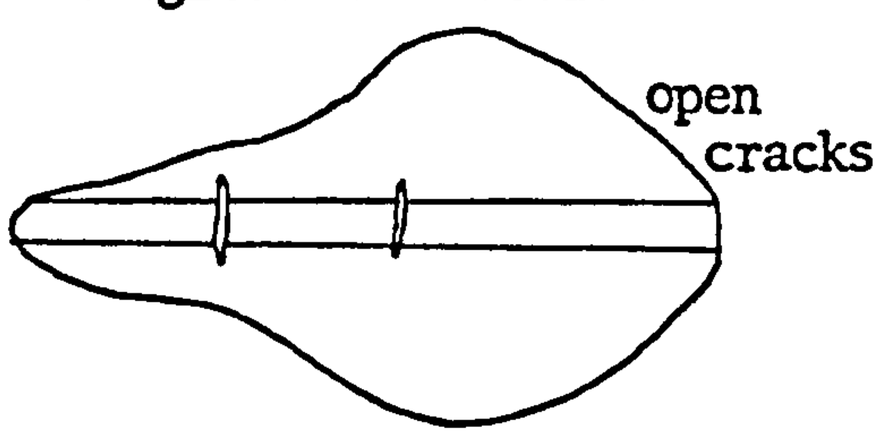


longitudinal section

crest settlement



longitudinal section



plan

FIGURE 3.3: Examples of Types of Settlement which may lead to Overtopping

Although it is difficult to specify a single reason for failure several analyses of types of failures have been carried out over the years and show significant trends. It can be seen from the data presented in Table 3.1 and Table 3.2 that failures involving overtopping and inadequate spillway capacities account for about 35 per cent of recorded failures and accidents. In 1961 the Spanish publication Revista de Obras Publicas[30] listed 1620 dams and noted 308 serious accidents between the years 1799 and 1944. Of the failures listed, 163 were earthen embankments, 14 were classed as dykes, 70 as concrete gravity, 9 as arch dams and 52 as other types. The causes of failure, as published, are shown in Table 3.1.[31] An earlier statistical review covering 128 earth dam failures resulted in the figures given in Table 3.2.[32] The dam failures concerned occurred between 1799 and 1922. Again it can be seen that approximately one third of all failures have been due to overtopping. No soil is totally resistant to erosion, and with an unprotected soil large quantities of material can be quickly eroded if appropriate conditions exist.

In relation to gross land area Britain may be described as a densely populated country in which there exists a large stock of elderly, mainly earthfill, dams. Many of these dams are built in potentially dangerous situations, in as much as they occupy sites in the Midlands and North West, above highly populated areas, often in series with other reservoirs, and also often in confined river valleys. These dams may be described generally as being of small or medium size and elderly, many being built at a time when technology of design was fairly rudimentary, if not actually non-existent. Data gathered by the Department of the Environment in preparation for the 1975 Act indicate a total of some 5000 dams in the UK, excluding Northern Ireland, of

Cause of Failure	% of Total
Foundation Problems	40
Inadequate Spillway	23
Poor Construction	12
Uneven Settlement	10
High Pore Pressure	5
Acts of War	3
Embankment Slips	2
Defective Materials	2
Incorrect Operation	2
Earthquakes	1

TABLE 3.1: Analysis of Failures[31]

Cause	Number	Percentage
Inadequate Spillway	38	30
Inadequate Spillway - Overtopped by Failure of Dam Above	6	4.5
Inadequate Cut-off Porous Foundation - Leakage/Erosion	17	13
Faulty Construction	9	7
Inadequate Cut-offs Round Conduits	15	12
Faulty Design - Slopes	5	4
Inadequate Stream Control During Construction	5	4
Improper Use of Clay	6	5
Improper Maintenance and Operation	2	1.5
Burrowing Rodents	2	1.5
Poor Materials	4	3
Unstable Foundation	1	1
Conduits not Properly Supported to Prevent Failure	2	1.5
Erosion from Backwash Below	3	2
Earthquakes	1	1
Miscellaneous & Undetermined	7	5
Failure of Bottom in Small Reservoirs	5	4
TOTAL	128	100

TABLE 3.2: Analysis of Failures[32]

which approximately 2000 qualify for inclusion within the ambit of the 1930 legislation.

Dams are subject to deterioration not solely from time and the effects of the elements but can also be rendered obsolete and possibly dangerous if the assumptions on which they have been designed are later proven unrealistic or inadequate.

3.3 Safety Regulations and Legislation

Dam safety laws and inspection programmes vary widely throughout the world.

In the United States of America legislation regulating licensing and inspection procedures have been the responsibility of each individual state. Great differences exist in the manner in which the states carry out their responsibilities to the safety of dams built within their jurisdictions.

The United States Public Law 92-367, 1972 authorised the Secretary of the Army through the Chief of Engineers to undertake a national programme of inspection of dams, to compile an inventory of dams in the United States, and to prepare for Congress recommendations for implementing a comprehensive national programme for dam safety, including assignments of responsibilities. This Law was enacted following the near failure of the Lower Van Norman Dam during February 1971, California earthquake, the Buffalo Creek failure in February 1972, and the Rapid City dam failure of June 1972. Public concern over these incidents was such that Congress was moved to react quickly through the enactment of federal legislation. The Public Law, applicable

to dams greater than 7.6 metres (25 feet) provided it impounds more than 18.5 t.c.m. (15 acre-ft), or to a dam greater than 1.8 metres (6 feet) if it impounds more than 61.7 t.c.m. (50 acre-ft), indicated the existence of approximately 49000 dams. The responses of the states and territories indicated that 11 have no laws regarding any aspect of dam supervision.[33] The Chief of Engineers recommended that a comprehensive National Dam Safety Programme should be implemented, followed by an inspection of those dams with a high or significant hazard potential, and that the National Dam Inventory should be maintained.

In France legislation also applies to all dams greater than 20 metres (66 feet) in height, while in Japan a dam is subject to legislation if any of the following criteria are satisfied:

(a) the dam is greater than 15 metres (50 feet) in height;

(b) the dam has spill-gates and has a total length of reservoir, measured along the river bed including tributaries, of more than 9.6 kilometres (6 miles);

(c) a cascade system.

In Great Britain the Reservoirs (Safety Provisions) Act 1930[11] was one of the earliest attempts to regularise dam safety procedures. The 1930 Act was applicable to reservoirs storing more than 22700 cubic metres (18.3 acre-ft) of water. One of the new provisions of the Reservoirs Act 1975[13] is the rounding up of the qualifying storage capacity to 25000 cubic metres (20.2 acre-ft).

The American legislation is seen to exclude any dam smaller than 6 feet (1.9 m) in height regardless of its storage capacity, and any

dam with a capacity of less than 15 acre-ft (61700 m^3) regardless of height. British legislation will exlude any dam with a capacity of less than 25000 m^3 and will apply only to reservoirs retaining water and therefore exclude industrial lagoons, some of which are subject to the Mines and Quarries Act (1969) and canal embankments.

3.4 A Study of Failures

In engineering much is learned from failure - relatively little from success. The knowledge of problems encountered in construction and operation of dams enables safer structures to be built and more effective operating procedures to be devised. Disasters have been demanding teachers, but properly structured studies of the mechanisms by which dams fail can offer an instructive approach to design problems, and it could be said that the more disastrous a failure the more valuable are its lessons.

The safety of dams, while of concern to their designers and constructors, is important to those whose lives and property may be endangered. Inherent in the development of any design criteria, procedures or legal requirements calculated to ensure that a dam is safe against any reasonable contingency, there must be thorough understanding of past failures.

With this in mind, an understanding of past failures due to overtopping was though to be instructive, and so an extensive survey of literature was instigated. Local authorities, water authorities, libraries, technical institutions, panel engineers and individuals with an interest in dam engineering were contacted and the resulting data stored. For statistical analysis and reference, however, a computer file listing consisting of a line entry for each dam was written and the

most pertinent data stored in this way. Data could be added or changed at will by use of an editing programme. A copy of the resulting 'print-out' is presented in Appendix A, and a 'sub-file' containing only British dams is given as Appendix B. The file is entitled "Known Instances of Dams which have been Overtopped" since the Author has included incidents which did not involve the loss of the embankment or total loss of storage but which damaged the embankment to the extent that remedial measures had to be instigated.

The International Committee on Large Dams published a text on incidents and failures at a number of dams.[34] Incidents and failures were put into a category selected from one of the following:

1. Complete failure - leading to abandonment. (F1)

2. Severe failure at time, but repairs enabled the dam to be put back into operation. (F2)

3. Accident, prevented from becoming a failure by remedial works or operations - such as drawing down level. (A1)

4. Accident during initial filling, immediate measures such as drawing down before putting into operation. (A2)

5. Accident during construction or before impoundment. (A3)

It is thus apparent that very differing emphasis may be put on the word 'failure'.

The listing in Appendix A involves some structures which may be better described as levees, canal banks, weirs, natural dams, debris barriers and ice-bar barriers, and it may be questioned as being of any value to the design and construction of man-made barriers across

natural water courses. These structures, as man-made barriers, do impound water and can be a potential hazard.

Accidents to, and failures of, earth or rockfill diversion cofferdams, where a degree of risk is acceptable for economic reasons, are common. As a result of the incomplete state of the dam some damage may be done if the upstream cofferdam is overtopped, but this damage is usually repairable and the dam is serviceable after completion.

Several ancient dams, about which very little is known, are also included, but they are of interest as they show the progress in dam construction through the ages, and from some of the descriptions it is apparent that many of these early engineers did realise the devastation that could result from the failure of a dam due to overtopping.

Unfortunately, it must be recognised that these notes may include reporting errors. Especially in the case of early incidents, various references to the same failure contain conflicting data regarding names, dates, dimensions. An attempt to cross-reference dams with alternative names has been made. It may appear from the listing that the United States is the major offender, but this is believed to be due to more complete reporting in reports and periodicals.

In addition there is a natural tendency of reporting, first hand, accidents having occurred recently and therefore in vivid recollection, while accidents having occurred many years ago have more or less been forgotten. Failures can also be expected to be better reported than accidents. Accidents in recent years are more completely reported than those occurring a long time ago. An additional difficulty in a study of this type is that incidents/failures occurring to small dams, unless

causing death or injury, are rarely published, and these incidents/ failures are therefore more difficult and time-consuming to research.

Inspection of the list and the completed files on each incident reveals several lessons to be learnt:

3.4.1 Small Dams

There is a tendency to a degree of complacency about the very small dams, to which no legislation is applied. Small dams can cause as much damage as a large dam in certain circumstances. Some authorities have realised that a small dam is an important structure and that, just as in the case of a 'large' dam, the damage caused by the failure of a small dam is a function of its location, both in distance from and height above a downstream community, the dam height, the reservoir volume, the amount of development downstream, whether the dam is a part of a series of dams, the shape of the valley downstream of the dam, and the speed at which the failure, or partial failure, occurs.[35] In addition, many of the old dams, especially those in this country and the United States, were planned and initially built in remote areas, often where only limited hydrological data was available. Now, due to population growth and urbanisation, the areas below many dams are becoming more densely populated. Development may also have affected the run-off characteristics of the catchment. Demand for water has also resulted in reservoirs being grouped in series in confined river valleys.

It has been argued that large prominent dams are safer than small obscure dams, since they are more likely to be properly designed and maintained. The fact that a dam is 'large' results in people being

aware of the potential danger of failure, whereas it is often forgotten that the failure of a small dam at the head of a catchment can trigger a cascade effect. As an example, the failure due to overtopping of Canyon Lake Dam, America in 1972 caused the loss of over 330 lives, and damage which at the time was estimated at sixty million dollars.[36,37,38] A study of Appendix A reveals that dams smaller than Canyon Lake, a 'small' dam by ICOLD standards, have caused even greater damage in terms of human life, again illustrating that the height of the dam or reservoir volume are not the only factors to consider from the point of view of risk. Several examples are shown in Table 3.3.

The minimum dimensions etc. adopted by the International Commission on Large Dams before a dam is listed in the World Register of Dams[6] seems rather restrictive. They specify a dam with a height of not less than 15 metres from the foundation to crest, or small dams, if they have a crest length of not less than 500 metres, if the capacity of the reservoir is not less than one million cubic metres, if the dam has difficult foundation problems, the dam is of unusual design, or if the maximum flood discharge is not less than 2000 cubic metres per second. Dams of less than ten metres in height are excluded from the Register. Of the list of 567 failures/incidents only 333 (58% of the total) have occurred to dams conforming with the ICOLD requirements for registration. Many authorities apply less stringent conditions to ascertain whether or not a reservoir should be subject to safety legislation.

The Author considers that even small dams must be officially controlled by safety legislation. Babbi-Yar dam, in the Ukraine[39,40], failed due to overtopping in 1961, killing 145 and causing damage estimated at the time at three and a half million dollars. The dam was

Country	Name	Height (m)	Year of Incident	Loss of Life
Bolivia	Potosi	8	1626	4000
Great Britain	Whinhill	13	1835	40
Great Britain	Cymcarn	12	1875	12
Canada	Alton	2	1889	2
America	Staunton	5	1896	5
America	Oakford Park	6	1903	12
Great Britain	Skelmorlie	5	1925	5
America	Canyon Lake	6	1972	330

TABLE 3.3: Examples of Loss of Life caused by the Failure of 'Small' Dams

only 10 metres high. An ornamental goldfish pond at Henbury Hall caused extensive damage when destroyed by a flood in 1872.[41] It gave rise to the lawsuit Nichols v. Marsland[42] in 1876, and it still serves as a precedent in British jurisprudence.

In addition, there are many dams in this country whose capacity is marginally below that which is required to be subject to the Reservoirs (Safety Provisions) Act 1930. These reservoirs may temporarily store a volume of water in excess of that required for inclusion within the Act due to small and inadequate outlets, and this, albeit a temporary storage, must represent a weakness within the legislation.

The graph in Figure 3.4 represents the number of recorded incidents/failures due to overtopping plotted against the height of the dam.

Any statistical analysis of this type is not entirely valid, since it assumes that each country listed reports failures to the same degree, that such reports are readily accessible, that the same environment exists, that the same materials are used, and the same construction methods and safety standards are implemented. Such an analysis can be used for comparison and to indicate possible trends.

It can be seen from Figure 3.4 that the most common failures appear to be to those dams with a height of the order of 10 metres - a relatively 'small' dam. However, as there are a greater number of small dams than large dams, and it is not known how many incidents/accidents have not been reported, this statement must be interpreted with caution.

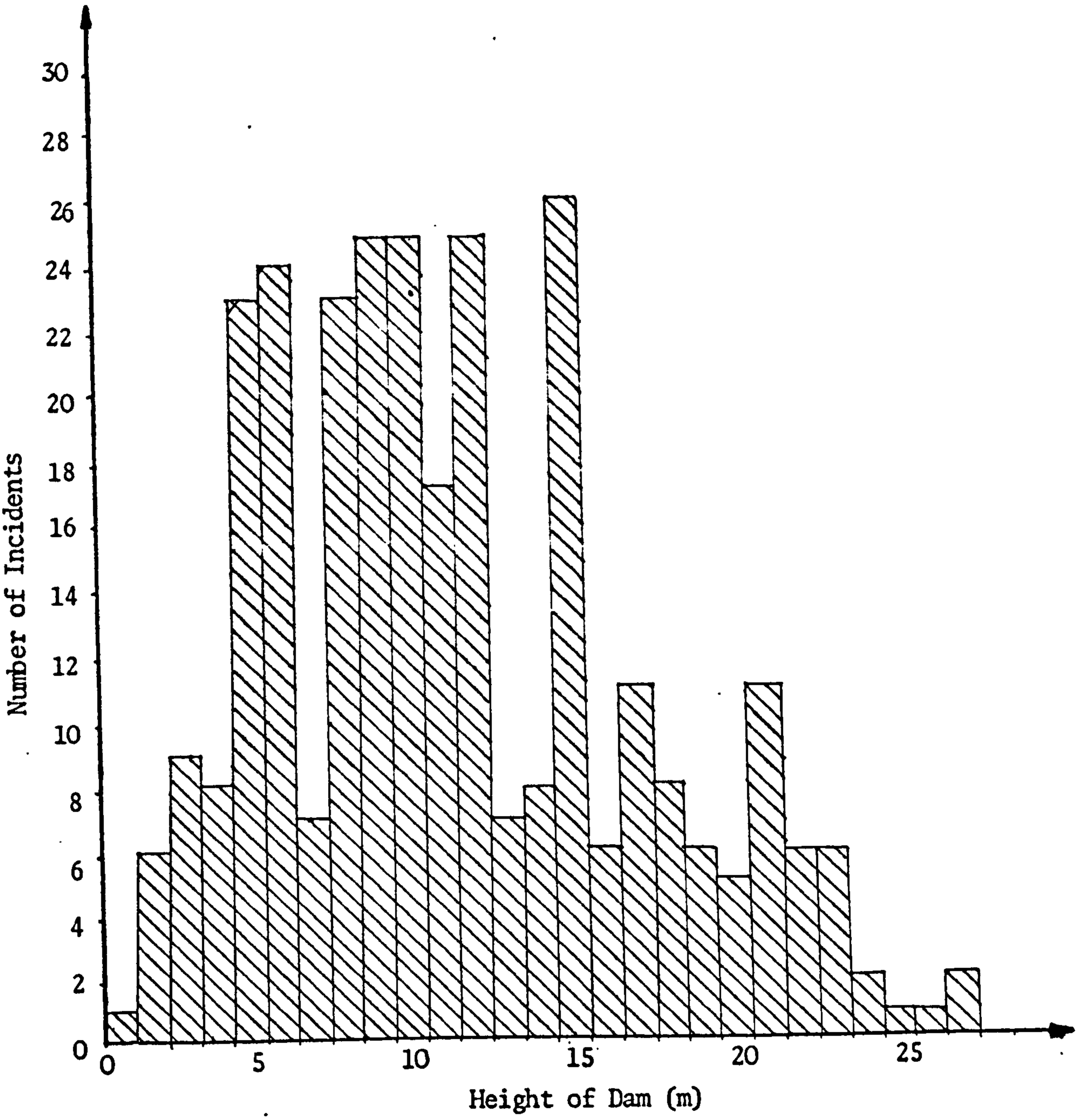


FIGURE 3.4: Number of Recorded Incidents v. Height of Dam (World dams)

The high incident/failure rate for small dams and not only due to overtopping may be due to several factors which are listed below:

1. The increasing demand for water and lagoons for industrial processes has led to small dams being designed and constructed by persons who may have no previous experience with design and construction and maintenance of dams. The increasing use of waste and collecting ponds and demand for water will obviously lead to an increase in the number of small dams.

2. Owners and maintainers of small dams may be lulled into a false sense of security by virtue of the very size of their structure.

3. The practice of relating fees to construction costs is not valid, since a good design for a small dam is likely to cost as much as that for a large dam.

4. Those responsible for design, construction and maintenance may only be concerned with the use to which the reservoir is to be put to the possible detriment of the structure in design, construction, supervision and later maintenance.

3.4.2 Engineering Supervision during Construction

Dams appear to have been one of the few civil engineering structures which can cause devastating loss of life and which have on occasion been designed and constructed without any engineering supervision, and indeed it has not always been considered necessary to employ a qualified engineer with some knowledge of the requirements for dam construction. This was the case at Holmfirth (failed 1850)[43], Greenwood (1936)[44], Mill River (1874)[39,45,46] and Kilantringan Loch

(1843)[47,48], the latter failing in November 1843, four months after construction was completed. No engineer was employed during the construction, and the embankment was filled as soon as it was completed. The fill had not been compacted during construction with the result that on impounding the embankment settled and was overtopped.

A similar experience occurred with the 100 year old American Greenwood dam[44] and with the Mill River dam[39,45,46] which failed in May 1874 with the loss of 143 lives. No engineering supervision was employed during the design and construction phases. Lower Otay dam, which failed in January 1916, nineteen years after its construction, is said to have been 'recklessly conceived and crudely construction'.[49,50] The fact that a dam does stand, however small the factor of safety, is often sufficient proof of its safety as far as the general public is concerned.

Engineering supervision has frequently not been employed because the cost of the works exceeded the funds available for the project, as in the case of the Holmfirth (Bilberry) failure of 1852.[43] The letter inviting George Towlerton Leather to be engineer for the scheme stated: 'You will only be required to come over now and then, as circumstances may require', because they would employ a surveyor 'to attend the works'. The first tender was accepted in 1838 and later transferred to another contractor in 1843. Leather appears only to have visited the site twice, and on the second occasion, after finding that his instructions from his first visit had not been carried out due to lack of funds, he refused, rightly or wrongly, to give further assistance. Construction was continued and, in due course, total failure occurred some eight years later.

3.4.3 Spillways/River Diversion during Construction

The construction of a dam presents the designer with a problem concerning river diversion and provision of flood by-pass facilities. A compromise must be made between size, and hence cost, of these often temporary works, and the possibility of the arrival of a flood exceeding the capacity of the by-pass measures within the construction period. It was possible that severe economic pressure could be put on the designer to keep these costs to a minimum; however, the damage caused by a flood and the delay incurred must be considered when deciding the capacity of by-pass measures.

Instances of damage due to overtopping during construction, due to floods, include: Akosombo (1963)[58], Alibey (1974)[52], Big Santa Anita (1926)[34,53], Bartlett (N.A.)[54], Buffalo Creek (1970)[34,55], Cheeseman (1900)[56], Credit River (1910)[39,57], Croton (1841)[58], Folsom (1953)[34], Embsay Moor (1908)[59], Hell Hole (1964)[60] dams and many others.

3.4.4 Spillways

The need for proper spillways was demonstrated by the early failures of quite ambitious schemes engineered by the early civil engineers. One of the first failures involving, as already mentioned, one of the first dams built was at Sadd-el-Kafara.[3] The failure of this dam seemed to deter the Egyptians from building dams, at least of masonry, for several thousands of years. The same lesson was learnt at Adhiem[61] in Iraq some time between 605 B.C. and 562 B.C., and one might think that man had realised the consequences of omitting spillway facilities, but failures due to hopelessly inadequate spillways have occurred more recently. A dam at Schenectady[62] built in 1853, which

failed in 1916, had as its only outlet a wooden waste gate on a pipeline which was closed when the flood reached the dam. Even more surprisingly, a 110 year old dam in Mohegan Park at Norwich, Connecticut, USA failed in March 1963[63] by overtopping, releasing 200000 cubic metres and causing the loss of 12 lives. No spillway was provided, the only outlet works being a small sluice through the base of the dam built to feed a mill race. This situation was allowed to exist for 110 years until failure in 1963.

Meadows Dam, Utah[64] was being constructed in 1937 without the provision of a spillway. Only protests by the local populace, indicating that at least someone was aware of the possible consequences of the omission, led to incorporation of a spillway.

A possible reason for some dams not being provided with a spillway was that the land downstream was not developed, and a simple comparison between construction costs and damage caused would show costs of the latter to be smaller, given no loss of life.

Several dams are known to exist whose spillway capacities have been drastically reduced by construction of roads across the crests, and several conventional spillways have been filled in to carry road decks and replaced by small culverts.

3.4.5 Interruption of Work during Construction

If work is interrupted during construction for any significant length of time, whether it be due to lack of funds or any other reason, the means of flood discharge should be assessed in terms of the calculated risk and the losses likely to be incurred if a flood occurs.

Extensive damage and the loss of 160 lives was the result when the Sempor Dam, Indonesia[65] was overtopped by a flood in December 1967. Construction was started on the rockfill dam in 1963, but due to lack of funds work ceased in 1965. Monsoon rains breached the unfinished rockfill dam two years later, in 1967.

3.4.6 Access

Dams, and their control structures, must be provided with good means of access so that if a potentially hazardous situation does develop the necessary personnel and equipment can be rapidly moved on to site.

Hatchtown Dam, Utah was described as being "in a most inaccessible location"[66], although even here when a rising stream coupled with spring floods, high winds and a spillway of an inadequate capacity caused overtopping, strenuous attempts were made to save the dam by opening culvert gates to their maximum height and by temporarily raising the crest.

Adequate access must be provided to control equipment. When overtopping occurred at Middlefield Dam, it was impossible to gain access to gates and valves.[45,67]. The approach bridge to the outlet tower had not been completed on the Panshet Dam[68] when the monsoon arrived in 1961. There were no means for resetting the outlet gates during the flood, resulting in vibration, excessive settlement and eventual overtopping and failure.

If access is good, then it is very likely that the dam is more likely to receive regular and adequate maintenance.

3.4.7 Mine Tailings/Chemical and Industrial Waste Lagoons

Improvement of mining techniques and the increased demand for minerals and fuels has led to an increase in the number and size of embankments storing industrial and mining waste slurries. Penman[69] quoted the production of mine tailings in the United States alone in 1974 as being over 2×10^9 t, and that this was expected to reach 3×10^9 t by 1980.

Mine tailings and industrial waste lagoons often present a very real potential hazard to the surrounding area due to the fact that material is not properly compacted, or placed as a hydraulic fill, and also the design may be crude in the extreme. Due to the requirements of operation, these embankments often impound water as well as waste products and must have adjustable spillway facilities to cater for the rising mud levels. This latter fact alone, as Penman points out, resulted in an accident in 1974 in Brittany with a small 'experimental' dam retaining kaolin.

In the past, it was common practice not to provide any type of spillway, but to rely on seepage through the material as part of the filtration process to provide flood relief. However, there is a tendency with time for the embankment to become progressively more impermeable. It is now common practice to use pipe spillways, usually installed to discharge directly onto the downstream face with the obvious associated problem of erosion. These spillways, frequently of inadequate capacity, are provided on an embankment which may have inadequate freeboard, and they have the additional disadvantage of being easily blocked.

An alternative is an 'open-cut' spillway which results in rapid erosion of the fill.

An embankment in Texas, retaining gypsum waste, failed due to liquefaction in 1966, but perhaps more serious was the failure of a mine tailings dam in West Virginia in February 1972.[70] Waste material had been deposited in a steep, narrow, natural valley. This created a natural depository for wash water as well as impounding the run-off from the surrounding area. It created a reservoir of depth 15 metres and a surface area of approximately 7 ha. Heavy rainfall filled the reservoir to within 0.3 metres of the crest when it was decided to cut an emergency spillway. The emergency spillway rapidly eroded to a breach through the embankment releasing 682 t.c.m. of waste and water into the narrow valley below, where the high velocity of flow was maintained and the resulting flood caused the loss of 166 lives.

Waste lagoons create an additional problem since failure of such a structure will release water which is usually of a highly acidic nature, and thus cause a long-standing problem of water quality in the surrounding area.

In the UK, if a lagoon is not adjacent to a mine or quarry, then the Mines and Quarries Act does not apply, with the result that many of these structures are not subject to any legislation.

It is the opinion of the Author that, although costly, these structures should be subject to the most stringent legislation and that more thought should be given to design and construction and to the provision of adequate spillway capacity.

3.4.8 Awareness of Danger

A remarkable lack of awareness of the possible dangers involved when water or waste liquids are impounded was exhibited at Gros Ventre Dam, USA in 1925 when a natural barrier was created when 40 million cubic metres of material slid from the hillside and dammed the river.[71] A reservoir with a capacity of 80 million cubic metres was created and seepage through the dam corresponded to the normal flow of the river. Engineers and local inhabitants believed that since most of the material in the slide would have been suitable for embankment construction the barrier was stable and that rapids would form down to the old streambed. However, in May 1927 heavy rains caused the barrier to be overtopped, with the result that the 56 metre high dam was breached and seven inhabitants of the town of Kelly, 5.6 kilometres downstream, killed.

At Mission Creek Dam (Horton)[72], engineers were advised of the settlement of a section of the crest, but no action was taken. In June 1925 heavy rainfall caused overtopping at the low points on the crest, and volunteers narrowly averted a catastrophe by filling the rapidly forming breaches with trees and timbers weighted with sandbags.

A similar story can be told about the Pinkston Dam in Missouri.[73] Pinkston Dam first came to the attention of the authorities in June 1962 when landslides on the sides of the dam and two active areas of water loss were noted. It was also calculated that the dam could not withstand a "50-year" flood. The dam was visited once a year and progressive deterioration of the earthen material in the base of the dam was noticed, as a result of continuous leakage. The leakage was so excessive that the dam would not fill. The dam failed in May 1968, and even though the deficiencies in the construction and maintenance of Pinkston Dam were

recognised as early as 1962 no action was taken to correct the faults.

The cascade failure of the Akerman Farm reservoirs[1] were caused because the spillways of both dams were small ditches cut around the ends of the embankments with a capacity only slightly larger than that of the dry weather streambed flow.

The spillway for Balsam Dam[74], New Hampshire was an 8 feet (2.4 metre) by 6 feet (1.8 metre) box culvert discharging onto the downstream slope. When the spillway discharged all support to the box was removed and the spillway dropped and broke up under its own weight.

Elk City Dam, Oklahoma[75] is said to have had its spillway capacity reduced by the city engineers in order to increase the storage capacity. Heavy rainfall in May 1936 caused the dam to be overtopped by 0.45 metres over the entire crest length of 615 metres removing a 50 metre section of the embankment.

The failure of Pittston Dam, Pennsylvania[76] in 1901 was the result of overpumping into a reservoir in which no outlet was provided.

Kelley Barnes Dam[77] collapsed in November 1977 killing 39 people. This tragedy occurred even though the dam had shown signs of instability for at least three years.

3.4.9 Sabotage and Vandalism

Instances have been reported where the failure of a dam has been thought to have been caused by an act of sabotage. This was thought to have been the case at Camden Dam, New Jersey[78]. This earth embankment was filled by pumping from the Delaware River. Failure

occurred in October 1902 due to overtopping caused by excessive pumping at night, and it is thought that either the valve had been left open mischievously or the valve may have been left open deliberately.

The Chaba Dam in Syria[39] was deliberately breached in 1947 by the local populace who disagreed with the scheme.

Hyde[79] reported a case of vandalism/sabotage at Coombs Dam, owned by the British Waterways Board, where the spindles were sawn off the valves so that the reservoir level could not be lowered.

3.4.10 Enlargement/Embankment Raising

Suitable sites for reservoirs are becoming increasingly difficult to find. As a result, many reservoirs are being enlarged by raising the embankment. When an embankment is raised, it must be ensured that the loading imposed by an addition to the structure and by the additional water loading can be withstood by the original structure and its foundation.

This was obviously not the case with Anderson Dam, a masonry dam which failed in December 1912.[49,80] A 6 metre high masonry addition was made to the dam and secured by anchor bolting to the original structure. The dam was subsequently overtopped by 2 metres of water, with the result that the upper section of the dam, the new section, overturned.

3.4.11 Control Structures

Flashboards

The use of flashboards has had a mixed history. Flashboard supports can catch cebris, which is always associated with any high run-off

during a storm, reducing spillway capacity and eventually blocking the spillway.

This was the case with Dells No. 4 Dam, Wisconsin.[81] Flashboards were installed to a height of 1.5 metres on the 137 metre long spillway. During a flood in October 1911, the flashboard supports caught debris brought down by the flood. This obstructed the free discharge of water over the spillway and caused the bank to be overtopped. Similar failures occurred at Deanhead (1841)[82], Harrington (1964)[82], Pagara (1923).[34] Canyon Lake, America, with a capacity of only 9092 cubic metres, failed during a flood in June 1972 with the loss of 330 lives. The dam withstood the first wave of the flood, but debris blocked the spillway and the arrival of the second wave of the flood caused overtopping.[84]

The spillway of Hatfield Dam, Wisconsin was also blocked by debris caught by the flashboard supports, and such was the pressure on the piers of an access bridge over the spillway that the bridge was destroyed, which prevented access to the valves in the dam. The dam ultimately failed due to the release of water from the breached Dells Dam situated upstream.[46]

3.4.12 Roller Gates

At Panchet Dam on the River Ambi, India temporary chains were put on the fixed-axle roller gates for the construction period due to the late delivery of the gates. In 1961 a flood hit the dam which was passed through the spillway gates where, due to the temporary chains, vibrations were set up which caused settlement of the crest, and this, coupled with the incomplete and vulnerable state of the works,

permitted overtopping to occur.[85,86] Panchet Dam subsequently failed, the resulting flood washing out the Khadakwasla dam situated downstream.

3.4.13 Power Supplies

A flood occurred in August 1958 at the Kaddam Dam in Andra Predesh, India. The dam had just been completed, and although there was a power supply on site, the electricity driven equipment for the spillway gates could not be used and the gates had to be raised by hand. This proved to be too slow, and the 41 metre high dam was overtopped by half a metre of water and breached.[17]

This emphasises that an adequate power supply and emergency supply must be provided for all power driven equipment and that it must be kept in a working state of repair in order that when and if required it can be put into immediate use.

Any type of gate should be opened periodically in an attempt to ensure that the gate is always in working order.

3.4.14 Icing of Gates

Measures must be taken to prevent ice forming on gates to the extent that the gates cannot be opened. This may be achieved by the provision of heating elements or 'bubbling tubes', the latter circulating the water near the gate to prevent ice formation.

The gates on the incomplete and abandoned Bulley Creek Dam, Oregon became blocked by ice, causing the water level to rise and failure to occur due to overtopping. It is interesting to note that this dam, which was built in 1916 and condemned in 1916, remained standing until its failure in 1925, and although built to store 2800

cubic metres above the outlet gate[87], it had 1680 cubic metres remaining in the bottom of the reservoir as dead water which contributed to the flood and caused a flood wave 14 metres high.

Generally it may be argued that from the point of view of safety, control by gates is very satisfactory since they permit rapid release of water from a reservoir after a flood in preparation for a possible second flood. Gates also permit manipulation of discharges to suit the downstream conditions. However, dams which have no gates are not subject to human or mechanical failure, and have the advantage of automatic operation. Siphon spillways have become popular since they have no moving parts, are designed to avoid cavitation and vibration and require very little maintenance, but unfortunately they are expensive, lack the 'fine' control of gates, have limited capacity, are easily blocked and usually require model tests.

The use of gates assumes that the gates will function when required, that there is a flood forecasting system giving data necessary for gate operation, that the gate operators are present, and that they understand the significance of this data and know how to act on the received information. Due to infrequent operation of gates, complacency often exists, and it is often difficult to obtain competent, reliable staff.

Gateless dams and the associated automatic operation are not subject to human error and discharge at the beginning of inflow into the reservoir (when the reservoir is full) and are free-flowing to a predetermined maximum. However, gateless operation usually necessitates a larger dam.

Small catchments which are often subject to flash floods and which may have adequate flood forecasting systems present difficulties in as much as the manipulation of the gates becomes very complex. In the case of a series of dams with a small catchment, the only method of determining the timing of the intricate manipulation of the gates is by using data available after the flood has passed.

3.4.15 Maintenance

Regular and thorough maintenance must be provided at all times. At times when this necessary maintenance has not been provided failures and accidents have resulted.

Valves and Gates

A broken valve on the outlet conduit of Lake Lidderdale Dam, Colorado resulted in the embankment being overtopped since the discharge capacity of the 12 metre long spillway could not be supplemented by the discharge of the outlet.[62] Although the capacity of the outlet works should never be relied on for flood discharge, in some cases use of the outlet conduit may avert failure.

The failure of the Rot-in-Baden Dam, Wurttemberg, Germany in 1969 was partly the result of a lack of maintenance.[17] On application of force to the valve on the outlet conduit the valve jammed and the spindle broke. As a result the water level rose, the embankment was overtopped, and 500 t.c.m. of water was released.

The 24 inch (0.6 metre) scour valve at Buckieburn Dam, Stirlingshire was found to be seized when an attempt was made to open it after a slide in the downstream face of the dam had occurred. It is thought that the valve had seized as a result of being kept partially

open for many years to provide compensation water, and although in this instance overtopping was not a danger, it is an example of the consequences of lack of maintenance.[88]

The capacity to rapidly lower the level or to empty a reservoir during a crisis can be extremely important. In some cases distress in a dam can be alleviated by reducing the elevation of the reservoir surface by perhaps as little as a metre. On such occasions it is clear that a properly functioning and adequate outlet facility is essential.

A case where there was an emergency which required immediate drawdown of the reservoir was that of the Lluest Wen Dam near Maerdy, Glamorganshire.[89,90] This is a 20 metre high earth embankment dam with a puddled clay core, built just before 1900, impounding 1100 t.c.m. gathered from a catchment of 6 square kilometres and with a mean annual rainfall of 2.28 metres. Outlet facilities consisted of a 15 inch (0.4 metre) pipe which could discharge about 13 t.c.m. a day. During the emergency 75 millimetres of rainfall fell within 24 hours over the catchment, representing an inflow into the reservoir of approximately 450 t.c.m. Clearly the 15 inch main was quite inadequate for the purpose of lowering the reservoir. The decision was made to bring pumps onto the site. However, the layout and condition of the access road was a serious restriction, and a Royal Air Force helicopter was used to ferry some equipment to the site and proved especially useful to move pumps as the reservoir level dropped, and also for moving stores to less accessible parts of the reservoir.

3.4.16 Settlement

Another item which may be described as maintenance is restoration of those crest areas of a dam which have sustained progressive settlement.

Herrin Dam, Illinois was 0.6 m lower in the centre than the rest of the crest, as shown in Figure 3.5.[91]

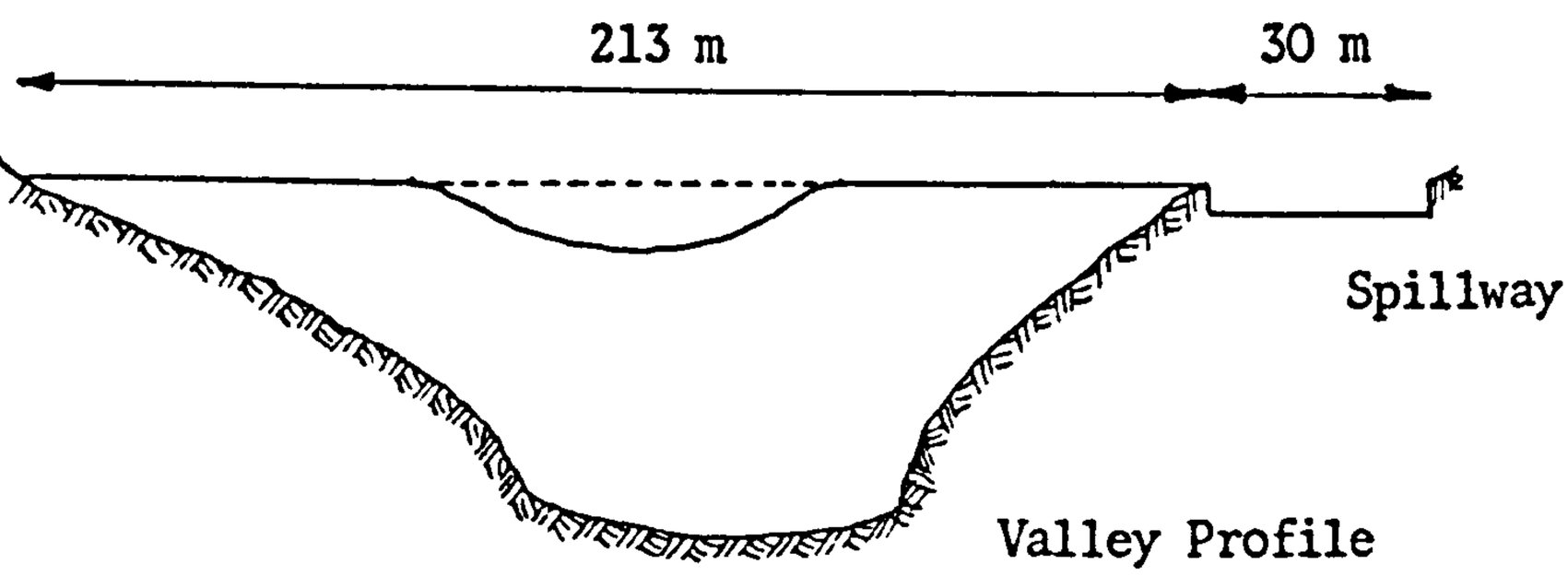


FIGURE 3.5: Longitudinal Section of Herrin Dam

Overtopping took place, with the consequent loss of the dam. Insufficient care with compaction can promote settlement, and since the first heavy smooth roller was developed in the 1860's and the first sheeps-foot roller in 1904, dams built before this time should be eyed with suspicion.

Johnstown was another instance where the dam was up to one metre lower in the centre than at the ends. The failure in 1889 caused the loss of 2280 lives.[92,93,94]

Thorough and regular inspection should be part of any safety programme since especially in the case of privately owned dams settlement can go unnoticed, and it is common practice to find fish screens and even chainlink fences across spillways.

3.4.17 Empty/Abandoned Reservoirs

Many, perhaps thousands, of small abandoned reservoirs exist in this country, particularly close to the industrial areas of the North. The dams, built to serve the woollen or mining industries, were built in the eighteenth and nineteenth centuries, and in many cases their ownership is obscure. There is, perhaps not unexpectedly, a reluctance for bodies or individuals to accept responsibility for the structure, and it will therefore fall on the enforcing agency, under the Reservoirs Act (1975) to ensure the necessary maintenance is provided, or to generally enforce the Act.

Many of the dams which are privately owned are, more often than not, unlikely to receive maintenance unless they are subject to a Statutory Inspection, and even then they are unlikely to receive maintenance between inspections.

An empty or abandoned reservoir can be potentially more hazardous than a structure that is in use. Unless provisions are made so that the structure cannot be filled it is possible that the reservoirs could refill in periods of heavy rainfall even if any remaining outlets are left open. A reservoir in such a state and with no supervision can be particularly hazardous.

3.4.18 Cascade Failures

A relatively small dam at the head of a watershed can be over-topped by a short duration, high intensity rainfall. If this structure fails it can trigger a cascade failure or 'domino' effect on dams downstream, and so even a well-designed lower dam with adequate spillway capacity for normal situations can fail in this way with serious

cumulative effects downstream.

Examples of cascade failures and the number of dams involved are shown in Table 3.4.

The Credit River Dam of the Erindale Power Company of Ontario, Canada failed when the rainfall had become so severe on April 6 1910 that a small dam in the upper part of the catchment failed.[57] By the following morning this had so aggravated the flood conditions that other small dams lower down the river failed completely or in part, creating a flood which overtopped the dam of the Erindale Power Company. The possibility of such a situation had been anticipated, and attempts to save the dam by increasing its height with sandbags were made, but to no avail.

If possible, allowance should be made in the design for the situation when an upstream dam fails, both in reservoir capacity and in discharge capacity at the spillway and outlet works. The provision of increased height and freeboard in a dam for flood storage is an economic method of reducing flood bypass costs so long as the reservoir area is large, and this may be especially useful where a possible cascade situation exists. There is, however, a great temptation on the operator's part to use this allowance for normal storage since this storage capacity may only be used once every 25 years.

The discharge capacity of outlet facilities should be such that the water level of a reservoir may be lowered by, say, half its total depth comparatively quickly (of the order of, say, 28 days) and so avoid situations similar to the one at Lluest Wen already mentioned, but it is appreciated that in the case of an extremely large reservoir,

Name of Dam(s)	No. in Cascade	Year of Incident	Name of Dam(s)	No. in Cascade	Year of Incident
Akerman	2	?	Sherburne	2	1905
Beaver Creek	2	1931	Telluride	2	1909
Clinton	2	1938	Upper Shelton	2	1903
Cocker Cobbs	2	1970	Yuba	2	1907
Coedty	2	1925	Zerbino	2	1935
Dells	2	1911	Gallinas	3	1957
Desert	2	1930	Harlem	3	1893
Escanaba	2	1930	Horse Creek	3	1935
Fairmont	2	1822	Middlefield	3	1901
Fishkill	2	1897	Bostwick Pond	4	1934
Iron River	2	1923	Embsay Moor	4	1908
Log Falls	2	1923	Frackville	4	1901
Lowercroft	2	1852	Citydam	5	1909
Middle	2	1909	Kenwood	5	1975
Norwich	2	1905	Alton	5-6	1889
Panchet	2	1961	Erindale	16	1910
Providence	2	1901	Hatchtown	16	194
Randells Pond	2	1901	Swift	26	1964

TABLE 3.4: Examples of Cascade Failures

the expense would be prohibitive.

The facility of a large outlet capacity may aid, if sufficient time exists, in a cascade situation, in as much as it may be possible to reduce the level of the reservoir at a rate which approaches or equals the inflow. Obviously this sort of facility would have to be the subject of an economic analysis since an increase in reservoir levels while passing maximum floods which are generally of low frequency raises the cost of the dam considerably. It is therefore advisable to compare the cost of providing more flood storage capacity with other alternatives such as restricting the maximum level in the reservoir by providing extra discharge capacity through spillway or outlet sluices.

Where a potential cascade situation exists the upstream dams should be inspected. Elimination from a hydrological analysis of those reservoirs whose size would not jeopardise the dams downstream would be automatic. Dams not eliminated in this process should be analysed hydrologically and their spillway adequacy checked.

3.4.19 Canals

Canals are not subject to the Reservoirs Act (1975) or to the Reservoirs (Safety Provisions) Act 1930. There are 3100 kilometres of canal in Britain, some of which is at a considerable elevation above houses, villages and towns. Hyde[95] argued that breaches occur very infrequently, that the volumes of water involved are very small, and the velocity of flow would be small. The fact remains that accidents have occurred; Peak Forest Canal (1973)[96] and the Leicestershire and Northampton Canal (1865)[97] are examples. The latter failed during a

storm in August 1865 when water overtopped the embankment causing a breach and releasing 200000 cubic metres of water since the distance between locks at the point of failure was 16 kilometres.

It is suggested that a canal can present a very real hazard, and that since access is often very limited to sections of canals any breach could be difficult to repair. The fact that the task of applying reservoir legislation or indeed any type of legislation to canals would be complex is not a valid reason for excluding canals from a safety programme.

A Report released by the Department of the Environment in November 1977 confirmed that much of Britain's 3100 kilometres of canal are a threat to public safety due to inadequate repairs and maintenance. It is estimated that £60 million are needed to make the necessary repairs, £5 million of which is urgently needed to protect public safety. Approximately sixty per cent of the costs are required for repair and reconstruction of bank protections.[98]

3.4.20 Landslides

Landslides have been known to cause overtopping, the most notorious instance being at Vaiont arch dam which was overtopped by a flood wave with a height in the order of 100 metres.[34] The left bank of the reservoir was known to be moving as early as 1960. When the reservoir level was slowly reduced the creep movements eventually ceased. In October 1963 the water level rose and the bank started to creep again with a velocity of less than 1 centimetre per day. Slow reduction of the reservoir level could not reduce the sliding velocity on this occasion, and the velocity increased to 20 centimetres per

day on the day of the catastrophe. The creep changed to a slide with a velocity of 2500 centimetres per second, and 250 million cubic metres of material moved into the reservoir.

Material which has slipped from the reservoir side can displace water, creating a situation equivalent to a huge storm inflow, and cause overtopping. Reservoir slopes should be regularly surveyed and examined.

A landslip caused the failure of Fruitgrowers Dam in June 1937, in as much as material covered the outlet pipe. A man-made outlet was made in the crest at a point where the embankment was less than 2 metres high to alleviate the situation. Although the trench was only 1 metre wide, the flow of water deepened and widened the channel so rapidly that the 41215600 cubic metres of water stored was released in nine hours.[99]

A mountain landslide which blocked the spillway and heavy rainfall combined to cause the failure of Ogaryarindo Tameike Dam, Japan in 1944.[100]

Landslides can also form a dam in themselves - a natural dam. This was the case on the River Bireh Ganga when a landslide in September 1893 completely filled the valley of the stream to a depth of 243 metres. Run-off from the catchment of 233 square kilometres was held behind the natural dam for nearly a year, forming a reservoir having a capacity of 60 million cubic metres. Breaching occurred due to overtopping and seepage on August 25 1894. Warnings had been given with the result that no lives were lost.[101]

3.4.21 Slide/Failures

Wave spray overtopping a dam or passing through gaps in a wave wall as well as water deposited by overtopping can initiate a slip or slide failure in the downstream shell of the dam which may lead progressively to the structural failure of the bank and overtopping or to the reservoir breaking through. Slips or erosion of the mitres have occurred at Auchendores[102], Bilberry[43], Blithfield[103]; Ballochdowan[47], Greenwood[44], and Mill River.[39,45,46]

The mitres of Bilberry Reservoir were eroded when run-off from a severe storm was directed down the mitres.

The design of a wave wall must consider where wave spray is directed when it hits the wall in order to prevent spray being directed upwards which would then be blown onto the downstream face. Wave spray should be deflected back towards the reservoir.

At Auchendores Reservoir water was driven through spaces in the joints of the wave wall which then collected at the toe of the embankment and ran along the base of the embankment, apparently weakening the structure. The poor nature of the sub-soil on the down-stream side of the channel and to some extent the presence of an old bund all seem to have contributed to the failure which occurred along the junction plane of the old and new banks. Spray passing through the gaps in the wave wall cut knife-like gaps in the turf for a distance of up to 5 metres, posing the question whether a less well manicured embankment might be more resistant to erosion caused by heavy, wind blown spray.

3.4.22 Damage caused in the Valley downstream of the Dam

The damage caused downstream of a dam when failure occurs depends on whether the storage is released slowly, in what is sometimes termed "in a controlled manner", or whether the storage is released virtually instantaneously.

An additional factor which affects the amount of damage caused is the configuration of the valley. A steep narrow valley results in the velocity of the flood wave being maintained, whereas a gently sloping valley or a wide valley results in the water spreading out over the valley floor with a consequent reduction in velocity and therefore in the destructive energy of the flood wave.

The effect of valley shape is demonstrated when one looks at the failure of Bilberry (Holmfirth) Dam. The volume of water released from Bilberry reservoir (24 metres deep) was 390 t.c.m. The construction of the Bilberry Dam was such that the embankment yielded and eroded relatively rapidly, discharging the whole of the reservoir's storage into a steep and narrow valley down which the water flowed with velocity.

Johnstown Dam (South Fork) which failed in 1889 with the loss of over 2200 lives was not a large dam by today's standards, having a height of 22 metres, although it did have a capacity of 22370 t.c.m. Again, the reservoir failed comparatively quickly, releasing its contents over a period of about 45 minutes into a steep narrow valley where the destructive force of the water was maintained.[104]

It is therefore clear that the manner in which the water is released, the configuration of the valley downstream of the dam and

the velocities maintained by the flood wave are important factors which can affect the amount of damage caused.

The average velocity of flow experienced with some known failures are shown in Table 3.5.

Obviously as the flood wave proceeds down the valley energy dissipates and the velocity decreases as the valley floor widens. This is demonstrated by the velocities observed in the valley below the Saint Francis dam which are shown in Table 3.6.

3.4.23 Corewalls - Steel/Concrete

The use of 'rigid' corewalls of concrete or of steel diaphragms is now relatively rare.

A puddled core is likely to erode progressively, but this is not the case with a 'rigid' core. When overtopping does occur, the downstream shell of the dam is usually removed exposing the core. If the core becomes exposed it may remain in position and intact, in which case it can be advantageous since some storage is retained. However, in more cases than not, since any core must be designed to accommodate any deformations that are likely to occur in the dam and therefore must not be totally rigid, the core fails catastrophically and releasing storage instantaneously rather than progressively.

This was the case with Lower Otay Dam built in 1897 and which failed in January 1916. Water overtopping the dam removed the downstream rockfill exposing the core. The diaphragm then tore and 'opened like gates' releasing the storage.[49]

Dam/Flood	Approx. Velocity of Flood Wave (km/hr)	Height of Dam (m)
Sepultrida	14	20
St. Francis	16	56
Lower Otay	20	41
Hell Hole	23	61
Dale Dyke	29	29
South Fork	29	23

TABLE 3.5: Observed Velocities of Flood Wave at Selected Failures

Distance from Dam (km)	Velocity of Flood Wave (kph)
15	20.6
28	20.1
48	19.8
62	18.0
74	11.9
87	9.5

TABLE 3.6: Observed Velocities of the Flood Wave from the St. Francis Dam[105]

The core remained intact and in position when the 14 metre high Cowlyd Dam was overtopped on the night of 31 December/1 January 1924/25. The downstream clay shoulder of the dam was removed, leaving the concrete core unsupported and acting as a cantilever but retaining the reservoir storage. The downstream shoulder was quickly backfilled, narrowly averting a disaster.[106]

3.4.24 Extreme Rainfall

A study of data in Appendix B, 'Known Instances of Dams which have been Overtopped', seems to indicate that 1968, with 19 known instances of overtopping, was something of a 'freak' year. Of these 19 known instances at least five occurred between the 14 and the 16 September.

Rainfall data[107] for that year reveals some interesting facts. The total rainfall over England and Wales for 1968, though marginally less than 1967, was substantially greater than average, and on this occasion for the fourth consecutive year. September 1968 was described as 'wet and cyclonic with heavy rain over most of the British Isles from the 5 till the 23rd'. The total rainfall over the period of the 14 and the 15 of September was just over 200 millimetres.

During the period 1863 to 1967 rainfalls exceeding 5 inches (125 millimetres) have only been recorded on 151 occasions of which 73.5 per cent occurred in mountainous regions of the West and North (e.g. Dartmoor, S.W. England, N. Wales, Lake District and the Scottish Highlands).

Rainfall of these magnitudes during the early autumn have previously been limited to four known occasions, and it is interesting

to note that they occurred outside the areas listed above. These rainfalls are shown in Table 3.7.

In general the Meteorological Office describes the periods 1912-1924 and 1952-1968 as being 'wet' periods.

3.5. General Considerations

Analysis of data from a sample of 539 British dams is presented in Figure 3.6 and Figure 3.6a. While it is thought that this sample is statistically suitable for the purpose of analysis, it is realised that the data did not include service reservoirs which would tend to move the curve in Figure 3.6 to a new position as shown, while inclusion of the many elderly small privately owned reservoirs, about which very little is known, would tend to restore the curve towards the original position. Precise calculation is virtually impossible due to the absence of records for many dams.

The data displayed in Figures 3.6 and 3.6a indicates that the mean age of dams in Great Britain is between 80 and 90 years, the age distribution curve showing a decline in dam building, as one might expect, during the First and Second World Wars.

Analysis of some 150 British dams, still in service, which have required modifications to their spillways to increase capacity, shows a large number of dams built in the latter half of the nineteenth century to be deficient by present day standards in spillway capacity as shown in Figure 3.7 and Figure 3.8.

Figure 3.9 shows the number of incidents of overtopping that have occurred per decade as defined by Appendix A. The peak at 1900 is at a

Date	Location	Number of Known Instances of Overtopping
25 August 1912	Norfolk	2
26 September 1933	Fleet, Hampshire	3
16 September 1947	Wisley, Surrey	1
5 October 1958	Knockholt, Kent	1

TABLE 3.7: Rainfalls exceeding 125 millimetres (5 inches) for the Period 1863-1967[107]

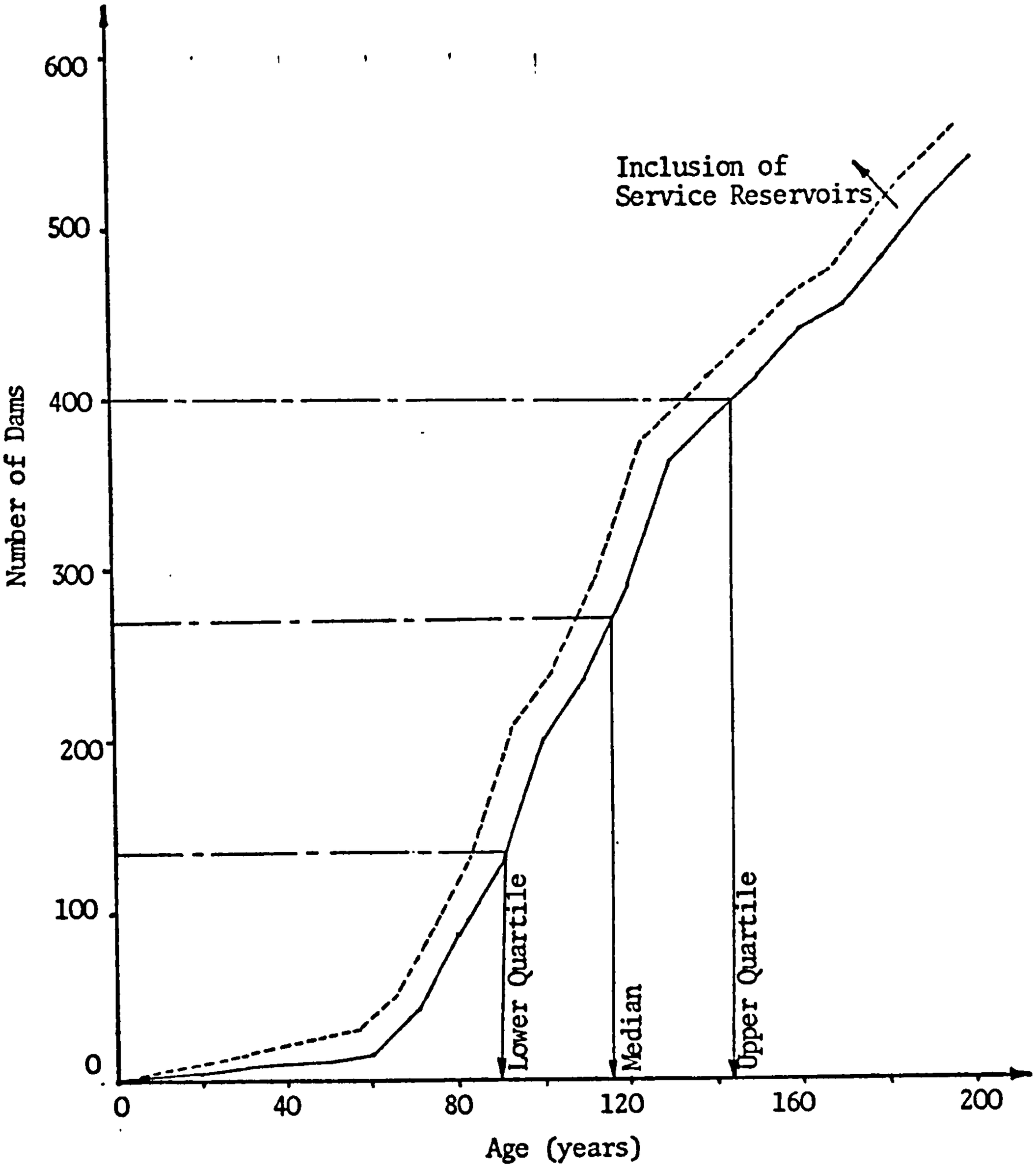


FIGURE 3.6: Cumulative Distribution of a Sample of British Dams (Dams >22.7 t.c.m.)

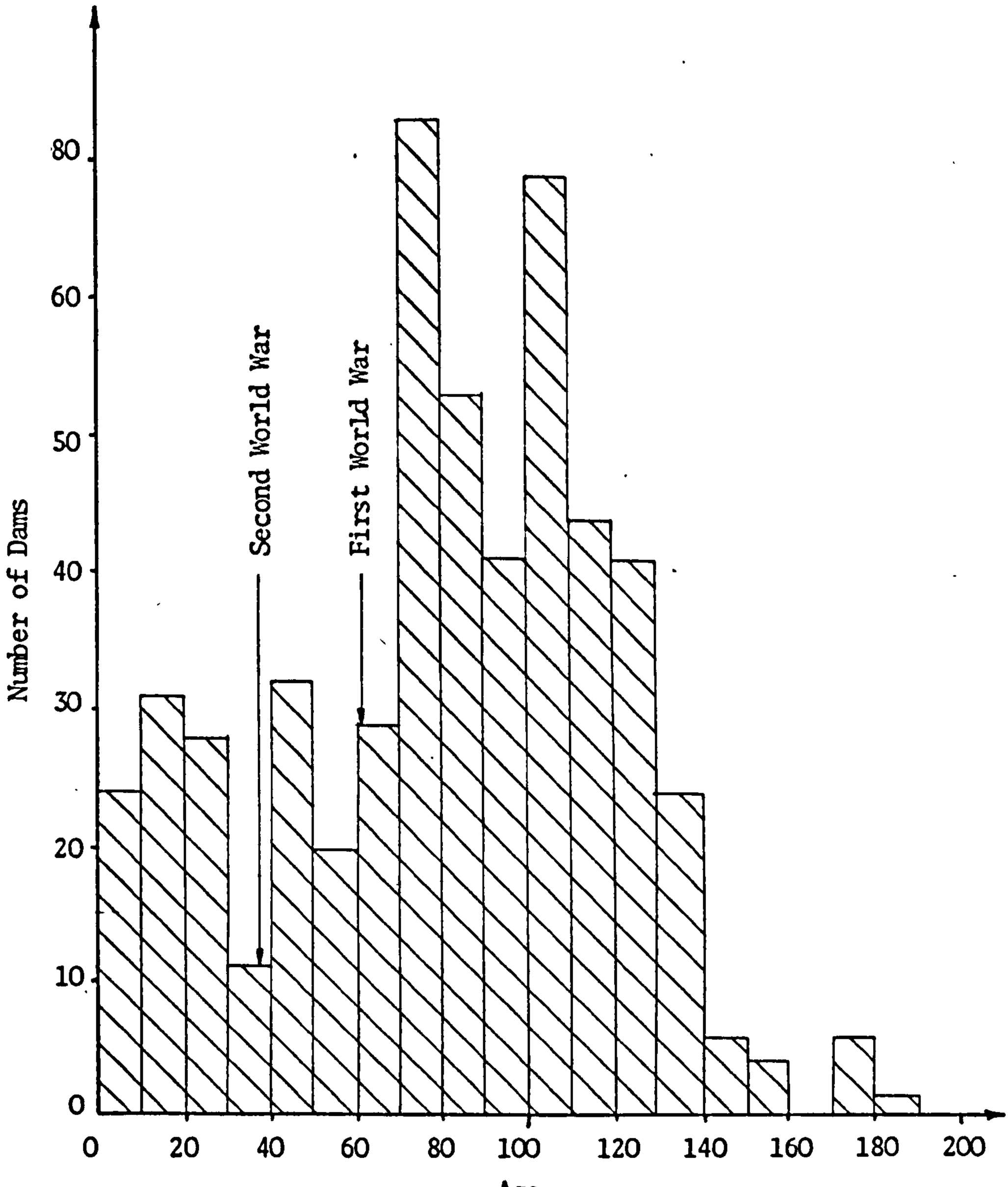


FIGURE 3.6a: Age Distribution of a Sample of British Dams



FIGURE 3.7: Year of Construction v. Number of Dams known to have required Remedial Measures (British Dams)

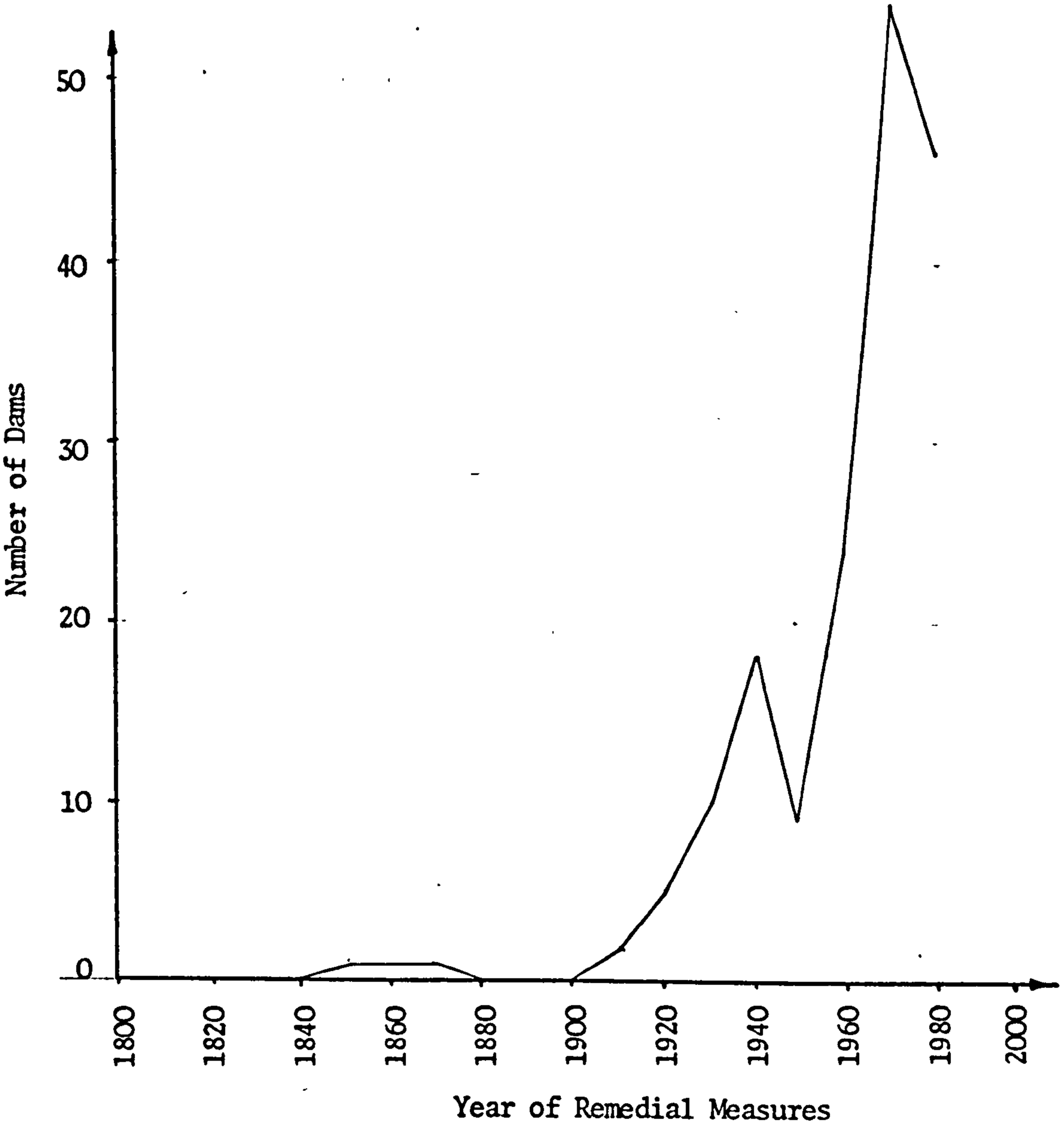


FIGURE 3.8: Year of Remedial Measures v. Number of Dams (British Dams)



FIGURE 3.9: Year of Incident/Failure v. Number of Incidents/Failures (545 known Incidents/Failures throughout the World)

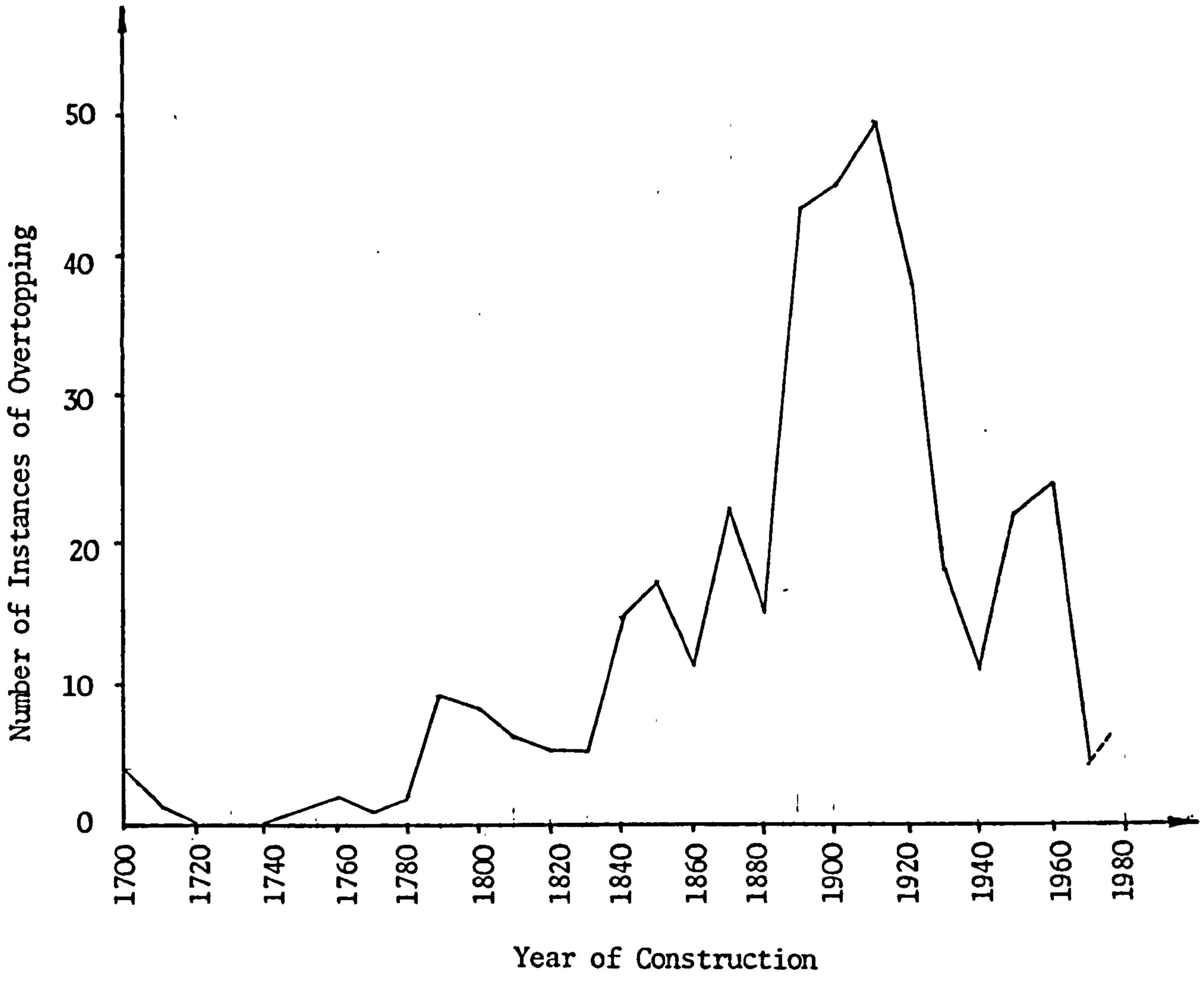
time when large numbers of dams were being constructed and when dam construction was becoming more adventurous. However, the peak during the period 1960-1970 is thought to be due to improved reporting and surveillance techniques, although it may be argued that this may mark the end of the 'design life' of many of the dams built around the turn of the century.

This trend is emphasised by the curve shown in Figure 3.10 showing a high overtopping incident rate for dams constructed around the turn of the century and for dams constructed during the 1960's, although once again it may be argued that the 1960 peak is a result of improved reporting techniques.

The results of a study of the age of the dam at the time at which overtopping occurred is shown in Figure 3.11. It is interesting to note that the majority of incidents, some 38 per cent of cases studied, occurred either during construction or within the first year of service. The construction phase of any project is a particularly hazardous time when the works are in an incomplete and obviously insecure and vulnerable state, with only temporary flood by-pass works, which represent a compromise between safety and cost. Also displayed in Figure 3.11 are 'peaks' at fairly regular intervals which might be interpreted as the expiry of the flood return periods used in the design of the dams concerned. The corresponding graphs for the British 'sub-sample' are shown in Figures 3.12, 3.13, 3.14 and 3.15.

It has been suggested that many of the failures and incidents due to overtopping which occurred at the turn of the century were due to a desire to promote bolder schemes at a time when the necessary design

FIGURE 3.10: Year of Construction v. Number of Instances of Overtopping (376 Incidents throughout the World)



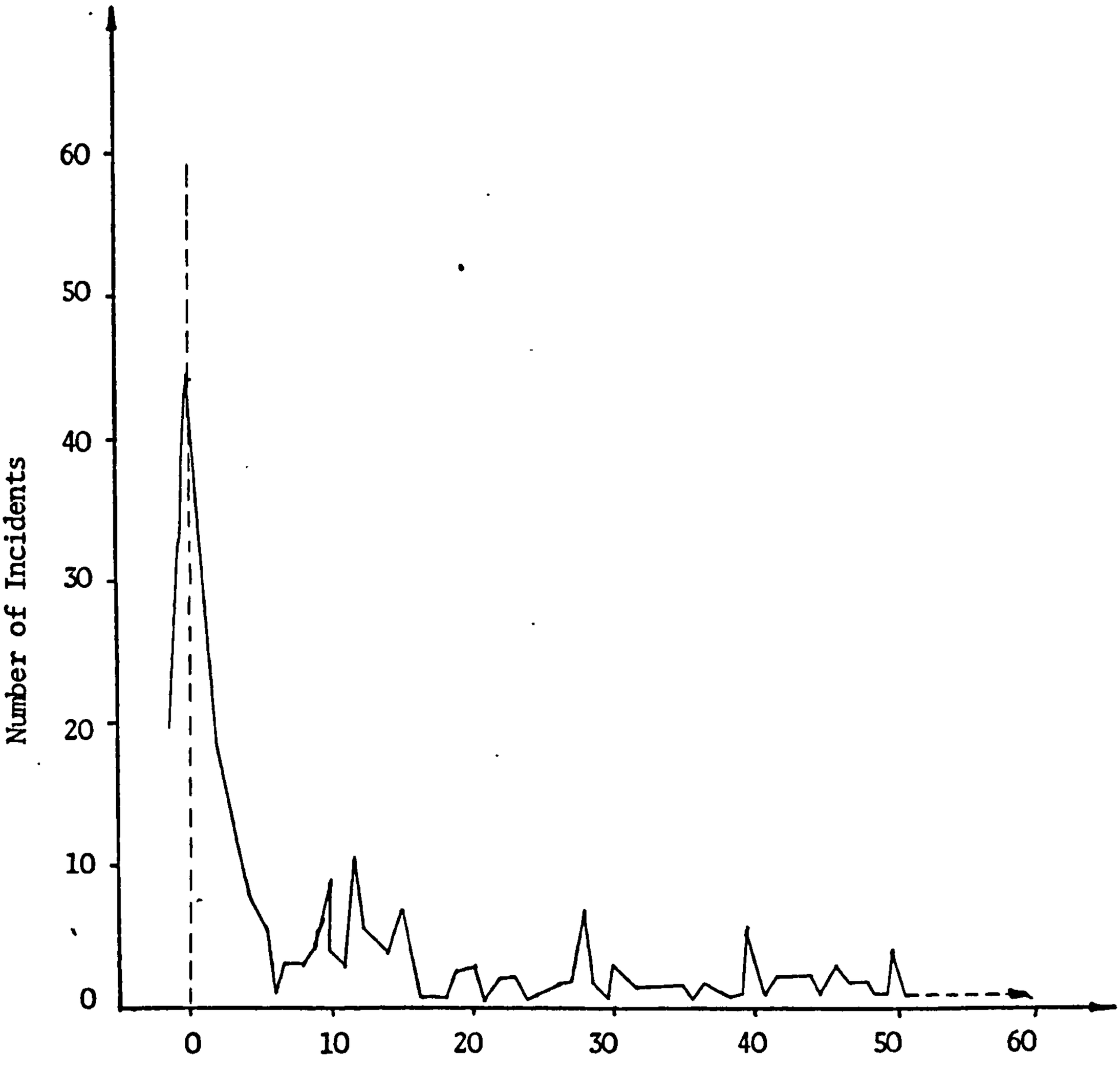


FIGURE 3.11: Age of Dam at Time of Incident/Failure v. Number of known Incidents/Failures (Incidents/Failures to 318 Dams throughout the World)

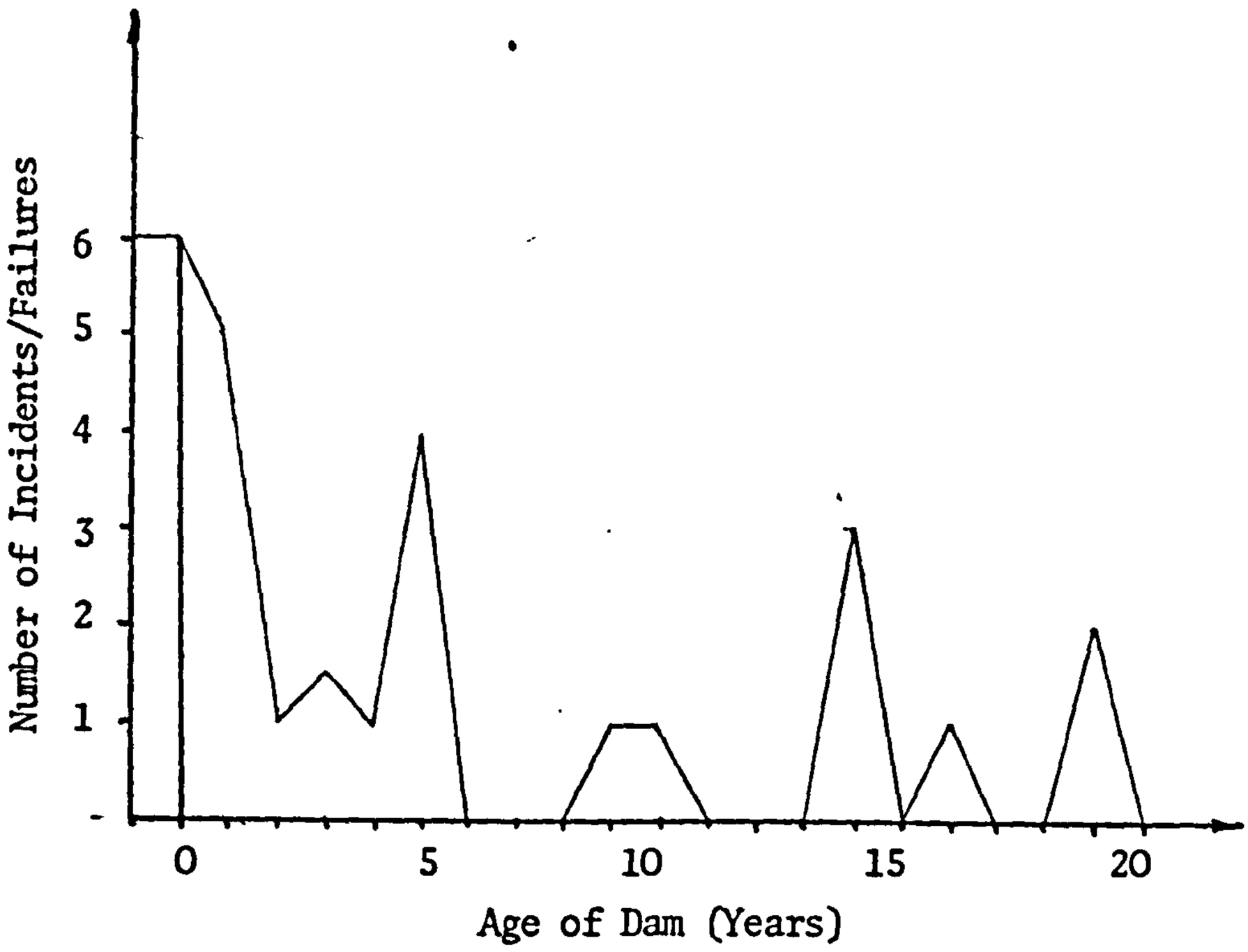


FIGURE 3.12: Age of Dam v. Number of Incidents/Failures (Incidents/Failures to 121 British Dams)

FIGURE 3.13: Year of Incident/Failure v. Number of Incidents/Failures (Incidents/Failures to 121 British Dams)



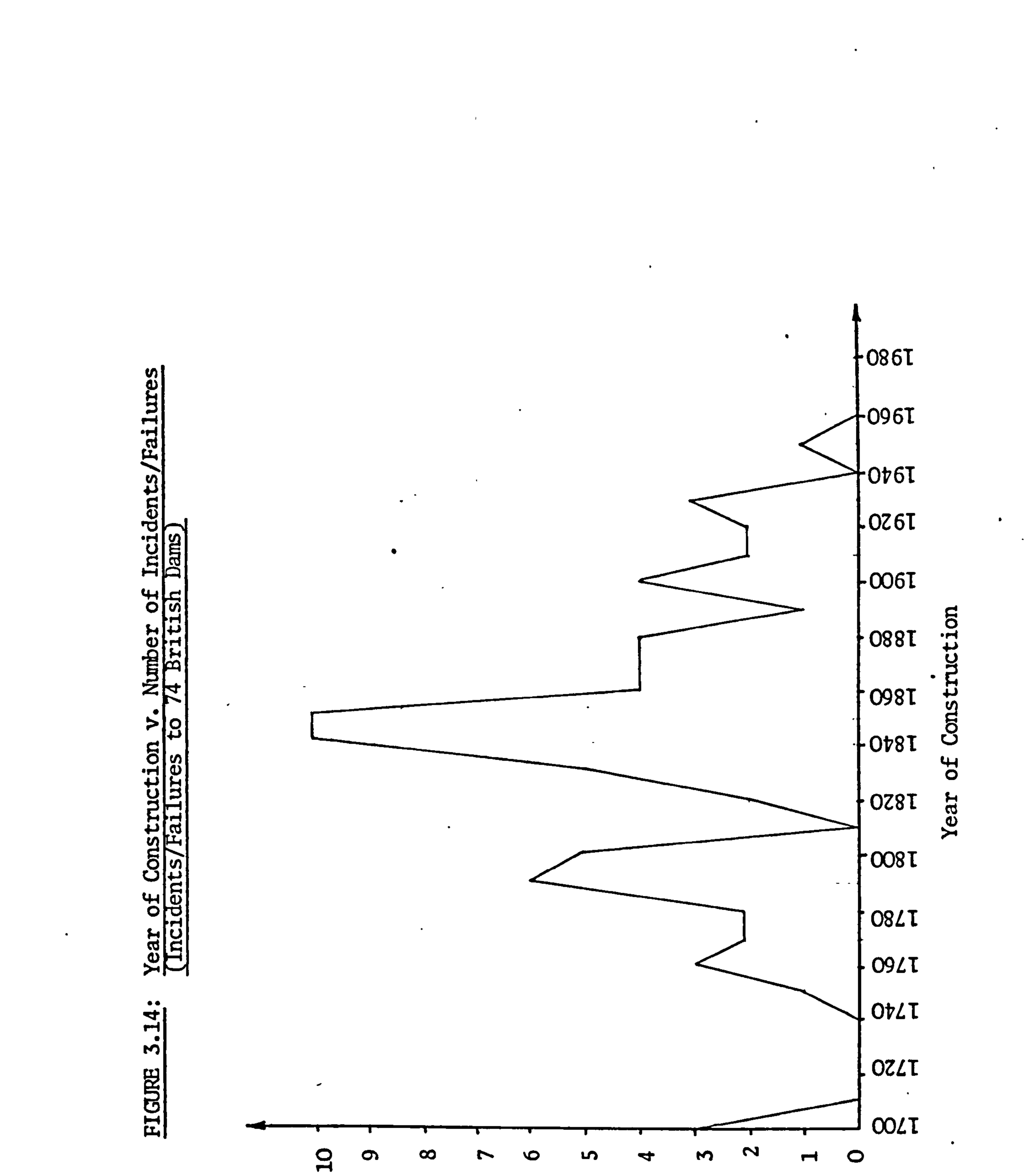

Year of Construction v. Number of Incidents/Failures
(Incidents/Failures to 74 British Dams)
Number of Incidents/Failures
0
1
2
3
4
5
6
7
8
9
10
Year of Construction
1700
1720
1740
1760
1780
1800
1820
1840
1860
1880
1900
1920
1940
1960
1980

FIGURE 3.14:

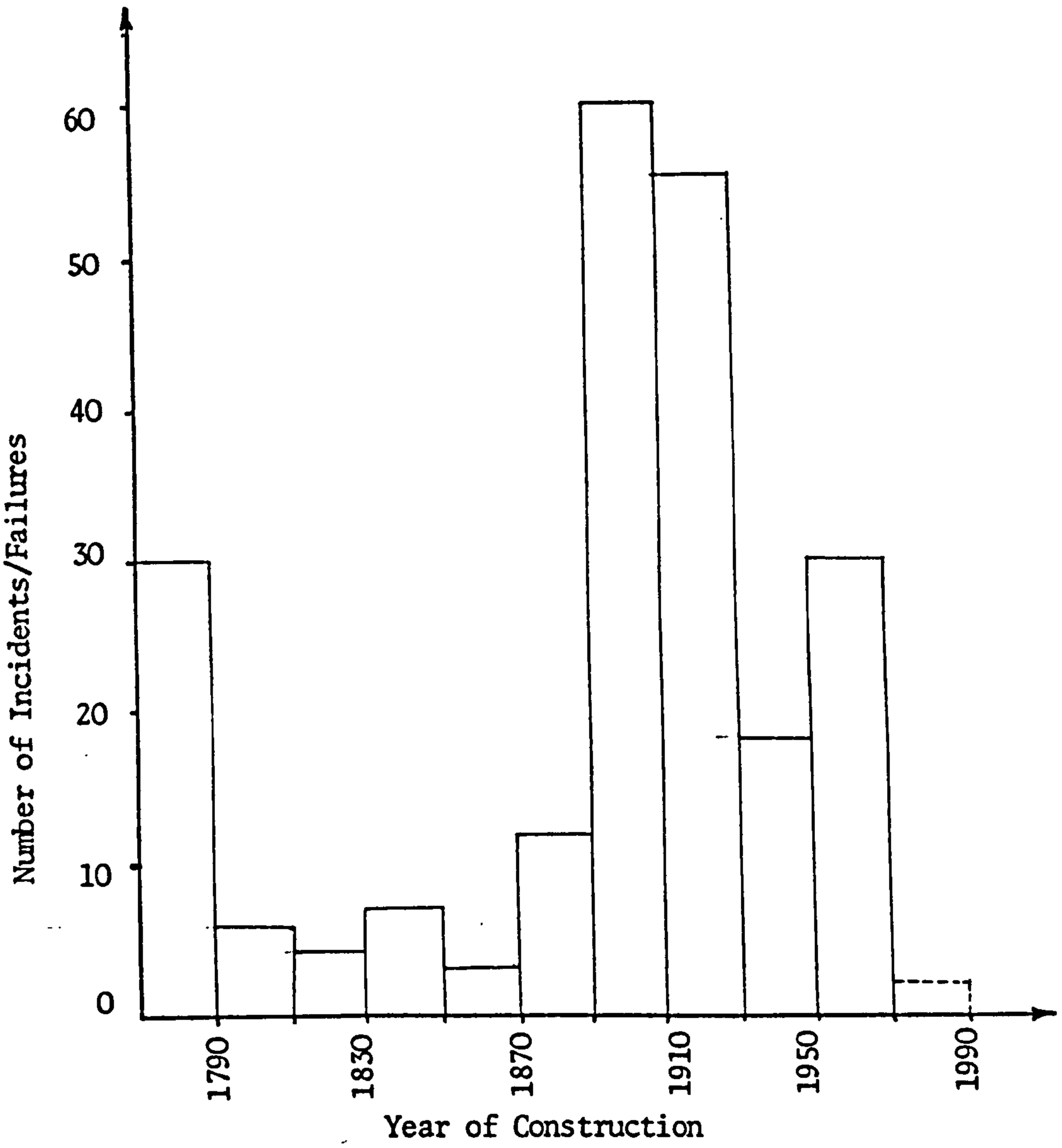


FIGURE 3.15: Year of Construction v. Number of Incidents/Failures (227 Failures/Incidents throughout the World)

criteria had not been established. Dams built at this time were usually quite massive structures having shallow slopes, large crest widths and very large base thicknesses. This may afford, by virtue of the immense quantities of material involved, some 'protection' against overtopping, and this is often the 'defence' offered by some supervisory authorities when questioned about the safety of dams.

It is clear that this country has been rather fortunate, as experience in other countries shows what could happen. The good record must be attributed very largely to good fortune.

An attempt to assign an index to dams in order that the potential hazard offered by any one dam could be established has been and is the subject of research.

3.6 Hazard Index and Safety

Sarkaria[108] advocated a hazard indexing system for dams. Since no two dams are alike and can differ in type, size, location, foundation, geology, design, construction method, maintenance programme and age, it follows that not all dams are equally safe. The degree of risk represented by a dam can change with age and circumstance, e.g. development downstream and in the surrounding area. It can be said that the safety of a dam is governed by natural factors, design and constructional factors and evolutionary factors.

The system Sarkaria suggested was a classification of hazard potential based on the volume of the reservoir (V_r), the height of the dam (H_d), distance to any principal downstream population and property centres (L), the elevation drop between reservoir surface and the

population centre (D), a value index of endangered population and property (P), and a numerical index representing the inherent relative overall stability of the different types of dams (T_d).

Thus Sarkaria's Dam Hazard Potential Index (DHPI) was expressed as:

$$DHPI = f(V_r, H_d, \frac{1}{L}, D^2, P, T_d)$$

The last index, T_d, makes a distinction between types of dam, and Sarkaria suggested that an arch dam represents one quarter of the hazard of a mine-tailings dam, one-third the hazard of a hydraulic fill dam, one half the hazard of a rockfill or rolled earthfill dam and two-thirds the hazard of a concrete gravity or buttress dam. Thus, this very empirical index T_d can be expressed in the following relative terms:

Arch dams	1.0
Concrete gravity and buttress dams	1.5
Rockfill and rolled earthfill dams	2.0
Hydraulic fill dams	3.0
Mine-tailings dams	4.0

Sarkaria also suggested that other factors representing age of the dam, spillway and outlet capacity and reservoir situation might also be quantified and added to the expression for DHPI. Ease of access, shape of the downstream valley, both for maintenance and in times of emergency, type of control structure and upstream reservoirs which may lead to a cascade failure are not included, and it is the Author's opinion that these factors should be included in any such indexing

system. Related work is being carried out by Moffat[109] on a Reservoir Hazard Index classification, and these factors will be accounted for.

It has been made clear that, in the Author's opinion, the arbitrarily set value of 22700 cubic metres used to define whether a dam is subject to any statutory safety legislation is far from satisfactory. The value of 25 t.c.m. stems from a rounding up of the 5000000 gallon criteria used for the 1930 Act, which was adopted because the failure at Skelmorlie in 1925 involved a reservoir whose volume was 5000000 gallons (22700 cubic metres). One may question what criteria would have been chosen if the failure in 1925 had involved a reservoir of 5000 cubic metre capacity. It would seem to be much more logical to use a Hazard Index rating, although it may be initially expensive to apply it to all dams. In this way even the so called 'small' dams which threaten communities in remote areas of this country would be subject to supervision appropriate to the risk, and the approval of small remote dams which were not even potentially dangerous would be almost routine upon application, provided that there had been no change in circumstances since the previous application. In this way the Hazard Index rating is used to establish the relative importance with respect to safety of a structure.

3.7 Warning Systems

The flood released from a dam becomes dangerous when it exceeds the capacity of the river channel. The height of the possible flood wave can be estimated using Di Marchi's formula[27], and the height to which a flood would rise in the event that the complete storage is released should be marked. It is interesting to note that slow-moving

water may not be particularly hazardous unless greater than one metre in depth after which it becomes difficult to stand. Markers at the flood level would enable the populace to ensure they are clear of the path of the flood wave at time of emergency.

Inundation maps showing the areas inundated if a dam fails are sometimes prepared, but it must be appreciated that, if common knowledge, these measures may create a situation whereby people assume that the preparations indicate that the dam is at the point of failure.

The use of a warning system has been advocated by many engineers, and is considered to be a necessary design feature, but care must be taken in the design and implementation of any warning system. The populace must know what to do and where to go if the alarm is sounded. The installation of any warning system, as already stated, often creates a fear of dams just by virtue of its presence, but when it is considered that over 120000 people were evacuated from the area beneath the Oros Dam in Brazil before failure it can be a very useful safety feature, especially for dams for which the downstream area is densely populated. False alarms must be avoided since they would lead to a loss of faith in the system. Automatic alarms have also been suggested, but these are surely subject to natural disturbance, vandalism and therefore to false alarms. It is therefore considered that actuation of an alarm should be controlled by one responsible person in authority.

3.8 Erosion Resistance

The laboratory study reported in Chapter 5 and 6 and information reported in Appendices A and B also provided additional evidence to suggest that compacted clay fills may be considerably more resistant

to erosion by overtopping than is traditionally accepted to be the case. Considered in the context of the implications of the NERC Flood Studies Report (1975)[21] and related documents, appreciable resistance to erosion under exceptional flood conditions could be significant in relation to many older embankment dams with spillways marginally inadequate by today's standards.

It is appropriate at this stage to refer to such field data on erosion as is available from overtopping failures and incidents to earthfill embankments. Examples of failures where semi-qualitative observations have given some insight into the failure mechanism and rate of failure include:

1. The South Fork Dam, Johnstown, Pennsylvania, some 23 metres high, failed on the 31st May 1889, causing the loss of some 2280 lives.[92] The original specification stated that the dam should have a freeboard of 10 feet (3 metres). The dam was originally completed in 1839 to supply a canal. The dam ceased to be used for this purpose in 1857. The reservoir was subsequently used to supply a railway, ownership finally passed to a local hunting and fishing club who repaired a relatively minor breach caused by a flood which partially overtopped the dam during the winter of 1880. Unfortunately in repairing the breach, the fishing and hunting club "took out the five sluices in the dam, lowered the embankment 2 feet (0.6 metres) and subsequently partially obstructed the wasteway by gratings etc. to prevent the escape of fish.

The original design had specified two spillways with a total channel width of more than 150 feet (46 metres) 'cut in the solid

rock'. As constructed, however, the single spillway channel was actually less than 70 feet (21 metres) in width. The earthfill upstream shoulder of the embankment was constructed on a rockfill downstream shoulder, the earthfill being prevented from being washed through the rockfill by "hay and brush". The embankment had an upstream slope of 2 to 1 and a downstream slope of 1½ to 1, both slopes being covered with rip-rap.

An account of the failure by J.G. Parker[92], a civil engineer working in the area at the time, tells of attempts to save the embankment as overtopping threatened:

"Half of the men (8) were cutting a ditch through the shale rock at one end of the breast. This ditch was cut through the original ground and about 25 feet from the constructed portion of the breast. The shale was so tough that they could not cut it more than about 14 inches deep and about 2 feet wide, but when it was cut through to the lake the water rushed in and soon made it a swift stream, 25 feet wide and about 20 inches deep, but the rock was so hard that it could not cut it any larger than this."

The remaining men "had a plow at work throwing up a furrow, and thus raising a temporary barrier or breast to retard the water flowing over the dam, should it reach that height which it was gradually doing". "The water in the lake rose until it was just passing over the breast, notwithstanding that the lake had then two outlets (the waste weir and the one cut by the laborers). The breast was slightly lowered in the center and the water washed away our temporary embankment thrown up by the plow and shovels, and the water was passing over in many places in a

distance of 300 feet about the centre of the breast."

Parker reported: "The water was cutting the outer face of the dam, but not as badly as I feared it would, its greatest effect was on some portions of the roadway which crossed the breast where the roadway had been widened on the lower side by the addition of a shale earth or disintegrated shale, upon which the action of the water was instantaneous, but the heavy rip-rapping on the outer face of the dam protected this wash and the water cut little gullies between each of the large stones for rip-rap."

Parker considered cutting a spillway through the embankment at one end, but decided against this action as he realised that it would have meant the destruction of the dam, and he also hoped that the level of the reservoir would cease to rise. He later found that "the water on the breast had washed away several large stones on the outer face and had cut a hole about 10 feet wide on the outer face and about 4 feet deep, the water running into this hole cut away the breast in the form of a step both horizontally and vertically, and this action went on widening and deepening this hole until it was worn so near to the body of the water in the lake that the pressure of the water broke through, and then the water rushed through this trough, and cut its way rapidly into the dam at each side and the bottom; and this continued until the lake was drained." Reports state that, on total breaching, the water level dropped some 18 m in a 'matter of minutes'.

Parker stresses that "the dam did not break, but was washed

by the water passing over it from 11.30 o'clock a.m. until nearly 3 p.m. until the dam was made so thin at one point that it could not withstand the pressure of the water behind it, and the water once rushing through this trough nothing could withstand it."

The storage, some 22 x 10^6 cubic metres of water, was released into a steep deep valley and the resulting flood causing great devastation and, partly through fires caused in Johnstown, the loss of some 2280 lives.

2. The Oros Dam, on the Jaguaribe River in Brazil, was under construction in 1960 when a flood breached the embankment, causing destruction of property, but fortunately with comparatively little loss of life because of large-scale evacuation of the population.[110]

Attempts were made to raise the embankment by placing uncompacted fill on top of the embankment at the rate of 1 metre rise per day. The continuous rainfall made the use of heavy machinery practically impossible, and since the lake was rising at the rate of 1.3 metres per day it soon became obvious that the embankment would be overtopped.

Evacuation of the populace had commenced, but attempts were continued to save the embankment. Metal sheets were dropped from aircraft with the object of protecting the surface from erosion.

The embankment was eventually overtopped over its total length of 620 metres by approximately 0.8 metres of water. A breach formed and began to widen towards midday of March 26th 1960, some 12 hours after water had started to flow over the embankment. The resulting breach was 150-200 metres wide and

released the storage of the reservoir, some 650 x 10^6 cubic metres, over a period of approximately 18 hours.

3. Lake Malloya, New Mexico was overtopped in 1942 for a period of six hours but did not fail.[111]

4. De Mello[112] has first-hand experience of a compacted clay embankment withstanding water flowing over it to a depth of up to one metre for several hours without failure.

5. The failure of the Euclides da Cunha and Armando de Salles Oliveira dams, two of the three dams on the Pardo River used by the Centrais Eletricas de Sao Paulo (C.E.S.P.) for hydro-electric power generation, occurred on the 20th January 1977.[113] An exceptional rainfall of nearly 230 millimetres in 24 hours caused the failure of Euclides da Cunha dam, the failure of the latter causing the cascade failure of the Armando de Salles Oliveira dam downstream.

Work on the Armando de Salles Oliveira began in 1953, and the project was commissioned in September 1958. The 32 metre high earthfill embankment was under construction when a flood ruptured the upstream cofferdam and overtopped the embankment in September 1957, washing out 15000 cubic metres of earthfill.

The Armando scheme consisted of a homogeneous earthfill embankment with inclined and horizontal drainage filters through which passed the river diversion tunnel, ultimately used as an emergency outlet, a water-intake structure, a powerhouse, and finally a spillway structure with two taintor gates catering for a maximum discharge of 1800 cubic metres per second. In addition,

the 660 metre long crest carried a service road.

The Euclides da Cunha dam was commissioned in 1960 for a maximum flow of 2040 cubic metres per second through two taintor gates, a diversion and outlet tunnel on the left abutment with a discharge capacity of 300 cubic metres per second and a river intake structure connected to an underground power structure.

The dam is a homogeneous earthfill embankment 63 metres high with a curved axis, and a crest length of 312 metres. The rip-rapped upstream slope was 1:3.3 while the grassed downstream slope was, from toe to crest, 1:3.5, 1:2.8, 1:2.1 and 1:2.3 between berms.

Both projects failed by overtopping as reported by the operating personnel at the sites. Overtopping commenced at the Euclides da Cunha dam at 8.00 p.m. on the 19th January and continued until a breach started to form at 3.30 a.m. on the 20th January, some seven and a half hours later.

The flood wave released from Euclides da Cunha dam reached the Armando de Salles Oliveira dam 15-30 minutes later, overtopping the embankment and resulting in a breach and total failure.

Hydrologists estimated at the time the inflow to Euclides da Cunha as being 2400 cubic metres per second, i.e. the flood was very close to the spillway capacity of the Euclides da Cunha dam.

Evidence existed to show that Euclides da Cunha dam had been overtopped by at least 1.26 metres of water. The overtopping water eroded the downstream slope "quite severely". The erosion eroded the filter zone material 3-5 metres deeper than the surrounding

impervious clay. Aerial views of the breached embankment are shown in Plate 3.1 and Plate 3.2 while Plates 3.3 and 3.4 show the effects of erosion in greater detail.

High water marks at the Armando de Salles Oliveira dam indicate that the dam was overtopped by at least 1.3 metres. Failure occurred through breaching at a low saddle in the reservoir rim, about 200 metres from the end of the dam embankment, and at the right end of the embankment, although all the downstream slope was eroded before the breach formed, as shown by Plates 3.5 and 3.6.

Centrais Electricas de Sao Paulo stated at the time that "as a result of the January storm, plans for the revision of the assessment of the maximum flood likely at other impounding schemes are being developed by C.E.S.P., as well as the revising of reservoir operation rules and standing instructions to personnel at all dams."

Preliminary estimates indicated that reconstruction of Euclides da Cunha and Armando de Salles Oliveira projects will cost nearly U.S. $25 million.

The main features of the schemes are shown in Table 3.8.

3.8 Flow through and over Rockfill

The Author does not intend to review in detail the related research carried out on the problem of flow through and over rockfill. It is sufficient to say at this point that floods have been successfully passed over rockfill dams when the downstream face has been protected by a welded mesh of reinforcing bars anchored into the rockfill.[114,115,116,117,118,119,120,121]

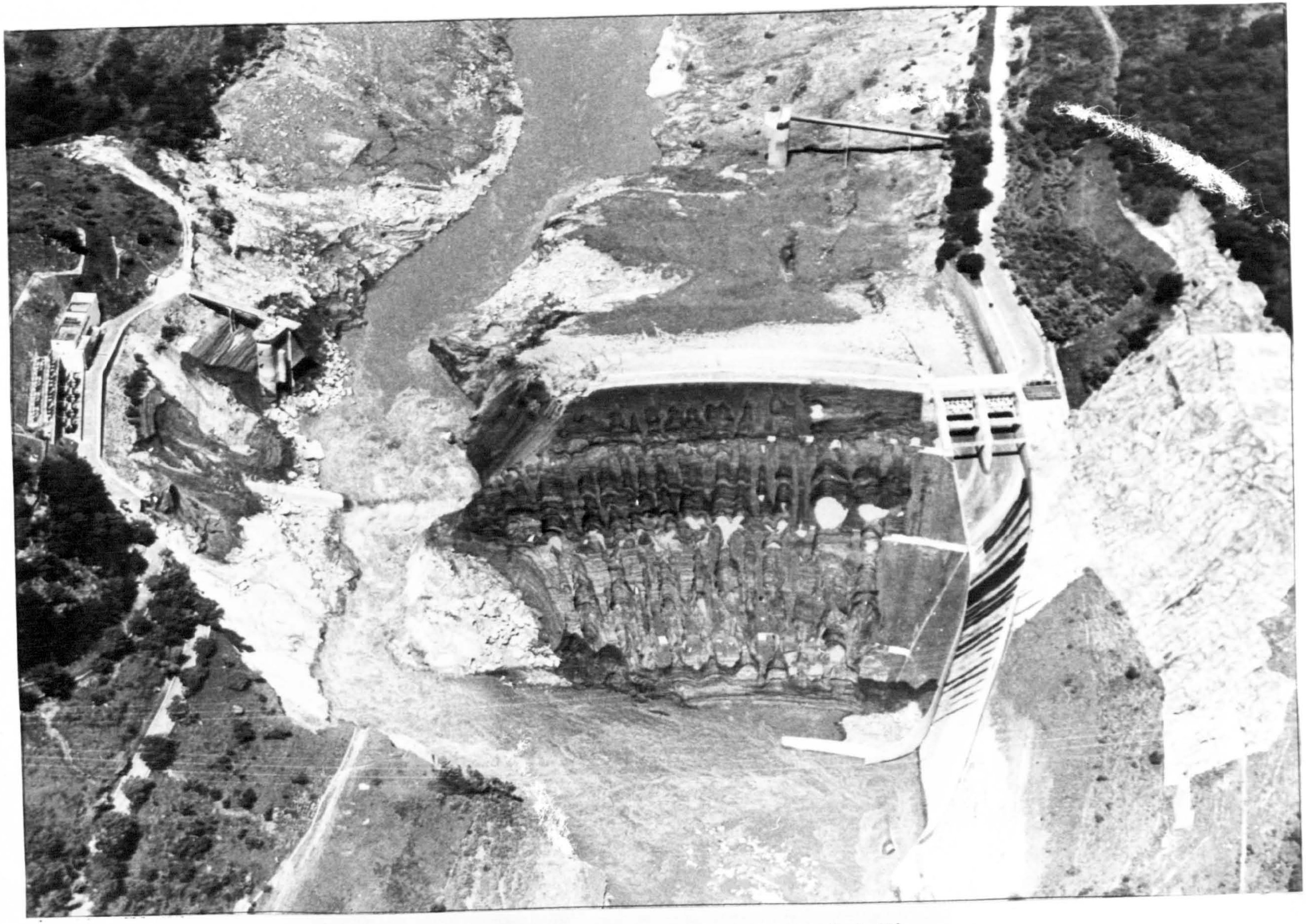


PLATE 3.1: Euclides da Cunha Project - Aerial View
(Courtesy of the Centrais Electricas De Sao Paulo - C.E.S.P.)

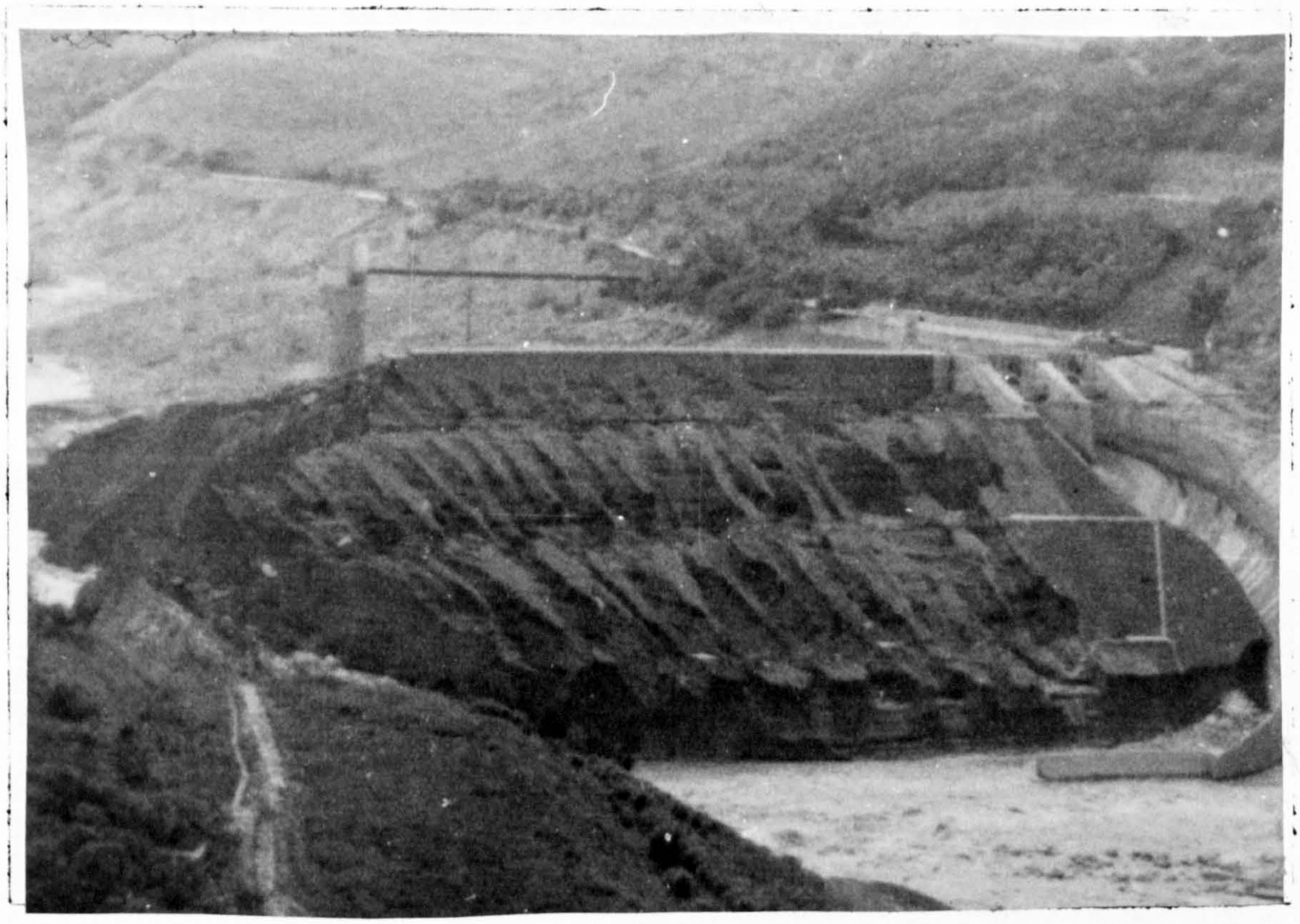


PLATE 3.2: Euclides da Cunha Project - Aerial View taken from downstream (Courtesy of the Centrais Electricas de Sao Paulo - C.E.S.P.)

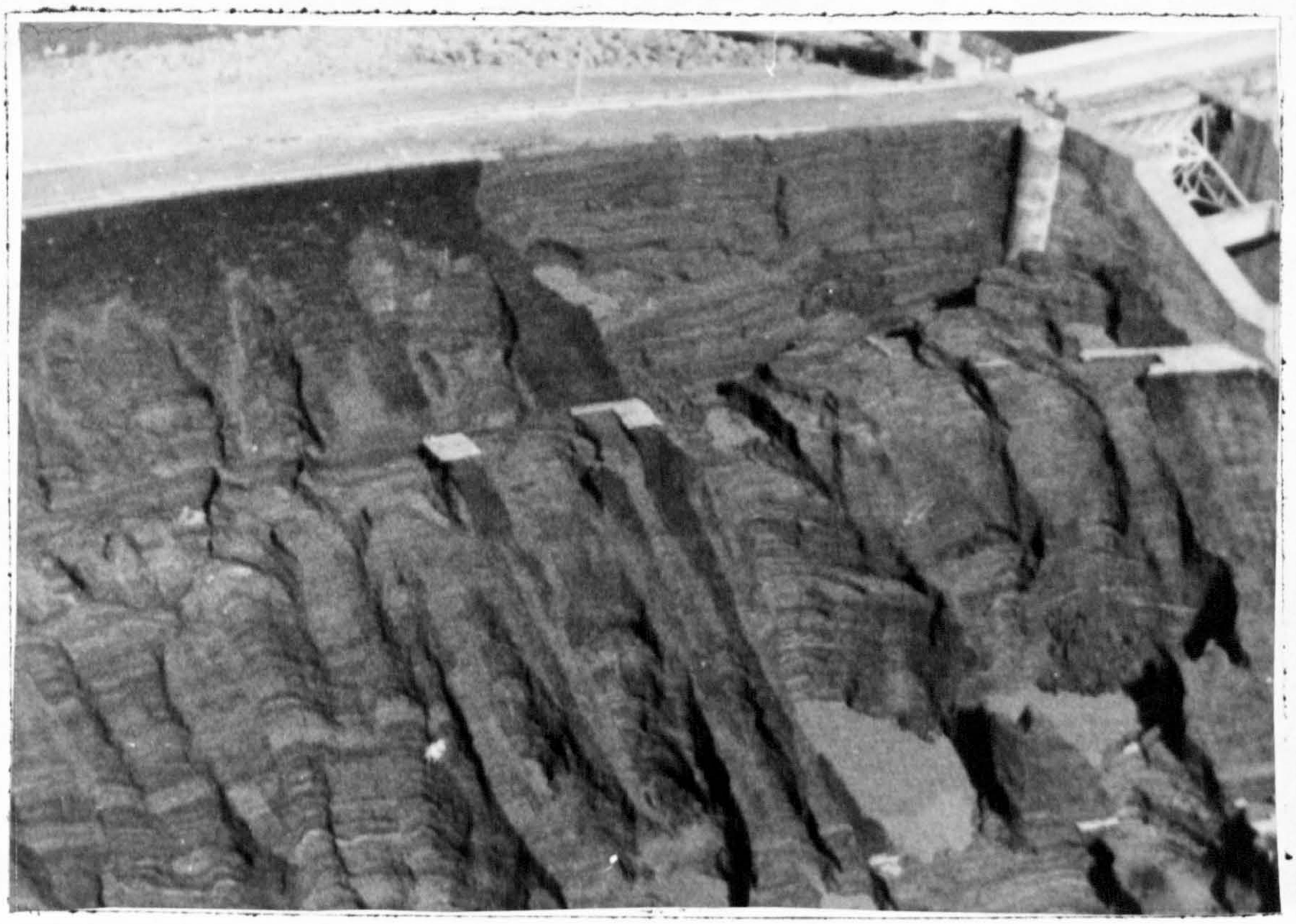


PLATE 3.3: Euclides da Cunha Project - Detail of the Erosion on the Downstream Slope (Courtesy of the Centrais Electricas De Sao Paulo - C.E.S.P.)



PLATE 3.4: Euclides da Cunha Project - Erosion caused by Overtopping on the Downstream Slope (Courtesy of the Centrais Electricas De Sao Paulo - C.E.S.P.)



PLATE 3.5: Armando de Salles Oliveira Project - Aerial View
(Courtesy of the Centrais Electricas De Sao Paulo - C.E.S.P.)



PLATE 3.6: Armando de Salles Oliveira Project - Erosion caused by the Overtopping on the Left Abutment (Courtesy of the Centrais Electricas De Sao Paulo - C.E.S.P.)

	Armando de Salles Oliveira Scheme	Euclides da Cunha Scheme
Pardo River		
Yearly Average Flow (m^3/s)	83.9	83.9
Catchment (km^2)	4053	4050
Overall Storage Capacity (m^3)	25.4×10^6	13.4×10^6
Useful Storage Capacity (m^3)	16.7×10^6	5.0×10^6
Crest Length (m)	660	312
Height (m)	63	32
Slopes - upstream	1:2.5	1:3.3
downstream	1:2	1:2.1
downstream (at toe)		1:1.5
Type	Homogeneous with inclined and vertical sand filters	Homogeneous with inclined and vertical sand filters
L.L.	10-80	22-75
P.L.	25-45	22-40
Optimum Moisture Content	18%	11-28%
Max. Dry Density (t/m^3)	1.19-1.71	1.47-1.91
Average Cohesion (kg/cm^2)	0.6	N.A.
Average Angle of Internal Friction	15°	N.A.
Natural Moisture Content	15-28%	
Freeboard (m)	2.5	2.5
Spillway Capacity (m^3/s)	1800	2040
Gates	2 Taintor	2 Taintor

TABLE 3.8: Main Features of the Armando de Salles Oliveira and Euclides da Cunha Schemes

This technique is not relevant to earth fill dams unless the downstream slope is covered with rip-rap.

3.9 Conclusions

It is apparent that much can be learned from a study of the failures of earth embankment dams which is relevant to design, maintenance and safe operation. Weaknesses in safety legislation may also be highlighted which may, in the long term, result in improved standards of reservoir safety.

The study of embankment failures has also provided additional evidence to suggest that compacted clay fills may be considerably more resistant to overtopping than is generally assumed in design. The following chapter will review work that has been carried out in connection with the erosion of clay soils while later chapters will describe the experimental and theoretical work of the Author. The latter work attempts to derive qualitative information on the mechanism of overtopping erosion and relationships between rate of erosion and key engineering parameters, including the physico-chemical nature of the clay, slope geometry, velocity of flow and other related factors.

SECTION II

CHAPTER FOUR

EROSION OF CLAYS - A LITERATURE REVIEW

4.1 Introduction

It has long been observed in the field that some soils are very susceptible to erosion while other soils, occurring under similar conditions of climate, topography and supporting similar vegetation, are much more resistant to erosion. The problem of erosion and transportation of material, mainly cohesionless coarse material, has been studied by many investigators. The present chapter reviews the research carried out by those investigators who have studied the erosion of cohesive soils.

4.2 Erosion of Cohesive Soils

Much of the research carried out in recent years has been with particular reference to the stability of unlined open channels, maintenance of minimum depths in estuaries and the life of reservoirs, and many empirical and semi-empirical relationships have been derived for coarse cohesionless material. The design methods are largely based on the equilibrium of forces acting on individual grains.

None of the relationships are valid for cohesive soils and, in fact, for cohesive soils very little published information exists

although in recent years some laboratory studies on erosion and deposition have been made in many instances; however, the results are far from conclusive and results from the different investigators are often conflicting. The principal difficulty lies in the fact that some of the resisting forces in cohesive soils are of an entirely different nature to those for cohesionless material, and furthermore they are not constant.

Since the basis of the resisting forces in cohesive material is mainly electrochemical in nature, it is felt that a brief summary of the nature of these forces and of the fundamental physico-chemical and mechanical properties of clays should be presented. Only those properties which seem to be related to erosion of cohesive materials are summarised. Such relevant properties essentially include the interparticle attractive and repulsive forces, the components of shear strength and the mechanism of flocculation.

4.3 Properties of Cohesive Material

Clay particles are of colloid size, having large ratios of surface area to volume and are generally plate-like in shape. The specific surface for some common clay minerals and a sand for comparison.are tabulated in Table 4.1.

Clearly, mineral composition is a factor which will affect the engineering behaviour of clay.

From the engineering standpoint, a cohesive soil is a soil containing a sufficient proportion of fines and colloids to make it plastic within a certain range of water content, and to be able to

Clay Mineral	Specific Surface (m^2/g)
Montmorillonite	200-600
Illite	50-100
Kaolinite	5-20
Sand-fine	2×10^{-2}

TABLE 4.1: Values of Specific Surface for Some Common Clay Minerals

resist a certain finite magnitude of shear-stress without confinement within a further range of water content. This last property is commonly known as cohesion.

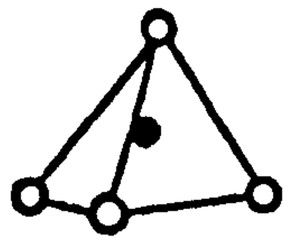
The colloids are responsible for the plastic and cohesive properties of clays, i.e. particles whose specific surface (area per unit weight) is so high that their behaviour is controlled by surface rather than gravity forces.[122,123,124]

4.3.1 Common Clay Minerals

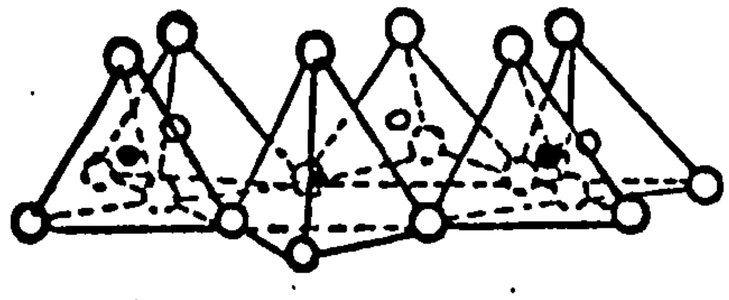
Clay particles are composed essentially of one or more clay minerals. Chemically the clay minerals are silicates of aluminium and/or iron and magnesium. There are two fundamental building units for clay structures. One is a silica tetrahedral unit composed of four hydrogen or hydroxyls having the configuration of a tetrahedron enclosing a silicon atom. The second unit is an octahedral unit of an aluminium, iron or magnesium atom enclosed in six hydroxyls having the configuration of an octahedron. The mineralogical origin of a clay will specify the relative degree of activity, i.e. whether the clay is a two- or three-layer clay and whether the clay lattice is expandable. The most common clays are two- or three-layer clays. These are bonded units of silica tetrahedra, T, and alumina octahedra, O, sheets as shown in Figure 4.1.

The most inert clay is the two-layer, OT, bonded by hydrogen bonds with strong attraction between the O and T layers and with little isomorphous substitution within the lattice structure. These minerals are called kaolinite.[125]

FIGURE 4.1: Sheet Silicate Minerals



Silica Tetrahedral Unit

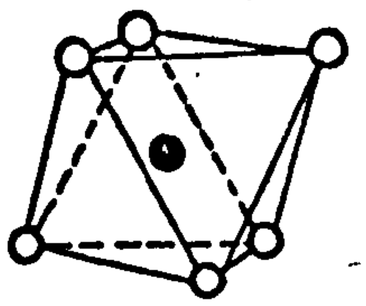


Tetrahedral Sheet

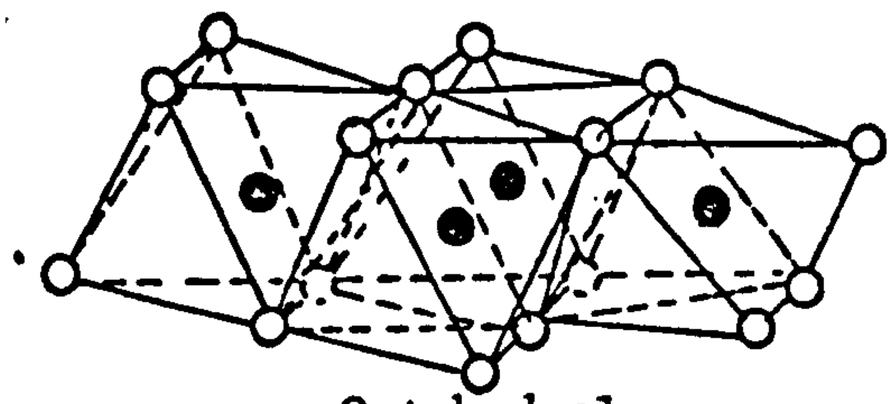


Oxygen

Silicons



Octahedral Unit



Octahedral Sheet

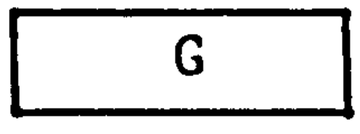
Hydroxyls

Aluminiums, Magnesiums etc.

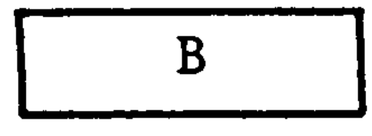
Combining silica tetrahedra = silica sheet - symbolic structure =

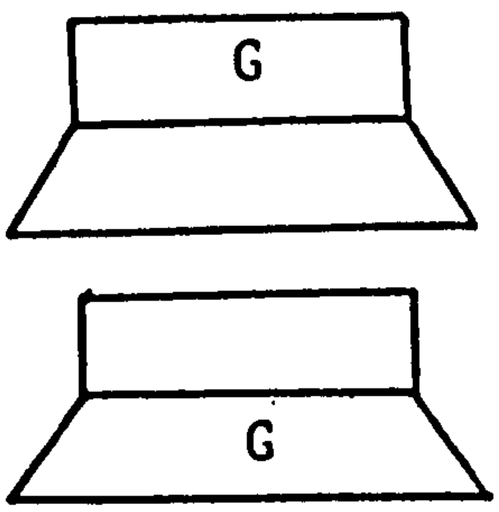


Combining aluminium-oxygen octahedrons = gibbsite

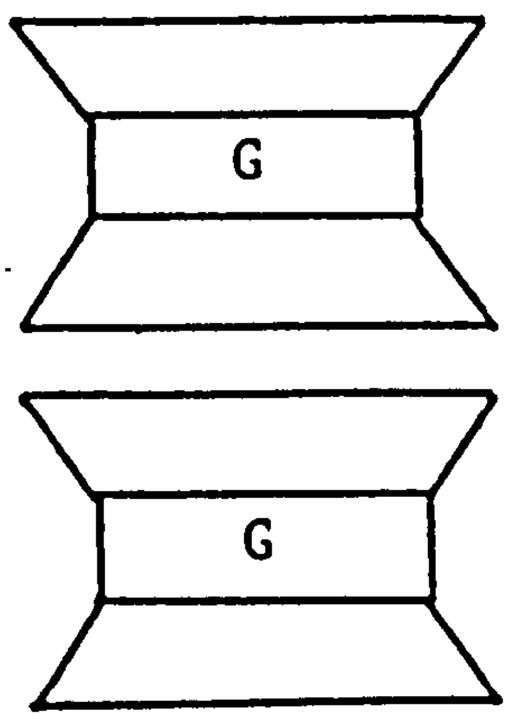


Combining magnesium-oxygen octahedrons = brucite

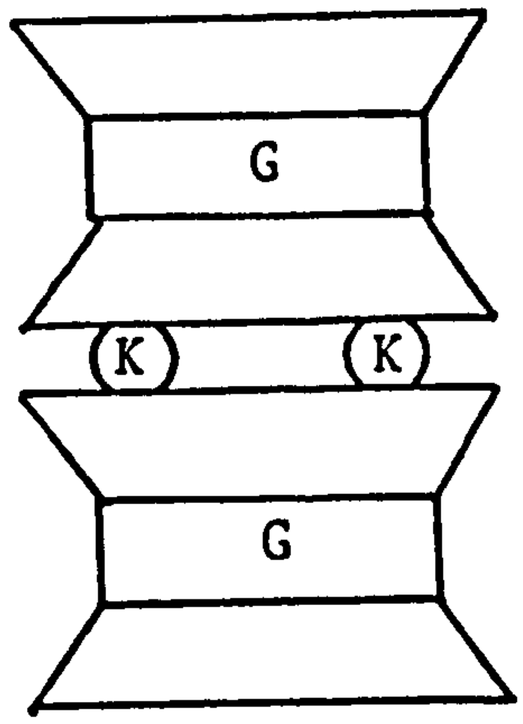




Kaolinite



Montmorrilonite



Illite

Kaolinite ($Al_2 Si_2 O_5 (OH)_4$)

There is substantially no substitution of cation for cation within the kaolinite structure, so that the mineral has the fixed formula $Al_2 Si_2 O_5 (OH)_4$, and the mineral is relatively inert. It shows little or no ability to absorb water within the lattice absorption which might cause swelling. The thickness of the unit is about 7 Å, and kaolinite particles are often one to ten microns in diameter.

Three-layer systems, TOT, exhibit extensive surface activity due to large-scale isomorphous substitution, resulting in weak bonding between the layers, which permits entry of water and hence large volume changes. An example of this type of system is the mineral montmorillonite.

Montmorillonite

The mineral is composed of two silica tetrahedral and one octahedral sheet. The thickness of the sheet is about 9.5 Å, but the dimensions in the other two directions are indefinite. The sheets are stacked one above the other like the leaves of a book with the oxygen layers facing each other. There is therefore very little bonding between successive sheets and water may enter causing the mineral to swell.

The octahedral elements may be aluminium, iron or magnesium, or a combination of these elements. The mineral is electrochemically unstable and as a result replacements occur within the structure, always producing a net positive charge deficiency. This deficiency is balanced by adsorbed cations which are held on the outside of the

sheets, and which in general are readily exchangeable. It is therefore very difficult to write a general chemical formula for the montmorillonite minerals. Montmorillonite is the most active clay mineral.

Illite

The structure of illite is similar to that of montmorillonite but with a greatly reduced likelihood of swelling. This is due to the replacement of silicon ions by aluminium ions in the tetrahedral layers and potassium ions between the layers serving to balance the charges resulting from the replacement and tie the sheet units together.

4.3.2 Isomorphous Substitution

Isomorphous substitution of lower valence ions for higher valence ions results in charge deficiencies at the surface of the clay particle resulting in a net negative charge at the surface. The magnitude of the charge depends on degree and type of ionic substitution. Particles of colloidal size with highly charged surfaces respond to the laws of electrical attraction and repulsion to an equal or greater degree than they do to ordinary Coulombic attraction between layers. The greater the percentage by weight of clay in a given soil is, the greater will be the overall effect of the surface behaviour.

4.3.3 Plasticity Index and Activity

An index commonly used to classify a clay soil is the Plasticity Index (P.I.) and is defined as the difference between the Liquid Limit (L.L.) and the Plastic Limit (P.L.) of the soil. The Liquid Limit may be defined as the moisture content at which the soil stops acting

as a liquid and starts to act as a plastic solid, while the Plastic Limit may be defined as the moisture content at which a soil will exhibit no permanent deformation under shearing stresses but simply fractures with no plastic deformation, i.e. it acts as a brittle solid. Figure 4.2 illustrates the relationships between the consistency limits.

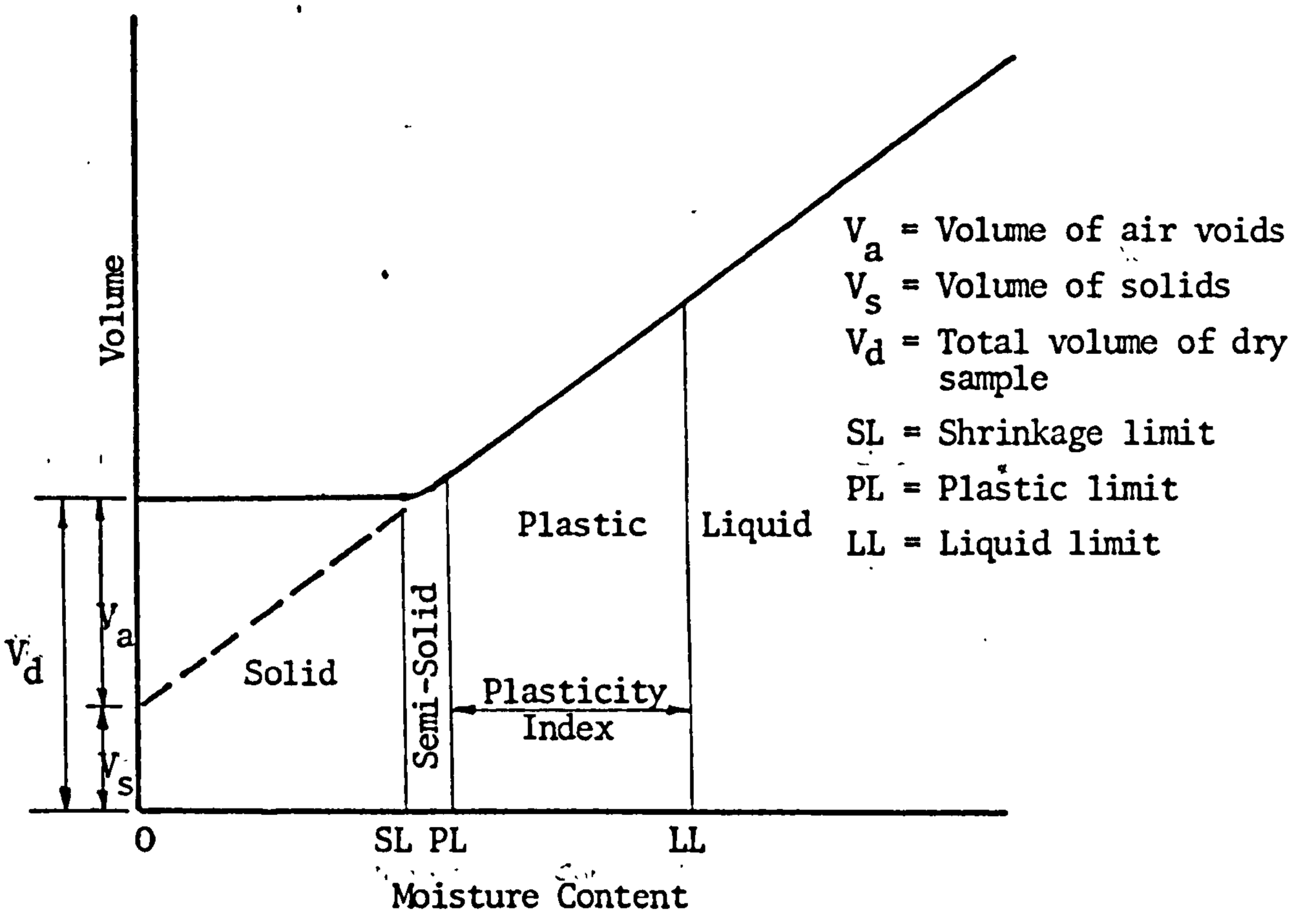


FIGURE 4.2: Consistency Limits

The Liquid Limit of the soil mainly depends on three factors:[126]

1. The amount and nature of the clay minerals;

2. The degree of electro-chemical saturation;

3. The polarizability of the absorbed ions.

As the plastic properties of a soil increase, the soil will deform rather than fracture under shear stresses. The increase in the ability

of a soil to absorb stress through deformation at specific values of shear stress without brittle failure is due to (a) an increase in the percentage of clay mineral (by weight) of the total soil fraction, and (b) the variance in the type of clay mineral. Both factors indicate that the strength of a soil becomes less dependent on the structure of the interparticle contacts and more dependent on the intercolloidal force structure, as a function of such factors as surface charge density, arrangement of particles and electrolyte concentration.

The term 'activity' serves as an aid to classifying soil types and gives more understanding to the significance of the concept of plasticity index:

$$\text{Activity} = \frac{\text{(change in) Plasticity Index}}{\text{(change in) percentage by weight of clay fraction}}$$ [127]

Seed et al[6] found this to be a 'reasonably reliable index of swelling co-efficient', and 'activity will accurately classify the soils with regard to their Liquid Limit versus clay content relationships, or with regard to their swelling potential regardless of the clay mineral composition of the clay fraction'.

4.3.4 Aggregation and Dispersion of Soil Particles

4.3.4.1 Electrical Forces

The electrical forces acting between atoms fall into three categories: primary valence bonds, hydrogen bonds and secondary valence bonds.

Primary valence bonds, which are the strongest, hold atoms together in the basic mineral units. The three most important bonds

are the ionic, covalent and the heterpolar bonds.

The hydrogen bond is responsible for the linkage between water molecules but is considerably weaker than primary valence bonds. It operates not only in water but is the main linkage between the basic units of kaolinite and other clay minerals.

Unlike primary valence bonds secondary valence forces are effective over relatively large distances. The secondary valence forces, also known as Van der Waals forces, arise from electrical moments existing within the units. The net effect of secondary valence forces is attractive.

Because of the non-symmetrical distribution of electrons in the silicate crystals (arising from the heterpolar bonds), these crystals act as a large number of dipoles. They can attract other dipoles, the most important being water molecules, the non-symmetrical configuration of which renders them dipolar.

The secondary valence forces are important since they contribute to clay strength and cause soils to hold water.

Another important primary valence linkage is the electrostatic attraction or repulsion of electrically charged units of matter. The negatively charged soil particle attracts cations; cations and anions attract each other; cations repel each other; and anions repel each other. These electrostatic forces are similar to primary valence bonds in that they involve units with net electrical charges.

In summary, the internal electrochemical forces that can act within the clay structure include:

(a) Attractive Forces: The most important ones are:

(i) Van der Waals-London forces, or secondary valence forces: They are electrochemical-attractive forces and act between all units of matter. They result from the mutual influence of electronic motion between the atoms which generate electric moments within the units. The magnitude of Van der Waals forces depends on the mutural distance between the mineral particles and on adsorbed cations.

(ii) Hydrogen bond: It occurs when an atom of hydrogen is rather strongly attracted by two other atoms.

(iii) Cation bonds: Exerted by cations attracting and trying to neutralise negatively charged particles.

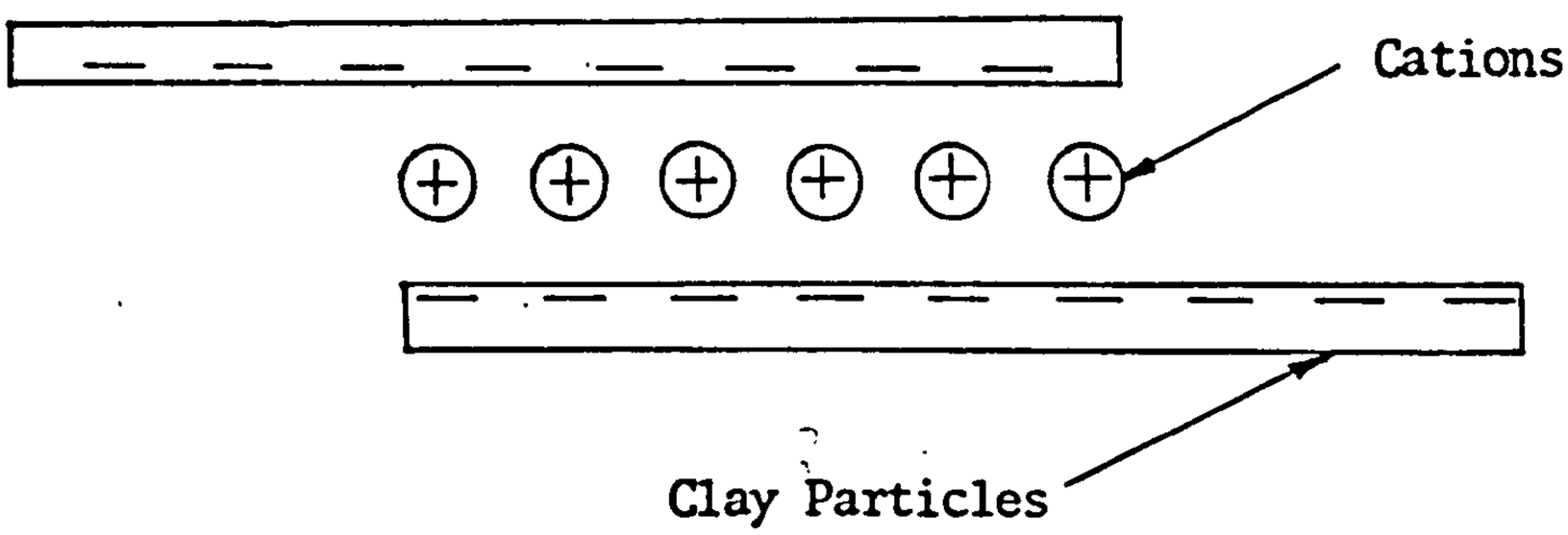


FIGURE 4.3: Cation Bond

(iv) Electrostatic attractions between negatively charged particle surfaces and positively charged edges.

(v) Chemical cementation between particles by various compounds.

(vi) Dipole-cation-dipole arrangement.

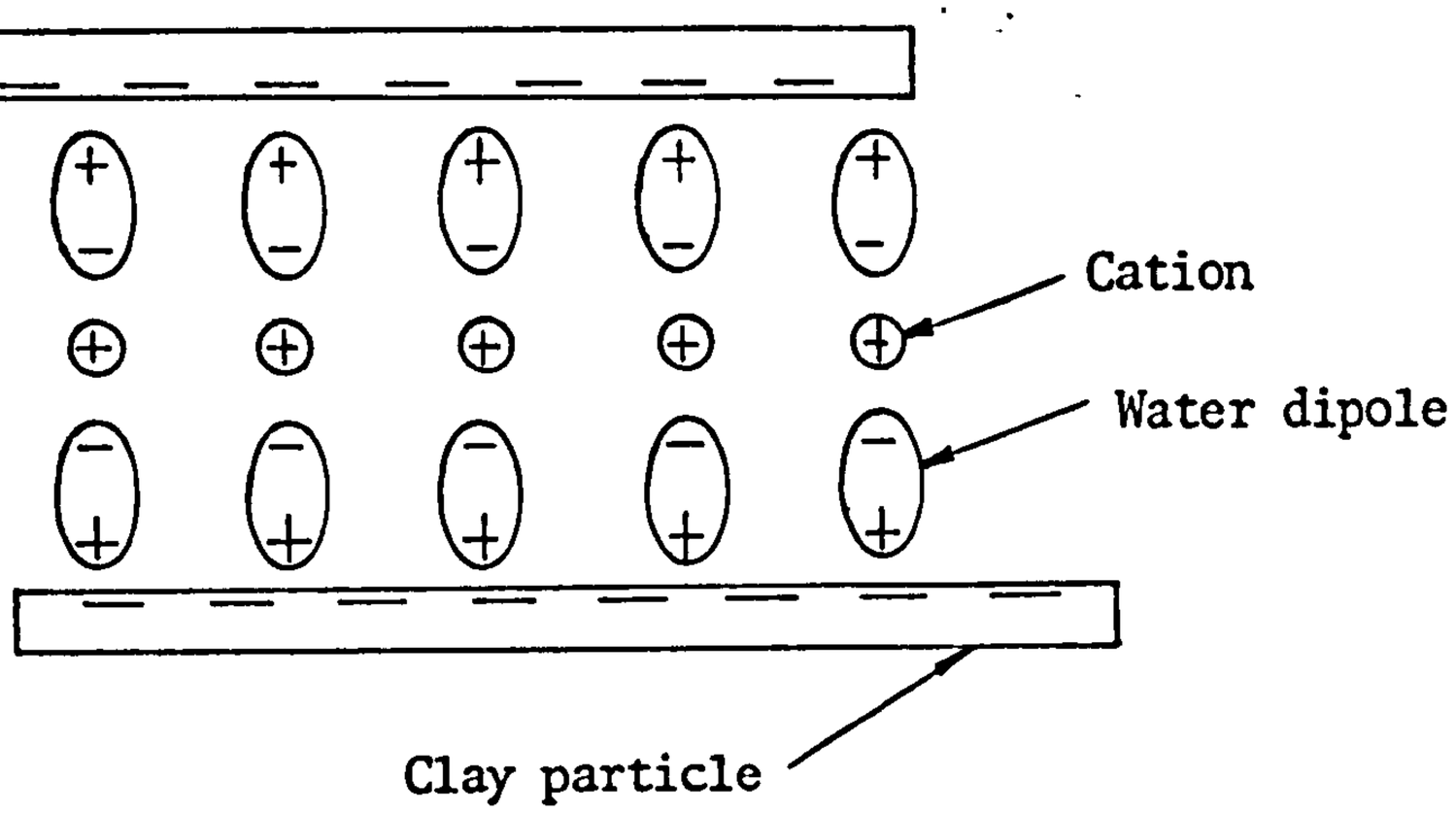


FIGURE 4.4: Dipole-Cation-Dipole Arrangement

(vii) Water dipole linkage.

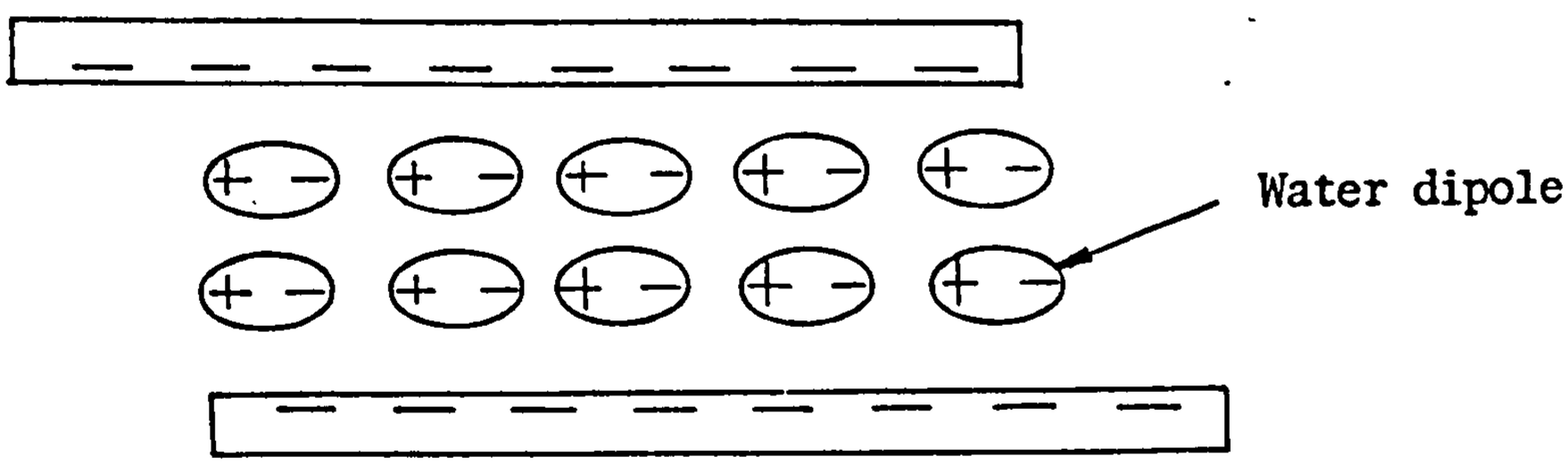


FIGURE 4.5: Water Dipole Linkage

(b) Repulsive forces: They include:

(i) Particle charges which being all negative repel each other.

(ii) Cation repulsion.

4.3.5 Interparticle Forces in Clay Soils and Clay Structures

A cohesive soil is distinctively a soil in which the behaviour of the clay particles is controlled by the influence of surface rather than gravitational forces, electrochemical inter-particle forces controlling the general behaviour of a clay. There are several types of such forces, both attractive and repulsive, all of which contribute to some extent to the inter-particle bonds.

The individual clay particles possess an electro-negativity resulting from isomorphic substitution of positively charged ions of lower valence within the lattice and from the structure of the minerals which concentrates negative oxygen and hydroxyl ions on the particle surface, and it has also been shown that the edges of the clay particles carry positive charges due to adsorption of protons.[128]

In a dry clay, cations are lightly held by the negatively charged clay surfaces. Cations in excess of those needed to neutralise the electro-negativity of the clay particles and their associated anions are present as salt precipitates. When water becomes available to the soil structure the precipitated salts go into solution. Because the adsorbed cations are present in much higher concentrations near the surface of particles, there is a tendency for them to diffuse away in order to equalise concentrations throughout. Their freedom to do so is, however, restricted by the negative electric field originating on the particle surface. The tendency to diffuse and the opposing electrostatic

attraction lead to a cation distribution adjacent to a clay particle in suspension.

The negative surface and the distributed charge in the adjacent phase are termed together the diffuse double layer. Several theories have been proposed for the description of ion distributions adjacent to charged surfaces in colloids and are shown schematically in Figure 4.6.

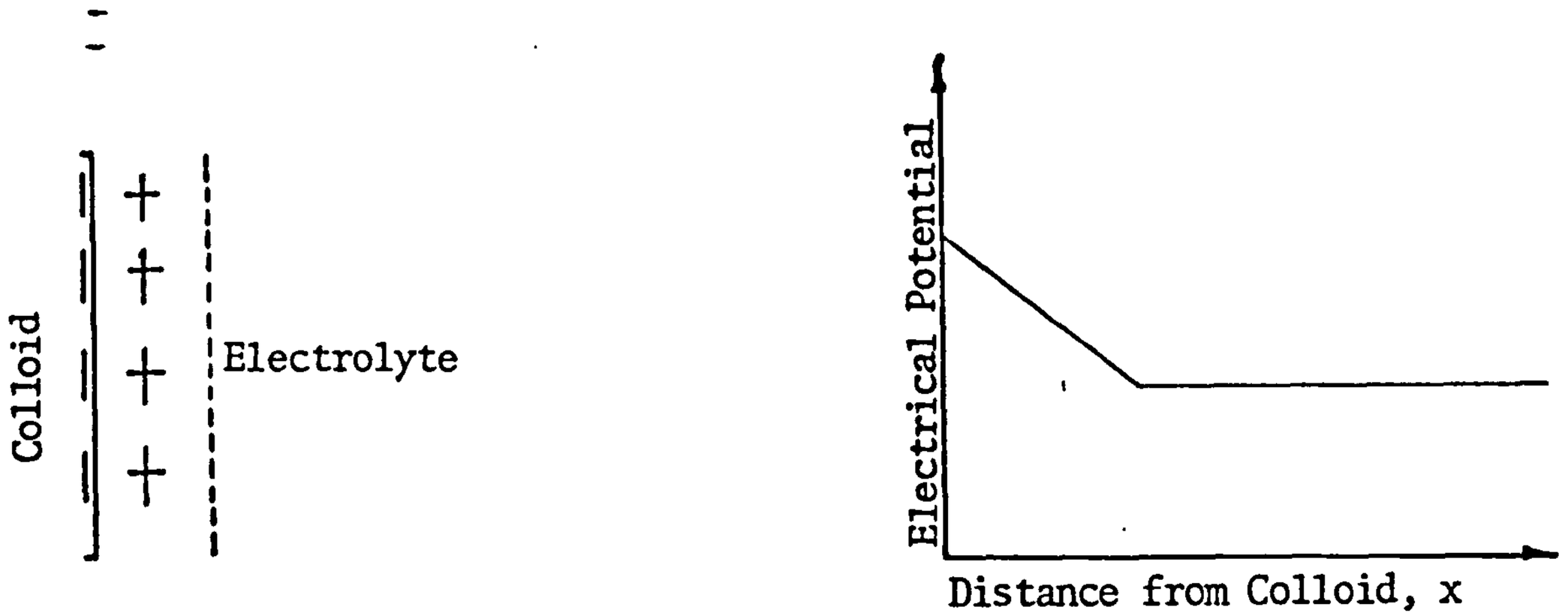
The Guoy Chapman theory of the diffuse double layer has received the greatest attention, and it has been applied to studies of the behaviour of clays.

4.3.6 Factors Influencing the Behaviour of the Double Layer System

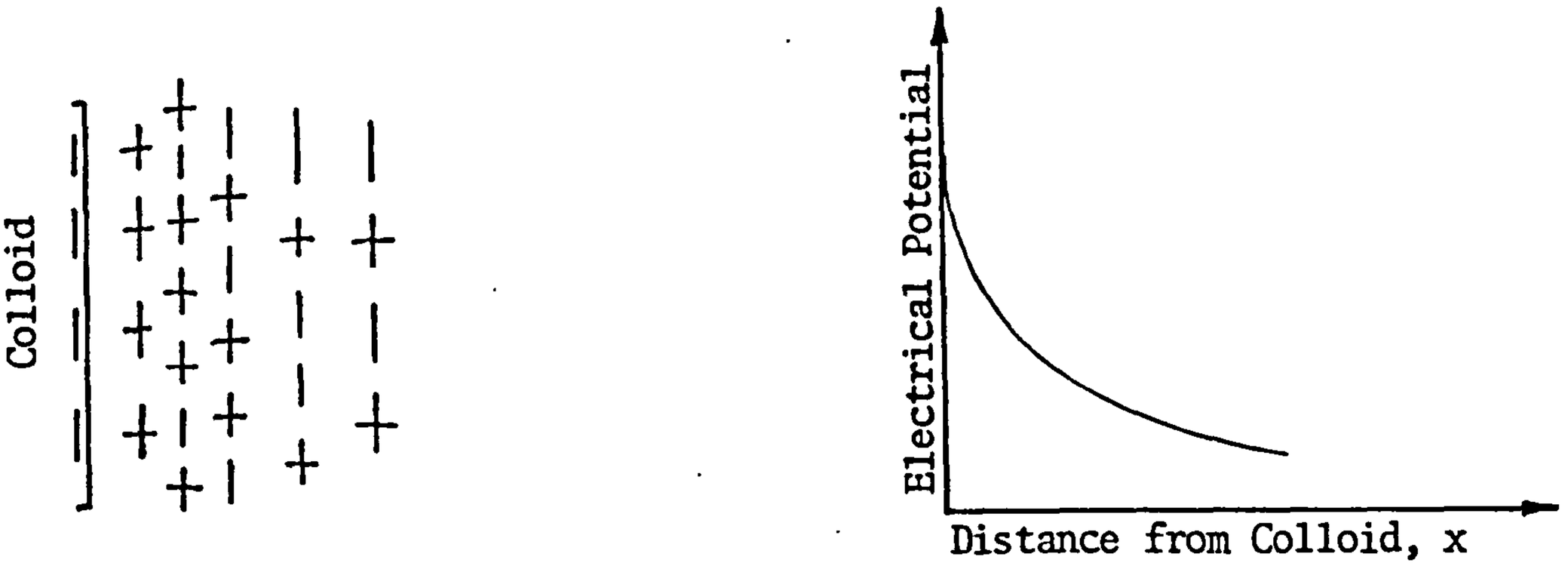
Among the variables in a soil water system which can affect the colloidal stability are:

1. Electrolyte concentration
2. Ion valence
3. Dielectric constant
4. Temperature
5. Size of hydrated ion
6. pH
7. Anion adsorption

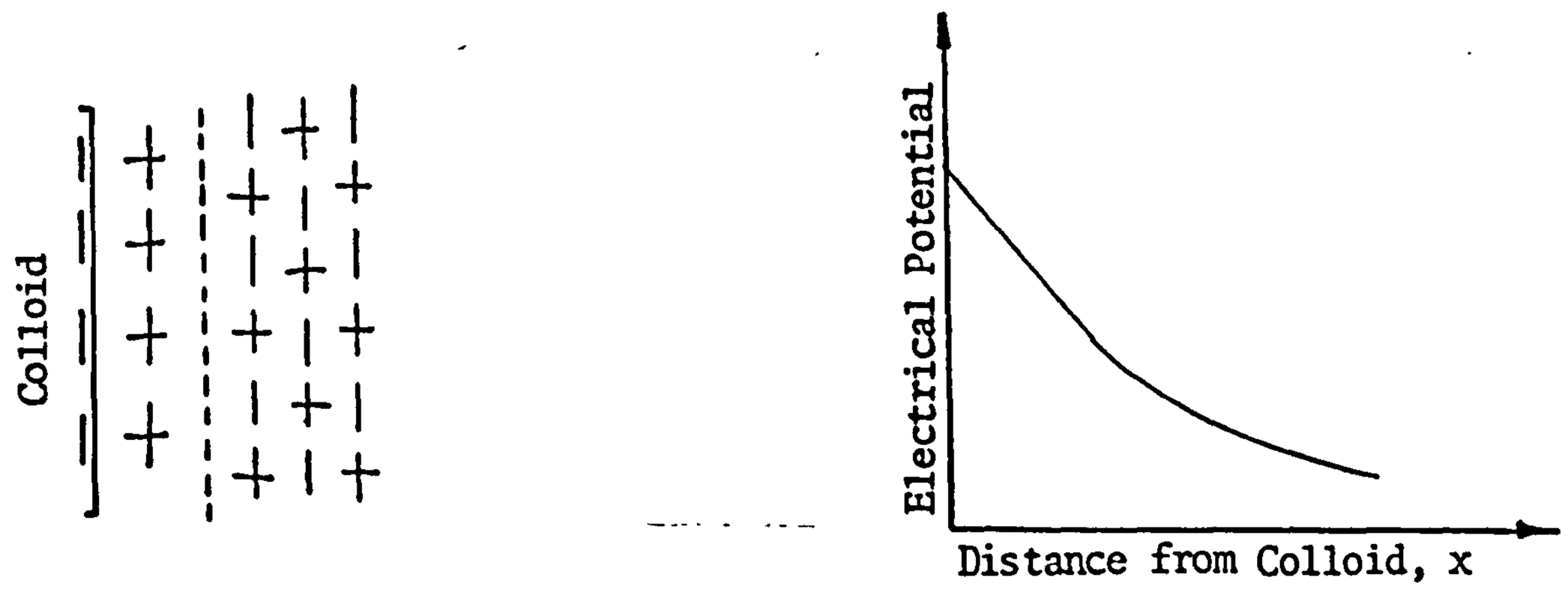
Equations developed by Bolt[129] indicate that, if the potential energy of a clay system is reduced when adjacent particles approach each other, they will so approach and "flocculate", i.e. form aggregates. If the energy of the system increases when the particles approach each other, they will move apart or 'disperse'.



1. Helmholtz-Quirk (1861)



2. Guoy-Chapman (1913)



3. Stern (1924)

FIGURE 4.6: Diffuse Double Layer Theories

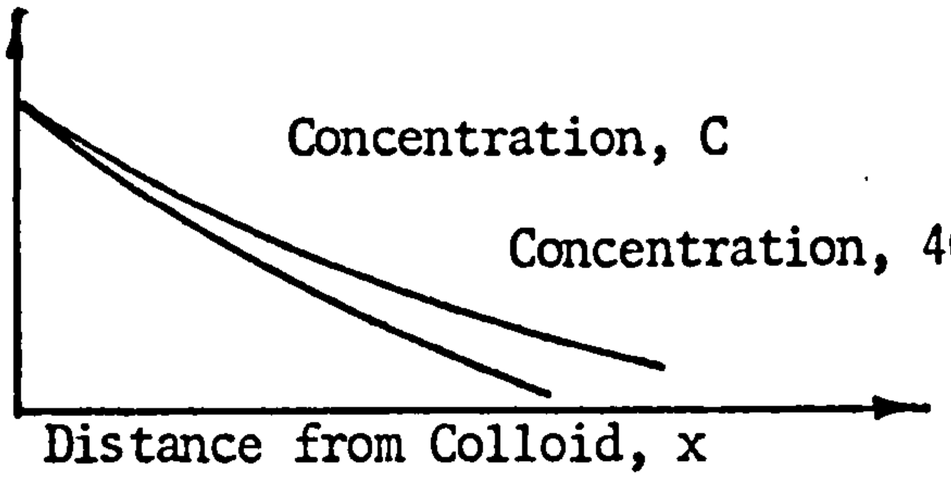
The Guoy-Chapman theory indicates that, in general terms, a tendency towards flocculation is usually caused by:

Increasing:	electrolyte concentration
	ion valence
	temperature
Decreasing:	dielectric constant
	size of hydrated ion
	pH
	anion adsorption

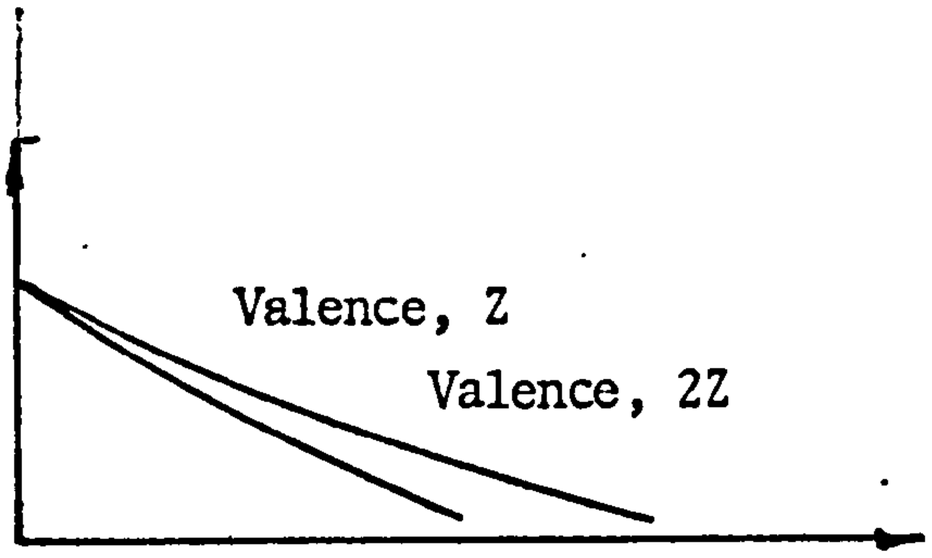
Obviously the variables which cause a decrease in the double layer thickness and reduce the electrical repulsion promote a tendency to flocculate. The effects of changes in system properties on the diffuse double layer are shown in Figure 4.7. It is clear that the smaller an ion plus its 'shell' of hydration water, the closer it can approach the colloidal surface; thus the smaller the hydrated ion, the smaller the double layer and the more likely flocculation.

The pH of the pore fluid affects the negative charge on a soil particle by altering the extent of dissociation of OH groups on the edges of the particle. High pH encourages the dissociation and increases the net charge, thus expanding the double layer and tending to cause dispersion; low pH does the reverse, and tends to cause flocculation.

Adsorption of anions increases the charge on the particles and hence tends to cause dispersion.

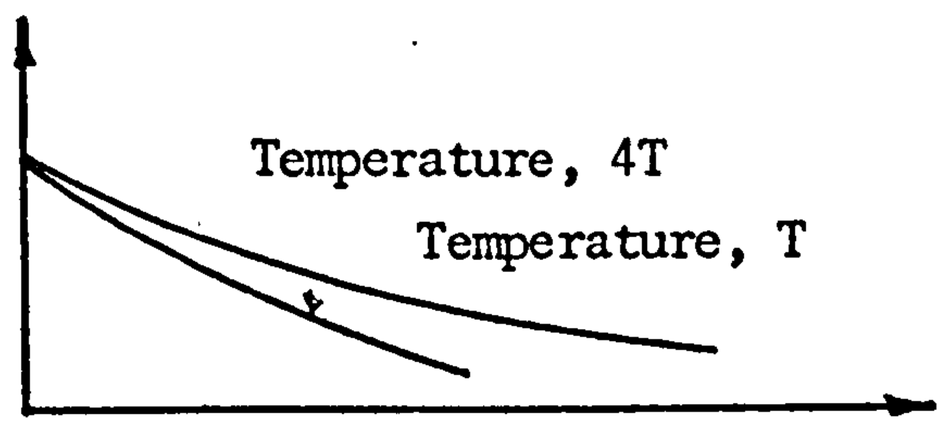


(a) Electrolyte Concentration only variable

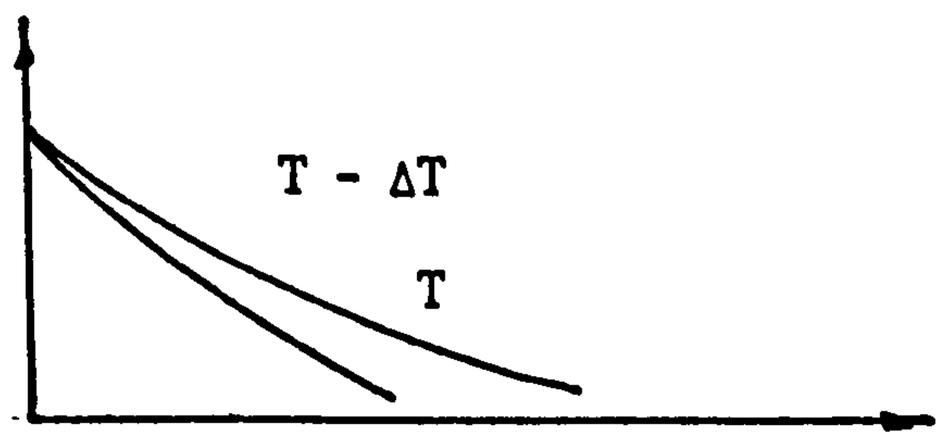


(b) Valence only variable

(c) Dielectric Constant only variable



(d) Temperature only variable



(e) Hydrosol-considerin Effect of Temperature per se and of T on Dielectric Constant

FIGURE 4.7: The Effects of Changes in System Properties on Diffuse Double Layer

4.4 Non-Salt Flocculation

The Gouy-Chapman theory assumes that the colloid charge is spread uniformly over the entire colloid surface. This implies that the edges of the clay particles, which may be described as platelets, carry the same charge as the flat faces. Several researchers[130,131] have shown that the edges of clay particles can be oppostively charged from the faces.

Two types of flocculation exist, salt type with an orientation approaching parallelism, and non-salt, with orientation tending to a perpendicular array. The appropriate sediment structures are shown in Figure 4.8.

Adding a small amount of electrolyte to kaolinite, for instance, will result in dispersion; further salt results in salt flocculation as shown in Figure 4.9.

4.5 Nature of Water in Clay

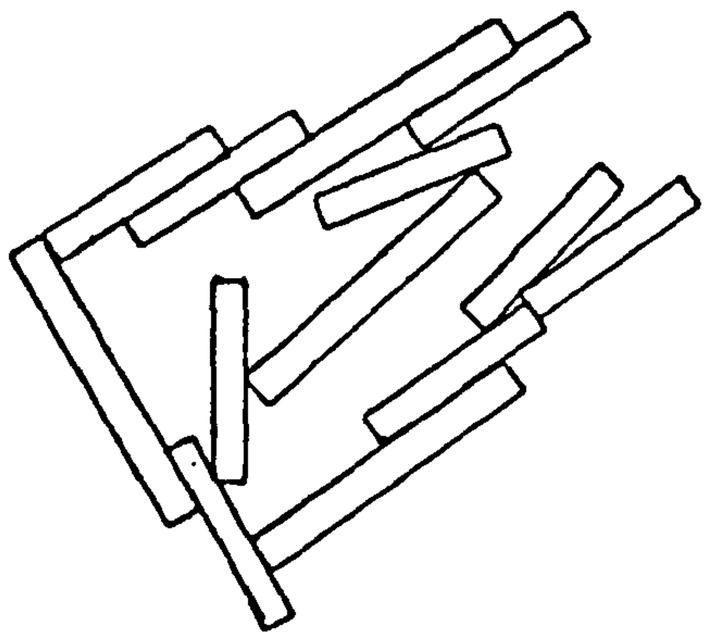
The influence of water on the behaviour of clay is very marked The water phase of a clay mass can be arbitrarily divided into three zones:[132]

(i) Adsorbed water held very strongly by the clay particles;

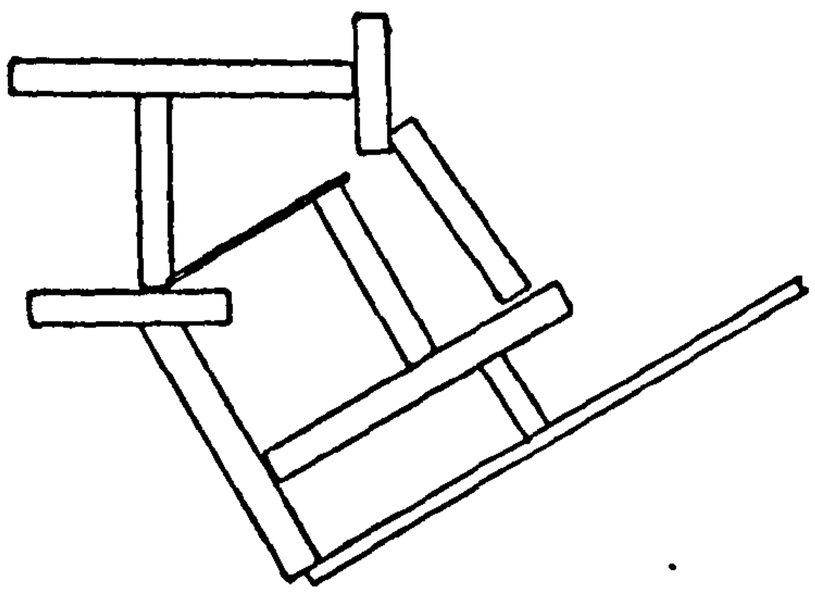
(ii) Water with a high ion concentration held mostly by the cation positive charge called "double layer water";

(iii) Free pore water subject to the laws of hydrodynamics.

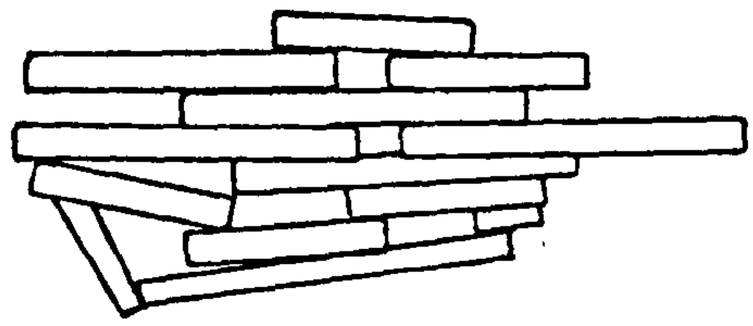
The adsorbed water is that which is strongly held by the soil; the double layer water is all the water attracted to the soil; and free water is water which is not subject to force fields and therefore not



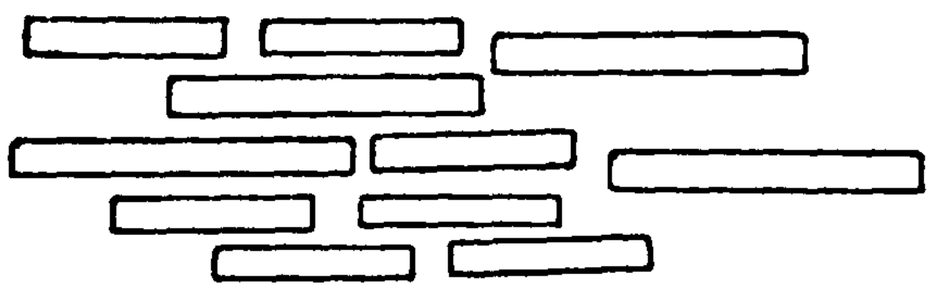
Salt Flocculation



Non Salt Flocculation



Dispersion



Remoulded

FIGURE 4.8: Clay Structure

held to the soil at all. Adsorbed water is, therefore, the innermost part of the double layer water. While this classification of water is accepted by some, many others use 'adsorbed' to describe all of the double layer water. Tne nature of the adsorbed water continues to be a very controversial subject.

4.6 Compaction

The effects of compaction on structures is shown in Figure 4.10. At A there is not sufficient water for the diffuse double layers of the soil colloids to fully develop. The small amount of water present at A gives a very high concentration of electrolyte which depresses the double layer. The low density and random particle orientation associated with the double layer depression is shown in Figure 4.10.

Increasing the moisture content from W_A to W_B expands the double layer around the soil particles and also reduces the electrolyte concentration. The reduced degree of flocculation permits an arrangement approaching parallelism, and thus a higher density. A further increase from W_B to W_C results in a further expansion of the double layer and a continued reduction in the net attractive force between particles. At this stage, however, even though a more orderly arrangement of particles exists than at the W_B stage, the density at C is lower because the added water has diluted the concentration of soil particles per unit volume.

Figure 4.10 also illustrates the change in soil structure due to changes in compactive effort. The greater the input of work, the more nearly parallel are the clay particles and, usually, the closer together they are. At high moulding moisture contents increased

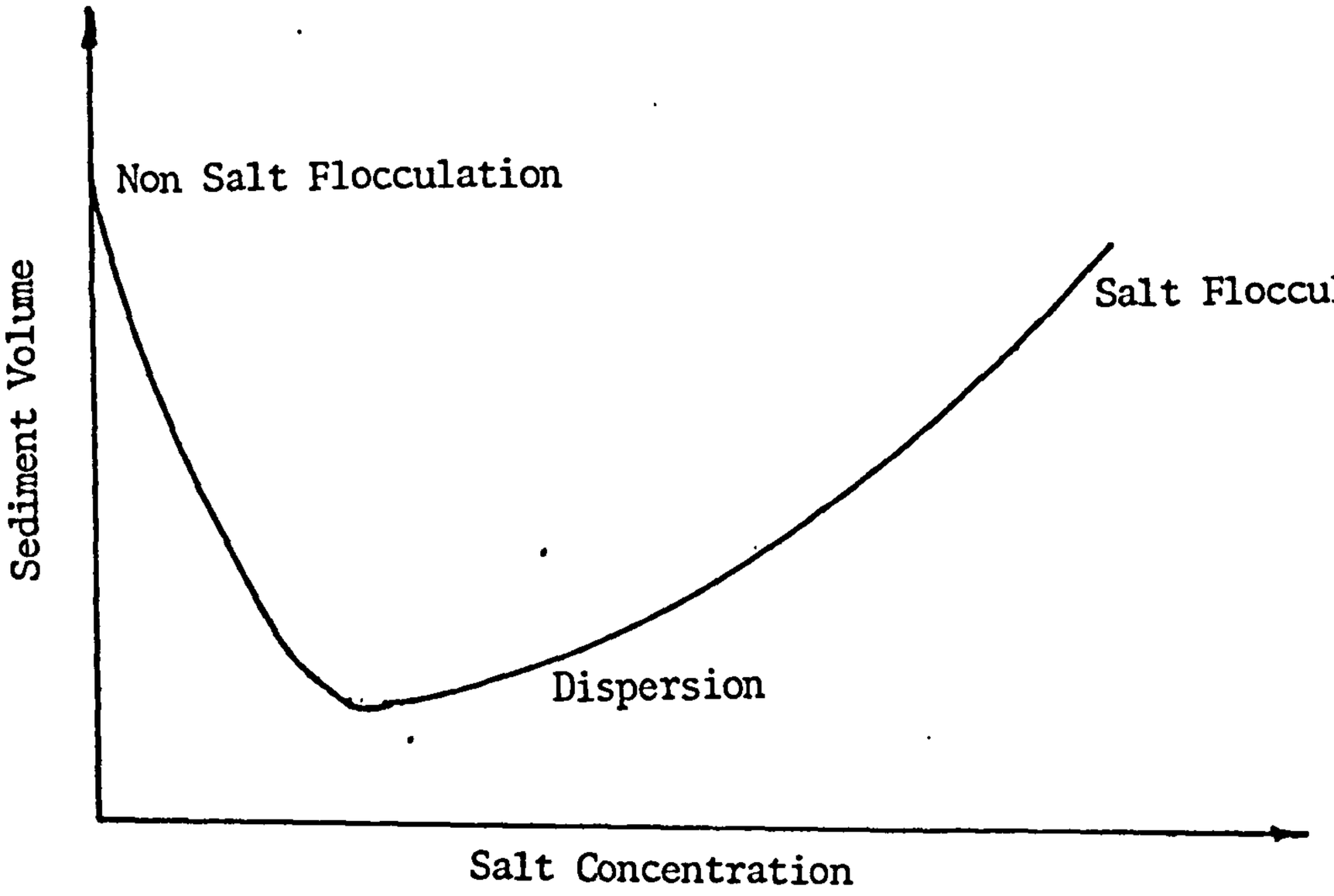


FIGURE 4.9: Flocculation and Dispersion of Kaolinite in Water

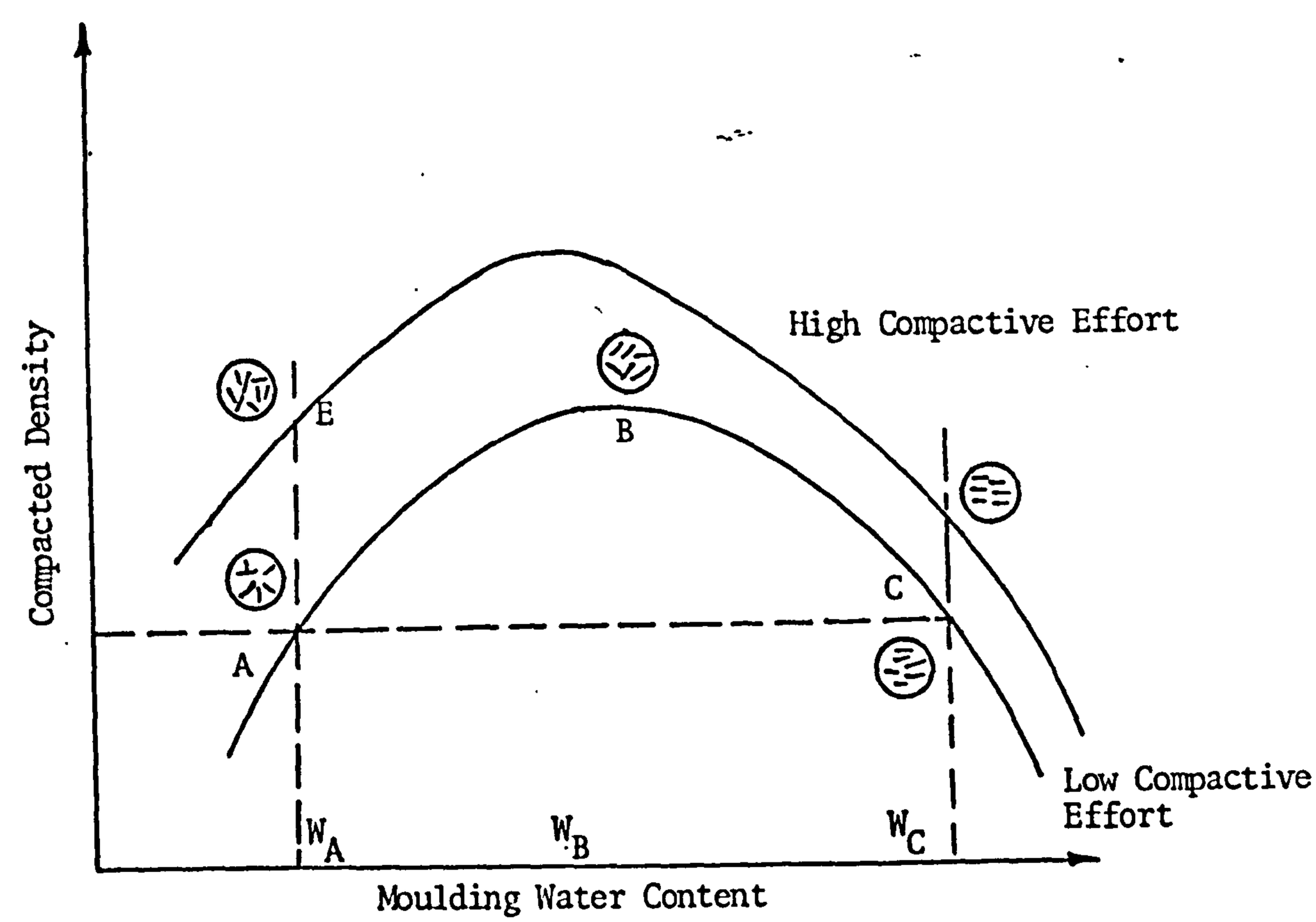


FIGURE 4.10: Effect of Compaction on Structure

compactive effort may merely align particles without significantly altering the particle spacing.

4.7 Shear Strength of Compacted Clay

4.7.1 Forces between Particles

For two adjacent colloidal particles there exists a force system of four horizontal forces between the colloids; these are:

1. $\bar{\sigma}$ = the externally applied intergranular stress
2. A = the electrical attraction
3. R = the electrical repulsion
4. I = the contact pressure.

The attractive potential is basically the Van der Waal forces and is dependent on the volume of the attracting mass, its geometry and the distance of separation of the clay plates and compared to the repulsive forces can be assumed constant. The net force acting between the particles is thus more strongly dependent on the relative magnitude of the repulsive force (R), since $\bar{\sigma}$ is the effective stress and is derived from some source other than electrical stresses originating within the particle and the contact pressure, I, can arise between mineral surfaces or between molecules of tightly held water and is electrical in nature.

It can be said that:

(R - A) < 0 - net attractive force - tendency to flocculate

(R - A) > 0 - net repulsive force - tendency to disperse

(R - A) = 0 - equilibrium position.

The shear strength of a clay is completely determined by the electrical forces acting between the particles, i.e. A, R and I. The electrical forces are thus the primary cause of strength; the four factors particle spacing, particle orientation, externally applied stresses and characteristics of soil water system,which determine the electrical forces, are secondary contributors to 'shear strength'.

4.7.2 General Strength Equation

$$S = c + \bar{\sigma} \tan \phi$$

The above equation represents the Coulomb relationship where S is the shear strength, c is cohesion, ϕ is the angle of internal friction and $\bar{\sigma}$ is the pressure normal to the failure plane.

4.7.3 The Components of Shear Strength

The shear strength of clay is essentially composed of two components:

1. Physical or dilatent components due to frictional resistance and interlocking between particles. They are proportional to the effective normal stress on the shear plane and can be divided into:[132]

   (a) Macro-dilatent components caused by the lifting of one mineral on the other;

   (b) Micro-dilatent components caused by lifting of one clay mineral over the roughness of the other;

   (c) Non-dilatent components due to mass forces between the atoms in the one mineral and the atoms of the other; and

2. Physico-chemical components due to the interparticle attractive and repulsive forces whose net effect is defined as 'true cohesion'. The fact that true cohesion is a purely physico-chemical and mineralogical property of soil was demonstrated by Skempton[133] who found the ratio of true cohesion, c_e to $\bar{\sigma}$ tan ϕ_d, where ϕ_d is the angle of drained shear resistance increases in direct proportion with the activity.

Cohesion can therefore be defined as the part of the soil strength that is present independently of any applied pressure, either mechanical or capillary, and which would remain, though not necessarily permanently, if all applied pressure were removed. It is due to a bonding of particles within the soil mass by physico-chemical mechanisms of an interatomic, intermolecular or interparticle nature.[134]

4.8 Thixotropy of Clays

Thixotropy of clay may be defined as an increase of the strength of clays occurring following disturbance or remoulding. It arises from the re-arrangement of clay particles due to internal energy and internal stresses resulting from the removal of an external stress which caused remoulding.[135]

4.9 Erosion Studies

In the past accumulated field data has been the basis for the design of channels. In recent years, investigators have produced a considerable amount of information on scouring and the rate of transportation of sediment for sandy soils. The data relates mainly to the grain size distribution of sandy materials. There was an obvious

need to extend research into the range of cohesive soils. The parameters, many of which have been postulated by researchers to have effect on erosion, are given in Table 4.2.

Work done on the design of stable channels has been carried out using one of the following design methods:

1. Maximum permissible velocity
2. Tractive force theory
3. Regime theory.

The first major attempt to formulate a regime theory was made by Kennedy[136] at the end of the last century. The original formula was:

$$V_c = \alpha y_o^{\beta}$$

where V_c is the critical velocity for stability

y_o is the depth of flow

α, β are empirical coefficients dependent on channel geometry, soil composition and general environment.

Cohesionless material obtains its main resistance to erosion from the submerged weight of the particles whereas the resistance to erosion for a cohesive soil is due to net attractive interparticle surface and electrochemical forces, the latter not remaining constant but a function of the fluid quality and also of time.

Early research was conducted on the erosion resistance of channels, and in the nineteenth century the first set of 'critical values' at which erosion would commence were published by Du Buat[137], e.g. "scouring velocity-Potters' clay 0.3 ft/sec", and this research was later extended to more soils.

Characteristic	Parameter
Physical	Soil Type (clay mineral) Percentage of Clay L.L. and P.L. and Activity Specific Gravity
Physico-chemical	Base Exchange Capacity Sodium Adsorption Ratio Pore Fluid Quality Pore Fluid Environment
Mechanical Properties	Shear Strength Cohesion Thixotropy Swelling and Shrinkage Properties
Conditions of Environment	Weathering (wet-dry) Freeze and Thaw Prestress History

TABLE 4.2: Soil Properties used in Evaluating Erosion of Cohesive Beds by Some Researchers

In 1916 Etcheverrey[138] published a set of critical velocities ranging from 0.8 m/s to 0.9 m/s (2.75-3.0 ft/sec) for loams, and corresponding velocities of 1.2 m/s to 1.5 m/s (4-5 ft/sec) for stiff clays. At the same time a special committee on irrigation hydraulics[139] presented similar values:

loams 0.5 m/s-0.9 m/s (1.5-3.0 ft/sec)

stiff clays 0.8 m/s-1.5 m/s (2.75-5.0 ft/sec),

the values being based on available field data.

In 1936 Russian engineers[140] included the effect of density, which resulted in the values for velocity and shearing stress being slightly higher than the values reported by the Americans:

stiff clays 1.2 m/s-1.7 m/s (4-5.5 ft/sec)

The American engineers considered that average flow velocity was the only flow parameter relating to erosion, although as early as 1874 the concept of tractive force or bed shear stress was introduced by Du Boys[141].

The Russians later considered shear stress and also introduced soil classification and void ratio of the bed material as a criteria of erodibility. It was found that cohesive materials with the same classification and very similar size grading may have widely different resistances to erosion. It was recognised that besides mechanical composition other factors may control the soil erodibility. It was also recognised that the resistance to erosion of artificial channels may increase considerably with time.

Studies by the U.S.B.R.[142] attempted to correlate maximum tractive force with Plasticity Index (P.I.) and mean grain size on the basis of field data. These studies seemed to show that the critical tractive force had very little correlation with mean grain size, but there was some indication that the critical tractive force increased with Plasticity Index. The critical tractive force was also found to be higher for ephemeral channels than for continuously wetted channels, suggesting that dessication may increase resistance to erosion.

Åke Sundborg[143] demonstrated that the critical velocity decreases with decreasing particle size as low as silt, i.e. 50 microns, and presented two sets of curves relating critical velocities to grain diameter. For sediments finer than 50 microns the critical velocity increased with decreasing particle size. Sundborg derived an equation for critical velocity for erosion, but unfortunately the equation needed comprehensive laboratory experiments to determine the necessary parameters to enable calculation and so he suggested that, in view of the meagre relationships between grain size and erodibility, the most rational approach would be to examine the shear strength of the bed material in situ by means of some kind of shear test.

Laboratory research, in general, has been done on two classes of cohesive soils:

1. Medium to high strength consolidated clays;
2. Soft freshly deposited, mid to low strength older deposits.

Sundborg[143] assumed the validity of Coulomb's Law for shearing resistance during erosion. This assumption led to a linear relationship between cohesion and critical shear strength of the bed material.

Dunn[144] also assumed the validity of Coulomb's Law and tried to correlate experimentally the shear strength as measured with a shear vane with the measured critical tractive force. A laboratory study was conducted on remoulded consolidated clay subjected to erosion by a submerged vertical water jet impinging on the clay sample whose upper surface formed part of the bottom of a small cylindrical container as shown in Figure 4.11. The critical flow was arbitrarily defined as the flow at which "the water becomes cloudy and no subsequent clearing occurred".

However, before that stage was reached limited erosion occured. For each soil tested, the plot of critical shear stress (τ_c) versus the vane shear strength (S_r) gave, according to Dunn, a straight line. The slope of that line was subsequently related to Plasticity Index, the percentage of silt and the percentage of clay and a statistical parameter called the 'dos Santos soil constant', 't'.[145] The latter parameter described the mechanical distribution of the material based on material passing the No. 200 (75 microns) sieve and statistical constants based on the phi probability curves.[146] Dunn did not record density during any test, the only sample identifying index used being the Plasticity Index, and the seven samples chosen varied over the narrow range of 6 to 16. This would suggest that the clay minerals were relatively inert and the soil as a whole not subject to adverse swelling. However, in isolating Plasticity Index and vane strength it can be assumed that there is a great deal of similarity in the soil structure and mode of erosion. The plot of increase of τ_c with S_r was based on an inadequate number of points and is therefore open to question. There was an indication, however, that τ_c on average increases

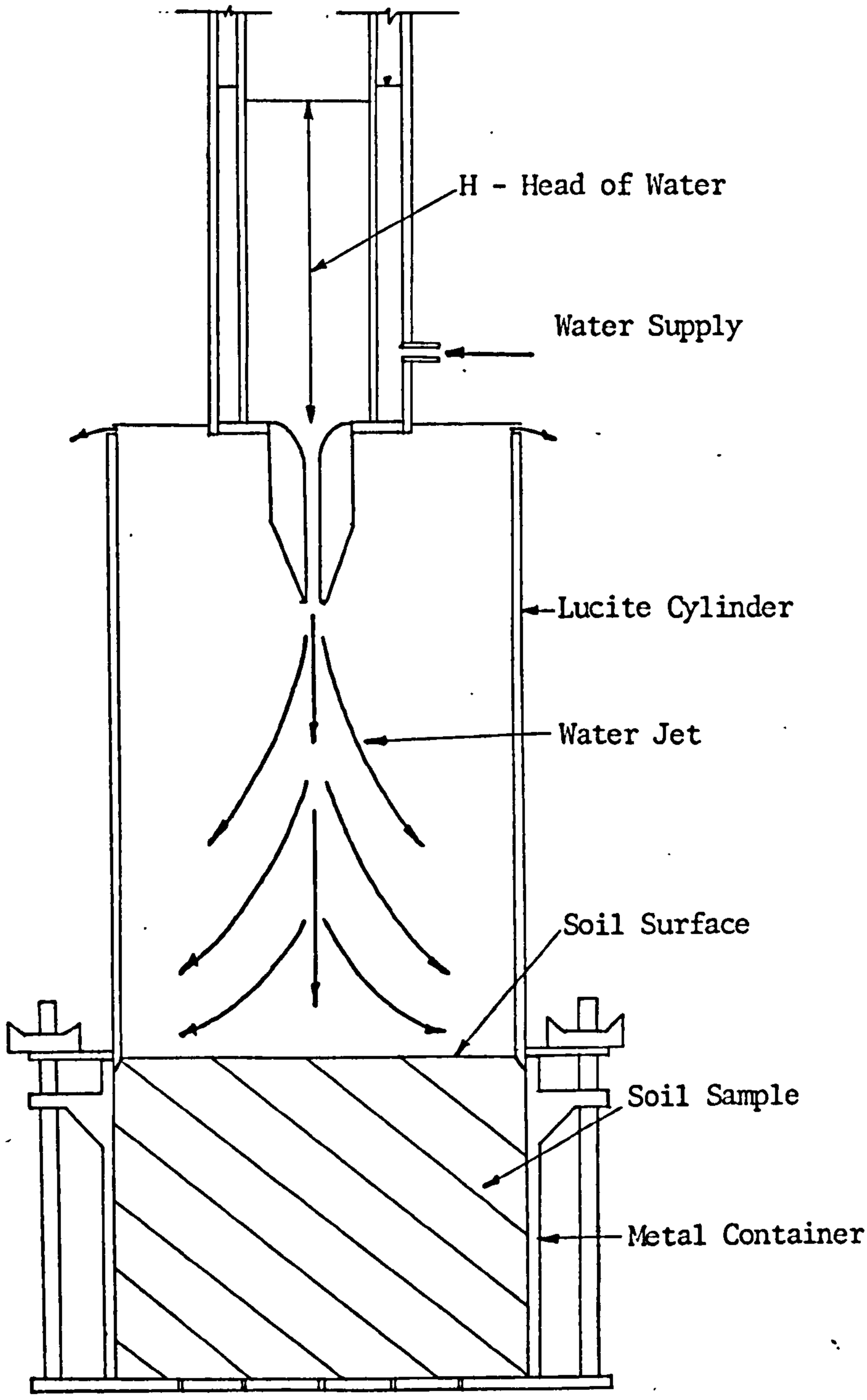


FIGURE 4.11: Dunn's Jet Test Apparatus

with increasing S_r.

Smeardon and Beasley[147] attempted to relate critical shear stress to Plasticity Index to percentage of clay and some statistical parameters of the sediment. These parameters yielded only descriptive information for the soil, which was compacted at a given moisture content and to a given density. Samples were placed in the base of a flume and subjected to erosion, but again the 'point of bed failure' was arbitrarily and vaguely defined as "the tractive force at which the bed material was in general movement".

A logarithmic relationship between critical tractive force and soil properties was established, showing best correlation with Plasticity Index and poorest correlation with mean particle size. Cohesion was decided to be the controlling erosive factor, but it was not evaluated as a parameter.

The relationship showed that for a critical tractive force of 0.03 p.s.f. (0.138 kN/m^2) a soil would have a Plasticity Index of 8, or a dispersion ratio of 40. However, the first soil in their study, number one, had a greater Plasticity Index of 10.2 and a small dispersion ratio of 22.2 for a tractive stress of 0.0199 p.s.f. (0.137 kN/m^2).

The tests were performed in an open flume with loosely placed soil, but no soil strength tests were carried out. The correlation with Plasticity Index obtained is particularly interesting from the point of view of comparison with results of other investigators:

$$\tau_c = 0.0034 \ (PI)^{0.84}$$

However, for a Plasticity Index range of 10 to 20 τ_c ranged between 0.02 p.s.f. (0.138 kN/m^2) and 0.047 p.s.f. (0.324 kN/m^2). For the same range Dunn obtained low strength values of τ_c between 0.35 and 0.50 p.s.f. (2.4-3.45 kN/m^2). In general the two investigations led to critical shear stresses differing by a factor of 10 to 15.

Moore and Masch[148] and Espey[149] attempted to devise a small-scale test apparatus to evaluate the scouring resistance of cohesive soils. Moore and Masch used the principles of a submerged water jet similar to Dunn's.[144] As an erosion criterion they used an "indirect scouring index" which is a dimensionless coefficient relating erosion depth with time. They found that this index at a certain velocity varies with the Reynolds number corresponding to the particular velocity and with the Reynolds number corresponding to 'incipient failure'. Again failure was arbitrarily defined, and depended on the judgement of the observer, making utilisation of results difficult.

Espey used a system of rotating coaxial cylinders with the inner stationary cylinder composed of clay samples. Neither investigation showed conclusive results. Espey obtained a range of shear stress between 0.18 p.s.f. (1.24 kN/m^2) and 2 p.s.f. (13.8 kN/m^2) depending on the interpretation of test data. No correlation with soil properties was attempted. Erosion took place by removal of relatively large pieces of the sample.

Carlson and Enger[150] performed field and laboratory studies on various cohesive soils. 'Critical' forces were obtained from a test in which clay samples were set into a well flush with the bottom of a circular tank and water circulated over the sample with a rotating

impellor. Critical tractive forces were measured as well as the Liquid Limit, Plasticity Index, density, soil gradation, vane shear strength and shrinkage limits.

Retoric[151] followed with an approach similar to Epsey's and presented linear relationships between τ_c and the vane shear strength, S_r. For S_r between 20 p.s.f. (138 kN/m^2) and 200 p.s.f. (1379 kN/m^2) the critical shear strength ranged between 0.3 p.s.f. (2 kN/m^2) and 2 p.s.f. (13.79 kN/m^2) a range quite close to that of Espey. Both ranges are higher by an order of magnitude than the recommended design values for soft to medium strength clays quoted by most texts. Attempts to correlate τ_c with P.I., percentage clay or exchangeable calcium-sodium ratio did not lead to conclusive results.

Abdel-Rahman[152] conducted experiments on artificially compacted clay beds in an open channel. In all Rahman's experiments erosion eventually stopped. Variables other than the shear strength of the bed and the gross shear strength were not considered, and no critical stress as such was defined. Attempts were made to express the mean depth of scouring and roughness of eroded bed material in terms of the shear strength of the soil, the bed shear stress and the water properties. Plasticity Index of the soil was 25. Experimental shear stresses ranged between 0.015 p.s.f. (0.10 kN/m^2) and 0.09 p.s.f. (0.62 kN/m^2). For the same Plasticity Index, Smeardon and Beasely gave τ_c = 0.0045 p.s.f. (0.03 kN/m^2) whereas Dunn's experiments indicated much higher values.

Berghager and Ladd[153] conducted experiments in a small flume to study the effect of shear strength for zero effective stress on the

resistance to erosion of Boston blue clay. No correlation was attempted between flow and soil parameters.

Saturated samples of the clay having a Plasticity Index of 12.5 and a mean grain size of 3.5 microns were preconsolidated to 2000 p.s.f. (1.37×10^4 kN/m^2) and did not display any observable erosion at shear stresses as high as 0.7 p.s.f. (4.8 kN/m^2) whereas Smeardon and Beasley[147] for a similar Plasticity Index gave τ_c = 0.03 p.s.f. (0.21 kN/m^2).

Middleton[154] was one of the first researchers to attempt to relate soil erodibility to measurable physical characteristics of the soil. He determined the silt plus clay content in the soil in the absence of chemical or mechanical dispersion, called the 'Suspension Percent'; the silt plus clay after standard dispersion, called the 'Ultimate Silt plus Clay'; and two other common physical characteristics of the soil, the 'Colloid Percent' (by a water vapour absorption method) and the 'Moisture Equivalent'. Each of the physical measurements was expressed as a percentage of the dry weight of the soil fraction which was less than 2 millimetres in diameter.[155] The 'Dispersion Ratio' and 'Erosion Ratio' were defined by Middleton as:

$$\text{Dispersion Ratio} = \frac{\text{Suspension Percent}}{\text{Ultimate Silt plus Clay}}$$

$$\text{Erosion Ratio} = \frac{\text{Dispersion Ratio}}{\text{Ratio of Colloid Percent to Moisture Equivalent}}$$

Middleton suggested that the dispersion ratio and the erosion ratio are the most significant soil characteristics influencing soil erodibility and in general that an erodible soil has a dispersion ratio

greater than 10 and an erosion ratio greater than 15 while a non-erodible soil has a dispersion ratio and an erosion ratio less than these values. Middleton did not, however, define an erodible soil in real terms, i.e. the amount of erosion likely to occur. Also suggested as factors influencing erosion were the amount of organic matter and the silica-sesquioxide ratio. In general Middleton found that soils that were comparatively resistant to erosion had low dispersion ratios, low silica sesquioxide ratios and high ratios of colloid to moisture equivalent.

Bennett[156], a few years previous to Middleton's work, had also suggested that for humid-tropical and humid-temperate soils degree of erosion was related to the silica-sesquioxide ratio.

Middleton, Slater and Byers[157,158] stated, after exhaustive physical and chemical studies of soils subjected to erosion, that the quality of colloidal material was more important as an indication of erodibility than quantity. Again the researchers stated that the ease with which fine-textured soils become dispersed in water, as indicated by the Suspension Percent and Dispersion Ratio, appeared to be a good qualitative index of soil erodibility.

Lutz[159] studied several types of clay and found that the silt and clay of a relatively non-erodible soil was aggregated into large water stable granules while the fine material of a highly erodible material was readily dispersedin water. Lutz found a soil with high plasticity to be non-erosive, whereas a low plasticity soil was erosive.

Several investigators[160,161,162,163] found that some soils which were relatively resistant to erosion were readily permeable to water. Soils with high clay contents were found to exhibit greater swelling characteristics than soils with lesser clay contents. Moisture equivalent and saturation capacity were used in these studies as an indication of water holding capacity. Musgrave[161,162,163] suggested that one of the primary factors affecting the amount of surface run-off, in his case from fields, is the soil's infiltration capacity, i.e. the process by which liquid enters the zone of aeration. This process includes the formation and building up of films around soil grains through wetting by rainfall, melting snow or temporary overtopping and the subsequent progressive downward movement of mobile water through the soil by 'film-flow' after the demand for pellicular water has been satisfied. For example, if two soils differ by as much as 10 millimetres infiltration per hour then over a four-hour period the amount of protection would differ by 40 millimetres.

An attempt to study the influences of definable soil properties on erosion was made by Grissinger.[146] Rather than stating a critical shear stress the determination of critical shear was given in terms of mass erosion rates. Subsequently Partheniades and Paaswell[165] using Grissinger's defined points estimated the critical shear stress as 0.04 p.s.f. (0.28 kN/m^2), of the same order as Smerdon and Beasley and Rahman.

Grissinger found that the orientation of the clay particles had a pronounced effect on erosion, the erosion rates decreasing with increasing degree of orientation. In general it was found that resistance to erosion varied with the type and amount of clay minerals,

clay mineral orientation, sample bulk density and antecedent water, and the temperature of the eroding water.

The percentage of clay was found to affect erosion; as the percentage of clay increased the resistance to erosion increased for all soil mixtures used, with the exception of Ca-montmorillonite additive. The Ca-montmorillonite was the only soil that showed, except at high density, decreasing stability with increasing clay content. This was attributed to the increasing swelling properties of the Ca-montmorillonite. The Plasticity Index of the Ca-montmorillonite was smaller than that of the Na-montmorillonite, 11 to 17 compared with 11 to 35 at 2 per cent to 10 per cent clay. The Na-montmorillonite showed greater resistance to erosion at a moisture content of 10 per cent. The samples were compacted on the dry side of optimum, the soil being in a flocculated state with a lesser degree of particle orientation, and hence the same soil is more susceptible to erosion.

A slight increase in the resistance to erosion was shown to occur with increasing bulk density of the soil, although the results were not conclusive. In this case it was thought that while bulk density referred to average density of the entire sample it is likely that surface densities, due to compaction techniques and expansion under flow, would not show the same marked degree of variance from sample to sample. In this case it was thought that bulk density was not as good an index as would be moisture content or void ratio in the erodible zone for those samples not fully saturated prior to testing.

The antecedent moisture, defined as the moisture content after compaction but prior to testing, was found to have a significant effect

on erosion rates which decreased with increasing antecedent moisture down to a minimum value. From there the erosion rates increased as shown in Figure 4.12. The erodibility of kaolinite mixtures was found to be dependent upon the size of the kaolinite particles.

Due to a wide range of Plasticity Index and overlapping of Plasticity Index between various sample mixtures no correlation between Plasticity Index and erosion was made. Thus, while the Plasticity Index was constant, erosion was dependent on the packing arrangement of the clay and the absorbed moisture content.

Ageing of the sample did not seem to have a pronounced effect on the erodibility after an ageing time of four hours, as shown in Figure 4.13.

Evidence indicated that erosion rates increased with increasing water temperature, the rates at 35°C being about twice those at 20°C, as shown in Figure 4.14.

Grissinger later extended tests to a wider range of natural soils and developed an equation relating erosion rate to the rate of sample wetting.[166] This empirical relationship is given by:

$$ER = bp\ (\Delta \text{ water/time})$$

where ER = erosion rate

b = regression constant

p = sample porosity

Partheniades[167] studied erosion and deposition of fine cohesive sediments in an open flume with recirculating water at ocean salinity

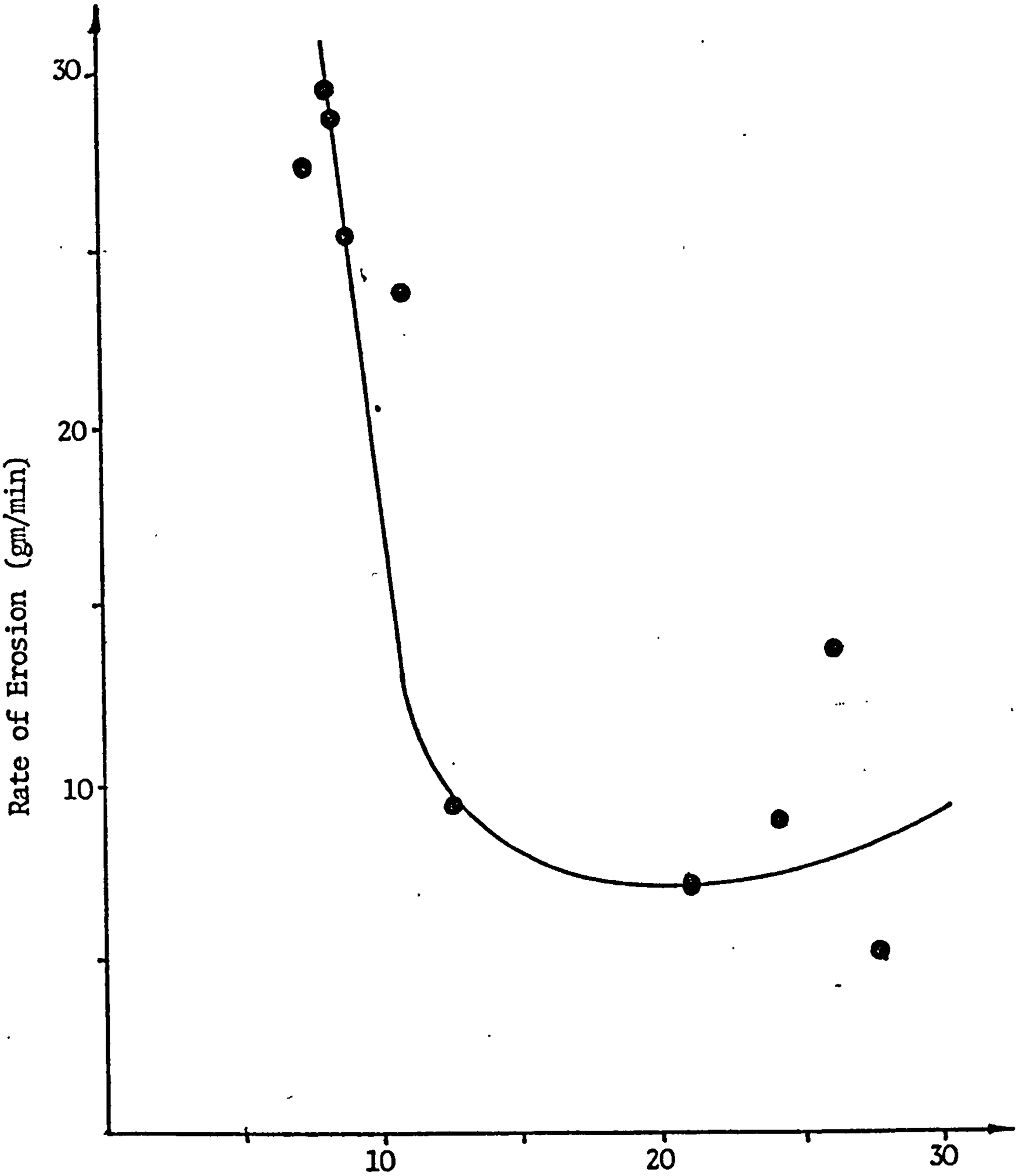


FIGURE 4.12: Influence of Antecedent Water on the Erodibility of a 10% Natural Silt-Grenada Silt Loam Mixture, packed at 8% Water to a 1.4 Bulk Density - after Grissinger[164]

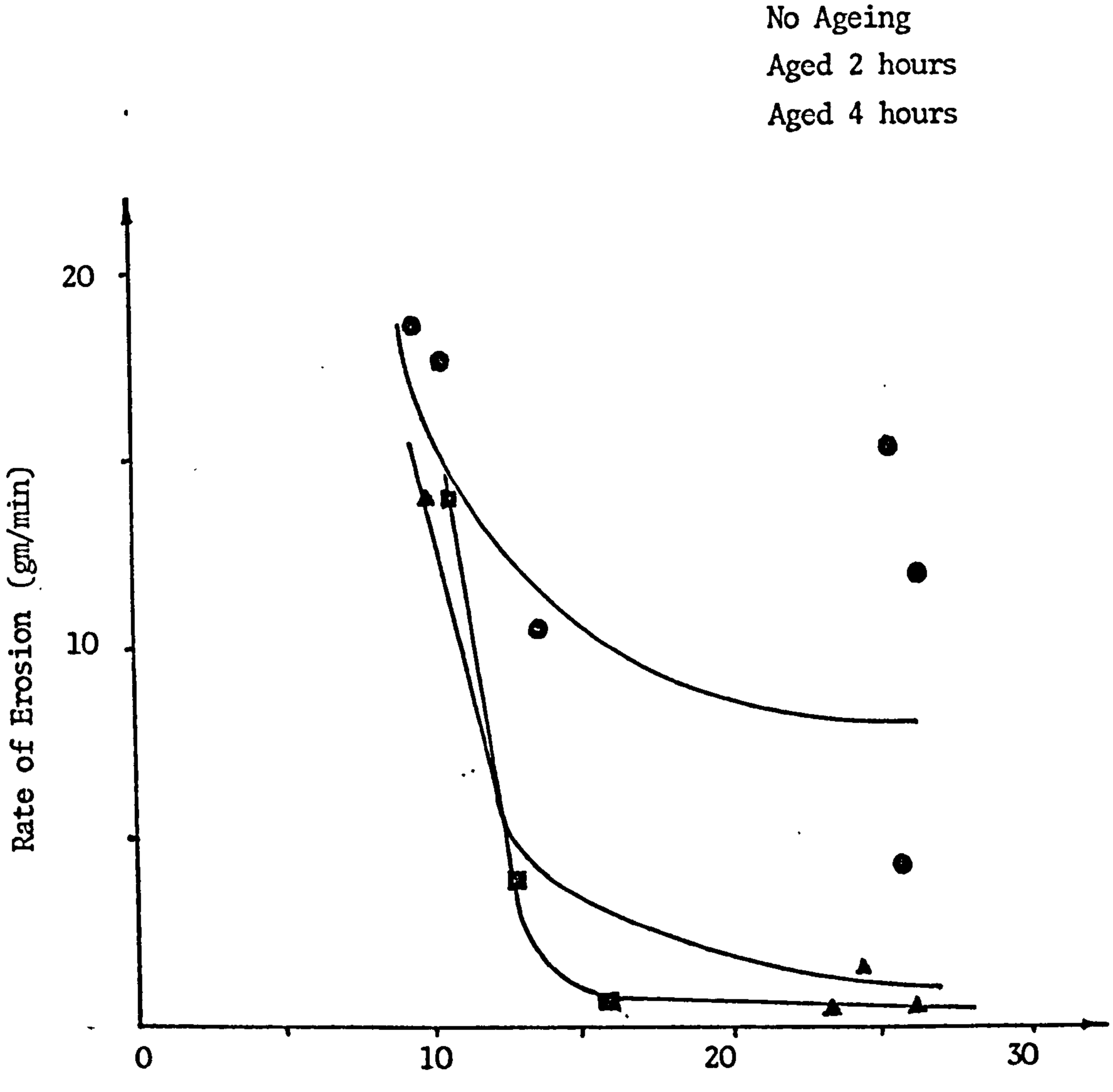


FIGURE 4.13: Effect of Ageing on the Erodibility of a Grenada Silt Loam packed at 10% Water to a 1.4 Bulk Density - after Grissinger[164]

Sample		Bulk Density (gm/cc)	Erosion Rate at 20°C (gm/mm)
G	Grenada Silt Loam	1.5	2.2
K	+20% Coarse Kaolinite	1.4	6.0
M	+5% Sodium Montmorillonite	1.4	0.5

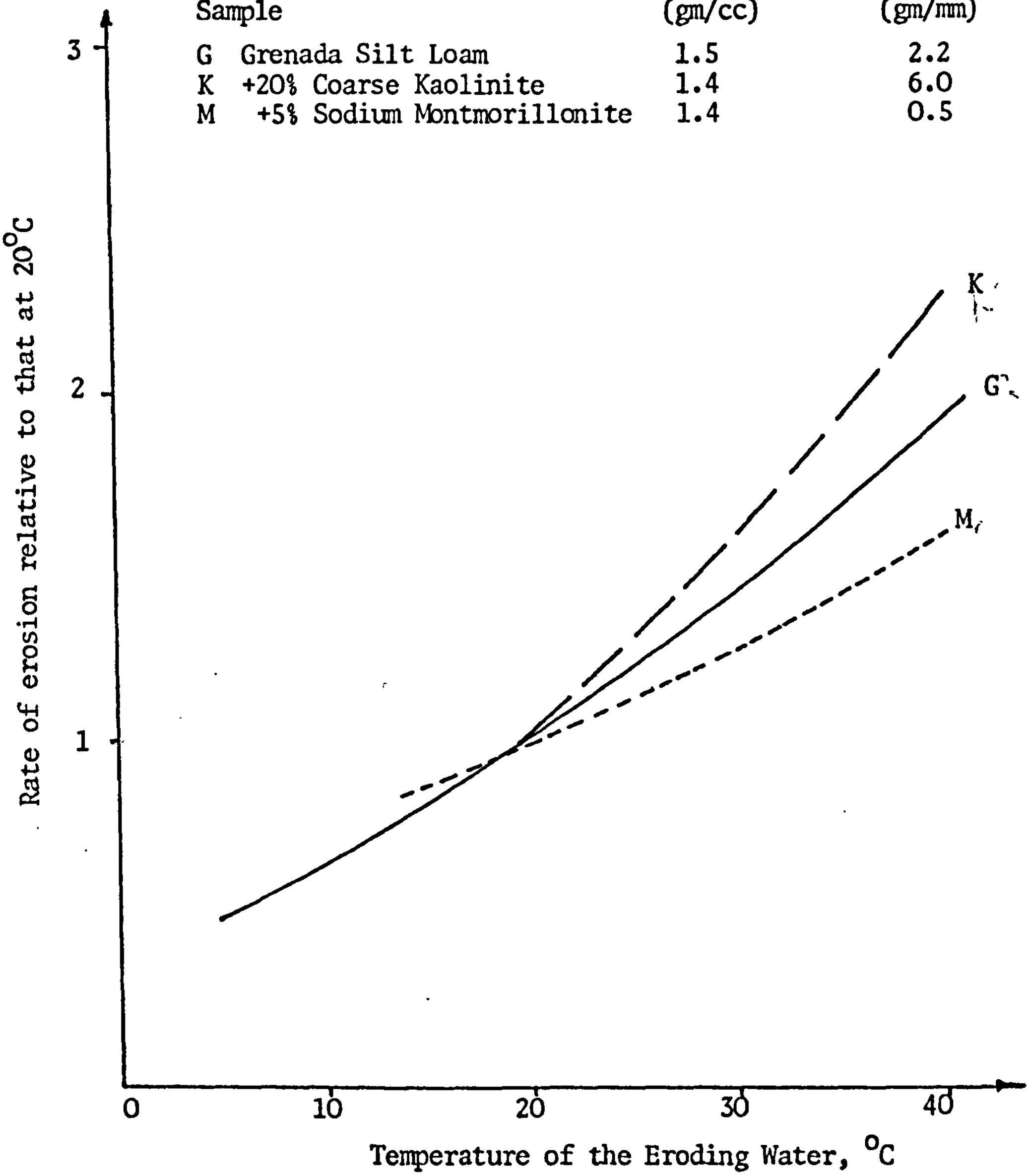


FIGURE 4.14: Influence of Temperature on Erodibility of Grenada Silt Loam with Admixtures and tested at 10% Water - after Grissinger[164]

and constant depth. Most of the sediment used was composed of silt, clay and traces of sand and some organic matter sampled from San Francisco Bay (L.L. 99, P.I. 55). Two beds were used, one remoulded at field moisture content of about 110 per cent and with a remoulded shear strength of 20 p.s.f. (137 kN/m^2), while the other was flocculated and deposited directly in the flume from suspension at very low flow velocity. The ratio of strengths of dense to flocculated beds was of the order of 100 to 1.

Conclusions arrived at by Partheniades included:

1. Minimum velocity or shear stress at which erosion was first observed was about the same for both beds.

2. The minimum scouring shear stress was found to be of the order of 0.0010 p.s.f. (6×10^{-3} kN/m^2).

3. For these tests the erosion rates were found to be independent of the macroscopic bed shear strength.

For the two tests on the dense beds lower shear strength was achieved at higher moisture contents, and this was attributed to cementing iron oxides dissolved in the water.

The concentration of the suspended sediment in the eroding water was seen to have no effect on erosion rates.

Partheniades observed an unexpected and striking bed erosion pattern consisting of a well-defined, relatively straight, narrow zone of erosion near the centre of the flume, the rest of the bed having a wavy appearance with small and smooth ripples. This was attributed to secondary boundaries generated at the frictional boundary and wall of

the channel which might have affected the instantaneous shear stress. The hard crust formed on the clay surface was thought to be due to an increase in the electrochemical attraction between clay particles caused by dissolved iron oxides which were thought to have been derived from the barrels in which the material had been stored and also from the flume.

Partheniades showed the erosion rates to depend strongly on the increase of average bed shear stress past a threshold value. Figure 4.15 shows erosion rates plotted versus the bed shear stress, τ_b.

A mechanistic model was developed by Partheniades to explain the observed erosion phenomena. It was assumed that the instantaneous shear stress, τ_o, has a normal distribution with mean $\bar{\tau}_o$, and standard deviation τ_o no. $\bar{\tau}_o$ is given by the logarithmic velocity distribution law:

$$\sqrt{\frac{\bar{\tau}_o}{\rho}} = Ky \frac{du}{dy}$$

or by $$\tau_b = \gamma_w RS_e$$

in which ρ = density of water

y = distance from the bed

R = hydraulic radius

u = local time average velocity

K = Karman's Universal Constant

S_e = slope of the energy line

The interparticle attractive forces, C, were assumed constant and proportional to the cohesion, c, due to these forces. Also σ_{max}, the

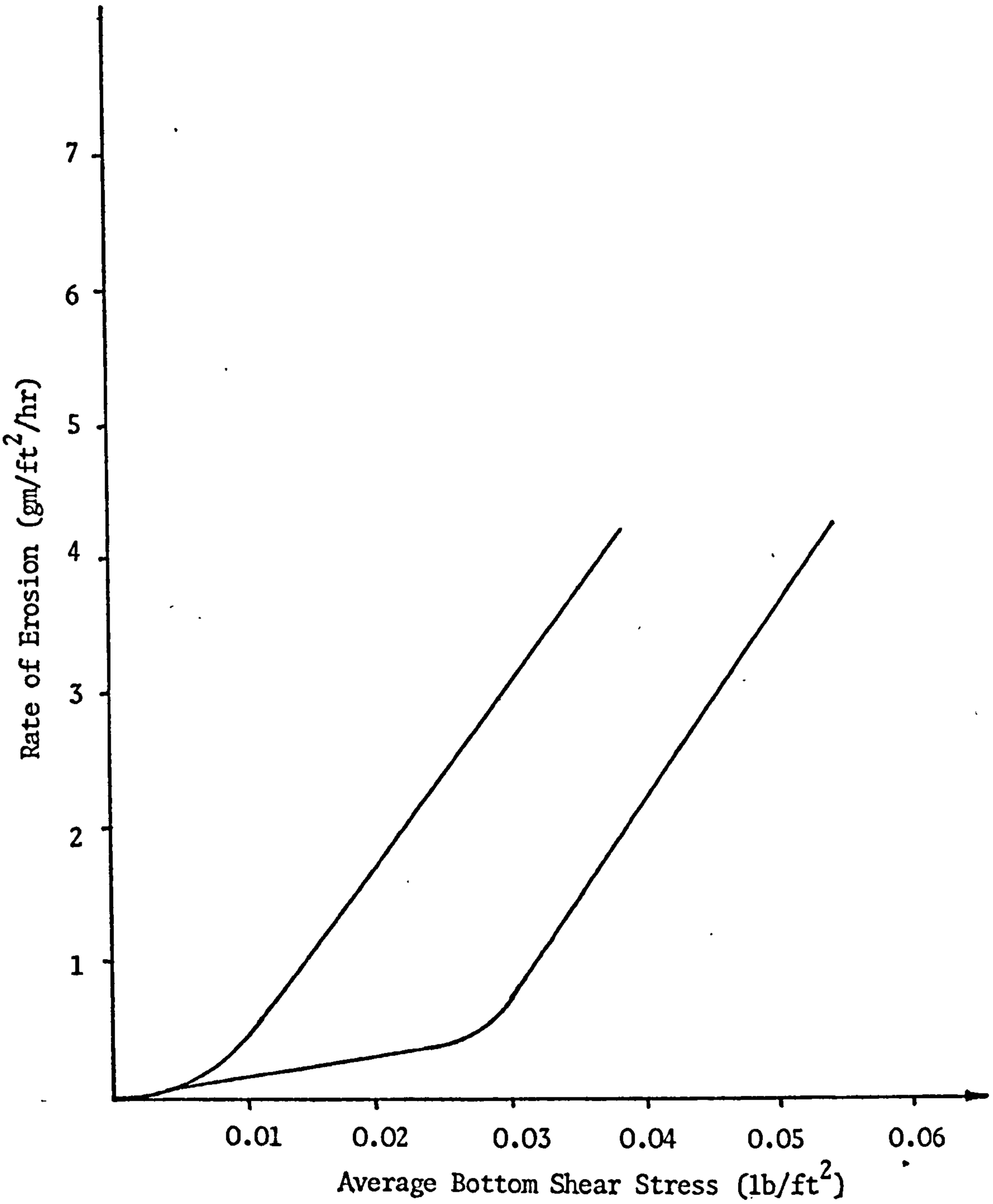


FIGURE 4.15: Variation of Erosion Rates with Bottom Shear Stress - after Partheniades[167]

maximum tensile stress induced by the flow on the discontinuities of the clay particles or within the particles themselves must be proportional to the local shear stress, τ_o.

Using as a criterion for erosion the condition:

$$\frac{\sigma_{max}}{C} = \frac{k\ \tau_o}{c} \geq 1$$

in which k is an overall proportionality constant. The following general equation for the erosion rate was derived:

$$E = \frac{A'\ D_s\ \gamma_s}{t(\tau_o)} \left[1 - \frac{1}{\sqrt{2\pi}} \int_{-(c/k\ \tau_o\ \eta_o) - (1/\eta_o)}^{(c/k\ \bar{\tau}_o\ \eta_o) - (1/\eta_o)} \exp.\ (-\frac{w^2}{2})\ dw \right]$$

where D_s = the average diameter of the eroded clay particles or clay clusters

γ_s = unit weight

A' = dimensionless shape constant

$t(\tau_o)$ = time required for a stress $\sigma_{max} \geq c$ to act for the removal of a particle.

The erosion rates, E, are given in terms of eroded dry weight per unit bed area and unit time. The product $(A'\ D_s\ \gamma_s)/\ to_t(\tau_o)$ is a function of the soil properties. η_o depends on the bed surface configuration and the flow conditions.

Martin[168] investigated the effect of the salt concentration of the eroding water on erosion resistance. Martin indicated that changing the salt concentration of the fluid would have noticeable effect on the strength of clay, both in its natural and remoulded states.

The general effects of electrolyte replacement on strength were considered by Bjerrum and Rosenqvist[169], who established that clays deposited in a saline environment and then subsequently leached with pure water will suffer a decrease in shear strength and a reduction in the liquid and plastic limits. Their analysis also indicated that the effect of weathering, coupled with thixotropic effects, resulted in an increase in shear strength.

Studies of the influence of chemical additives on the erodibility of cohesive soil have been made by several investigators. In tests by Liou[170], $Na_2 CO_3$ and $Ca(OH)_2$ were added to the soil to permit ion exchange before erosion took place. An attempt made to correlate vane shear strength of the soil with erodibility for different weights of chemical additives was successful only with $Na_2 CO_3$. Liou described the physical process for erosion of the Na-montmorillonite as "the mixture dispersed small clay particles and some chemicals; ... the soils were eroded away, layer by layer, approximately uniformly across the sample. For the Ca-montmorillonite "... the soils treated with 2.0 per cent and 5.0 per cent $Ca(OH)_2$ additives were eroded away in large flocs... The soils were eroded away locally..." The Na-montmorillonite displayed a sustained erodibility that increased as concentration of sodium ion increased. The Ca-montmorillonite showed erodibility over a range of low percentage of additive but showed a peak at 5 per cent Ca additive. Liou suggested that the variation in erodibility between Na- and Ca-montmorillonites occurs from the structure, flocculated or dispersed, that is established by the quantity of salt added.

The relationship of erosion susceptibility to water quality was shown in a study by Arulanandan et al.[171] Water quality of both the pore

fluid and the eroding fluid was changed in a series of rotating cylinder shear tests. The sodium adsorption ratio was varied, affecting the internal structure of the soil. The eroding fluid also had Na Cl concentrations varying from 0 to 0.1 N. The results showed that as the adsorbed sodium ratio increases the critical shear stresses decrease, the rate of decrease being a function of the salt concentration in the pore fluid. With low concentration of salt in the pore fluid, high shear stresses are resisted only when little of the original Ca or Mg ion has been displaced. When the pore fluid contains much salt (0.1 N) the resistance to erosion is sustained to much higher sodium adsorption ratios. Arulanandan et al attributed this to the ability of the soil to form strong flocculated structures with either higher valence exchange ions or high concentrations of salt in the pore fluid.

The eroding fluid was also shown to be used to stabilise the soil as increasing the concentration of salt in the eroding fluid resulted in lower erosion rates. Thus a quantity of 0.005 N.Na Cl was enough to reduce the erosion rate to 20 per cent of distilled water acting on the soil. Thus it could be said that in cases where the Na Cl concentration in the pore fluid is greater than in the eroding fluid, erosion can be expected, whereas when the reverse situation exists erosion is not expected. Reduction of salt concentration at the surface of the soil relaxes the internal force system and permits erosion to commence.

Christensen and Das[172] developed a technique in which samples were placed inside a brass tube as a lining. The clay was actually compacted by squeezing into a tube and the surface made smooth by removal of the inner tube which affected the orientation of the particles at the surface. Moulding moisture content, flow velocity and temperature of the eroding

fluid were varied. The samples showed an initial rate of erosion followed by steady state conditions, followed by rapid erosion due to the nature of the changing boundary forces as the surface becomes rougher and particle re-orientation occurs. A mechanistic model was derived which makes the understanding of erosion as an internal/ external energy system possible:

$$\dot{E} = \beta_E \exp(\alpha\tau) \quad (\tau < \tau_c)$$

and $$\dot{E} = E_{CR} \exp\left[\alpha_1 (\tau - \tau_c)\right] \quad (\tau > \tau_c)$$

where $\dot{E}$ = steady state erosion rate

τ_c = critical stress

α and β = soil and test parameters

The authors noted that as moulding moisture increased, total erosion weight decreased.

Wischmeier and Meyer[173] related the data developed since 1930 by the U.S. Department of Agriculture on run-off and erosion to properties of detachability and transportability of the soil. Using multiple regression techniques the authors found that significant parameters affecting erosion were soil texture, organic matter content, aggregation, surface pH and permeability of the surface. An erodibility nomogram was constructed which is used to predict the erodibility of soils at construction sites from soil parameters easily obtained through surface sampling.

4.3 Conclusions

The research cited here has shown that the problem of cohesive soil erosion is very complex. Definitions of erosion bed failure and critical shear stress in many studies have been arbitrary and based entirely on visual observation and the judgement of the observer. It becomes difficult, therefore, to predict erosion.

In general, low shear strength is not the only soil property governing erosion, and no definite correlation has been found between strength and erodibility and no well-defined empirical relation has been developed.

CHAPTER FIVE

EXPERIMENTAL APPARATUS, MATERIAL AND PROGRAMME

5.1 Introduction

This chapter introduces a laboratory investigation into erosion, describing the material used and experimental apparatus. The preparation of the test specimens is described as well as all items of equipment.

The techniques used in testing the samples are also outlined. In cases where procedure used varied substantially from British Standard Methods or is not covered by any standard, the techniques are described in greater depth.

5.2 Design Parameters for Apparatus

Velocity was one of the most important parameters to be considered since in earlier experiments[174] the velocity of flow was too low and no erosion of the clay took place. An estimate of the velocity at which clay might be expected to erode was obtained from Webber[175] who states that the maximum permissible water velocities for the design of stable channels in stiff clay, very colloidal in clear water as 1.15 metres per second and in water transporting colloidal silts as 1.5 metres per second .

Velocities likely to be experienced during overtopping were calculated by Bernouilli's equation[175], and while it is appreciated that the values obtained are very empirical they did aid the design process. Assuming no tailwater and no friction on the slope and with reference to Figure 5.1, Bernouilli's equation becomes:

$$h_D + h = \frac{V_1^2}{2g}$$

If, in the prototype a depth of water of 0.5 metres is assumed to overtop the dam, then the velocities attained at the base of the downstream slope for a number of different heights of dam is shown in Table 5.1.

Perhaps slightly more realistic values of velocity may be calculated by use of the Manning Formula:[175]

$$V = \frac{1}{n} R^{2/3} S^{\frac{1}{2}}$$

Assuming an initial breach 1 metre wide and 0.5 metres deep in the crest, then:

$$R = \frac{A}{P} = 0.25$$

and assuming slopes of 1:1, 1:1.5, 1:2, 1:2.5 and 1:3 and a coefficient of surface roughness, n, for an "unlined channel of earth, in a poor condition with weeds" of 0.05, then the velocities likely to be experienced on the downstream slope of an embankment are shown in Table 5.2.

H = Height of dam
h = Depth of overtopping water
V_B = Velocity of base of slope
S = slope

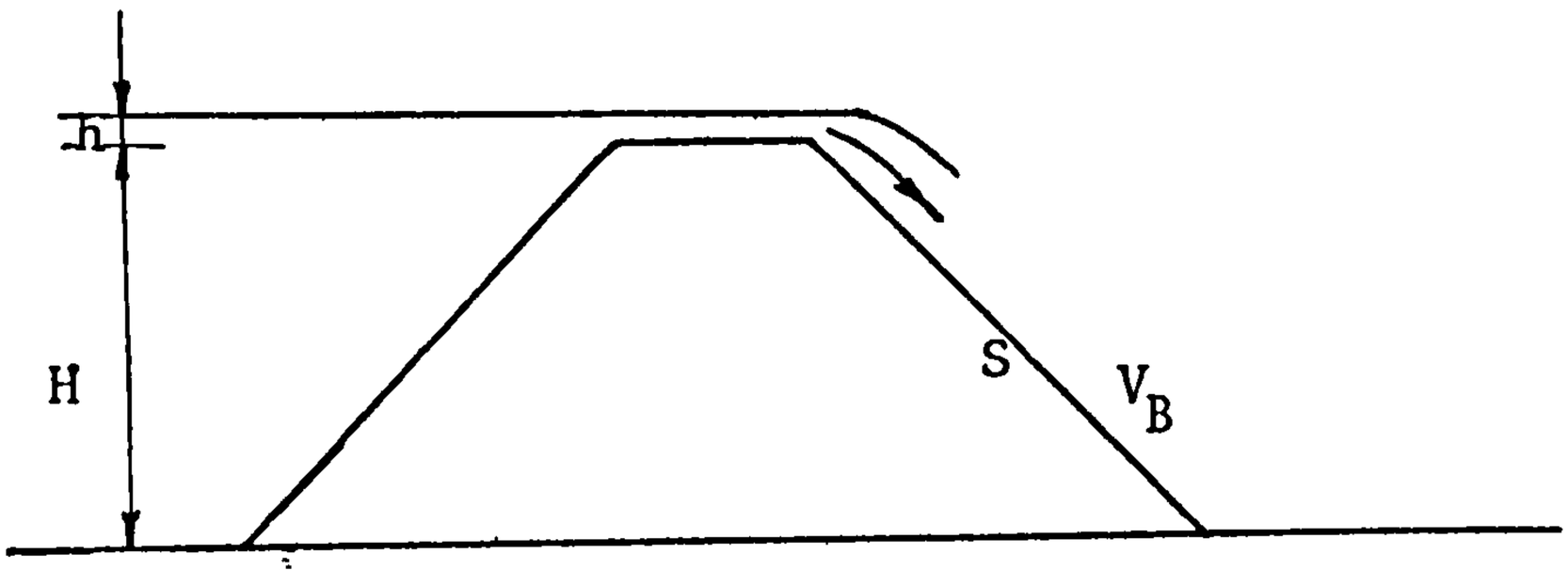


FIGURE 5.1: Flow over a Dam

H (m)	h + H (m)	2g	$V_B = 2g\ (h + H)^{1/2}$ (m/s)
10	10.5	19.62	10.15
20	20.5	19.62	14.19
30	30.5	19.62	17.30
40	40.5	19.62	19.93
50	50.5	19.62	22.26

TABLE 5.1: Estimated Velocities achieved during Overtopping - Bernoulli's Equation

S	$(S)^{\frac{1}{2}}$	$(R)^{2/3}$	1/n	V (m/s)
1:1	1.000	0.397	20	7.94
1:1.5	0.816	0.397	20	6.48
1:2	0.707	0.397	20	5.61
1:2.5	0.633	0.397	20	5.02
1:3	0.577	0.397	20	4.58

TABLE 5.2: Estimated Velocities achieved during Overtopping - Manning Formula

Clearly the velocity was an important parameter, and a compromise had to be made between the range of velocities required with that of the range obtainable. A minimum velocity of 2 metres per second was adopted for design of the channel. Other design considerations included:

1. Utilisation of as much existing equipment as possible including an available pump.
2. 'Closed-circuit apparatus' - due to material in suspension, the test apparatus was designed to be self-contained with a recirculating water system.
3. Adjustable channel - hinged at the uppermost end so that the slope of the channel could be altered.
4. Perspex "viewing section" - adjacent to sample.
5. Measuring equipment - measurement of discharge, sediment concentration etc.
6. Head losses in system.
7. Discharge control.
8. Overall size of equipment.

9. Sample size, as large as possible, to eliminate 'side effects'.

10. Cost.

5.3 Principal Apparatus

5.3.1 Closed-Circuit Test Channel

The basic requirement of the closed-circuit channel was to provide an accurately reproducible flow of sufficient velocity to erode a sample of compacted clay.

The main apparatus consisted of three principal components: (a) an upstream reservoir, (b) a channel and (c) a downstream reservoir, as shown in Plate 5.1, Figure 5.1 and Figure 5.2.

5.3.2 Upstream Reservoir

The upstream reservoir consisted of a one metre square metal tank, having a depth of one metre and fitted with a bell-mouthed exit.

Water entry to the tank was streamed through a flow diffuser within the tank, constructed from 100 millimetre diameter rigid PVC piping.

A piece of aluminium 'honeycomb', used as a flow-straightener, was installed in the bell-mouthed exit to ensure a 'smooth' water surface in the channel.

A tap was provided in one corner of the base of the tank to facilitate drainage. The tank was positioned on a 1.35 metre high timber staging.

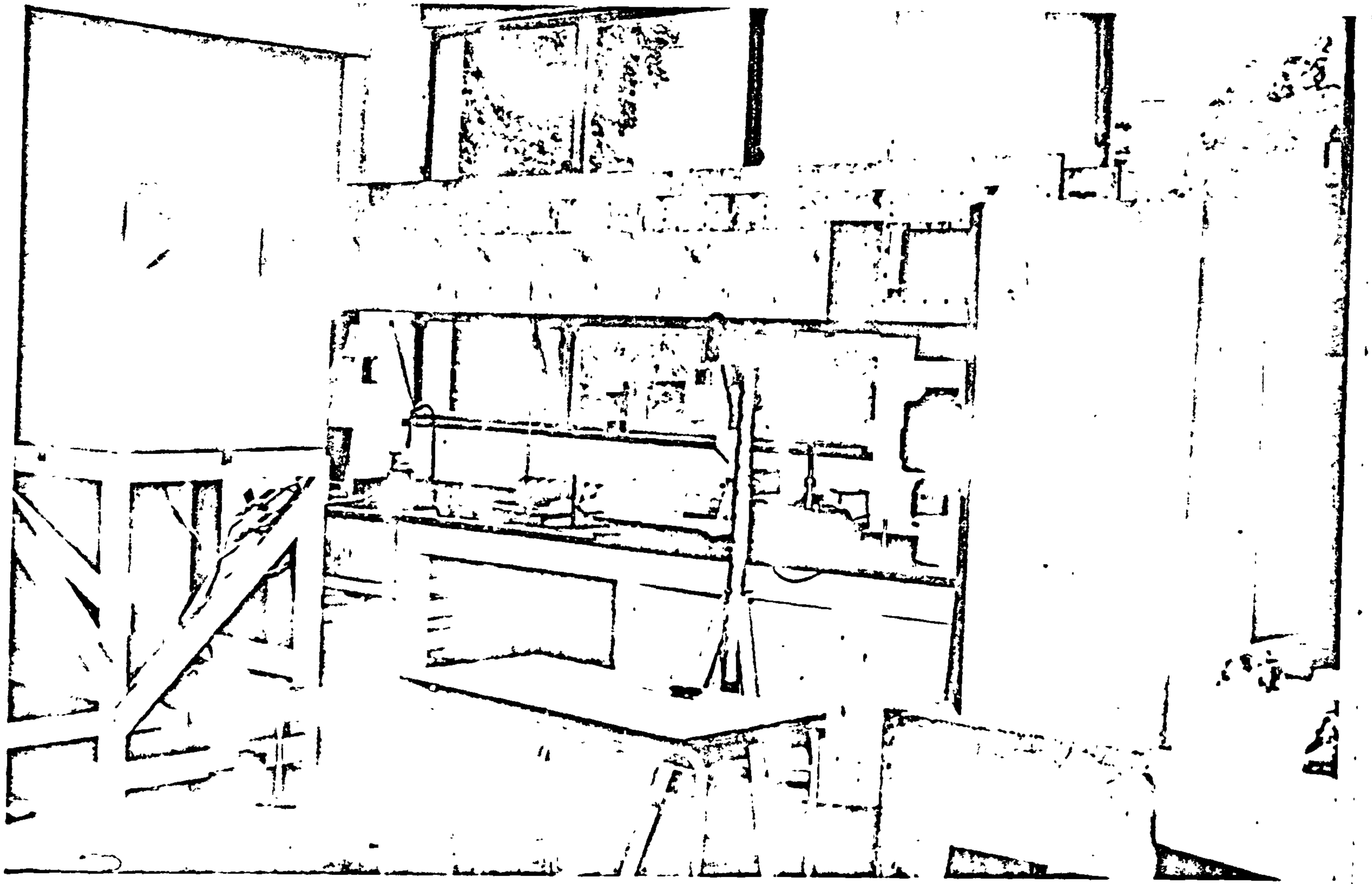


PLATE 5.1: Closed-Circuit Test Channel

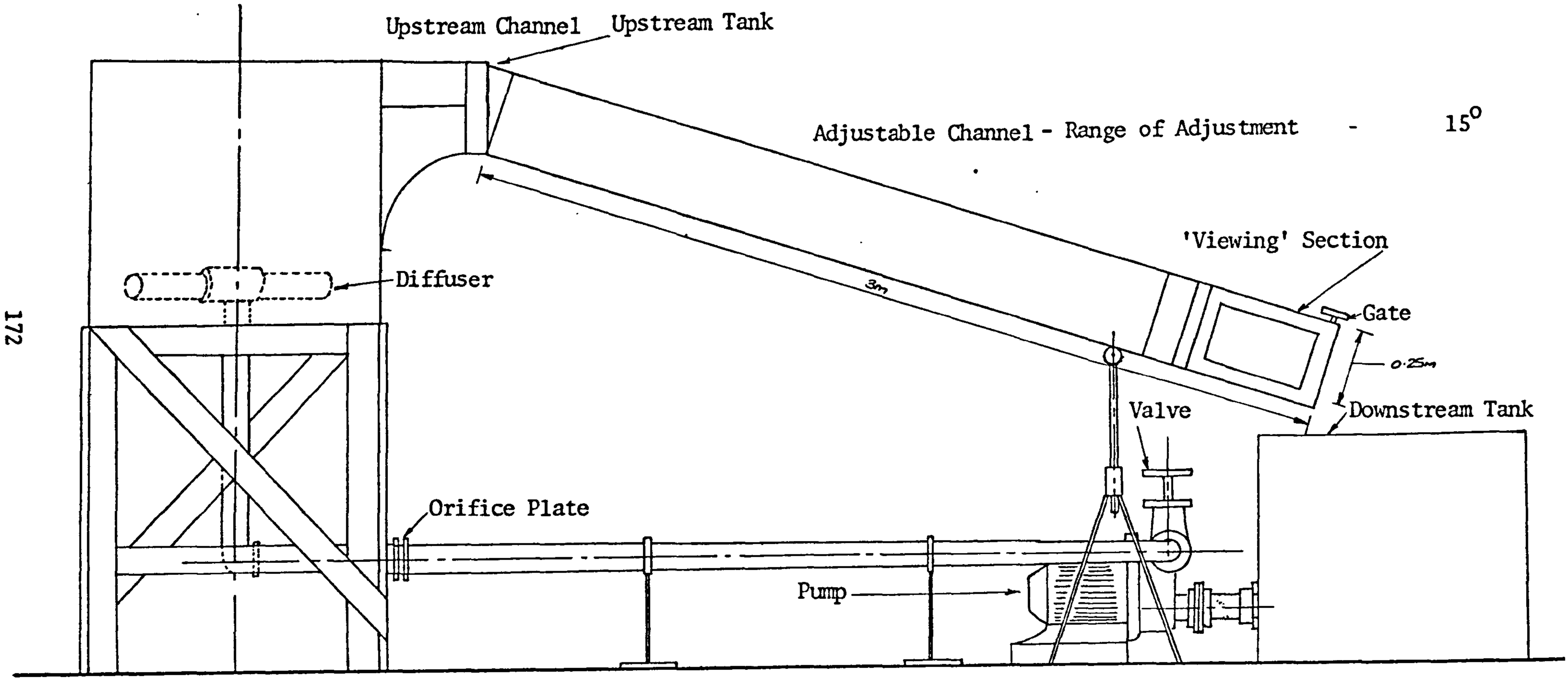


FIGURE 5.2: Front Elevation of Closed-Circuit Channel Apparatus (Scale 1:20)

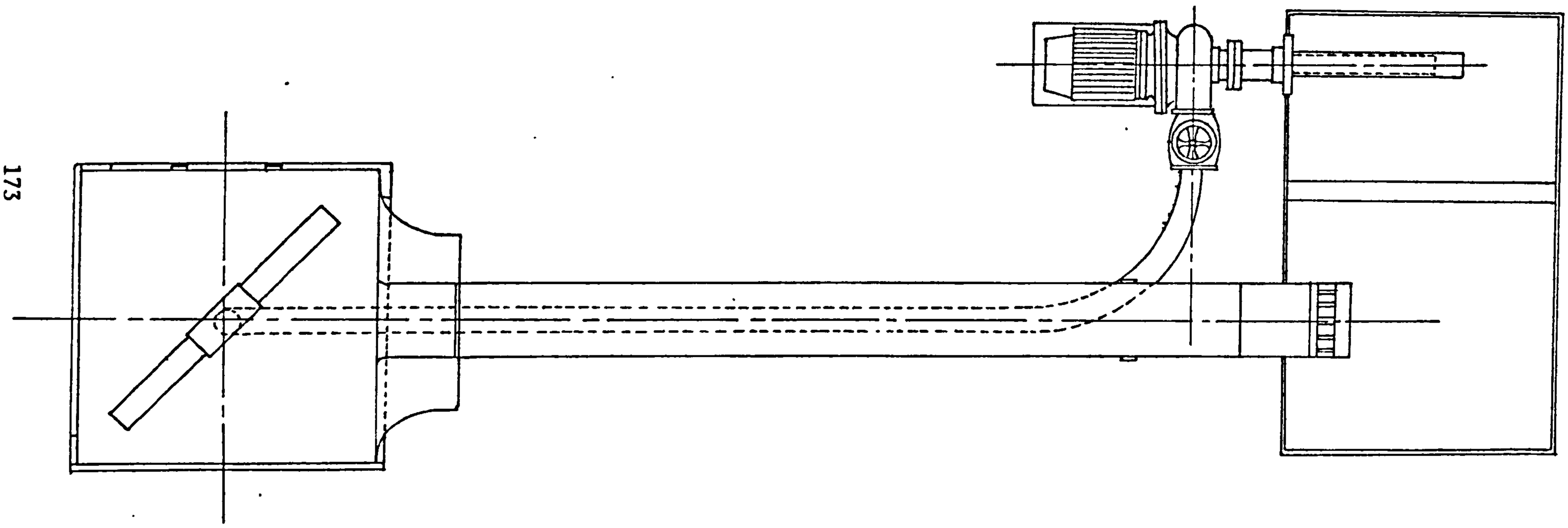


FIGURE 5.3: Plan of Closed-Circuit Channel Apparatus (Scale 1:20)

5.3.3 Channel

The channel was 3 metres in length and constructed of 12 millimetre marine plywood, protected by polyurethane varnish, and fitted with a 40 millimetre deep depression for the clay samples at the downstream end. The channel was 0.25 metres wide and 0.25 metres deep, the 'sample section' being of rectangular cross-section and 0.25 metres wide, 0.512 metres long but 0.04 metres deeper than the rest of the channel as shown in Figure 5.4.

Perspex 'windows' were installed in the walls of the channel, next to the sample section, these windows being marked with linear scales to enable the erosion process to be recorded.

A brass control gate was fitted to the downstream end of the channel.

The channel was hinged from the upper tank and supported by an adjustable prop which enabled the channel to be positioned horizontally or at varying angles of depression to a maximum of 15 degrees.

5.3.4 Downstream Reservoir

The downstream reservoir consisted of a galvanised steel tank, 1.5 metres long, 1.0 metres wide and 1.0 metres deep.

Water drawn from the tank by the pump passed through an intake constructed from 100 millimetre diameter rigid PVC pipe and in such a manner that water was drawn from the lowest portion of the intake to prevent air entering the pump. As an added precaution to prevent air entering the pipework system a 100 millimetre wide filter, consisting of an expanded wire cage filled with 37.5 millimetre single sized

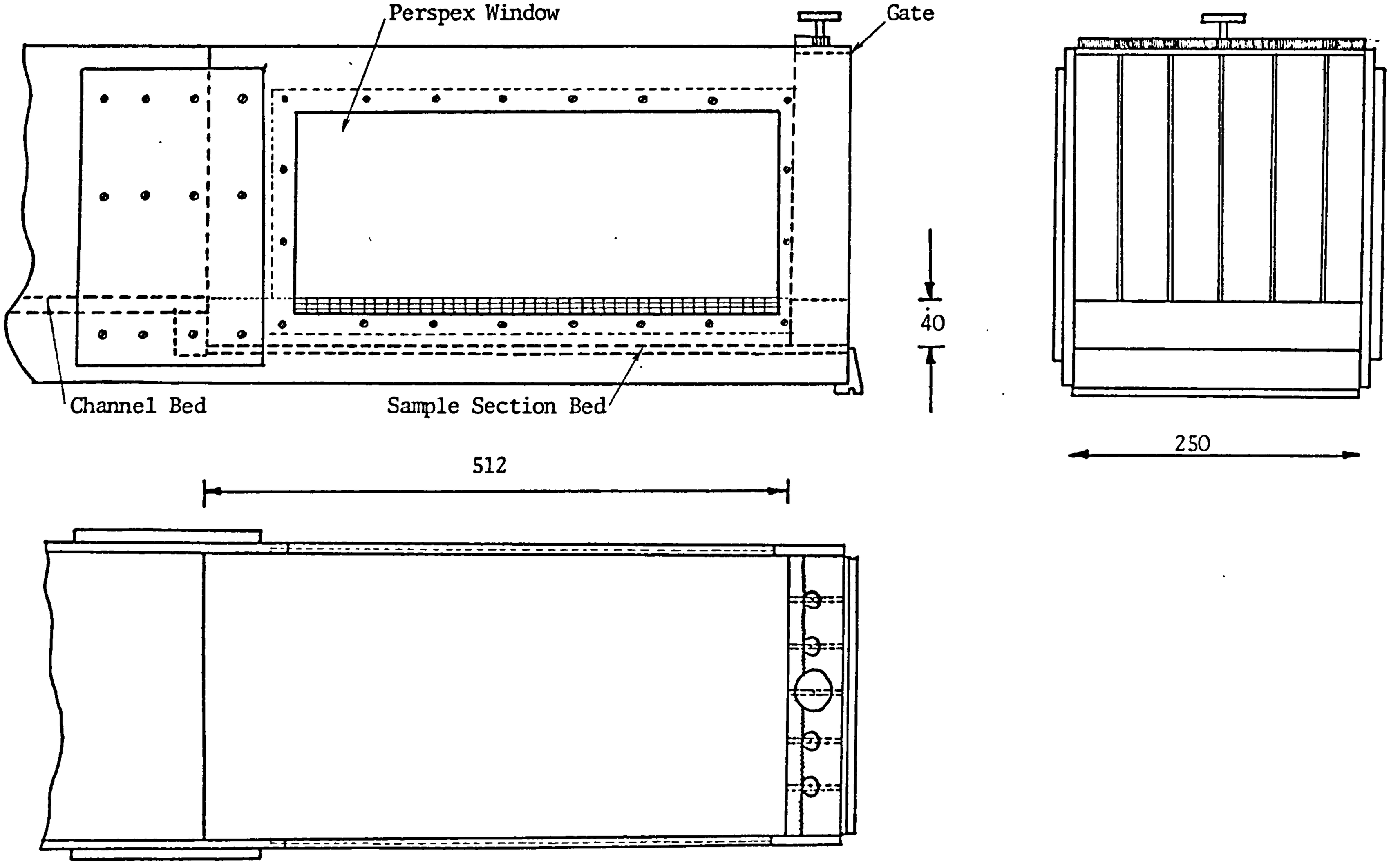


FIGURE 5.4: 'Sample Section' of Channel (Scale 1:5)

gravel, was constructed in the downstream channel. This filter effectively 'stilled' the water falling from the channel to the tank before the water was recirculated by the pump.

A tap was provided in the wall of the tank near the base to enable the system to be drained.

5.3.5 Pump

The pump employed was an Ingersoll-Rand Pumpak (Model AB04B) driven by a Crompton-Parkinson motor running at 1420 r.p.m., 4 kW power. The pump's characteristic curve is shown in Appendix C.

The pump discharged into a 100 millimetre diameter rigid PVC pipe running beneath the channel to the upper tank. The discharge was controlled by a 100 millimetre valve positioned next to the outlet of the pump.

5.4 Discharge Measurement

5.4.1 Orifice Plate

The criteria, as stated by BS 1042:Part 1(c), determining the position of an orifice plate in a pipeline include the requirement that the plate must be in such a position that it is a distance of at least the equivalent of thirty pipe diameters from any bend or valve.

This criterion could not be satisfied in the system as designed, and so there was a risk of turbulence and non-uniformity of the flow which might affect the accuracy of the readings obtained from the orifice plate. Thus, since the orifice plate was installed at a distance of only twenty-two pipe diameters from a bend, a turbine flow-

meter was used to alleviate the problem of turbulence and to accurately calibrate the system, i.e. relate the discharge predicted by the pressure drop across the orifice plate and obtained from the manometers with the actual discharge of the pump as recorded by the turbine flow-meter.

5.4.2 Turbine Flow-Meter

The turbine flow-meter, Type M6/2000/B250, manufactured by Meter-Flow Limited of Baldock,Hertfordshire, was installed in the closed circuit pipeline as shown in Plate 5.2.

The flow-meter, in conjunction with water and mercury manometers, enabled the orifice plate to be accurately calibrated against discharge.

The flow-meter assembly was 1.6 metres long and consisted of two transitional cones, reducing the diameter of the pipe from 100 millimetres to 50 millimetres, two 'flow straightening' sections, and in the centre the turbine flow-meter itself.

Due to the limited availability of the flow-meter the apparatus was designed to enable the flow-meter to be installed into the system for the duration of the calibration period and then removed. Installation involved moving the upper tank on its supporting frame back 1.6 metres and inserting the flowmeter assembly into the pipeline. This, of course, resulted in the end of the channel being 1.6 metres short of the lower reservoir,and so a temporary channel extension complete with splash boards was bolted to the end of the test channel for the calibration exercise.

The sample section of the channel was filled with a wooden block while the calibration exercise took place. The test apparatus is shown in Plate 5.2 and the flow-meter and associated electronics are shown in Plate 5.3, Plate 5.4 and Plate 5.5.

Calibration was achieved by varying the flow by use of the control valve to obtain pre-determined values of frequency of rotation of the turbine,which in turn corresponded to known values of discharge at a known head of water and at the same time recording the pressure head, as shown by the manometers connected across the orifice plate.

The advantages of the turbine flow-meter for calibration are:

1. There is only one moving part, and therefore the instrument is comparatively rugged.
2. It is simple to instal in any system.
3. High degree of accuracy - better than ±0.15%.
4. Excellent reproducibility - better than ±0.05%.
5. Low cost.

The turbine flow-meter can measure discharge of fluids over a wide range of flow-rates. The rotation of the turbine is detected by a pick-up attached to the side of the flow-meter, and the resulting recorded frequency of rotation is directly proportional to the fluid velocity. The pick-up is separated from the fluid by the body of the flow-meter which is locally reduced in thickness, thus allowing high internal fluid pressures without the problem of providing a reliable seal.

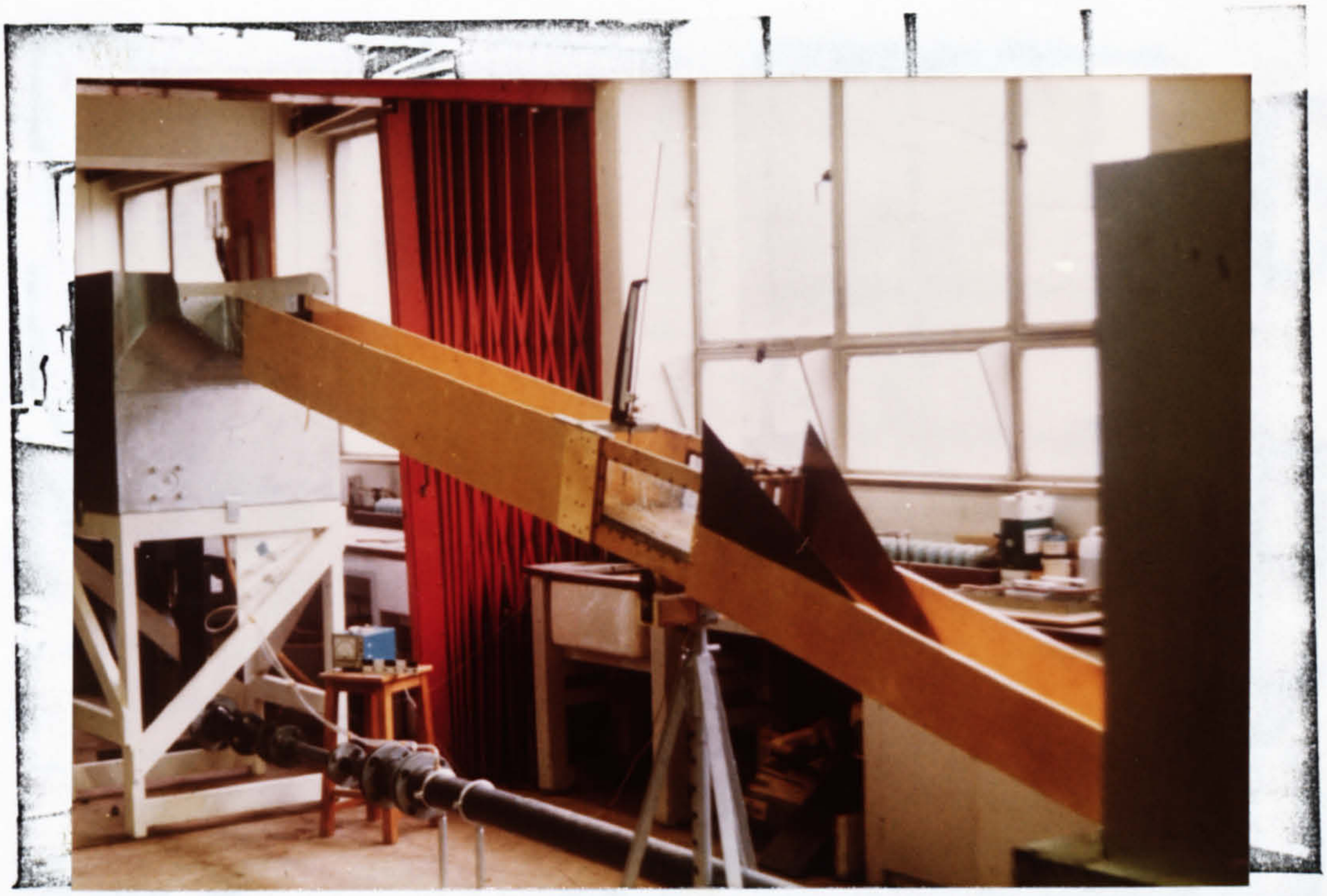


PLATE 5.2: Calibration of Channel

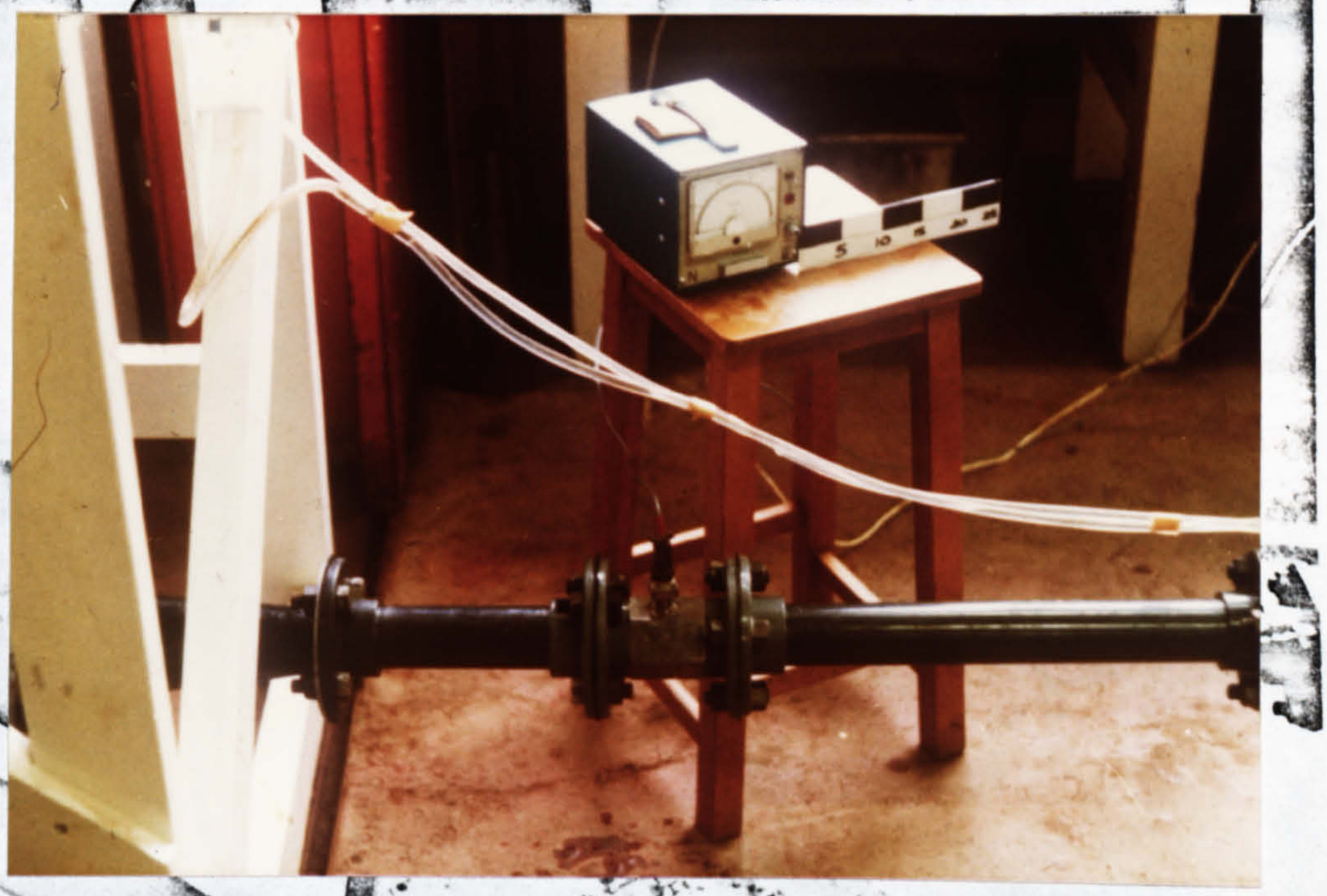


PLATE 5.3: Turbine Flow-meter

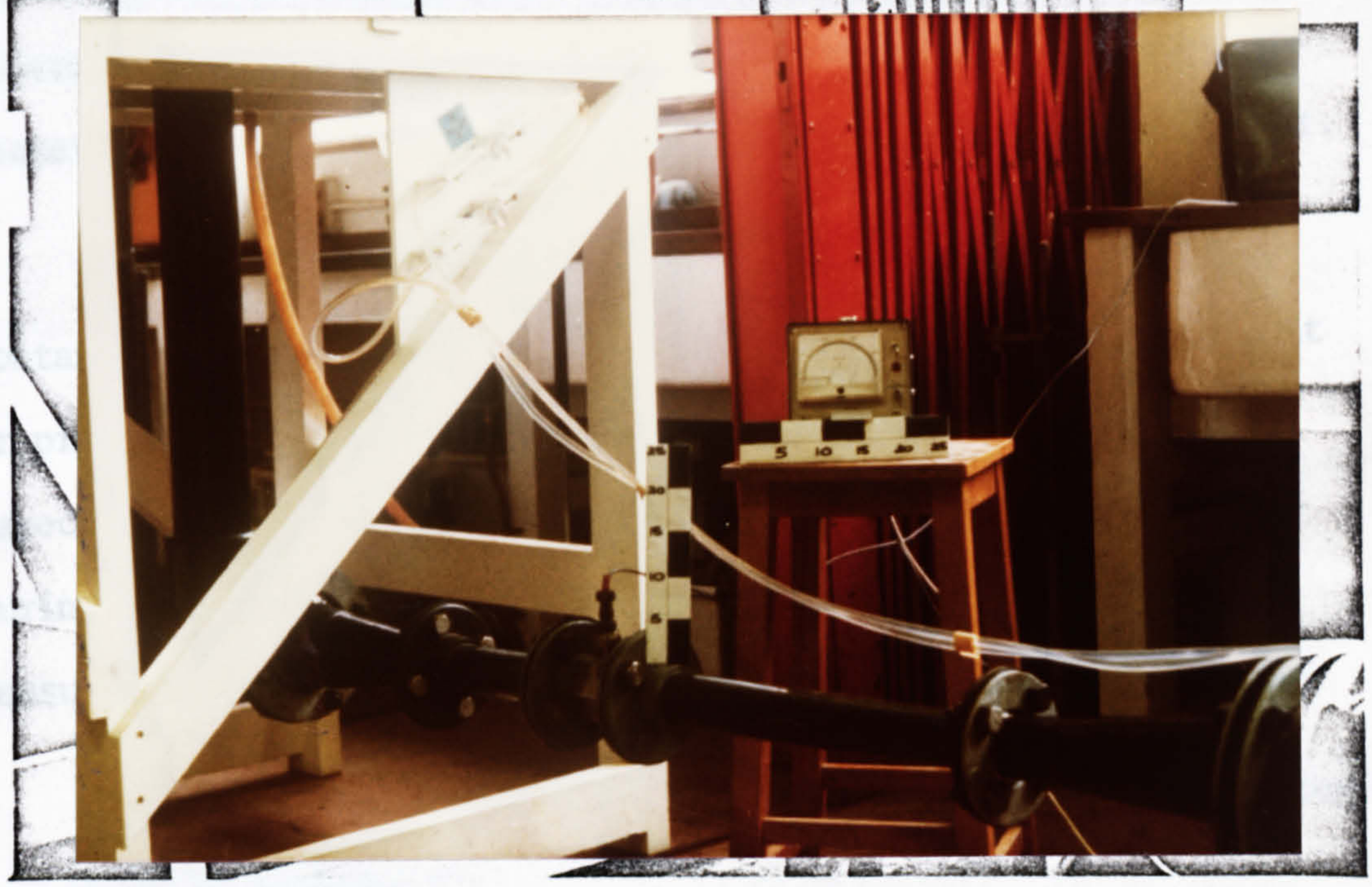


PLATE 5.4: Turbine Flow-meter

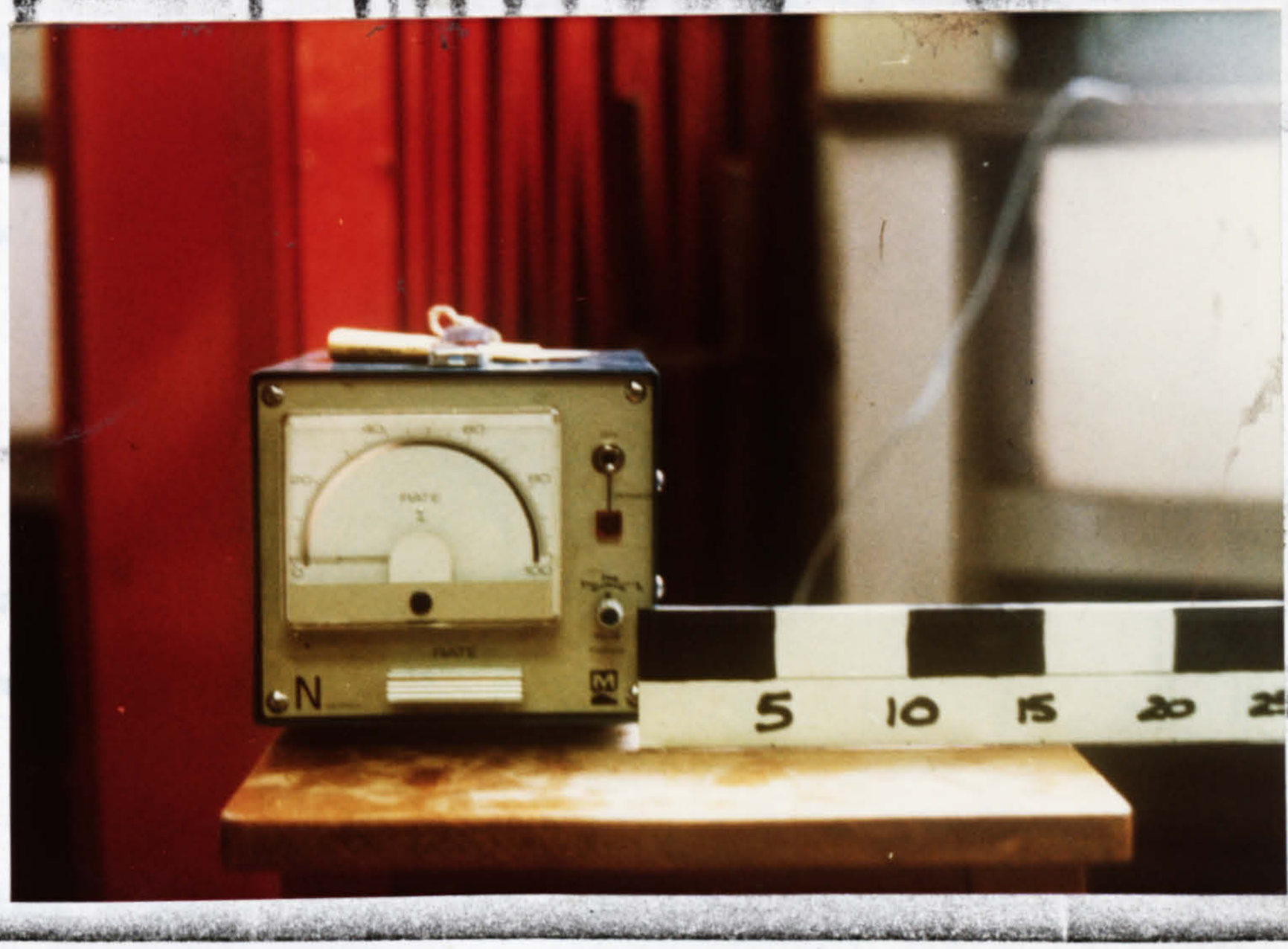


PLATE 5.5: Flow-meter Revolution Counter

The attitude of the meter has no effect on its accuracy, although flow in one direction only is permitted. The flow itself must not be swirling, and the manufacturer recommends that there are no obstructions within ten pipe diameters upstream and five pipe diameters downstream of the instrument. This recommendation was met.

The flow-meter used was capable of measuring flow rates in contaminated fluids such as raw water, crude oil, or where sediment or pipe scale might have been present, and this fact proved to be especially useful when checks were made on the orifice calibration during the laboratory programme. The M6 model used was capable of measuring a flow-range of 2.3 to 1136.5 litres/second.

The theoretical and observed discharge/manometer difference curves are shown in Appendix C.

Plastic tubing and 'two-way' glass taps led from D and D/2 brass manometer tappings in the pipe to a water and a mercury manometer mounted on the side of the upper tank, the water manometer being used for the lower flow-rates while the mercury manometer was used for high flow-rates.

5.5 Water Depth Gauge

The depth of flow was obtained using a point gauge held on a framework supported by the side walls of the channel which could be moved along the channel.

The point was mounted on a graduated scale and actuated by a slow-motion screw. A vernier scale enabled readings accurate to 0.01 millimetre to be made.

5.6 Pitot Tube

The velocity distribution in the channel was obtained using a pitot tube of the Prandtl type and using the basic equation:

$$v = \alpha \sqrt{2g \frac{P_t - P_s}{\gamma_w}}$$

where α - is usually taken as 1

v = velocity at any point in the channel

P_t = the total pressure indicated by the connecting manometer

P_s = the static pressure on the same manometer

γ_w = the specific gravity of water.

The mouth of the pitot tube inserted in the water had a diameter of one millimetre.

The pitot tube was mounted in a clamp attached to the point gauge support, enabling accurate positioning of the pitot tube and accurate determination of the depth of the mouth of the pitot tube. Frequent back-flushing was necessary in order to wash out sediment and air at the high points of the system.

5.7 Shear Vane

A portable hand shear vane manufactured by Pilcon Construction Limited was used to measure in-situ the shear strength of the bed material.

To measure the shear strength of the bed material the vane was pushed into the soil to the desired depth at the required position and then the handle turned until failure occurred. The angle turned by the handle is recorded on a scale within the instrument, and by use of a

calibration curve, which is given in Appendix C, the shear strength of the bed material can be found. The diameter of the vane (D) was 20 millimetres and the height of the vane (H) was 30 millimetres, the ratio of the height to diameter $(\frac{H}{D})$ being 1.5.

To ensure that the base of the channel did not affect the results of the shear vane tests some readings were taken with the flanges of the vane totally inserted into the soil, while in others readings were taken with the flanges only partially inserted into the soil.

Account was taken of this on the basis of the following theory:

First case: When the vane flanges are totally inserted in the soil.

If it is assumed that the soil fails over the surface of a cylinder, diameter D and height H, and that the shear stress is uniformly distributed:

The torque resisted by the curved surface at failure is:

$$\text{Torque} = \text{cohesion} \times \text{area} \times \text{moment arm}$$

$$= c\pi DH \frac{D}{2} = c\pi \frac{D^2H}{2}$$

Torque resisted by the ends (uniform stress) is

$$\frac{c\pi D^3}{6}$$

Hence total resisting torque is

$$\underline{\underline{\text{Torque} = c\pi(\frac{D^2H}{2} + \frac{D^3}{6})}}$$

Second case: When the vane flanges are only partially inserted in the soil,

i.e. H^* = inserted depth which is less than H

H = vane height.

$$\text{Torque} = c\pi DH^* \frac{D}{2} + c\pi \frac{D^2}{4} \frac{2}{3} \frac{D}{2}$$

$$= c\pi \frac{D^2 H^*}{2} + \frac{c\pi D^3}{12}$$

$$\underline{\underline{\text{Torque} = \frac{c\pi D^2}{6} (3H^* + \frac{D}{2})}}$$

5.8 Suspended Solid Concentration Measurement

Sampling of the water at specific times was achieved by collecting all the flow at the end of the channel by moving a perspex sampling box 150 millimetres wide, 250 millimetres long and 120 millimetres deep momentarily across the end of the channel, thus capturing the total flow. This sample was then continuously agitated and a 500 millilitre sub-sample obtained by filling a graduated measuring cylinder.

The sample was then filtered through Whatman GF/C Glass Microfibre filter papers of 110 millimetre diameter. The filter papers were oven dried overnight and then weighed on a balance, sensitive to 0.001 grams. They were then placed in funnels on Buchner flasks and a vacuum applied. The 500 millilitre sample was then added and filtration began. The residue was washed with distilled water. When filtration was complete the vacuum was released, the filter paper removed and dried overnight. The papers and sediment were then weighed on a balance, and

initially measuring time was kept to a minimum in order to reduce error due to varying moisture absorption during weighing. However, an experiment carried out showed that over a range of temperatures from 18°C to 25°C and over a wide range of humidity conditions the filter papers had a very high degree of weight stability. The papers used were chosen because they were made of ultra-fine fibres to retain the finest particles, in fact they are described as 'the standard paper for suspended solids in water'. Made from boro-silicate micro-fibres with no binders or additives to affect results, they are chemically and physically inert.

The "threshold" concentration of suspended solids was obtained before the start of the test. Thus the amount of suspended matter could be determined in grams per litre of the flowing water at each particular sampling time.

5.9 Swelling Characteristic Apparatus

The apparatus used to evaluate the swelling characteristics of the bed material is shown in Figure 5.5 and Plate 5.6.

The apparatus consists of a 100 millimetre diameter perspex tube with detachable base containing a porous disc, and a perforated circular aluminium plate with fixed vertical aluminium rods weighing 56 grams, which transmit any movement of the sample to dial gauges mounted on top of the apparatus.

The material and overlying aluminium plate were covered with water to a depth of 25 millimetres.

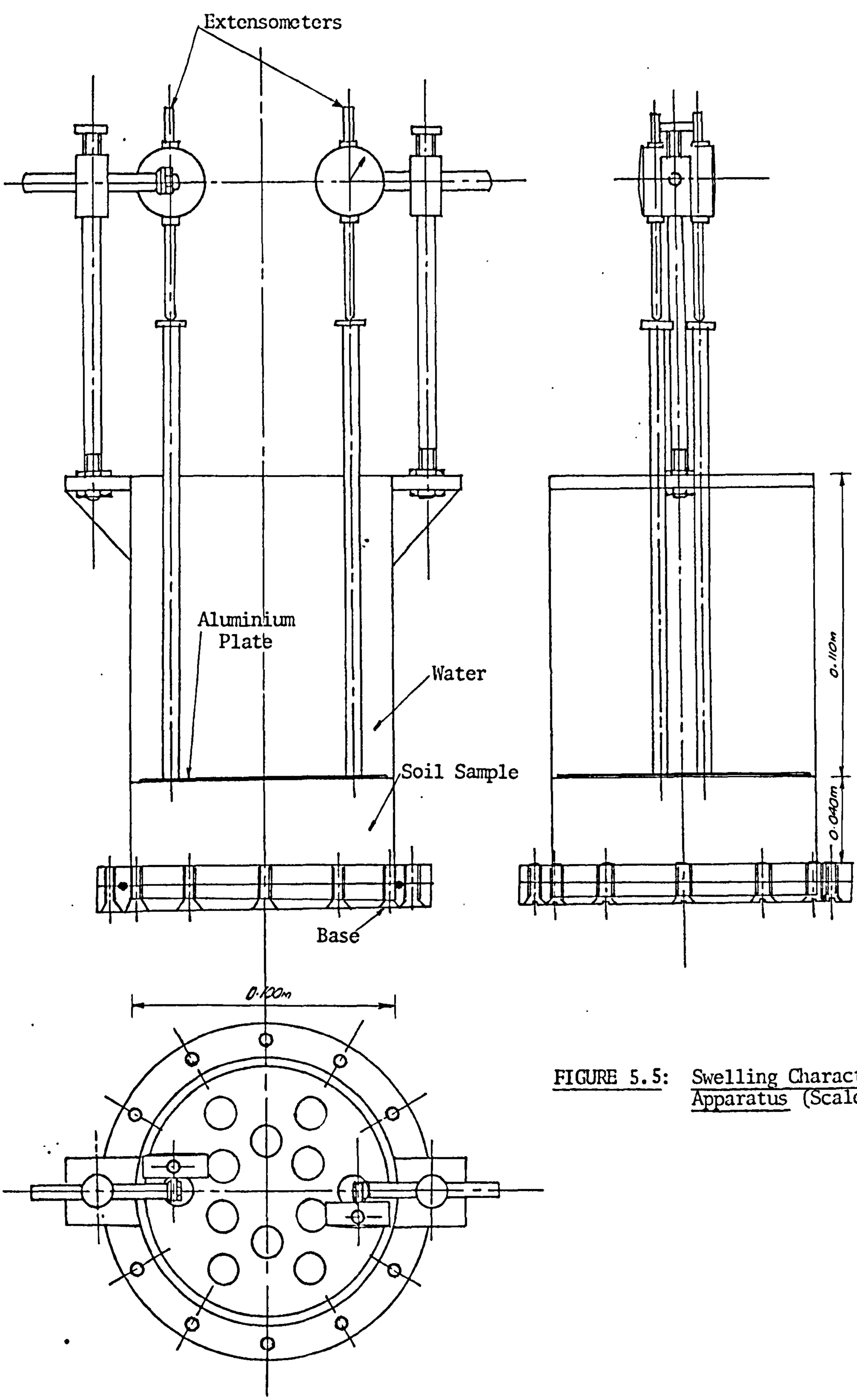


FIGURE 5.5: Swelling Characteristic Apparatus (Scale 1:2)

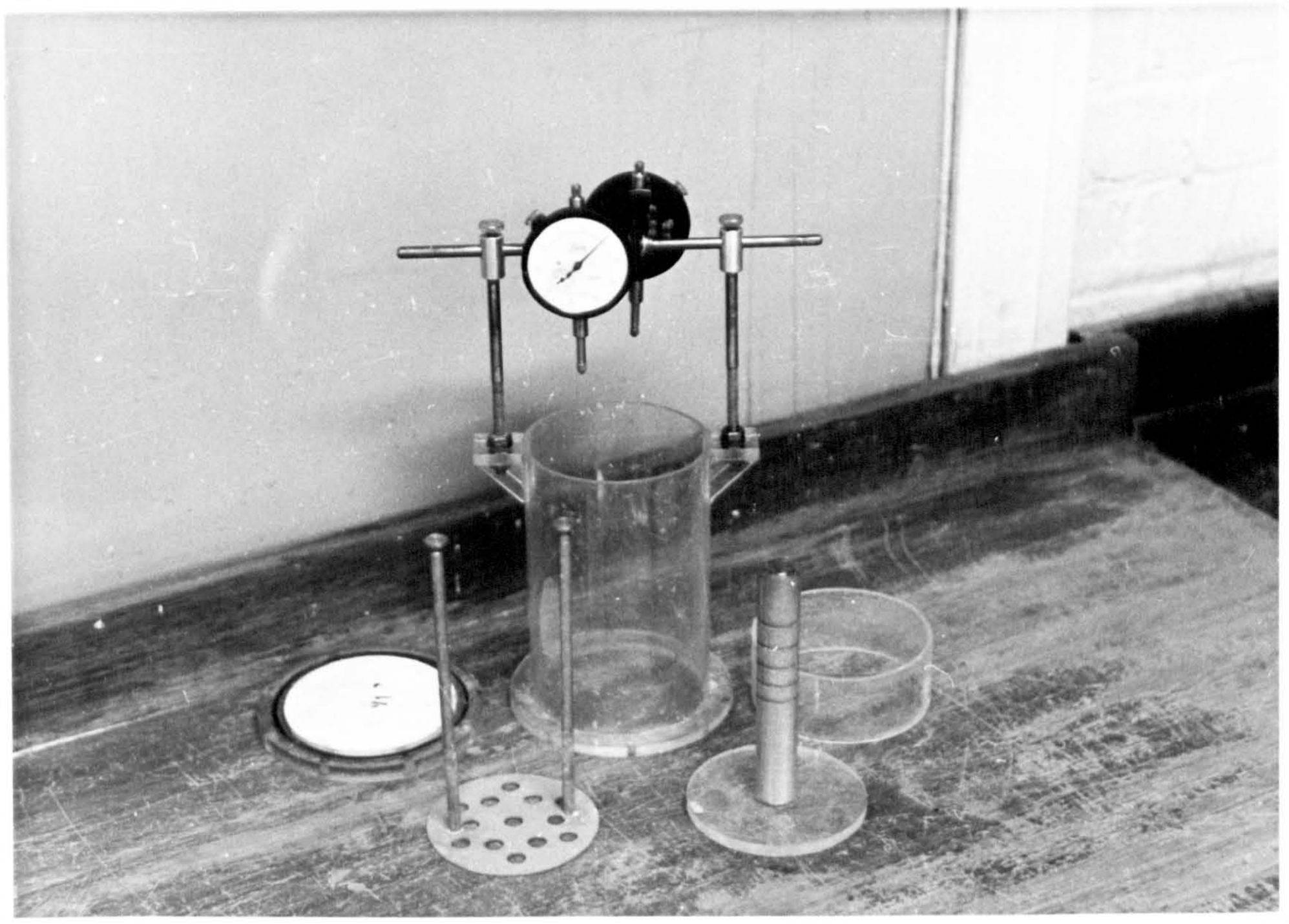


PLATE 5.6: Swelling Characteristic Apparatus

5.10 Recording/Photographic Apparatus

In addition to the results obtained from the test photographic records of the tests were made using a single lens reflex camera and a time lapse camera, the latter being shown in Plate 5.7.

5.11 Clay Material

5.11.1 Classification

The clay soil used in the laboratory investigation was classified as being a clay of intermediate plasticity, falling between the CL and CH groups in the Casagrande system of classification.

This dark brown clay, a typical North-East, overconsolidated glacial boulder clay, was sampled from the Whitley Bay area of Tyneside. Approximately two cubic metres of material was dug by hand and returned to the laboratory where it was air-dried, all stones removed, and then broken down to a powder whose particle size did not exceed three millimetres using a 'Fritsch Pulverisette' crusher. The material was then stored in covered plastic bins, under laboratory conditions of between 14°C and 24°C, and at a humidity of approximately 55%, until required.

5.11.2 Grain-Size Distribution

The grain-size distribution was determined by pipette analysis, as described in BS 1377:1975[176] and showed the material to be composed of approximately 29 per cent by weight of clay (less than 2 microns in size), 40 per cent by weight of silt (between 2 microns and 60 microns) and a small amount of fine sand. The particle size distribution is shown graphically in Figure 5.6 showing, perhaps surprisingly, very little fine and medium silt to be present.

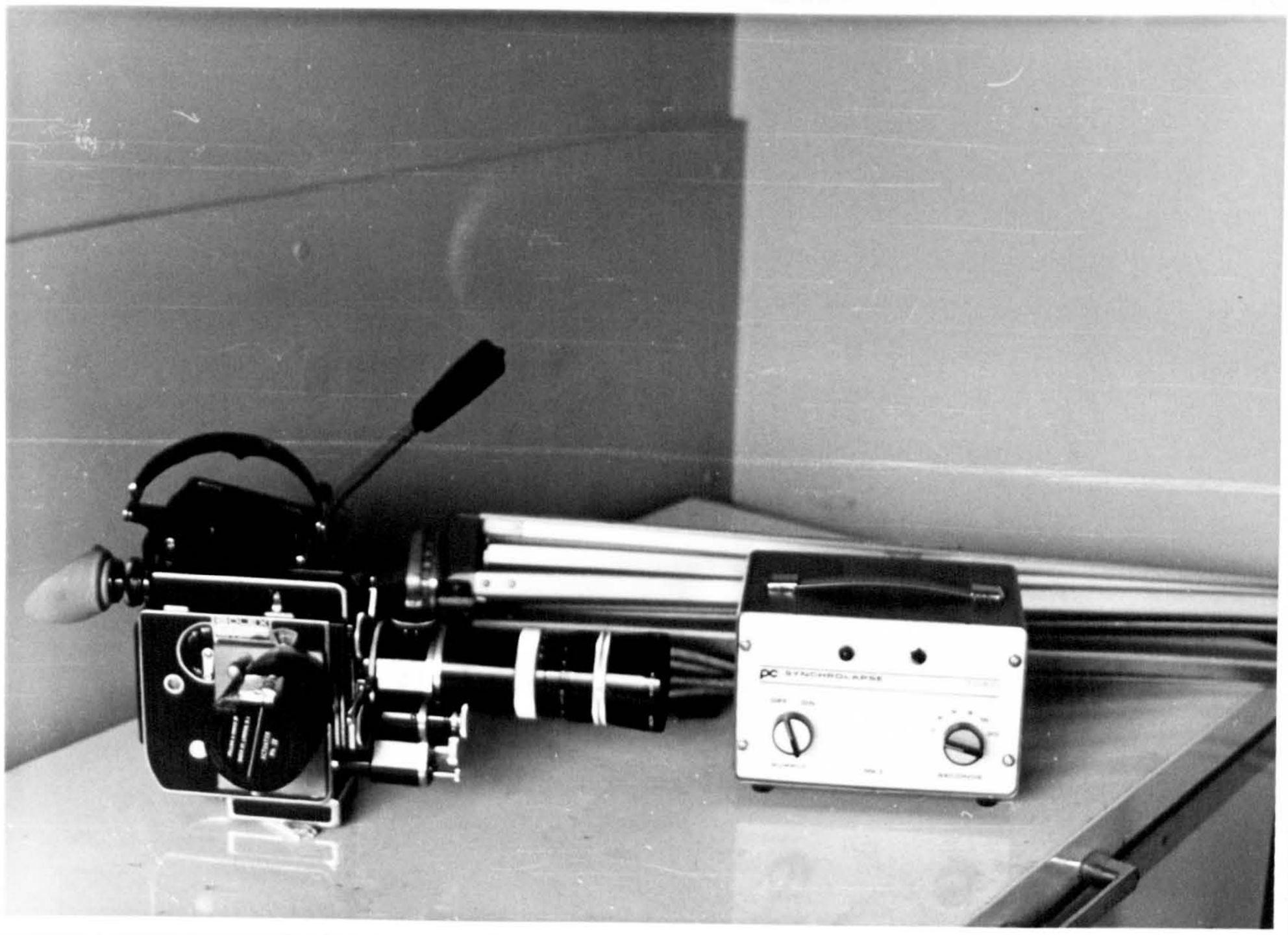


PLATE 5.7: Time-Lapse Photography Equipment

5.11.3 Mineralogy

The mineralogy of the soil was investigated using X-ray diffraction techniques (XRD) from which it was found that the clay fraction was composed of several clay minerals which are presented in Table 5.3.

Mineral	Percentage by Weight
Albite	5.4
Chlorite	6.6
Illite	2.6
Muscovite	1.3
Orthoclase Felspar	8.5
Quartz	75.6

TABLE 5.3: Mineralogical Analysis of Clay Fraction

Some clay particles are shown in Plate 5.8.

5.11.4 Atterberg Limits

The Liquid Limit was determined using the cone penetrometer method as described in BS 1377:1975[176]. The Atterberg Limits for the clay material are shown in Table 5.4.

Liquid Limit	39
Plastic Limit	18
Plasticity Index	21

TABLE 5.4: Atterberg Limits

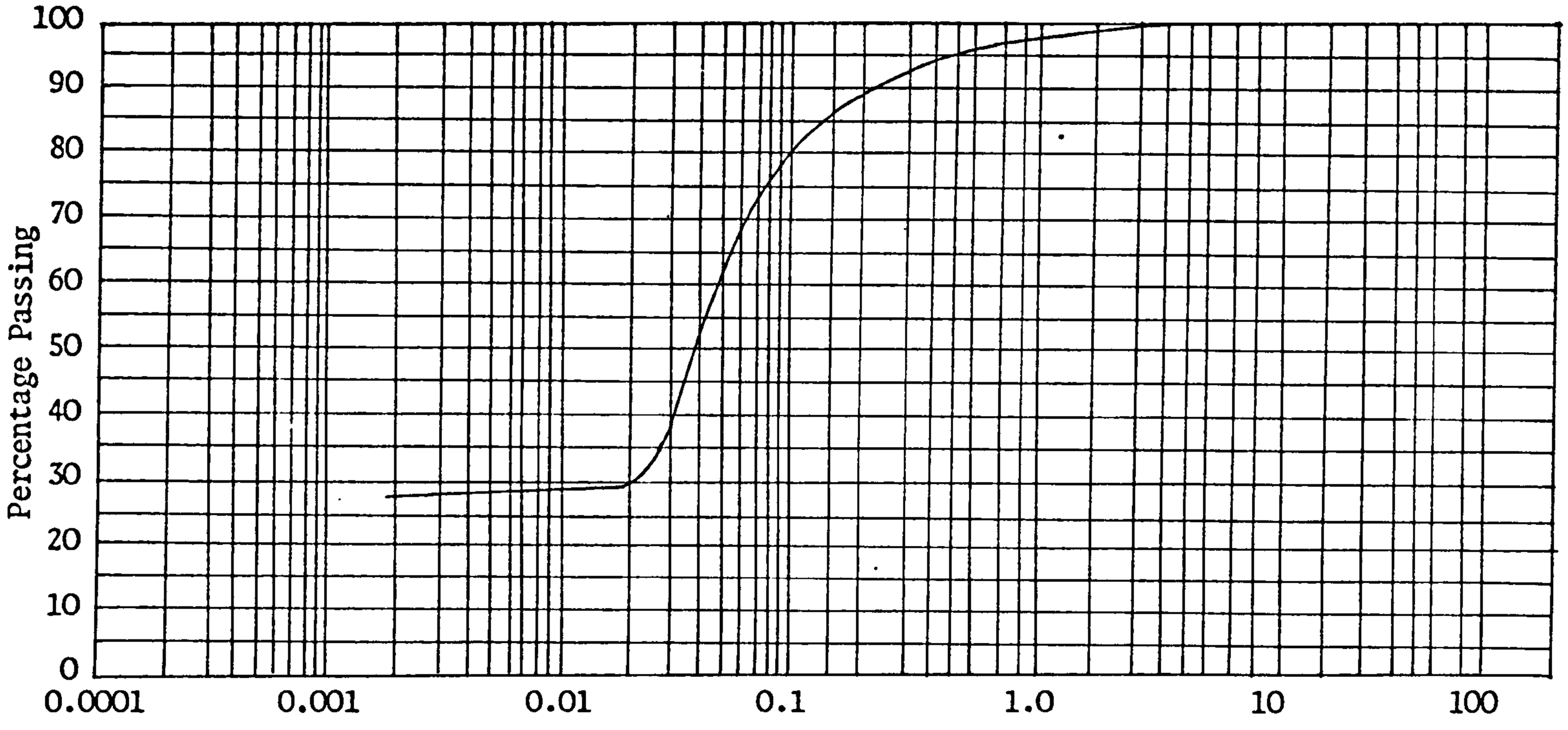
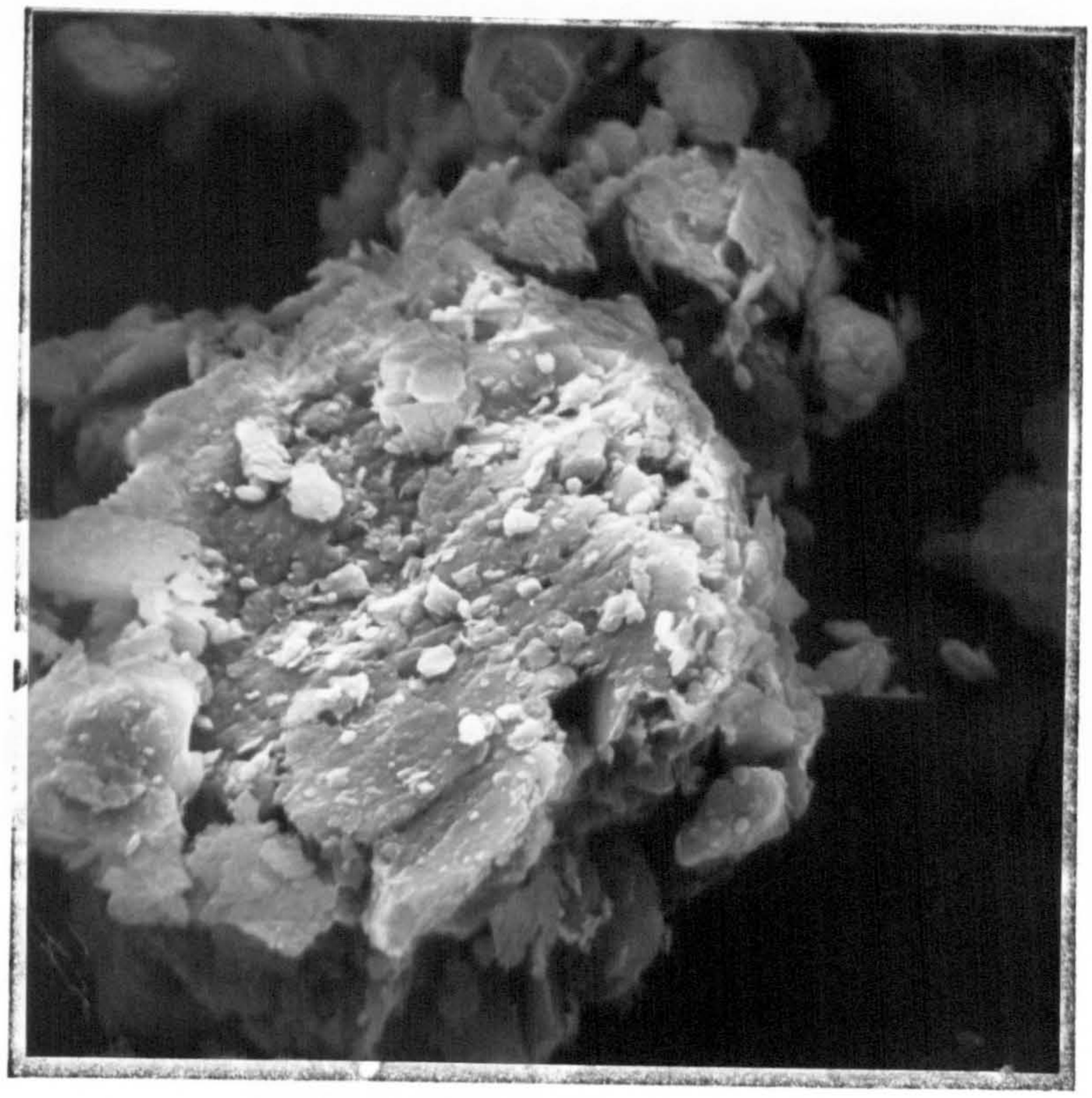
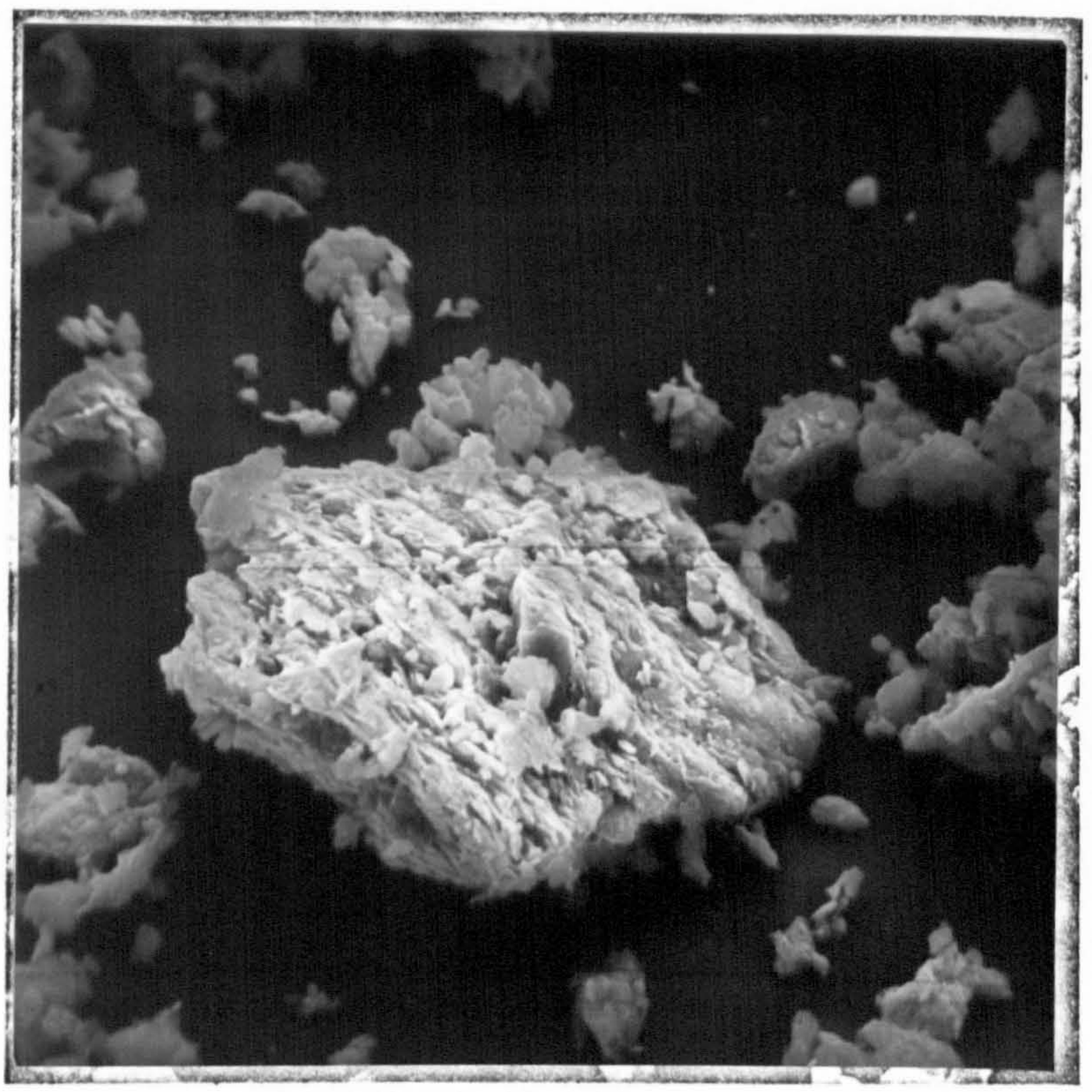


FIGURE 5.6: Particle Size Distribution for Clay Material

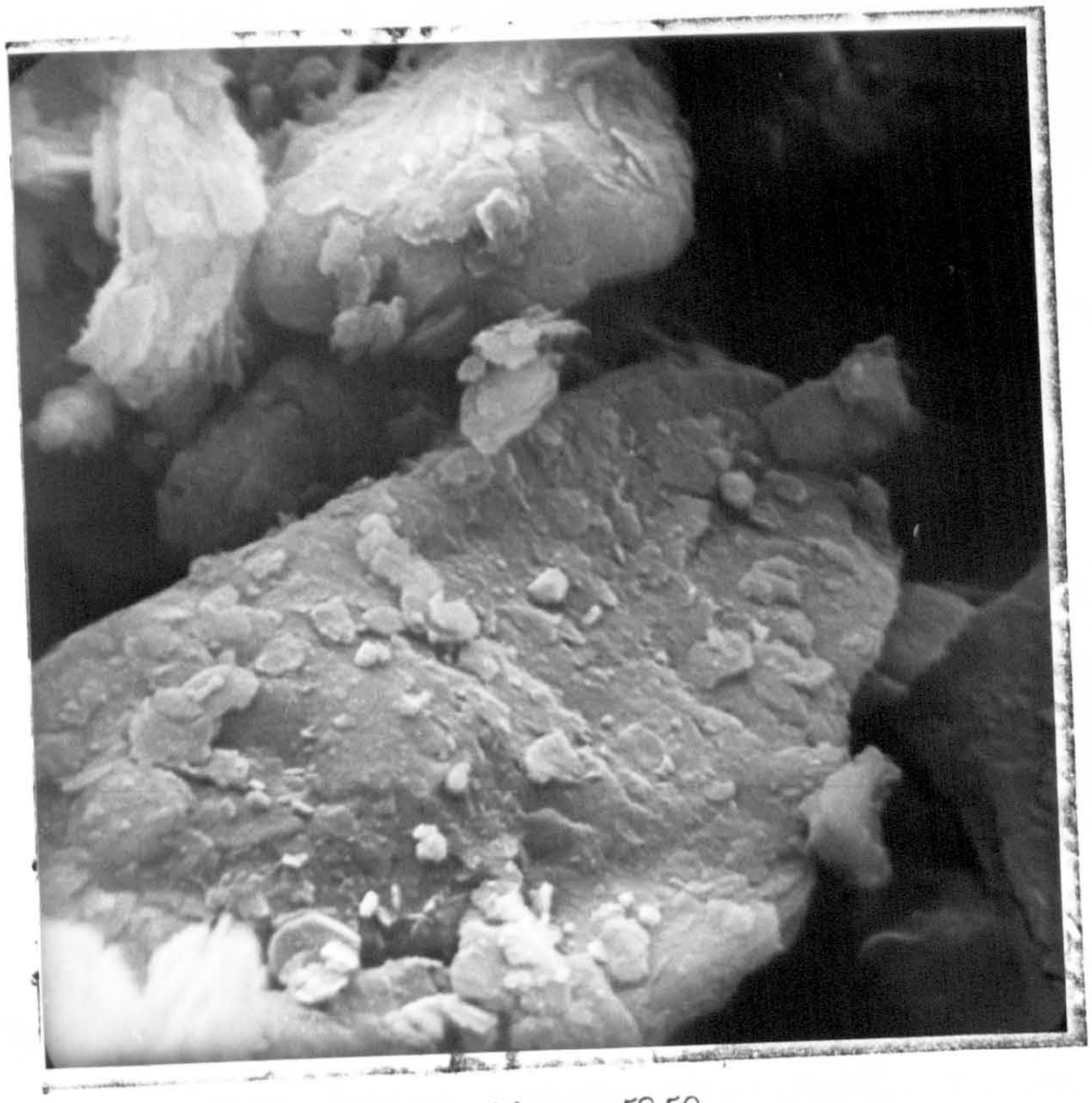


Magnificationx 2080.

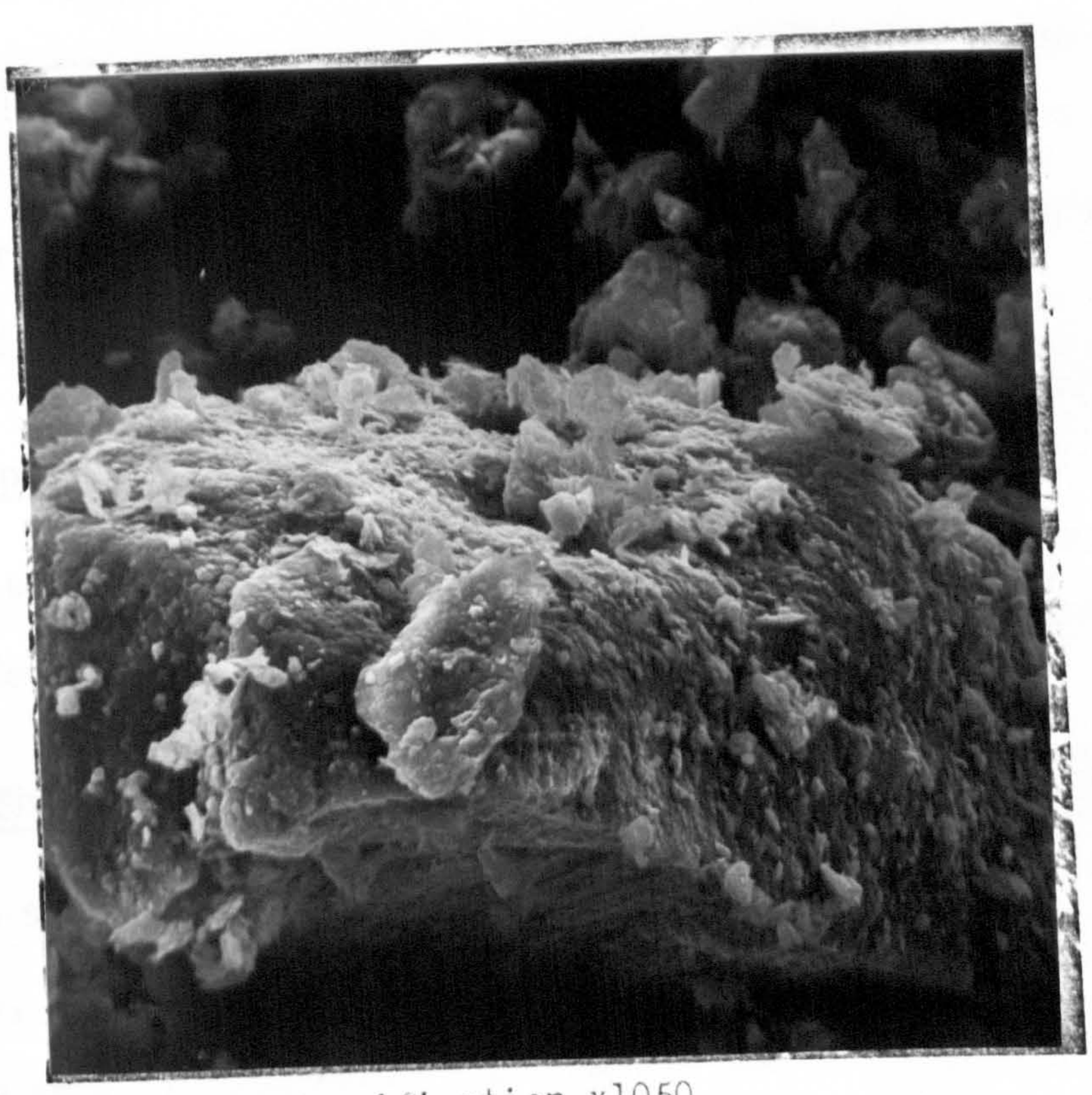


Magnification x940.

PLATE 5.8: Clay Particles



Magnification x5250



Magnification x1050.

PLATE 5.8(a): Clay Particles

5.11.5 Chemical Characteristics

The results of a 'water soluble' chemical analysis are summarised in Table 5.5.

Ca^{++}	0.64 mg per gram of wet weight
Mg^{++}	0.49 mg per gram of wet weight
Cl^{--}	0.12 mg per gram of wet weight
Mn^{--}	0.28 mg per gram of wet weight
Fe	2.00 mg per gram of wet weight

TABLE 5.5: Water Soluble Chemical Analysis

5.11.6 Soil Compaction Test

The dry density/moisture content relationship for the clay soil is shown in Figure 5.7. The test used a 2.5 kg rammer falling through a height of 300 millimetres, i.e. 'normal' compaction, as specified in "Methods of Test for Soils for Civil Engineering Purposes, BS 1377:1975"[176]. Maximum dry density achieved was 1.78 Mg/m^3 at an optimum moisture content of 16.5 per cent.

5.11.7 Specific Gravity

The specific gravity, G_s, of the clay particles was found to be 2.64[176] determined by the method outlined in BS 1377:1975[176].

5.11.8 Shear Strength - Undrained Triaxial Tests

The shear strength of the clay, at various densities and moisture contents, was investigated in a series of undrained triaxial tests. For each density, the amount of material required to form a 100 millimetre long, 38 millimetre diameter sample was calculated. The material was then forced into a 38 millimetre diameter sample tube and

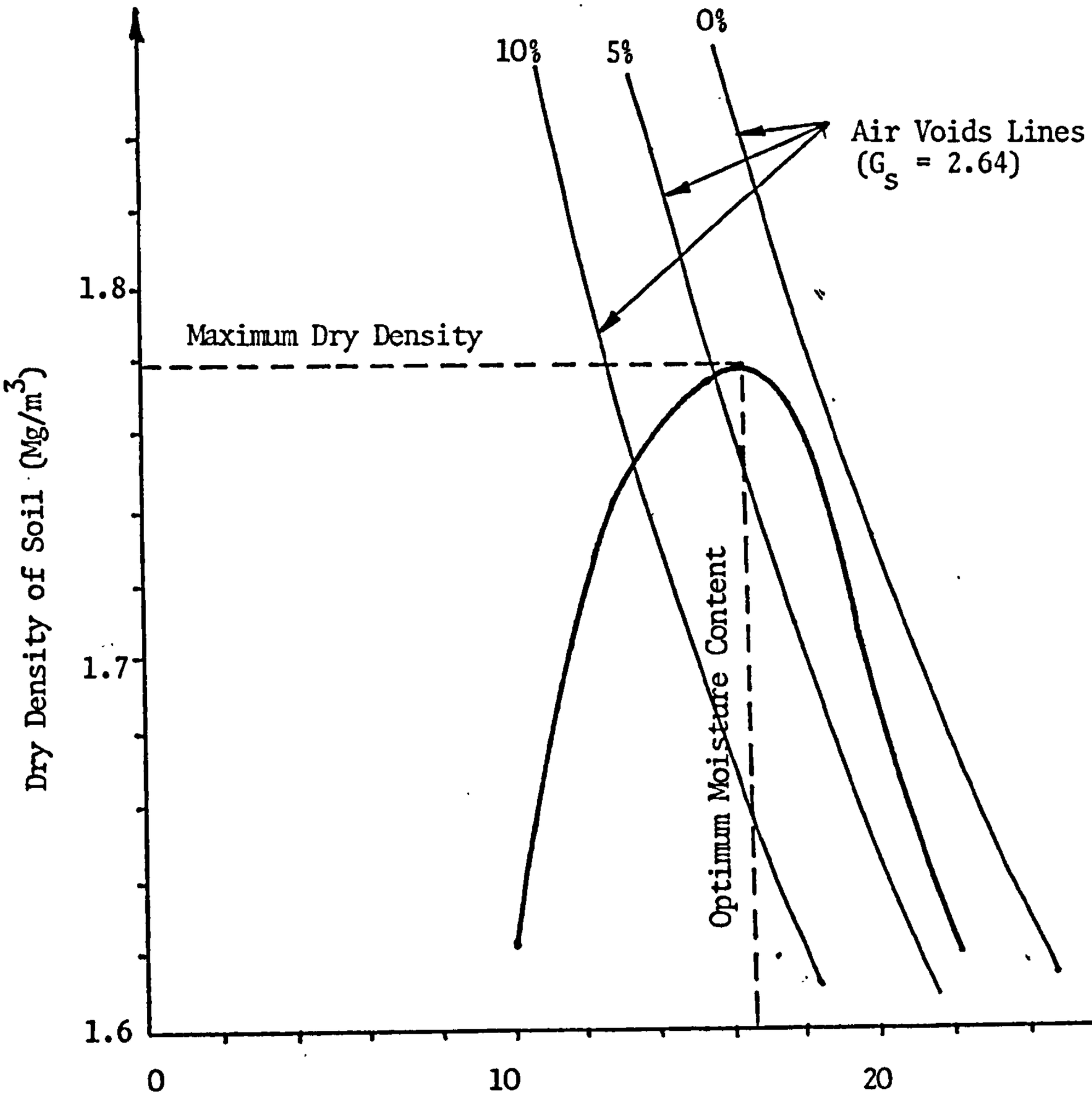


FIGURE 5.7: Dry Density/Moisture Content Relationship for Clay Soil Sample

a 100 millimetre specimen extruded which was then trimmed to give the 'standard' 76 millimetre long, 38 millimetre diameter sample.

Figure 5.8 shows the relationship between moisture content, cohesion and bulk density for the clay.

5.12 Modification of Soil Properties

In order to vary the soil properties it was decided rather than attempt to locate clays with widely varying properties, to add sand to the clay soil.

5.12.1 Sand Material

The sand added to the clay, was supplied by Tilcon Quarries Limited from their Styford Quarry. The properties of the sand are as shown in Table 5.6.

Sand was added in the proportions of 5%, 10%, 15%, 20% and 25% of the dry weight of the clay. The resulting changes in Liquid Limit and Plastic Limit are shown graphically in Figure 5.9 and Figure 5.10.

Variation in shear strength of the soil due to the sand was monitored by use of the shear vane.

Change in the soils' compaction characteristics was evaluated by testing samples to which 5%, 10%, 15%, 20% and 25% of sand, by weight of the original sample, had been added. The results of the compaction tests are shown in Figure 5.11. A gradual reduction in optimum moisture content is seen to occur with increasing amounts of sand.

Sieve Analysis - Size	Percentage passing
10 mm	100
5 mm	91.3
2.36 mm	71.7
1.18 mm	56.9
600 μm	41.5
300 μm	26.4
150 μm	10.6
Silt Content	6%
Organic Impurities	O
Water Absorbtion	6% of Dry Weight
Specific Gravity	
Saturated Surface Dry	2.65
Oven Dried	2.41
Apparent Relative Density	2.85
Density	
Compacted Density (kg/m^3)	$1641 > \gamma > 1613$
Loose Density (kg/m^3)	$1525 > \gamma > 1498$

TABLE 5.6: Sand Properties

FIGURE 5.8: Relationship between Cohesion, Moisture Content and Bulk Density for Clay Soil

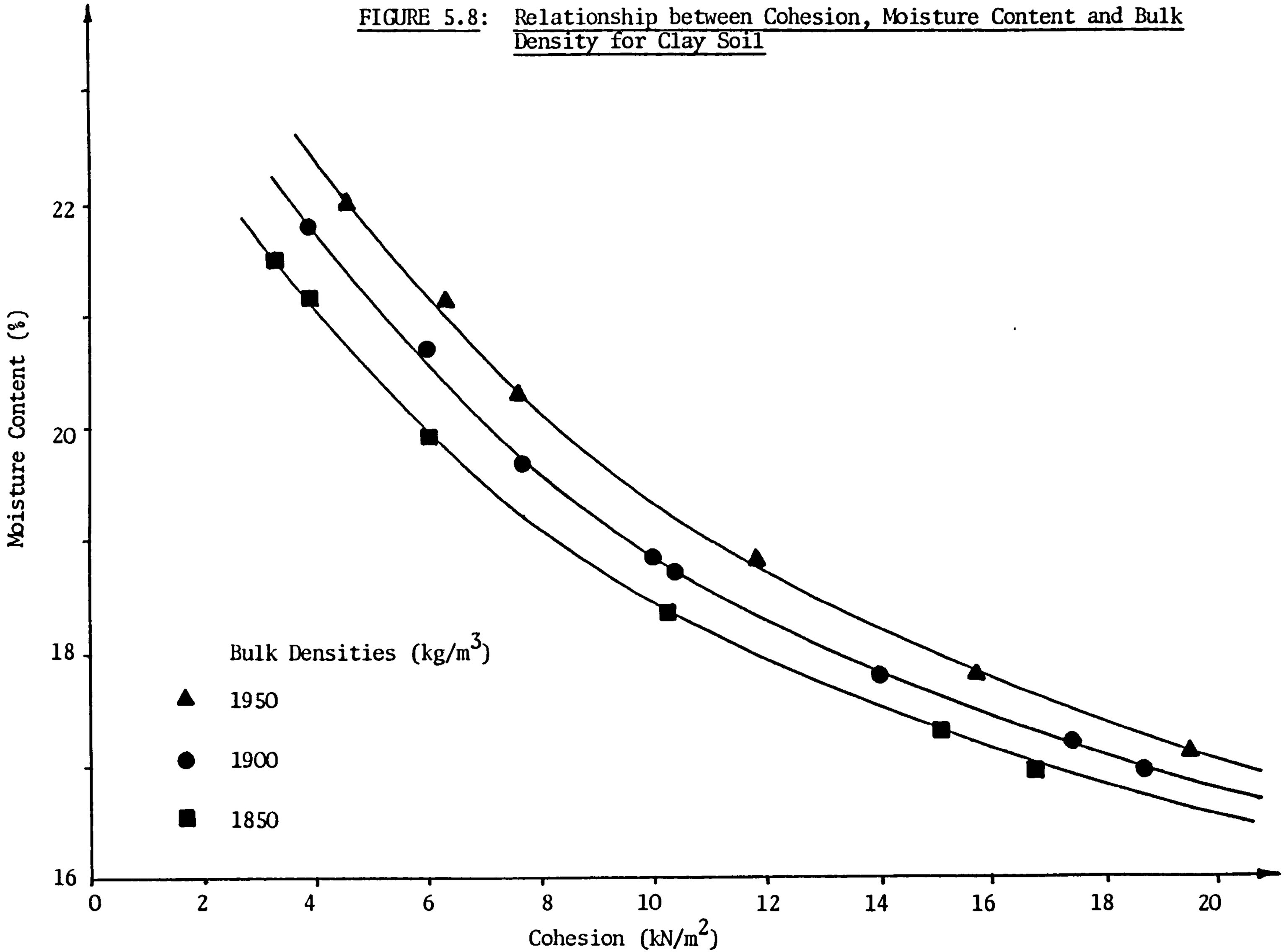
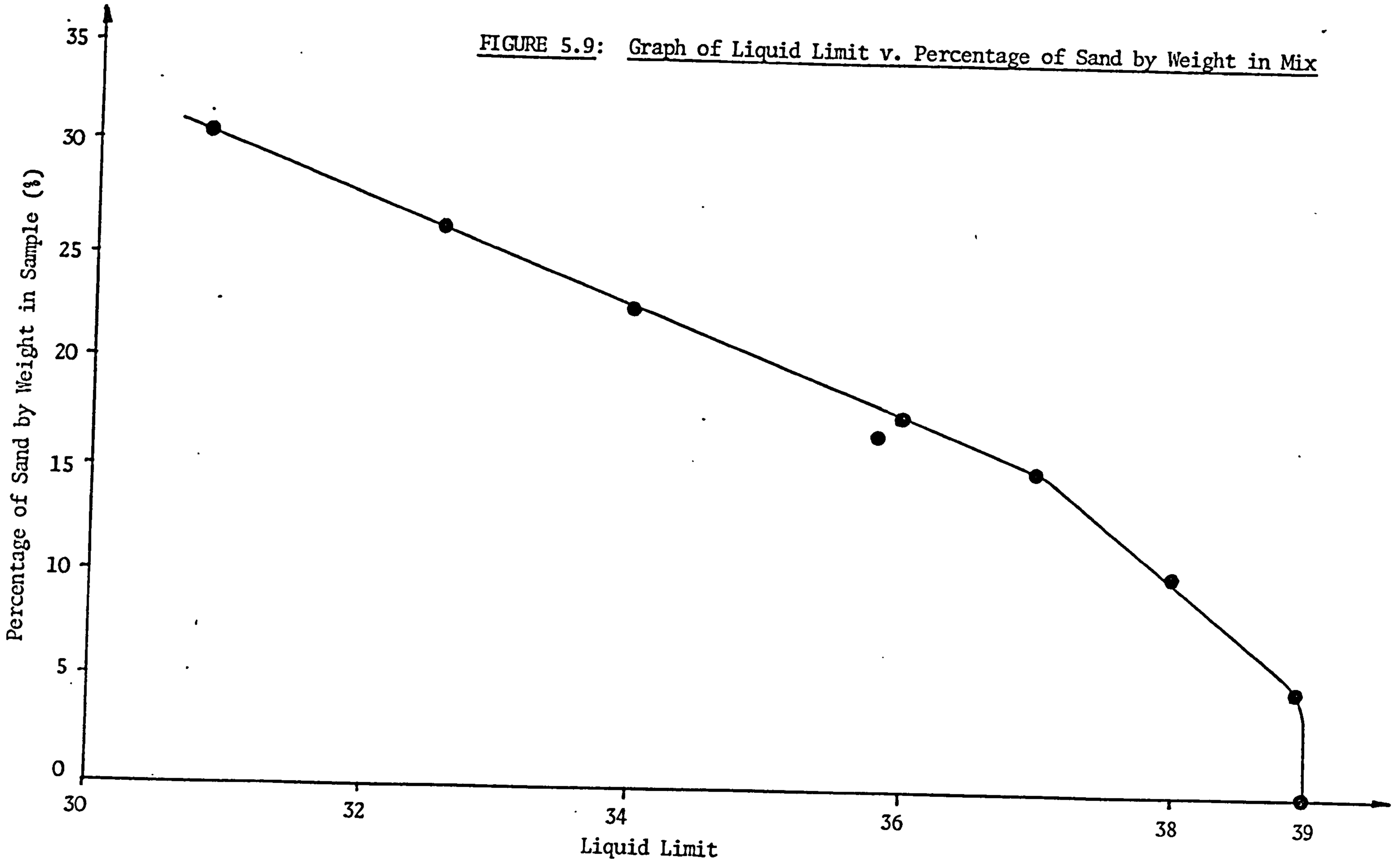


FIGURE 5.9: Graph of Liquid Limit v. Percentage of Sand by Weight in Mix



661 199

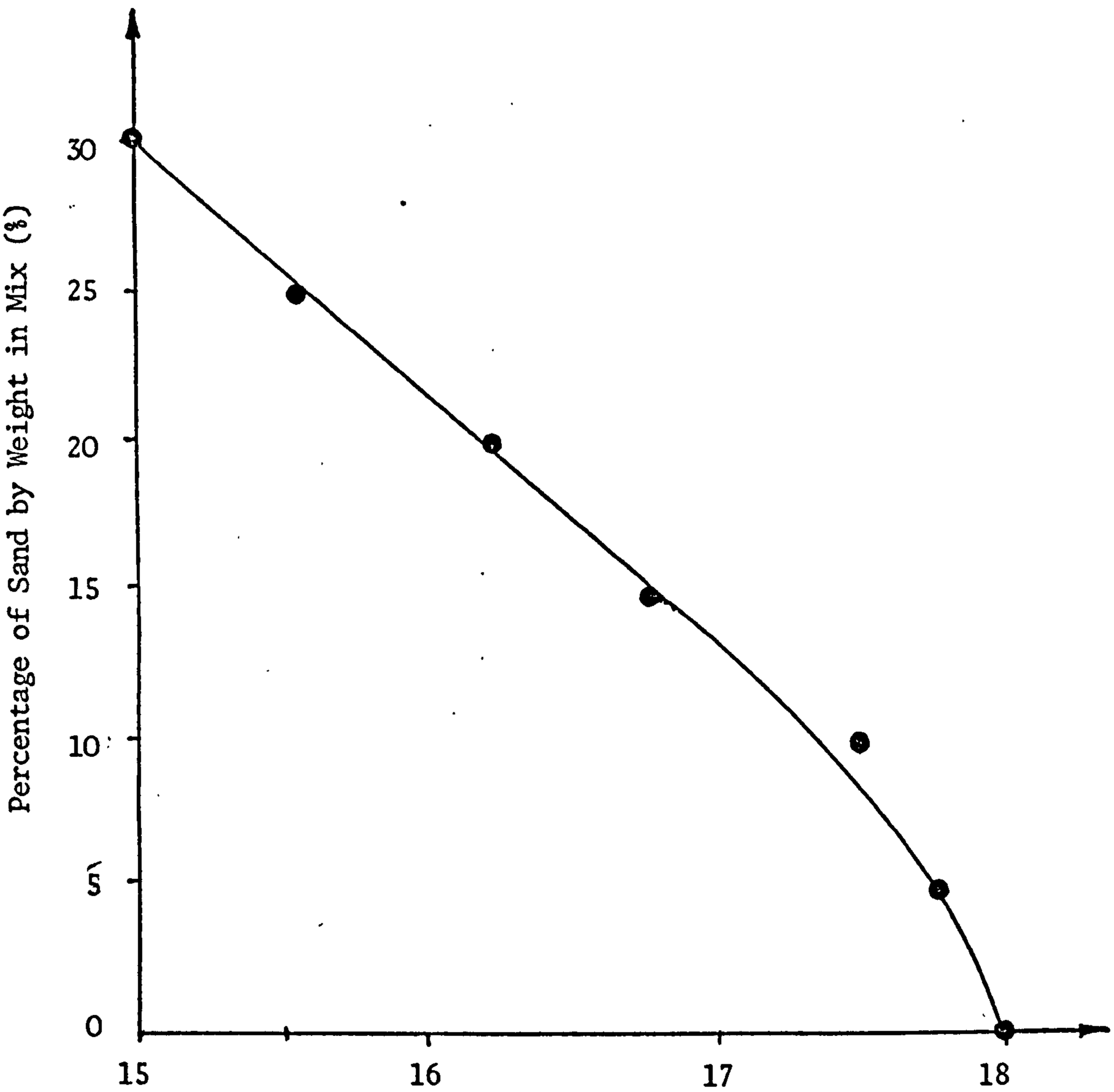


FIGURE 5.10: Graph Plastic Limit v. Percentage of Sand by Weight in Mix

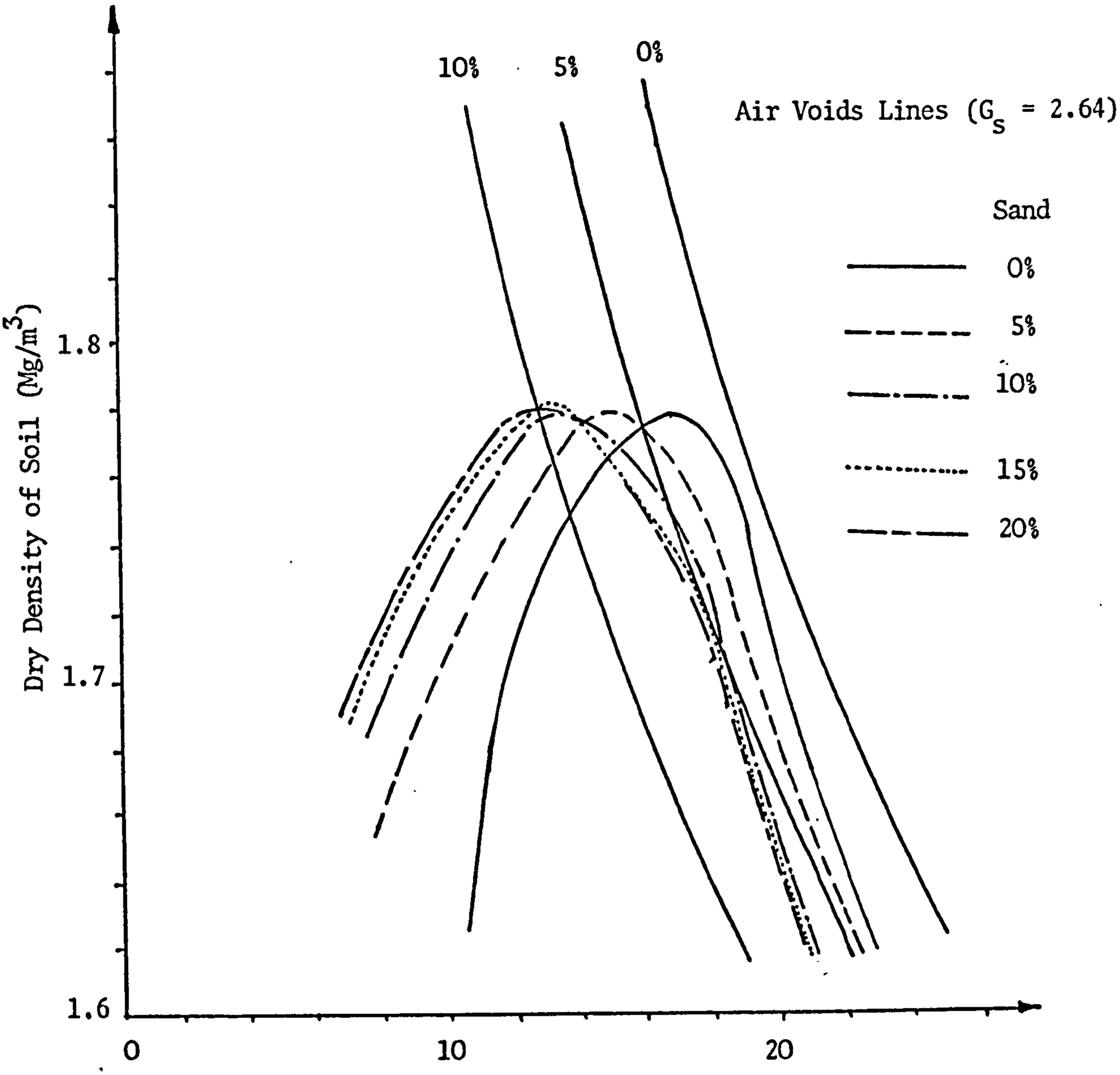


FIGURE 5.11: Dry Density/Moisture Content Relationship for the Clay Soil Sample with Various Percentages of Sand by Weight in the Mix

5.13 Development of an Experimental Programme

Primary objectives of the experimental programme were to investigate the washout process of earth dams, to study the mechanism of erosion, and to investigate how erosion rates may vary with change in water velocity and basic soil parameters, such as Plastic Limit, Liquid Limit comparison and shear strength. Dual series of experiments were devised in order that these objectives might be fulfilled.

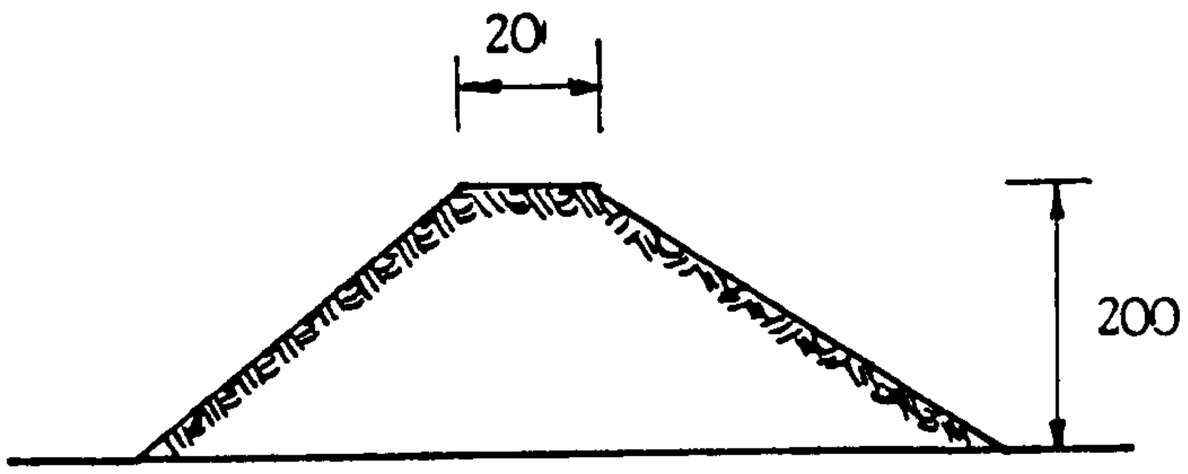
The first series of tests (Series I) involved compacted clay samples of which the properties could be varied, i.e. density, moisture content, Plastic Limit, Liquid Limit etc., 512 millimetres long, 250 millimetres wide and 40 millimetres deep which would fit into the 'sample section' of the channel flush with the base of the channel and which could then be subjected to erosion by the water flowing over the sample. A study of the effects of varying the soil parameters or erosion rates and a study of the mechanism could thus be made.

Parameters also to be considered in Series I tests were the effects of temperature and sediment concentration of the eroding waters.

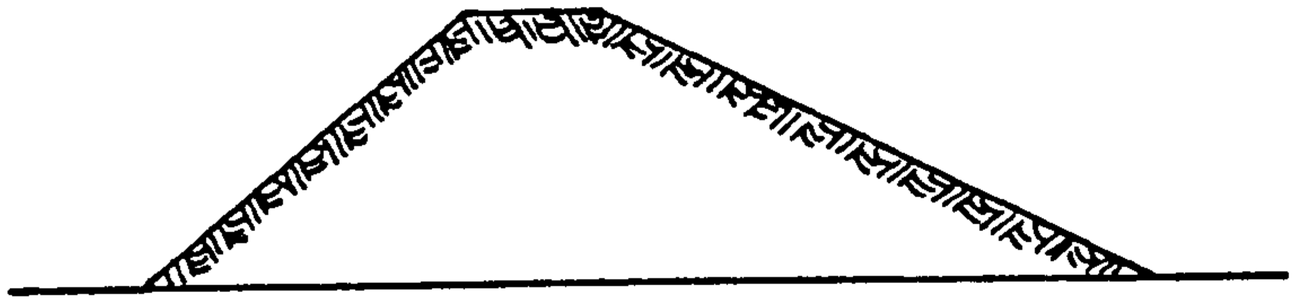
Small scale model tests (Series II) were made using models whose profiles are shown in Figure 5.12. This series of tests were used to study the effect of the orientation of compacted layers in the dam relative to the vertical axis of the dam, the effect of changes in the downstream slope, and also to study breach configurations, and the mechanism of failure of a dam subject to overtopping.

5.14 The Procedure of Work

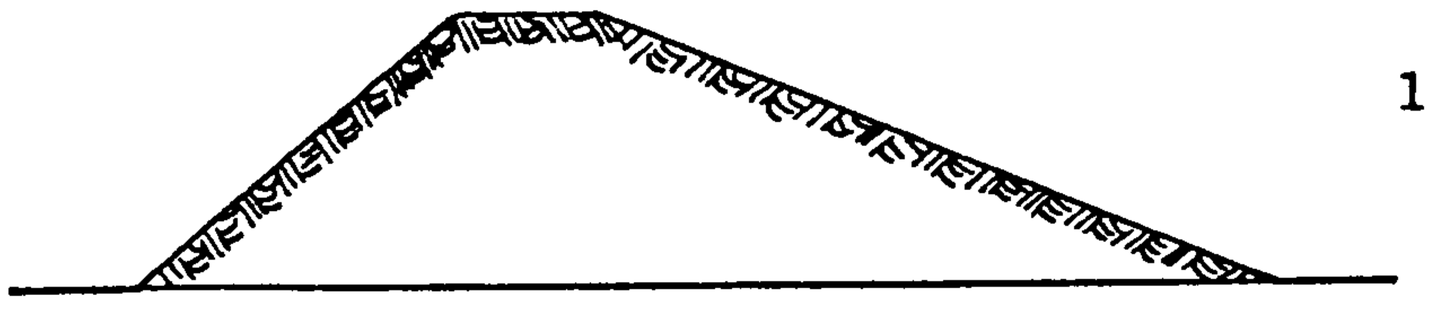
The following sections describe the preparation of each specimen, the measurements taken and, finally, the results from a typical test are presented.



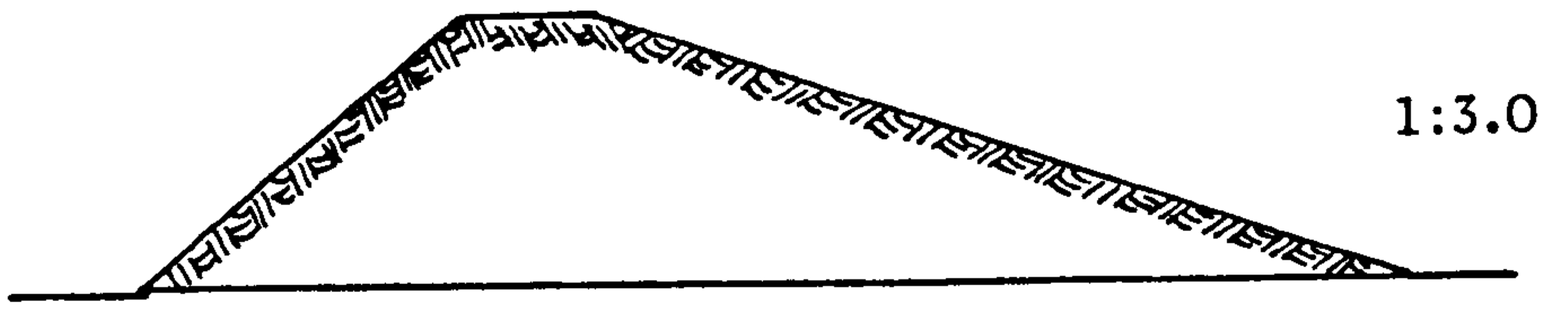
1:1.5 Downstream Slope



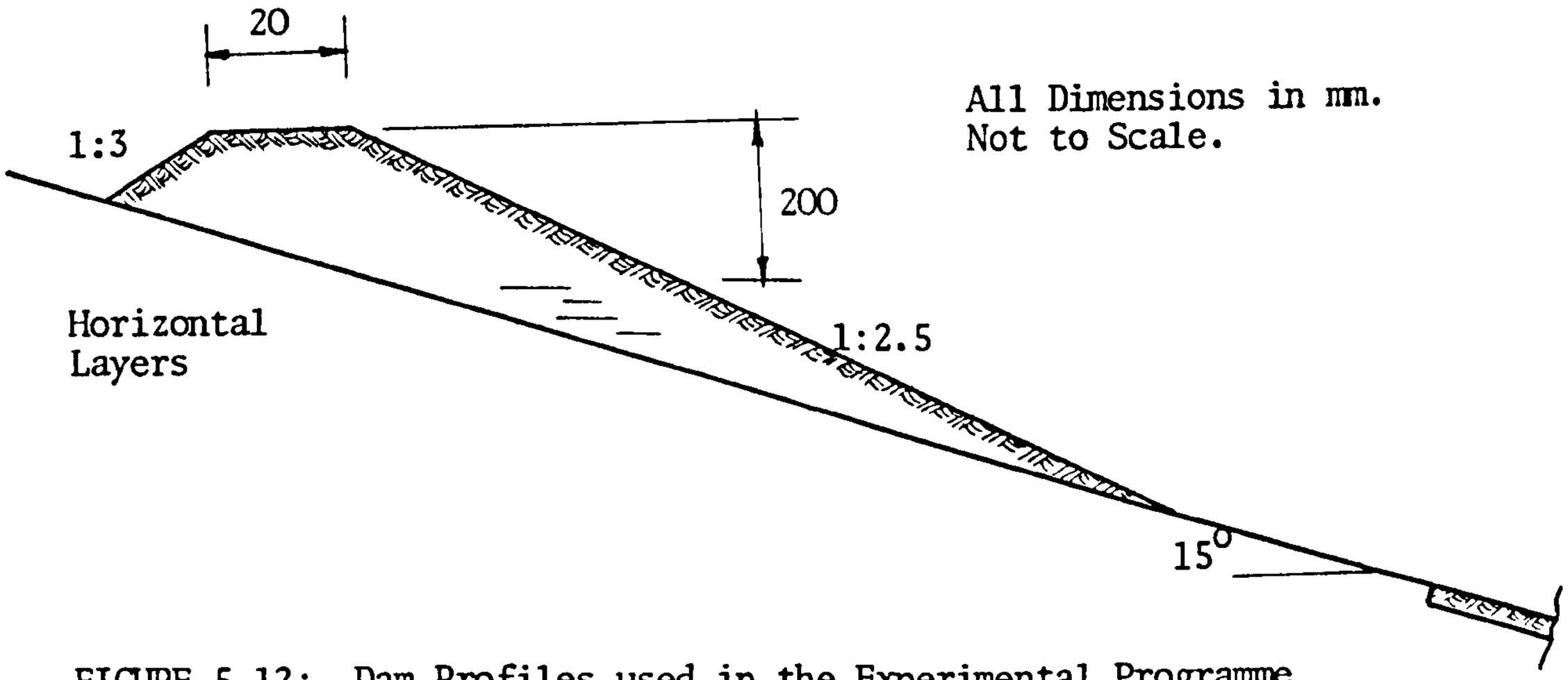
1:2.0 Downstream Slope



1:2.5 Downstream Slope



1:3.0 Downstream Slope



All Dimensions in mm.
Not to Scale.

FIGURE 5.12: Dam Profiles used in the Experimental Programme
Series II Trials

5.14.1 Preparation of the Experiments

Preparation for each experimental run included filling the test apparatus with water and expelling all air from the manometer system, and also the formation of a homogeneous cohesive sample with a smooth surface. The samples were prepared in four stages as follows:

(a) Mixing;

(b) Compaction;

(c) Laying out the model;

(d) Treatment of the surface.

5.14.2 Mixing

In this process the quantity of water required to produce the desired moisture content was calculated making allowance for the initial moisture content of the clay particles. Selection of the moisture contents to be used during the tests was based on the optimum moisture content obtained from the compaction tests. A range of moisture contents was chosen to encompass the optimum moisture content as found from the standard compaction test. The initial moisture content of the clay particles was measured in a 'Speedy' moisture meter. The clay was stored indoors under laboratory conditions of temperature and humidity ($\simeq 20^{o}C$, 60% RH), and, in the latter stages of the work, due to no change in the moisture content of the clay, the use of the 'Speedy' was discontinued.

The water required to produce the required moisture content was added to 10 kilograms of clay particles and mixed for three minutes in a 'Multiflow' pan-mixer of 0.015 m^3 ($\frac{1}{2}$ ft^3) capacity. The arrangement of the paddles in this mixer gave thorough mixing in the horizontal and vertical directions, although care had to be exercised, especially in the case of the 'drier' mixes, to ensure that material in the corner of

the pan was distributed through the mix.

A sample was removed from the pan immediately after mixing and an accurate determination of moisture content obtained by the method described in BS 1377:1975[176].

5.14.3 Compaction

Compaction of the clay sample, especially in the earlier tests, proved to be a problem. Several methods of compaction were tested before a method was chosen to be used throughout the laboratory programme. Due to the limited working space available within the channel it was decided to pre-form the clay samples and then transfer them to the channel for final surface finishing and sealing to the channel sides and base.

Initially a hand tamping method of compaction was used, but it was found that the density within the sample was very variable and that the sample, which was formed in an angle-iron frame whose internal dimensions were the same as that of the required sample, was often so variable that large voids existed, especially near the corners of the sample.

Compaction was attempted using a standard compaction test rammer in an attempt to get uniform compaction. However, once again large voids were found to be present, especially at the corners of the sample, and the density was found to be variable. Tests carried out with these samples showed widely varying rates of erosion due to the variation in compaction techniques. It was therefore essential to devise a method of compaction and sample preparation which enabled reproducibility in the sample and thus reproducibility in test results. This was achieved by a method of compaction which was subsequently used throughout the

laboratory programme and which will now be outlined in detail.

A mould was designed and constructed consisting of an angle iron frame screwed to a wooden base. A wooden base the size of the sample and some 5 millimetres thick was oiled to prevent the clay adhering to the plate and then it was placed in the mould. The amount of the prepared soil required to achieve the desired density under test was then kneaded into the mould by hand in order that few voids were created in the compaction process. A steel plate some 10 millimetres in thickness was then placed on top of the soil sample, within the mould, and the apparatus was then transferred to a compression testing machine where load was applied to the top plate via a steel beam. The dimensions of the angle iron and plate were such that the sample reached the required thickness (40 millimetres) and hence the required density when the top of the loading plate became flush with the top of the angle iron. The beam and steel plate were then taken off the mould.

The mould and sample were then removed from the compression testing machine, and the screws holding the angle iron mould to the wooden base board released, enabling the angle iron wooden base and sample to be inverted and the sample removed by applying hand pressure to the wooden base. The wooden base could then be slid off the sample to leave a homogeneous, virtually perfectly smooth sample, ready to be transferred to the channel sample section.

The basic mould assembly is shown in Figure 5.13. Other frames were constructed to accept the samples made from the basic mould assembly to enable samples to be cut at an angle in order that the sample could be laid in the models horizontally, i.e. horizontal layers with vertical compaction, but also form the downstream and upstream

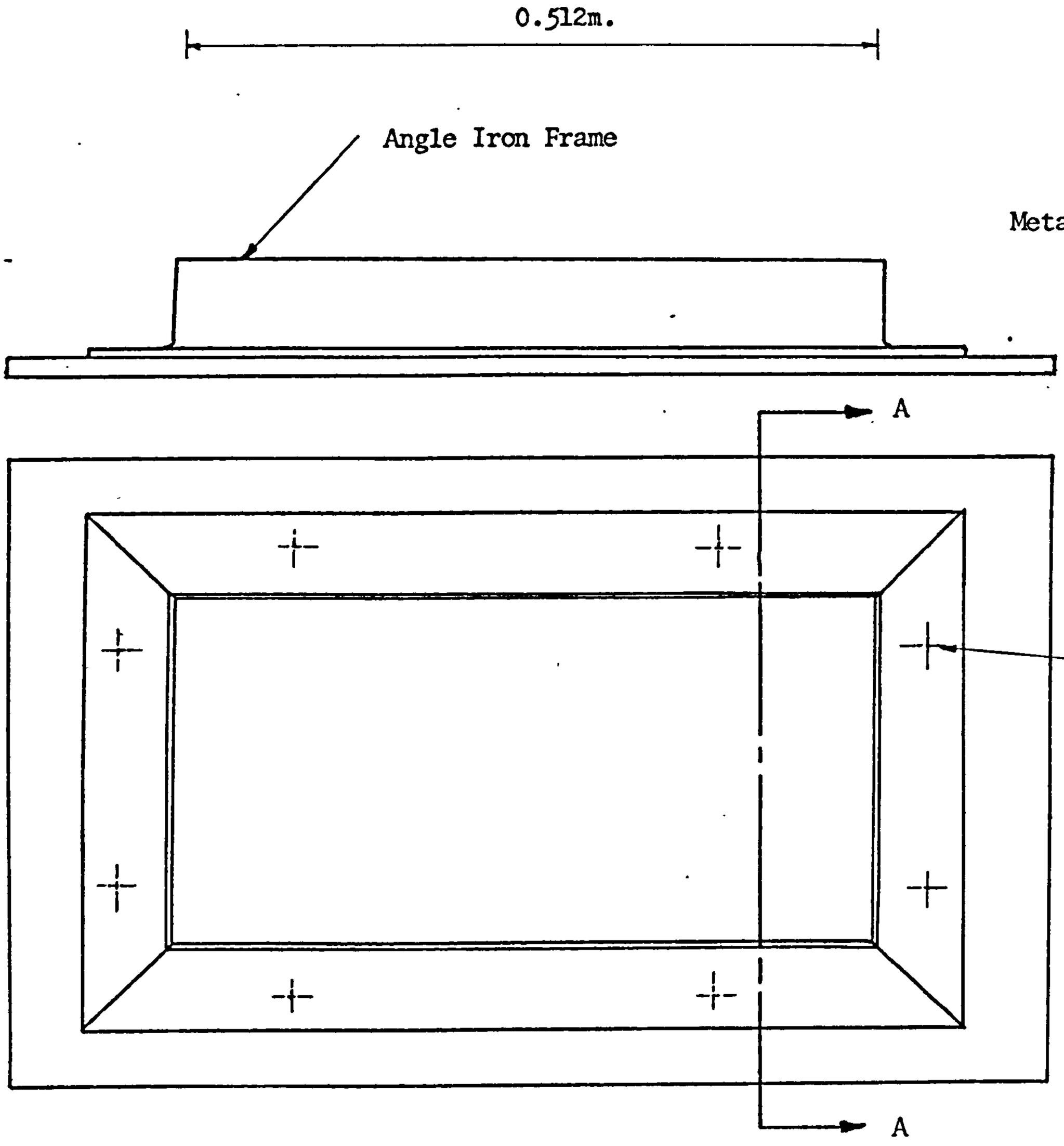


FIGURE 5.13: Mould Assembly (Sca

slopes of a model. The compaction and sample forming apparatus is shown in Plate 5.9.

5.14.4 Laying out the Model

Laying out the model, whether for tests in Series I or Series II, involved placing one or more of the samples prepared in the manner described in the previous section into the channel.

Series I experiments involved one sample only placed in the 'sample section' of the channel, a palette knife being used to smooth the clay against the side of the channel and the base of the channel.

Samples for experiments in Series II experiments were formed in the same manner but the ends cut to an angle in order that when placed one on top of the other the obliquely-cut faces formed the required downstream slope. The top and bottom faces of the sample were roughened to ensure some bond between layers. Final preparation of the surface involved smoothing with a palette knife.

5.15 Determination of Experimental Data

5.15.1 Initial Soil Properties

The density of the sample was accurately determined before placement of the sample in the channel by weighing the sample on a balance accurate to 0.25 g, and by knowledge of the sample volume the density could be calculated.

Due to the possibility of frictional effects at the boundary of the clay sample with the channel wall, it was decided to monitor erosion by weighing the remaining material at the end of the test within an area in the centre of the sample. This 'sub-sample' was 200 milli-



PLATE 5.9: Compaction and Sample Forming Apparatus

metres long and 150 millimetres wide. Material outside this area in the centre of the sample could be used,if required, to measure shear strength of the sample using the hand shear vane.

Six values of shear strength were usually obtained using the vane at various depths of penetration and the mean shear strength obtained used for calculation. Local disturbance of the clay due to the vane was smoothed with a palette knife before the test commenced.

5.15.2 Test Procedure

The manometer reading corresponding to the desired discharge for the test was read from the manometer reading v. discharge graph (Appendix C,Figure C1). The flow regulating valve on the pump was opened allowing water to be pumped from the lower tank to the upper tank until the manometers recorded the required discharge. A stopclock was started as soon as water began to flow over the sample. Water temperature was monitored by a thermometer placed in the lower tank.

The depth of flow was measured using the depth gauge and at the same time the Pitot tube was used at three sections across the channel and four depths, as shown in Figure 5.14, in order that the velocity distribution could be determined.

The concentration of suspended solids in the water was calculated before the test began using the apparatus described in Section 5.8. Samples were collected at intervals of 2½ minutes for the first 10 minutes and at intervals of 5 minutes thereafter. The samples were filtered in the manner already described using apparatus shown schematically in Figure 5.15. The change in suspended solid concentration as the test progressed could thus be calculated.

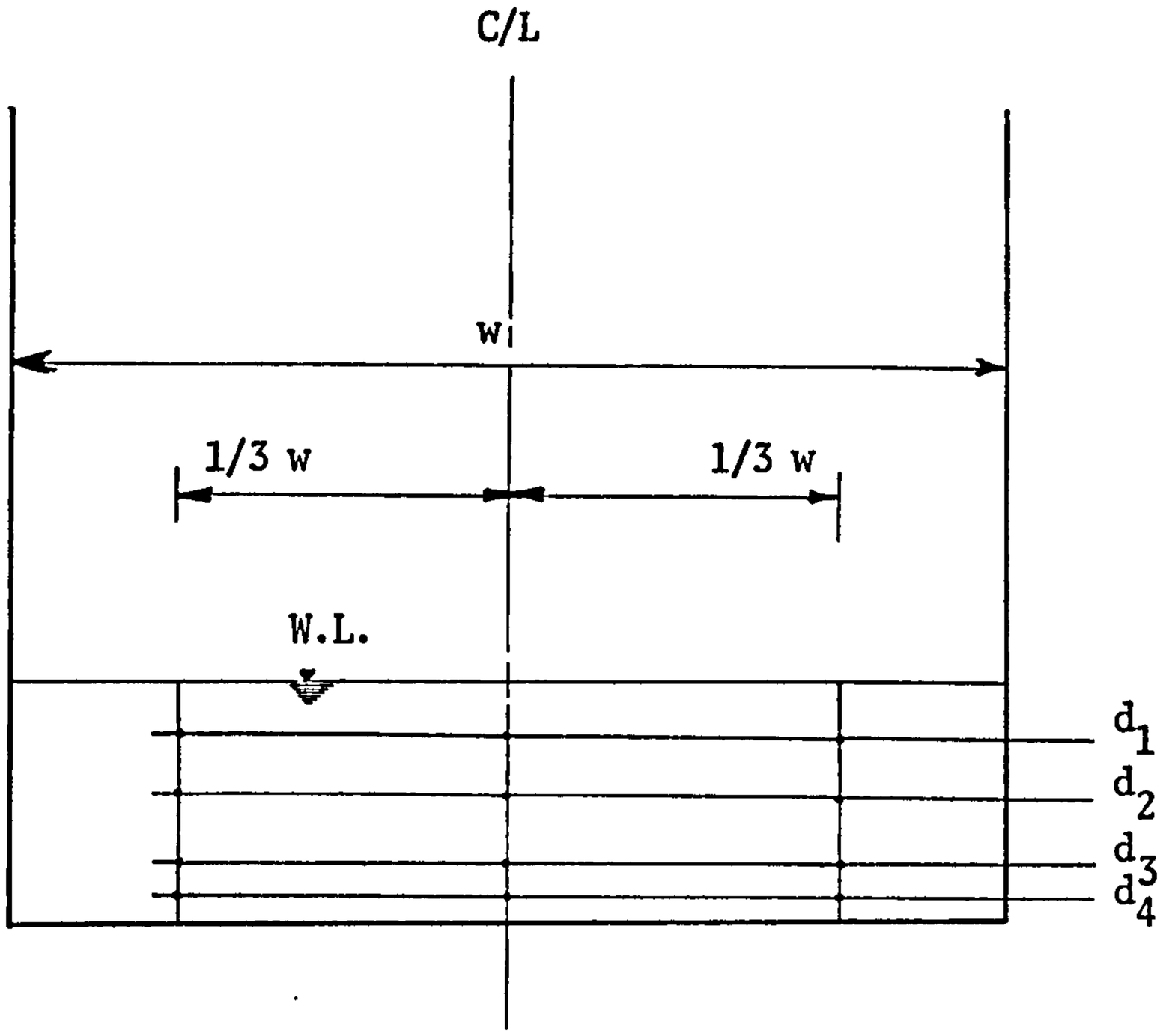


FIGURE 5.14: Velocity Measurement Positions

On completion of the test, which in the case of sample tests in Series I was at the end of 50 minutes, chosen because this was the time at which samples formed at low bulk densities began to break up, although some tests were run for longer periods, the valve controlling flow was closed and the pump switched off. The shear strength of the sample after the test was found by using the hand vane and then the sub-sample cutter, shown in Figure 5.16, was used to cut a sub-sample from the centre of the test specimen. The cutter, of known weight, and sub-sample could then be weighed to enable the loss in weight of the sub-sample due to erosion to be evaluated.

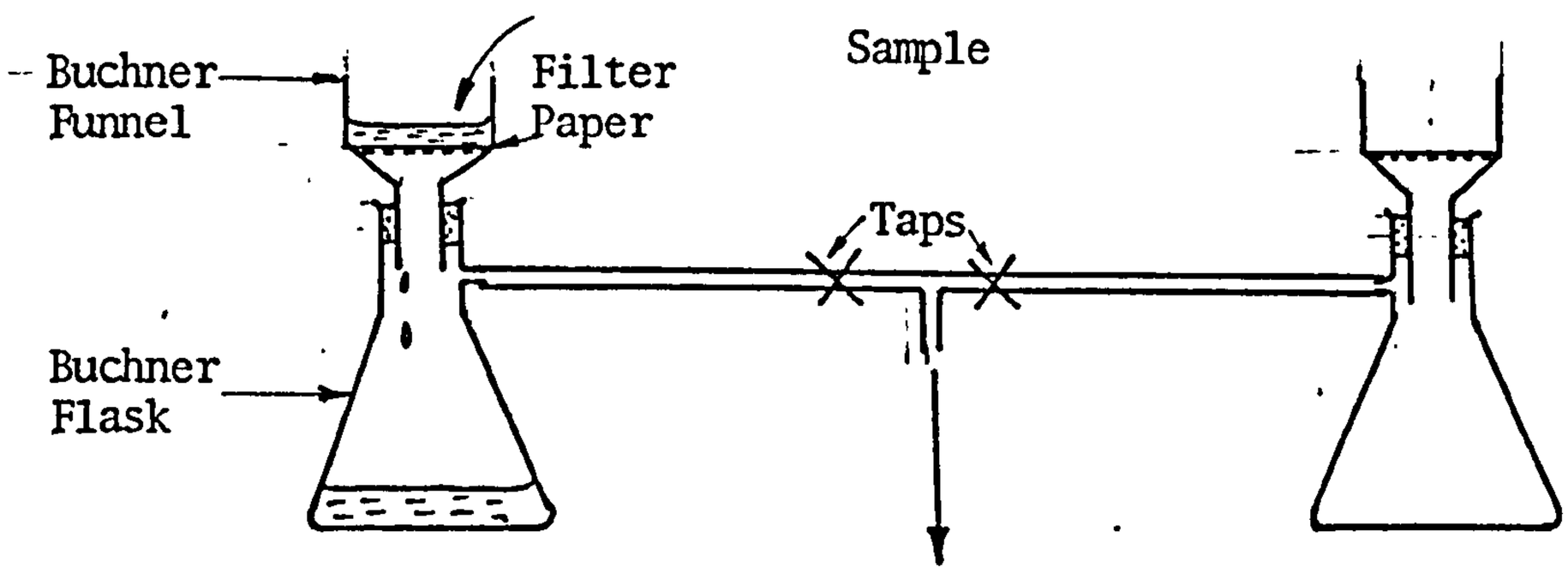


FIGURE 5.15: Suspended Solid Filtering Apparatus

The remaining material was also weighed to verify whether the channel walls had any effect on the erosion process. This provided a check, in as much as the erosion rates per unit time and per unit area could be calculated for the full sample and for the sub-sample and compared. The final moisture content of the sample was obtained by taking a sample from the test material and in addition moisture content was calculated for samples taken at depths of 10, 20, 30 and 40 millimetres. 'Still' photography was used in some tests from Series I to record the final condition of the clay sample.

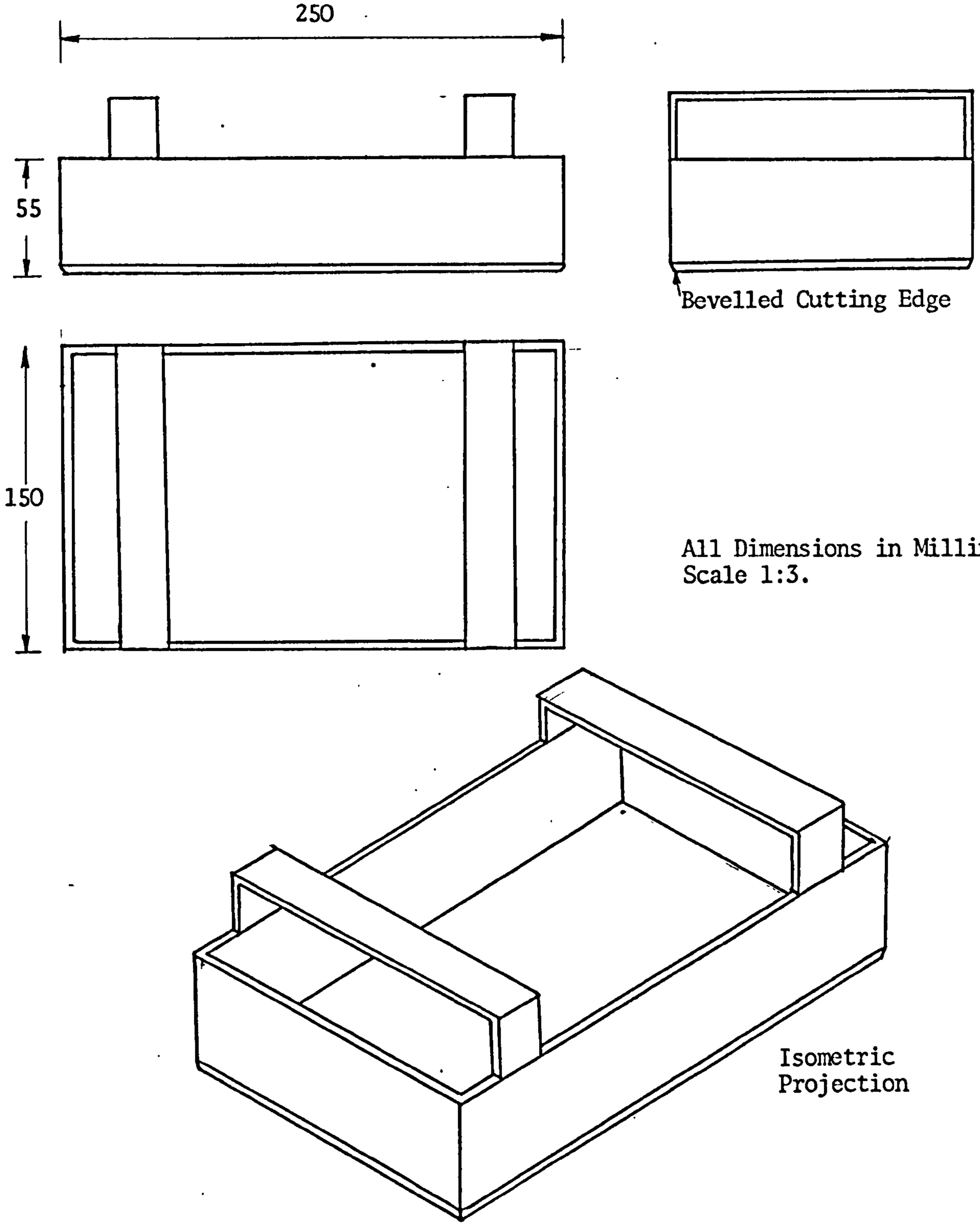


FIGURE 5.16: Sub-Sample Cutter

Series II tests were designed primarily to study the mechanism of failure due to overtopping. Time lapse photography was used on occasions to record the failure process. Processed films could be shown on a projector with a facility that allowed the film to be advanced one frame at a time enabling the erosion mechanism and process to be studied.

Swelling tests were carried out on material having the same properties as that used in the channel tests. Cylindrical sample, 100 millimetres in diameter and 40 millimetres deep, were formed in the apparatus shown in Plate 5.6 to the desired density and moisture content and then extruded into the apparatus shown in Figure 5.5. The perforated circular disc and the rods were then placed in the soil surface and the two gauges moved to make contact with the top of the rods. After zeroing the gauges, the material and overlying plate were covered with water to a depth similar to that experienced in the channel.

Readings of the extensometers were taken every 2½ minutes for the first 10 minutes and at intervals of 5 minutes thereafter until 1 hour, when they were then taken hourly. Readings were taken until there was no further change, i.e. less than 0.25 mm. in a 24 hour period.

5.16 Sample Calculation

Date of test:	30.7.76
Clay:	Standard - no sand
Weight:	9.696 kg
Volume of sample:	5.14×10^{-3} m^3
Density of sample:	1886.38 kg/m^3

Initial moisture content = $\frac{\text{weight of water}}{\text{weight of solids}}$

= $\frac{17.92}{80.42}$

= $\underline{\underline{22.28\%}}$

Shear Vane Readings

Penetration (mm)	Shear Vane Reading (°)	Shear Strength (kN/m^2)
30	60	24.52
20	36.5	24.02
10	24.0	30.39
10	25.5	30.54
20	39	26.00
30	67.5	27.76
10	21.5	26.96
20	39	26.00
30	66.5	27.65

Slope of Channel: 15°

Depth of Flow: 14.6 mm

Discharge: 10.3 l/sec.

Calculated Average Velocity: $\frac{Q}{A}$

= $\underline{\underline{2.82 \text{ m/sec}}}$

Pitot-Tube Readings

Depth from Surface (mm)	Head on Pitot Tube (mm)			Velocities (m/s)		
	Section I	Centre Line	Section II	Section I	Centre Line	Section II
7.6	400	410	400	2.67	2.73	2.67
8.2	250	290	285	2.06	2.24	2.22
9.3	235	275	260	1.98	2.17	2.10

These values can be plotted to produce a velocity distribution, as will be shown in Chapter 6.

Weight of 'sampler': 1.065 kg

Weight of 'sampler' + clay sub-sample: 3.199 kg

Weight of clay 'sub-sample' 2.134 kg

Volume of sub-sample (before erosion): $1.2 \times 10^{-3}\ m^2$

Assumed initial weight of sample:

= density x volume

= $1886.38\ kg/m^3 \times 1.2 \times 10^{-3}\ m^3$

= 2.263 kg

∴ Loss in weight of 'sub-sample': 0.129 kg

Total weight of clay remaining at end of test = 8.351 kg

Total loss in weight of clay sample = 1.245 kg

Duration of test	-	50 mins
Temperature of water	-	$21^{o}C$
Loss in weight/unit time (large sample)	-	42.68gm/min
Loss in weight/unit time (sub-sample)	-	2.60gm/min
Loss in weight/unit area (large sample)	-	$9688gm/m^2$
Loss in weight/unit area (sub-sample)	-	$432.1gm/m^2$
Ratio of areas:- (large sample to sub sample)		3.67 : 1

(N.B. It can be seen that the figures for the large sample and for the 'sub-sample' do not show good correlation with sample size but these results are from an early test when the experimental programme was in its infancy and a good experimental technique had not been established. It will be noticeable in later results, when sample forming techniques had been perfected, that a better correlation was established).

Moisture Contents

The moisture content at various depths within the sample was calculated at the end of every test in addition to a check on the moisture content of the clay obtained from a sample of full thickness. Typical results will be presented graphically in the next chapter.

Sediment Sampling The following results were obtained:-

Sampling time (mins)	Sediment Concentration (gm/litre)
0	0
2.5	0.140
5.0	0.160
7.5	0.175
10.0	0.190
15.0	0.210
20.0	0.220

Sampling time (mins)	Sediment Concentration (gm/litre)
25.0	0.250
30.0	0.264
35.0	0.246
40.0	0.320
45.0	0.330
50.0	0.354

The above figures are shown graphically in Figure 5.17 which indicates an initial high rate of change of concentration of material, reducing to a lesser value after several minutes.

Swelling

The results of a typical swelling test are shown graphically in Figure 5.18. Again an initial high rate of swelling is shown which gradually reduces until swelling virtually ceases.

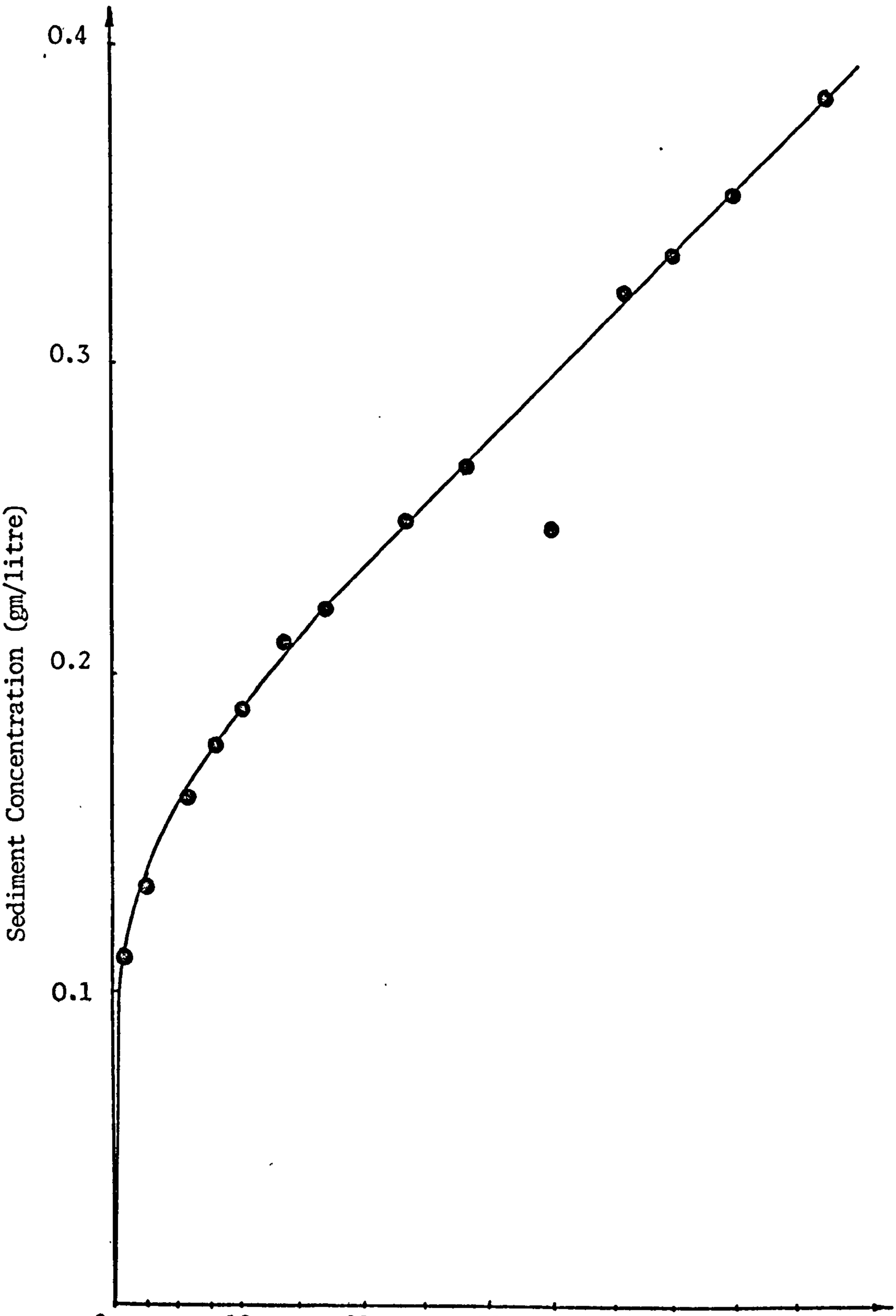


FIGURE 5.17: Graph of Sampling Time and Sediment Concentration

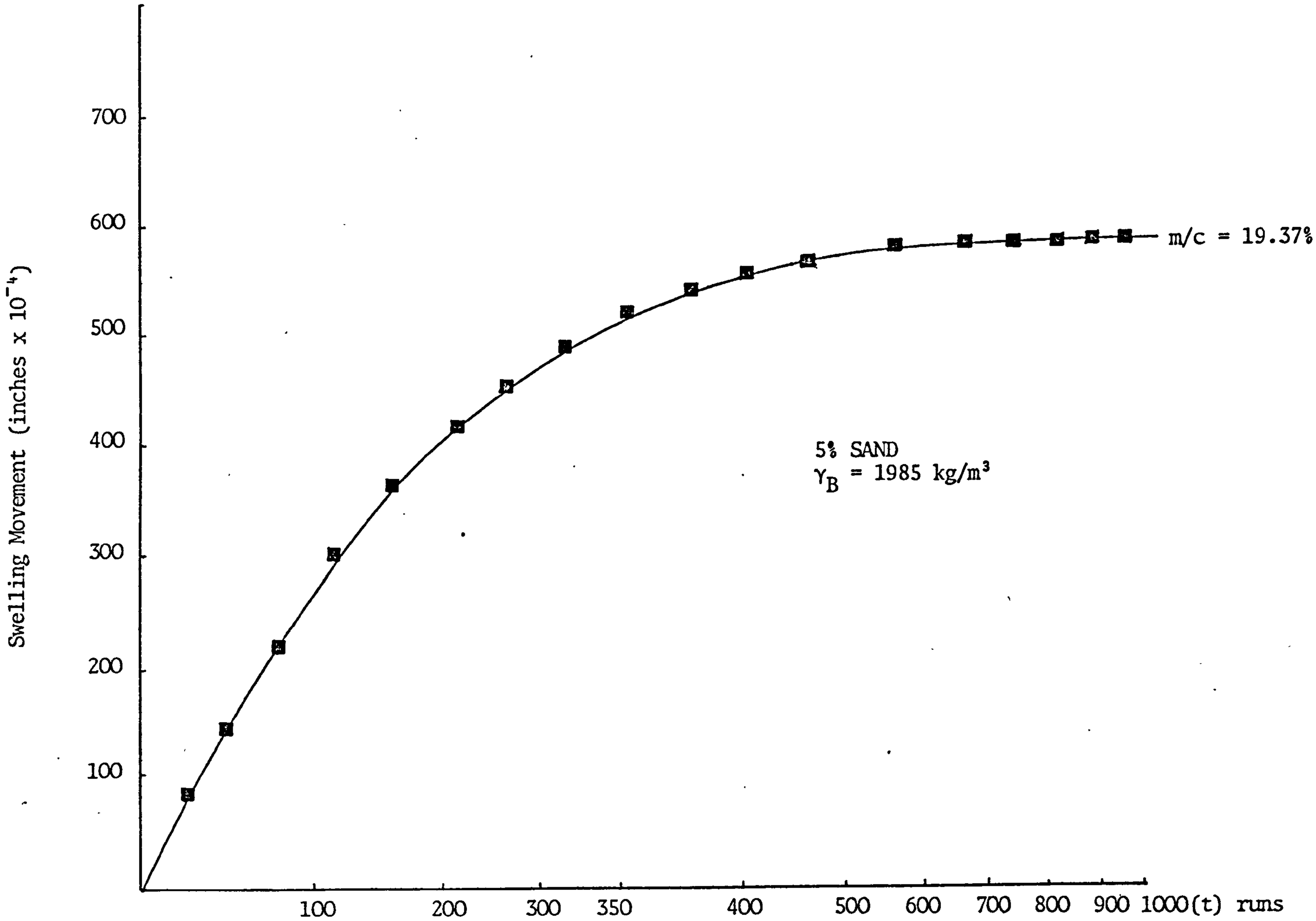


FIGURE 5.18: Graph of Time and Swelling Movement

Notes from Series II Test 53

Date of test: 5.4.1978

Model:- Cross-section as shown in Figure 5.19

Cross-sectional area	219860 mm^2
Width of Model	252 mm
Volume of Mwdel	5.54 x 10^{-2} m^3
M/C	19.75%
density (bulk)	1985 kg/m^3
Head over crest	65 mm
Head over toe (pt A)	14.4 mm (at start of test)
Manometer readings	410 mm
Discharge	11 litres/sec
Duration of test	9 hours

Test

1. Immediate plucking and removal of small pieces of clay occurred at the toe at the interface between channel and clay, where velocity and turbulence were highest.
2. 20 min - a vertical step 10 mm high, had formed at toe and in addition erosion was taking place on sloping face of dam
3. 25 min - rounding of crest and isolated 'pock' marks on downstream face.
4. 1 hr - plucking from crest - step feature forming 120 mm down from crest measured down the slope, and at approximately 300 mm intervals. By this time the toe had receded 180 mm, measured down the slope.
5. 2 hrs - increased surface erosion - extending to the area between the steps.

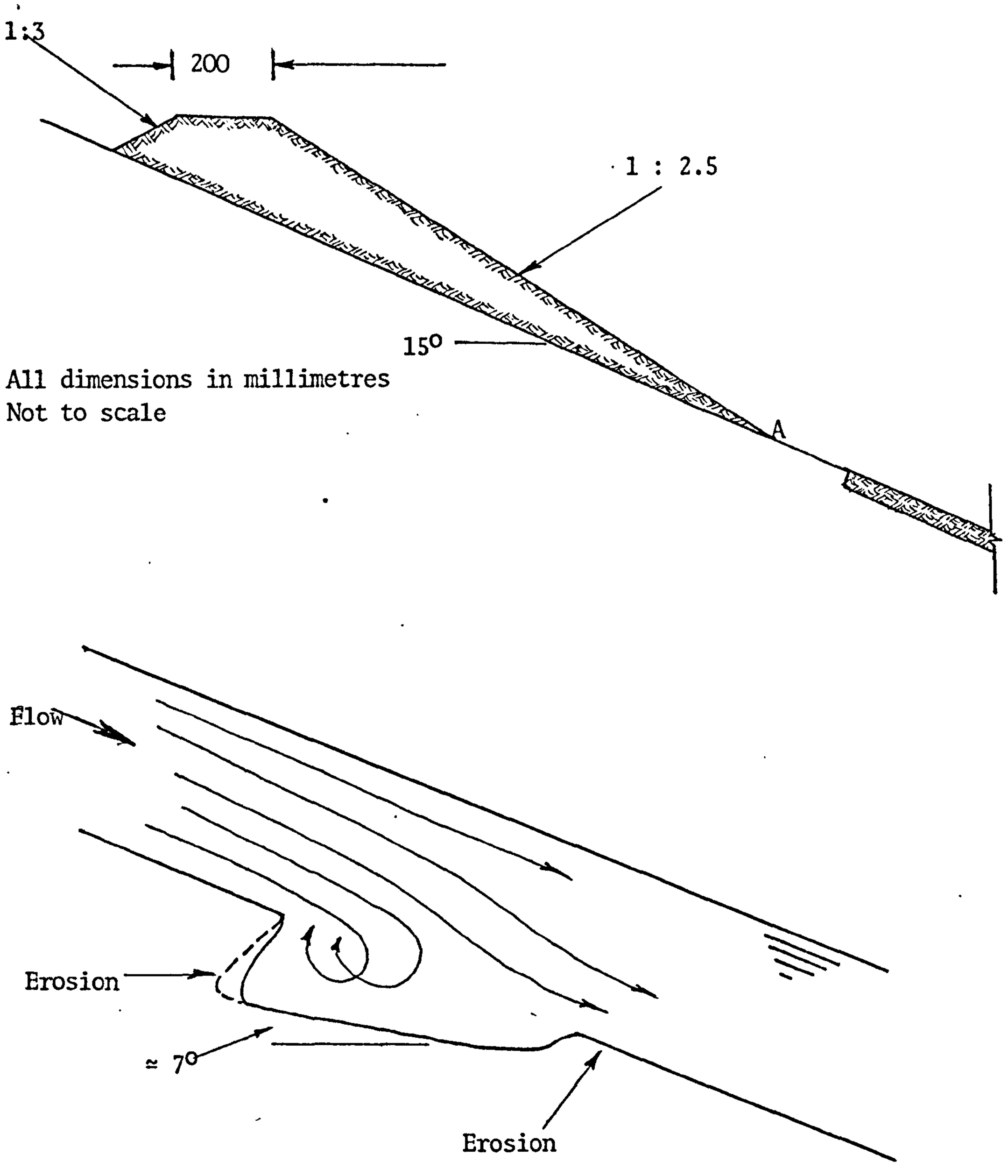


FIGURE 5.19: Model and Erosion Process for Series II, Test 53



PLATE 5.10: Series II, Test 53; Model before test.



PLATE 5.11: Series II, Test 53; Initiation of Erosion just below the Crest - 2 hours after start of test



PLATE 5.12: Series II, Test 53; Erosion at the Toe of the Model - 40 mins. after start of test



PLATE 5.13: Series II, Test 53; Erosion at the Toe of the Model - 40 mins. after start of test

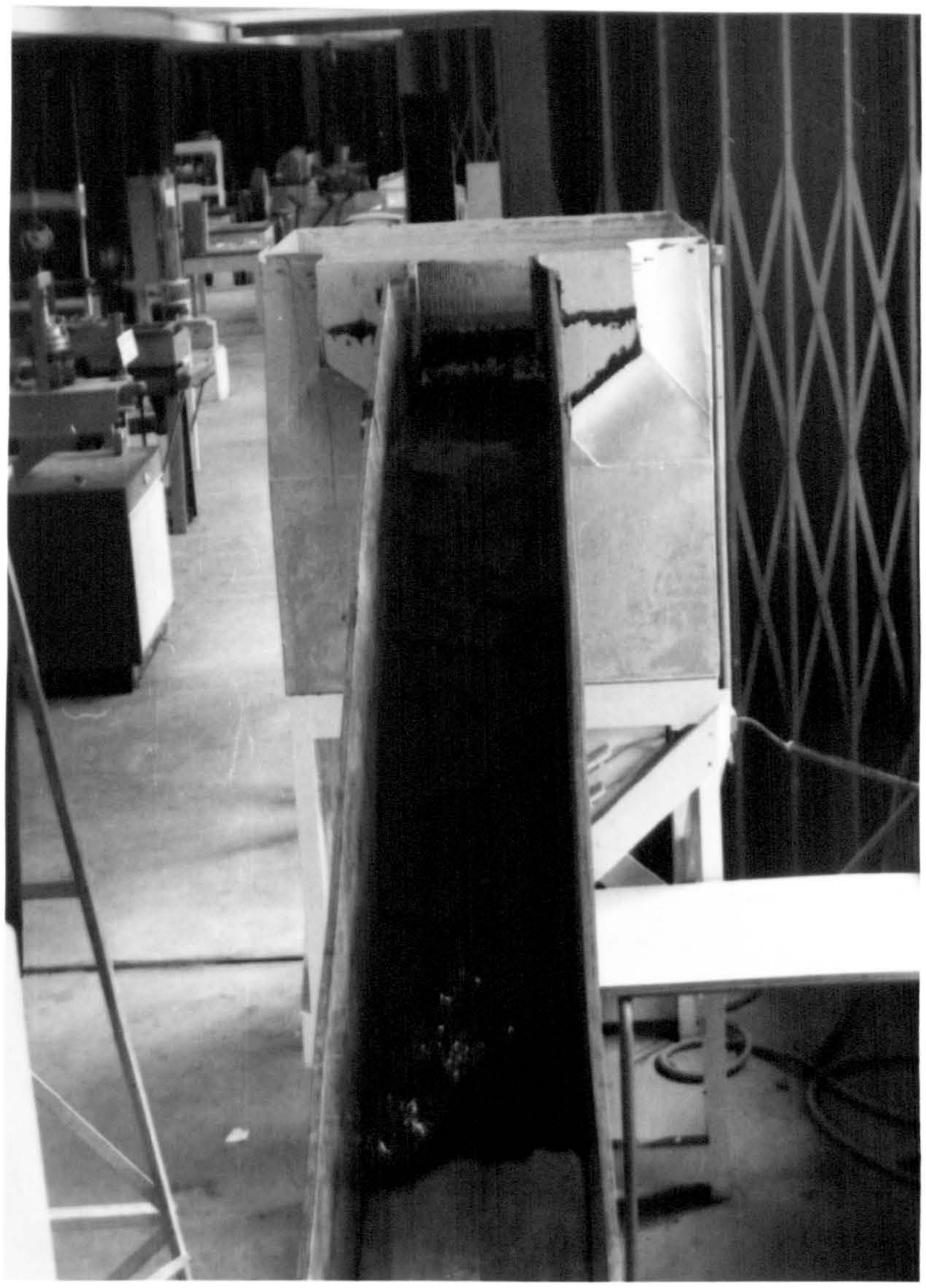


PLATE 5.14: Series II, Test 53; Erosion Pattern - 4 hours after start of test

5. - Flat of steps sloped at 7° from horizontal.

Toe now receded 200 mm.

6. $6\frac{1}{4}$ hrs - 1st step 60 mm high

Lowest step 60 mm high

The erosion pattern and process is shown in Figure 5.19 and Plates 5.10, 5.11, 5.12, 5.13 and 5.14.

5.17 Conclusion

Chapter Five describes the development of the experimental programme, the apparatus and the procedure, together with the results of tests carried out to establish the basic engineering properties of the test material and of the 'modified' material.

CHAPTER SIX

PRESENTATION AND DISCUSSION OF EXPERIMENTAL RESULTS

6.1 Introduction

This chapter presents the experimental observations and results, which are then, where appropriate, discussed with reference to the following:-

(1) Erosion process, mechanism and patterns

(2) Clay structure

(3) Clay properties

(4) Properties of the eroding fluid

(5) Current construction techniques in dam engineering

(6) Current design in dam engineering

It is generally observed that some soils erode easily while others, under the same conditions of rainfall, vegetation and topography erode very little. A substantial difference exists between cohesionless and cohesive soils in their interaction with flow induced hydrodynamic forces. For instance, cohesionless material relies on the submerged weight of the material i.e. gravitational forces provide the main resistance to erosion. It is also clear that a earthfill is, in general, a very complex material, and that fill material is invariably not single sized, making graphs similar to that reproduced in Figure 6.1

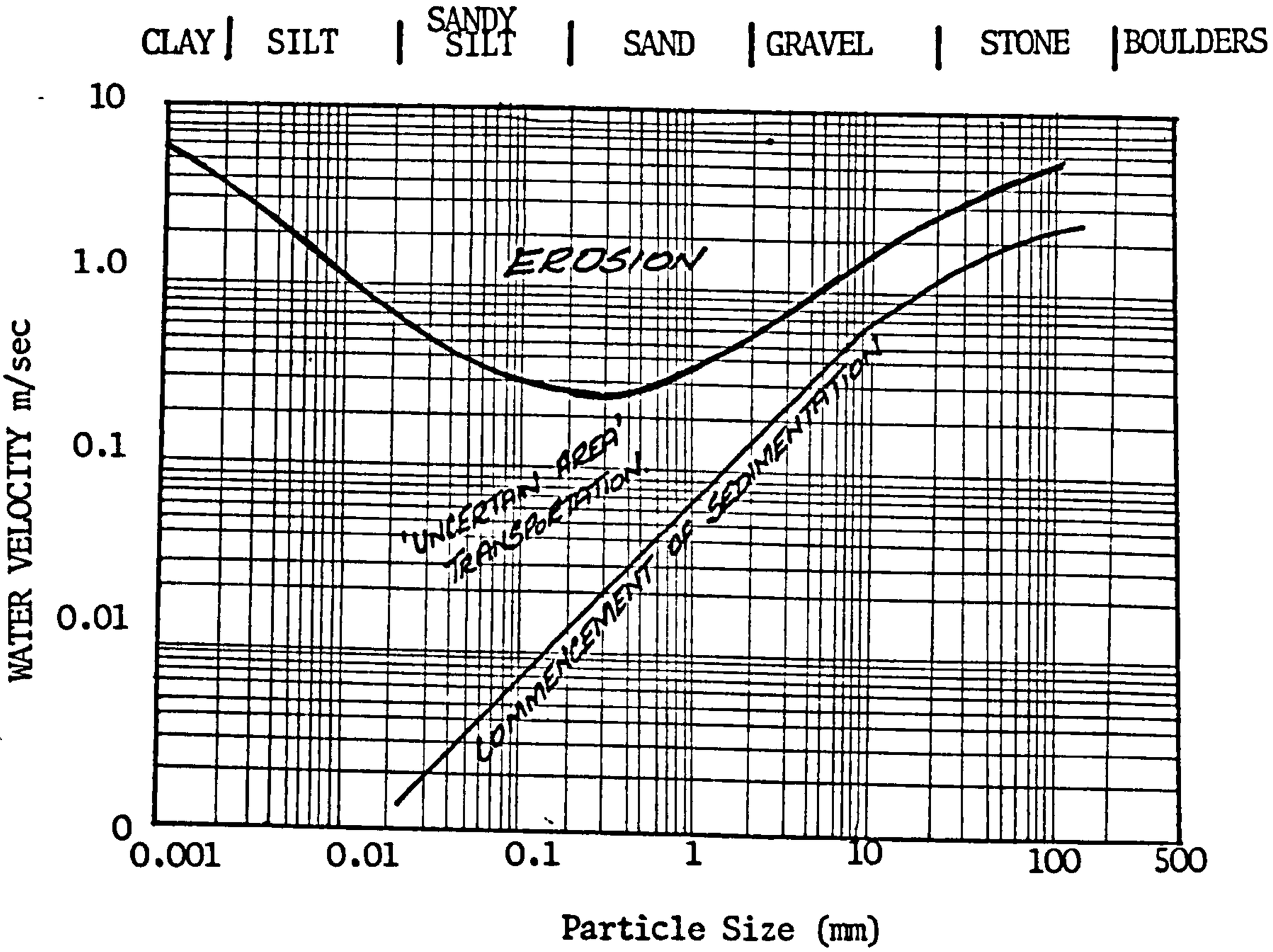


FIGURE 6.1 : Erosion Susceptibility in Relation to Water Velocity and Particle Size[177]

of limited use. Keen and Raczkowski[178] state that the behaviour of most soils is largely determined by the percentage of clay that they contain. The following chapter will discuss the results obtained in the experimental investigation.

6.2 Erosion

6.2.1 Mechanism

The phenomenon of erosion due to overtopping may be said to occur when the fluid induced shear stresses on particles reach values great enough to cause particle removal from the surface. Erosion of the soil involves detachment and transportation of the soil. Generally soil detachability increases as the size of the soil particles increases and thus it can be seen that a clay particle is more difficult to detach than sand particles but is subsequently easier to transport.

Particle removal will occur when the resultant of the induced forces exceeds those forces resisting motion. In the case of water flowing down the face of an embankment dam, or in the case of parallel flow in an open channel, the resultant hydrodynamic force causing erosion will be a shear force, whereas in the case of problems involving exit, the resultant will be a tensile force and, in the case of vertically impinging water jets, the resultant will be a combination of impact and shear. Samples in a smooth flume may not be fully representative of the shear forces that exist since as the flow encounters the soil sample there is an abrupt change in bed roughness and velocity distribution. It is felt therefore that critical shear from average tractive forces cannot be used as an 'erosion index' but that the technique used in this investigation is superior to the rotating blades used by other investigators, which create a whirling effect behind the blade and at the edges of the sample, and is also superior to water jets.

Particle removal was seen in these investigations to occur as movement of individual clay particles, or as small clay clusters (flocs), defined as 'surface erosion', and in some instances the removal of relatively large pieces of soil, defined as 'mass erosion'. These terms are further illustrated graphically in Figure 6.2.

If, at any instant, there are no external forces acting on the clay mass, the system is in a state of temporary equilibrium, for the 'static' conditions to which it is exposed. Imposition of an external force system, such as boundary shear, would create deformation. This may be slight, as with some small translation or rotation of individual particles or particle groups or, as will be explained later, be the result of swelling. Movement of the particles would then cause a re-adjustment in the force system which, if no longer stressed, would continue to deform. If the interparticle bond is broken and the freed particle entrained in the fluid then erosion could be said to have occurred.

Erosion is controlled by two sets of forces:-

(1) hydraulic - which break and remove particles from the bed

(2) resisting - which are, in the case of clay predominantly electrochemical in nature, but also include gravitational (negligible in magnitude) and viscous forces in the absorbed water layer surrounding the clay 'platelets'.

6.2.2 Movement of particles

Due to differential velocity distribution, the fluid past a particle of soil is faster over the top of the particle than around the

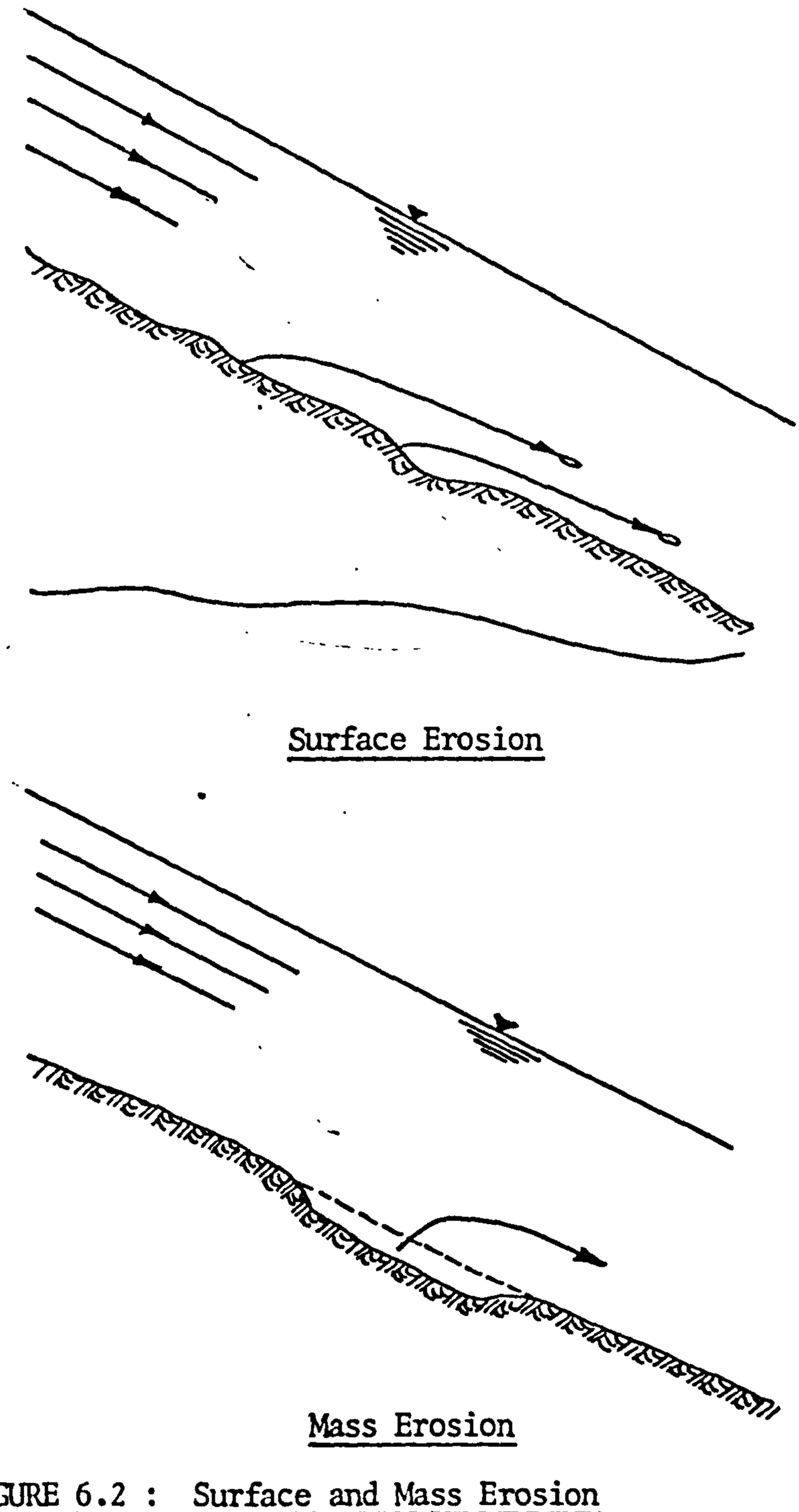


FIGURE 6.2 : Surface and Mass Erosion

sides and, indeed, at contact between particle and boundary the fluid is theoretically stationary. When it is remembered that clay particles are effectively thin plat shaped particles with one or two of their dimensions much larger than the other, the bed surface will appear on the microscopic scale as having small surface irregularities of the order of a few Å, while swelling, as will be explained, may result in particles projecting through any laminar sub-layer which might exist, and hence it is clear that particle size and shape are important factors influencing flow conditions, as shown in Figure 6.3.

Einstein and Huoi Li[179] postulated the theory of a periodically forming and disintegrating laminar sub-layer, which would result in high stresses on the bed at the start of the formation of the layer, decreasing as the layer is formed. It is considered that, since the bed becomes rougher as erosion and swelling occurs, then a variation of shear stress must occur with time due to fluctuations in the surface of the clay. As a result there must be a variation of shear stress with time, and hence the tractive force is not generally uniform over the bed. Consequently, particular maximum values are of primary interest rather than average tractive force values, as used by other investigators, and there is not usually a constant shear stress since, as the erosion process begins, roughness will provide additional resistance to the flow, and there will be a variation of shear stress.

It is considered that the first stage of erosion can very often be attributed to swelling which starts as soon as water begins to flow over and saturate the soil, a phenomenon to which clay soils are particularly susceptible by virtue of their high molecular weight and high silicate content. As the material swells, particles may

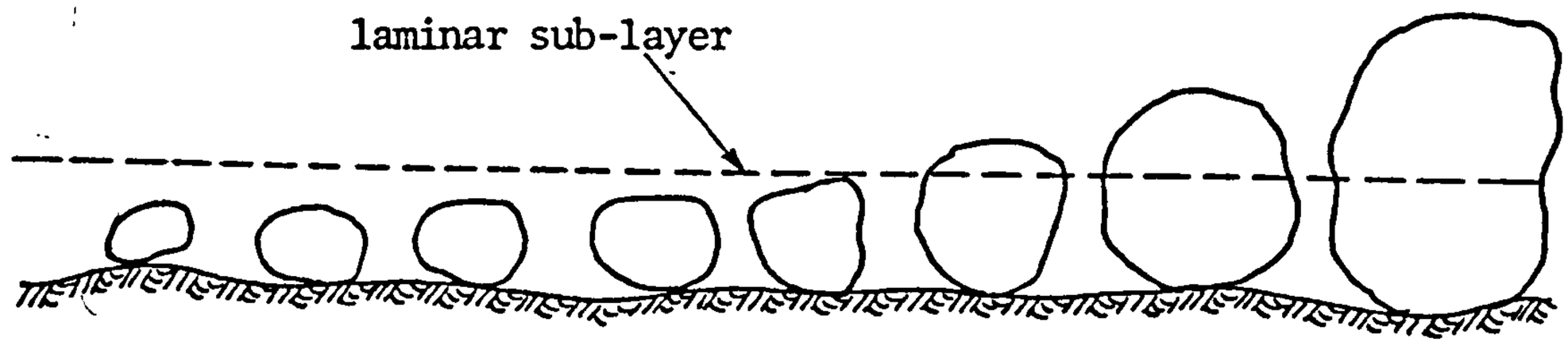


FIGURE 6.3 : Influence of Particle Size on Flow Conditions Around a Particle

consequently project through any laminar sub-layer which might exist.

In a partially saturated soil, such as the 'surface' of the downstream shoulder of an earth dam, the flow of water on the surface can suck out gases in the pores and carry the gas along, causing a net suction at the surface of the bed and resulting in cavitation. As a result of these processes the surface acquires a certain roughness, consisting of small knolls of low strength and perhaps some small stones projecting from the surface.

From Bernoulli's principle it follows that, for a particle projecting into the flow, the fluid pressure is greater on the underside of a particle than on the upperside, and as a consequence a net upward force is exerted on a particle. At the rear of a particle, and assisted by the contraction of streamline flow past a particle, there exists a negative pressure which assists in the removal of clay particles and small stones, and it is clear that if these particles slide over the surface they are likely to have an additional abrasive effect. As soon as the net upward force becomes larger than the submerged weight of the particle, the particle will move up from the bed and become incorporated in the flow pattern. White[180] suggested that the net upward force on a particle was very small, but in 1961, Hinze[181] showed the lift force to be comparable in magnitude with the drag force.

As particles move away from the bed the lift force decreases rapidly, since the magnitude of the lift force is related to the rapid decrease in the velocity gradient, and therefore pressure gradient, at increasing distances from the boundary surface. A position is reached where the lift force is smaller than the submerged weight of the particle and here deceleration, and eventually acceleration back

towards the bed, begins. In the absence of drag forces a particle could be expected to oscillate continuously about a critical height at which the lift force and the submerged weight are equal. Superimposition of the drag forces on the motion as described results in an overall 'bouncing' motion in the direction of flow known as saltation as shown in Figure 6.4. The height of bounce, (h), is directly proportional to the ratio of particle density to fluid density.

In addition to material carried along by saltation some material was observed to be carried in suspension, i.e. remaining in suspension without contact with the streambed, and the remainder as bed-load - material in almost continuous contact with the streambed.

6.2.3 Removal of a Single Particle

It was stated earlier in this Chapter that the surface clay particles form protrusions which are large compared to the water molecules. They are therefore subjected to hydrodynamic forces which can be resolved into a drag force in the direction of flow, and a lift force, normal to the drag force.

After swelling the surface particles will probably have suffered a slight reorientation. The edge of each particle will in general be attached to some low spot of the surface irregularities of some other particle. Thus a connection will be formed, perhaps surrounded by adsorbed water, and the particles may actually be in direct contact or be separated only by a few molecules thickness of the adsorbed water. Due to the nature of the forces acting between the two particles and the viscous properties of the adsorbed water the connection will possess some flexibility, and will therefore deform and yield plastically or

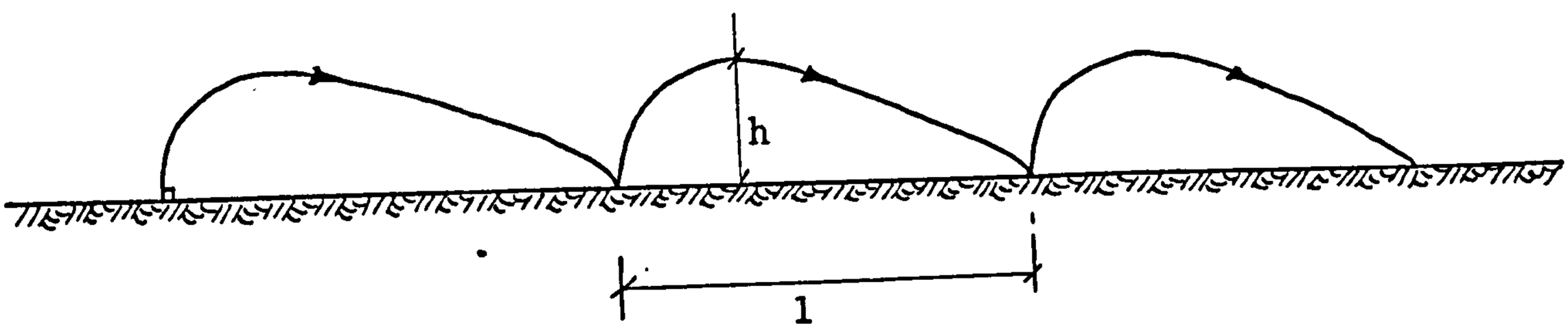


FIGURE 6.4 : Saltation

elastically if subject to a small rotation. However, if the upper surface clay particles are attached to only one end, and the other end is completely free to rotate, application of a moment will induce high tensile stresses at the connection which may break the connection between particles. The particle will then be picked up by the drag and uplift forces and taken into the flow. If the particle is supported at more than one point there is still a possibility of failure due to bending stresses at some critical position within the particle.

Each 'interior' particle will be attached to at least two other particles and the entire system forms a statistically highly indeterminate three-dimensional framework. Shear failure of this framework will occur within the soil mass when the strength of the connections or of the particles themselves is exceeded.

It is thus evident that the surface particles have at least one connection less than the 'interior' particles and that some are subject to hydrodynamic forces which induce bending stresses at interparticle connections. In addition surface particles are subject to the phenomenon of swelling, alteration in the double layer thickness, and the distribution of adsorbed cations, all of which will affect the equilibrium of the soil system.

Obviously the minimum rather than the average interparticle cohesive bond governs erosion, since particles can easily be removed if a few weak connections are broken, whereas the macroscopic shear strength will be governed by some average value of the interparticle cohesive bond.

6.2.4 Removal of a 'Floc'

In a flocculated structure the basic units are small clusters

of particles called 'flocs'. A flocculated clay can consist of 'floc aggregates', groups of 'flocs' and these 'floc aggregates' can group together to create an 'aggregate network' in a state of equilibrium. If subsequently consolidated by a load at the surface, the voids in the 'floc aggregate' network will reduce and a new equilibrium position would be achieved. A further increase in load will lead to a state where the aggregates will be at their densest spacing but still maintaining their original interparticle floc distance. Yet further increase in load will cause bonds between flocs to yield, and to move into stable positions, making the clay denser and eventually will cause flocs to deform. The inter-floc voids will then disappear, maintaining the average density of the same order as the original density of the flocs. Any further increase in load will result in a decrease in interfloc particle spacing.

If stress is applied to each of these states then it is clear that clay in the first state will fail along some plane containing the points of contact between aggregates while in the final state the failure plane will have to pass through a much larger number of bonded contact points and hence have a higher shear strength (since the shear strength is proportional to the number of particle contacts).

When subject to erosion, the surface of the first state will have a similar floc density to one of the later states and, although the surface will be more irregular, these irregularities are of the order of only a few microns. The surface behaviour of the surface layer will be similar in both cases but if the hydrodynamic forces are sufficiently large to induce mass shear the erosion rate will be higher in state first described.

6.2.5 Mass Erosion

Mass erosion occurs when the flow induced forces on the bed cause mass shear stresses which exceed the soil strength along some microscopic scale and deep seated plane. The same type of failure may also be caused by other physico-chemical phenomena, such as swelling, and represents a cause of rapid failure. In general, mass erosion may occur if the induced shear stresses are considerably larger than the macroscopic shear strength of the bed.

6.2.6 Resulting form of the Bed

The surface of the bed after a test was generally flat; with an even pattern of ripples approximately 2 mm high confirming uniform drag. The form of the bed after a period of erosion for a Series I and a Series II test can be seen in Plates 6.1 and 6.2 respectively. Where small stones occurred in the bed, the stone often caused the water to erode the bed to a greater extent than elsewhere on the downstream side of the stone creating.small 'pock-marks' or holes, even though the stones were only the order of 5 mm square and 1 or 2 mm thick. Figure 6.5 illustrates this form of erosion.

Another form of erosion was noticed during the experimental programme; these were 'pockets' of erosion as shown in Figure 6.6, where eddies whirled pebbles about and gradually eroded a deeper hole.

Partheniades[182] found that a straight narrow groove was eroded near the centre of a clay bed which was tested, while the rest of the bed remained flat with small irregularities. This type of erosion pattern is the type which would be associated with secondary current generated along channel boundaries. These currents were observed and described by Prandtl[183] and proved experimentally by Einstein and Li[184]



PLATE 6.1 : Series I, Test 12 - Resulting Form of Bed

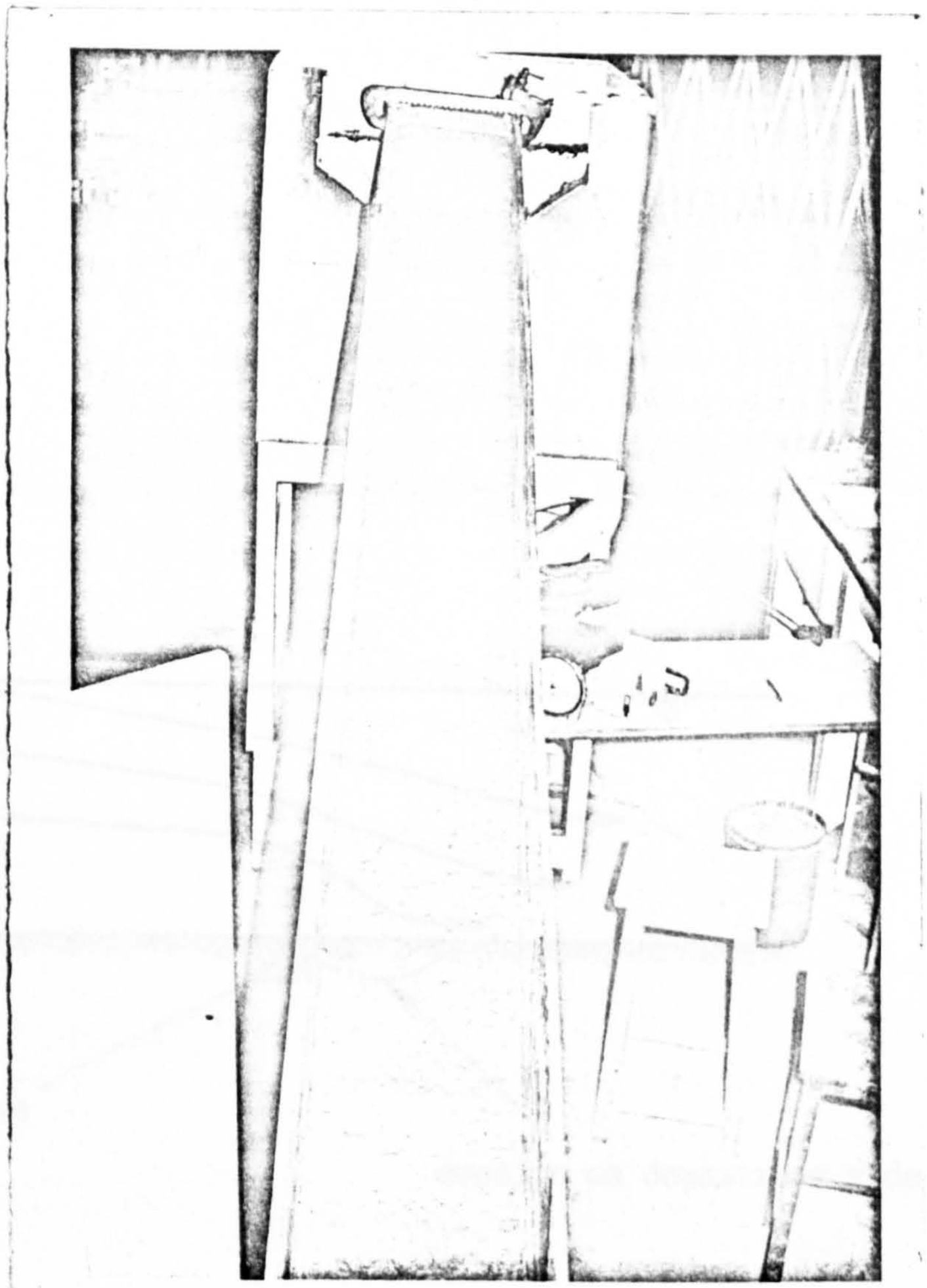


PLATE 6.2 : Series II, Test 56 - Resulting Form of Bed - 2 hours after start of test

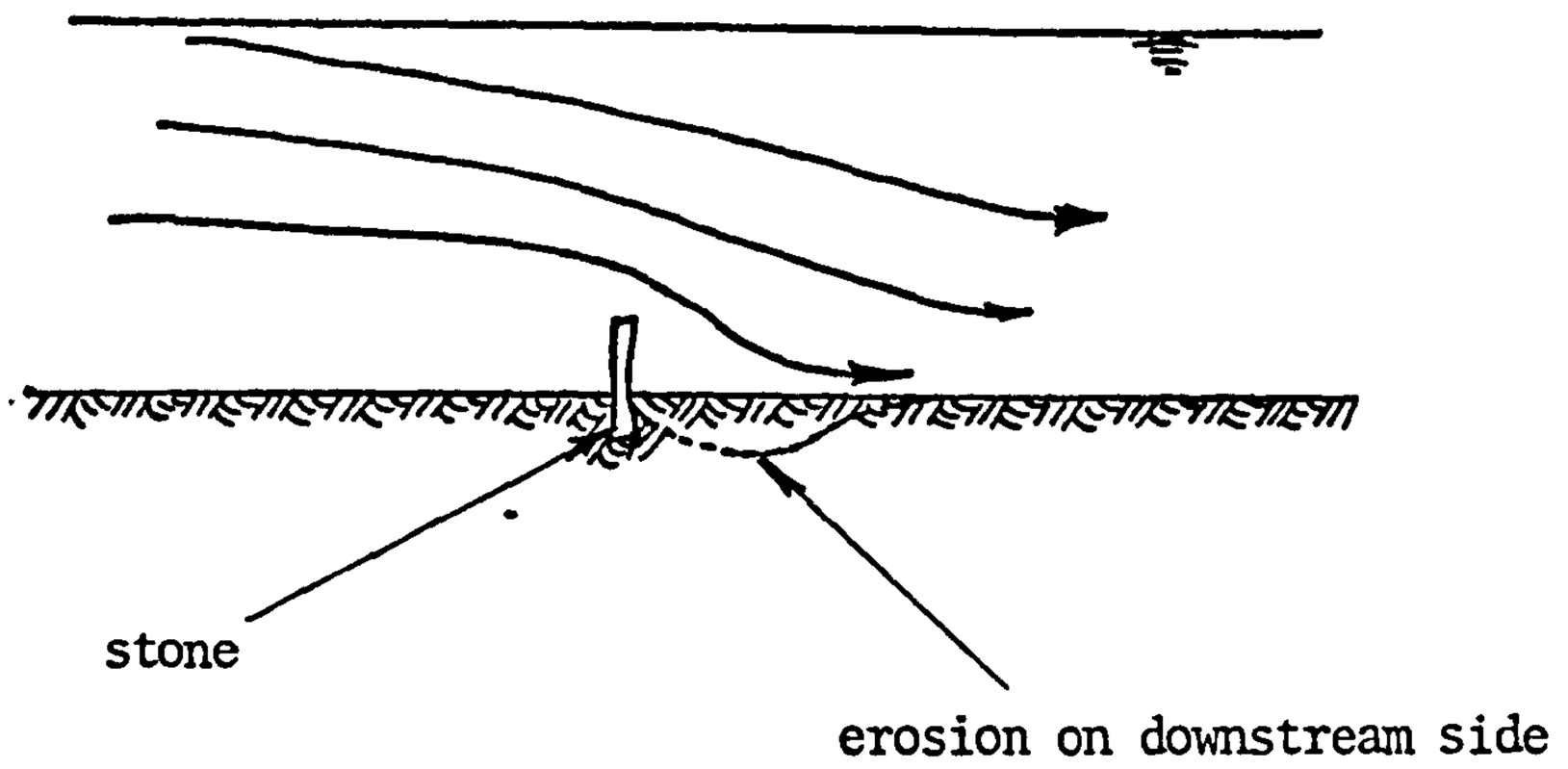


FIGURE 6.5 : Erosion due to Protruding Stones

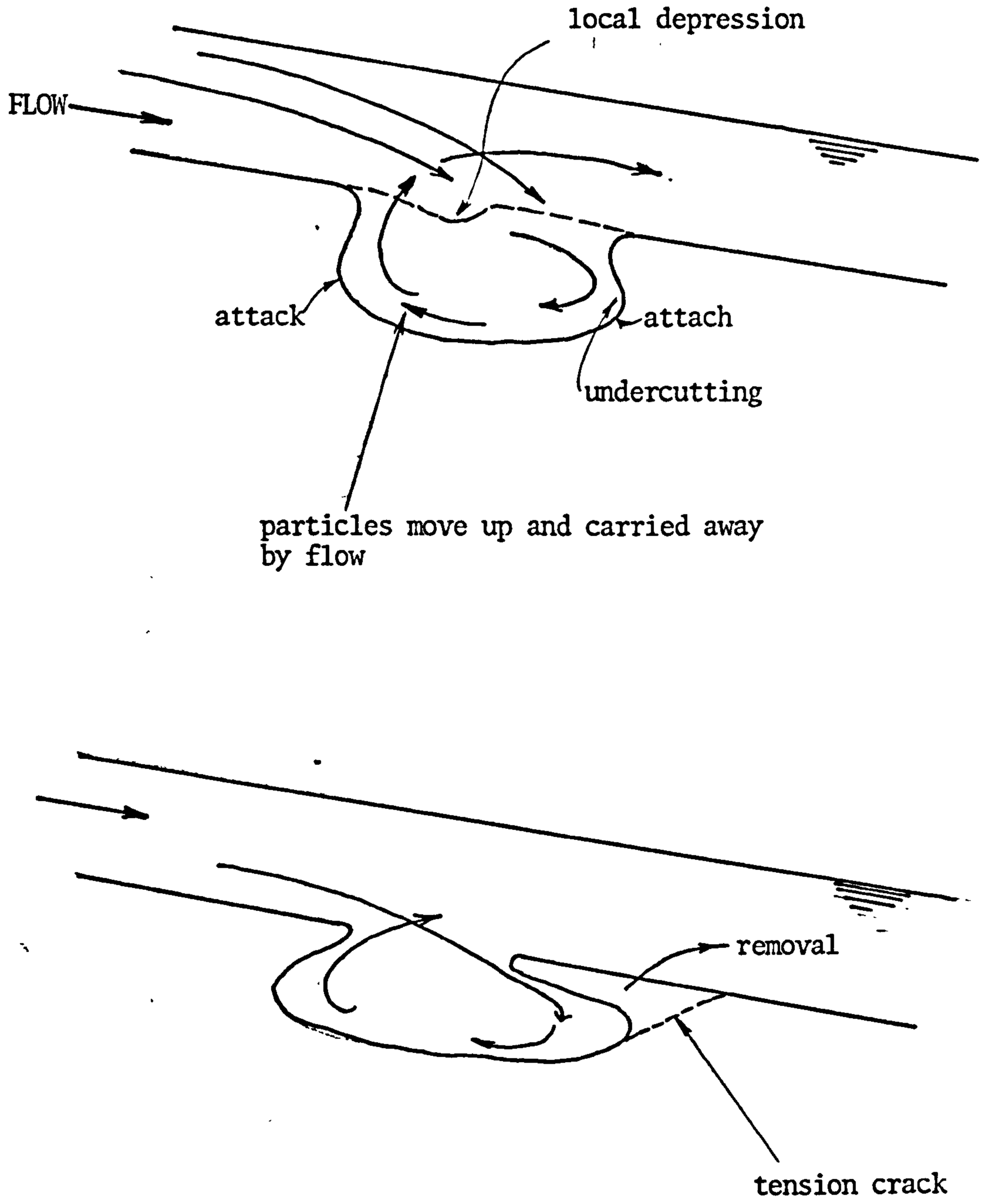


FIGURE 6.6 : Erosion due to Small Stones and Local Depressions

to be caused by the frictional boundaries.

The downward velocity components of the water at the outer part of the channel set up two boundary layers across the channel bottom. In a straight channel two boundary layers can be formed from the walls and within these the water can move in opposite directions[185]. These layers join each other near the centre of the flume where a narrow zone of high turbulence is created, which would obviously tend to cut a narrow groove near the centre of the channel.

The secondary currents affect the time variation of the shear stress at any point, although the time average shear strength may remain the same. Secondary currents are not accounted for in quasi-analytical methods and tend to make velocity distributions, and hence wall shear stress distributions, highly complex by the super-imposition of cross-flow components on the uni-directional base flow.

6.3 Formation of a Breach

If the inflow to an impounding reservoir is such that the retaining embankment is overtopped over a considerable length, or if 'local' overtopping occurs due to settlement of the crest, erosion will generally quickly concentrate and develop in one area, at the weakest or the lowest point, although some mass movement of the turf and topsoil may also occur and gullies will be cut into this upper layer.

The breach formation observed during the course of this research was found to follow the same form as that found in earlier experiments[186]. The model tests in this earlier series were constructed in pulverised fuel ash (P.F.A.) which was found to have a finite value of cohesion and to possess some of the properties of clay.

In tests carried out during the course of the current research,

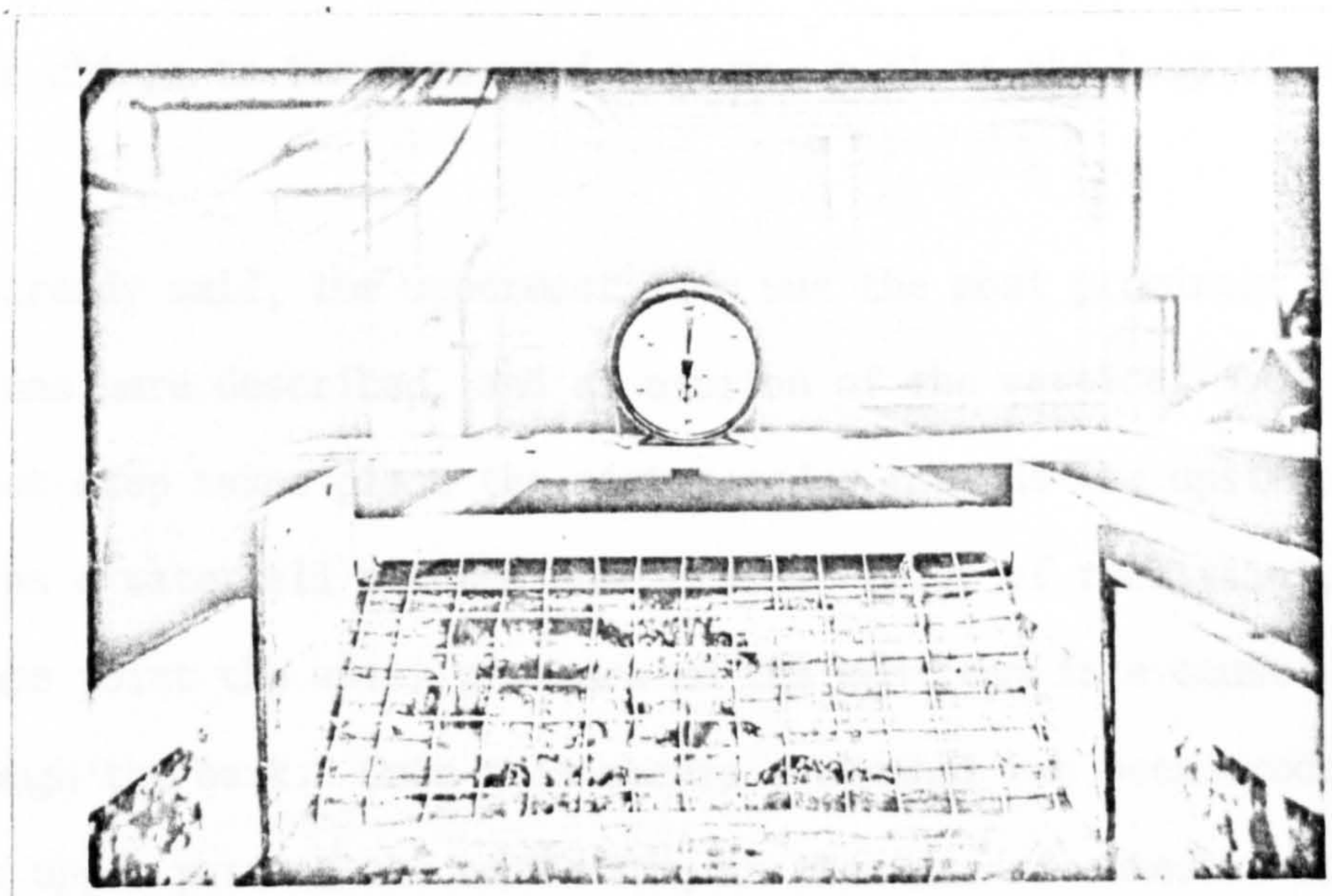


PLATE 6.3 : Step Features on the Downstream Face of a P.F.A. Model

as with earlier tests, several striking features were observed.

Overtopping waters were seen to cause a rounding of the crest and, at the same time, to form what will be described as a "step-feature" near the crest. This feature was repeated at fairly regular intervals down the downstream face, the step nearest the crest being the most prominent. In some cases, where the toe of the models met the base of the test channel, a further 'step' feature occurred. Plate 6.3 illustrates the step feature found in one of the early P.F.A. models.

The 'step' feature forms in a manner similar to that in which a waterfall erodes; establishing a vertical face, with some undercutting if the water clings to the face, and a plunge pool at the base of the face.

As already said, the uppermost step was the most prominent in the investigations here described, and as erosion of the vertical face of the uppermost step takes place the step recedes towards the upstream face, just as a waterfall recedes and forms a 'gorge of recession', until at some point the water pressure on the upstream face causes a breach through the bank. Once this occurs a channel has been eroded through the upper part of the bank with, in the case of cohesive soils, vertical sides. Reservoir storage will, at this stage, cause an increase in discharge.

Stability of the vertical sides will depend on whether any undercutting occurs and on the Culmann theory:-

For a stream cutting vertically down in clay the critical height (Hc) before instability occurs is given by:

$$H_c = \frac{4c \sin i \cos \phi}{\gamma_B \left[1 - \cos(i - \phi)\right]}$$

the symbols being defined in Figure 6.7, and γ_B being the bulk density of the soil.

Assuming that the failure surface is a plane passing through the toe of the slope

$$H_C = \frac{2c}{\gamma_B} \frac{\sin i}{\sin (i - a)(\sin a - \cos a \tan i)}$$

Field observations have shown that this expression is often satisfactory for vertical slopes

If $a = \frac{1}{2} (i + \phi)$

$$H_C = \frac{4c}{\gamma} \frac{\sin i \cos \phi}{[1 - \cos (i - \phi)]}$$

and if the stream cuts vertically then $i = \pi/2$

$$H_C = \frac{4c}{\gamma_B} \frac{\cos \phi}{1 - \sin \phi}$$

or written in a common form

$$H_C = \frac{4c}{\gamma_B} \tan \left[\frac{\pi}{4} + \frac{\phi}{2}\right]$$

Hence instability of the vertical face can be predicted.

Instability of the vertical faces of the breach occurred in several P.F.A. models, the unstable material falling into the breach and being carried away. Erosion continues through this channel, deepening the channel until the reservoir is empty (or storage is insufficient to maintain further flow sufficient for continued erosion in the case of smaller reservoirs).

On the upstream face, as more and more water is"channelled"

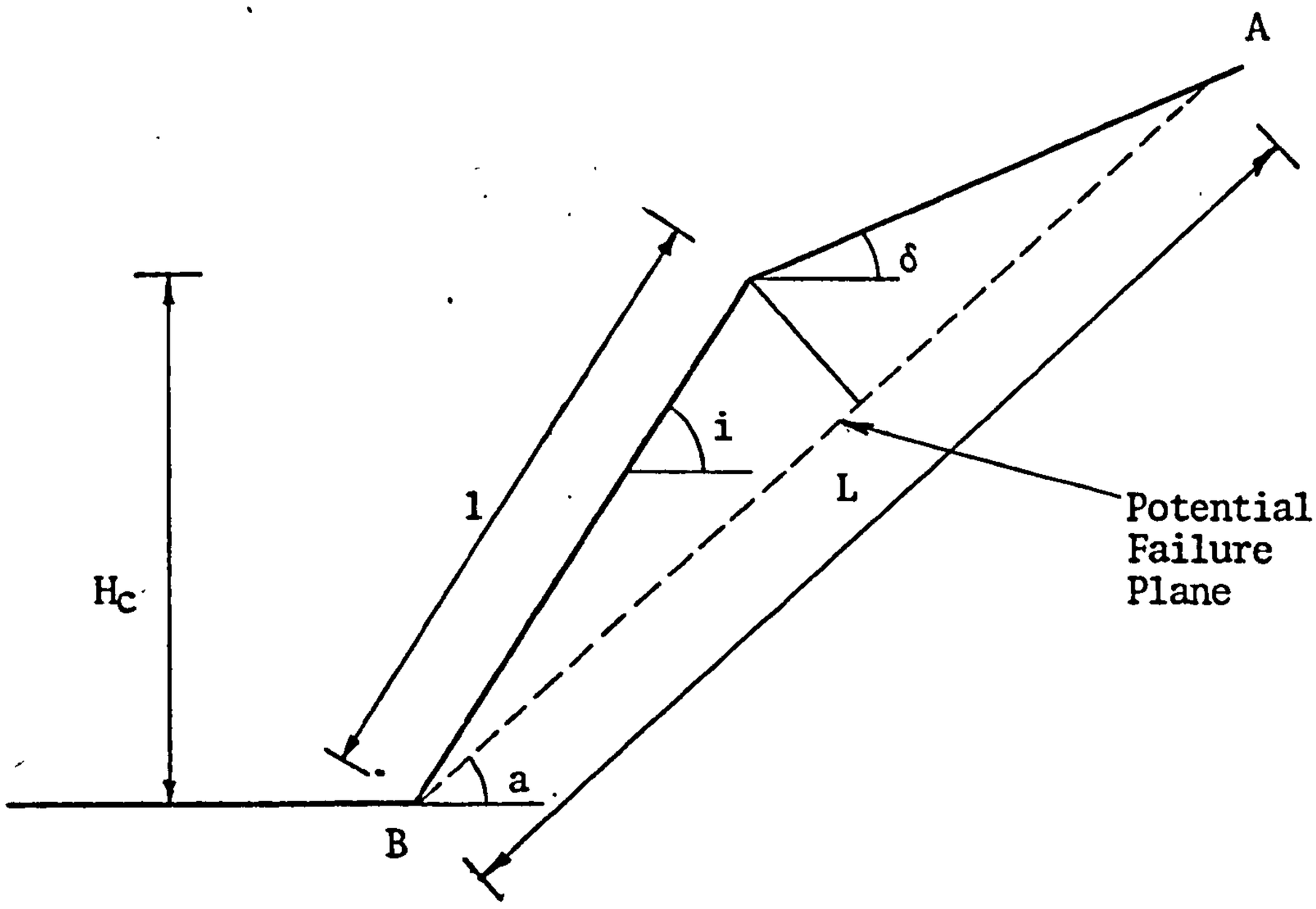


FIGURE 6.7 : Slope Stability

towards the breach a widening of the upstream end of the channel occurs as shown in Figure 6.8 and may result in some undercutting of the upstream end of the channel which again can cause sections of the crest to fall into the breach and to be carried away.

The tests reinforced observations by Foster and Martins[187] and by White and Gayed[188] that the shape of the eroding face depends on the nature of the material. In the case of uniformly graded cohesionless material without fines, the slope of the eroding faces are gradual and the material rolls continuously into the flow, but for materials with some apparent cohesion, or mechanical interlocking, the face stands closer to the vertical and some undercutting is possible.

If the dam sits on an erodible base a deep plunge pool will be formed at the junction of the toe of the bank and the original ground due to turbulence. If the base is not susceptible to erosion then the turbulence caused at the toe will result in material being eroded from the toe of the bank and a step feature being formed as shown in Figure 6.9.

In addition, theoretically at least, the toe is where the overtopping water will attain its highest velocity.

Until the channel forms through the embankment the erosion process will obviously cause roughening of the downstream slope which in turn causes turbulence and hence increased erosion.

Observation of the models suggests that if any discontinuity exists then surface damage is likely to occur as the result of local turbulence. Holes caused by projecting stones or stones being whirled around them result in the streamlines flowing into the hole attacking the downstream edge, turning the water back, taking suspended material

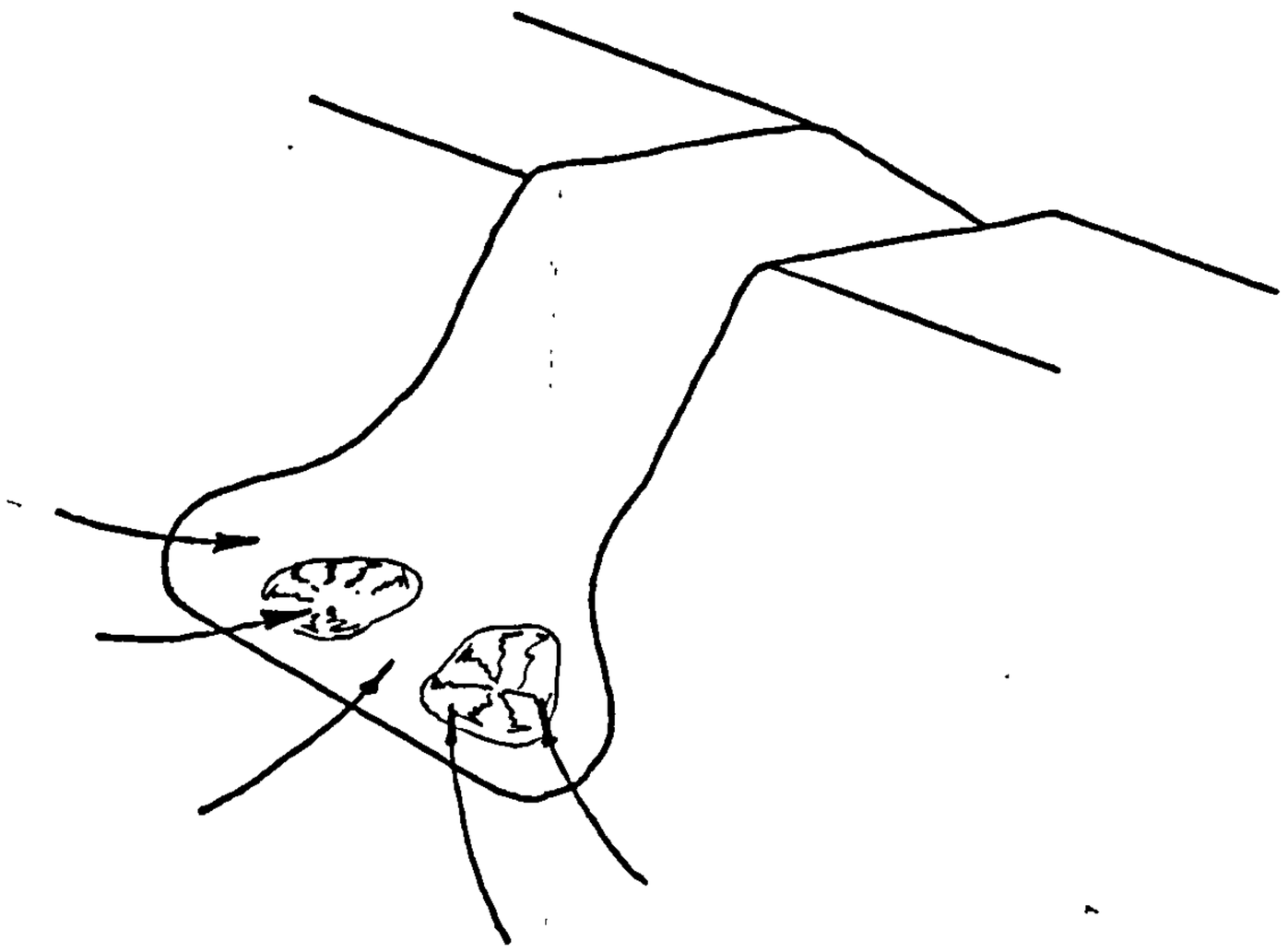


FIGURE 6.8 : Erosion of the Upstream Face by Channelling

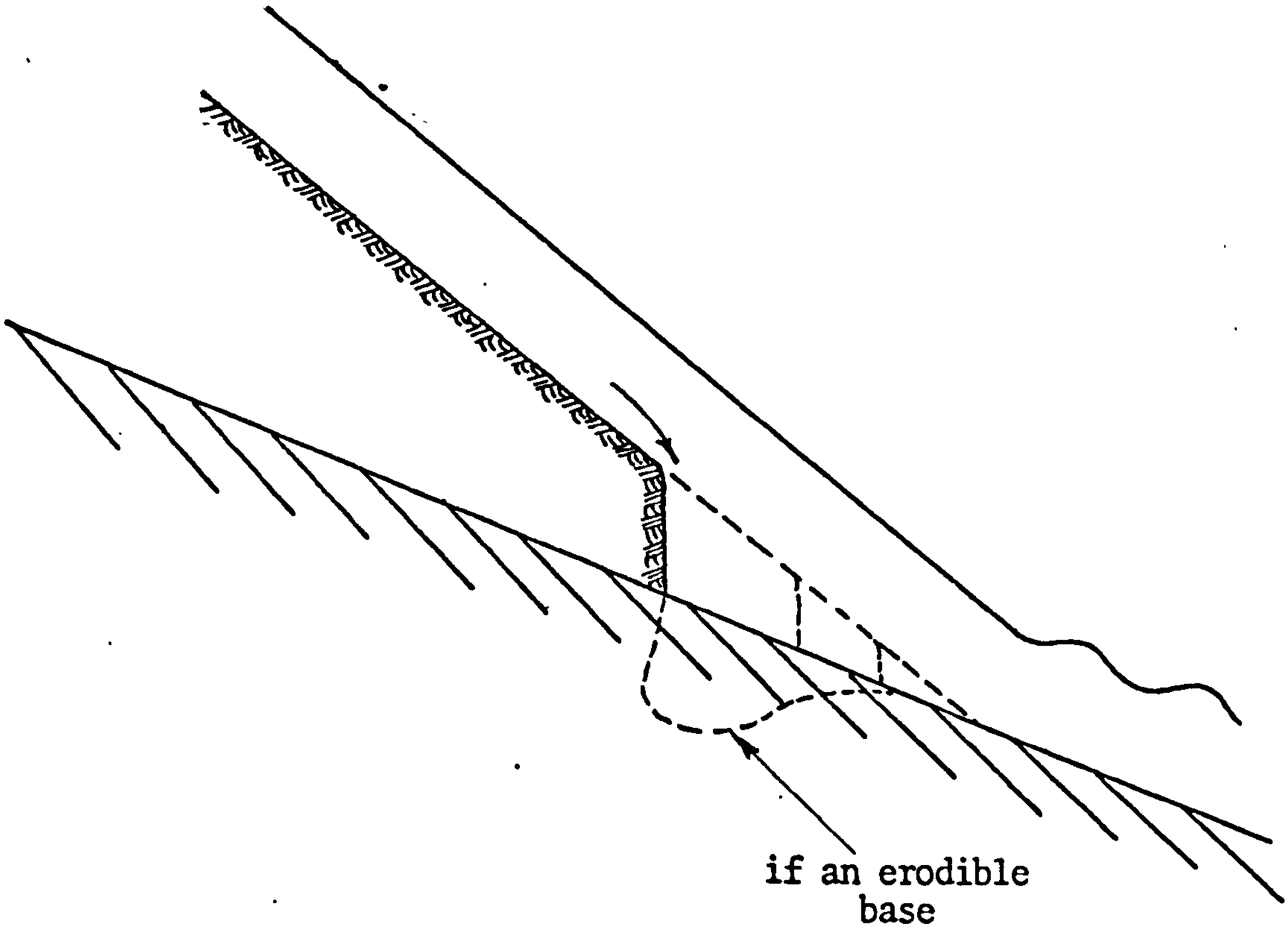


FIGURE 6.9 : Erosion of the Toe

with it, and attacking the rear face. As the water moves the particles up the back of the void these particles are captured by the main flow and removed. In some cases undercutting will eventually result in the removal of large pieces of the bed. This process is shown in Figure 6.6.

If filter zones exist within the embankment shoulder then these zones, being granular material, will erode more quickly than the surrounding cohesive material, and it is also clear that berms on the downstream shoulder will affect the erosion pattern. Berms will result in increased turbulence at the junction of the berm with the downstream slope and cause the formation of a step feature. This feature may be observed on the photographs of the Enclides da Cunha dam in Brazil, which failed by overtopping in 1978 (pages 111 and 112).

It is interesting to note that the 'step'feature was recorded in the eyewitness report of the much earlier failure of the South Fork Dam in 1889.

"the water running into this hole cut

away the breast in the form of a step"

(See page 104)

It is thought that the 'step' feature results from the change in velocity of the overtopping waters as they flow down the downstream face.

Assuming the velocity of the water as it crosses the crest to be negligible, and assuming topsoil and grass to have been removed, then as the water flows down the face it will accelerate to some point where the induced hydrodynamic forces, coupled with the effect of swelling, will cause removal of the particles and the formation of a hole. Water

flowing into this hole forms a plunge pool and the vertical 'step' feature. Water leaving the plunge pool will have decreased in velocity and will again accelerate down the slope again until at some point the hydrodynamic forces will be such that another step is formed. This theory thus explains why the step features are fairly evenly spaced down the face. This process is shown in Figure 6.10, the full process being shown in Figure 6.11a and 6.11b.

6.4 Breaching of a 'Cored'Dam

If a dam composed of an impervious clay core with granular upstream and downstream shoulders is overtopped, then as shown by tests carried out by the Author[186], Esmail[189], Jayasinghe[190] and Tunney[191], the downstream shoulder erodes comparatively quickly, leaving the core unsupported and standing as a cantilever. The core then acts as a weir until failure occurs. This phenomenon was observed with the Cowlyd dam in 1924, albeit with a concrete core. Jayasinghe[190] observed cracks in the core, just before failure similar to those shown in Figure 6.11, and similar to those one would expect from a core restrained on three sides, as one would have in the case of concentrated flow through a channel eroded within an embankment.

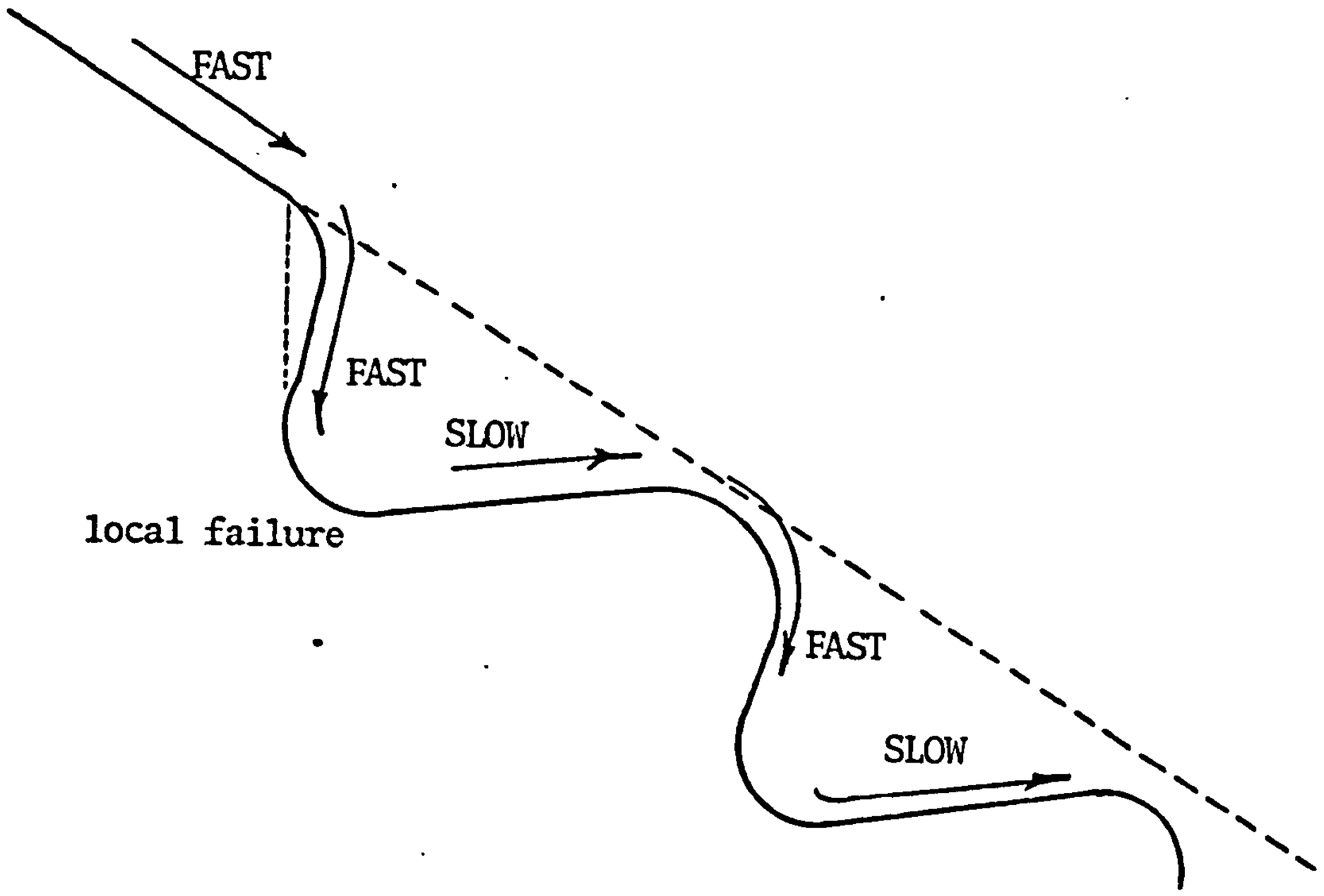
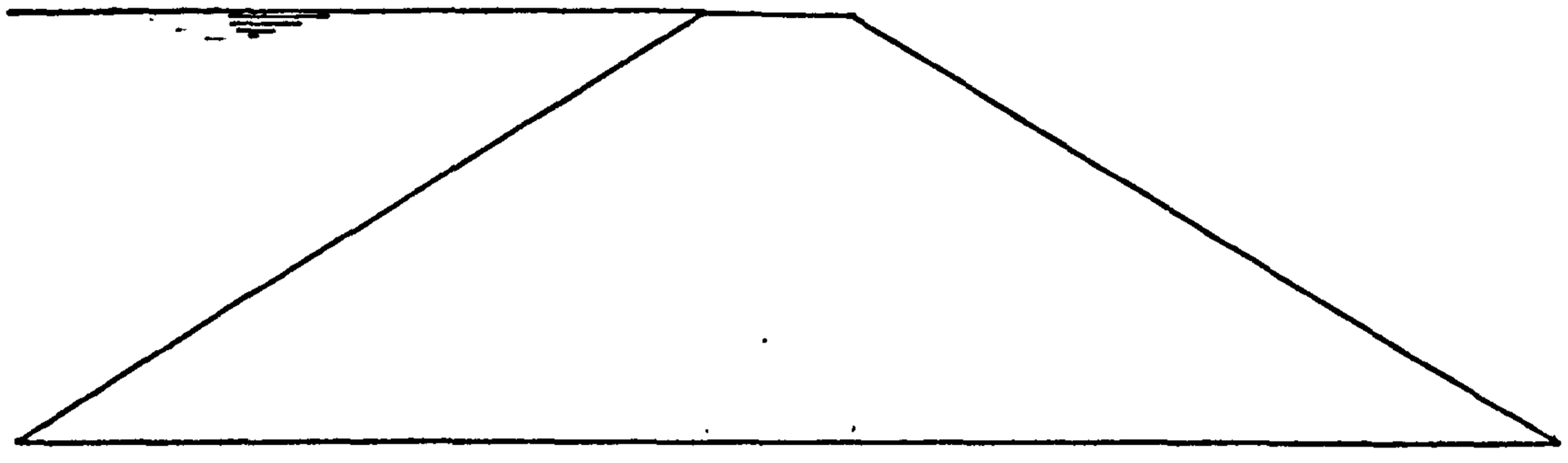
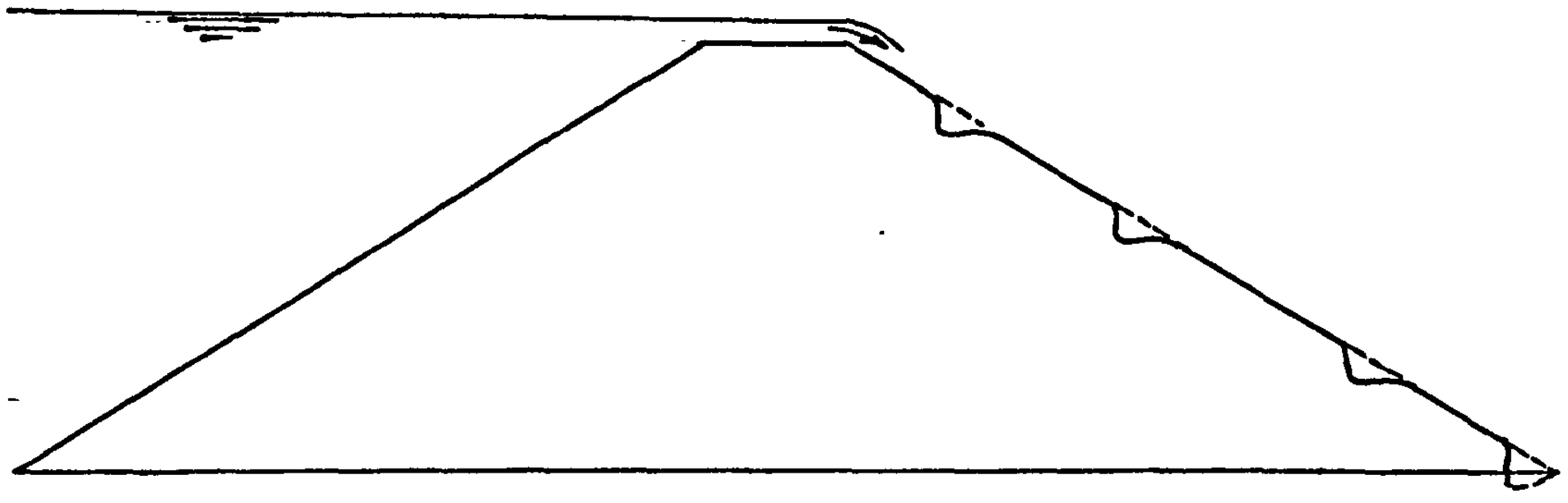


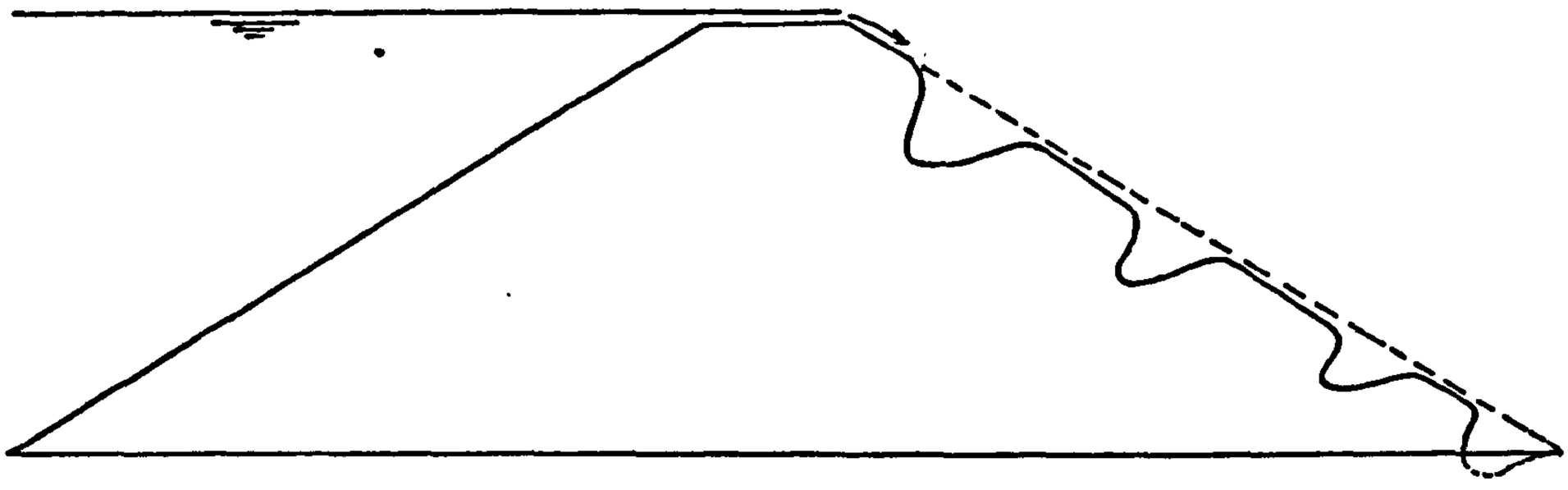
FIGURE 6.10 : 'Step' Formation



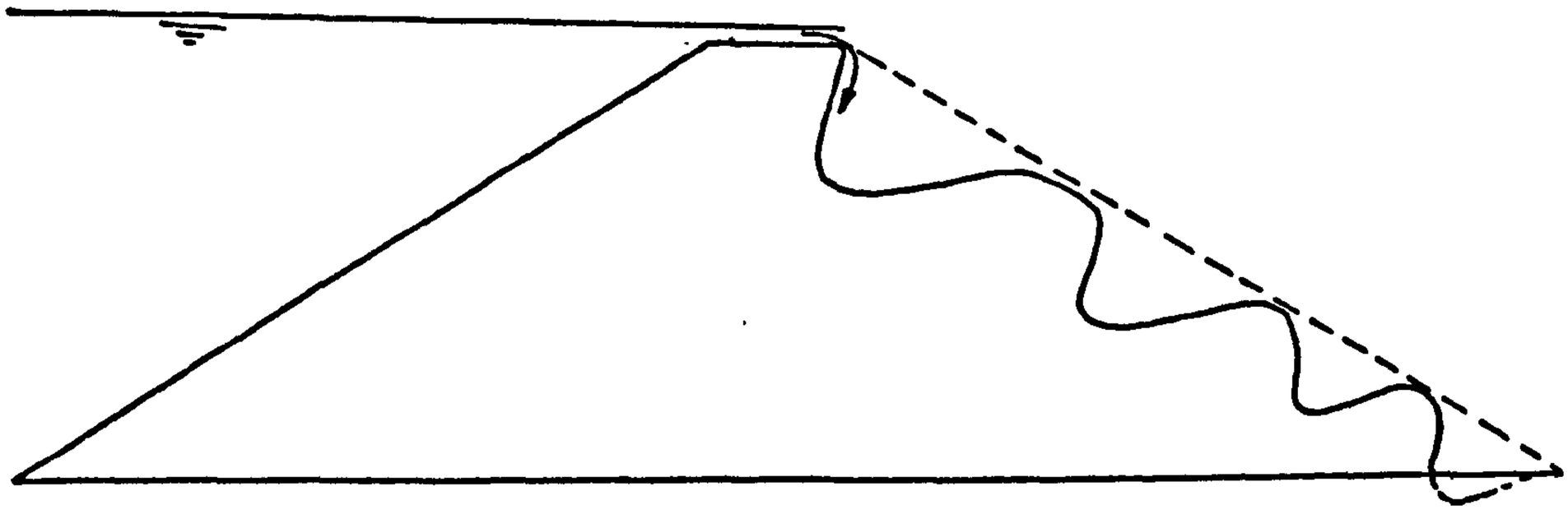
(1) Water approaches the crest



(2) Initial overtopping - Steps start to form

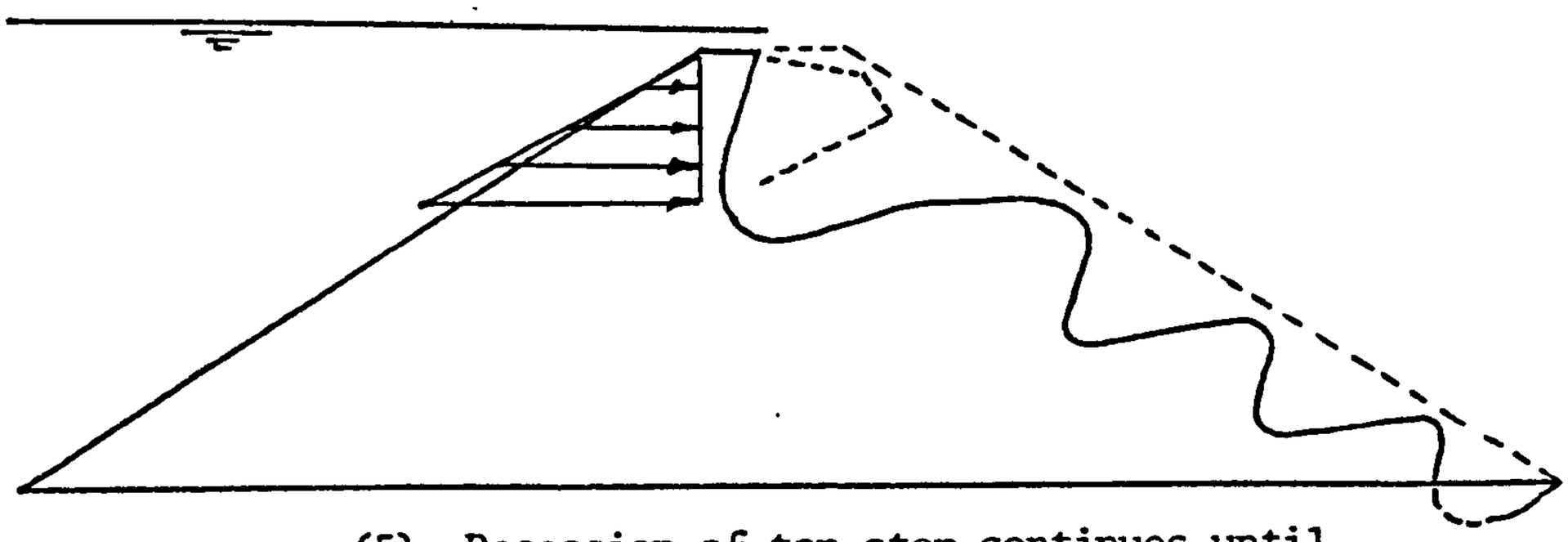


(3) Steps increase in size

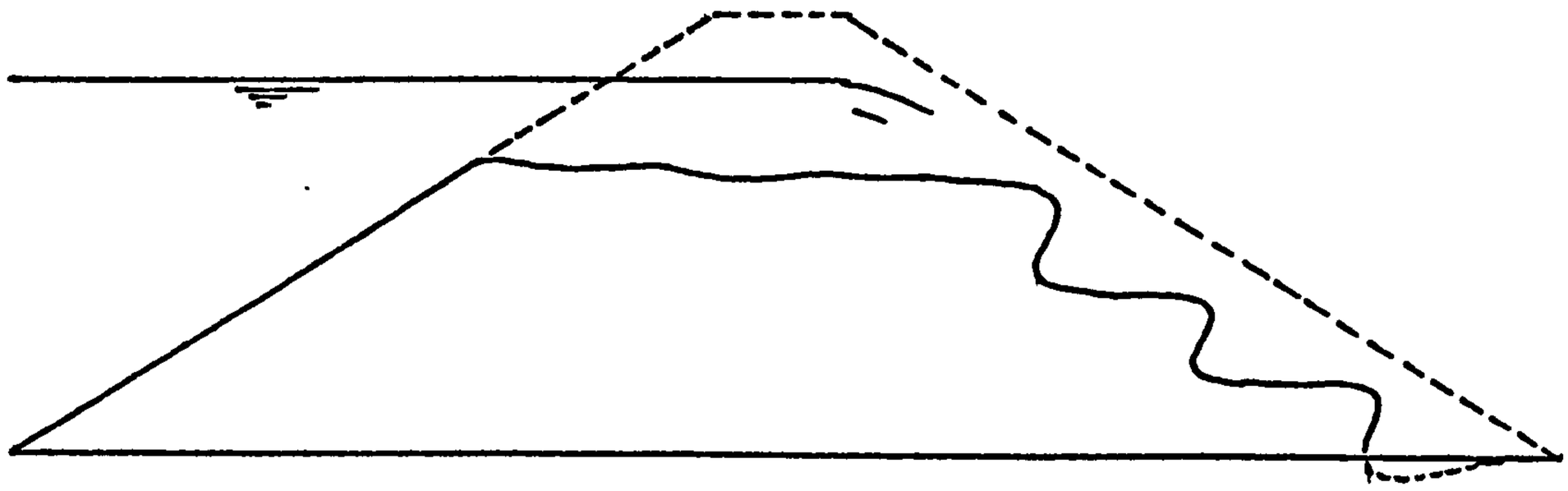


(4) Recession of top step

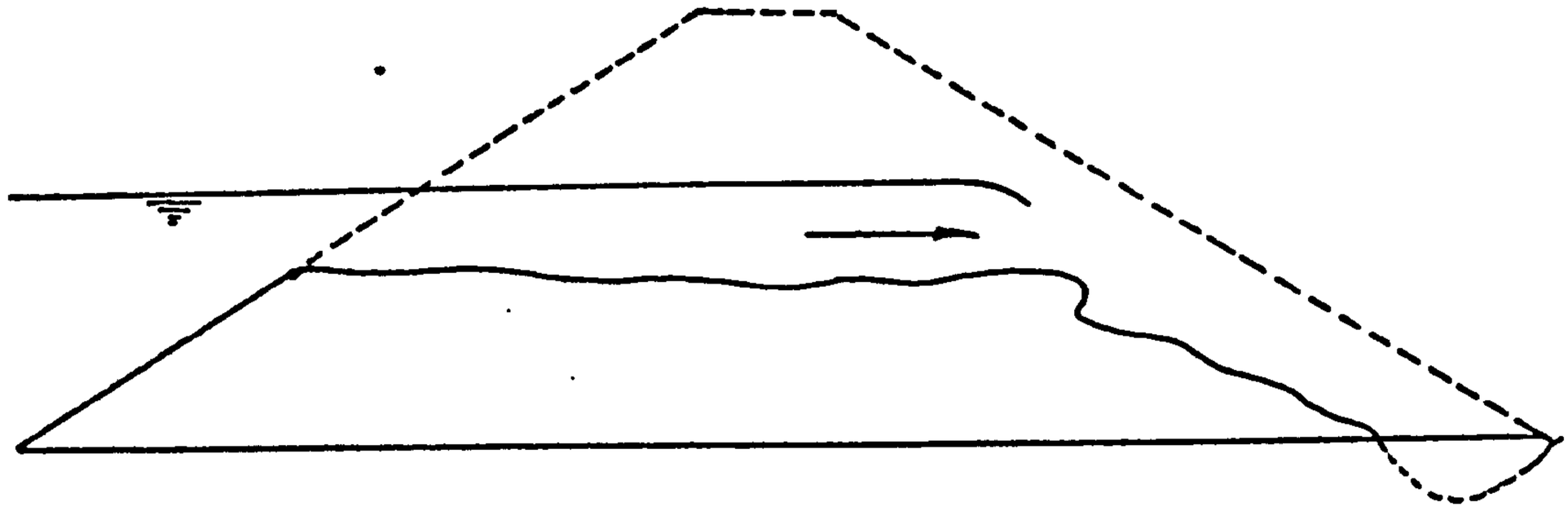
FIGURE 6.11a : Stages in the Development of a Breach



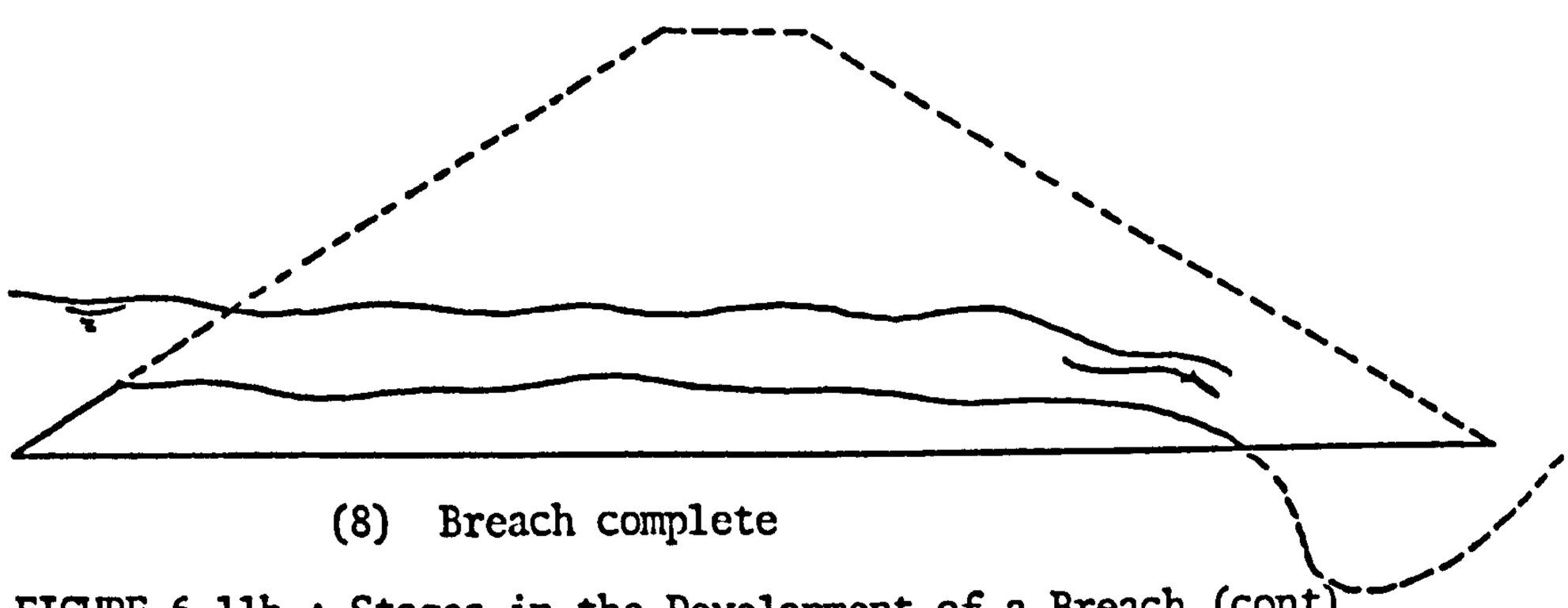
(5) Recession of top step continues until water pressure causes breach



(6) Continued horizontal erosion



(7) Erosion continues



(8) Breach complete

FIGURE 6.11b : Stages in the Development of a Breach (cont)

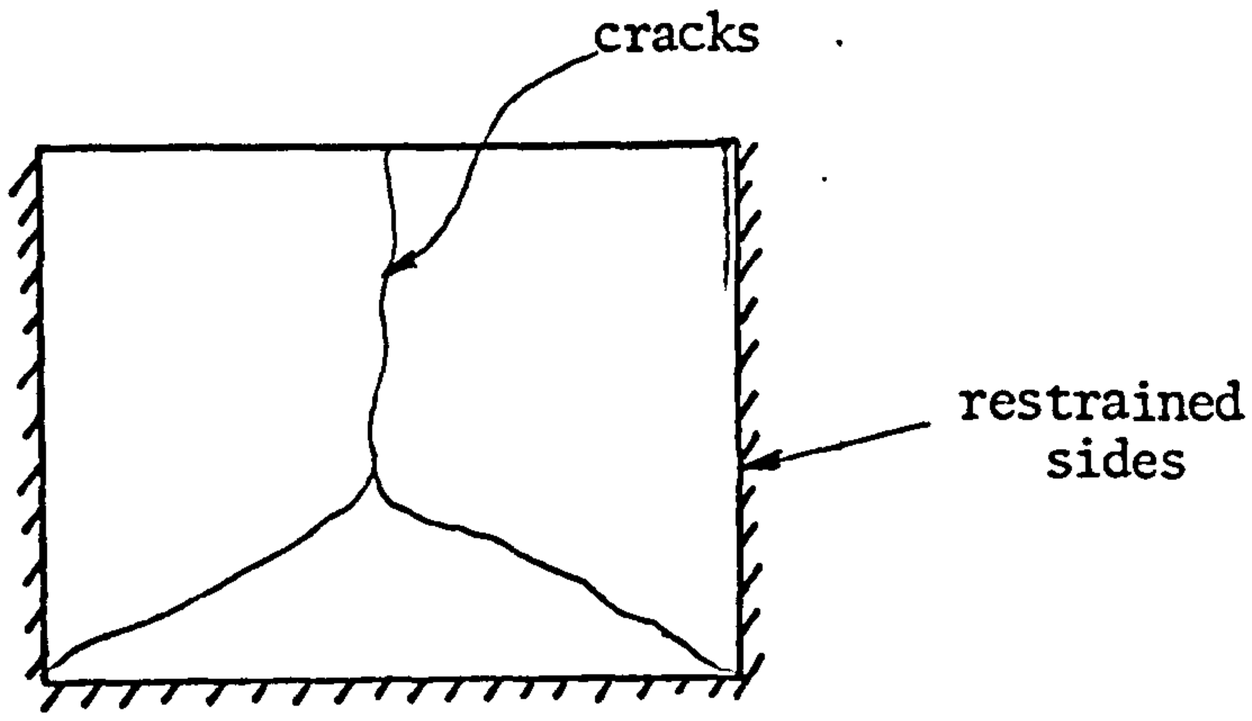


FIGURE 6.12 : Typical Cracking to Clay Core, Restrained on Three Sides when Subject to Overtopping

6.5 Effect of Temperature on Erosion

In the laboratory tests described temperature was made an independent variable by keeping the temperature of both the laboratory and the water constant; in actual fact, throughout the course of the Series I and Series II experiments the temperature of the water in the closed cycle experimental rig did not vary by more than ± 1° C from a nominal 21° C, the temperature being monitored by a thermometer installed in the downstream tank.

Theoretically, and in some circumstances practically, the capacity of a stream to carry suspended sediment is inversely linked to water temperature[192].

6.6 Effect of Suspended Solids on Erosion

Two tests of Series I were run using clays with the same

engineering characteristics and subjected to the same velocity of flow for the same period of time. One test run employed 'clean' water and the other 'dirty' water, i.e. water which contained a significant amount of suspended solids. The results, shown in Table 6.1 indicate that the concentration of the suspended solids of the eroding water has no significant effect on the rate of erosion, at least for concentrations up to 1.0 gm/litre of solids.

	velocity (m/sec)	amount of material eroded during test (kg)
'clean' water	2.86	1.954
'dirty' water	2.86	1.928 (mean of two tests)

TABLE 6.1 : Suspended Solid Tests

Although these results indicated that the concentration of suspended solids did not significantly affect the results of the tests, the experimental apparatus was always purged at high speed without samples, prior to any test run to disturb sediment which might have settled in the pipes, tanks etc. From a practical standpoint it was found necessary to change the water in the experimental apparatus after two or three experimental runs because of the difficulty experienced in observing erosion processes and taking photographs when the water contained higher concentrations of suspended solids.

6.7 Initiation of Erosion

One test, in Series I, was carried out to determine the velocity of flow at which erosion began. A sample was installed in the test

channel and flow rates were gradually increased by small increments of 2 litres per second, allowing sufficient time between increments to ensure that stable conditions were achieved.

Samples of the eroding water were taken at each stage in order that the amount of material eroded could be determined and the velocity at which erosion commenced calculated.

The graph of mean velocity against suspended solids is shown in Figure 6.13. The following observations and deductions were made, all velocities quoted being average values

(1) at 1.0 m/sec no motion of solids was observed

(2) at 1.4 m/sec some particles went into suspension but the concentration of suspended solids remained constant after a few minutes indicating removal of 'loose' particles but suggesting that erosion of the sample as a whole had not commenced.

(3) at 1.75 m/sec a slow but definite increase in the concentration of suspended solids marks the beginning of erosion.

(4) at approximately 2.2 m/sec a marked increase in the concentration of suspended solids indicated the commencement of mass erosion and the removal of flocs.

The results of this test are tabulated in Appendix F and it may be noted that for this clay, at least at the density and moisture content employed in this particular test, the velocity at which erosion began was somewhat higher than that suggested by Webber[175] and quoted in Section 5.2.

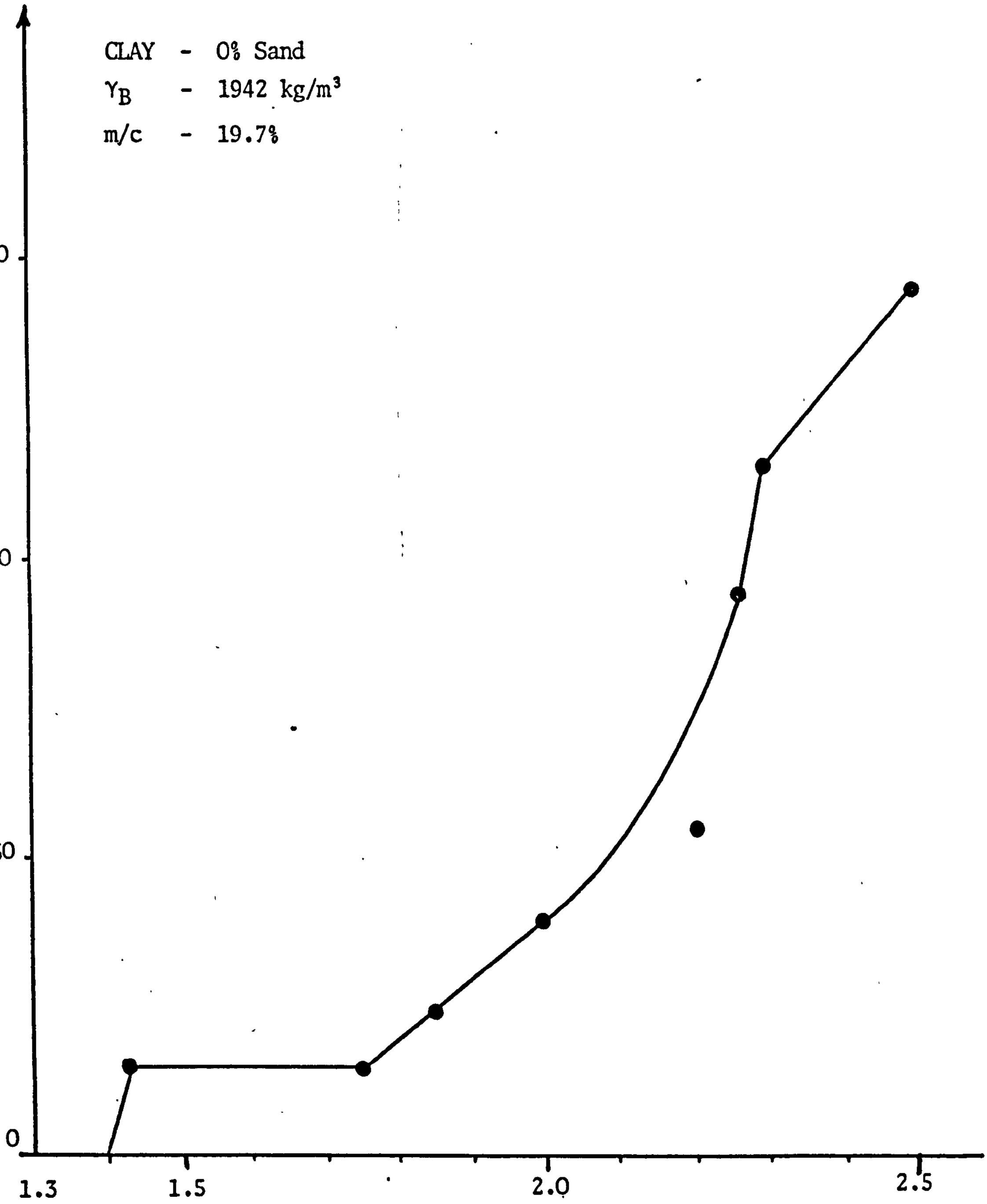


FIGURE 6.13 : Graph of Mean Velocity v. Suspended Solids Concentration

6.8 Swelling

Swelling may be said to have occurred in a soil when dimensions become enlarged and cohesion is diminished but the soil does not lose its apparent homogeneity.

It has already been established that the water in the multiphase structure of a soil consists of a fluid in an 'absorbed' and in a 'free' state. Absorbed water is that attracted by forces within the soil matrix strong enough to influence its behaviour, and consists of an inner layer immediately adjacent to the soil particles (adsorbed layer), and an outer layer which is partially bonded to the particles. Free water, on the other hand, behaves as a normal fluid with respect to flow etc.

A soil, by virtue of its composition, will attempt to imbibe water until it has sufficient water to fully develop its double layer. Swelling occurs as the soil takes in water, resulting in a decrease in cohesion as the interparticle distance increases until a maximum eventually occurs. Swelling results in the blocking or partial blocking of the larger pores and, as the flow through a pore is proportional to the fourth power of the pore radius[193], then appreciable decreases in permeability may result when the amount of swelling is large.

Swelling will also occur, due to movement of water to the surface of the clay particles when the concentration of salts is lower in the eroding fluid than in the soil pore fluid, since the osmotic potential of the eroding fluid is less than that of the soil solution.

As overtopping occurs the migration of water into the surface layers can lead to local swelling, resulting in movement of particles and a reduction in the bonds between particles which, coupled with the applied hydrodynamic stresses and turbulence generated by the eroding waters can lead to the removal of individual particles and flocs.

The log time v. swelling curves for the 'standard' clay and for clay containing 5%, 10%, 15% and 20% of sand by weight of the amount of clay in the sample are shown in Figures 6.14, 6.15, 6.16, 6.17 and 6.18.

All graphs show an immediate tendency to swell as soon as the water comes into contact with the soil.

It can be seen that the results for each series of tests show that the lower the moisture content of the bed material the greater the amount of swelling and the shorter the time to reach the same degree of swelling, or in other words decreasing the moisture content of a soil will increase the tendency of that soil to swell.

Only one test sample showed a tendency to shrink but this quickly reverted to a swelling, and may have arisen from insufficient care being taken to ensure that the sample was in contact with the porous base plate and the aluminium disc during installation.

It can be seen that all tests exhibited an initial rapid but fairly constant, rate of swelling, the rate increasing as the moisture content decreased. This was followed by a period when the rate of swelling reduced gradually to zero.

Figure 6.19 compares the swelling of all the 'modified' soils at various moisture contents, with the amount of clay in the sample tested. A linear relationship between the amount of swelling and the

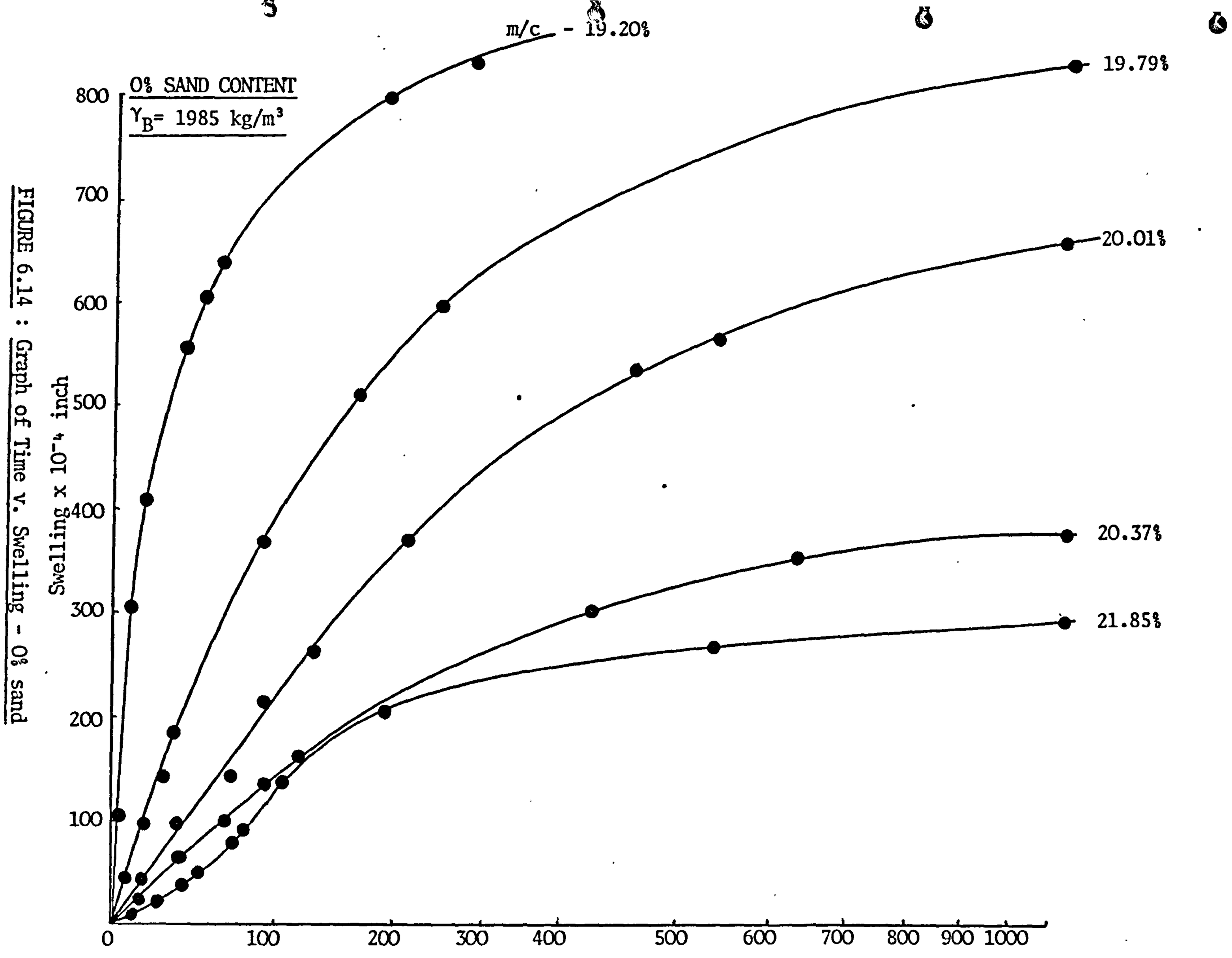


FIGURE 6.14 : Graph of Time v. Swelling - 0% sand

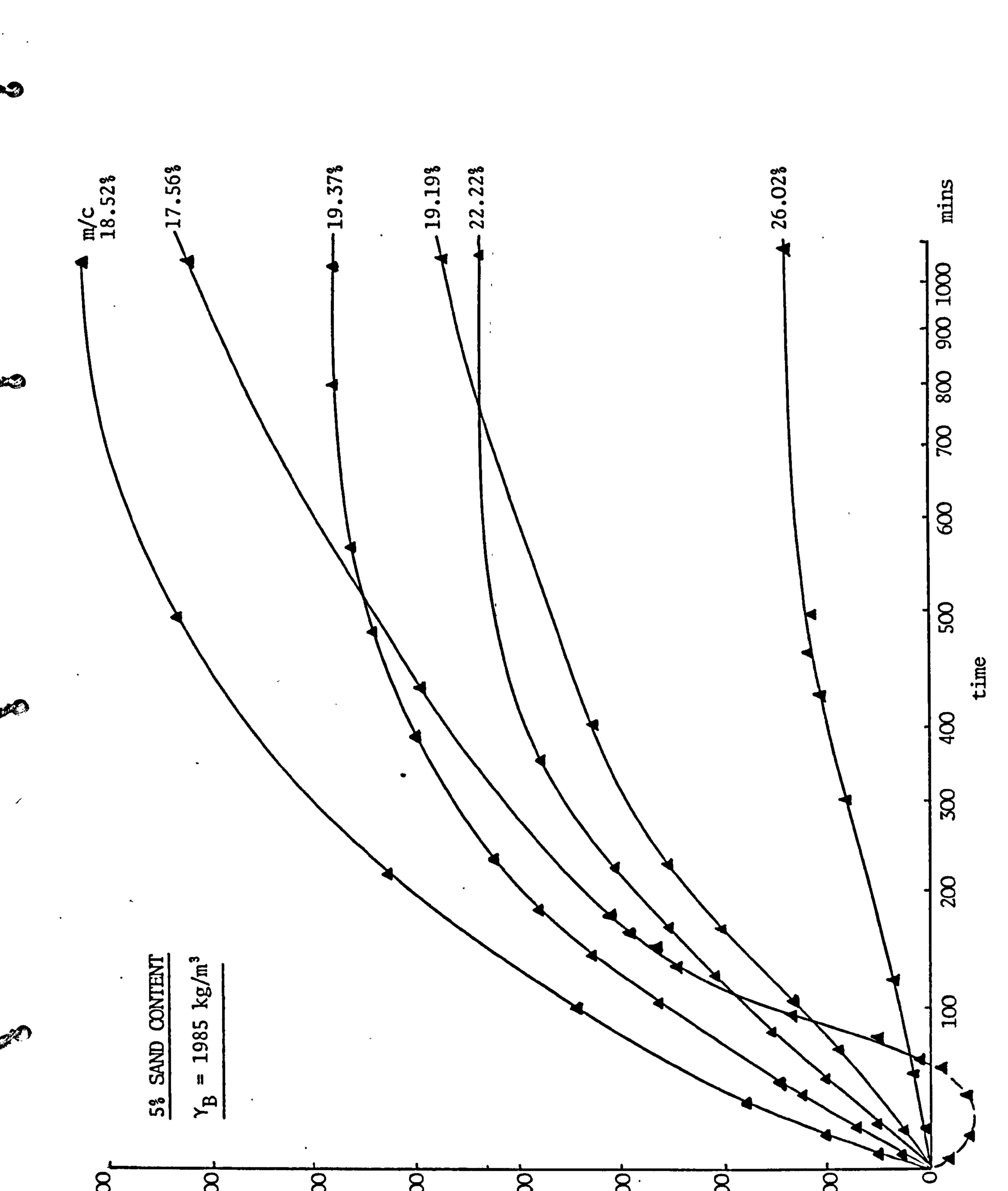


FIGURE 6.15 : Graph of Time v Swelling - 5% sand

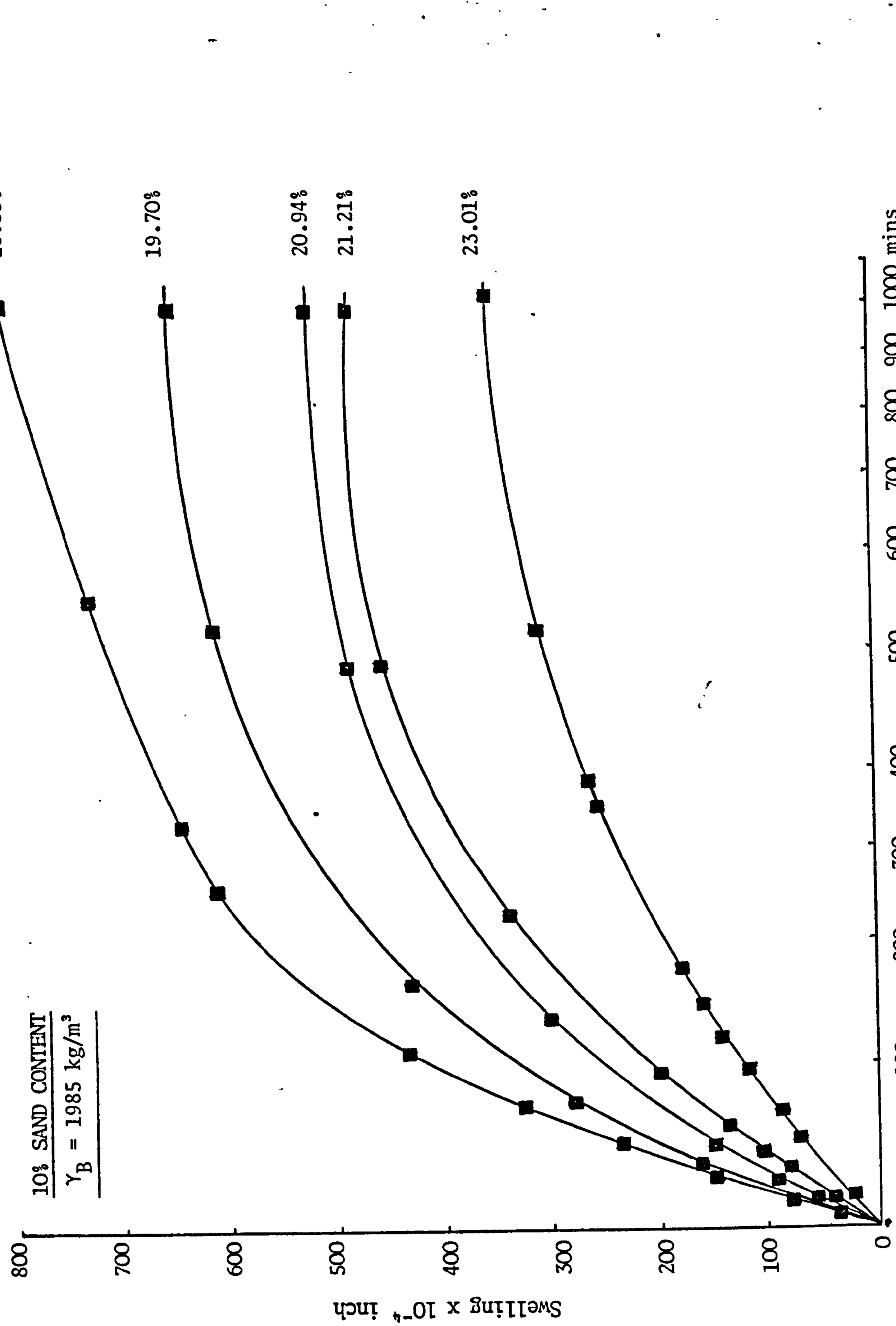
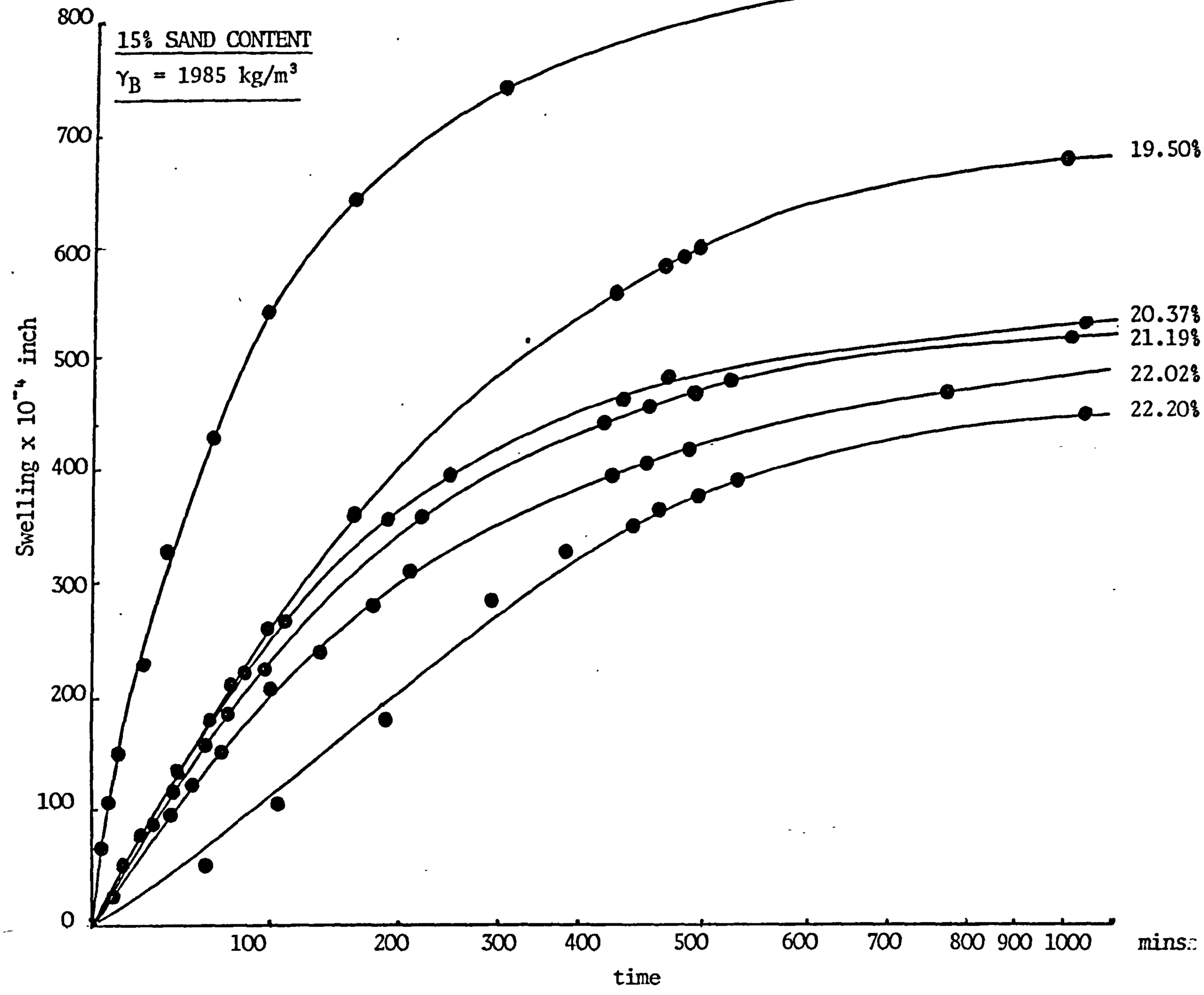


FIGURE 6.16 : Graph of Time v. Swelling - 10% sand

FIGURE 6.17 : Graph of Time v. Swelling - 15% sand



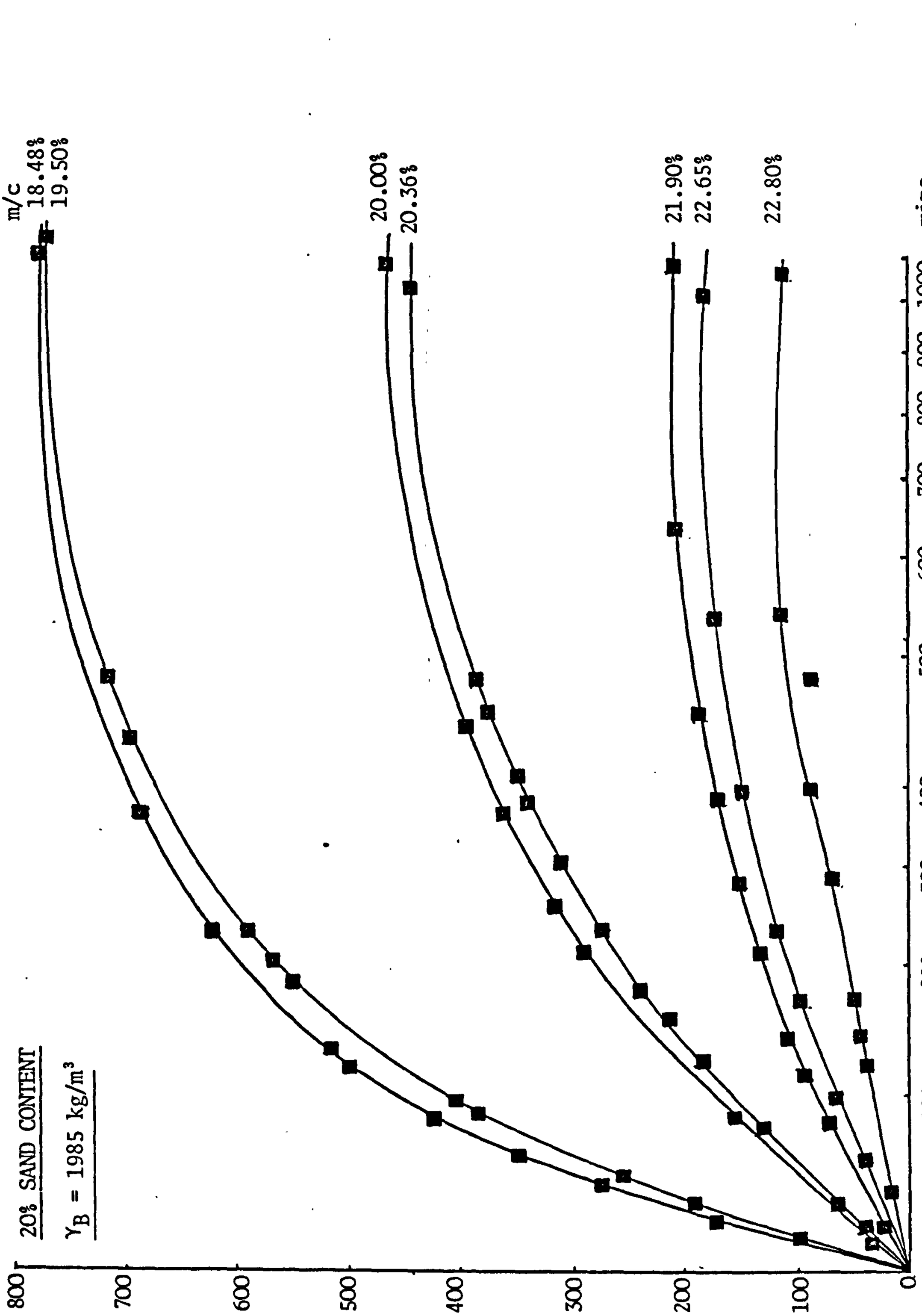


FIGURE 6.18 : Graph of Time v. Swelling - 20% sand

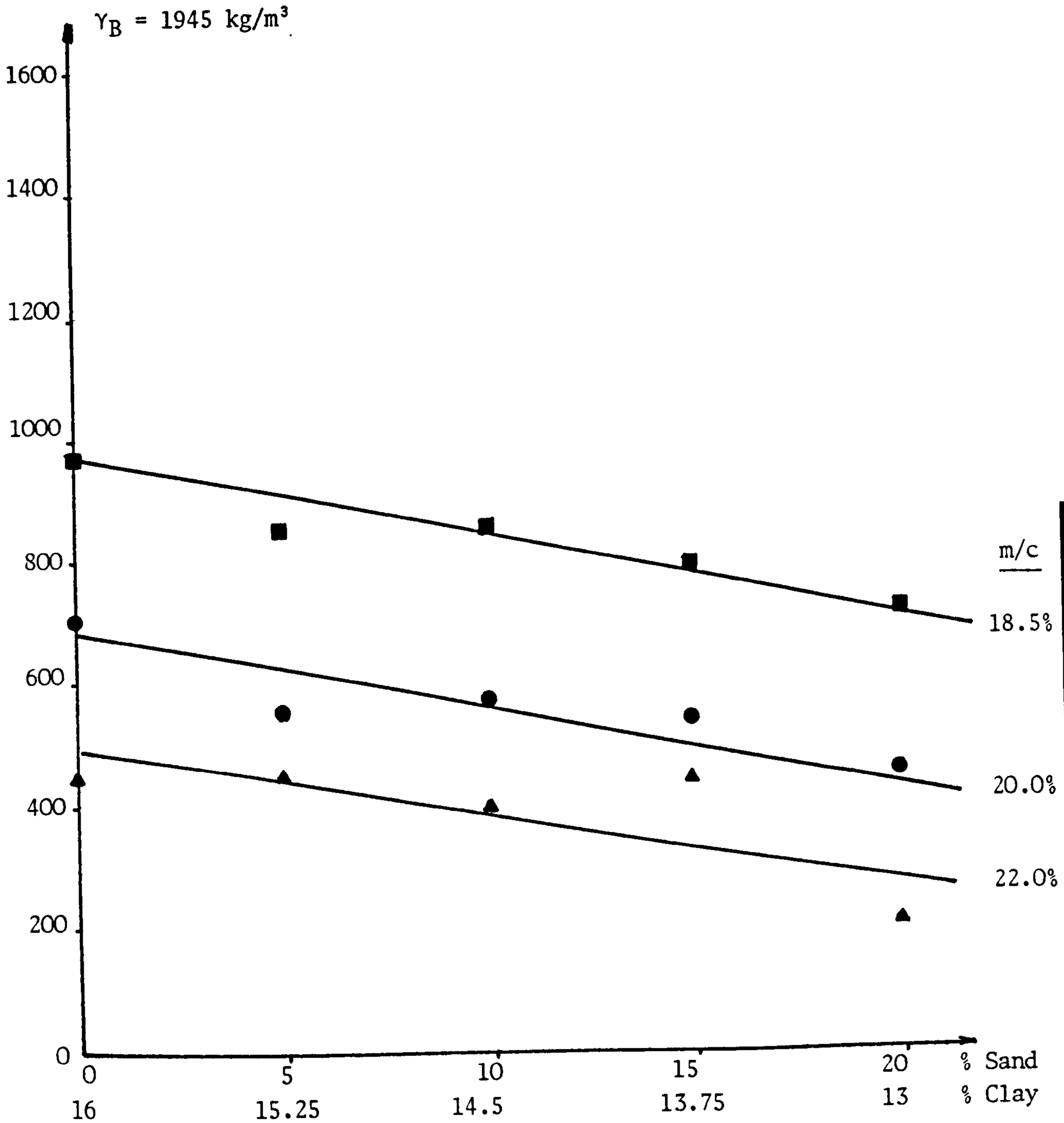


FIGURE 6.19 : Graph of Percentage of Clay v. Swelling

percentage of clay in a soil is indicated, dependent on the moisture content. The results plotted in Figure 6.19 have been adjusted and plotted against the percentage of clay in the mix, the latter obviously decreasing as the percentage of sand added to the soil increases. The result obtained is similar to that reported by Keen and Raczkowski[194], insofar as a linear relationship was obtained between swelling and the percentage of clay in a soil, but Keen and Raczkowski's results showed no dependence on moisture content as shown here in Figure 6.20.

Swelling may cause slight changes in particle orientation; small rotations of the particles occur and hence the particle arrangement will deviate from a generally horizontally dispersed structure to a less dispersed form.

6.9 Effect of The Chemistry of the Eroding Water

It is clear that the chemistry of the eroding fluid can effect the erosion process since, if the salt concentration of the eroding fluid is less than that of the pore fluid of the soil. An initially flocculated structure will change to a dispersed structure, perhaps leading to a marked reduction in attractive forces between particles and thus to reduced resistance to erosion[195].

In general, erosion can result from a decrease in or an exchange of, pore fluid-cations which reduces the magnitude of interparticle forces. In addition, research has shown that the lower the content of dissolved salts in the water the greater the susceptibility of the clay to erosion[195].

However, to keep the chemistry of the eroding fluid an independent variable all tests were run with mains water so that a comparative

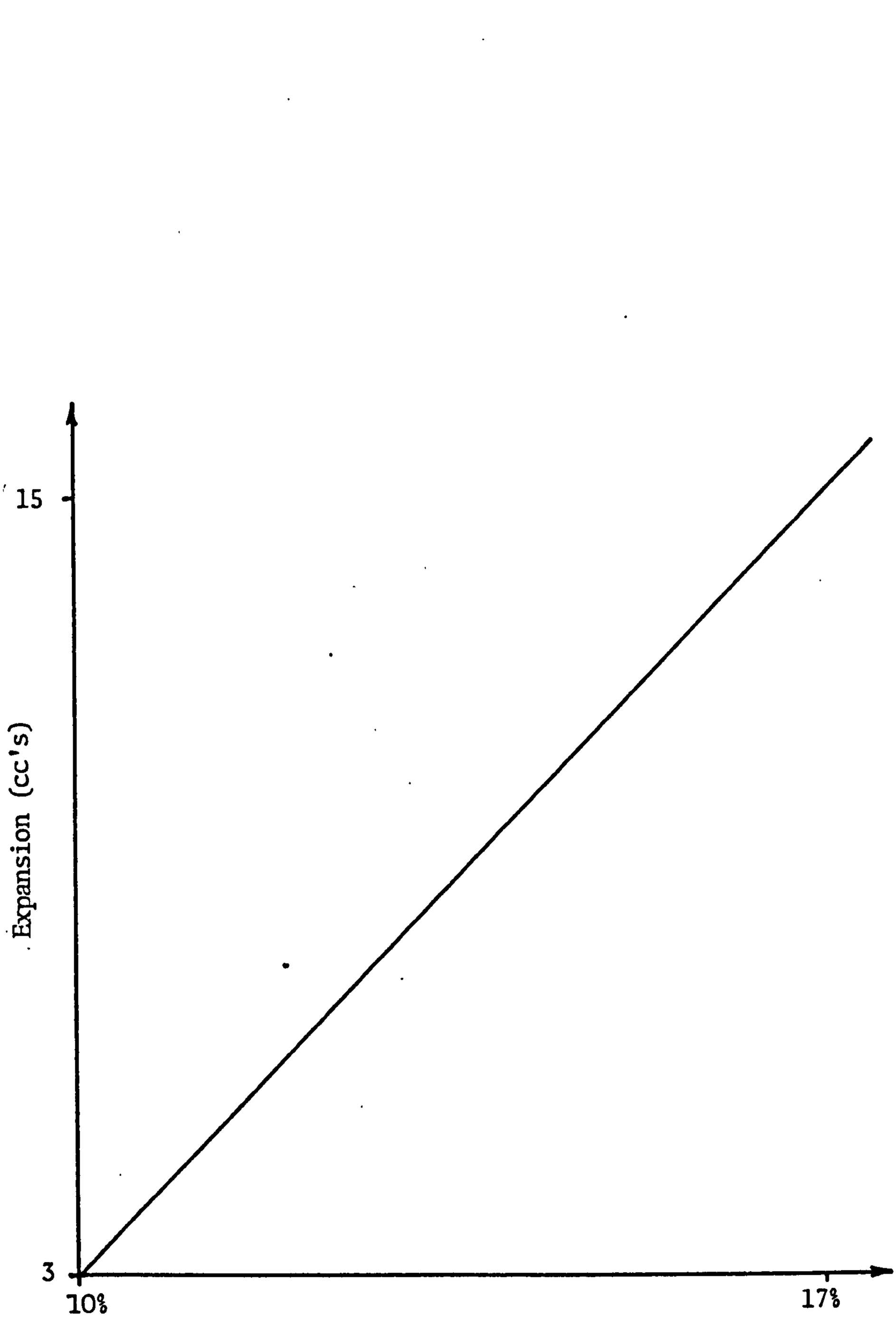


FIGURE 6.20 : Graph of Percentage of Clay v. Expansion

(after Keen & Raczkowski[194])

study could be made.

Chemical analyses of the mains water before and after tests were carried out and showed a slight increase in the chloride content of the water as shown in Table 6.2

chloride content (mean values)	
before test	after test (after 2 tests)
34 mg/1	55 mg/1

TABLE 6.2 : Chloride Contents of Eroding Fluid

In addition a slight increase in the pH of the water was apparent as shown in Table 6.3

pH of eroding fluid	
before test	after test (after 2 tests)
7.3	7.7

TABLE 6.3 : pH Values of Eroding Fluid

6.10 Suspended Solids

The concentration of suspended solids was found for each test by the methods outlined in Section 5.8. The results of several tests are shown in Figures 6.21, 6.22, 6.23, 6.24, 6.25, 6.26 and 6.27. It will be noticed that most of the curves are of the same general

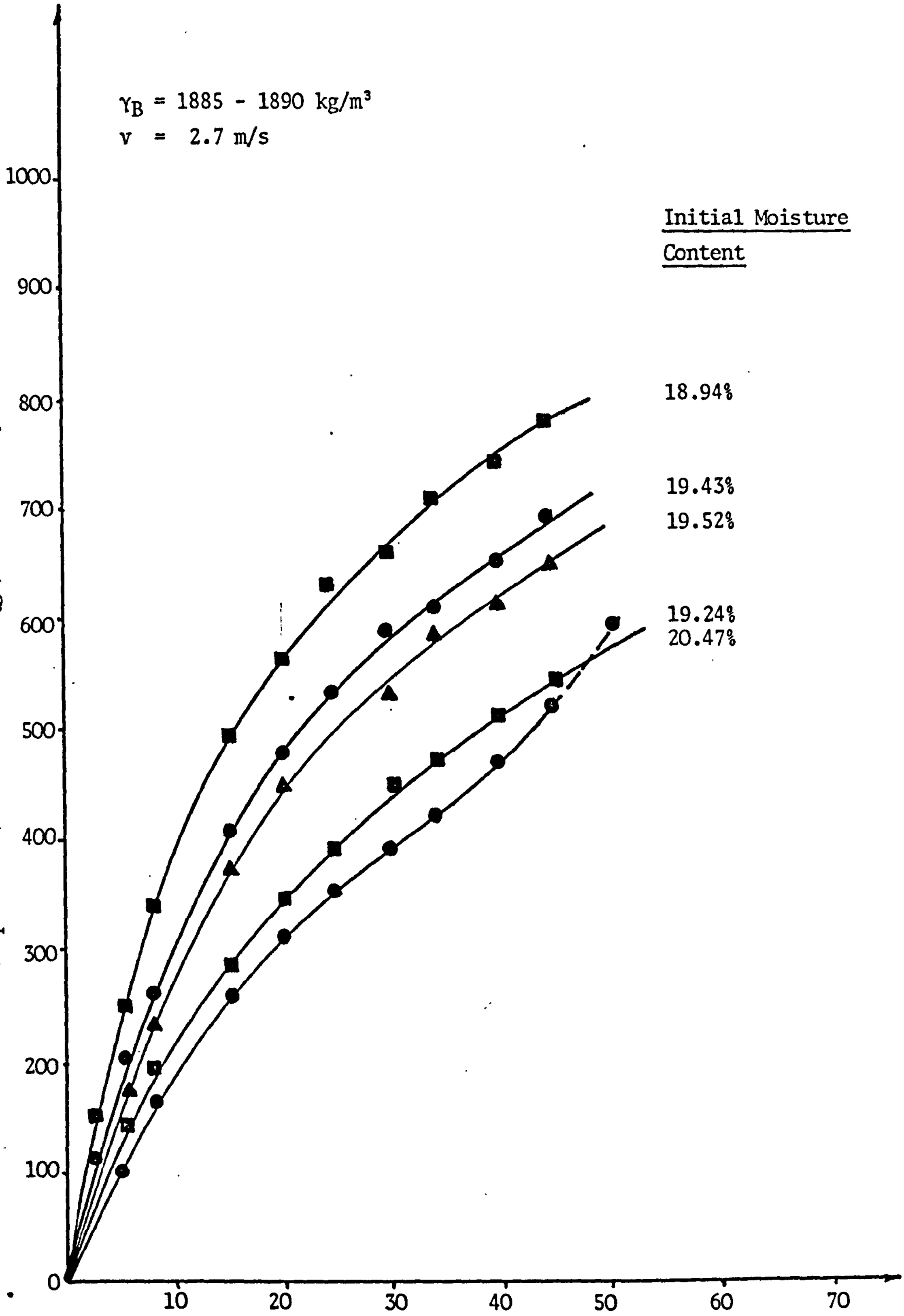


FIGURE 6.21 : Graph of Suspended Solids v. Time

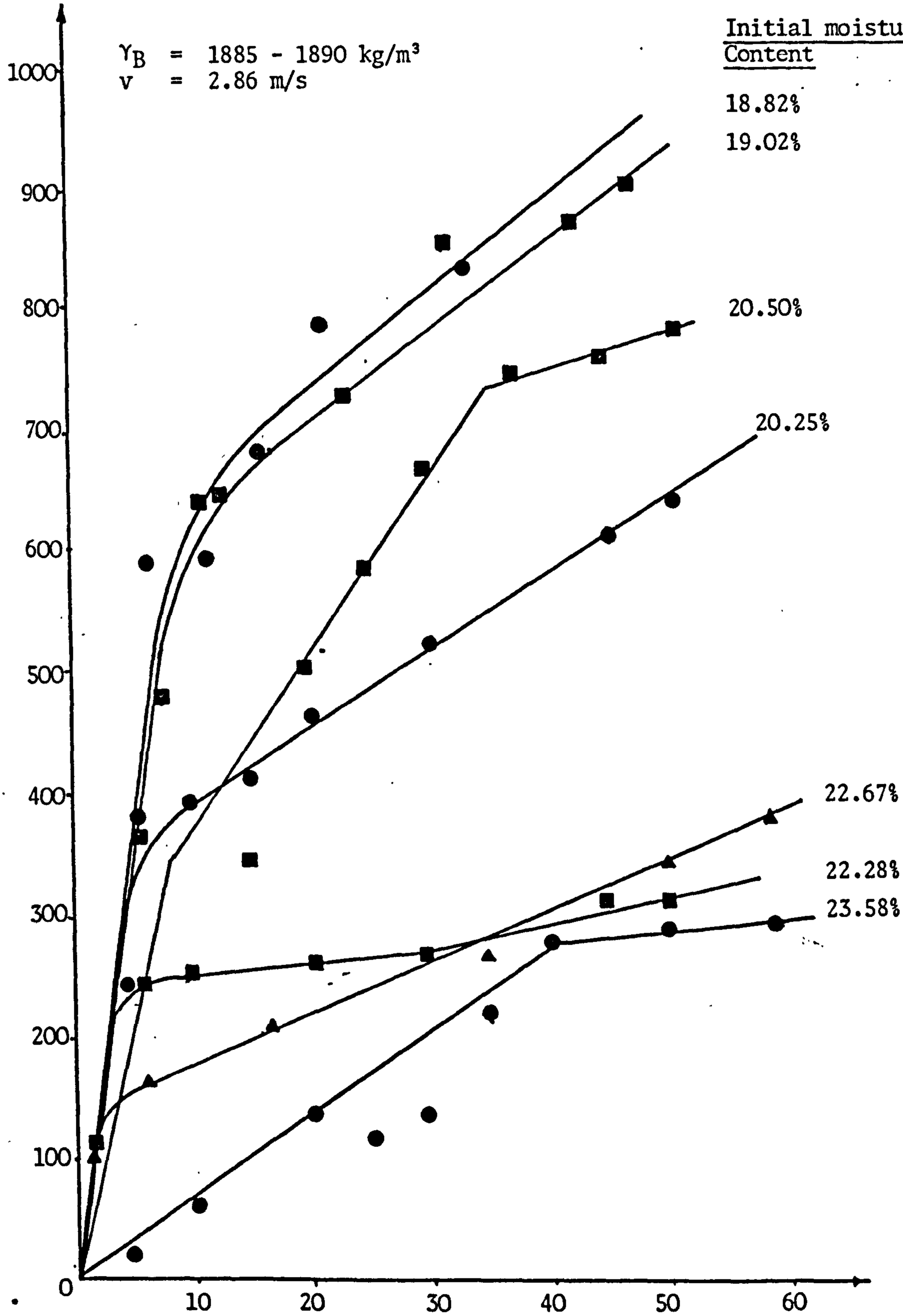


FIGURE 6.22 : Graph of Suspended Solid Concentration v. Time

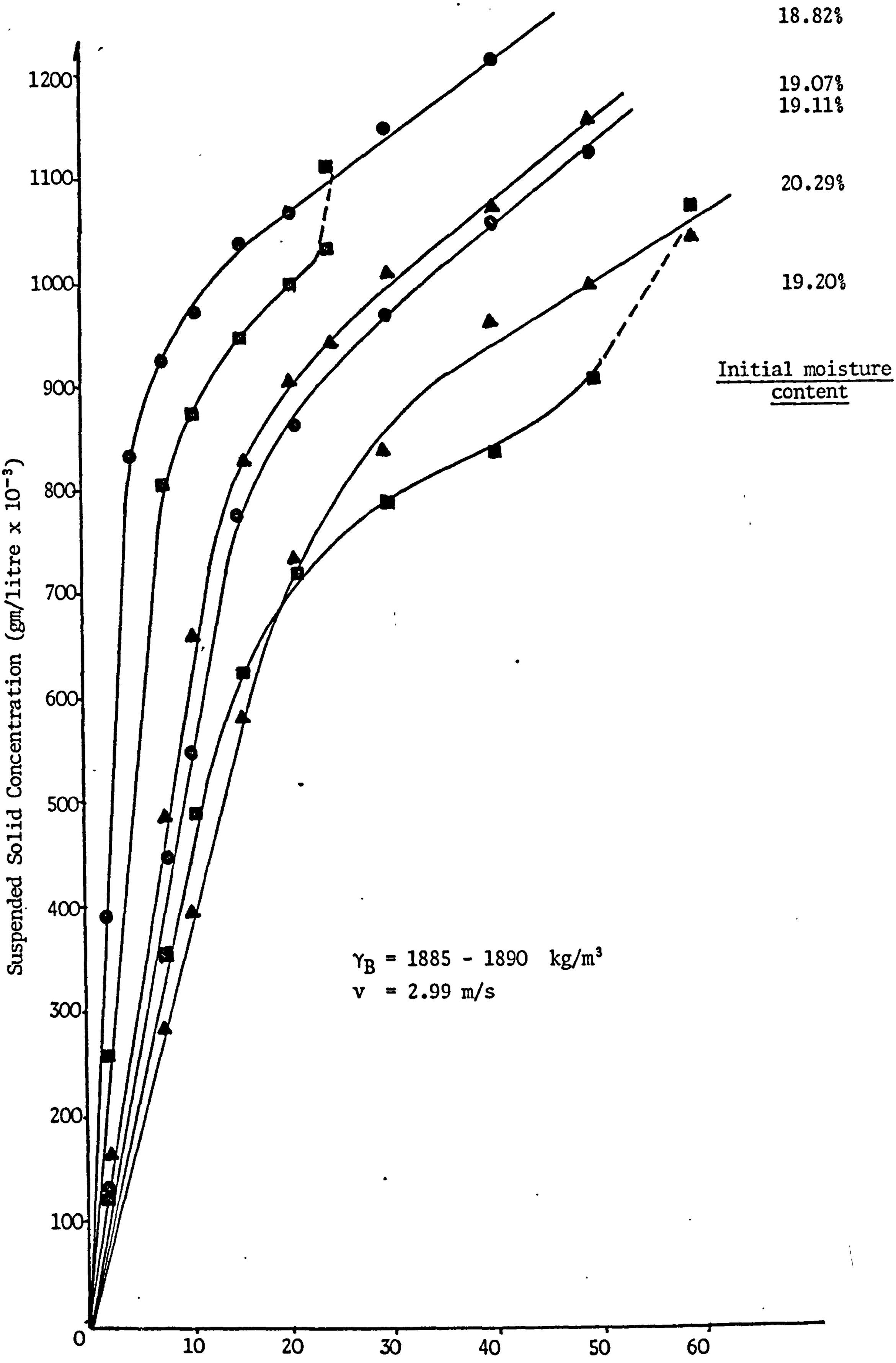


FIGURE 6.23 : Graph of Suspended Solids v. Time

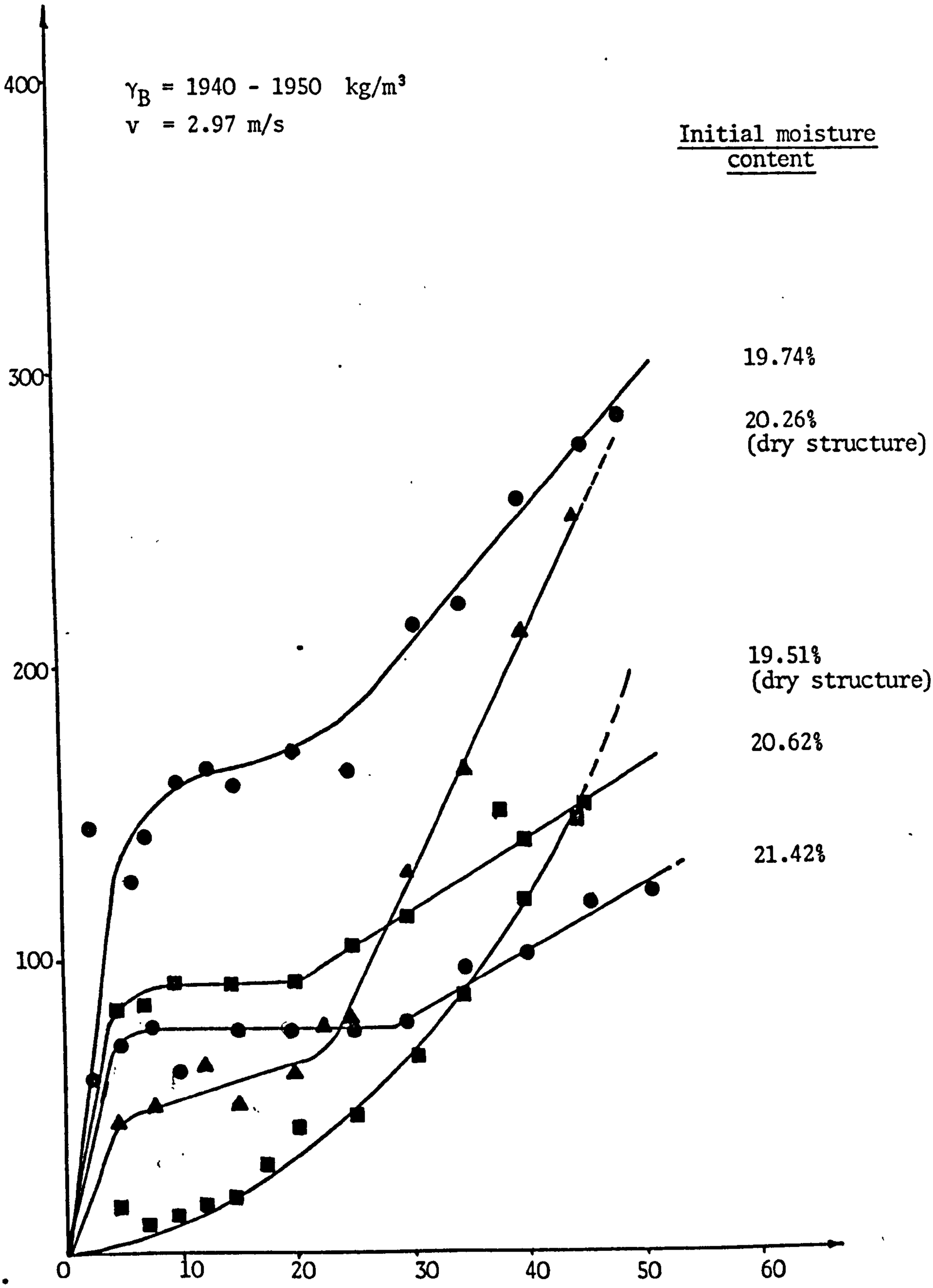


FIGURE 6.24 : Graph of Suspended Solid Concentration v. Time

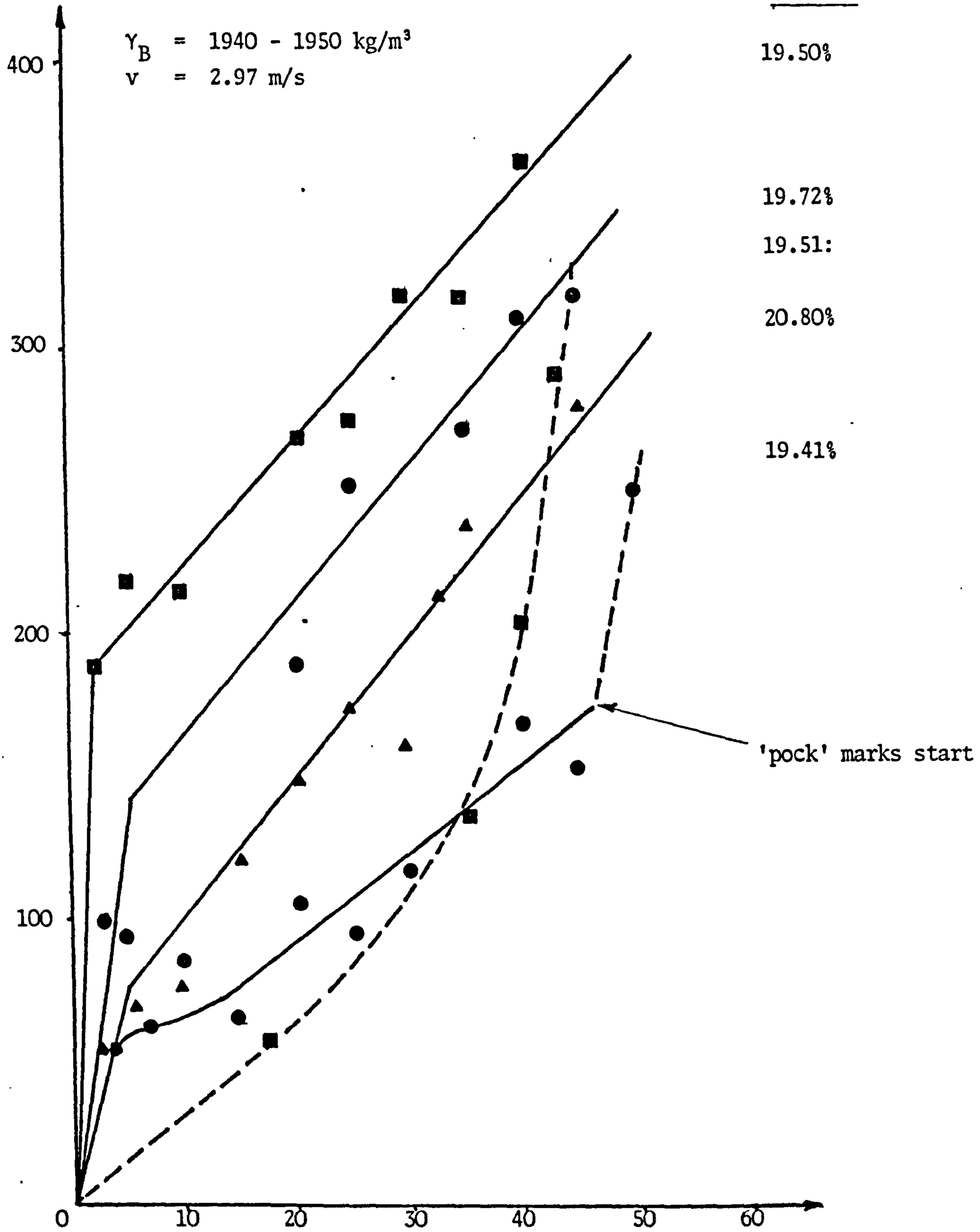


FIGURE 6.25 : Graph of Suspended Solid Concentration v. Time

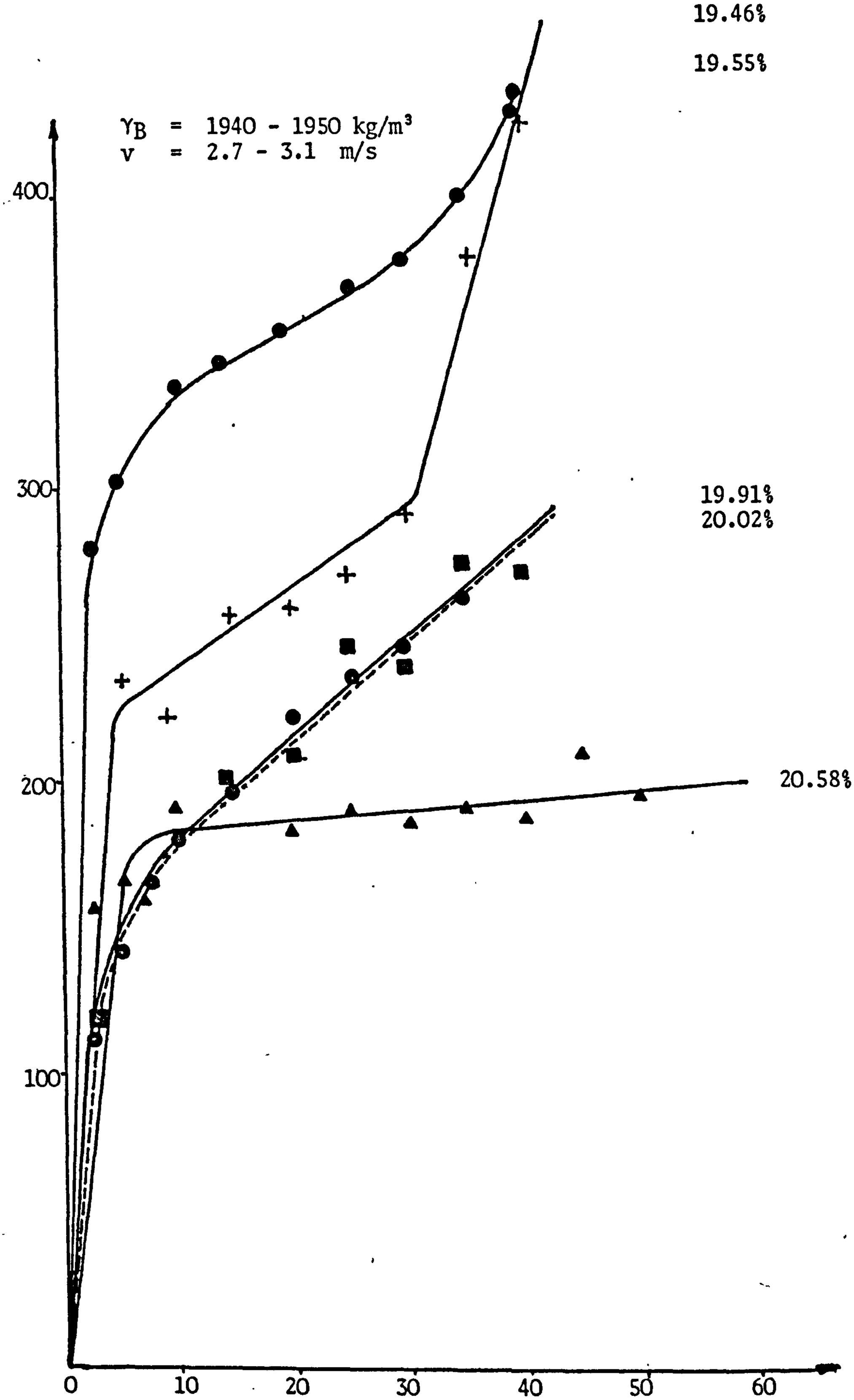


FIGURE 6.26 : Graph of Suspended Solid Concentration v. Time

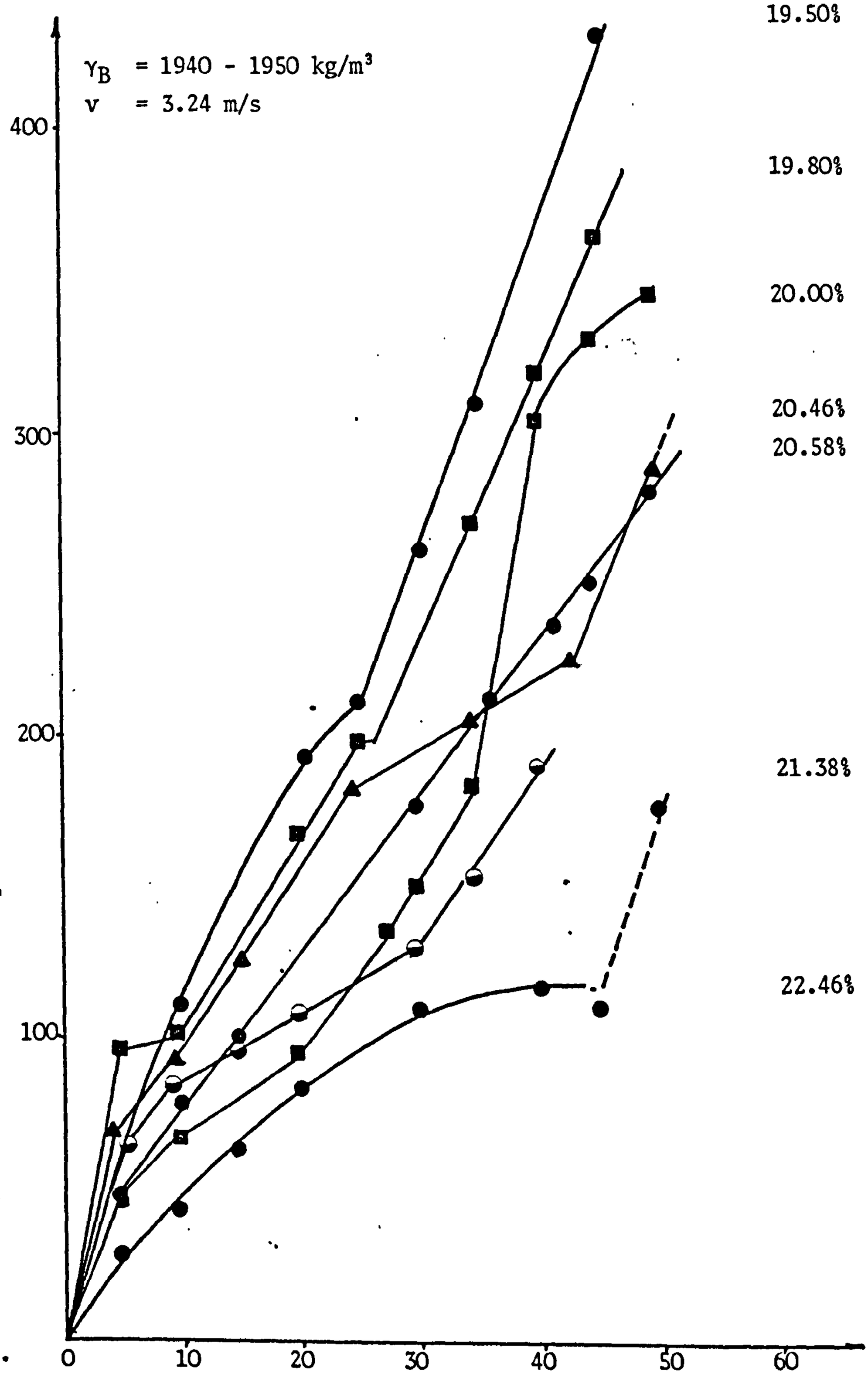


FIGURE 6.27 : Graph of Suspendid Solid Concentration v. Time

form, consisting of a steep initial phase, indicating a rapid, constant erosion rate and an increase in the concentration of suspended solids, followed by a second flatter phase.

The straight line relationship again indicates a constant but lesser rate of erosion. The examples shown which do not follow this pattern will be discussed in detail later in this section.

The general form of the graphs obtained is slightly different from those obtained by other research[195], in that earlier results were of the form shown in Figure 6.28 with an initial section at a constant erosion rate followed by a reduction in erosion and a tendency to a steady state condition where no subsequent erosion took place.

A typical result from the present investigation is shown in Figure 6.29; it must be remembered, however, that the time scale for Figure 6.28 was much larger than that for Figure 6.29. However, evidence from Series II experiments suggests that a steady state condition would not occur on the downstream face of a dam. The various sections indicated in Figure 6.29 may be explained in the following manner:-

1. Section 1 - 'clouding' of water due to material carried in suspension - dispersion of platlets of clay as water is absorbed - swelling - loss of adhesion and reduction in cohesion - erosion of the surface as a whole.

2. Section 2 - slight reduction in the rate of erosion as 'local' areas are eroded rather than the full surface of the bed, coupled with increased turbulence.

3. Section 3 - if this final section appears it

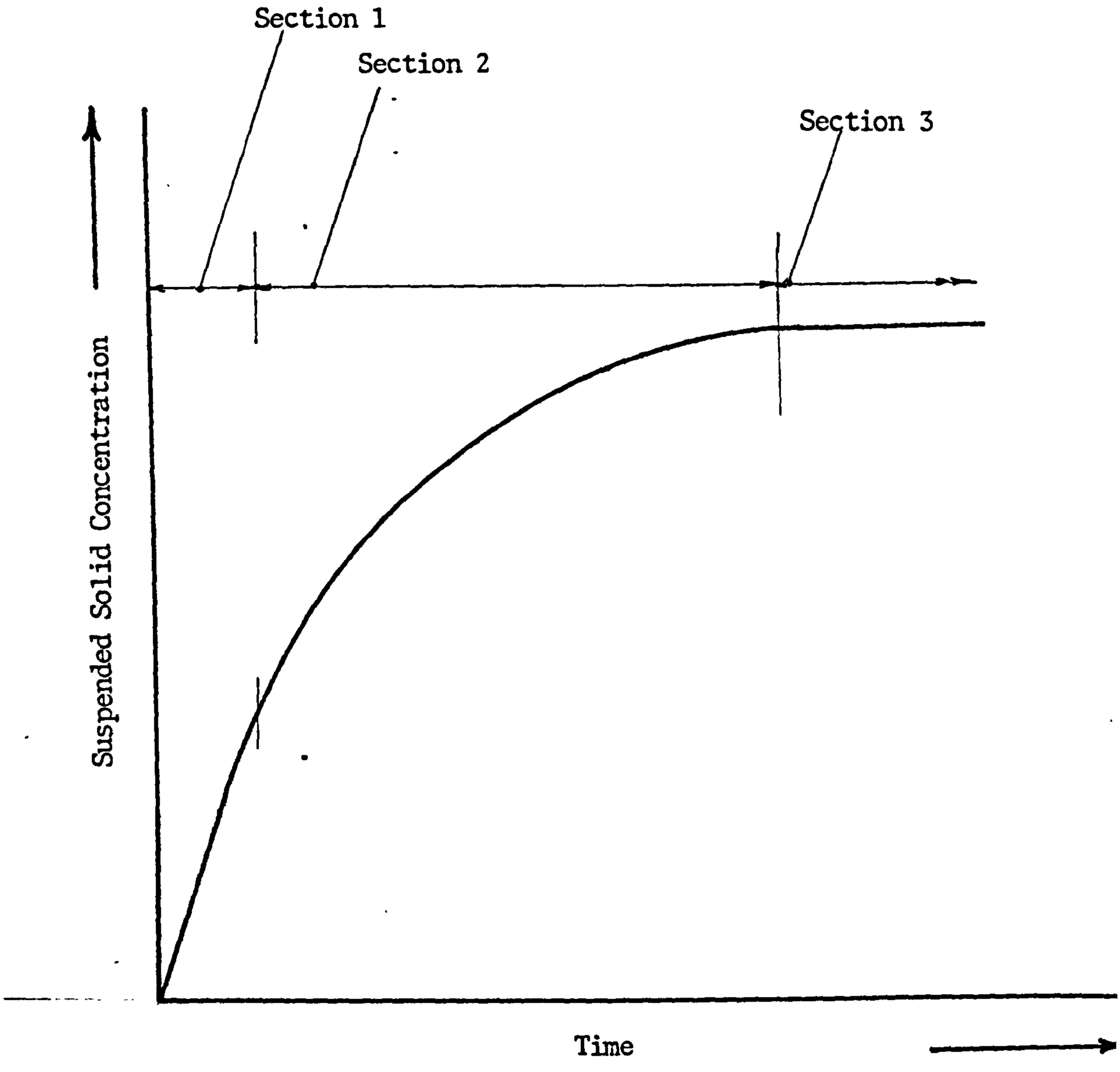


FIGURE 6.28 : Typical Suspended Solid v. Time Graph

(after Arulanandan)

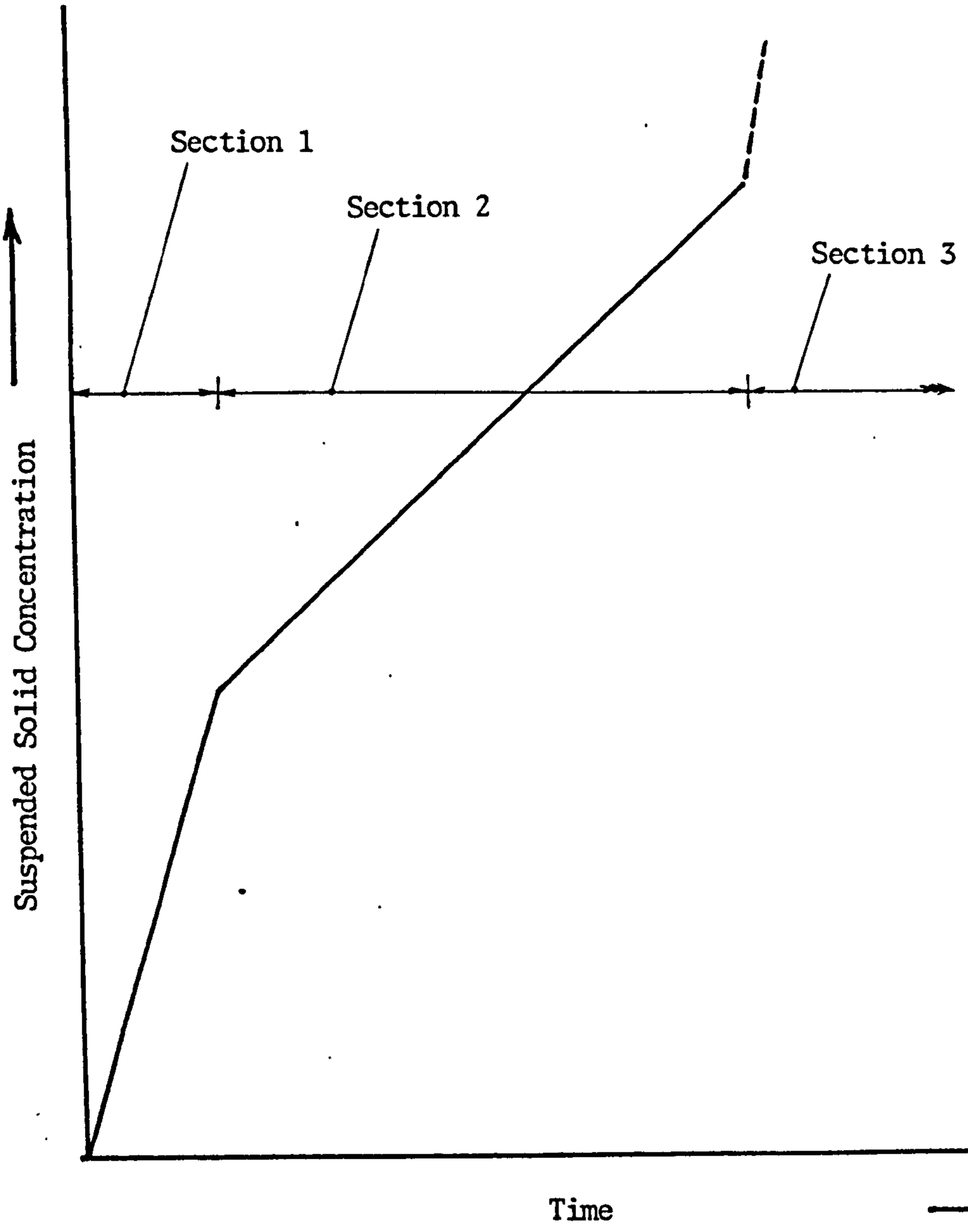


FIGURE 6.29 : Typical Suspended Solid v. Time Graph

signified an increase in the rate of erosion, caused by the detachment of flocs and/or mass erosion, accompanied by increased turbulence.

Tests run with soils having an initial molding moisture content of less than about 20% resulted in mass erosion and disintegration of samples.

6.10.1 Unusual Forms of Suspended Solid v. Time Graphs

Figures 6.24 and 6.25 in particular present curves which indicate a tendency to an 'infinite' rate of erosion. This was due to mass erosion of the bed samples from the start of the experimental run attributable, in some early tests, to non-uniform compaction causing areas of low density and low moisture content.

6.11 Effect of Permeability on Erosion

The amount of water absorbed by a soil is influenced by the degree of mechanical compaction and the soil type and structure. Lutz[196] states that high permeability leads to rapid absorption and that the quantity of water absorbed by a soil is affected by the amount of organic material present and by the swelling properties of the soil colloids.

It may be argued that if a soil is permeable then some water is taken in through the open pores, and less is available for run-off. This may be significant where the total amount of water involved is small but in the case of overtopping flows the amount of water taken in by the soil would be negligible. If, however, a soil has low permeability, then the amount of water absorbed is relatively small and the consequent decrease in cohesion small, other than at or near the surface exposed to water.

6.12 Effect of Compaction on Erosion

It is clear that compaction and the dependent variable, moisture content, are parameters which will affect erosion rates, a 'loose' material being more easily eroded than a compacted material.

6.12.1 Historical Development

Very little is known about compaction, construction methods and materials before 1902, although it is known that before 1800 compaction was carried out by hand or foot. By 1820 cattle and sheep had been used in Britain, and by 1850 heavy concrete and metal rollers had been used in Europe. Sheepsfoot rollers had been developed for use on roads in 1904 but it is thought that these rollers were not used on dams until 1907.

It is evident that continuation of many older embankments has relied on the compaction of layers, usually about 0.3 m thick, by horses and wagons and, in some instances, by steam rollers with gross weights of 10 - 20 tonnes. By 1930 most engineers positively recommended compaction, mainly by the use of sheepsfoot rollers, to give bond between successive layers of fill. In 1933 Proctor strongly endorsed the use of sheepsfoot rollers because of the high unit pressures and kneading action they achieved and a large number of failures of embankment dams built between 1900 and 1920 in the United States of America are believed now to have been the result of insufficient compaction. The introduction of limit-equilibrium stability analysis theories and of Proctor's control procedures, or derivatives thereof, have resulted in fewer failures of this type.

Most of the failures referred to above have arisen from excessive settlement attributable to insufficient compaction, but the

results of the experimental programme will show that the erosion characteristics of a clay are also affected by the degree of compaction attained and by the moisture content.

6.12.2 Experimental Results

The surfaces of flat 'platelets' and sheet minerals associated with clays and micas display considerable repulsion which resists compaction, and it is also known that density and moisture content for any soil are inter-related. A knowledge of one consequently predetermines the value of the other, for a given state of compaction.

Soils which have been deposited and then compacted in-situ, or natural deposits which are subsequently artificially compacted, have structural characteristics significantly different from those of entirely natural soil deposits. Mechanically compacted soils lose any natural interparticle bonding that occurred in the parent material, their entire strength depending on subsequent bonding that may occur as the deposit is compacted.

Each result presented in the preceding and subsequent sections is accompanied, and qualified by, values of bulk density. It is apparent from the results that density of the soil plays an important part in determining the amount of erosion likely to occur.

The moisture content of the bed material was determined prior to and on completion of each test in Series I. The results of several tests are illustrated on Figures 6.30 and 6.31. It will be noted that as the density of the soil increases the uptake of water by the soil and the final moisture content relative to the initial moisture content decrease.

The final structure of a soil depends to a large extent on the

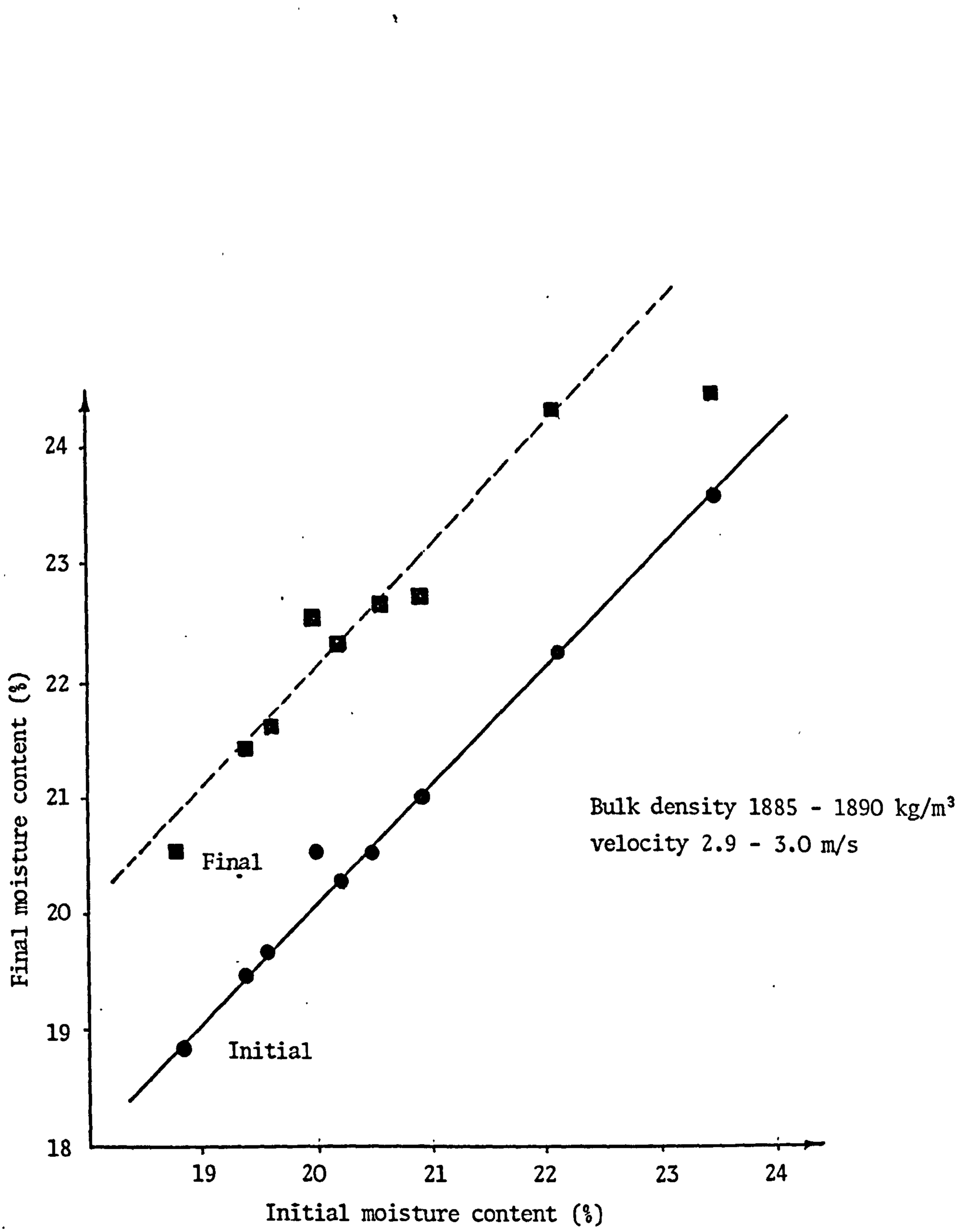


FIGURE 6.30 : Graph of Initial Moisture Content v Final Moisture Content

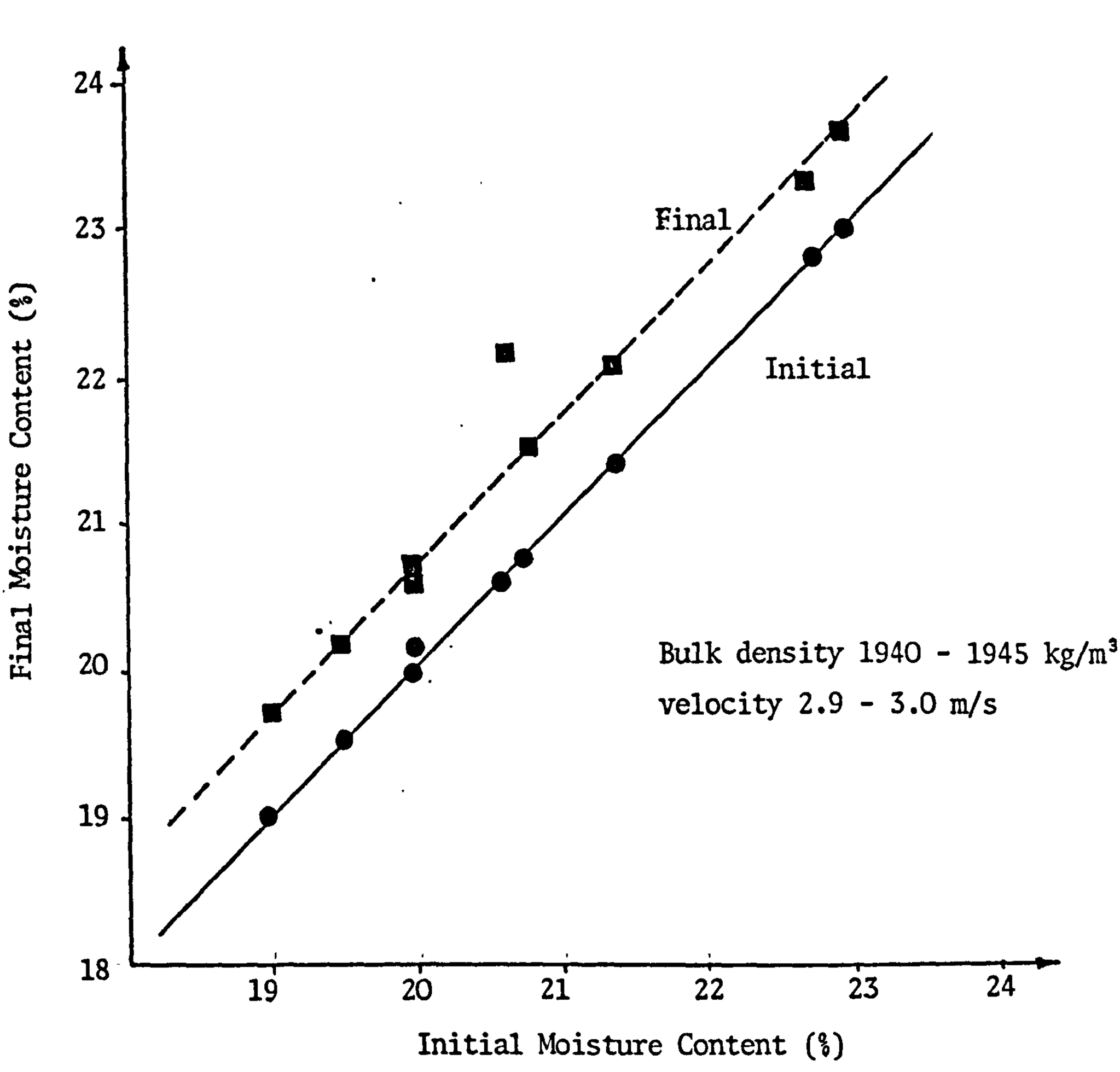


FIGURE 6.31 : Graph of Initial Moisture Content v Final Moisture Content

moisture content at time of compaction since, as the moisture content increases, the structure becomes more orientated. It is apparent from Figures 6.30 and 6.31 that the final moisture content, and consequently the strength, of a compacted clay will depend on method of compaction, initial density and moisture content and, in certain cases, any surcharge load applied.

A sample of soil compacted dry of the optimum moisture content will develop an essentially flocculated structure with relatively low pore water pressures, while a sample compacted dry of optimum will have an essentially dispersed structure and will develop much higher pore water pressures at low strains.

It has been shown earlier in this chapter that, for soils soaked after compaction, the amount of swelling that is likely to occur depends on the initial structure and moisture content of the soil; swelling increasing as moisture content decreases.

However, the final 'equilibrium' water content can be influenced by a surcharge pressure.

Generally a soil soaked after compaction will suffer an increase in moisture content and volume and a decrease in dry density and shear strength.

Embankment dams are examples of structures where compacted fills, initially placed at or near optimum moisture content, may in subsequent service approach saturation. In dam construction the lower layers, placed in the early stages of construction, are subject to increasing geostatic stress as the fill is raised, and the initial density at time of placing will be increased. For soils subject to high surcharge pressures, such as the lower layers of an embankment,

that pressure will control the tendency to swell and result in a gradual increase in moisture content. The face layers of fill are not, however, subject to high surcharge pressures and consequent reduction on swelling.

The moisture content of a fill material is clearly one of the more important parameters to be considered with respect to its susceptibility to erosion, but it is also an important parameter in the design process generally in the case of all fills of significant magnitude.

A clay soil compacted dry of optimum moisture content will facilitate construction in as much as lower construction pore water pressures will be generated and there will also be less tendency for construction plant to get bogged down. A clay fill containing a high proportion of air voids will, however, display a tendency to cracking and a tendency to swelling. Increasing the moisture content of a fill from an initial value of, say, 2 - 3% dry of optimum to optimum will substantially increase the flexibility of a compacted clay. In the case of embankment dams it is thus more usual to use material slightly on the wet side of optimum[229], producing a fill which is less crack susceptible, less prone to swelling and, in addition, can sustain some differential settlement. It must be appreciated, however, that at comparable moisture contents any increase in compactive effort will substantially decrease flexibility.

An empirical rule commonly employed in embankment dam construction is :-

compact first 40 - 50% of ht. from base at opt.m/c ± 1%

compact next 40 - 50% of ht. from base at opt.m/c - 0.5% + 1.5%

compact top 10 - 20% of ht. from base at opt.m/c + 2%

Perhaps a primary consideration for clay soils should, however, be the equilibrium moisture content, since a soil compacted wet or dry of this final equilibrium value will subsequently shrink or swell.

6.13 Effect of velocity of flow on erosion

The 'erosive power' of a flowing fluid varies as the second power of the fluid velocity , since the carrying capacity of the water in terms of the maximum size of particle which can be transported varies as the square of the velocity, and in addition the maximum mass (or quantity) that can be carried by the fluid varies as the sixth power of the velocity, the relationship being temperature dependent. In theory, doubling the velocity enables water to move individual idealised particles of a size 64 times greater and allows a carrying capacity, in suspension, 32 times greater. In effect, therefore, the 'erosive power' is theoretically four times greater. 'Erosive power' in this context may be defined as the ability of the flowing fluid to carry eroded material away.

Typical forms of the velocity distributions obtained during the experimental programme are shown in Figures 6.32 and 6.33. It can be seen that as one might expect, the average velocity is approximately 85 percent of the velocity at the surface, and occurs at about 0.6 of the full depth below the surface.

6.14 Reduction in shear strength

As already outlined in Chapter Five, the shear strength of the bed material was measured before and after each test with a hand shear vane. A reduction in shear strength was observed on each occasion.

Typical graphs showing the variation of shear strength both with depth and as a result of wetting are shown in Figures 6.34 and 6.35.

It can be seen that, as may be anticipated, a reduction in shear strength takes place as the sample is wetted and swelling occurs. This reduction varied from a maximum of 27 percent to a minimum of 3 percent of the original vane shear strength.

The greatest percentage reduction in vane shear strength for a particular bulk density occurred for clay samples having a comparatively 'high' moisture content whereas the greatest reduction in vane shear strength occurred with clay samples having 'low' moisture contents as shown in Table 6.4. No relationship is apparent linking moisture content, vane shear strength, and density and this is thought to be due to local variations in the moisture content as illustrated by Figure 6.36 showing the moisture content distributions within the clay following a test run.

6.15 Effect of Rainfall on Erosion

Rainfall, and the subsequent infiltration, causes a movement of water downward through a soil or a fill towards the water table. There is, as a consequence, a continuous leaching of the soil, which in the initial stages, may involve the removal of salts and alkalis.

The leaching of such minerals as muscovite,biotite, and other clay minerals such as illite and chlorite, will tend to remove cations from between the silicate layers. Additionally, the removal of potassium and magnesium ions tends to destroy the links between the silicate sheets, and hence permits the entry of water between those sheets, with an associated rise in moisture content which will affect erosion rates.

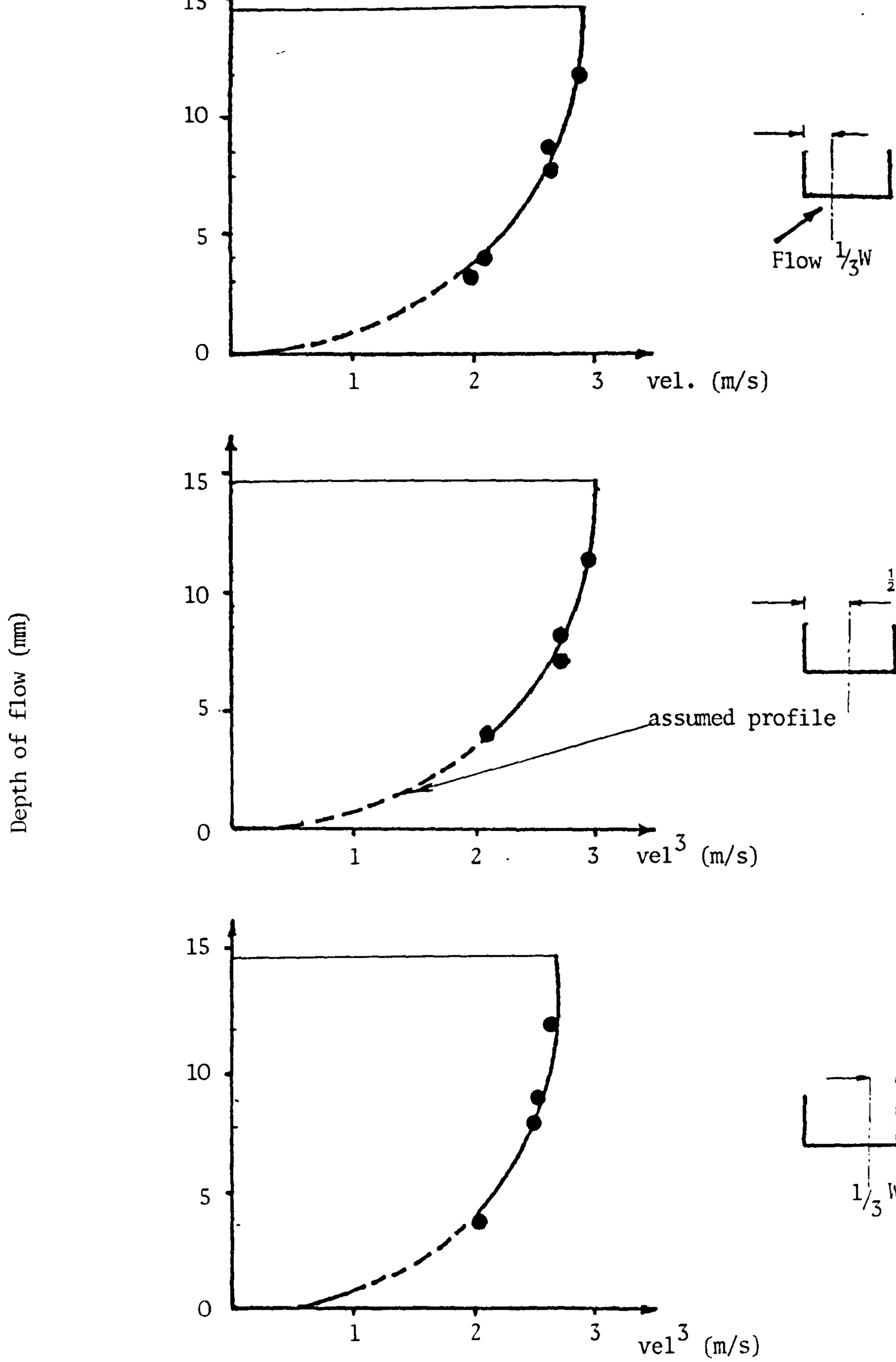
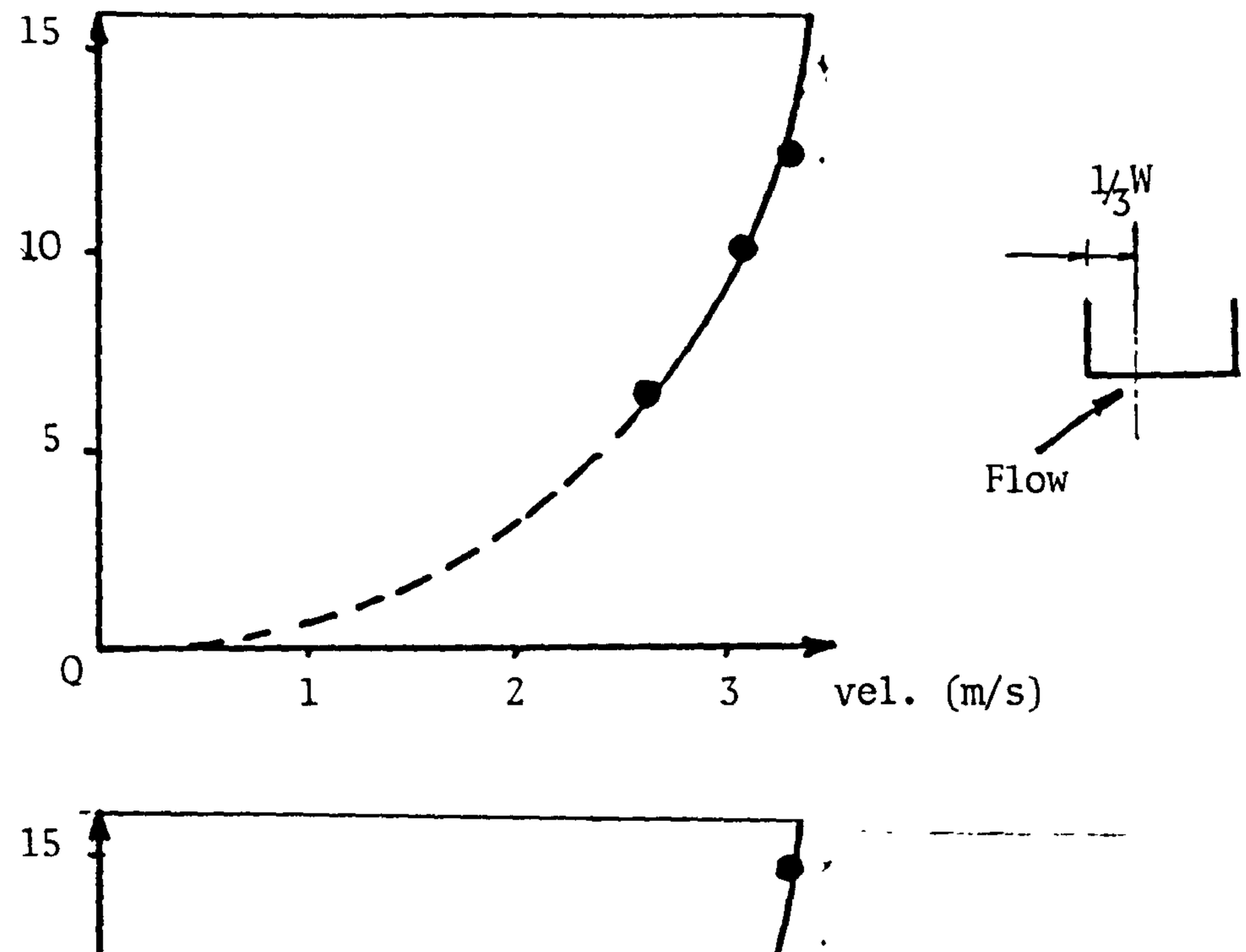
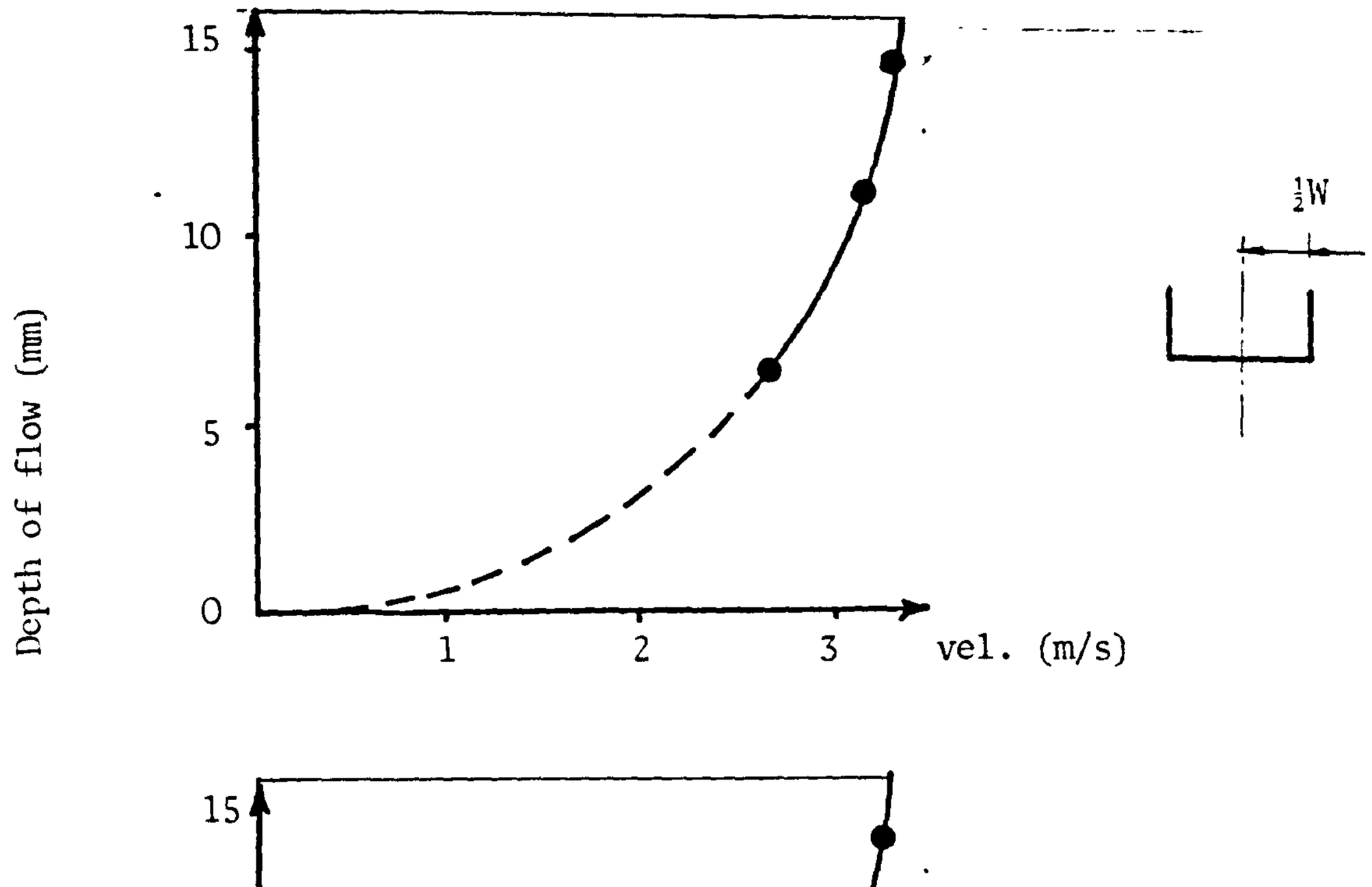


FIGURE 6.32 : Velocity Distributions for Flow in Channel
(Test Series I, No.1 $\bar{v}$ = 2.86 m/sec, d = 14.6 mm)





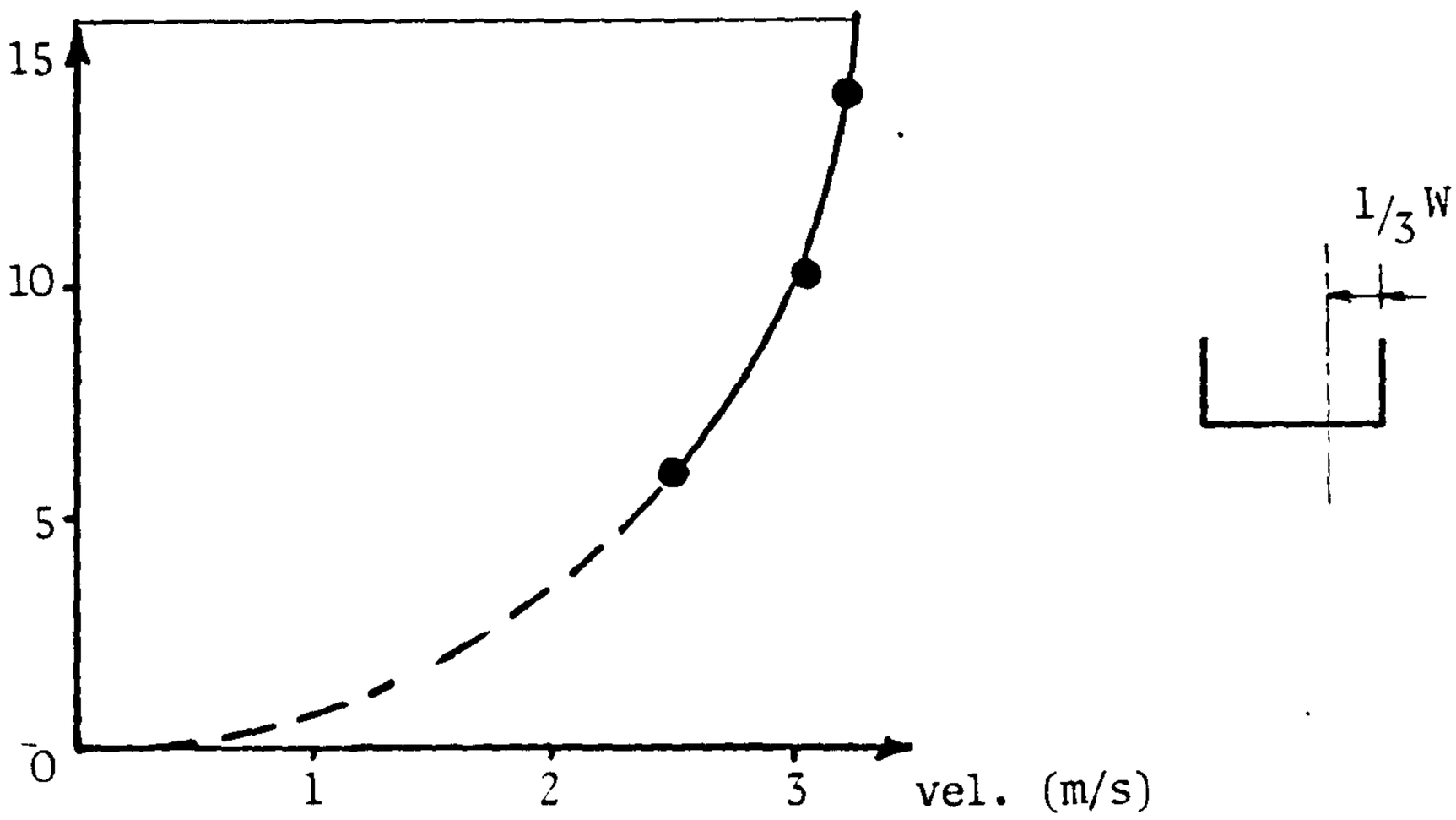


FIGURE 6.33 : Velocity Distribution for Flow in Channel
(Test Series I, No. 27) $\bar{v}$ = 3.13 m/sec, $\bar{d}$ = 15.8 mm.

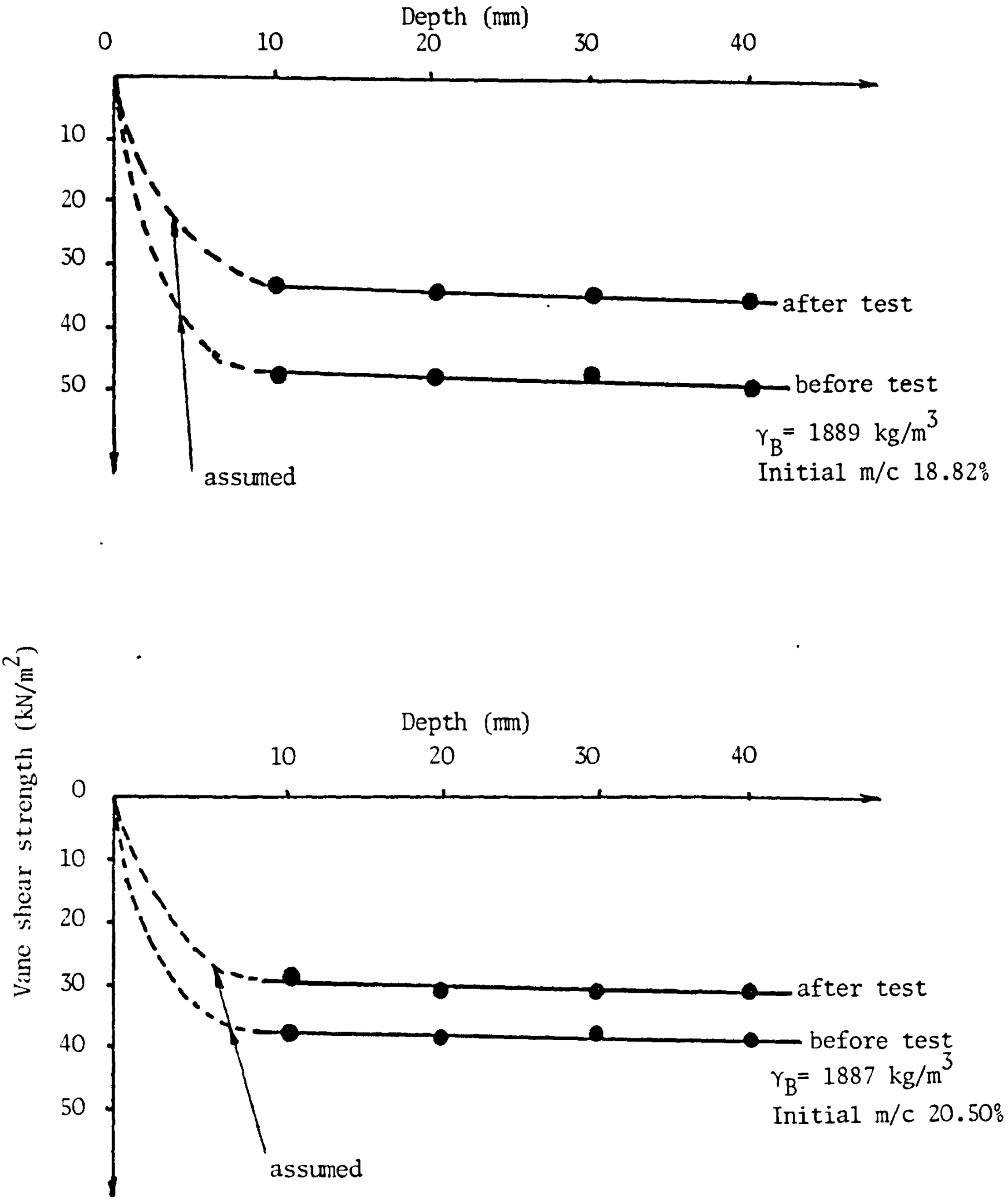


FIGURE 6.34 : Variation of Vane Shear with Depth

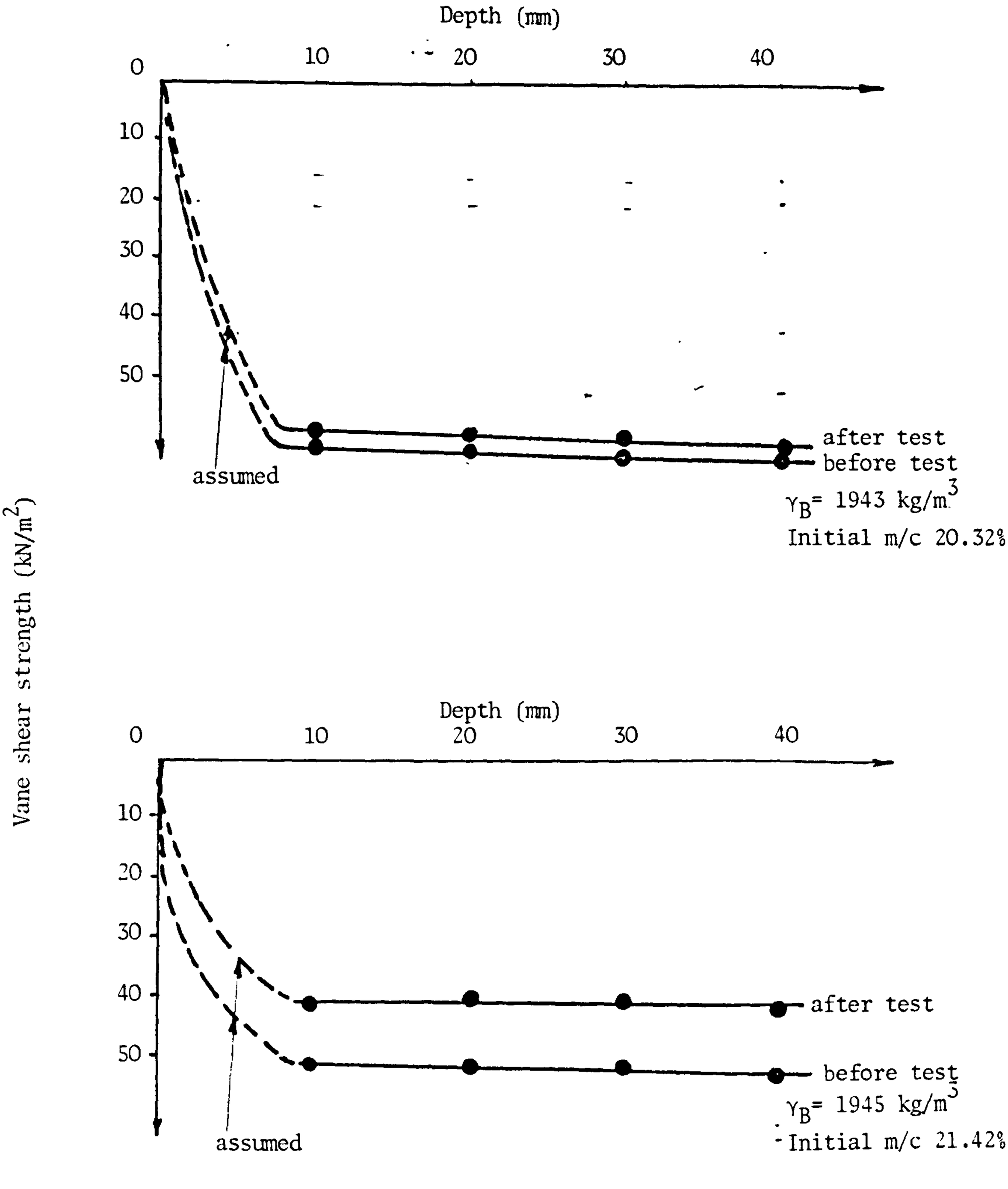
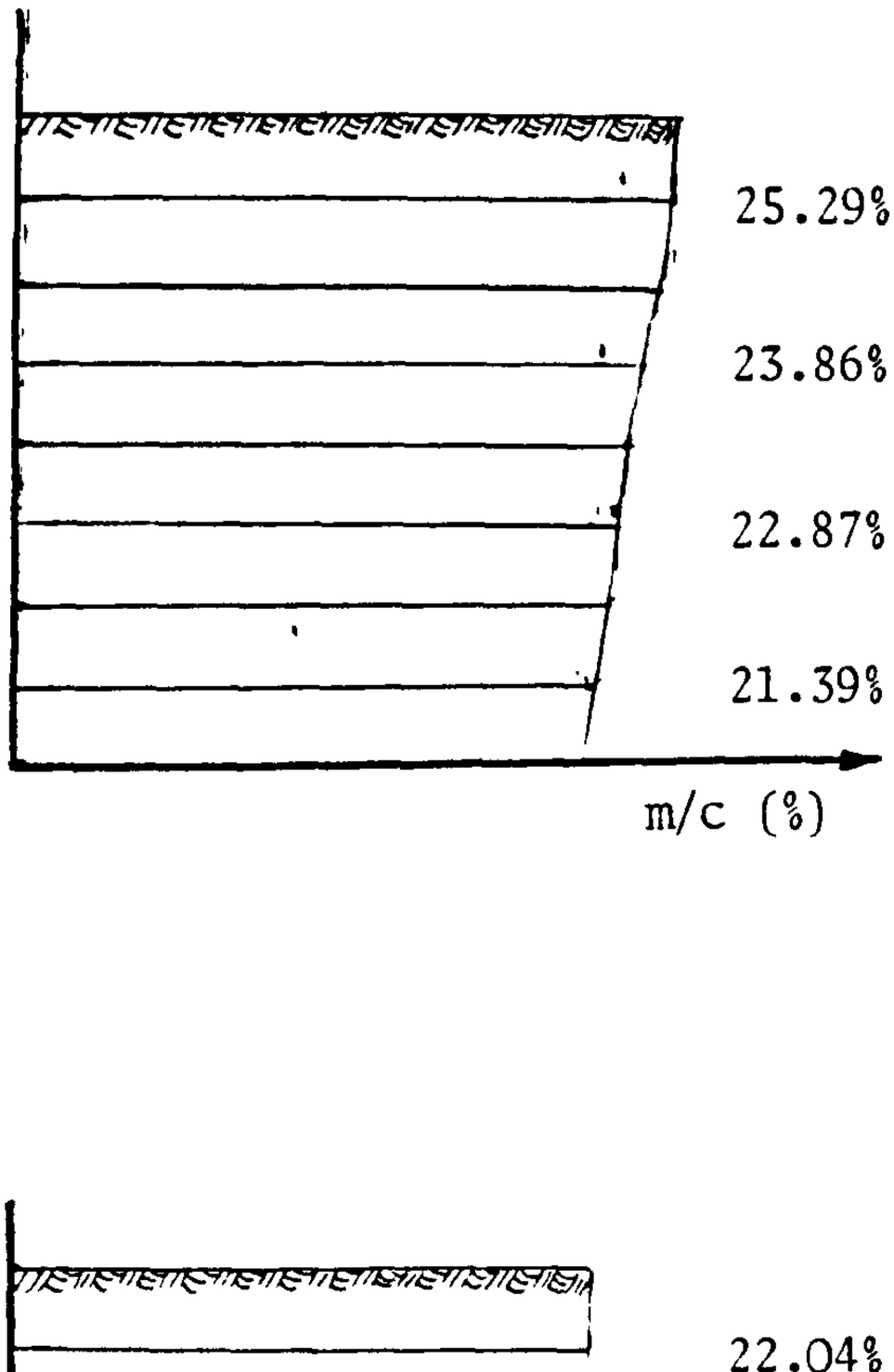


FIGURE 6.35 : Variation in Vane Shear with Depth

Initial m/c (%)	Final m/c (%)	Increase m/c (%)	Vane Shear Strength Initial (kN/m^2)	Final (kN/m^2)	Decrease (% of initial)	Actual Decrease kN/m^2
23.58	24.41	0.83	24	22	8.3	2
22.25	24.32	2.07	35	31.5	10	4.5
20.50	22.61	2.11	38.5	30	22	8.5
19.57	21.55	1.98	56	45	22.22	11
18.82	20.81	1.99	49	38.5	23.2	11.5

TABLE 6.4 : Vane Shear Results (γ_B = 1840-1850 kg/m^3)



Moisture content on sample as a whole (%)

23.61

21.57

22.00

FIGURE 6.36 : Moisture Content Distributions (γ_B = 1940 kg/m^3)

6.16 'Modified' Clay

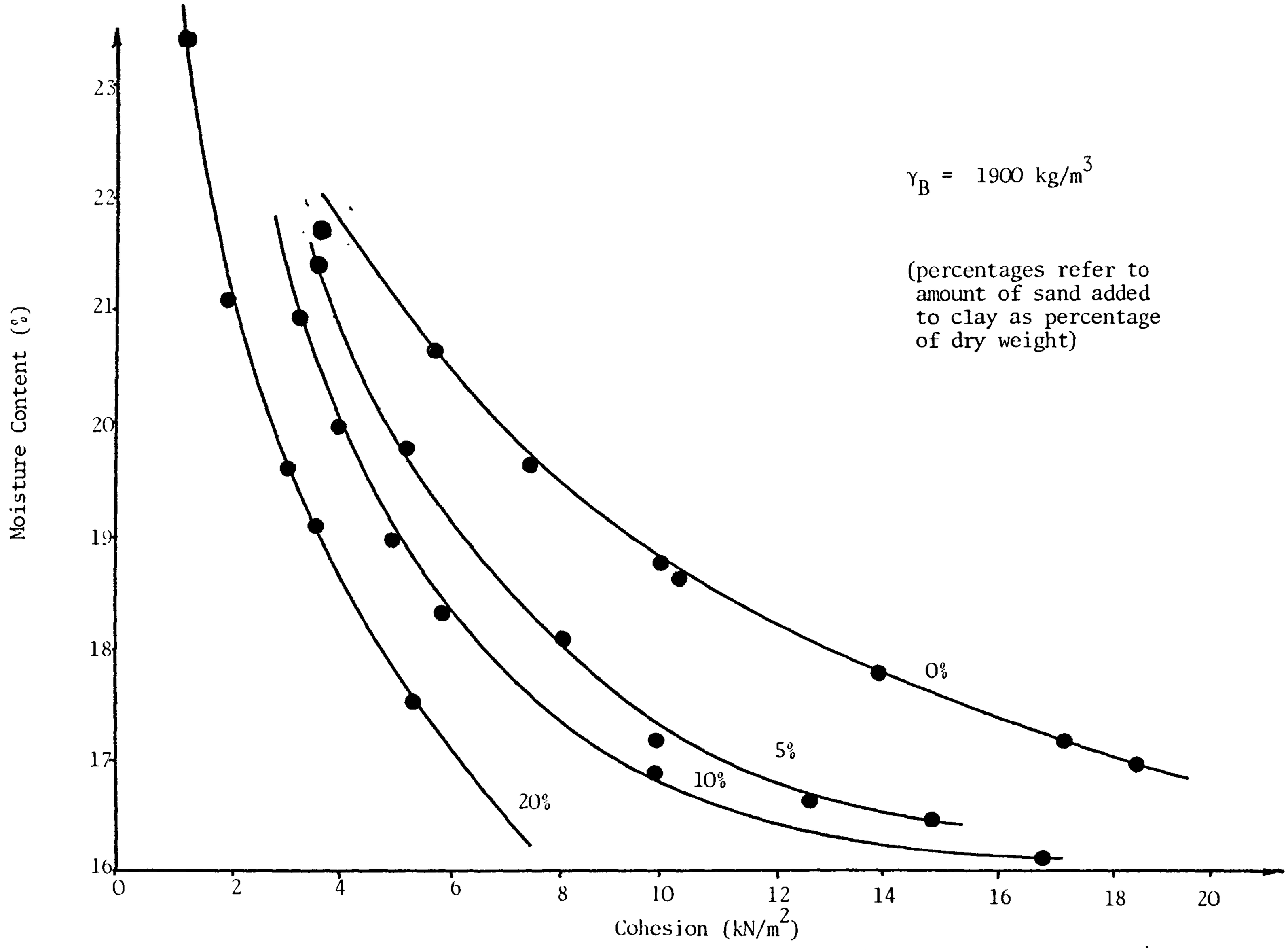
As already stated the clay used in the laboratory programme was 'modified', by the addition of sand, to obtain 'soils' with properties which differed from that of the original. It may be argued that such 'artificial' materials may not exist in nature and that they are not truly representative of materials used in the construction of embankment dams, but it should be appreciated that many of the experiments devised are essentially for comparative purposes only, and similar materials may well, in fact, exist naturally.

6.16.1 Cohesion and Moisture Content

The relationship between cohesion, moisture content and the percentage of sand added to the clay is shown in Figure 6.37. For comparison, the curve for the 'standard' clay material is included, and it can be seen that for a given moisture content and density an increase in the percentage of sand leads, as is to be expected, to a reduction in cohesion.

6.16.2 Swelling/P.L/LL

The results of swelling tests on ' modified' clay samples are shown in Figures 6.15,6.16,6.17 and 6.18. It has already been shown that, for a particular moisture content, as the percentage of sand increases, the amount of swelling decreases (Figure 6.19). In addition the experimental results have shown that an increase in sand (i.e. decrease in clay content) leads to a reduction in the Liquid Limit and Plastic Limit of the soil and, in consequence, to a reduction in Plasticity Index. It will be shown that the modification of the soil due to the addition of sand also leads to changes in erosion rate.



γ_B = 1900 kg/m^3

(percentages refer to amount of sand added to clay as percentage of dry weight)

FIGURE 6.37 : Relationship between Cohesion Moisture Content and Bulk Density for 'Modified' Clay

6.17 Effect of Slope on Erosion

Foster and Martin[220], using slopes of 3 :1, 2 : 1 and 1 : 1, and densities of 80-85, 90 and 95 lb/ft^3 (1280-1360, 1440 and 1520 kg/m^3) found that, for a given unit weight, there is a unique slope, from which a maximum amount of erosion will occur. This theory was not in agreement with the conclusions of other investigators [231,232,233,234]. The latter suggested that an increase in slope (without qualification of slope limits) results in an increase in soil loss.

Foster and Martin concluded that for a given density there is a unique slope from which the maximum erosion will occur, or, for a given slope there is a given unique density for which maximum erosion will occur, as shown in Figure 6.38.

Results obtained from Series II tests in this experimental investigation do not agree with the conclusions of Foster and Martin but are in general agreement with other investigators, although the latter investigations in test gave little information with which the results of these experiments could be directly compared.

In general it was found that as the slope of the downstream face became steeper the amount of erosion increased. Results of a full range of tests in Series II are shown in Table 6.5. It can be observed that at first sight the amounts eroded appear to vary very little, but when it is considered that as the slope steepens the length decreases, then it can be seen that the erosion rates have indeed increased. This may be partly due to the theoretical increase in velocity as the water flows down the longer slope.

Some tests in Series II, involving models composed of

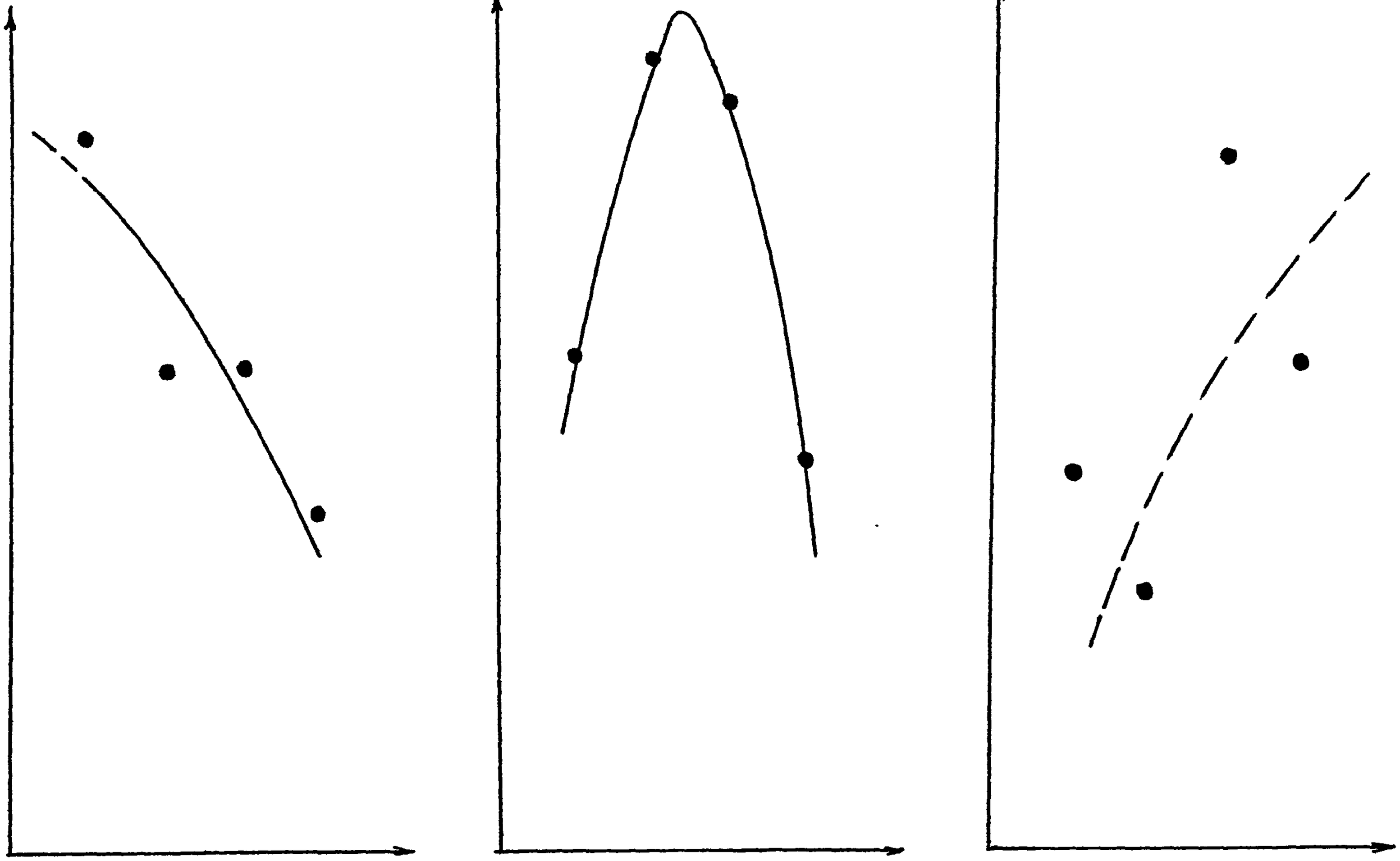


FIGURE 6.38 : Erosion Rates for Various Slopes
(after Foster & Martin)

γ_B = 1920 - 1950 kg/m^3 - standard clay

m/c = 19%

v = 2.56 m/sec (at crest)
Period of test: 50 mins.

Slope	Length of Slope (mm)	Ration $\frac{\text{Length Slope}}{\text{Length of 1:15 slope}}$	Eroded Material (gm)	Erosion Rate gm/m^2/mm
1 : 1.5	360	1 : 1	1804	397
1 : 2.0	447	1 : 1.25	1784	316
1 : 2.5	539	1 : 1.5	1720	253
1 : 3.0	632	1 : 1.75	1719	216

TABLE 6.5 : Erosion Rates for Various Slopes

'layered' clay samples, eroded in such a way that the step features coincided with the junctions of the layers of clay. It is thought that this was as a result of inadequate bonding between successive layers, a conclusion which reinforces the need to ensure that the compaction process is such that adequate bonding between layers is achieved. The erosion rates for the slopes tested are shown graphically in Figure 6.39.

6.18 Erosion Rates

The erosion rates, calculated from loss of weight in the sample per square metre per minute, are shown graphically for each test in Figures 6.40, 6.41, 6.42, 6.43, 6.44 and 6.45.

The figures each show two curves, one for the small 'sub-sample' and one for the full sample. The area of the latter is 0.13 m^2 while the area of the sub sample is 0.03 m^2, the ratio of areas thus being 4.3 : 1.0. Inspection of the graphs in figures 6.40, 6.41, 6.42, 6.43, 6.44 and 6.45 show fairly good agreement between results for the full sample and for sub-sample.

The difference, although small, is undoubtedly due to the change in surface roughness as the water encounters the sample and for this reason it is considered that the erosion rate calculated from the results obtained from the sub-sample may in the event be slightly more accurate.

It is also apparent that a time-dependent variation of shear stress must occur as non-homogeneities in the internal force system of a soil will result in some particles proving more resistant than other particles. In addition, the discontinuity at the upstream edge

γ_B = 1920 - 1950 kg/m^3 -'standard'

m/c = 19%

v = 2.56 m/sec (at crest)

Period of test : 50 mins.

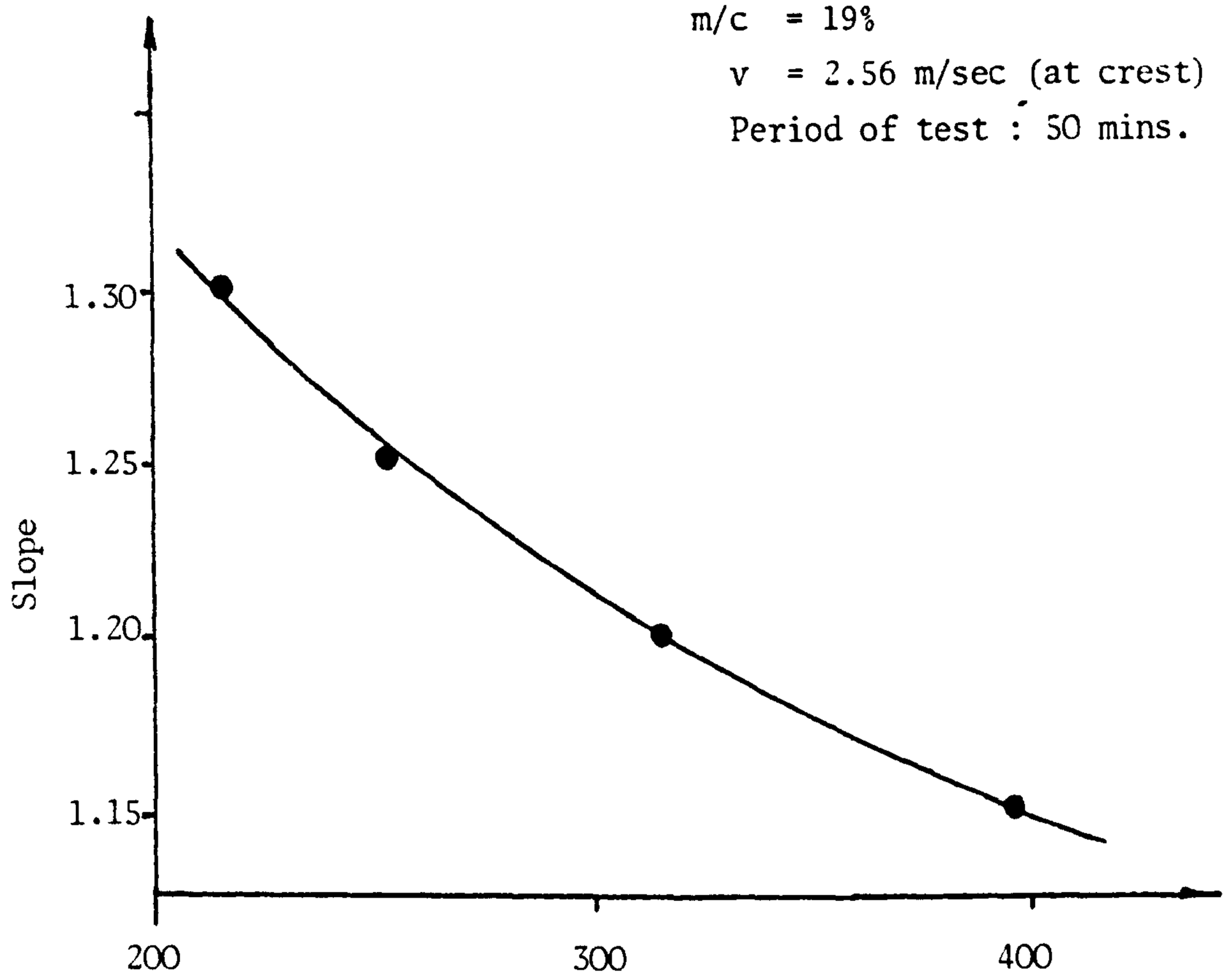
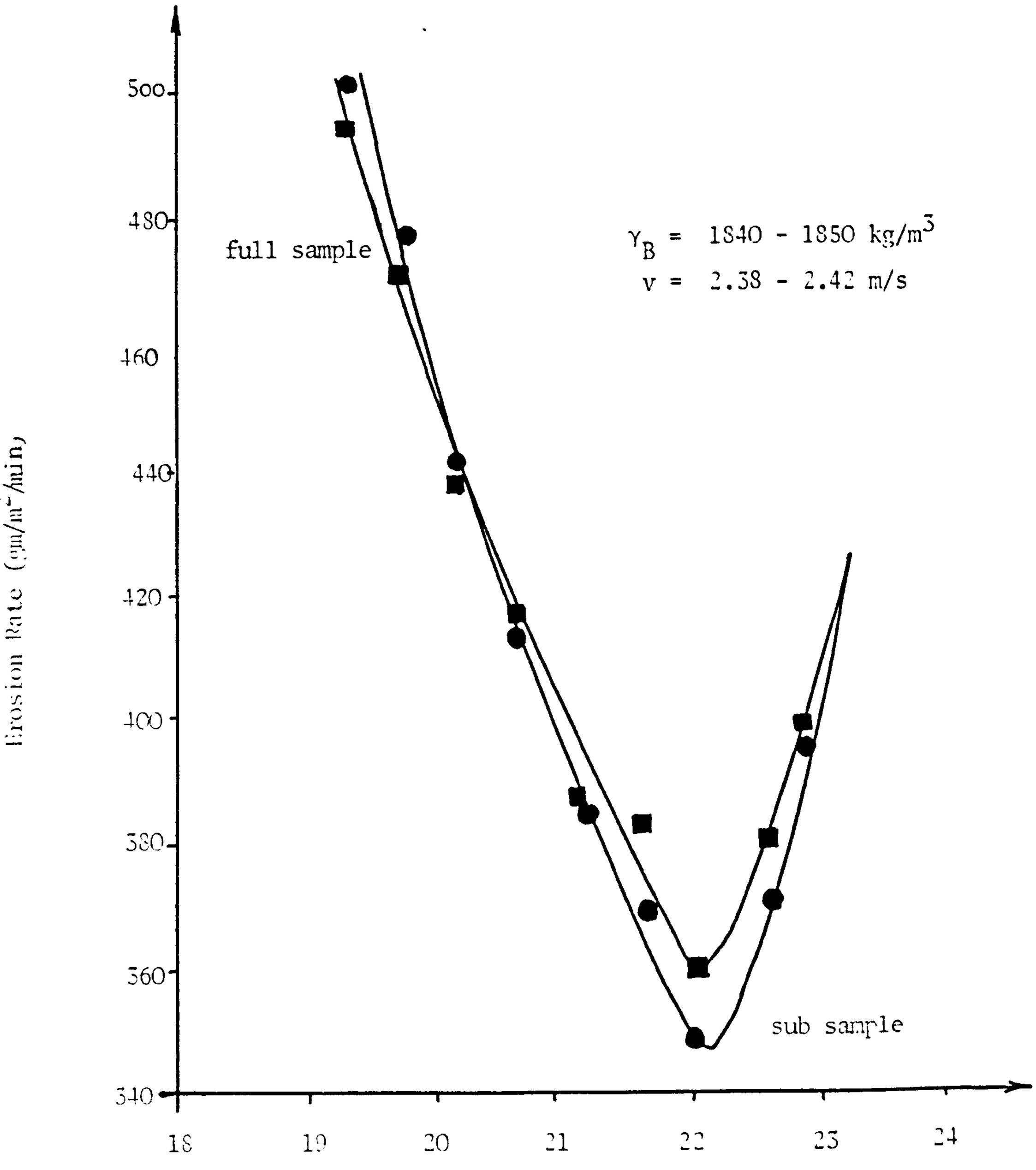


FIGURE 6.39 : Erosion Rates for Various Slopes



γ_B = 1840 - 1850 kg/m^3

v = 2.38 - 2.42 m/s

FIGURE 6.40 : Moisture Content v Erosion Rates

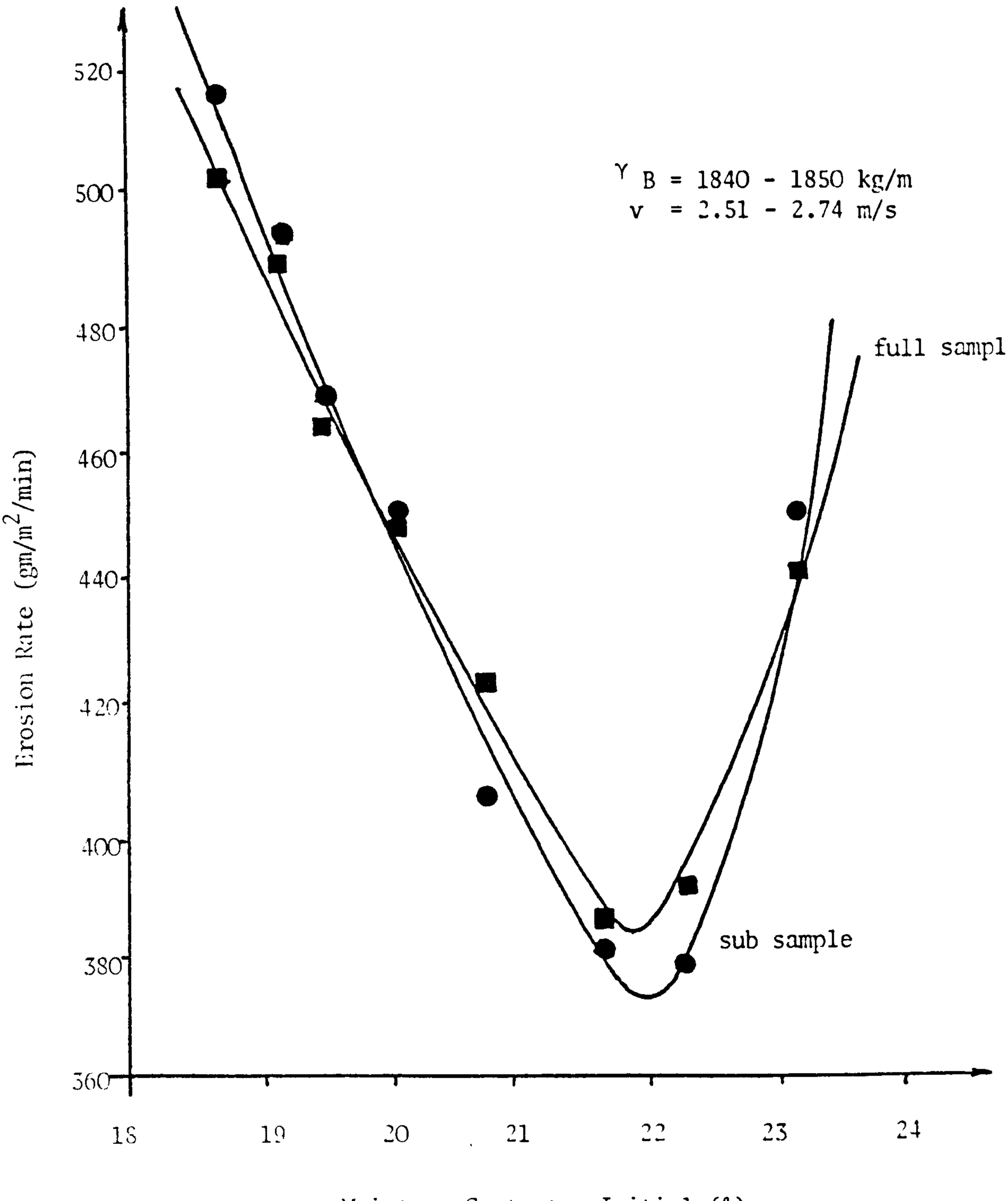


FIGURE 6.41 : Moisture Content v Erosion Rates

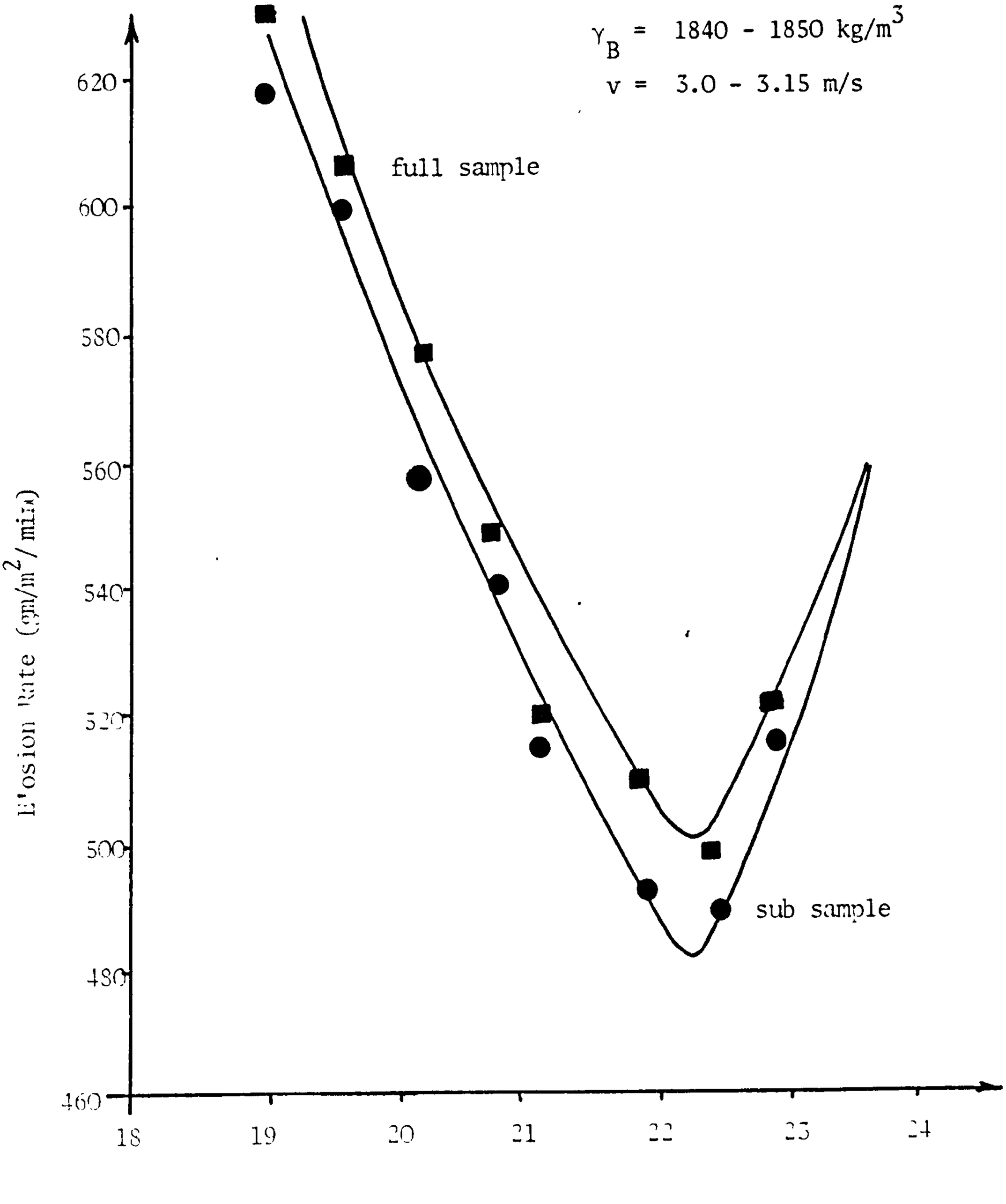


FIGURE 6.42 : Moisture Content v Erosion Rates

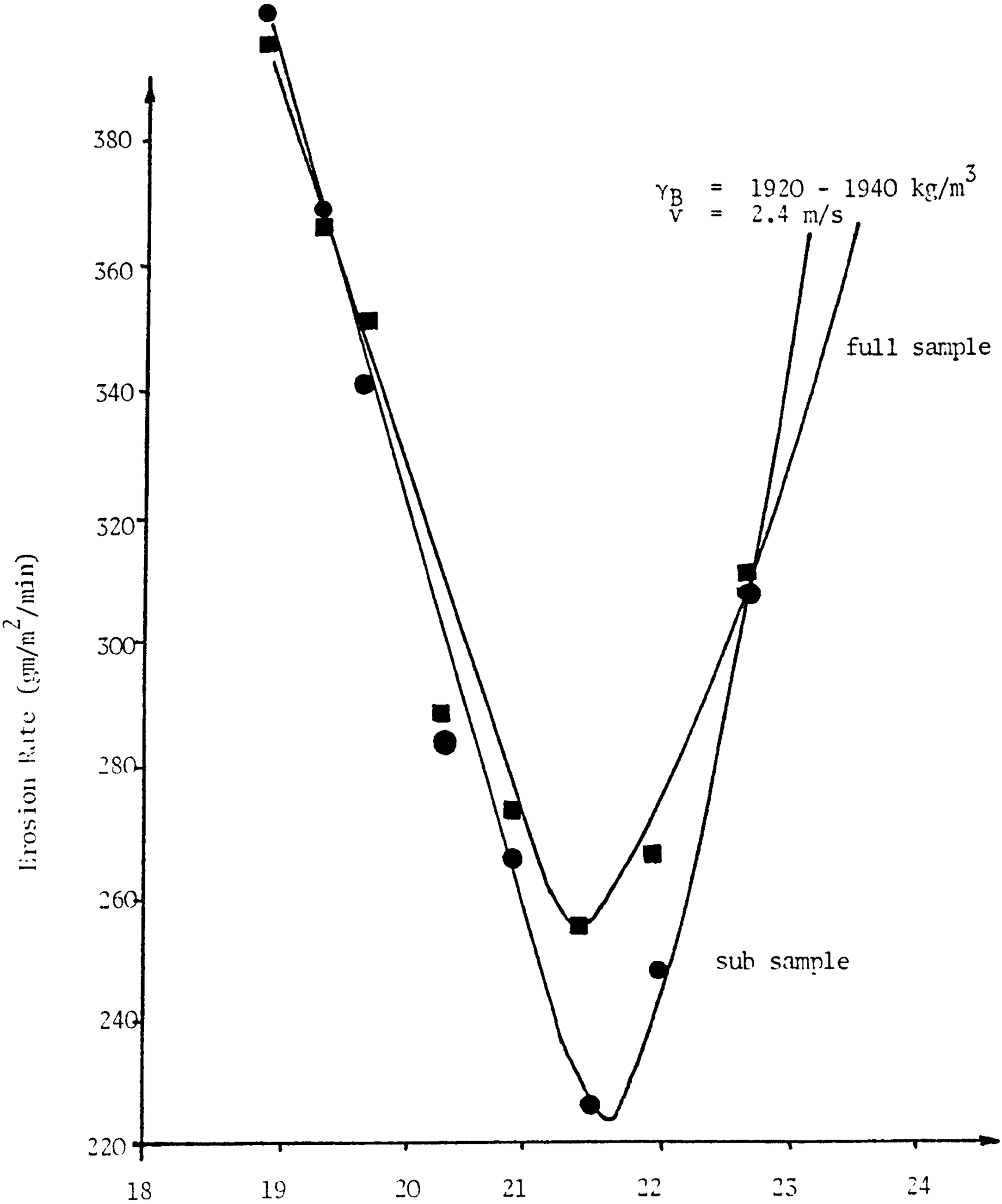


FIGURE 6.43 : Moisture Content v Erosion Rates

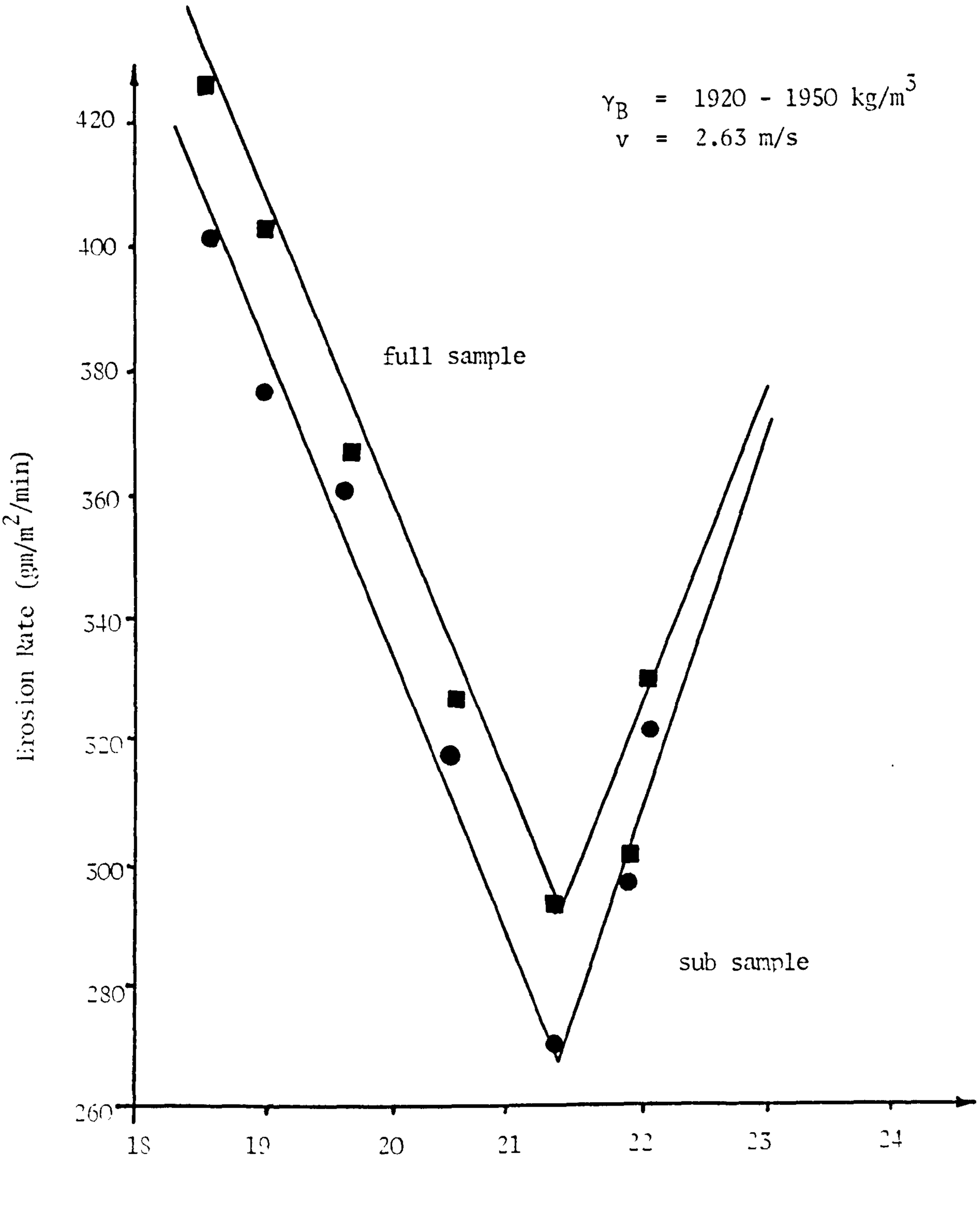


FIGURE 6.44 : Moisture Content v Erosion Rates

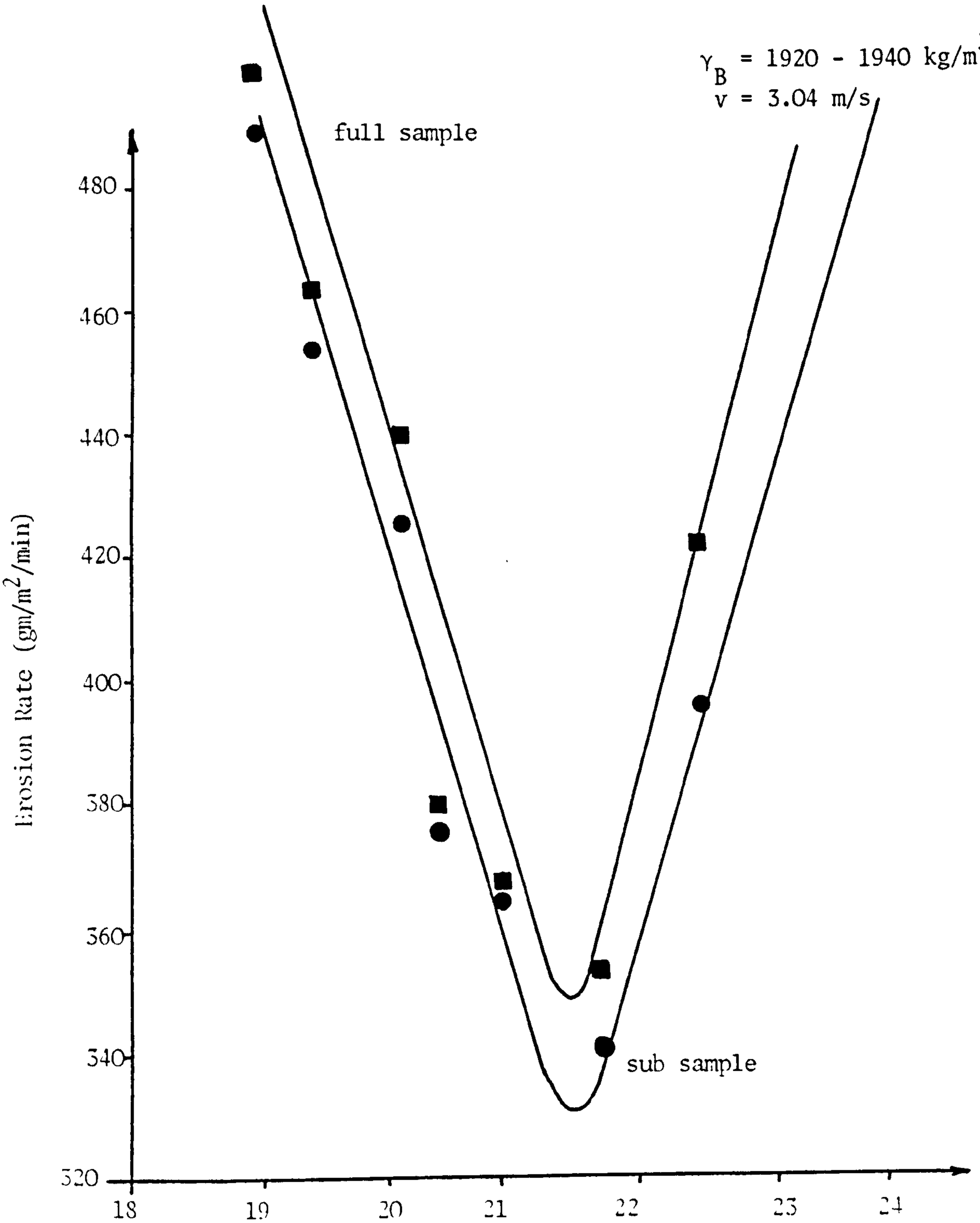


FIGURE 6.45 : Moisture Content v Erosion Rates

of the clay sample may cause some variation in shear stress with time.

With laminar flow the maximum shear stress on the side wall occurs at the free surface, while as turbulent flow occurs the position will change. The maximum shear stress on the channel base always occurs along the centre line of the cross section with laminar flow, while for turbulent flow the maximum shear stress may be off centre. With both laminar and turbulent flow present the shear stress appears to be zero at the base corners of the cross-section. It is also conceivable that the water pressure on the flow boundaries will vary with time and from point to point because of disturbances in the flow that are induced by the roughness and geometry of the sample itself, with the result that pressure can drop momentarily at a point, with water actually flowing upward and outward from the clay bed, carrying surface particles with it.

Plots of erosion rate v. moisture content all show the same basic form, exhibiting a <u>minimum</u> value of rate of erosion at a particular moisture content, while rates of erosion increased rapidly at moisture contents high or low of this 'optimum'. The rate of increase in erosion rates appears to be fairly constant for the range of bulk density and velocities investigated in these experiments, and is approximately 55 $gm/m^2/min/\%$ of moisture content.

Erosion rates are seen to increase as the velocity of flow increases and also as bulk density decreases.

The 'optimum' moisture content at which the erosion rates reach a minimum would appear to be sensibly constant for this clay, having a

value of 21.5 - 22%. The erosion rates quoted above are, it should be noted, average values calculated from loss of material during a 50 - 60 minute test. It has already been shown that the rate of erosion varies during the test period, as shown by the graphs of suspended solid concentration, but it is felt that for the calculation of the amount of erosion resulting from a period of overtopping of 50 - 60 minutes, the overall rate of erosion will enable a first estimate of the resultant erosion to be estimated. Further tests should desirably be conducted to establish the rates of erosion for periods in excess of 60minutes, but constraints on available time did not permit investigation within this experimental investigation.

The erosion rates for a clay having a moisture content of only two or three percent will obviously be high, as will the erosion rates for a clay nearing a liquid state, and hence it is clear that the graphs in Figures 6.40 - 6.45 will show a tendency to even higher erosion rates as the moisture content is increased or decreased outside the range of moisture contents tested during the investigation.

Inspection of the suspended solids graphs shows a decrease in suspended solid concentration for the 'high' moisture contents, whereas the erosion rate graphs seem to suggest that erosion rates should increase from a minimum value as the moisture content increases. The limited number of tests at higher moisture contents do not allow any definite conclusions to be drawn, but it is considered that the results obtained from bulk sample weighings are likely to be most accurate. The suspended solid concentration graphs should be used to study the erosion mechanism, and the erosion rate graphs used to predict the amount of erosion likely to occur due to overtopping.

6.19 Models

If a model and prototype are built with corresponding materials exerting proportionate resistance to the dislodging of particles, then both must erode to a similar shape,and this at rates proportionate to the square root of their geometrical scale. If the inertia stresses completely overshadow those due to viscosity, it is safe to conclude that both dynamic and static pressures are substantially proportional to the model size, at least when the water exceeds 5 millimetres in depth and where the velocity exceeds 0.5 metres per second. Even close to the bed, where local velocities may be less, this conclusion remains sensibly valid, since in such locations the motion is turbulent, and local eddies formed by irregularities in the bed and set up by cross flows are many times greater than the molecular interchange of viscous action. Thus, the stress available for moving individual particles from their settings is on average proportional to the size of the model and the velocity available for transportation of loosened particles is proportional to the square root of the linear scale.

In order to eliminate surface tension effects Webber[175] suggests that the head on any model should exceed 6 mm.

6.20 Methods of Reducing Erosion Rates

The erosion rate of a clay may be reduced by:

(1) cultivation of vegetation on the surface, (i.e. grass) to bind the surface and shield it from direct impact and 'drag' effects,

(2) protection of the surface with grouted rip-rap, or with some other similar, possibly proprietory, surface protection systems,

(3) mixing active clay, e.g. bentonite, in the top 100 mm

of the surface material - ion exchange can influence the interparticle forces, and the substitution of multivalent cations, in sufficient concentration to displace any monovalent cations in the soil, will set an upper limit on the amount of swelling likely to occur.

6.21 Resistance to Erosion - Documented Evidence

Documented instances of overtopping which have included evidence of the resistance of embankment fill material to overtopping include:-

Dam	Incident/Accident
1. Baldwin Hills	"It took 77 minutes to release 897 acre-ft (111 ha.m) of water".
2. Buffalo Creek	While under construction the dam was overtopped and over a period of 2.3 days 50×10^6 yd^3 ($34 \times 10^6 m^3$) of material was removed.
3. Cheaha Creek	Overtopped for 3 hours.
4. Detention	Overtopped by 0.76 m of water (average value) which eroded some 0.3 m of material off the whole of the downstream shoulder.
5. Fort Meade	A rockfill dam with a concrete upstream face. A 1 in 100 year flood caused 0.46 m of water to flow over the crest - did not fail
6. Holden	Overtopped by 0.1 m of water for 15 minutes.

7.	Malloya	Overtopped by 0.38 m of water for 6 hours - did not fail - d/s slope 2 : 1, PL = 20, LL = 37, D_{50} = 0.009, % clay = 44%, S.G. = 2.54.
8.	Nanpantan	Overtopped for at least 24 hours.
9.	Oros	Fill placed on the crest withstood overtopping by 0.5 m of water for several hours.
10.	Pinkstone	'Overtopped for 2 hours before any measurable erosion occurred.'
11.	Pittston	"It took 2 hours to release 4 million gallons of storage" (18 Ml)
12.	Schaeffer	"It took 1½ hours to release 3852 acre-ft" (477 ha.m)
13.	Santelle	"It took 39 minutes to release 5.5 million cubic ft. of water" (155 t.c.m)
14.	Unknown	"Overtopped by 0.80 m for 8 hours before failure.

Reference should be made to Appendices A and B for detailed information on these and other incidents.

6.22 Failure of a Dam - Sequence of Events

From the experiments carried out during the course of this investigation it is possible to suggest the sequence of events involved in the failure of a dam as a result of overtopping.

The sequence of events may be as follows:-

(1) As inflow to the reservoir increases the reservoir level rises.

(2) If a wavewall exists then the static flood level may encroach on the wavewall to a point where the wavewall becomes unstable and part of the wall collapses resulting in a sudden high flow across the crest at comparatively high head and down the face of the dam. If a break exists in the wall, such as a gateway to a valve tower, then obviously a concentration of flow will occur at this point.

(3) If no wavewall exists then the water will flow across the crest of the dam more or less evenly unless there are areas of the crest which are significantly lower than the rest.

(4) If the flow across the crest is even then the erosion which occurs on the downstream face will be comparatively even with gullies, forming across the entire downstream face until some discontinuity or variation in the fill material of the downstream shoulder results in local increase in erosion rates compared with that of the rest of the shoulder. The situation resulting is then the same as the concentrated flow case.

(5) If the wavewall remains stable, and is continuous, then even flow would occur over the top of the wall which would act as a weir. Water falling to the crest at the base of the wall would cause undermining of th foundation and eventually the wall would fail, probably over a short section, resulting again in a concentration of flow.

(6) A concentration of flow is thus a highly likely occurrence, possibly in consequence of a low spot on the crest, as a result of a hole or local failure in the wavewall, or may possibly arise from tussocky grass or bushes/stones on thc downstream face of an embankment.

(7) Filter zones within the shoulder of the dam will erode more quickly than the more cohesive fill, but again, an area of concentrated flow will eventually result.

(8) The step features previously discussed in this Chapter, will occur down the face and, as swelling, reduction in shear strengths and local erosion progress, in general the upper step will recede towards the crest, and thus the upstream face, of the embankment. There may be some small instability of the sides of the channel formed by the receding step, but this is likely to be very slight, the sides standing almost vertically.

(9) If a core exists within the embankment and the material from which the core is constructed is very different to the material in the shoulder then the core will erode at a rate which will, in general, be less than that of the fill in the shoulders. As a result a 'waterfall' may occur at the interface of the downstream face of the core with the downstream shoulder. Erosion will continue on the downstream side of the core within the channel until the fill and water pressure on the upstream side of the core cause the core, now acting as a cantilever, to fail. The core is then carried away by flow in the eroded channel. If the material in the core is less

markedly different to that in the shoulders, and the rates of erosion of the materials are hence only slightly different, the 'waterfall' formed will be relatively low and the channel will recede into the upstream shoulder until, eventually, water pressure may break through the remaining shoulder material creating a channel through the embankment.

(10) The reservoir storage is now "channelled" towards the breach and the breach is further eroded along the entire length of base of the breach down to the base of the dam thus releasing all the impounded water.

6.23 Dimensional Analysis

Dimensional analysis allows the form of a possible relationship to be established in terms of definite parameters.

Various investigators have tried to use Plasticity Index (P.I.) as a measure of erodibility, a high plasticity (> 40) being relatively non erosive while low plasticity clay (< 25) tends to be associated with an erosive soil, but it is possible that two soils with the same P.I. may erode very differently depending upon soil structure. On occasions soils with no appreciable Plasticity Index may be found to show considerable resistance to erosion. For the reasons given above. P.I. has not been included as a measure of erodibility since it proves to be a reliable index in certain instances only.

Assuming the amount of material eroded (E) likely to occur for any sample will depend upon the velocity of flow (v), the bulk density of the soil (γ_B) the surface area of the sample exposed to the eroding fluid (A), and the period of overtopping or duration of test (t) then, using Buckingham's Π theorem :

			Dimensions
E	=	wt. of clay eroded	M
v	=	mean velocity of flow	LT^{-1}
γ_B	=	bulk density of soil	ML^{-3}
A	=	area of sample	L^2
t	=	duration of test or period of overtopping	T

$$\dagger (E, v, \gamma_B, A, t) = 0$$

no of Π's = 5 - 3 = 2

Let $\Pi_1 = \gamma_B^{x_1} A^{y_1} t^{z_1} v$

and $\Pi_2 = \gamma_B^{x_2} A^{y_2} t^{z_2} E$

For Π_1

$$(ML^{-3})^{x_1}(L^2)^{y_1}(T)^{z_1}LT^{-1} = m^0L^0T^0$$

Equating M's

$$x_1 = 0$$

Equating L's

$$-3x_1 + 2y_1 + 1 = 0$$

$$=> y_1 = -\tfrac{1}{2}$$

Equating T's

$$z_1 - 1 = 0$$

$$z_1 = 1$$

i.e. $x_1 = 0$, $y_1 = -\frac{1}{2}$, $z_1 = 1$

$$\therefore \Pi_1 = \frac{tv}{A^{\frac{1}{2}}}$$

For Π_2

$$(ML^{-3})^{x_2}(L^2)^{y_2}(T)^{z_2}M = M^0L^0T^0$$

Equating M's

$$x_2 + 1 = 0$$

$$x_2 = -1$$

Equating L's

$$-3x_2 + 2y_2 = 0$$

$$3 + 2y_2 = 0$$

$$y_2 = \frac{-3}{2}$$

Equating T's

$$z_2 = 0$$

i.e. $x_2 = -1$, $y_2 = \frac{-3}{2}$, $z_2 = 0$

$$\therefore \Pi_2 = \frac{E}{\gamma_B A^3/_2}$$

Since $\Pi_1 = k\Pi_2$ where k is a constant

$$\frac{E}{\gamma_B A^3/_2} = k \frac{tv}{A^{\frac{1}{2}}}$$

$$E = \gamma_B A^{3/_2} k \frac{tv}{A^{\frac{1}{2}}}$$

or $= k_2 \gamma_B A\ tv$ where k_2 is another constant.

Applying this equation to the curves of erosion rates shown in Figures 6.40 - 6.45 at the 'optimum' moisture content for minimum erosion the following values of k_2 were obtained.

For γ_B = 1840 - 1850 kg/m^3

$k_2 = 7.6 \times 10^{-5}$

For γ_B = 1940 - 1950 kg/m^3

$k_2 = 5.4 \times 10^{-5}$

the final equations being

$E = 7.6 \times 10^{-5} \gamma_B A\ t\ v$ for γ_B = 1840 - 1850 kg/m^3

and

$E = 5.4 \times 10^{-5} \gamma_B A\ t\ v$ for γ_B = 1940 - 1950 kg/m^3

6.24 Conclusions

The erosion problem considered in this investigation is that of surface erosion, when the top layer of topsoil and grass has first been removed, rather than deep internal erosion or deep seated shear. For this reason any factors which minimise the surface interparticle forces and minimise the stability of the surface layers will maximise the erosive condition. However, difficulty arises in predicting surface erosion when an attempt is made to establish those parameters of the soil which control the susceptibility to erosion. From an engineering standpoint it is obviously desirable to rely upon simple and readily determined measurable properties to characterise erosion resistance.

The experimental investigation has shown:-

(1) Erosion of clay occurs by the removal of individual particles, small clusters and, in some more extreme instances, by mass erosion. Small stones in the clay material were observed to affect the erosion process, especially when they protruded from the bed, resulting in undercutting on the downstream side of the stone. In addition small stones were seen to be whirled around in depressions causing holes to be formed.

(2) Material eroded from the bed was seen to roll, slide, bounce and to be removed in suspension.

(3) The bed was generally observed to erode fairly evenly, and to be covered with ripples having a height of approximately 2 mm, with occasional small stones protruding from the bed.

(4) The concentration of suspended solids in the eroding fluid (for the clay under test and for concentrations of up to

1 gm/litre) had no effect on the erosion process

(5) Models of embankment dams showed the formation of a breach, attributable to overtopping follows a set sequence of events:-

(i) the crest becomes rounded

(ii) 'steps' are eroded at regular intervals down the downstream face.

(iii) the uppermost step becomes most prominent and cuts back through the crest towards the upstream face.

(iv) when the water pressure on the upstream face exceeds the tensile strength of the clay the water will break through to form a breach.

(v) If the fill material has any apparent cohesion then the sides of the breach will stand almost vertically.

(vi) If a clay core exists within the embankment then the downstream shoulder may be eroded leaving the core standing and acting as a cantilever.

(vii) If filters exist in the downstream shoulder then these zones will erode more deeply than the compacted fill of the shoulder.

(viii) If there are any features which result in concentrations of flow or increased turbulence then increased erosion will result. These features include berms, tussocks, etc.

(6) Swelling occurred as soon as the eroding fluid encounters the soil.

(7) Swelling results in an increase in void ratio.

(8) Swelling increases as the moisture content of the soil decreases.

(9) For a particular moisture content and density the amount of swelling increases as the percentage of clay in the soil increases.

(10) Flocculated clays swell more than dispersed clays.

(11) All soil samples tested absorbed water during the test, increasing their moisture content. For a particular density of fill the increase in moisture content was found to be sensibly constant.

(12) In general, erosion increases as the slope of the downstream face becomes steeper.

(13) Erosion rates decreased to a minimum value as the moisture content increased, then increased once again as the moisture content exceeded the 'optimum' value in terms of minimum erosion. Optimum moisture content for minimum erosion for the clay investigated was approximately 21.5 - 22%.

(14) The minimum amount of material eroded for a particular density can be predicted by the following equations.

For γ_B = 1840 - 1850 kg/m^3

$$E = 7.6 \times 10^{-5} \gamma_B A \text{ t.v.}$$

For γ_B = 1940 - 1950 kg/m^3

$$E = 5.4 \times 10^{-5} \gamma_B A \text{ t.v.}$$

CHAPTER SEVEN

SLOPE PROTECTION

7.1 Introduction

Much of the preceeding experimental work relates to a situation where overtopping has occurred and topsoil has been removed exposing the compacted clay shoulders of the embankment.

It has been suggested that vegetation will protect an embankment against infrequent overtopping[197]. The present chapter reviews conventional methods of slope protection and suggests protection methods which might be used to protect an earth embankment against overtopping.

7.2 Vegetation

Emergency spillways and also the downstream faces of dams are cases where conventional treatment for the stabilisation of surfaces subject to erosion by the intermittent flow of flood water are uneconomic. The frequency of occurrence of flow can be so low that expensive insitu concrete works or alternatives such as asphalt or blockwork cannot be justified.

G.M. Binnie was one of the first to suggest that a grassed spillway could be used for emergency spillways, if sufficient reliance could be placed on it's engineering properties, although evidence does exist to suggest that a well chosen grass cover can withstand a velocity of up to

2 m/s for 'prolonged' periods, of say, more than 10 hours; a velocity of 3-4 m/s for several hours, and velocities of approximately 5 m/s for 'brief' periods, say less than 2 hours[198].

A reasonable appreciation of the erosion resistance and hydraulic frictional properties of different types of vegetation was considered essential, and attempted by the Construction Industry Research and Information Association (C.I.R.I.A.)[197].

Grass protects earth surfaces by reducing the velocity of flow to a value which will not erode the soil. The root growth reinforcing the upper soil layers also retards the erosion of the soil, and thus it is clear that ground cover is an importanct factor to be considered[199].

Vulnerability of bare patches appears to be dependent on their size relative to the height of the grass, which shades them if it is upright or mats over them if the velocity is high enough to lay the grass down.

Failure occurs when scour around the plant roots has weakened the anchorage of the plant to a degree that permits the flow to strip the cover. Once a point of attack has been established the scour is progressive and may therefore be considered time dependent.

Flow over a vegetated surface can be divided into three basic regimes:-

1. when the flow dept is significantly less than the height of the vegetation, which is not deflected.
2. when the flow depth is close to or greater than the height of the vegetation, which deflects and oscillates.
3. when the velocity is high enough to lay the vegetation down and a relatively smooth, stationary surface is presented to the flow.

The criteria taken into consideration in design may include:-

1. hydraulic loading; velocity, frequency of immersion, interval between immersions.
2. secondary purpose of land - e.g. grazing, hay, parkland.
3. management that can be provided reliably.
4. climate
5. type of soil

Grass cover for dams must provide good ground cover, and good root development, be resistant to disease and drought and be naturally hardy. A perennial species is used which has the ability to recover quickly after immersion and to establish a good ground cover quickly in the soil used in the construction of the dam.

A low maintenance species is usually chosen for dams due to the difficulties in providing regular maintenance, although machinery has now been developed which will work on steep slopes. It is debatable whether it is preferred to have a well manicured bank on which deformations and leakages can be easily seen or to have long grass which would deflect under the influence of overtopping waters to protect the bank. Sheep and cattle are often put to graze on embankments, the former being the most common as less damage is caused and sheep graze the sward down closer to the bank but unfortunately sheep form tracks, particularly along the crests, where the reduction in grass cover increases the vulnerability of the bank to erosion. An additional factor to be considered is that for a well mown grass the leaf area is low, and the water loss by transpiration from the soil less. No cutting results in the grass becoming rank, the bottom structure becoming destroyed and there is a tendancy for the grass to form clumps and as will be explained, this

may lead to increased erosion.

Factors which may cause a break in the continuity of the grass cover and cause a reduction in the level of protection against hydraulic erosion include:-

1. any feature which causes a concentration of flow at one point, e.g. local gullies, low areas, shrubs - should be avoided.
2. shrubs, tussocky grasses, which can cause bare patches susceptible to erosion, due to the concentration of flow around them.
3. slip areas which disrupt the continuity of the slope.
4. a continuous break in grass cover, usually along the crest caused by frequent passage of people or animals.
5. gateways or fences which lead to a concentration of traffic through a small area and total destruction of all grass cover in the immediate vicinity.
6. soil shrinkage between the junction of a rigid structure and a grass structure e.g. concrete side wall and a grass surface.
7. differential settlement between a rigid structure and a neighbouring grass surface which can quickly lead to preferential erosion of the softer surface.

If overtopping does occur, then at low flows, when the depth is less than the height of the grass then the grass is not deflected and the retardance is high. When the flow depth is close or greater than the height of the vegetation the retardance drops rapidly with increasing flow while with high flows the retardance approaches a constant low value. Australian work[197] suggests on slopes described as 'steep', velocities are so high that the friction factor approaches a low constant

value independent of grass length.

Grass cover is said to reduce evaporation and prevent the embankment drying out, however, it is also accepted that grass with deep roots will cause a bank to dry out by evapo-transpiration leading to fissures forming in the clay. Hence the presence of vegetation may intensify shrinkage and fissuring of the surface but a dense cover of grass may provide an effective defence against erosion, especially where spray may be thrown up above the parapet to be carried along by the wind and its own movement, and driven on to the downstream face of the bank. Generally grass adapts to the environment and thus if the roots meet a hard compacted area they will not attempt penetration, except under unusual conditions, or where fissures exist, and so one would expect to find 'shallow' root growth in 'dry' areas, and 'deep' root growth in 'wet' areas the latter eventually causing shrinkage and fissuring.

7.3 Alternatives to seeded grass cover

7.3.1 Protection methods in general

A method which attempts to protect the downstream face of a dam against erosion by overtopping waters must satisfy certain conditions. Obviously the most important criteria to be satisfied is that the water should not detach soil particles from the face which are then able to be transported by the water. In addition any protective system should be structurally stable in itself and installed in such a way that the embankment remains stable. Obviously, cost is an item to be considered as is the method of installation. Any protective measure of this type must obviously be permeable in order that pore pressures within the embankment may dissipate but still retain fine particles while it must also be strong to resist the drag forces from the overtopping water and

and to resist impact from floating debris.

If damage does occur it would be an advantage if damage was located to small units which could be replaced. Installation should be simple, the materials being safe to handle, robust, and easily jointed on site. An impervious lining may be destroyed by overtopping, undermined or subject to upward pressure beyond their strength.

7.32 Turf

Turf may be used to provide a rapid method of protection, although the turfs are vulnerable at the joints between adjacent turfs.

7.3.3 Reinforcements

Reinforcement of grass surfaces may be broadly divided into two categories:-

(i) Surface reinforcements - designed to protect newly seeded areas against washout by rainfall and provide support for young plants e.g. jute mesh, glass fibre rovings and bitumen.

(ii) reinforcements; designed to permanently increase the erosion resistance of the combined surface above that of grass alone. These are normally completely or partially buried e.g. plastic mesh prescast concrete grids and in-situ perforated concrete slabs, e.g. 'Monoslabs', 'Dymex' blocks and 'Grasscrete'. Concrete slabs are very resistant to erosion in the absence of grass but are more prone to damage by undermining and of course involves a greater expense than grass alone.
Once erosion of the grass surface occurs, then just as with clay, the turbulence caused in the eroded area results in even greater erosion.

It is clear that grass cover will provide some protection against erosion if overtopping occurs, although it is difficult to quantify the amount of protection afforded. A compromise must be made between the length attained of the grass before some maintenance is provided or sheep are put out to grass.

7.3.4 Graded Filters

Graded filters are a well-established method of providing water permeability with fine particle retention. Traditional filters consist of combinations of graded sand and gravel. Their installation is time consuming and expensive, particularly if suitable materials are not available locally. Graded filters are awkward to lay on steep slopes and the use of graded filters usually require strict, expert supervision to ensure correct placement and subsequent satisfactory performance.

The design of any graded filter should be based on the particle size of the finest clay particle that can be eroded from the face of the surface rather than the D_{85} size. (i.e. the size of sieve through which 85% of the material will pass. The design criteria often used for graded filters are (198).

(1) $D_{15} < 5\, d_{15}$

(2) $d_{15} < D_{15} < 20\, d_{15}$

(3) $D_{50} < 25\, d_{50}$

(4) $\frac{D_{15}}{d_{85}} < 4 \text{ or } 5$

(5) $\frac{D_{15}}{d_{50}} > 4 \text{ or } 5$

(6) Gradation curve of the filter should have roughly the same shape as the gradation curve of the protected soil.

Notation

D_{15} - size of sieve through which 15% of the filter will pass.

d_{15} - size of sieve through which 15% of the material to be protected will pass.

(7) Where the protected soil contains a large percentage of gravels, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than the 25 mm sieve.

(8) Filters should not contain more than about 5% of fines passing the 75 μm (No 200) sieve, and the fines should be cohesionless.

(9) Thickness: Minimum thickness for horizontal filters:-

150 mm for sand, 300 mm for gravel

Vertical or sloping filters difficult to form.

7.3.5 Maccaferri Gabions

Maccaferri Gabions have the ability to pass through the rock filling without pore pressures being set up within the embankment. They have the advantage that if damage does occur it is localised and easily repairable. They have the ability to expand and contract and therefore can accommodate settlement. The gabions must be filled with graded stone or as an alternative 'Terram' may be placed beneath the gabions and as an additional safeguard against slipping wooden stakes can be driven through the centre of each gabion.

Approximate costs for placing gabions are:-

£5/m^2 placed in straight lines (1978 prices)

£10/m^2 placed in curved lines.

7.3.6 RENO Mattresses

Reno Mattresses 230 mm thick have been placed on a 1 in 1.5 slope. They are, in fact, 'thin' gabions, supplied in lengths of 6 metres and widths of 2 metres, and thicknesses of 170, 230 and 290 mm, the 230 mm thickness being advised for hard clay. Again a graded filter or 'Terram' would have to be used to prevent erosion.

7.3.7 Asphaltic Concrete

This method has been used for the downstream face protection of embankments but cannot be used on a fill where significant post constructional settlement is likely to occur. It is comparatively expensive, as are other forms of protection such as concrete, steel and copper. (201, 202).

7.3.8 Concrete Facing

Many of the available precast concrete products, or cast in situ products with grass filled apertures require a compacted sub-base to be formed before either precast blocks can be placed or in situ concrete can be cast. The minimum depth of sub-base is of the order of 100 mm. Some protection systems are shown below:-

'Mono-slabs' (Manufactured by Mono Ltd.) May be placed on slopes of up to 45°, but every third slab should be staked.

'Grassblock' (Manufactured by Grasscrete Ltd.) An interlocking pre-cast block system, 434 mm square and 100 mm thick, but again requiring a prepared sub-base.

'Grasscrete' (Manufactured by Grasscrete Ltd.) An in situ cast concrete system formed on a prepared sub-base, which results in a self-draining reinforced concrete facing but one which remains flexible. Expansion joints must be provided at 10 metre intervals. Costs are of the order of £13/m^2. (1978 price

7.3.9 Soil Cement

Soil cement consists of natural soil, cement and water (203) and can be especially useful where suitable rock is not available locally or long hauls are involved.

Soil cement has frequently been used for the upstream protection of

earth dams but theoretically it may be used to protect the downstream slope. Formation usually consists of soil cement placed and compacted in narrow horizontal layers"stair stepped" up the slope, the 'step-back' on layers is equal to the product of the slope multiplied by the layer thickness i.e. 2:1 and 150 mm layer gives a step-back of 300 mm (see Figure 7.1)

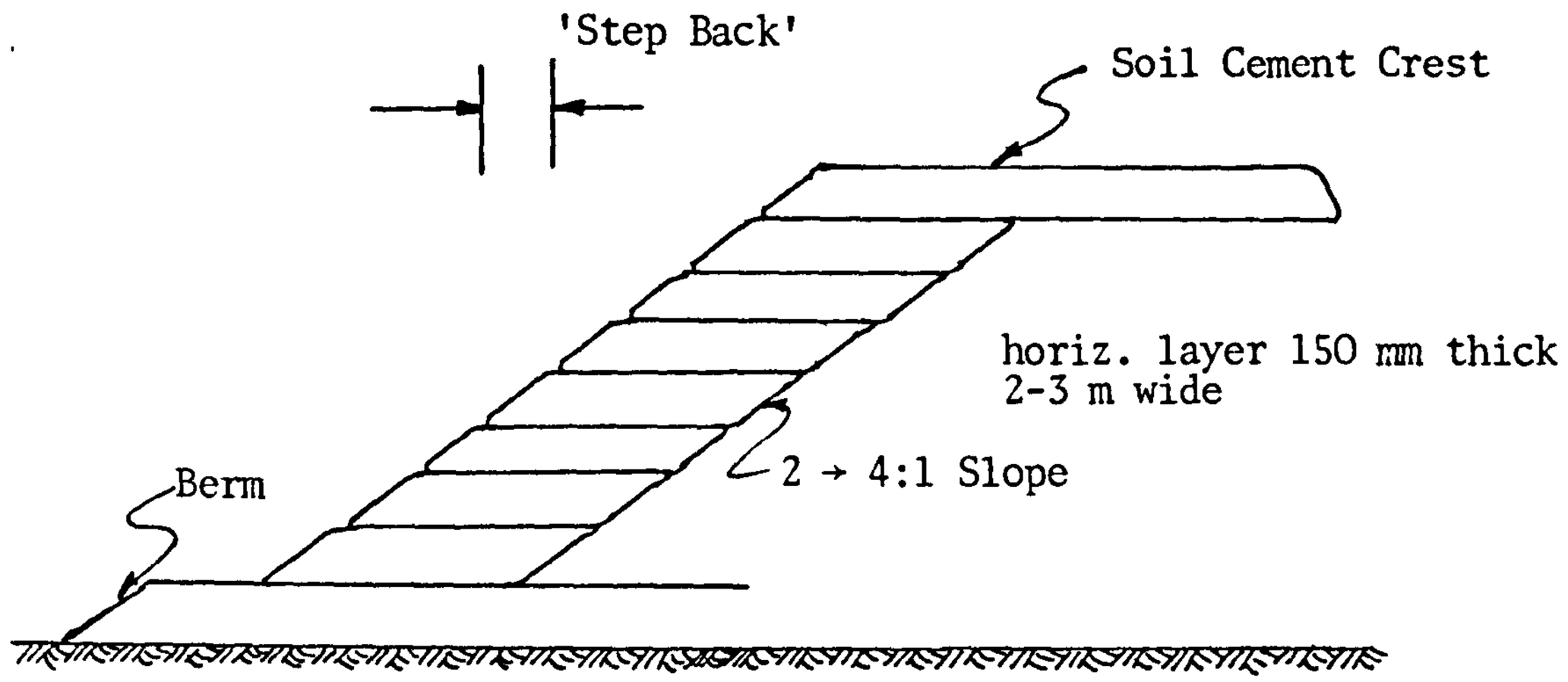


FIGURE 7.1 : Soil Cement Facing (204)

Soil gradation is important, coarse sandy or gravel soils containing 10-25 percent of material passing 75 μm (No 200) sieve being ideal. If the amount of material smaller than the 75 μm (No 200) sieve exceeds 35 percent then any effort to find a coarser material may be justified from a processing cost standpoint. Soils containing more than 50 percent finer than the 75 μm (No 200) sieve should not be used without pre-mixing

techniques. Pre-mixing ensures a more consistent material and aids faster placing. Compaction of layers is usually required to ensure that the shear strength of the fill is adequate for stability purposes. The cement content of the soils are of the order of 10-12 percent (204 205 206), of the compacted volume of the soil cement.

Curing may be aided by a curing compound, or water trucks with spray bars curing being sufficient within 7 days although no waiting time is required between layers but recently completed soil-cement layers should be protected with earth ramps.

Soil cement was used in the construction of the Vallon des Bimes dam, in France[207]. This 9 m high dam, 36 m long was constructed by 8 men in 7 weeks to store water for use during forest fires and is shown in Figure 7.2

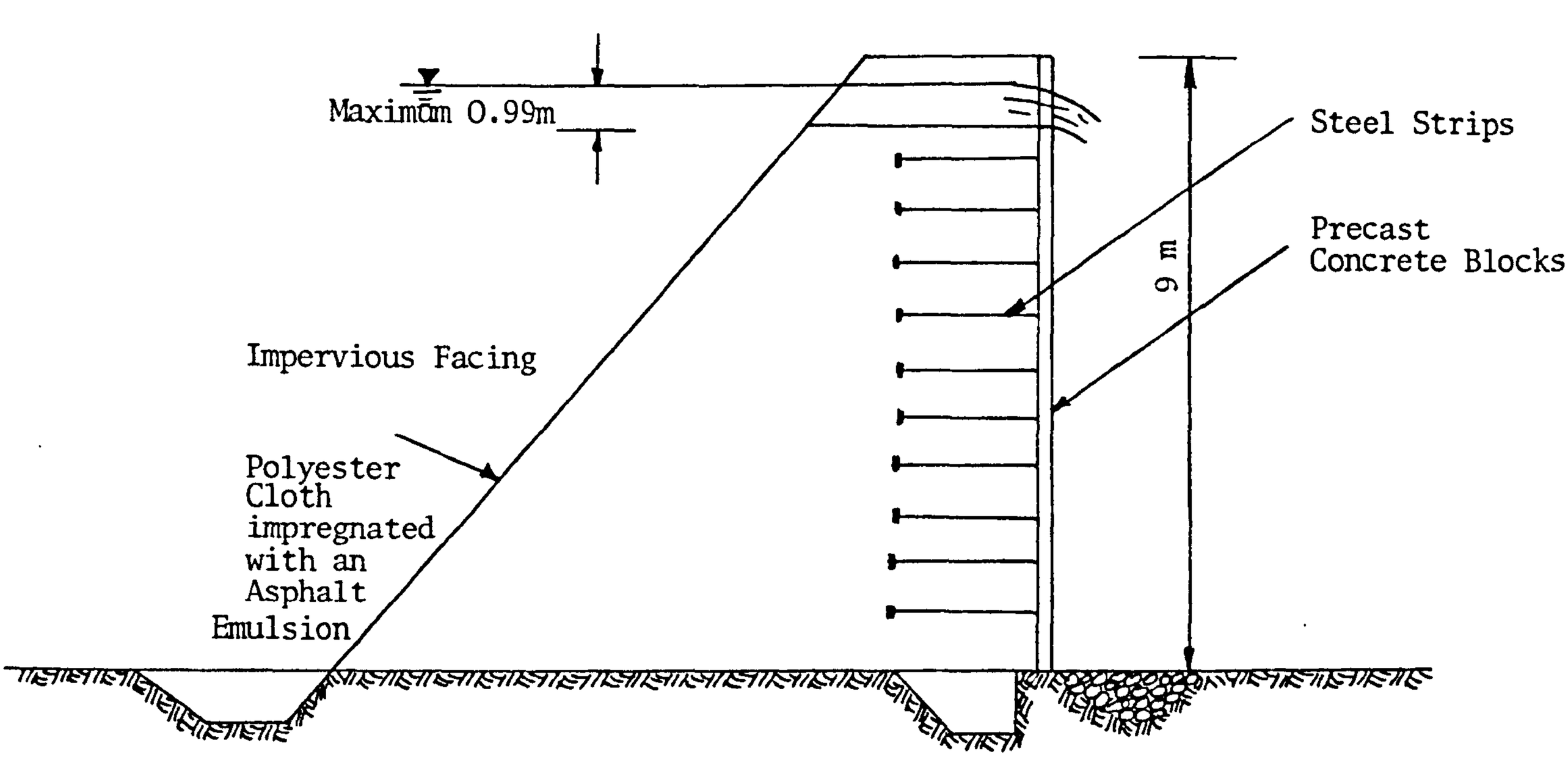


FIGURE 7.2 : Vallon Des Bimes Dam (207)

Flood water is passed over the crest, and to date flows of approximately 20 m/s have passed over the crest which acts as the spillway. The downstream face of the dam is vertical and is protected by 1.5 m square precast panels, and as a result a little more than one half of the fill required for a conventional dam was used. The cost of the structure was comparable with the cost of a conventional earth dam.

7.4 Suggested Protection Methods

7.4.1 'Terram' (I.C.I. Fibres)

Terram filter membranes have been used to replace granular filters and the Author suggests that Terram may satisfy the conditions stated in 7.3.1.

Terram has high strength and toughness, combined with good permeability and filtration characteristics, and is easy to instal[208]. Terram is available in various grades but the Author considers that only 'Terram 1000' or 'Terram 2000' would be suitable for the purpose envisaged.

Some properties of the material are shown in Table 7.1

Terram is essentially isotropic. The pore size distribution of the material is shown in Figure 7.3.

Prolonged exposure to UV light leads to gradual loss of strength and so the Author envisages that Terram would be buried especially as bacteria alkalis and acids have no effect on the material and it is not affected within a temperature range of - 40 to + 100° C.

Terram has additional advantages over conventional filters in that it has inherent tensile strength and can therefore be easily installed. It has high tear strength and abraison resistance and is lightweight with factory controlled quality and properties which are not easily impaired

Polymer Composition	Terram 1000 | 2000 67% Polypropylene 33% Polyethylene	
Fabric Weight (g/m^2)	140	240
Fabric Thickness (mm)	0.8	1.1
Tensile Properties (Wet & Dry)		
Tensile strength under uniform applied stress (N/m^2)	7500	12500
'Grab' Tensile strength (N)	800	1400
'Grab' Extension at break - %	80	90
'Wing' Tear resistance (N)	250	400
Permeability to Water ($\times 10^{-3}$ m/sec)	1.50	0.85
Specific Gravity	0.92	0.92

TABLE 7.1 : Some properties of 'Terram' Fibres
(For definition of terms see ref. 208)

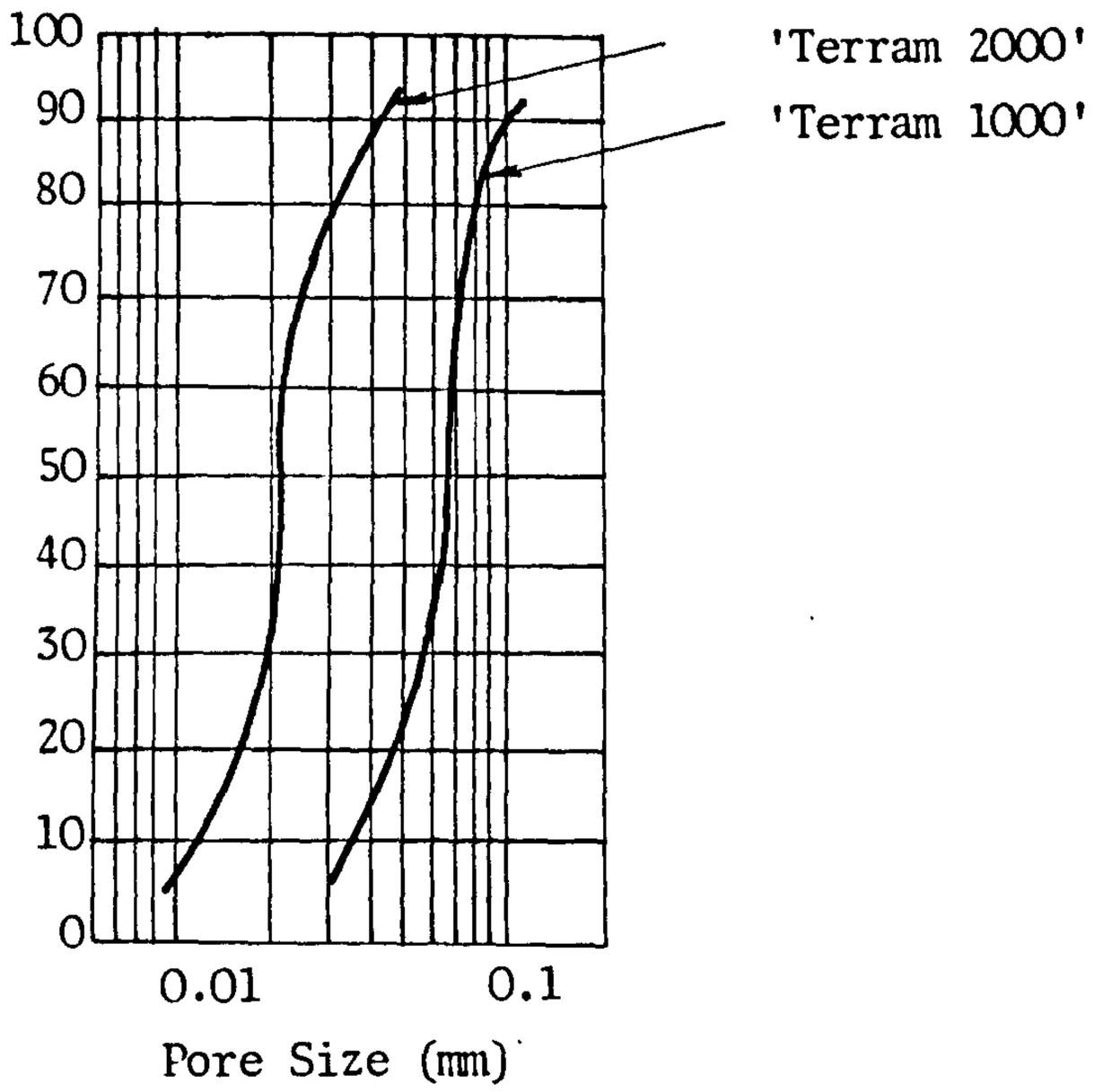


FIGURE 7.3 : 'Terram' Pore Size Distribution

by careless installation. Geographic location and availability of materials are eliminated as economic considerations in the design of the filter system, which in the case of reservoirs, can be important. Quick visual inspection assures the engineer that the filter is in place as designed. It is permanent and has uni-directional strength; being versatile and labour saving, and in wet weather machines are able to move on material placed on Terram. It is supplied in rolls 4.5m wide and 100m long. An additional property of Terram is its good frictional properties, in fact Terram has been used to improve the frictional stability of an embankment.

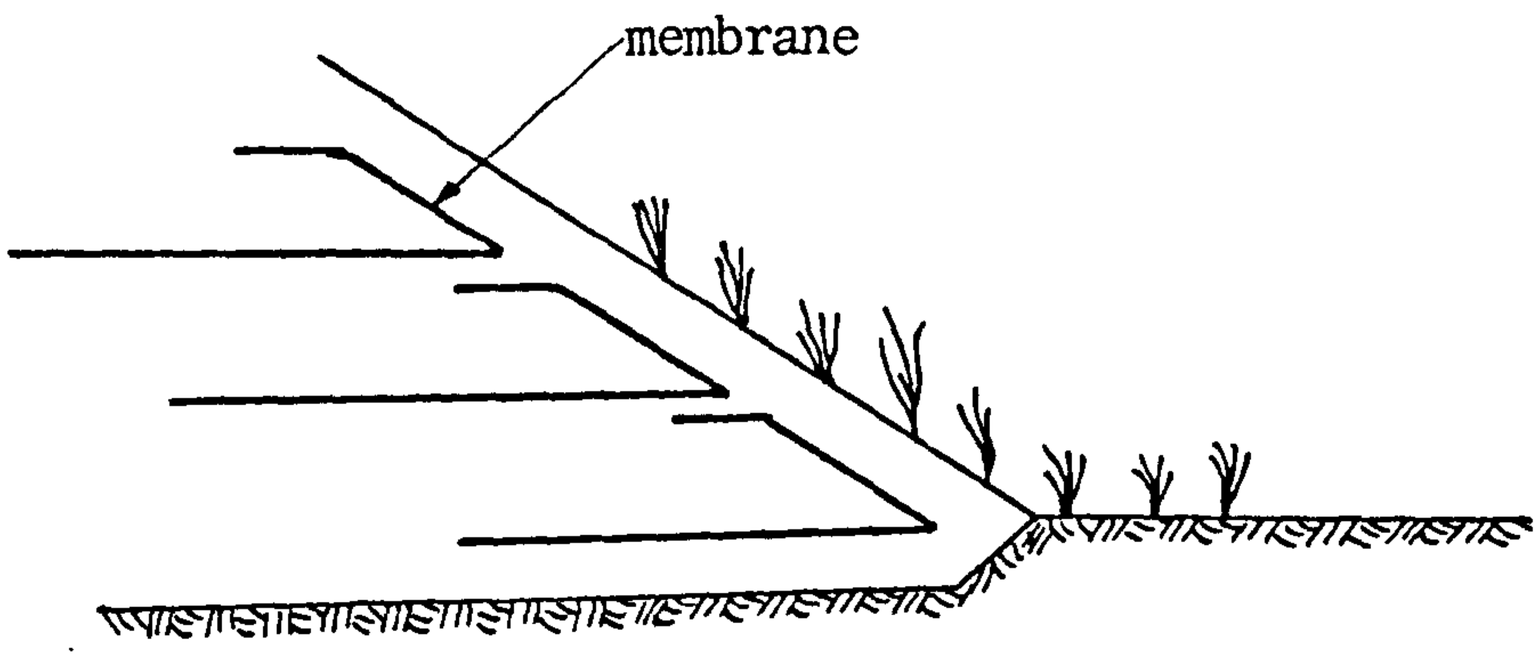


FIGURE 7.4 : Terram used to improve frictional stability (209)

If this idea could be adapted to produce a continuous layer of Terram, parallel to the downstream face, then this would act as a retarding layer if erosion due to overtopping reached this level, since the Terram will retain fine particles. This could be achieved quite easily, without disrupting the construction programme to a great extent, by virtue of the ease of fastening of the material (see Figure 7.4a) Joints should be overlapped by 0.3 - 1.0m preferably in the direction of flow. Construction equipment should not be allowed to travel directly on Terram, material being back-tipped onto the Terram and then rolled. On 600mm layers of a soil with a C.B.R. of 3% axle loads of 30000 kgs (equivalent to heavy motorized scrapers) can be sustained. Joints may be glued, stapled, or sewn, the latter being achieved by use of portable mains/battery/compressed air sewing machine.

Costs are of the order of:- Terram 1000 30 p/m^2

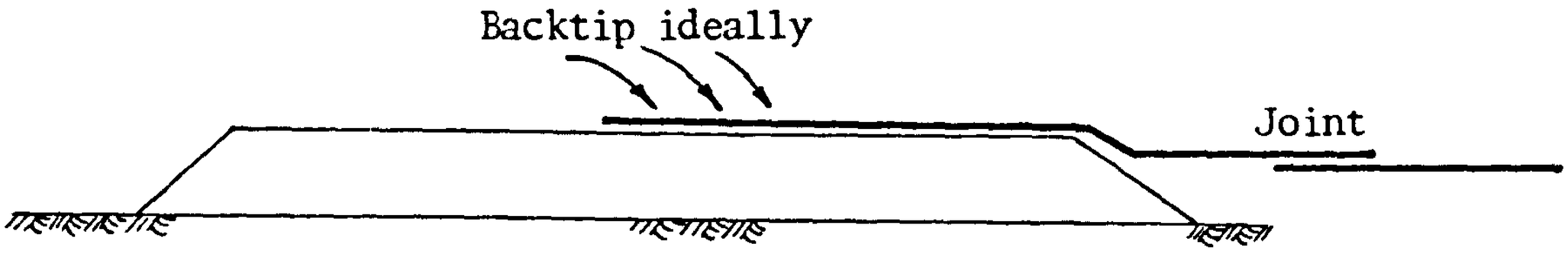
Terram 2000 54 p/m^2

7.4.2. 'Paraweb' (I.C.I. Linear Composites Limited)

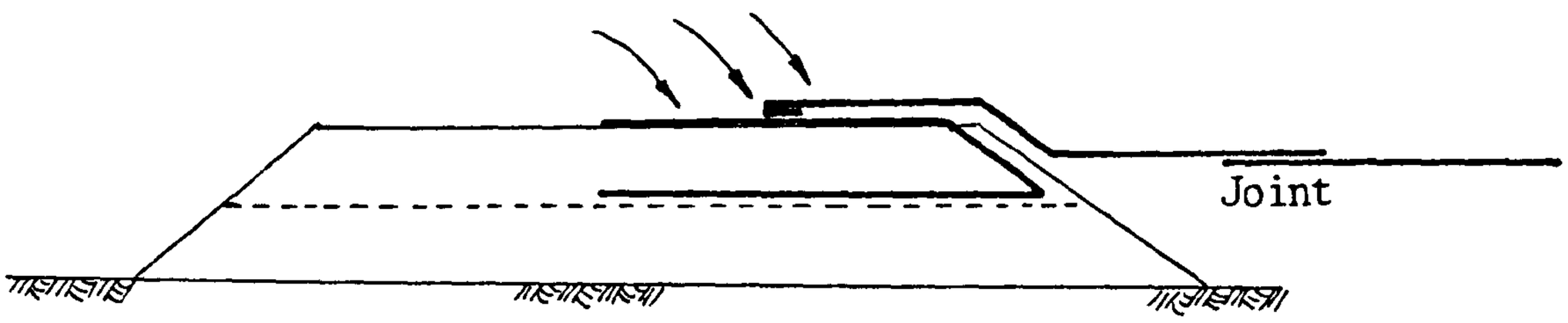
Interwoven Paraweb is a mat woven from Paraweb which is a flat high strength webbing constructed from high tenacity filaments of Terylene embedded in a tough durable sheath of black Alkathene.

The width of the webbing can be varied between 50 and 80mm. It can also be supplied with a Terram backing, and this is the form in which the Author suggests it be used to provide a surface protection. With a 'Terram' backing it is known as a 'duplex' structure consisting of '165kg woven Paraweb mat' with a 'Terram 1000' backing.

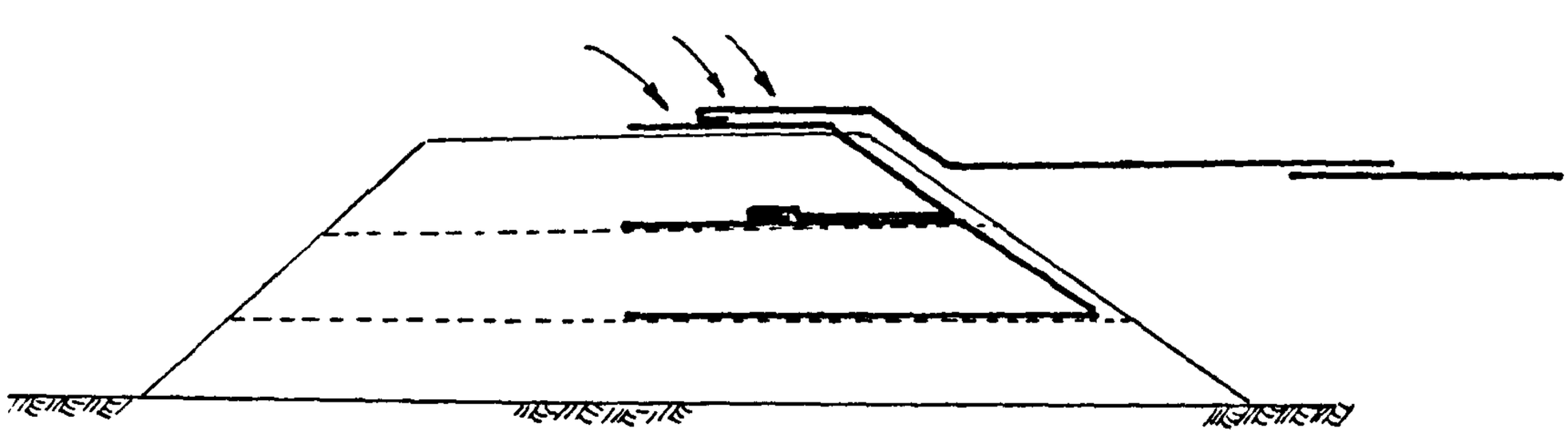
Paraweb in this form has several advantages namely :-



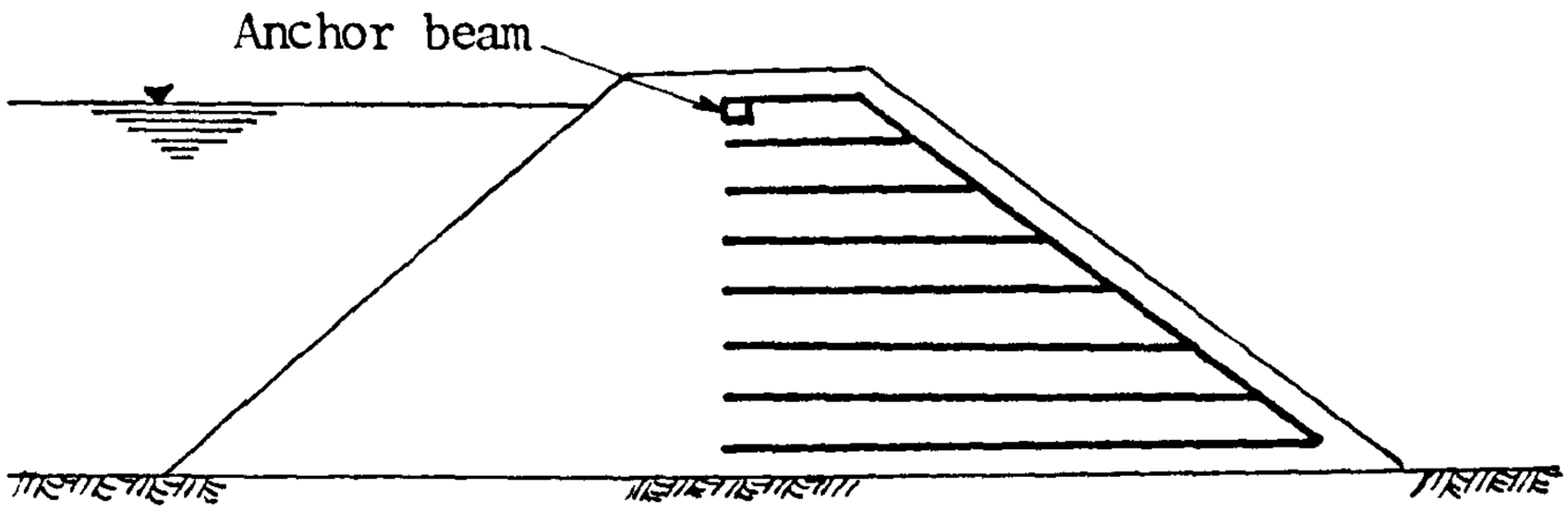
STAGE 1



STAGE 2



STAGE 3



STAGE 4 - Completed Protection

FIGURE 7.4a : Schematic Diagram showing the construction of a continuous downstream protection layer

1. A high strength, permeable membrane permitting the free flow of water but preventing the erosion effects of flowing water.
2. Lightweight.
3. Easy to instal.
4. Can be cut on site.
5. Robust.
6. Safe to handle.

Some of the properties of the duplex structures are shown in Table 7.2

	Paraweb (165 kg) + Terram 1000	Paraweb (400 kg) + Terram 2000
Tensile Strength/metre N/m	3135	7600
Weight	1.466	2.054
S.G.	0.858	0.875
Permeability (m/sec x 10^{-3}) (under const.head of 0.1 m)	6.7	6.7

TABLE 7.2 : Properties of Paraweb + Terram Duplex Structure

Obviously, the Terram in this situation prevents the removal of fines while the Paraweb affords some protection from the flood. debris. This type duplex structure has been successfully used where fast flowing water has transported up to rocks weighing up to 0.25 tonne. Again, if not buried the black alkathene (polyethylene) covering the high tenacity filaments of Terylene (polyester) suffers UV deterioration whereas if buried this problem does not arise and Paraweb is resistant to chemical attack.

Paraweb duplex consists of plain Paraweb with bonded peripheral

edges and with Terram bonded to the underside at the peripheral edges. The sheets of Paraweb, which are normally 100 metres long and 4.5 metres wide, must be securely fastened together, and to prevent loss of fines, an overlap between sheets must be provided of at least 20 millimetres with an additional one metre wide strip of Terram beneath the joint.

The methods of joining mats include:-

(1) 'Parafil' ; Mats may be joined by lacing the edges of the mats with 'Parafil', a 7.1 millimetre diameter rope of a similar construction to that of 'Paraweb', this providing the strongest joint (Figure 7.5)

(2) Bolts ; Twelve millimetre diameter holes in the edge of the mats allow the mats to be bolted, and/or allowing stiffening of the edge of the mat by bolting through metal or wooden strips (Figure 7.6)

(3) Rods and Bars ; Rods and bars may be inserted through the weave and then linked with bolts, fishplates or parafil (Figure 7.7

(4) Nails and Screws ; Nails and screws may be put through the edges of the mats into wooden battens beneath (Figure 7.8)

(5) Site welding ; Site welding is possible by use of a portable heat welding machine.

(6) Staples ; When mats have been overlapped they may be stapled with portable staple guns using a staggered staple pattern. Staples may be put through the mats alone, or through the mats into wooden battens (Figure 7.9)

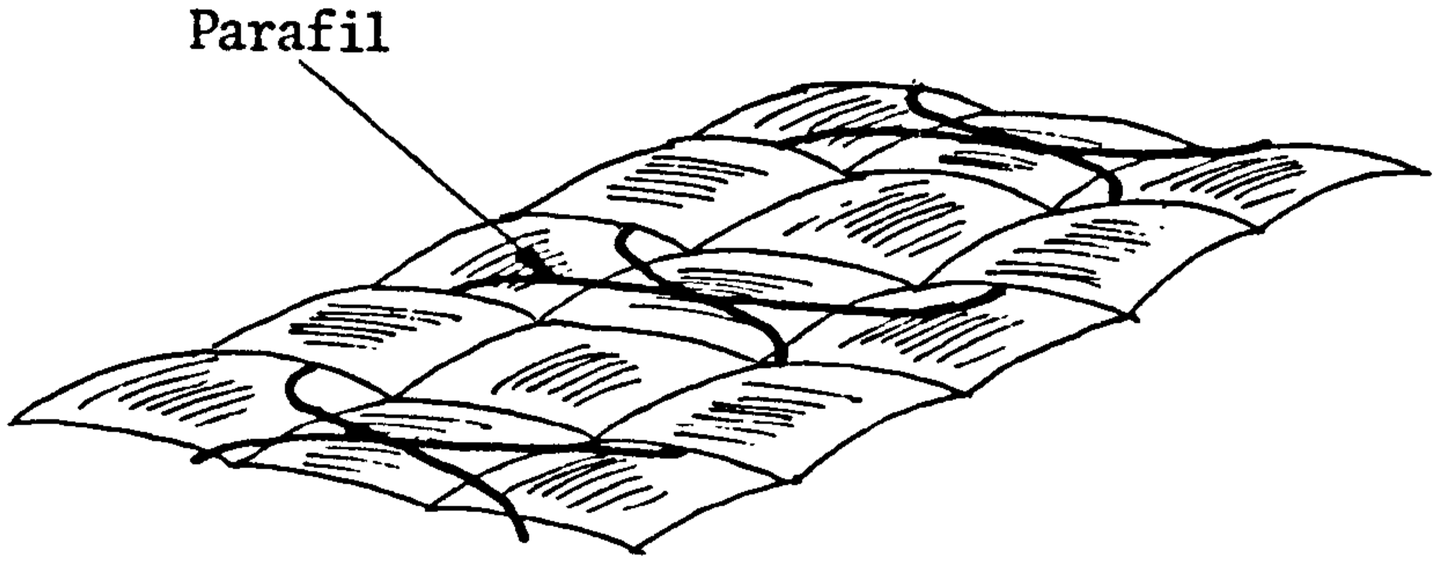


FIGURE 7.5 : Interlacing through Weft & Warp

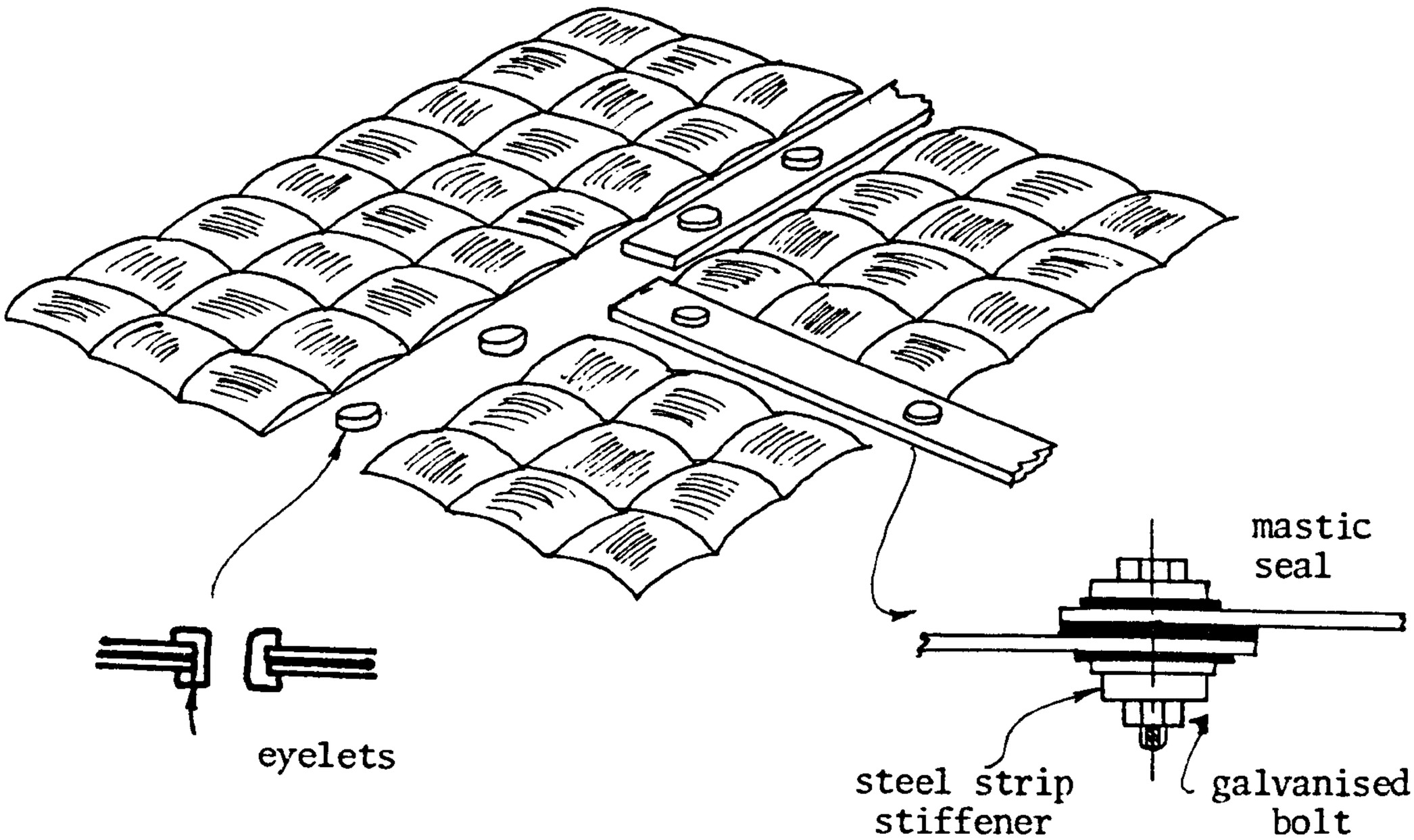


FIGURE 7.6 : Bolted Joints with stiffener or eyelets

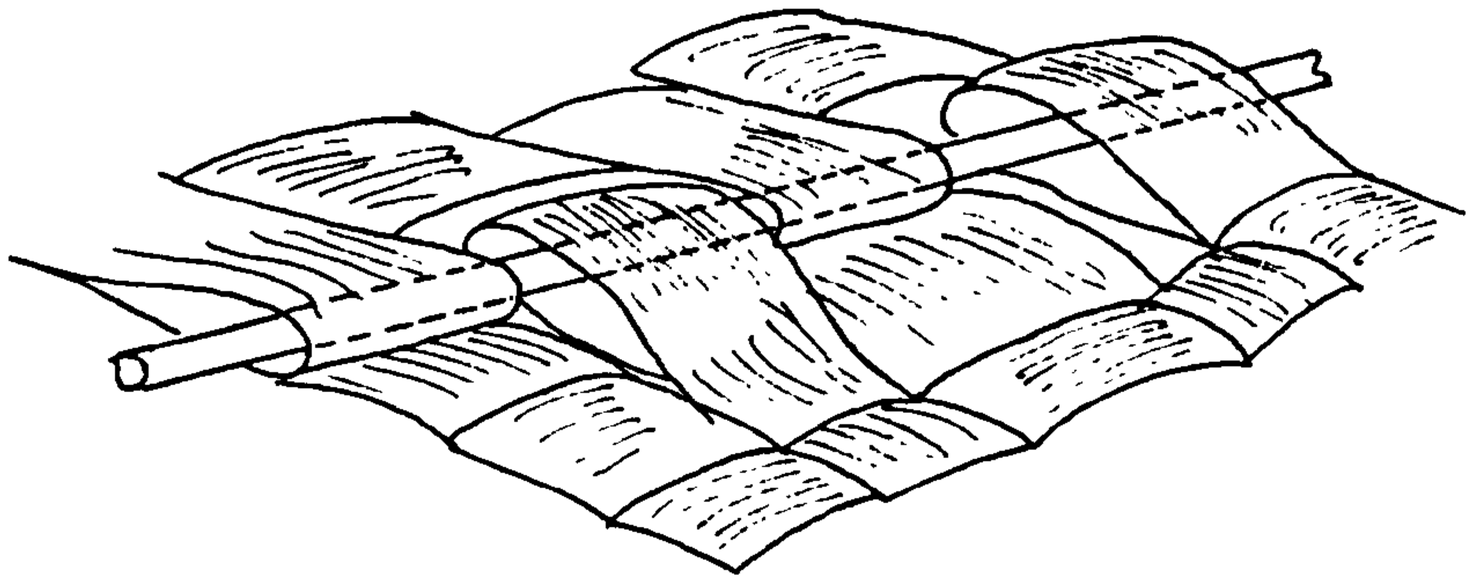


FIGURE 7.7 : Rods & Bar jointing

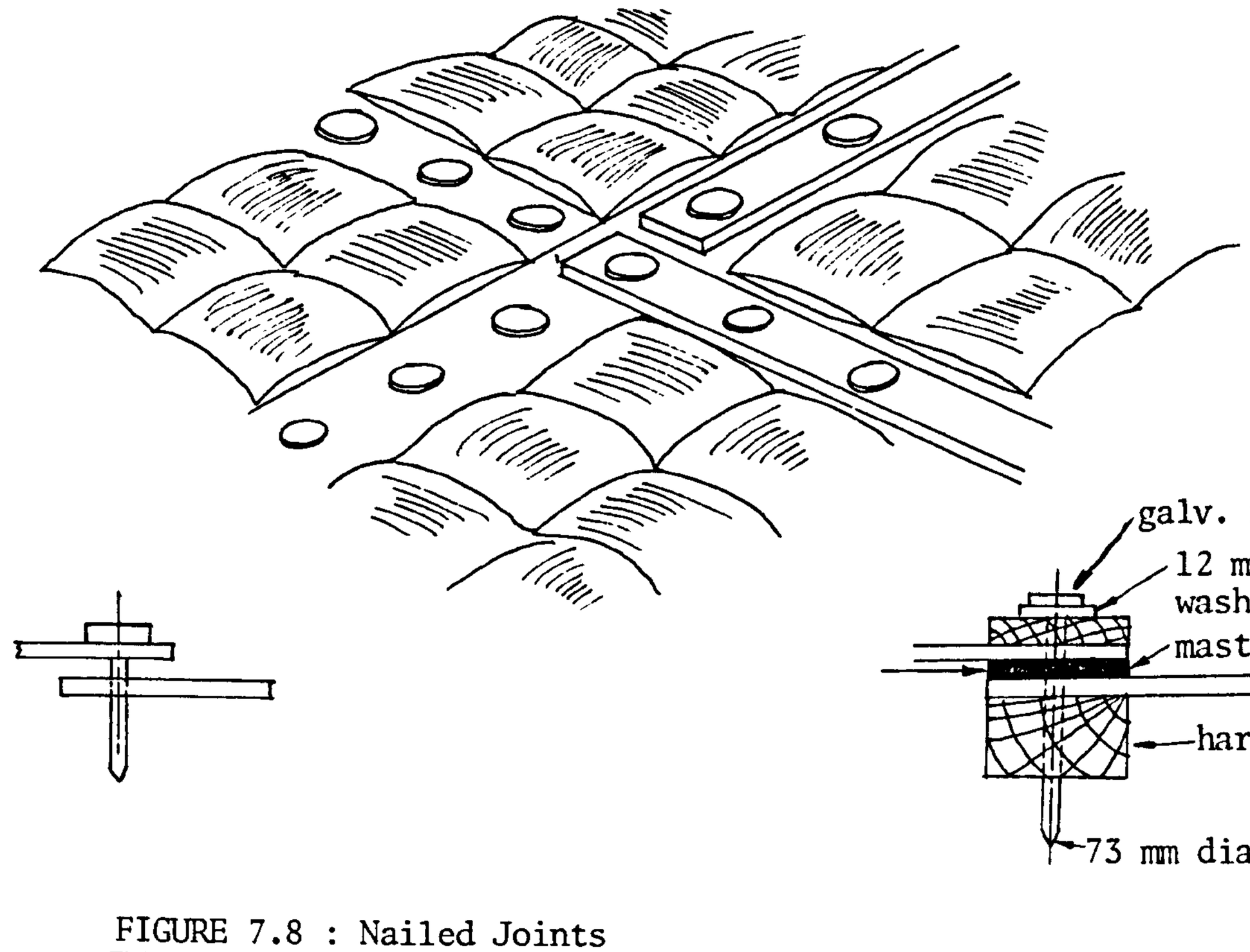


FIGURE 7.8 : Nailed Joints

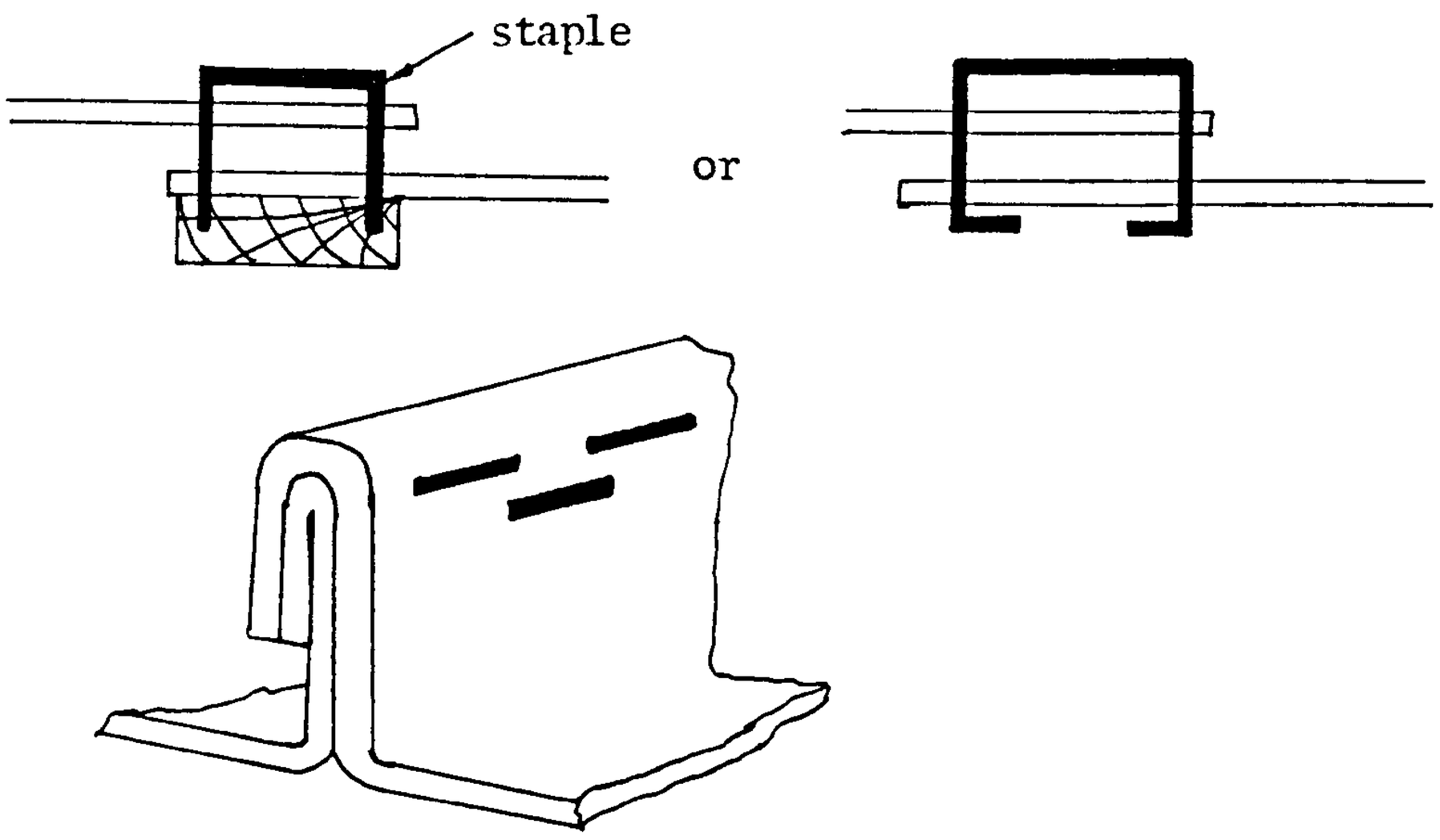


FIGURE 7.9 : Stapled Joints

In the case of an embankment slope the top of the protection method must be anchored at the crest of the embankment, and at the base, and ideally at points on the downstream slope. The crest anchorage usually would consist of a concrete beam set into the crest of the embankment (see Figure 7.10), and this form of construction, together with rip-rap protection may be adopted for the toe beam. Intermediate anchorages may be made by 800 millimetre long, 20 millimetre diameter steel pins with welded washer heads driven straight into the bank in the case of hard clay, - 24 spikes per mat, or by 200 millimetre long pins driven into 40 millimetre square wooden pickets 1.5 metres long driven into the bank in the case of soft material.

Approximate costs for placing Duplex Paraweb are:-

400 kg Paraweb + Terram 1000

£3.89/m^2 for up to 10000 m^2)
£3.89/m^2 for up to 150000 m^2)) but discounts available.

Additional Materials

500 kg Parafil - Cost per 300m coil	£38.00
Staple Gun (pneumatic)	£104.00
Staples - Nickel Silver	£13.00 per 1000

7.5 Indirect Methods of Slope Protection

Obviously, the alternative to controlled overtopping coupled with downstream protection is to provide measures which prevent overtopping. Restriction of the top water level will prevent overtopping but may lead to operation problems as well as a loss of stored water. A method which has been used to provide an emergency spillway capacity and which may be used more in the future is the fuse-plug or breachable embankment.

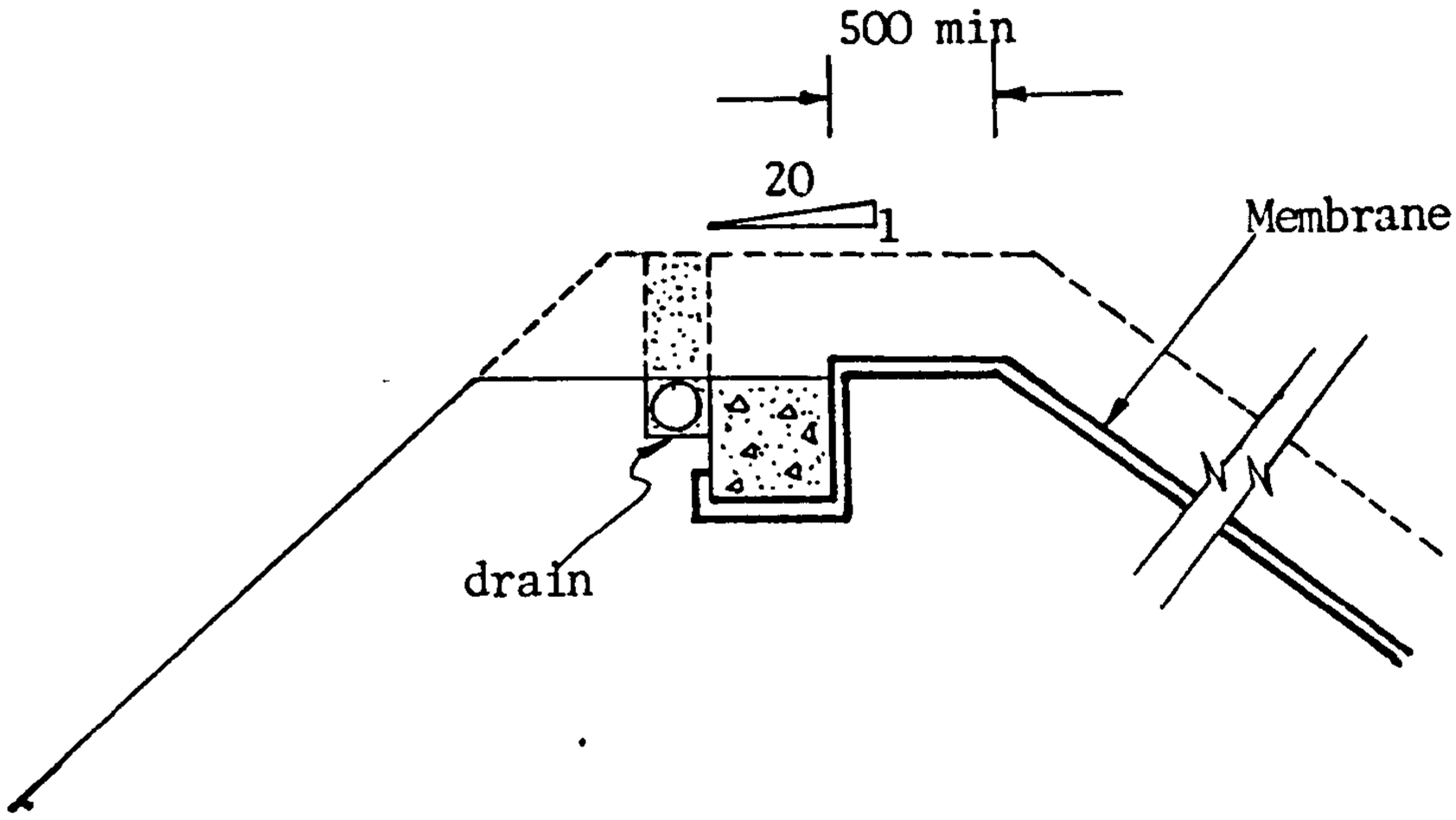


FIGURE 7.10 : Crest Anchorage for Protection Methods

7.5.1 Fuse-plugs A fuse-plug is an embankment which, in time of flood is overtopped and carried away, thus giving the necessary relief, while for ordinary floods a spillway of any of the usual types is constructed, the philosophy being that this plan permits a considerable saving in spillway costs, the interest on which would more than pay for the parts of the fuse-plug which require replacement as, at most, it would be washed away not more than two or three times a century.(210 211 212)

The spillway passage is controlled by an embankment of selected materials which, when an emergency arises, is automatically removed, just as a fuse is blown out in an electrical circuit. Upon removal the extra spillway capacity is provided to ensure the safety of the dam.

Topographic conditions may limit the length of a spillway on a dam, making a gate controlled spillway the only obvious solution to the problem. Gated spillways, especially to cater for large capacities, are costly and in this instance it may be economical to provide a fuse-plug spillway.

The provision of a fuse-plug, specially requires a second valley as shown in Figure 7.11, . or at least an area in which a wide shallow channel can be excavated economically and preferably in material, i.e. earth or rock, which can be used in the construction of the embankment. Ideally it should be situated so that the repairs can be made without affecting the water level in the reservoir.

The use of the fuse-plug has been very limited due principally to general scepticism about their use, and that very little research has been carried out on the subject of their behaviour and design. Fears exist about the effects of vegetation binding the material of the fuse-plug together and generally about the ageing effects which may alter the

erosion characteristics of the fill. Explosives dropped from a bridge built over the fuse-plug could remove any resistant fill and give the required spillway capacity in times of emergency.

The fuse-plug should be as low as possible so that, if washed out the loss of reservoir storage is a minimum. Sectionalisation of the fuse-plug, by steel sheet piling, gives a sequential operation from one end to another, keeping replacement costs to a minimum, and more advantageously, producing a controlled overflow (Figure 7.11a). An additional advantage is, that by manipulation of the main spillway gates, a virtually constant overflow can be achieved. To prevent unexpected breaching due to wave action in the reservoir, a generous freeboard, higher than the probable wave is usually provided between the normal full reservoir level and the invert of any pilot channel constructed to initiate erosion.

The preferred construction for a fuse-plug is a rockfill structure dam with an inclined impervious core. Rockfill is generally used, as it is a coarse cohesionless material for which the amounts of erosion under different conditions can be easily calculated, and the inclined core is preferred, because the dead weight and the water loading would aid failure when the downstream material has been removed.

The foundation of a fuse-plug must be non erodible, e.g. a concrete slab, since it is essential that the discharge is controlled during use, and it also facilitates the replacement of the plug after washout. Tractive theory should be used to ensure that material removed from the fuse-plug does not fill or obstruct the spillway channel.

The fuse-plug is designed as a normal dam or embankment to ensure it remains in place under all normal operating conditions until an

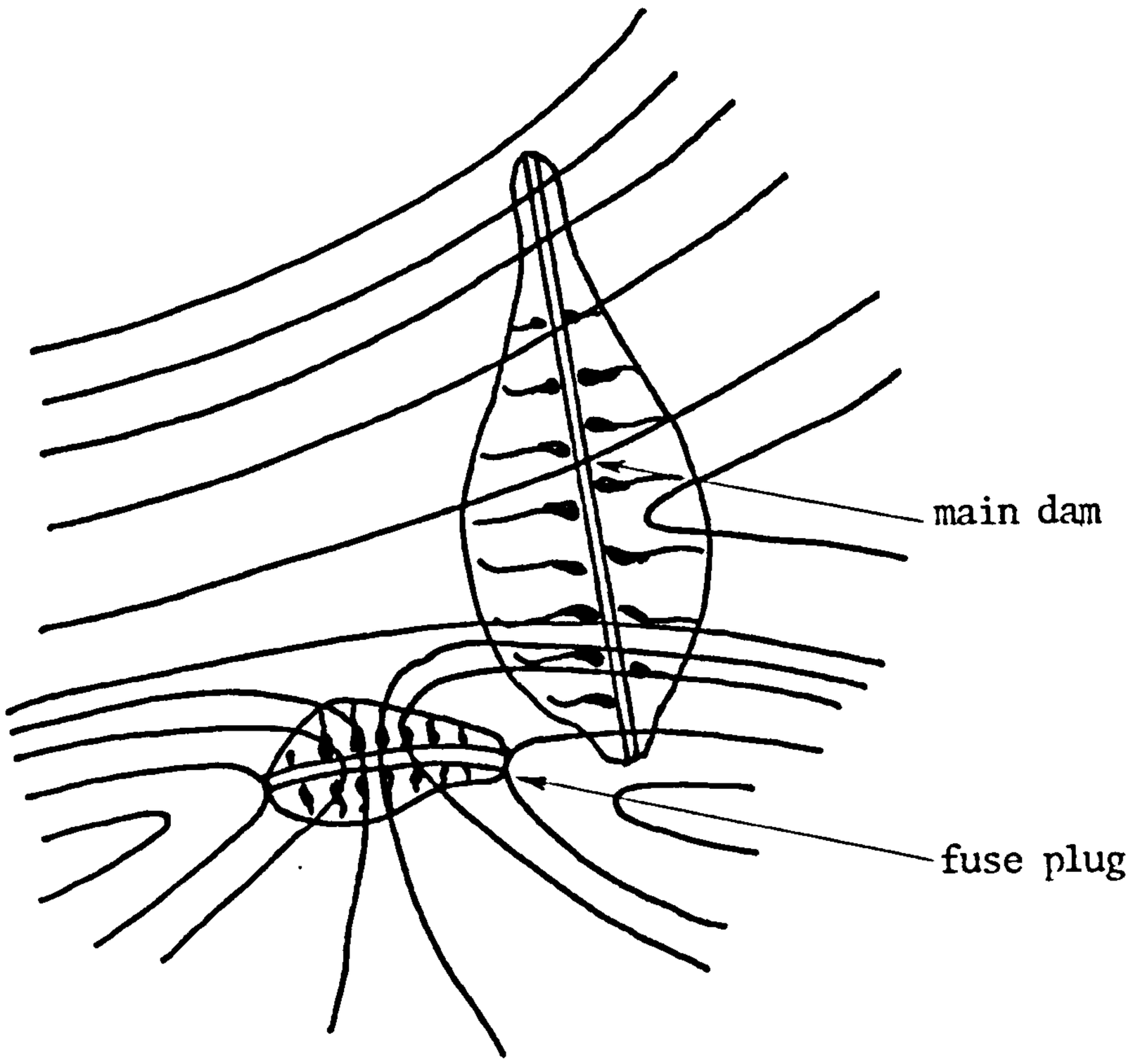


FIGURE 7.11 : Ideal Locality for a Fuse-Plug

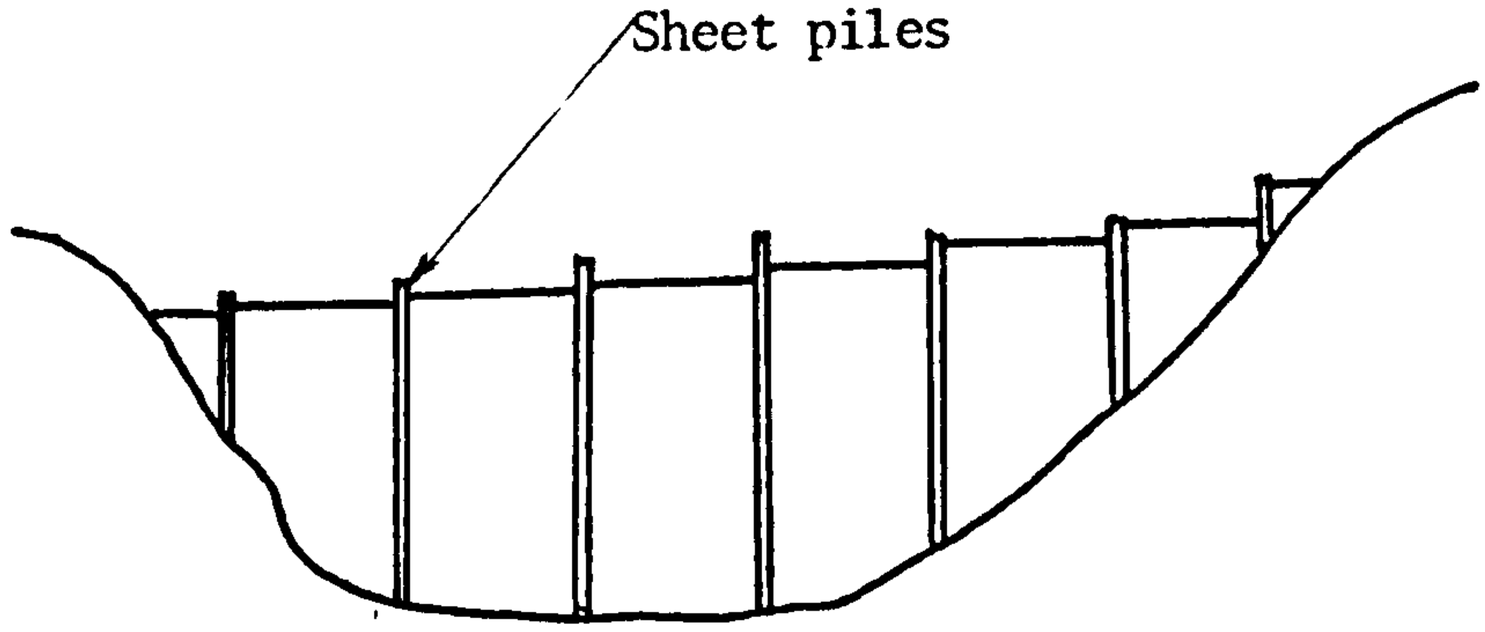


FIGURE 7.11a : Longitudinal Cross section showing Sectionalised Construction for Progressive Failure

emergency arises.

In 1959 the Idaho spillway of the Oxbow Project, on the Snake River, was used as the site of a large scale test on a fuse-plug. Initially the theoretical analysis was completed and a series of laboratory model tests used to establish principles such as the washout process and evaluate the lateral erosion process, and to find the relationship between the rate of washout and size of material, while the large scale test of the dam site was used to confirm the design assumptions.

Data obtained from the small and large scale tests indicated that:-

(1) It is essential to have clean uniform material for the downstream slope in order to get rapid breaching.

(2) The mechanics of washout can be explained by the laws of sediment transport.

(3) The shape of the eroding face depends on the material; uniformly graded material without fines produces a slope of the eroding face which is gradual and material rolls continuously into the flow whereas materials having some apparent cohesion or mechanical interlocking have an eroding face which will stand almost vertical.

(4) The washout rate is a function of the grain size, the larger the size the slower the rate of washout, although the effect is not pronounced.

(5) The washout rate for a fuse-plug is between 1.05 - 1.20 times faster than the corresponding half-scale model.

(6) The 'hold up' of the clay core is insignificant.

(7) Increasing the volume of rockfill (by reducing the thickness of the clay core) decreases the washout rate slightly.

(8) A pilot channel 1.0 metres deep and 3 metres wide was sufficient to initiate a breach in 37 millimetre concrete aggregate when 0.3 metres of water passed through it.

(9) A sloping crest, having a vertical difference of 0.3 metre between the right and left abutments, would avoid the possibility of general overtopping.

(10) The fuse-plug should be constructed of clean uniform material 18 - 37 millimetres ($\frac{3}{4}$ - $1\frac{1}{2}$") aggregate for the first fifteen metres to give a rapid removal of material while the remainder should be constructed with well graded material from medium sand to 75 millimetre gravel.

(11) The average washout rate for a uniform material was 1.7 metres per minute while the rate for a well graded material was only 0.4 metres per minute.

(12) The upstream slope of the plug should be protected from wave action by a layer of broken stone.

(13) The downstream slope of the fuse-plug should be protected from erosion due to rainfall by quarried rock of up to 150 millimetre maximum size.

(14) The breach will form before lateral erosion takes place. Jayasinghe[213] also found that:-

(15) The time of failure of the clay core can be controlled by 'weakening' the clay by the addition of quantities of sand.

(16) The rate of failure of the clay core of the fuse-plug is a log function of the percentage of sand in the core material.

Fuse-plugs have been used at a number of schemes throughout the world. A fuse-plug was installed at Bridgeport Dam on the Walker River[214] U.S.A. The reservoir capacity was 52000 t c m created by a dam 19.4 metres high and 183 metres long. Twenty-two years of flood

records showed the greatest flow to be 42 cumecs resulting in the 100 year flood being 254.7 cumecs. This value was used for spillway design, and it was found to be too expensive to provide one spillway to cater for this flow primarily due to topography of the site. As a result a small permanent spillway, four siphons, and a fuse-plug were provided.

In 1953, at Burford Dam[74] on the Chattahoochee River in Georgia it was found to be five million dollars less expensive to raise the embankment 12 metres to take advantge of a favourable saddle in the abutment than to build a concrete ogee spillway. Fuse-plugs have also been used at Kanopolis Dam, Kansas, Blakely Mt., Arkansas, Cherry Creek, Colorado, Buffalo Bayon, Texas and many other sites, but it is often argued that they have not been in service for a sufficient length of time to have proved their worth.

7.6 Additional Protective Measures

Considerable foresight was shown at the site of a reservoir in the Glencorse valley, 13 kilometres from Edinburgh[215] During the construction of the reservoir in 1821, a flood nearly overtopped and washed away the partly completed embankment. Tarpaulins and other sheets were temporarily spread over the soft material of the bank in case flood waters overtopped the bank.

Marsland[216] devised a buried louvre, as shown in Figure formed in asphalt and concrete, which would allow penetration of rainfall and the safe passage of floods. Being buried allows the asphalt to be laid inexpensively on a steep slope. Peat asphalt laid on the surface, through which grass could grow was a refinement since it is more resistant to erosion than turf, but porous.

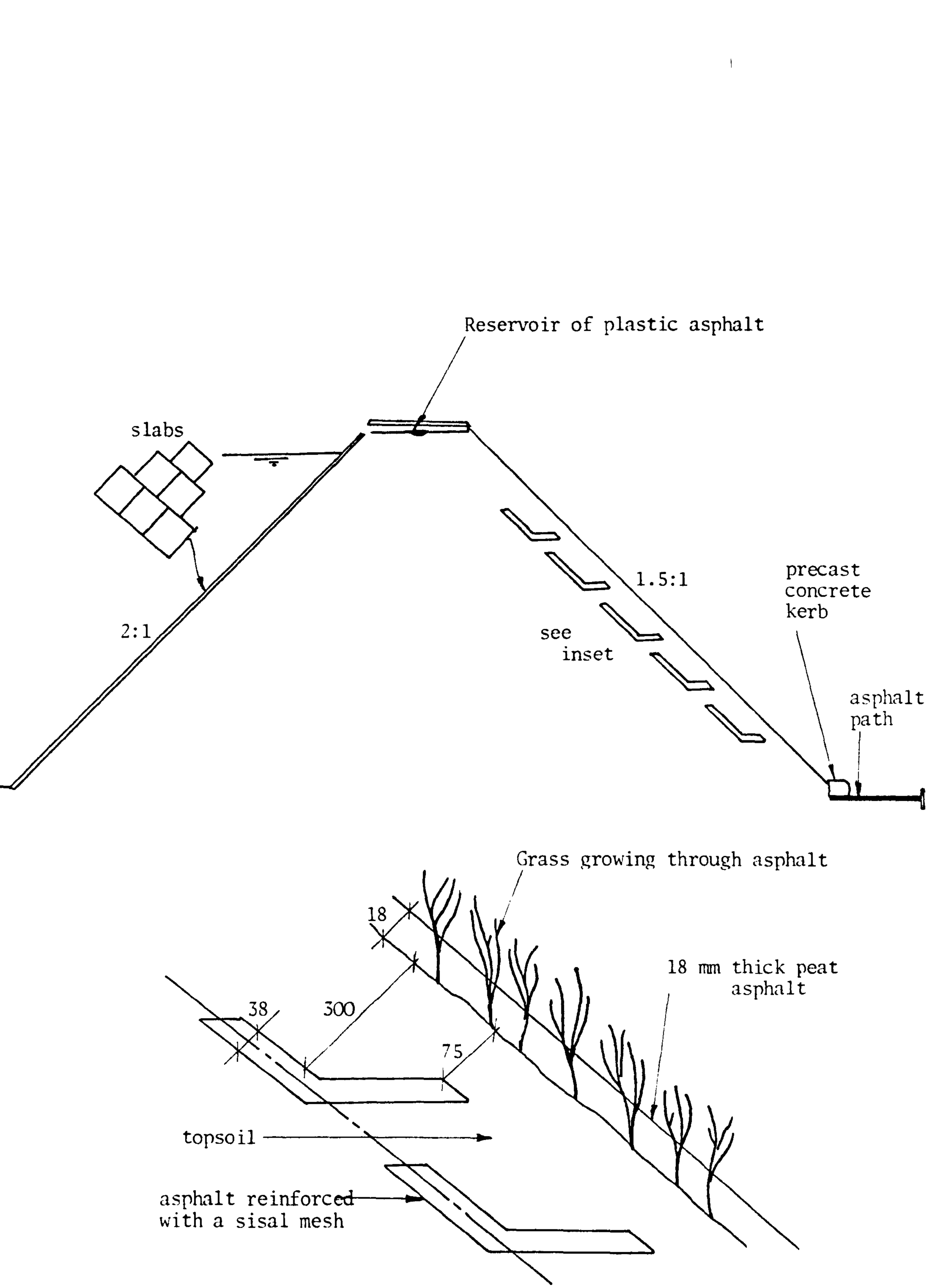


FIGURE 12 : Marslands' Buried Louvre (216)

7.7 Closing a breach

Unless the flood flow is decreasing then any attempt to close the breach may be futile since overtopping will occur at some other point. There is no general procedure for closing breaches but it is clear that initially an attempt must be made to prevent further enlargement of the breach, then reduce the velocity through the breach, seal the breach, and then strengthen the closure.

Much depends on planning, site conditions and access, depth of scour holes, velocity of water, and the availability of suitable plant and materials.

If flow has ceased or is slow, then sandbags placed well clear of any scour holes for stability, and placed in the same manner as brickwork may be sufficient to prevent further erosion. Gabions filled with sandbags may be used to fill a breach, but in any attempt to close a breach, it must be recognised that such an attempt, if unsuccessful, will result in a concentration of flow and increased erosion.

The most satisfactory method, if a breach has not developed, is to quickly increase the height of the embankment. This may be done with sandbags or, where mater l and plant is available, and a wide crest permits access, to place material on the crest. This was the case with the Oros Dam[110] where 0.4 metres of uncompacted fill was placed on the crest and withstood 0.8 metres of overtopping water before the dam failed.

7.8 Conclusions

It is clear that various methods can be used to protect the downstream face of an embankment and the choice of any protective measure must be based upon economic and physical considerations. The frequency of overtopping is such that expensive insitu concrete protections is

usually not feasible and for most dams, in this country at least, the downstream is grassed. Research has shown grass to be a valuable protection against overtopping flows but may not be sufficient protection for all flows. Vegetation is useful because it intercepts precipitation and breaks the impact of raindrops and thus prevents dispersion of the soil particles by rain splash and the formulation of a compacted surface layer with a low infiltration rate. Vegetation also interrupts to sub-divide overland flow around the plant stems and leaves,and roots physically bind the soil within the roof mat, improving soil structure and infiltration rates. A compromise has to be made between long grass which may protect bare patches and short grass which permits deformations and leakages to be seen. Sheep are often used to graze embankments but care must be taken to prevent tracks and bare patches being formed.

Fuse-plugs in certain instances may be a useful and economic alternative to expensive spillway facilities and their use should be considered where suitable sites and materials exist.

CHAPTER EIGHT

HYDRAULIC CONSIDERATIONS

8.1 Introduction

This chapter reviews those aspects of hydraulics and hydrology which are most relevant to dam failures, together with related features highlighted by the experimental investigation and the historical review of incidents and accidents.

8.2 Hydraulic Considerations

8.2.1 Peak Flows

It is often stated in texts that immediately after the instantaneous 'removal' of an entire embankment/dam the depth of flow will equal four ninths (44%) to one half (50%) of the initial depth of water retained in the reservoir before failure. There are clearly no types of failure which can conceivably result in the instantaneous removal of an embankment dam and to which this rule can therefore be applied.

Schoklitsch's equation for peak discharge relates to the width of a dam at the original water surface, (W_d) and the depth of water behind the dam before a breach occurs, (Y_o), to peak discharge, (Q_{max}); the equation is shown below:

$$Q_{max} = \frac{8}{27} W_d \sqrt{g} Y_o^{3/2}$$

Roughness was shown to have little or no effect on the magnitude of peak discharge which appears to be a function of breach size and shape. As a result the equation was 'modified' to include the variables, W_b, width of breach at original water surface and, D_b, depth of breach. The modified equation is:-

$$Qmax = \frac{8}{27} W_d \sqrt{g} D_b^{3/2} \frac{W_d}{W_b} \frac{Y_o}{D_b}^{0.28}$$

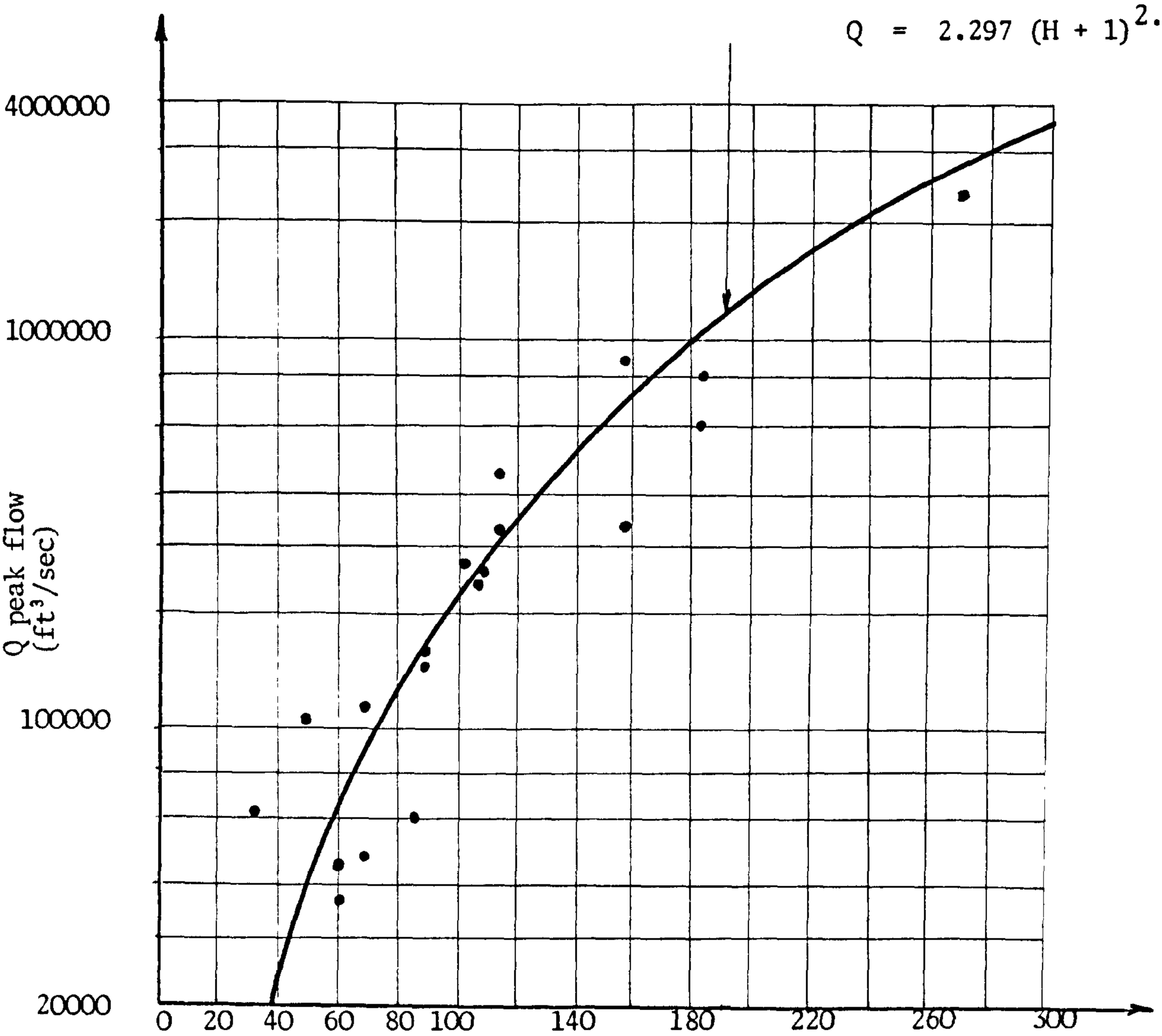
for any given breach provided that

$$1.0 < \frac{W_d}{W_b} \frac{Y_o}{D_b} < 20$$

While this equation may have some use in certain instances when considered in conjunction with the information presented in the following section, it is clear that each failure is unique in some respect and that peak discharge, and also the shape of the failure flood hydrograph, depends on many variables which must include reservoir level, storage capacity, size and shape of breach, position and rate of formation of breach and inflow to the reservoir.

8.2.2 Actual Peak Outflows

An empirical plot of height of dam if overtopped, or depth of water at time of failure if not overtopped versus estimated peak outflow is shown in Figure 8.3. This graph is reproduced in "Floods and Reservoir Safety : An Engineering Guide[24] and "Water Power and Dam Construction"[224]. Rossington[225] used this graph to estimate very approximately the time required to empty several of the reservoirs within the Pennine Division of the North West Water Authority, and



H = height of dam if overtopped, or depth of water at time of failure if not overtopped, in feet.

FIGURE 8.3 : Peak Floods from Dam Failures (after [224])

obtained the times shown in Table 8.3. However, these results were obtained using a rectangular hydrograph with constant discharge equivalent to the peak discharge as shown in Figure 8.4. It is felt, however, that a more realistic, although still idealised, hydrograph might be triangular, as shown in Figure 8.5. Calculations on this basis are shown alongside Rossingtons in Table 8.3

Dam Height (m)	Approx. Gross Cap.of Reservoir ($m^3 \times 10^6$)	Approx.Peak Flow from Fig.8.3 (m^3/sec)	Time to Empty (secs) Rossington	Hughes
40	5	13,000	388	776
38	3	10,500	298	596
37	0.6	10,000	61	122
34	3	8,000	388	776
33	5	8,000	654	1308
31	3	6,000	512	1024
30	0.2	6,000	35	70

TABLE 8.3 : Estimated Times to Empty Reservoirs

If the equation for the curve is applied to historical data from documented failures, such as at Dale Dyke (1864), the following results are obtained:-

In Imperial Units:-

Height of embankment 95 ft (28.96)

$$Q = 2.297\ (H + 1)^{2.5}$$

$$= 2.297\ (96)^{2.5}$$

$$Q = 207414 \text{ cusecs} \quad (5878\ m^3/s)$$

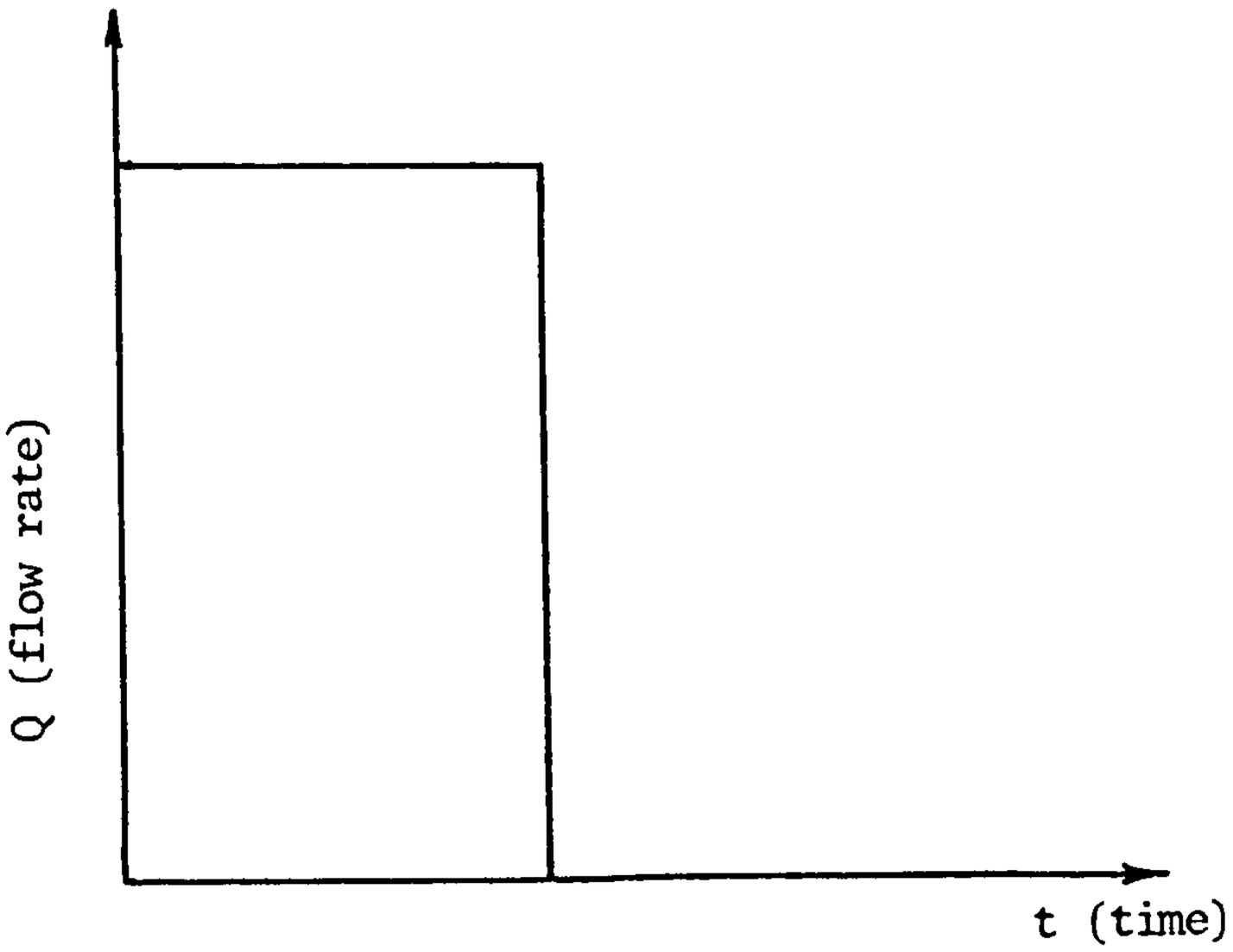


FIGURE 8.4 : 'Rectangular' Hydrograph

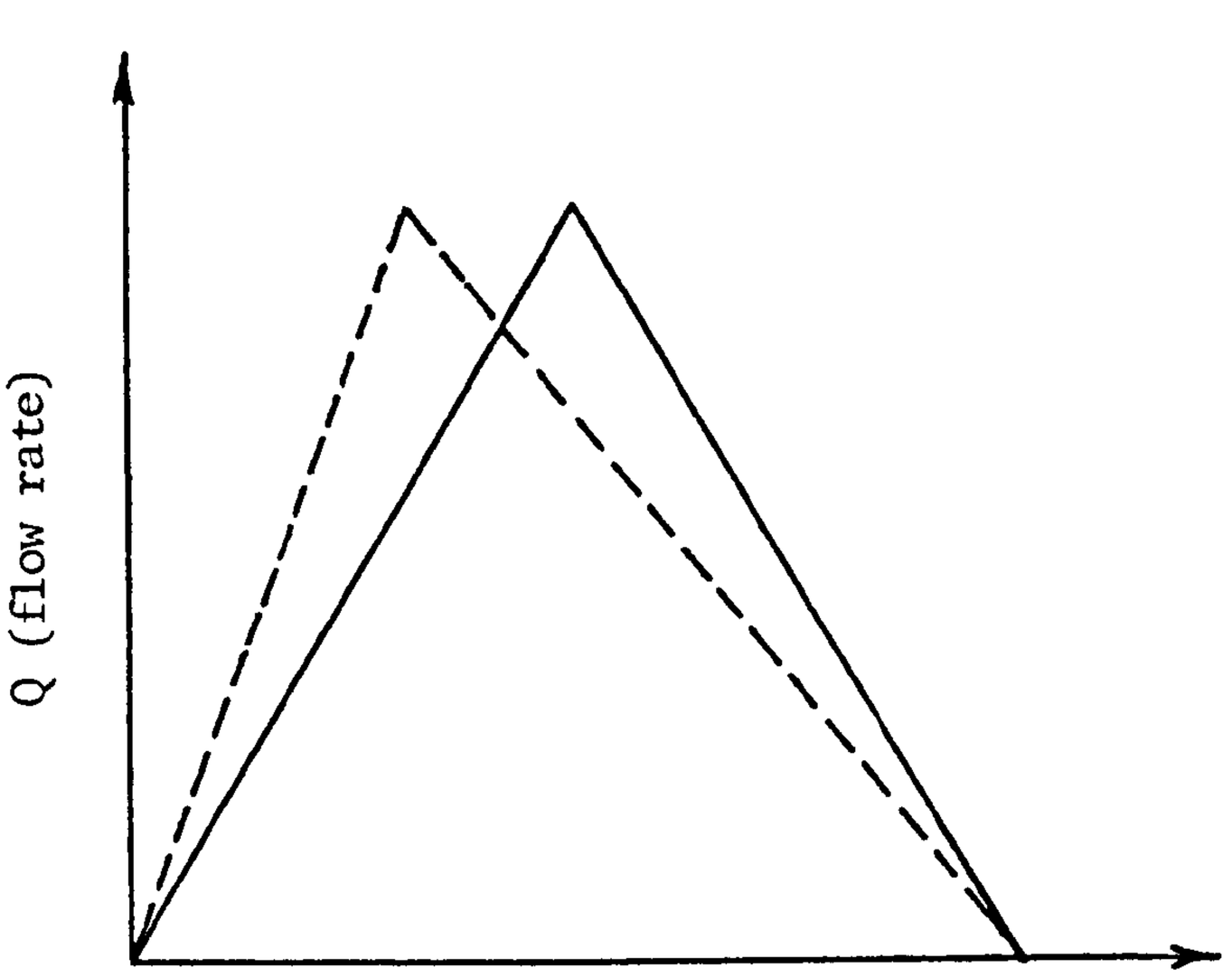


FIGURE 8.5 : 'Triangular' Hydrograph

Volume of reservoir 114,292,000 ft^3 (3.238 x $10^6 m^3$)

$\therefore$ Time to empty = 1102 sec. (18 mins)

It is known that the reservoir at Dale Dyke drained over a period in excess of three quarters of an hour.

While it is appreciated that the simplified approach used here is not entirely appropriate, the times to empty obtained would seem to suggest that the equation for peak discharge obtained from the graph in Figure 8.3 cannot be applied to relatively small capacity upland earth bank reservoirs, as in the Pennine examples quoted in Table 8.3.

8.3 Breach Dimensions

Data from some documented incidents and failures have enabled information on the dimensions of breaches to be recorded and analysed. The information is shown in Figure 8.1 and in Table 8.1.

The wide spread of results reflects the complexity of the problem, and the resulting breach configuration's dependence on such factors as reservoir capacity, slope protection, nature of the embankment fill etc.

The only general relationship obtainable from publicised breach dimensions is:-

$$w < 6d$$

and

$$w < 90$$

If, however, the width of the resulting breach, (w metres), at crest, is plotted against the height of the dam before failure (i.e. depth of breach, d), then the results shown in Figure 8.2 and Table 8.2 are obtained.

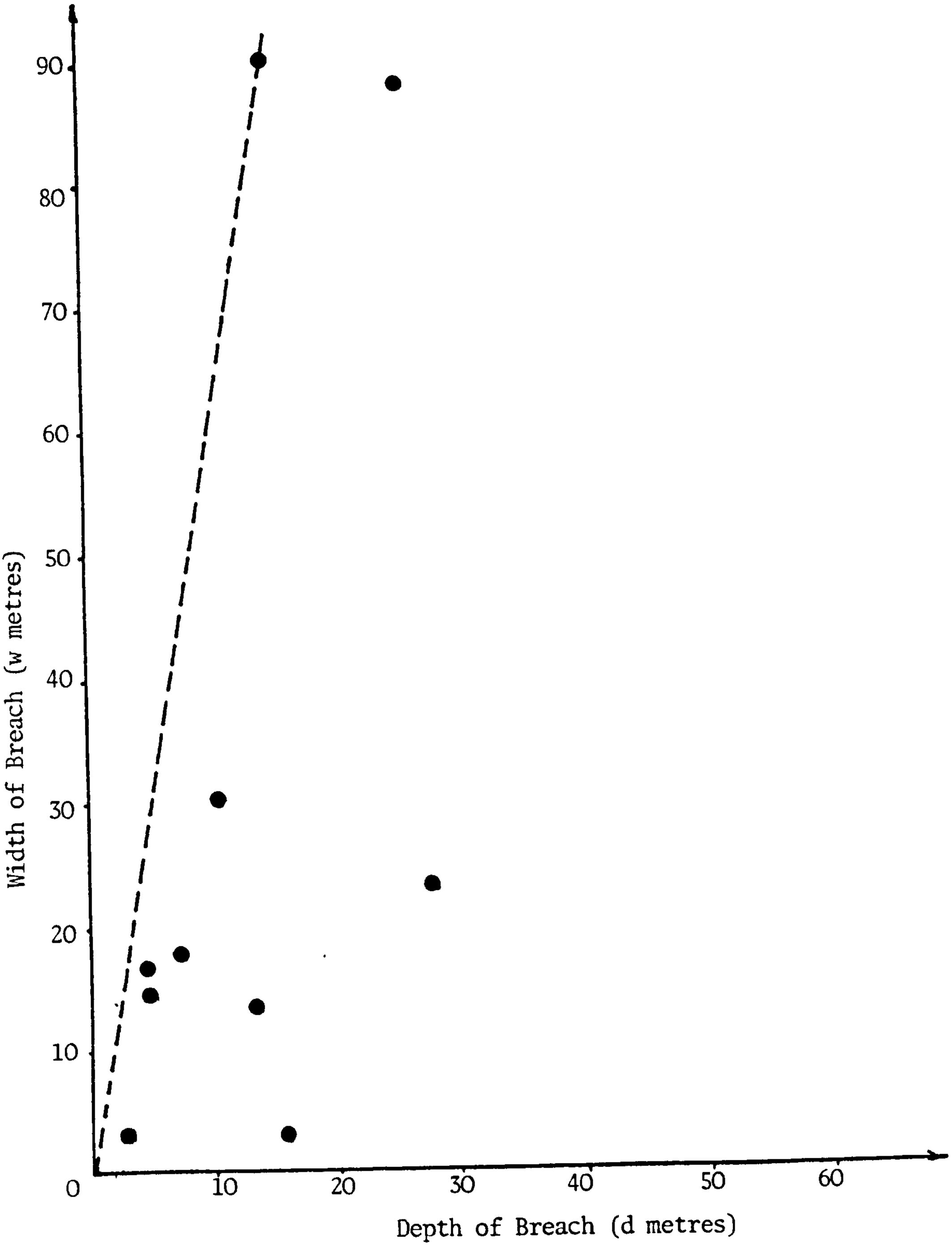


FIGURE 8.1 : Graph of Depth of Breach v. Width of Breach

Name of Dam	Width of Breach W (metres)	Depth of Breach D (metres)	$\frac{W}{D}$
Baldwin Hills	23	27	0.95
Brainerd	84	23	3.6
Breakneck	30	7	4.3
Coedty	18	11	1.7
Kaddam	137	41	3.3
Kepple Cove	24	16	1.5
Lurgashall	22	5	4.4
McMahon	6	17	0.4
South Fork	128	22	5.8
Confidential	12	4	3.0
Confidential	3	2	1.5

TABLE 8.1 : Reported Dimensions of Breaks in Dams which have been Overtopped

Name of Dam	Width of Breach, W (metres)	Height of Dam, H (metres)	$\frac{W}{H}$
Baldwin Hills	23	47	0.49
Brainerd	84	23	3.6
Breakneck	30	7	4.3
Coedty	18	11	1.7
Kaddam	137	41	3.3
kepple Cove	24	16	1.5
Lurgashall	22	5	4.4
McMahon	6	17	0.4
South Fork	128	23	5.8
Confidential 1	12	3	4.0
Confidential 2	3	2	1.5

TABLE 8.2 : Reported Dimensions of Breaks in Dams which have been Overtopped

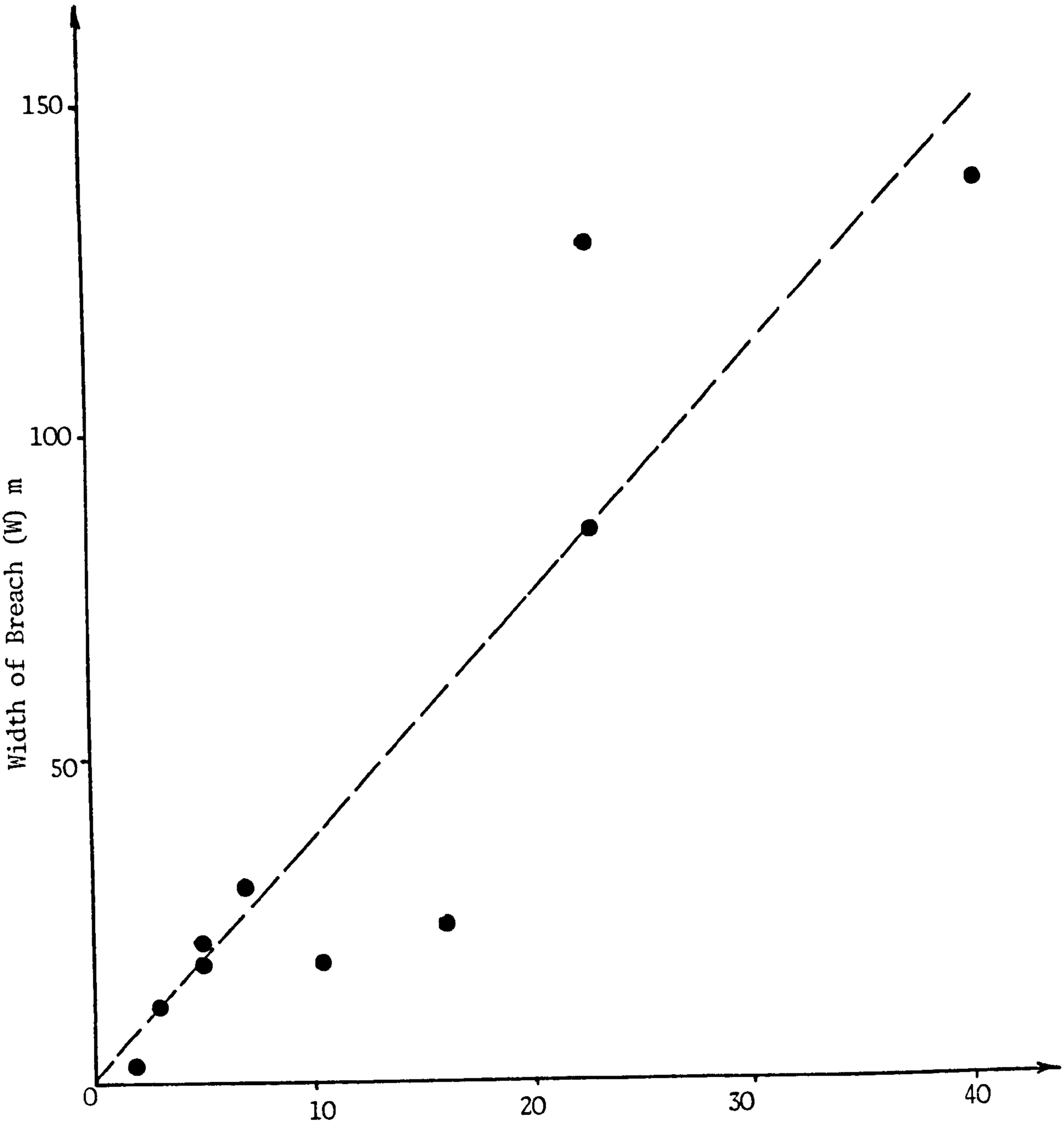


FIGURE 8.2 : Graph of Height of Dam v. Width of Breach

Some scatter of points is to be expected due to differences in embankment materials and reporting techniques/errors, but from the limited information available there appears to be a broadly linear relationship between the height of a dam before failure and the width of the breach resulting from overtopping. This relationship is shown below:-

$$W = 3.625H$$

where W = Width of Breach in metres

H = Height of Dam in metres

8.4 Erosion Retarding Layer

Fread and Harbaugh[217] introduced a conceptual method of alleviating downstream damage from a breached earth dam by providing a relatively thin erosion retarding layer at an optimum elevation within the embankment. The resulting breach would not develop continuously, but rather be delayed and thus produce two distinct flood waves of reduced amplitude, compared with the single larger wave from the corresponding breached dam without retarding layer.

While Fread and Harbaugh gave no details of practical aspects of the construction of such a layer they did formulate expressions for the optimal position of the retarding layer with respect to certain geometric, hydraulic and dynamic parameters, including the failure rate of the dam, i.e. the rate of formation of the V-shaped breach, and the ratio of the failure rate of the earthfill dam to the failure rate of the retarding layer. The protection layer proposed in Chapter 7 by the Author would obviously also serve as such an erosion retarding layer.

8.5 Discharge Coefficients for Breaches

Sandover[218] carried out experiments to find the discharge coefficients for a constriction in an open channel caused by end-tipping of loose fill. The sides of the constriction were assumed to have slopes of 2 : 1 and to remain stable. Sandover found that the discharge

$$Q = C_d Q_r T$$

where Q_r is rate of flow in the channel,

C_d is a coefficient of discharge,

Q is the rate of flow or the upstream Froude number, and

T is time.

C_d was seen to depend on:

1) The Contraction ratio, m

with $m = \frac{1 - b}{B}$,

where b is the gap width measured on the centre line of the dam at half water depth,

B is the river or reservoir width.

2) The geometry of the abutments of the constriction.

3) The distance from the gap centre line to the river or dam centre line and the angle the dam makes with the direction of flow and the ratio $\frac{d}{b}$ where d is the mean particle diameter of the tipped material.

While the results quoted by Sandover are not directly applicable to the case of a breached dam it can be seen that as the particle size

of the material within the embankment decreases, the value of C_D decreases and approaches 0.5.

8.6 Erosion Rates for Failure of Earthfill Dams

In 1965, Cristofano[226,227] developed an equation to compute the erosion rate at a breach in a dam. This equation was based on the solution of a differential equation resulting from a simple analysis of force vectors. The force of water flowing through his idealised breach was equated to the shear stress acting on a shear plane normal to the plane of the dam, i.e. the base of the breach.

The breach was assumed to trapezoidal with side slopes approaching the angle of repose of the compacted fill material, and therefore the broad crested weir formula could be used without any refinement in computation due to end contractions. An expression for the resistive shear strength was derived by equating shear force to the product of the plane area upon which this force would act and the average shear stress. Average shear stresses were equated to the developed cohesion plus the product of the intergranular pressure and tangent of the developed angle of friction. The length of the shear plane is a function of the developed angle of friction of the fill material. Substitution led to an expression for rate of change of vertical and lateral erosion within the breach with respect to the rate of change of water flowing through the breach in the form of a differential equation.

This equation was found to be too cumbersome for manual computations and so several simplifying assumptions were introduced:

(i) the width of the breach was assumed to be constant and the breach to remain trapezoidal in shape.

(ii) the length of the base of the breach was measured only to bedrock and it increased uniformly to the point at which bedrock inhibited erosion.

(iii) material transported by the water due to sloughing of the sides was ignored.

(iv) Slope of the base of the breach or shear plane was considered to be constant and equal to the developed angle of internal friction of the fill.

The final expression obtained is shown below

$$\frac{Qsoil}{Qwater} = Ke^{-x} \quad .$$

where $x = \frac{b\tan\emptyset}{H}$

where k = constant of proportionality,

Qsoil = the volume of soil carried away in each time period,

Qwater = the volume of water carried away in each time period,

e = the base of the natural logarithm

$\emptyset$ = the developed angle of friction of the soil material being considered,

b = the base length of the bottom of the overflow channel (in the direction of flow) at any given time,

H = hydraulic head at any given time.

The application of Cristofano's equation requires a trial and error solution with time increments of a few seconds.

This equation has been used by the Bureau of Reclamation and the

Tennessee Valley Authority to estimate the maximum flood resulting from a breached dam, and it may be useful for a 'first' estimate of peak flow but, in view of the complexity of the erosion process, as shown in this experimental programme, it is felt that the results from an analysis using this equation should be treated with caution. It must be appreciated, however, that assumptions must be made in any analysis of this kind in order that the resulting equation may be sufficiently manageable.

8.7 Design and Layout of Earth Emergency Spillways

The Soil Conservation Service of the U.S. Department of Agriculture have published a guide for the design and layout of earth emergency spillways for catchments in excess of 25 km^2 [228].

The emergency spillway will remain stable during passage of a flood if the resistance to erosion equals or is greater than the 'attack'.

The Soil Conservation Service defined 'attack' as the total volume discharged through the spillway during the passage of a flood divided by the bottom width of the spillway.

If O_e = total volume of onflow through spillway (acre.ft)

b = base width of spillway (ft)

then attack = $\frac{Oe}{b}$

The 'attack' can obviously be reduced by providing additional storage between normal T.W.L. and the crest of the emergency spillway, which would also reduce the frequency of operation of the spillway and hence reduce maintenance and repair costs, or alternatively by increasing the width of the spillway.

The resistance to erosion is measured by the resistance of the

soil to erosion or by the amount of soil that must be eroded to produce a breach. Soils were broadly divided into two categories:-

Easily Graded Soils (E):- Cohesionless soils

Soils with a PI less than 10

Recent alluviums, colluvial soils and organic soils.

Soils with a V.R. > 0.7

Dispersed soils and soils with a high shrink/swell potential.

while Erosion Resistant Soils (R) included:-

Cohesive soils with a PI between 10 and 40 and a V.R. < 0.7

Obviously the design of any spillway must be based on the soil in the spillway or beneath the spillway the geometrical parameters being as defined in Figure 8.6

The following empirical formulas were developed to express the relationship between resistance and attach:

If PI = plasticity index in percent

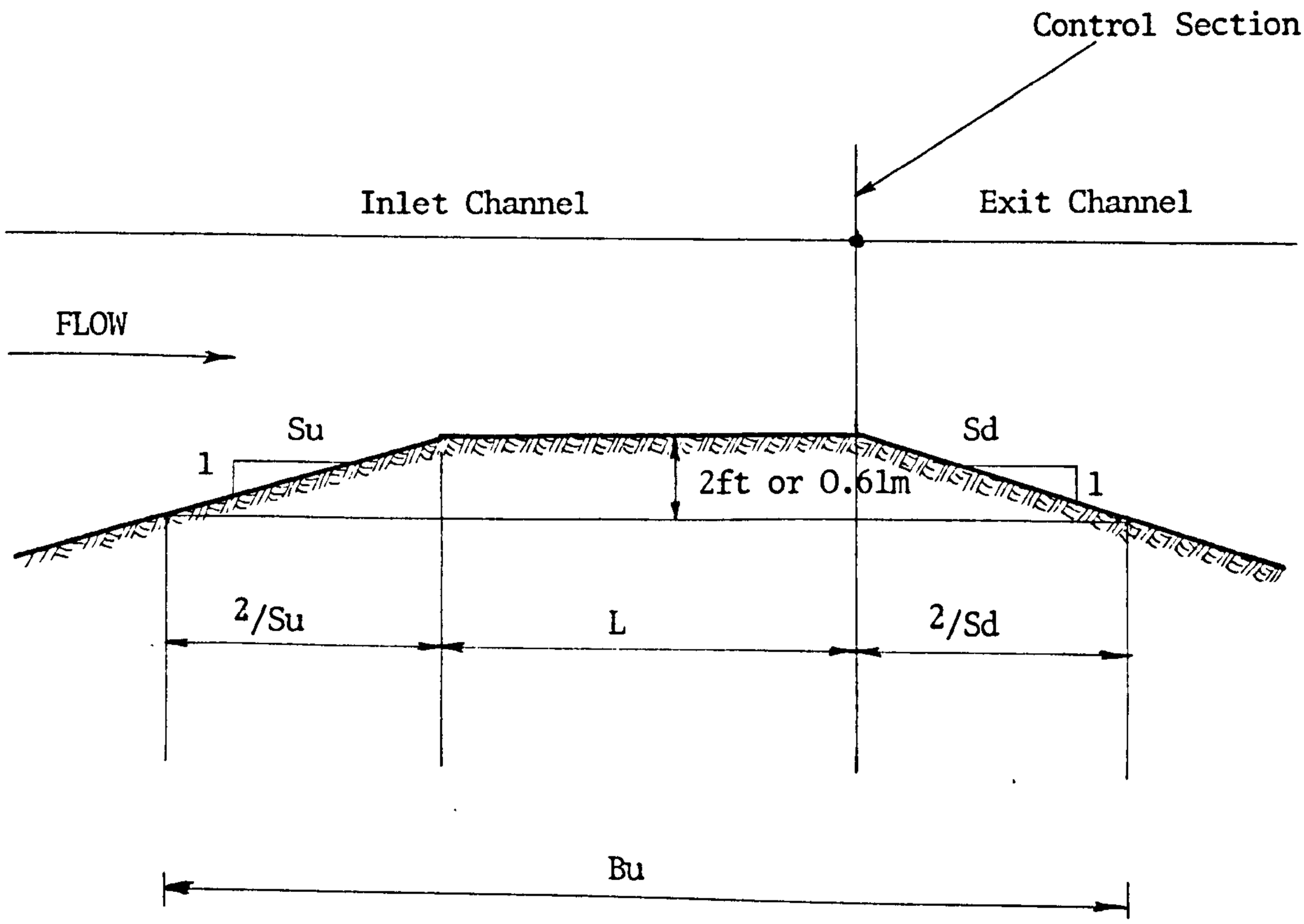
Bu = longitudinal bulk length

<u>For Erosion Resistant Soils (R)</u>

$$\frac{Oe}{b} \leqslant \left[PI + \left(\frac{Bu - 300}{20} \right) \right]$$

where the following limits are not exceeded.

$$\frac{Oe}{b} \leqslant 50 \text{ acre.ft. per ft.}$$



$$Bu = L + {}^{2}/Su + {}^{2}/Sd$$

L = length of level section of spillway in ft.

Su = slope of upstream part in ft./ft.

Sd = slope of downstream part in ft./ft.

FIGURE 8.6 : Layout of Earth Emergency Spillways (after [228])

$$10 < PI < 40$$

$$B_u > 100 \text{ ft.}$$

for Easily Eroded Soils (E)

$$\frac{O_e}{b} < \left[\frac{B_u - 100}{50} \right]$$

when the following limits are not exceeded:

For $5 < PI < 10$

$$\frac{O_e}{b} < 20 \text{ acre ft.per ft.}$$

$$B_u > 300 \text{ ft.}$$

For $PI < 5$

$$\frac{O_e}{b} < 10 \text{ acre ft.per ft.}$$

$$B_u > 300 \text{ ft}$$

Layout

The Soil Conservation Service also makes recommendations for the layout of earth emergency spillways which are not reproduced in this section.

In order to obtain a relatively simple expression the number of variables affecting erosion were kept to a minimum, in this case the only variable defining soil type being the Plasticity Index (PI).

It has already been stated that soils with 'high' Plasticity Indices may reasonably be expected to be eroded less than soils with low Plasticity Indices but that a situation can arise whereby two soils

with the same Plasticity Index can erode at very different rates depending on their structure. It can hence be seen that, once again, application of the equations quoted earlier must be circumspect.

8.8 Conclusions

Several methods of predicting peak flows from a dam failure have been suggested but, as has been shown, application of these methods must be cautious, as each failure is unique in some respect and the failure hydrograph can be dependent on so many variables.

Data from documented incidents and accidents suggests that there is a fairly well defined relationship between the width of a breach and the height of a dam, while it has been shown that equations recommended for the design of earth emergency spillways should be used with caution.

CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS

9.1 Introduction

The following chapter details the principal conclusions drawn from the investigation and the preceding chapters, and also makes certain recommendations for further research into the erosion of earthfill embankments.

9.2 Conclusions and Recommendations

9.2.1 General

The protection of earthfill dams against overtopping forms one of the primary considerations in the planning design and construction of these structures. To obviate failure, design has been focussed on hydrology and the prediction of the worst floods reasonably to be envisaged, coupled with the provision of spillway and/or outlet works of sufficient capacity, but this has not always been sufficient to prevent failure due primarily to the inaccuracies of the flood prediction methods employed during design and as shown by the recent failure of Machhu II dam in India, and the failures listed in Appendix A[219,220].

Evidence has been presented to suggest that compacted clay fills are considerably more resistant to erosion by overtopping than is traditionally accepted to be the case. Considered in the context of the implications of the N.E.R.C. Flood Studies Report (1975) and

related documents, appreciable resistance to erosion under exceptional flood conditions could be significant in relation to many embankment dams.

9.2.2 Documentation

The research carried out to obtain information on failures has shown that more complete documentation of construction procedures, engineering details and operational history is required.

Engineering details should include engineering drawings, historical details of remedial works, leakages, spillway and bywash capacities and where applicable the capacity of any other flood diversion works, reservoir volume and details of outlet works and freeboard allowances together with the details of the engineering properties of the embankment materials.

Old records are useful and all data of overtopping and/or failures do provide some aid in giving an understanding of the physical condition of an embankment, but it must be recognised that the deterioration of fill materials with time in in general little understood and documented.

9.2.3 Documentation of Failures

The failure of any dam should, of course, be fully investigated in an attempt to ascertain the reason or reasons for failure as should be known instances of overtopping. The results of such an investigation, as well as data on the incident, amount of erosion etc., should be published in the form of an engineering report available to interested parties. This report should ideally contain information on rainfall, runoff, breach details, soil parameters etc. etc.

Very little information is available on such incidents/failures as have occurred, and what information is available contains little or

no data on soil type. This is, in a sense, hardly surprising since, especially for the earliest recorded incidents/failures the influence of soil type was not appreciated and also no approach was widely available for defining quantitatively the various parameters involved.

9.3 Safety Implications

9.3.1 Embankment Behaviour

The behaviour of the embankment after heavy rainfall, and after rapid drawdown should be monitored perhaps by the use of instruments, e.g. piezometers etc.

9.3.2 Communication Network

The procedures for obtaining specialist assistance, including helicopters, radios, teletypes, telephones,pumps, and all emergency services in the event of an emergency should be known by the reservoir operational staff.

9.3.3 Interruption of Construction

If construction work is interrupted on a reservoir site provision must be made to safely pass floods which might both damage the works and release any temporary storage the partially completed works might be retaining.

As already stated, during the construction of a reservoir in the Glencorse Valley which was to cover 17 ha., having a maximum depth of 17.6 m, a capacity of 1673 t.cm., an embankment with a puddle clay core "considerable excitement and alarm was caused by the accumulated waters of a flood which nearly overtopped, and was likely to wash away, the half constructed embankment"; "but provision was made to prevent this by temporarily spreading tarpaulins or other materials over the soft

material of the embankment[221] illustrating the need to protect partially constructed embankments.

9.3.4 Access

It must be ensured that proper access is possible, both to the dam site and to control valves, and spillway gates even when the embankment is being overtopped.

9.3.5 Crest

9.3.5.1 Crest Widths

The crest width should be of such width that, in the event of an emergency, fill material can be placed easily and quickly on the crest. Five metres of loose fill were placed in 3 days on the crest of the Oros Dam in Brazil[110]. A breach did not occur until some 0.75 m of water was flowing over this loose fill.

Sandbags were placed on the crest at Danson Park Lake to avert a failure.

Ideally the crest should be sloped at 1 in 20 down towards the reservoir (see Figure 9.1). This assists drainage towards the reservoir, increases freeboard slightly and reduces the velocity of approach of overtopping waters.

9.3.6 'Small' Dams

It is considered that 'smal' dams, i.e. a capacity of less than 25000 m^3 should be included in safety legislation. It has been shown that failures have occurred to 'small' dams and, from personal experience during the summer of 1979 (late July, early August) when over 40 mm of rain fell in 2 days with higher than average rainfall during the rest of the week, several Pennine mill lodges gave serious cause for concern. In the North West as in other parts of the industrial

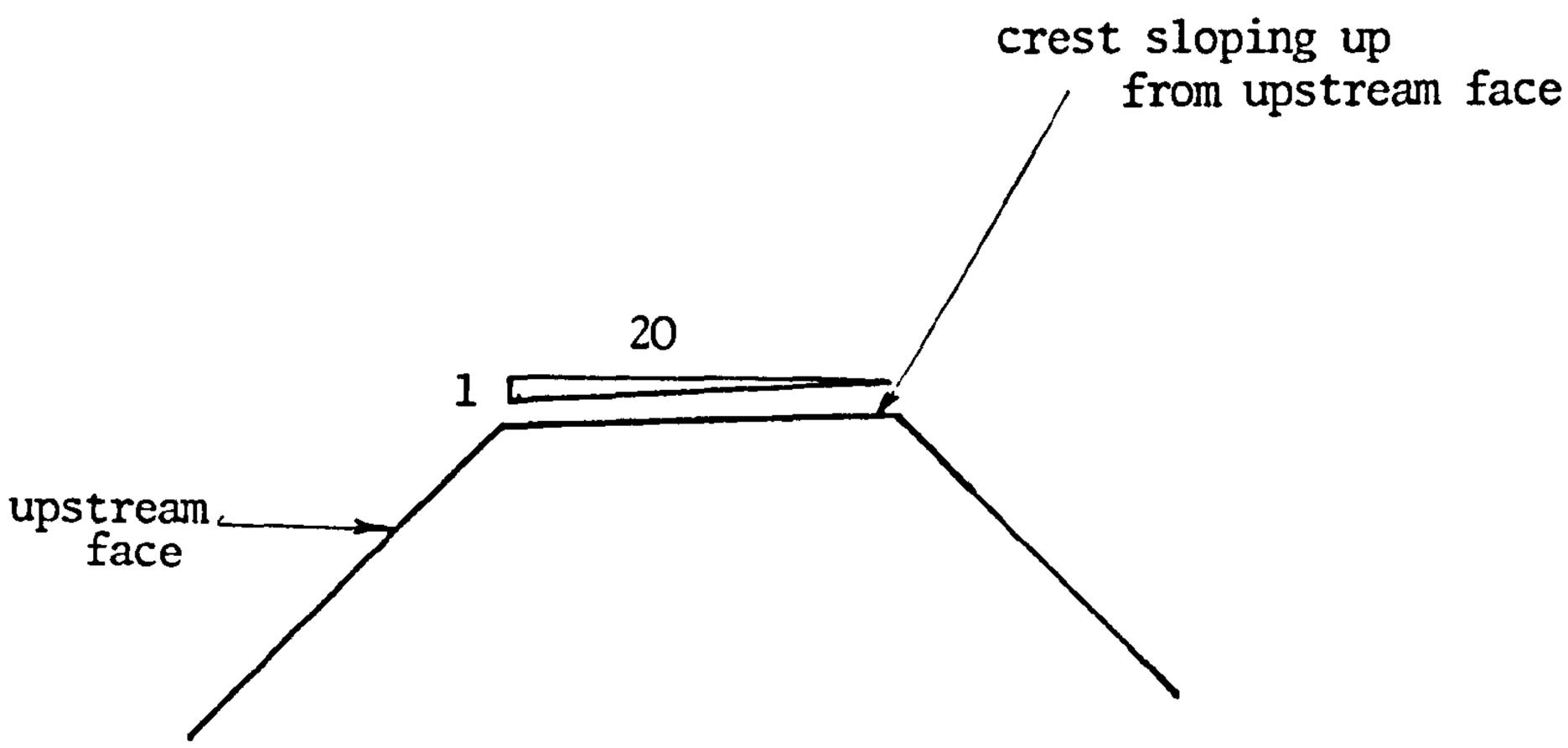


FIGURE 9.1 : Section through Crest

north, there exist large numbers of such mill lodges, some of which are one hundred years old and built to supply individual mills, are 'small' in terms of storage capacity, but are often built behind houses constructed for mill workers. They usually have minimal freeboard and often do not have spillways. These lodges can, by virtue of their proximity to the local inhabitants, be as hazardous as a large dam.

9.3.7 Control Equipment

Control equipment, especially spillway gates, should be chosen with care, and "back-up" operating methods always provided and maintained fully operational. This fact was emphasised with the recent failure of Machhu II dam in Western India where three out of 18 radial spillway gates failed to operate. The coils in the electric motors failed and manual efforts failed to force the gates open. The dam was overtopped and failed with very heavy loss of life although in this case it may be argued that failure would have occurred whether the gates had failed or not since the inflow to Machhu II, some 14000 cumecs, was more than twice the design flow of 6000 cumecs[220].

9.3.8 Vigilance

A programme of inspection and maintenance must be provided for all reservoirs which might threaten life. It is disturbing to read "of sixteen dams brought to the attention of the U.S. Corps of Engineers by local citizens as showing signs of trouble, the immediate draining of the reservoir was ordered in eight cases"[222].

9.3.9 Fuse Plugs

The use of fuse plugs should be encouraged; these fuse plugs can be of the 'Oxbow' type, employing an erodible subsidiary embankment, or automatic tilting gate type auxiliary spillways (similar to that used by the North West Water Authority at Greenfield Reservoir), and are

comparatively easy to install on existing dams.

9.3.10 Surface features

The formation of tracks due to sheep or people on the downstream shoulder of an embankment should be prevented. All surfaces should be ideally accurately and smoothly finished with no discontinuities which might cause a concentration of flow in the event of overtopping.

9.3.10.1 Berm Drains

Berm Drains on the downstream face of an embankment can capture local runoff and also low flows overtopping the embankment as well as providing access tracks for grass cutting and general maintenance, and increased resistance to erosion by virtue of the increased thickness of the downstream shoulder (Figure 9.2) and the resulting reduction in velocity although this is offset by increased turbulence.

9.3.11 Waves

Waves likely to overtop an existing wave wall may be damped out by a boom anchored to the upstream face.

9.3.12 Wavewall

If a wave wall is being relied on as part of the flood protection works at a reservoir, i.e. the wave wall is expected to retain part of the 'static' flood, then the wall must be continuous and be structurally sound. If a wave wall collapses, especially if only part of the wall fails, then flow will be concentrated in one particular area leading to accelerated local erosion under high head.

9.3.13 Rip-rap Protection to the Downstream Face

Tests carried out in 1975[186] suggest that stone laid on the downstream face of a dam will move down the face and build up at the

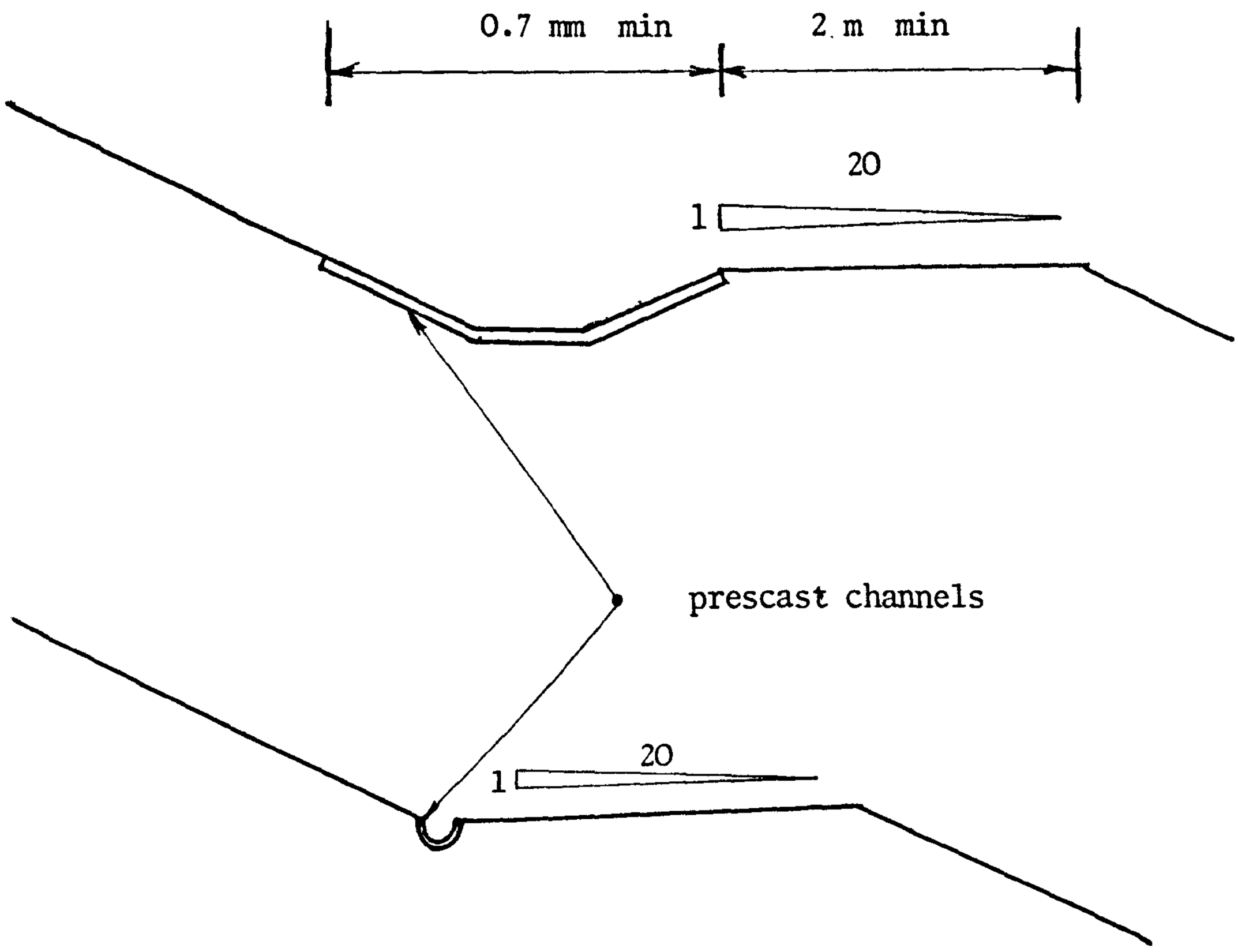


FIGURE 9.2 : Berm Drains

toe decreasing the amount of erosion at the toe but allowing erosion of the upper section. Grouted rip-rap will clearly afford greater protection to the downstream face.

Rip-rap on the upstream face would obviously afford greater protection to the upstream face and, in the tests of reference was seen to reduce 'lateral' erosion, as discussed in Chapter 6. Rip-rap was seen to be removed by water which flowed its way between the stones eroding the underlying fill leading to the removal of individual or small groups of stones into the flow.

9.3.14 Erosion of the Different Zones within a Dam

The different zones within a dam, i.e. core, filters, shoulders etc., will erode at different rates as shown in Plate 3.2 The core material is probably the most resistant to erosion but the thickness of the core will depend on economic factors as well as construction method, type of material, the design of filter layers and tolerable seepage loss.

9.3.15 Warning Systems

Care must be taken in the design and supervision of a flood warning system. The recent failure of Machhu II in Ihdia showed that a warning system can often be ineffective. As the water levels at Machhu rose efforts were made to alert those living downstream of the dam with loudspeakers. Few could be informed in time, and many who were informed ignored the warning.

9.3.16 Industrial Lagoons

Lagoons which store industrial wastes should, it is considered, be subject to safety legislation. This is justifiable on grounds of location and by reference also to the failures cited in Chapter 3.

9.3.17 Maintenance/Sabotage/Vandalism

Adequate maintenance must be provided at all times to such items as gates, valves etc. and attempts made to prevent sabotage and vandalism. Maintenance should include restoration of freeboard due to settlement, measures to prevent landslides within the reservoir basin, and measures to prevent icing of gates, etc.

9.3.18 Grass

More research is required in relation to choice of the type of grass best suited for embankment slopes, and also into the type of management that should be provided, i.e. whether the grass should be kept short or allowed to become long. Grass can cause heavy water losses in the top layer due to plant transpiration, leading to a reduction in moisture content and thus to the possibility of fissuring, both of which would tend to result in higher rates of erosion. In addition, the moisture content of unprotected surface layers of fill, when subject to heavy run-off, may increase by as much as 5 percent with an associated reduction in shear strength.

Long grass under overtopping conditions can bend and protect the surface, but it may also result in greater moisture loss and a less healthy sward than short grass. Long grass also makes visual inspection of the face of a dam for seepage and settlement etc., more difficult.

Irrespective of length of grass it is obviously important to 'key' the grass layer to any topsoil layer and thence into the shoulder of the embankment. Artificial reinforcing materials such as plastic netting pegged to the face have been used, and hydroseeded grass protected by sprayed fibreglass strands and cold bitumen have also been used in isolated instances[223]

9.4 Erosion of Clay

9.4.1 General

In the eighteenth and nineteenth centuries, when the foundations of modern mathematics and mechanics were established, and significant developments in other areas of civil engineering took place, little progress occurred in earth dam construction or in the proper understanding of the behavious of such dams. With the advent of the twentieth century, important developments took place. In the period 1910 - 1940 they included:-

(1) Recognition of the principle of soil compaction.

(2) Manufacture of larger mechanical and more efficient earth moving and compacting equipment.

(3) An understanding of the principles relating to shear strength and compaction of soils.

(4) The development of analytical tools for estimating shear forces, slope and foundation stability, and for predicting requirements as to spillway capacity.

Since that period research has proved to be the key to a better understanding of the way in which materials and structures behave.

9.4.2 Material Research

The differences between cohesive and non-cohesive soils from an erosion standpoint are:-

(1) Resisting forces are predominantly electrochemical in nature in cohesive soils.

(2) Electrochemical forces are not necessarily constant and may vary with salinity, soil disturbance etc.

The experimental investigation has shown that:-

(1) Some clay structures are composed of small clusters called flocs the size of which depend on such factors

as salinity, turbidity etc. rather than on individual particles.

(2) The bed material tested was, in most cases, eroded through by individual particles or flocs, although in some tests a degree of 'mass' erosion also took place.

(3) Eroded material was observed to include material in suspension, material which rolled, slid and/or bounced.

(4) In general bonds between clay particles within a soil cannot be expected to be uniform in strength. They will vary in magnitude from values close to zero, to some value x, with an average $\bar{x}$ and it is clear that the minimum rather than the average value of interparticle cohesive bond will govern erosion.

(5) Under test the bed material was eroded fairly evenly, the surface being covered with a regular pattern of ripples.

(6) If small stones existed within the bed material 'pock' marks were formed on the downstream side of these stones. Stones were also seen to be whirled around in 'pockets' gradually eroding a deeper hole.

(7) The concentration of suspended solids (for concentrations up to 1 gm/litre) had no significant effect on the rate of erosion.

(8) Temperature was an independent variable during this investigation but it is known that temperature is inversely linked to the capacity of a stream to carry suspended sediment

(9) For the clay used in these tests at a particular moisture content and density, initiation of erosion occurred when

the velocity of flow reached a value of approximately 1.75 m/sec.

(10) The first facet in the erosion process may be attributed to swelling. Swelling is a surface phenomenon which occurs as soon as the eroding fluid encounters the sample. The amount of swelling is affected by the moisture content of the sample, the percentage of clay in the sample and lesser factors. In general swelling occurs as the soil tends to achieve a state of equilibrium within the soil water system, and that in doing so, a reduction in the interparticle bonding occurs. Hence it can be concluded that, for the soils tested :

(i) swelling results in an increase in void ratio and a reduction in interparticle bonding

(ii) a soil with a 'low' moisture content swells more than a soil with a 'high' moisture content

(iii) flocculated soils swell more than dispersed soils since a soil compacted dry of optimum absorbs more water and swells more than a soil of the same density compacted wet of optimum

(iv) an increase in the percentage of clay in a soil sample will result in an increase in the soil's susceptibility to swelling and in general the greater the amount of clay in a sample the greater the overall effect of surface behaviour becomes.

(11) Erosion rates are moisture content and density dependent.

(12) For the soils tested, an increase in moisture content occurs as the eroding fluid passes across the sample. This

increase, for a particular bulk density, is sensibly constant.

(13) The chemistry of the eroding fluid was kept an independent variable, but it is thought that the chemistry of the eroding fluid could have a considerable effect on the erosion rate of a soil.

(14) The manner in which an embankment is breached by overtopping was found to follow a set sequence of events.

(i) a rounding of the crest occurs as the overtopping water passes across the crest and flows down the face of the dam.

(ii) step features as described in Chapter Six, are formed on the downstream face

(iii) the uppermost step recedes through the body of the dam until the pressure of water on the upstream face causes failure of the remaining material and the formation of a definite breach through the dam.

(iv) if the compacted fill from which the embankment is composed has some apparent cohesion, then the sides of the breach will remain close to the vertical as the breach deepens and widens.

(v) if a clay core exists within the dam then the downstream shoulder may be eroded more quickly than that core, leaving the core acting as a cantilever

(vi) any discontinuity on the face of the embankment (including berms) will cause increased turbulence and erosion.

(vii) Filter zones within the downstream shoulder will erode more quickly than the compacted fill.

9.5 Recommendations for Further Research

Although a considerable amount of information now exists on the process and problem of erosion there are certain areas which would particularly benefit from further investigation. Information gathered in research of this type can be used as a guide but no investigations can comprehensively explore all the complex problems associated with the erosion process. Tests are also very time consuming. Research conclusions must, if they are to be of value, refrain from the use of the obscure adjectives such as, 'reasonable' and 'sufficient' which have been used in the past to express the time to which a clay can be subjected to overtopping. They must also avoid arbitrarily defining erosion by some purely visual parameter which makes comparison with the work of other researchers extremely difficult or invalid.

Recommendations for further work are listed below:-

(i) The investigation should be extended to a carefully selected series of clays with different properties. Particular attention should be paid to clays with shear strengths and densities higher than the ones used in this study. Consideration should also be given to lower grade materials such as those used in industrial lagoons.

(ii) An investigation to study the effect of the chemistry of the eroding waters should be carried out. The chemistry of water may be especially important when it is appreciated that the characteristics of water even from adjoining

catchments can be very different. Waters flowing through peat will, for example, prove very different to water flowing through limestone. Industrial wastes, as often retained in lagoons, should also be included in this study.

(iii) The effect of environmental and micro-climatic changes on clay should be studied in some detail. The surface layers of the downstream shoulder of an embankment dam can, for example, be subjected to repeated cycles of freezing and thawing and/or wetting and drying. Cyclic variations may affect the erosion process perhaps by causing cracking or a change in chemical composition etc. base exchange which might fundamentally affect the character, structure and physical properties of the surface layers.

(iv) The effects of both intensity and frequency of the turbulence, and local changes in turbulence, on the process of erosion and the rate of erosion should be studied.

(v) The tests used to determine initiation of erosion in Section 6.7 should be repeated to see whether the velocity at which erosion commences is affected by moisture content and density.

(vi) Trials should be conducted to investigate the type of grass best suited to embankments, and the type of vegetation management which should be provided should also be studied.

(vii) A full scale test on, for example, an old embankment

would be of great value and enable the erosion process to be verified at the prototype scale.

Such an opportunity to carry out a full scale test occurred in 1974:-

Winscar dam, a rockfill structure being constructed for the Yorkshire Water Authority, lay immediately downstream of the old earthfill Dunford Bridge dam, an embankment dating from 1858. The Dunford Bridge embankment was known to have slipped during construction, and as a result the embankment was completed to an irregular plan. In addition, it is known that the embankment was built on made ground, and although appearing to average some 6 or 7 m high it is thought that the embankment could have had a maximum structure height of up to some 23 m with a length of some 210 m.

The reservoir capacity was calculated as being in the region of 520 t.c.m.

Comprehensive plans for a controlled breaching equipment on Dunford Bridge were prepared and put forward by the Department of Civil Engineering, University of Newcastle upon Tyne, following consultation with the Panel Engineer responsible for Winscar Dam under the Reservoirs (Safety Provisions) Act (1930). The plan provided for Dunford Bridge to breach and discharge into a 'cushion' of water retained by the almost completed and much higher Winscar embankment.

Approval in principle was given by the Yorkshire Water Authority, but the urgency with which Winscar was required and the fear of a period of excessive discolouration and turbidity due to the breaching process of Dunford Bridge led to the project being abandoned.

Details of the proposed test including details of flow rates likely to have been obtained are given in Appendix G.

(viii) Tests to study the effect of permeability and infiltration rates should be conducted. This investigation has shown erosion to be dependent on moisture content and hence change, and rate of change, in moisture content will affect the change and rate of change in erosion rate.

(ix) The rate of formation of the uppermost 'step' should be studied in detail in order that the rate of erosion of both the vertical face and the flat, together with the rate of recession of the face, can be determined.

(x) Having established the information in (ix) work should be carried out on the development of a mathematical model to demonstrate, perhaps on a visual display basis, the progressive formation of a major breach.

The hydraulic parameters required for the model can be obtained from the N.E.R.C. Flood Studies Report - namely inflow, duration, lag, outflow, etc. enabling an estimate of the volue of water likely to overtop the embankment and the height to which the static flood would rise above the crest to be obtained.

Having established these parameters the width of the breach likely to form for a particular dam could be calculated from the equation quoted in Chapter Eight. Once the width and geometry of the breach has been determined, and having assumed a nominal initial depth of breach, or low area on the crest of the order of, say, 0.3 m, an initial estimate of the velocity of flow through the notch can be made. The information gathered

from the investigation suggested in (ix) would enable the movement of the uppermost 'step' and the formation of the other 'steps' on the face to be modelled. Knowledge of the tensile strength of the embankment fill would then allow prediction of the point when water pressure against the upstream face would cause collapse of the remaining material and create a clear channel through the embankment.

From this point a recalculation of discharge through the channel and the velocity of flow would prove necessary. An increase of pressure head would obviously result, unless the reservoir storage was very small, and hence an increase in velocity of flow is very likely. Knowledge of the soil parameters of the compacted fill would then enable the subsequent erosion rate to be established to evaluate and model the rate at which the channel cuts down through the embankment.

Allowances would have to be made in any computer program to allow for progressive change in the length and geometry of the base of the channel as the deep breach forms through the embankment, and also for consequential change in reservoir level and hence in velocity and erosion rate.

Development and refinement of the basic program would permit allowance being made for the influence of such features as berms, filter zones and puddle clay cores.

(xi) In general, erosion increased as the slope of the downstream face became steeper.

The minimum amount of material eroded for a particular density can be predicted by the following equations.

For $\gamma_B = 1840 - 1850 \text{ kg/m}^3$

$$E = 7.6 \times 10^{-5} \gamma_B \text{ A.t.v.}$$

For $\gamma_B = 1940 - 1950 \text{ kg/m}^3$

$$E = 5.4 \times 10^{-5} \gamma_B \text{ A.t.v.}$$

SECTION III

APPENDIX A

KNOWN INSTANCES OF DAMS WHICH HAVE BEEN OVERTOPPED

The following computer file listing gives basic information on incidents/accidents/failures assembled by the Author.

Notes

(1) Use computer print-out pages to check whether the dam has an alternative name. If so consult all relevant entries.

(2) Entries are in alphabetical order.

(3) All dimensions to the nearest metre.

(4) Use of these print-outs should be supplemented by information on individual data sheets and files held by the Author.

KNOWN INSTANCES OF DAMS WHICH HAVE BEEN OVERTOPPED.

DEAD	COUNTRY	NAME	DATE	BUILT	HT	LT	TYPE	CAPACITY M3	INCIDENT TYPE
	IRAQ	ABU HABDA	605-562BC	600BC			M		F
	IRAQ	ADHEIM	/ /1258	2500BC	15M	175M	M		F?
	AMERICA	AFTERBAY	/ /1967	1965	5M	171M	R	NA	U/C-F
	AMERICA	AGUACALIENTE	/ /1973				E		F?
	TURKEY	AHMETOGLAN	/ /1967	1967	5M		E		F
	AMERICA	AKERMAN UPPER	/ /		5M		E		F
	AMERICA	AKERMAN LOWER	/ /		15M		E		F
20	GHANA	AKOSOMBO	28/07/1963	1963	141M	640M	R	NA	U/C-F
21	SPAIN	ALCORA	/ /1792						F
22	AMERICA	ALCYON N.J.	18/06/1896				E	37000000	F
23	GREAT BRITAIN	ALDENHAM	/ /1933	1795	8M	400M		78000	I
24	TURKEY	ALIBEY	12/05/1974	1974	39M	352M	E	NA	U/C-F
25	SPAIN	ALICANTE	13/11/1697	1594	41M	80M	M	4072200	F
26	100 ITALY	ALLA SELLA ZERBINO	13/08/1935	1923	12M	80M	CG	1727000	F
27	100 ITALY	ALLESSANDRIA	13/08/1935	1923	12M	80M	A	1727000	F
28	SPAIN	AL YOZARA							F
29	AMERICA	ALTMAN	19/05/1901						F
30	2 CANADA	ALTON	13/11/1889		2M		E		F
31	AMERICA	ALYCON N.J.	18/06/1896				E		F
32	AMERICA	ANACONDA,MONT.	28/07/1938	1898	22M		E-C	318220	F
33	AMERICA	ANDERSEN	28/12/1902	1897	13M	129M	M	288756	F
34	MADAGASCAR	ANTANIFOTSY	/ /1958	1958	4M		E		F?
35	AMERICA	AQUACALIENTE	/ /1973				E		F?
36	GREAT BRITAIN	ARI	02/01/1933	1933	20M	189M	C	22730	U/C-I
37	AMERICA	ARIZONA CANAL	/ /1905		10M	305M	T		F
38	BRAZIL	ARMANDO	/ /1958	1958	32M	660M	E		U/C-F
39	BRAZIL	ARMANDO	20/01/1977	1958	32M	660M	E	254000000	F
40	GREAT BRITAIN	ASHBURNHAM 1	/ /	1762	6M		E		I
41	GREAT BRITAIN	ASHBURNHAM 2	/ /	1762					I
42	GREAT BRITAIN	ASHBURNHAM 3	/ /	1762	8M	260M	E		I
43	GREAT BRITAIN	ASHFORD RES.	10/07/1968	1934	6M	230M	E	50005	I
44	JAPAN	ASHIZAVA	17/07/1956	1912	16M		E		F
45	AMERICA	ASHLAND	13/12/1901	1900	12M		E	987200	F
46	IRAQ	ATRUSH		700BC			M		F
47	GREAT BRITAIN	AUCHENDORES RES.	14/01/1968	1880	10M	348M	E	857000	I
48	8 AMERICA	AUSTIN,PENN.	13/07/1942		10M	174M	T	295490	F
49	AMERICA	AUSTIN,TEXAS	07/04/1900	1893	18M	388M	M		F
50	AMERICA	AVALON	03/08/1893	1893	13M	320M	E+R	4516440	F
51	AMERICA	AVALON	01/10/1904	1897	15M	420M	E/R	4516440	F
52	AMERICA	AVOCA	25/05/1894	1893	18M		E/R	1364	F
53	145 USSR	BABII YAR GORGE	/ /1961		10M		E		F
54	1000 AFGHANISTAN	BADJJA	/ /1971				L		F
55	1100 MEXICO		03/10/1976		9M		E		F
56	7 AMERICA	BAKER CITY	15/06/1865	1860	15M		E	1000120	F
57	5 AMERICA	BALDWIN HILLS,CAL.	14/12/1963	1951	47M	195M	E	1106898	F
58	GREAT BRITAIN	BALLOCHDOWAN	04/11/1843	1843		137M	E		F
59	AMERICA	BALSAM	03/05/1929	1894	18M	91M	E-C	2500000	F
60	GREAT BRITAIN	LAITHAYOCK FISHING	06/07/1916		5M		E	102285	F

A/1

61		AMERICA	BARLETT			COFFERDAM.		NA			U/C-F
62		AUSTRALIA	BARNEJACK	/ /1925	1913	73M	239M	CG	941000000		I
63		GREAT BRITAIN	BARROWFORD	/ /1976		9M	1000M	F	459145		I
64		CANADA	BATTLE RIVER	22/04/1956	1956	12M		E-C			F
65	200	AMERICA	BAYLESS	/ /1911	1909	14M	166M	M	909200		F
66	200	AMERICA	BAYLISS DAM	/ /1911	1909	14M	166M	M	909200		F
67		FRANCE	BAZACLE DAM	/ /1709	1171	5M	274M	E+R			F
68		GREAT BRITAIN	BEATH	15/03/1815	1796	13M	243M	E	262000		F
69	40	GREAT BRITAIN	BEATH	21/11/1835	1821	13M	243M	E	262000		F
70		AMERICA	BEAVER CREEK,GEORG	06/05/1931				E			F
71		GREAT BRITAIN	BEITH	15/03/1815	1796	13M	243M	E	262000		F
72	40	GREAT BRITAIN	BEITH	21/11/1835	1821	13M	243M	E	262000		F
73		GREAT BRITAIN	BESOM HILL	/10/1856	1856		73M	E	45450		F
74		AMERICA	BIG SANTA ANITA	/04/1926	1927	71M	187M	A	NA		U/C-F
75		AMERICA	BIG SANDY CREEK	/ /1965	1962	11M		E			#?
76		CZECH	BILA DESNA	/09/1916	1915	5M		E			F
77	81	GREAT BRITAIN	BILBERRY	05/02/1952	1938	20M	90M	E-C	390000		F
78		GREAT BRITAIN	BILBERRY	24/03/1941	1840	32M		E-C	390956		I
79		INDIA	BIRCH GANGA	25/08/1894	1893	246M	M	N	600000000		F
80		GREAT BRITAIN	BIRCHEN BRIDGE	14/09/1968		8M	60M	E	>22700		F
81		GREAT BRITAIN	BIRCHEN BRIDGE	/ /1974		8M	60M	E	>22700		F
82		GREAT BRITAIN	BIRSEMORE LOCH	23/04/1968	PRE1865	3M	425M	E	78000		F
83		AMERICA	BISHOP CREEK	05/09/1909	1908	11M	135M	E+M	141910		F
84		GREAT BRITAIN	BLACK MOSS	/ /1810	1800	6M	320M	E	85000		F
85		GREAT BRITAIN	BLACKBROOK 1	20/02/1799	1796	28M	160M	E	2300276		F
86		GREAT BRITAIN	BLACKBROOK 2	/ /1804	1801	28M	160M	E	2300276		D
87		GREAT BRITAIN	BLACKSYKE	21/09/1821							
88		GREAT BRITAIN	BLAEN-Y-CYM	/ /1912				E			F
89		GREAT BRITAIN	BLAENNELINDWR			M	73M	E			I
90	1	GREAT BRITAIN	BLAKLNEY BROCK	15/07/1975		5M		M	296000		F
91		AMERICA	BLANDIN	/ /1948	1902	9M		T			F
92		GREAT BRITAIN	BLEA BECK			7M	137M	E			F
93		GREAT BRITAIN	BLITHFIELD	/02/1962	1953	16M	856M	E	18123000		I
94		AMERICA	BLUE WATER	06/09/1909	1908	11M	135M	E+M	141910		F
95	25	PAKISTAN	BOLAN	06/09/1976	1961	19M	500M	E	88000000		F
96		AMERICA	BOLTON,CONN.	/ /1938				E	1044300		F
97		AMERICA	BOLTON,CONN.	08/06/1941	1940	10M		E	1044300		F
98		AMERICA	BONEY FALLS	/ /	1921	12M		M			F
99		AMERICA	BONNELL POND DAM	30/07/1905		9M	243M	E			F
100		AMERICA	BONNEY IRRIGATION	11/04/1903	1901	10M		E			F
101		AMERICA	BOSTWICKS POND,N.J.	03/08/1934				E			F
102		AMERICA	BOSTWICKS POND 2	03/08/1934				E			F
103		AMERICA	BOSTWICKS POND 3	03/08/1934				E			F
104		AMERICA	BOSTWICKS POND 4	03/08/1934				E			F
105		GREAT BRITAIN	BOTTONS	/07/1872	1877	20M	250M	E	NA		U/C-I
106		AMERICA	BOYDSTOWN	/ /1907	1896	9M	95M	E+T			F
107		AMERICA	BOYDSTOWN	/ /1903	1897			E+T			F
108	245	GREAT BRITAIN	BRADFIELD	11/03/1854	1859	29M	382M	E	3327660		F
109		AMERICA	BRADFORD 2	/ /1913	1913	15M	305M	E	730400		F
110		AMERICA	BRADSHAW	17/03/1936	1933	12M	610M	E			F
111		AMERICA	BRAINERD	12/02/1917	1888	23M		E-C	20114200		F
112		GREAT BRITAIN	BRAITHWAITE	02/11/1838							
113		AMERICA	BREAKNECK	29/05/1902	1887	7M	63M	E-C	54552		F
114		AMERICA	BRIDGEPORT	/08/1905	1855	12M	196M	M	1091040		F
115		SOUTH AFRICA	BRITS DRIFT	11/04/1967		15M		R			I
116	14	AUSTRALIA	BRISBANE	/ /1929		32M	107M	E+R	3409500		F
117		AMERICA	BROOKVILLE	/ /1912	1912	9M	110M	E-C			F
118		GREAT BRITAIN	BRUTON	/07/1917	1857				142312		F
119		AMERICA	BUCKHORN	/ /1923	1904	9M		E	24880000		F
120		GREAT BRITAIN	BUCKHORN RES.	01/11/1970	1900	23M	259M	E	988000		I

A/2

121		AMERICA	BUFFALO CREEK	12/08/1970	1973	32M	107M	E	NA	U/C-F
122	66	AMERICA	BUFFALO CREEK	26/02/1972	1966	15M		E+R	NA	U/C-F
123		SOUTH AFRICA	BUFFALO RIVER 1					R	NA	U/C-F
124		SOUTH AFRICA	BUFFALO RIVER 2					R	NA	U/C-F
125		SOUTH AFRICA	BUFFALO RIVER 3					R	NA	U/C-F
126		AMERICA	BULLY CREEK	05/02/1925	1913	38M	55M	E-C		F
127		AMERICA	BURLINGTON	27/06/1938	1893	6M	305M	F	340950	F
128		AUSTRALIA	BURRINJUCK	/ /1925	1913	73M	239M	CG	941000000	I
129		AMERICA	BUTTE	/ /1898	1895	18M	105M	T	NA	U/C-F
130		AMERICA	BYNUM, MONT.	08/06/1964	1926	11M		E	93000000	F
131		AMERICA	CALDERWOOD	/05/1930	1930	71M	279M	A	51000000	F
132		GREAT BRITAIN	CALLANDER	02/01/1933	1933	20M	189M	C	22730	U/C-F
133		AMERICA	CAMDEN	02/10/1902	1853	6M	259M	E	34095	F
134		GREAT BRITAIN	CAMLOCK	/02/1859	1801	8M				F
135		NATAL	CAMPERDOWN	/11/1943	1901			E	2273000	F
136	330	AMERICA	CANYON LAKE	/06/1972	1930	6M		E	9092	F
137		GREAT BRITAIN	CARTSBURN 1	/ /1815	1796	13M	243M	E	262000	F
138		GREAT BRITAIN	CARTSBURN 2	/ /1835	1821	13M	243M	E	262000	F
139	14	AUSTRALIA	CASCADE	04/04/1929	1926	19M		R	29000000	F
140		GREAT BRITAIN	CASTLE HOWARD	/ /1967	1798	6M	520M	E	45000	I
141	2	GREAT BRITAIN	CASTLE MALGWYN 1	10/08/1875						
142	2	GREAT BRITAIN	CASTLE MALGWYN 2	10/08/1875						
143	2	GREAT BRITAIN	CASTLE MALGWYN 3	10/08/1875						
144	2	AMERICA	CASTLEWOOD, COLO.	03/08/1933	1890	21M	183M	R	4319000	F
145		AMERICA	CAZADERO	29/01/1965	1906	21M	55M	T	16000000	F
146		AMERICA	STANLEY	/ /1927	1840	14M	185M	E	612000	I
147		AUSTRALIA	CETHANA	15/11/1968	1971	110M	213M	R		I
148		SYRIA	CHABHA	/ /1947	1944	10M		E		F
149		AMERICA	CHEAHA CK. NO. 6	19/03/1970	1970	23M	305M	E	NA	U/C-F
150		AMERICA	CHEESEMAN LAKE	/ /1900	1900	12M	183M	R	NA	U/C-F
151	10	ALGERIA	CHEURFAS	/ /1895	1892	40M	155M	M	160000000	F
152		GREAT BRITAIN	CHEW MAGNA.	/03/1989	1849	9M	96M	E-C	113650	I
153		GREAT BRITAIN	CHEW MAGNA.	10/07/1968	1849	10M	103M	E-C	113650	F
154		AMERICA	CITY DAM 1	/ /1909	1908	8M	45M	M		F
155		AMERICA	CITY DAM 2							F
156		AMERICA	CITY DAM 3							F
157		AMERICA	CITY DAM 4							F
158		AMERICA	CITY DAM 5							F
159		AMERICA	CLEAR FORK	/ /1894						I
160		RHODESIA	CLEVELAND	/ /1923	1913	11M		E		
161		AMERICA	CLINTON 1	23/07/1938		4M		M	136380	F
162		AMERICA	CLINTON 2	23/07/1938		6M		E-C	590980	F
163		AMERICA	CLOVER RUN	/ /1911		8M		T		F?
164		AMERICA	CIRCLEVILLE	16/05/1903	1903				NA	U/C-I
165		CANADA	COBDEN	13/10/1894		11M	76M	E		I
166		GREAT BRITAIN	COCKERMOUTH	02/11/1898	TEMPORARY DAM					
167		GREAT BRITAIN	COCKER COLBS RES.	24/11/1970					522800	I
168	16	GREAT BRITAIN	COEDTY	02/11/1925	1924	11M	244M	E-C	318220	
169		GREAT BRITAIN	COLDBS	/ /1878	1797	16M	304M	E	15450000	
170		AMERICA	CONNELLSVILLE	29/05/1902	1887	7M	63M	E-C	54552	
171		GREAT BRITAIN	COLLEGE 1	/ /	1850	6M	116M	E		I
172		AMERICA	COLOMBUS	28/12/1901	1901	12M	166M	M		F
173		GREAT BRITAIN	COTE LODGE	30/05/1944	1866	12M	73M	E-C		I
174		AMERICA	COTTON WOOD	/ /1947	1936	11M		E		F
175		AMERICA	COTTON WOOD	/ /1969	1965	11M		E		F
176		GREAT BRITAIN	COWLYD	31/12/1924	1921	14M	310M	E-C	10700000	I
177		AMERICA	CRANE VALLEY	/ /	1910	44M	549M	E+R	61700000	F?
178		CANADA	CREDIT RIVER	07/03/1910	1910	15M	213M	E-C	NA	U/C-F
179		CANADA	CREDIT RIVER	07/04/1912	1910	15M	213M	E-C		F
180		AMERICA	CROTON, NEW YORK	/ /1841	1842	15M	131M	E+M	NA	U/C-F

A/3

		INDIA	CONDUM TANK	PUBDJENT	1933	2.5M	375M	E	9254078		E
	12	GREAT BRITAIN	CYNCARN	19/07/1875	1793	12M		E	63215		E
		AMERICA	CASUERINA POINT,CAL.	/ /1963	1965			E			F
	245	GREAT BRITAIN	DALE DYKE	11/03/1963	1963	22M	382M	E	3227000		F
		AMERICA	DAM NO.3,MICH.	/ /1930	1916	8M		C+E			F?
		AMERICA	DAM NO.4,MICH	/ /1930	1921	12M		E			F
		AMERICA	DAM NO 4,PENN.	17/03/1930	1923	12M	510M	E			F
		GREAT BRITAIN	DANSON PARK	05/09/1903	1782	4M	162M	E	90000		I
		SPAIN	DARRO	/ /1928	1300	2M	11M	M			F
	13	GREAT BRITAIN	DARWEN	23/07/1849				E			F
		AMERICA	DEADMANS GULCH	09/06/1972	1924	17M		R	88500		I
		GREAT BRITAIN	DEANHEAD	/ /1941	1840	19M	320M	E-C	319220		I
		GREAT BRITAIN	DEBDON LAKE	13/08/1945		M	M	E	39704		I
		AMERICA	DEBRIS CARRIER 1	17/03/1907	1906	4M	373M	E			I
		SPAIN	DEL GASCO	/ /1799	1799	56M	56M	M	NA		U/C-F
	4000	AMERICA	DEL REY	15/03/1626		8M	500M	E			F
		AMERICA	DELLS	06/10/1911	1910	18M	87M	E-C	13056720		F
		GREAT BRITAIN	DENTON.	/ /1947	C1800	7M	1380M	E	278000		I
		AMERICA	DESERT	15/06/1933	1910	9M					F
		AMERICA	DETENTION M.I.	16/06/1956	1963	17M	110M	E	424000		F
		AMERICA	DIANDI	/ /1926	1924	15M	104M	E			F
		GREAT BRITAIN	DIGGLE	/ /1849	1830	12M	133M	E	55460		I
		AMERICA	DIXIE LAKE	/11/1940				E			
		IRAQ	DIYALA		500BC			M			F
		CZECH	DOLNA HODRUSA	/ /1959	1954	22M	318M	E	432000		I
		GREAT BRITAIN	DOVESTONE CLOUGH	29/05/1681							I
		GREAT BRITAIN	DURLEIGH	10/07/1963	1938	12M	433M	E	959206		I
		GREAT BRITAIN	EARLSWOOD	/ /1953	C1813	6M	40M	E	772820		I
		CANADA	EASTEND	/ /1952	1939	8M		E			F
		GREAT BRITAIN	EASTER DALTULLICH	15/08/1893							F
		GREAT BRITAIN	EDEN VALE IRONWORK	16/01/1912							F
		AMERICA	EDDY	06/08/1893	1890	14M	317M	R			F
		AUSTRALIA	EILDON(2DIG)			27M	993M	E/R			
	200	ALGERIA	EL HABRA 1	/ /1871	1871	34M	325M	M	17000000000		F
	400	ALGERIA	EL HABRA 2	/12/1881	1871	35M	330M	M	17000000000		F
		ALGERIA	EL HABRA 3	26/11/1927	1871	33M	340M	M	17000000000		F
		EGYPT	EL KAFARA	2778-2563BC		10M		M			F
		TURKEY	EL MALI	/ /1916	1893	20M	240M	E+M	1760000		
		SPAIN	ELCHE 1	/ /1793	1642	23M	66M	M	400000		F
		SPAIN	ELCHE 2	/ /1836	1642	23M	66M	M	400000		F
		AMERICA	ELK CITY,OKLA.	01/05/1936	1925	9M	615M	E-C	1136500		F
		AMERICA	ELK CITY,OKLA.	/05/1937	1925	9M	615M	E-C	1136500		F
		GREAT BRITAIN	EMBSAY MOOR	03/06/1939	1903		430M	E	NA		U/C-F
		AMERICA	ENGLEWOOD	/ /1921	1922	34M	1440M	F	NA		U/C-F
		AMERICA	ENGLISH	/05/1883		31M	100M	R			F
		CANADA	ERINDALE	07/03/1910	1910	15M	213M	E-C			F
		CANADA	ERINDALE	07/04/1912	1910	15M	213M	E-C			F
		AMERICA	ESCANABA 1,MICH.	25/06/1930	1907	9M		E	2468000		I
		AMERICA	ESCANABA 2,MICH.	25/06/1930	1910	5M		E	1172300		F
		TURKEY	ESMALI	/ /1916	1893	20M	240M	E+M	1760000		F
		SPAIN	ESTRECHO DE RIENTES	/04/1903	1739	46M		E			F
		BRAZIL	EUCLIDES	20/01/1977	1960	53M	243M	E	13000000		F
		AMERICA	FAIRMONT	/02/1822	1819	5M	366M	E			I
		GREAT BRITAIN	FALMOUTH		1950	6M	116M	L	331853		F
		AMERICA	FALL RIVER	/ /1909				T			F
		ALGERIA	FERGOUG	/ /1871	1871	34M	325M	M	17000000000		F
	200	ALGERIA	FERGOUG	/ /1881	1871	35M	330M	M	17000000000		F
	400	ALGERIA	FERGOUG	26/11/1927	1871	33M	340M	M	17000000000		F
		GREAT BRITAIN	FIELDS DAM	/ /1926				M+E			F
		AMERICA	FISHKILL 1	14/07/1907	1907	9M	76M	L	27275		F

A/4

241		AMERICA	FISHKILL 2	14/07/1897	1898	7M	67M	E	27276		F
242		AMERICA	FOLSOM	/01/1953	1955	COFFERDAM			NA		U/C-F
243		AMERICA	FOLSOM	/05/1953	1955	COFFERDAM			NA		U/C-F
244	12	AMERICA	FORT PITT	/ /1896	1895	3M	57M	E-C			F
245		AMERICA	FORT PITT	05/07/1903	1895	8M	58M	E-C			F
246		AMERICA	FORTE MEADE	09/06/1972	1924	17M		R	68500		I
247		CANADA	FOURTH LAKE	/ /1961	1960	22M		E			F
248		AMERICA	FRACKVILLE 1	13/12/1901				E			F
249		AMERICA	FRACKVILLE 2	13/12/1901				E			F
250		CANADA	FRASER VALLEY	/ /1948		4M		E			F
251		AMERICA	FRENCHMANS CREEK	/ /1952	1952	12M	884M	E	8650340		F
252		ARGENTINA	FRIAS DIQUE PARDO	06/01/1970	1939	14M		E-C			F
253		AMERICA	FRUITGROWERS	13/05/1937	1898	11M	277M	E	4195600		F
254		GREAT BRITAIN	FURNACE HOUSE POND	/ /1968				E			F
255		GREAT BRITAIN	FURNACE MILL COWDENBE	06/1898		6M	120M	F			F
256		AMERICA	GALLINAS 1	05/08/1957	1910	29M	64M	E+I	220000		F
257		AMERICA	GALLINAS 2								F
258		AMERICA	GALLINAS 3								F
259		AMERICA	GANTT	/ /1929	1925	16M	519M	E			F
260		AMERICA	GARRISON DAM	08/06/1964	1929	54M		A	129570000		I
261		AMERICA	GILBERT RUN 2	/ /1913	1913	15M	305M	E	758400		F
262	8	GREAT BRITAIN	GLANDERSTON	30/02/1948		M	M	E			F
263		GREAT BRITAIN	GLENMORDIAL	/06/1953	1880	SIDE BANK		E-C	63000		I
264		GREAT BRITAIN	GLENOGIL	10/03/1975	1907			E+CG	273000		I
265		GREAT BRITAIN	GLENOGLE	10/03/1975	1907			E+CG	273000		I
266		GREAT BRITAIN	GLEN CLACHAIG	/01/1811	1801	10M					F
267		GREAT BRITAIN	GLEN CLACHAIG	/02/1859	1801						F
268	7	AMERICA	GOODRICH CREEK,ORE.	15/06/1846	1850	15M		E	1000120		F
269		AMERICA	GOOSE CREEK	03/05/1900	1900	63M		R			F
270		AMERICA	GOOSE CREEK,SC.	14/07/1916	1903	7M	701M	E			F
271		INDIA	GOPALPURA	/ /1955	1955	14M	579M	E	2200000		F
272	10	ALGERIA	GRAIN CHEURFAS	/ /1985	1982	40M	155M	M	16500000		F
273		AMERICA	GRAND RAPIDS	02/07/1900	1874	8M	250M	E-C	27276		F
274		INDIA	GREAT ICE DAM	12/09/1928		44M	305M	ICE			F
275		GREAT BRITAIN	GREENFIELD	29/05/1881				E			
276		AMERICA	GREENWOOD,CONN.	1936	1936			T			F
277		AMERICA	GREEN PETER,ORE.	/ /1964	1965	COFFERDAM			NA		U/C-F
278	6	AMERICA	GROS VENTRE	18/05/1927	1925	56M		N	30000000		F
279		HUNGARY	GYONGYOSOROSZI MINE	/ /1965				E+R			F
280		AMERICA	HALF MOON BAY,CAL.	/1926		24M		E			F
281		GREAT BRITAIN	HANCH	/ /1960	1880	4M	259M	E	145472		I
282		AMERICA	HARLAN COUNTY	/ /1956	1952	31M	3600M	E	104390000		F
283		AMERICA	HARLAN COUNTY	/ /1958	1957	31M	3600M	E	104890000		F
284		AMERICA	HARLAN COUNTY	/ /1963	1963	31M	3600M	E	104390000		F
285		AMERICA	HARLEM	/ /1893		1M	107M	E			F
286		AMERICA	HARLEM 2	/ /1893				E			F
287		GREAT BRITAIN	HARRINGTON	30/11/1864	1864	6M	60M	E-C	NA		U/C-F
288		AMERICA	HARRISON	05/05/1931		3M	61M	E			F
289		AMERICA	HATCHTOWN	25/05/1901	1901	12M	238M	E	14808000		F
290		AMERICA	HATFIELD	06/10/1911	1908	7M	150M	E	12244382		F
291		AMERICA	HAUPT ESTATES 1	13/12/1901				E			F
292		AMERICA	HAUPT ESTATES 2	13/12/1901				E			F
293		AMERICA	HEBGEN	17/08/1959	1915	37M	219M	E	420000000		I
294		AMERICA	HEBRON	02/05/1942	1914	17M	1198M	E			F
295		JAPAN	HETSA-IKE POND	/ /1951	1940	20M		E			F
296		AMERICA	HELL HOLE DAM	21/12/1964	1964	61M	470M	R	NA		U/C-F
297		GREAT BRITAIN	HENBURY HALL 1	19/06/1872			72M	E			F
298		GREAT BRITAIN	HENBURY HALL 2	19/06/1872			45M	E			F
299		GREAT BRITAIN	HENBURY HALL 3	19/06/1872							F
300		AMERICA	HENRY	20/06/1935		19M	213M	E			F

301		SOUTH AFRICA	HLUHLUWE	/ /				A+E	NA	U/C-F
302		AMERICA	HOLDEN	/ /		5M		E		I
303	81	GREAT BRITAIN	HOLNEIRTH	05/02/1852	1838	20M	90M	E-C	390956	F
304		GREAT BRITAIN	HOLNEIRTH	/ /1944	1840	32M		E	390956	I
305		AMERICA	HORSE CREEK,COLO.	29/01/1914	1911	17M	1365M	E	3085000	F
306		AMERICA	HORSE CREEK 1	28/08/1935		12M	701M	E	2468000	F
307		AMERICA	HORSE CREEK 2	28/08/1935	1935	12M	701M	E	2408000	F
308		AMERICA	HORTON	18/06/1925	1924	10M	343M	E-C	1739	F
309	5000	PERU	HUARAS	/ /1943				L		F
310	3500	PERU	HUASCARAN	/ /1961				L		F
311		AMERICA	HYDRAULIC CO.	/08/1905	1855	12M	195M	M		F
312	200	KOREA	HYOKIRI	/07/1961		15M				
313		ALGERIA	IABIA	/ /1890	1876	25M				F
314		AMERICA	IRON RIVER 1	12/12/1923	1870	2M		T		F
315		AMERICA	IRON RIVER 2	12/12/1923	1921	5M		E	1209320	F
316	1200	JAPAN	IRUKA DAM	/ /1868	1633	29M	692M	E	15180000	F
317	1200	JAPAN	IRUKAIKA	/ /1868	1633	29M	692M	E	15180000	F
318		AMERICA	ITHACA	21/06/1905	1903	5M		E	3825400	F
319	17	POLAND	IVINY	/ /1967		17M		E		F
320	40	AMERICA	JEANETTE,PA.	05/07/1903	1893	5M	107M	E	300000	F
321		AMERICA	JEFFERSON COUNTY	09/07/1897				E		F
322		AMERICA	JOHNSTOWN	/ /1880	1852	21M	275M	E	22730000	F
323	2280	AMERICA	JOHNSTOWN	31/05/1889	1852	23M	283M	E	22730000	F
324		AMERICA	JOHNSTOWN MILL 1	09/07/1889				E		
325		AMERICA	JOHNSTOWN MILL 2	09/07/1889				E		
326		AMERICA	JOHNSTOWN MILL 3	09/07/1889				E		
327		INDIA	KADAM	31/08/1958	1957	41M	3777M	E		F
328		INDIA	KADAM	31/08/1958	1957	41M	3777M	E		F
329	1104	INDIA	KADAM	31/08/1958	1959	23M		E		F
330		CEYLON	KALAWEWA		1887	15M	5895M	E		F
331		JAPAN	KAMENO HARA-TAMEIKE	/ /1950	1942	8M		E		F?
332		JAPAN	KAMINSUE-TAMEIKE	/ /1964	1919	12M		E		F
333		USSR	KARACHUNOVSKAYA	/ /1933	1950	22M		R		U/C-F
334		USSR	KARACHUNOVSKAYA	/ /1934	1950	22M		R		U/C-F
335	4000	AMERICA	KARIKARI	15/03/1626		3M	500M	E		F
336		INDIA	KEDARNALA	/ /1964				E		F
337		GREAT BRITAIN	KENNEDYS MILL DAM	21/11/1835						F
338		AMERICA	KELLY	24/07/1912	1862	3M		T		F
339		AMERICA	KEENE N.H.	09/04/1893		5M		E		F
340		AMERICA	KENNEBEC	13/12/1928	1895	4M		M		F
341		GREAT BRITAIN	KENTMERE	/ /1848		BYWASH WASHED		OUT		F
342		GREAT BRITAIN	KENWOOD HOUSE 1	/08/1975	PRE1777	6M		E		I
343		GREAT BRITAIN	KENWOOD HOUSE 2	/08/1975	PRE1777	5M		E		I
344		GREAT BRITAIN	KENWOOD HOUSE 3	/08/1975	PRE1777	1M		E		I
345		GREAT BRITAIN	KENWOOD HOUSE 4	/08/1975	PRE1777	6M		E		I
346		GREAT BRITAIN	KENWOOD HOUSE 5	/08/1975	PRE1777	4M		E		I
347		GREAT BRITAIN	KEPPLE COVE	29/10/1927		16M	M	C		F
348		AMERICA	KETNER	/ /1911	1911	14M	182M	E-C	NA	U/C-F
349	100	INDIA	KHADAKVASLA	12/07/1961	1875	33M	1600M	R	30850000	I
350		INDIA	KHARAGPUR	/09/1961	1961	24M		E		F
351		GREAT BRITAIN	KILANTRINGAN LOCH	04/11/1843	1843			E	102285	F
352		GREAT BRITAIN	KILLAMARSH	/ /1968	C.1774	7M	240M	E	105000	I
353		GREAT BRITAIN	KILLINGTON	30/07/1836	1820	16M	220M	E	3482000	F
354		AMERICA	KILLINGWORTH 1,CON	23/07/1938		4M		M	135380	F
355		AMERICA	KILLINGWORTH 2,CON	23/07/1938		6M		E-C	590990	F
356		AMERICA	KITTANNING PT.	20/06/1894	1877	15M	178M	E	295490	I
357		AUSTRIA	KOEFLACH	/ /1965						F
358		JAPAN	KOGUE-TAMEIKA	/ /1959	1907	16M		E		F
359		JAPAN	KOMORO	/ /1928	1927	5M		C		F
360		INDIA	KUNDLI	/ /1925	1924	45M		CG	NA	U/C-F

361		JAPAN	KUSUI SHIMA	/ /1958		12M		E			F
362		GREAT BRITAIN	L. NAM BREAC BUIDHE	/ /1965	1801			P			F
363		MEXICO	LA JALPA	/ /1839	1650		1650M				F
364		AUSTRALIA	LAANECOORIE	/ /1909	1891	22M	399M	E	7771000		F
365		AMERICA	LAFARGE	/ /1928	1862	3M	18M	T			F
366	120	AMERICA	LAKE	/ /1893		16M		E+R			F
367		AMERICA	LAKE AVALON	03/08/1893	1893	13M	320M	E/R	45164400		F
368		AMERICA	LAKE BARCROFT	21/06/1972	1913	21M		G+E	3119000		F
369		AMERICA	LAKE DIXIE, TEXAS.	/1940				E			F
370		AMERICA	LAKE FRANCIS	21/10/1899	1899	15M	302M	E			F
371		AMERICA	LAKE GEORGE 1	19/08/1909	1907	6M	335M	E			F
372		AMERICA	LAKE GEORGE 2	11/07/1914	1907	6M	335M	E			F
373	1	AMERICA	LAKE HELMET	16/02/1927	1895	6M	83M	E	17276000		F
374	1	AMERICA	LAKE HEMET	16/02/1927	1895	6M	83M	E	17276000		F
375	5	AMERICA	LAKE LATONKA	/10/1966	1965	13M	700M	E			F
376		AMERICA	LAKE MALLOYA	22/04/1942	1914	15M		E	5000000		I
377		AMERICA	LAKE MALLOYA	/ /1955	1914	15M		E	5000000		F
378		AMERICA	LAKE MCDONALD	07/04/1900	1893	18M	333M	M			F
379		CHILE	LAKE PINIHUE	/1960	1960	29M		E			F
380		AMERICA	LAKE TARA	/11/1977				E			I
381		AMERICA	LAKE VERA	/ /1905	1880	15M		T			F
382		ITALY	LANGENFERNER			ICE	BARRIER.				F
383		AUSTRALIA	LAUGHING JACK	/ /	1957	17M	170M	E	24000000		F
384		AMERICA	LAY	/ /1936	1914	32M	489M	CG	154000		F
385		AMERICA	LEBANON, OHIO.	10/07/1882		9M		E			F
386	2	AMERICA	LEE, MASS.	/03/1968	1965	8M	145M	E-C			F
387		AMERICA	LEMONT	24/05/1903				E	109104		F
388		AMERICA	LEROUX CREEK	/07/1905		9M		E			F
389	5	AMERICA	LEWIS CREEK	30/09/1895		5M					F
390		AMERICA	LIDDERDALE 1	19/08/1909	1907	6M	335M	E			F
391		AMERICA	LIDDERDALE 2	11/07/1914	1907	6M	335M	E			F
392		GREAT BRITAIN	LIGHTWOOD BYWASH	31/07/1881		BYWASH		E			F
393		AMERICA	LIMA, MONT.	/05/1894	1902	12M	33M	E-C	154250000		I
394		GREAT BRITAIN	LLANHILLETH	16/01/1912							F
395		GREAT BRITAIN	LLYN BODGYNYDD BACH	/ /	1880	5M	M	E			F,
396		GREAT BRITAIN	LOCH AIRIDH NA LIC	27/08/1933		4M		E			
397		AMERICA	LOCH ALPINE	/ /1926	1926	8M		E			F
398		GREAT BRITAIN	LOCH DUNTELCHAIG	/ /1970	1930	1M	17M	C	6270000		I
399		GREAT BRITAIN	LOCH CLACHAIG	/02/1859							F
400		CANADA	LOG FALLS	/ /1923	1921	11M		E-C			F
401		GREAT BRITAIN	LONG DAM	03/06/1908							
402		AMERICA	LONG VALLEY CK.	/ /1892							
403		AMERICA	LOOKOUT SHOALS	/ /1916	1915	31M	832M	E	49320000		F
404		INDIA	LOW KHAJUR	/ /1949	1949	5M		CG			F
405		GREAT BRITAIN	LOWERCROFT 1	05/12/1852			27M	E-C	5228		F
406		GREAT BRITAIN	LOWERCROFT 2	05/12/1852			27M	E-C			F
407		GREAT BRITAIN	LOWER BLACK MOSS					E	295490		F
408		AMERICA	LOWER CANADA POND	11/03/1901	1816	5M	183M	E			F
409		AMERICA	LOWER HILL HOLE	/ /1964	1966	123M		R	NA		U/C-F
410		AMERICA	LOWER MEDICINE	/06/1964				E			F/
411		AMERICA	LOWER MONUMENTAL	/ /1965	1969	COFFERDAM			NA		U/C-F
412		GREAT BRITAIN	LOWER NEUADD	/ /18	1884		200M	E	195500		I
413	30	AMERICA	LOWER OTAY.	27/01/1916	1897	41M	172M	R-C			F
414		AMERICA	LOWER RESERVOIR	/ /1944	1887	8M	53M	M			F
415		AMERICA	LOWER SUCLICN	22/02/1903	1881	8M	33M	M			F
416		AMERICA	LOWER TALLASSEE	29/12/1902	1901	9M	396M	M			F
417		GREAT BRITAIN	LURGASHALL MILL	16/09/1968	PRE1650	5M	106M	E	92738		F
418	1000	INDIA	MACHERLA	30/09/1964				E			F
419		INDIA	MALDUCK MASUR	/ /	1844	33M	564M	E	1073580000		F
420		AMERICA	MAMMOTH, UTAH	24/06/1917	1916	21M		E-C	NA		U/C-F

A/7

421		AMERICA	MANCHESTER	/ /1922		9M	53M	M			E
422	4	ITALY	MANFREDI	/ /1973				E			F
423		AMERICA	MAQUOKETA	01/06/1927	1924	6M	137M	E-C	1488204		F
424		AUSTRALIA	MARIAM	/ /1954	1952	8M		CG			F?
425		YEMEN	MARIB	450657580	750BC	10M	580M	E			F
426		AMERICA	MARTIN DAVEY,TEX.	/11/1940				E	1752280		F
427		AMERICA	MAYFIELD	/ /1960	1963	75M		A	NA		U/C-F
428		AMERICA	MCCLOUD	/12/1964	1965	70M	201M	E-R	NA		U/C-F
429		AMERICA	MCMANON GULCH	/ /1926	1924	17M	137M	E	31000		F
430		AMERICA	MEADOW LAKE, ORE.	20/11/1962	1893	9M		E/R	740400		F
431		AMERICA	MELVILLE	15/06/1909	1907	11M	488M	E-C			F
432	7	AMERICA	MELZINGAH 1	14/07/1897	1897	9M	76M	E	36368		F
433	7	AMERICA	MELZINGAH 2	14/07/1897	1897	7M	67M	E	290944		F
434	40	AMERICA	MEMORIAL LAKE,OHIO.	/ /1969		12M		E/R			F
435		EYGPT	MEMPHIS	4000BC	3100BC						F
436		AMERICA	MERRILL	24/07/1912				C+E			F
437		AMERICA	MIDDLE,COLO.	/ /1909	1894	10M		T			F
438		AMERICA	MIDDLEFIELD MASS.	10/07/1874	1792	6M	152M	E			F
439		AMERICA	MIDDLEFIELD MASS.	21/04/1901	1874	6M	152M	E			F
440		AMERICA	MIDDLESEX PLANT 2	/ /1927	1896	14M		T			F
441		AMERICA	MIDDLETOWN CONN.	/04/1961	1850	11M	46M	E			F
442	140	AMERICA	MILL RIVER	16/05/1874	1862	13M	183M	E	2739480		F
443	165	AMERICA	MINE TAILINGS	26/02/1972		15M		E+R	681900		F
444		AMERICA	MISSION CREEK	18/06/1925	1924	10M	343M	E-C			F
445		GREAT BRITAIN	MOCHDRE	/06/1936				E	59000		F
446		AMERICA	MOHAWK 1,OHIO	/03/1913	1913	5M	122M	E			F
447		AMERICA	MOHAWK 2,OHIO	01/02/1915	1913	5M	122M	E			F
448		AMERICA	MOHEGAN PARK	06/03/1963	1853	6M		E+R	204570		F
449		ITALY	MOLARE	13/09/1935	1925	15M	146M	CG	18016400		F
450		CANADA	MOOSE JAW RIVER	/ /1916			91M	CG			I
451		AMERICA	MORENA	/ /	1912	51M	154M	R	80210000		T
452		AMERICA	MOUNT LAKE,MINN.	05/05/1938	1937			E			F
453		AMERICA	MOYIE RIVER	/ /1926	1926	17M	47M	A			F
454		AMERICA	MUD LAKE 1	12/12/1923	1870	2M		T			F
455		AMERICA	MUD LAKE 2	12/12/1923	1921	5M		F			F
456	24	JORDAN	NAGATALAN	/ /1963	100BC			E			F
457		SOUTH AFRICA	NAHOON	/ /1970				CG			F
458	100	INDIA	NANAKSAGAR	08/09/1967	1962	16M	3219M	E	209700000		F
459		GREAT BRITAIN	NANPANTAN	20/07/1875	1870	7M	4195M	E	131834		I
460		AMERICA	NASH BOG	22/05/1969	1914	10M	274M	E+T	6170000		F
461		GREAT BRITAIN	NEVER FEAR DAM	/ /1926							F
462		AMERICA	NEW CROTON	/10/1955	1905	90M	661M	CG	89000000		I
463		AMERICA	NEW DON PEDRO	26/01/1969	1971	178M	579M	E-R	NA		U/C-F
464		AMERICA	NIMBUS	/ /1953	1955	COFFERDAM			NA		U/C-F
465		GREAT BRITAIN	NORTHAMPTON CANAL	23/08/1965							F
466		AMERICA	NORTH SCITUATE	13/04/1926		2M		E	308500		F
467		AMERICA	NORTH VERNON	/ /1910	1909	3M		CG			F
468	12	AMERICA	NORWICH,CONN.	06/03/1963	1853	6M		E+R	204570		F
469		AMERICA	NORWICH, N.Y.LOWER	03/09/1905	1893	10M	91M	E	45460		F
470		AMERICA	NORWICH, N.Y.UPPER	03/09/1905	1892			E	90920		F
471		AMERICA	NR ACKERMAN MISS 1	/ /		5M		E			F
472		AMERICA	NR ACKERMAN MISS 2			15M		E			F
473		GREAT BRITAIN	NR. BRISTOL	C.1940	1700	10M		M			F
474		GREAT BRITAIN	NR. BRISTOL.	C.1960	1700	10M					F
475		GREAT BRITAIN	NR. BRISTOL.	C.1963	1700	10M		M			I+D
476		AMERICA	NR FRANKLIN VIR.	/09/1960							F
477		GREAT BRITAIN	NR.LOUGHBOROUGH	20/07/1875	1870			E	131834		I
478		GREAT BRITAIN	NR. PUMLUMON - BREACHED LEAD MINING DAM - NOTICED ON T.V. DOCUMENTARY								
479		AMERICA	NR TAMPA 1	/ /1960							F
480		AMERICA	NR TAMPA 2	/ /1960							F

A/8

481		RHODESIA	NYANDNDO	/ /1959	1959	10M		E			F?
482	40	AMERICA	OAKFORD PARK	05/07/1903	1895	6M	107M	E-C			F
483		AMERICA	OAKLAND	20/10/1896							F
484		AMERICA	OCHLOCKONEE RV.	28/12/1901	1871	21M	56M	E			F
485		JAPAN	OGAYARINDO-TAMEIKE	25/09/1963	1944	19M		C			F
486		AMERICA	OIL CITY	04/06/1892		3M	55M	E/R			F
487		GREAT BRITAIN	OLDHAM	29/05/1881				E			F?
488		AUSTRALIA	O'REILLY WEIR	/ /1950	1947	11M		CG			F?
489		SYRIA	ORONTES VALLEY	/1961				E			F?
490	1000	BRAZIL	OROS	25/03/1960	1960	26M	457M	E	987200000		F
491	6	SPAIN	ORTUELLA	/ /1954				E			F
492		NORWAY	OSFALLET	/ /1916				E			F
493		GREAT BRITAIN	OTTERHEAD LAKE	10/07/1968	1900	2M	76M	E-C	9092		F
494		YUGOSLAVIA	OVCAR BANJA	/ /1965	1952	27M	150M	CG+E	3000000		F
495	13	GREAT BRITAIN	OVER DARWEN	23/08/1849				C			F
496		AMERICA	OVERHOLSTER	/ /1923	1920	17M	250M	CG	18500000		F
497		AMERICA	OVERHOLSER SHAWNEE	/ /1963	1920	19M		E:C			F
498		AMERICA	OVOCA	/ /1914	1909	5M	53M	E-C			F
499		AMERICA	OWL CREEK,S.DAK.	/ /1912		35M	1981M	C+C			F
500		AMERICA	OXBOW	/ /1961	1961	47M	488M	E			F
501		CEYLON	PADAVIL	/ / 504BC		21M		F			F?
502		INDIA	PAGARA	/ /1923	1914	31M		M+E			F
503		INDIA	PAGARA	/ /1943	1927	31M		M+E			F
504	100	INDIA	PANCHET	12/07/1961	1957	52M	731M	C			F
505		AMERICA	PANGUITCH	16/05/1903	1903				NA		U/C-I
506		CEYLON	PARAKRAMA		500AD			E			F
507	33	ARGENTINA	PARDO DAM	06/01/1970	1939	14M	62M	E-C	4936000		F
508		AMERICA	PARIS	16/05/1939	1939	17M	226M	E+R	NA		U/C-F
509		GREAT BRITAIN	PEAK FOREST CANAL	/07/1973		60M	OF CANAL	E			F
510		AMERICA	PEAPACK BROOK,NY.	17/11/1927	1927	10M	110M	E-C	NA		U/C-F
511		ALGERIA	PERREGAUX	/ /1871	1871	34M	325M	M	17000000000		F
512	200	ALGERIA	PERREGAUX	/ /1881	1871	35M	330M	M	17000000000		F
513	400	ALGERIA	PERREGAUX	26/11/1927	1881	38M	340M	M	17000000000		F
514		GREAT BRITAIN	PICKUP BANK	/07/1974	1849			E	141000		I
515			PIERA	07/10/1976				E			F
516	350+	NATAL	PINETOWN	/ /				E			F
517		AMERICA	PINKSTON	23/05/1968	1939	21M		E	671296		F
518		GREAT BRITAIN	PIPINGFORD PARK	/ /1972	1696	5M	118M	E			F
519		AMERICA	PITTSFORD	/ /1947		12M		E-C			F?
520		AMERICA	PITTSTON	28/11/1901	1870	5M	300M	C	4936000		F
521		AMERICA	POCONO LAKE	/ /1955	1909	9M		E			F?
522		AMERICA	POINTA	/ /1929	1925	17M		E	515812000		F
523		AMERICA	POINT A	/ /1929	1925	17M		C	515812000		F
524		ITALY	PONTE ALTO	/ /1542	1537			T			F
525		AMERICA	PORTMAN SCHOALS	29/12/1901	1897	13M	129M	M	288756		F
526		AMERICA	PORTSVILLE	04/04/1903				E			F
527	4000	BOLIVIA	POTOSI	15/03/1626		9M	500M	E			F
528		AMERICA	POWER CANAL DIV.	/ /1916							F
529		AMERICA	PRAIRIE RIVER	/ /1912				E			F
530		AMERICA	PRATT FORK CK.	14/09/1938	1934	6M		E			F
531		AMERICA	PROSSER CREEK	3 /08/1929		11M	31M	CG			F
532		AMERICA	PROVIDENCE 1	11/03/1901	1816	5M	183M	E			F
533	1	AMERICA	PROVIDENCE 2	11/03/1901	1816	6M	183M	E			F
534		AMERICA	PUDDINGSTONE,CAL.	26/04/1926	1928	15M		M	NA		U/C-F
535	250	AMERICA	QUILCASA	/ /1963				N			F
536	13	GREAT BRITAIN	RADFIELD FOLD	23/09/1848				E			F
537		GREAT BRITAIN	RAINOW FORGE DAM	09/07/1926		2M	70M	E			F
538		AMERICA	RAINBOW LAKE	14/09/1938				C			F
539		AMERICA	RANDALLS POND 1	11/03/1901	1816	5M	183M	E			F
540		AMERICA	RANDALLS POND 2	11/03/1901	1816	5M	183M	E			F

541		AMERICA	RAPID CITY	09/06/1972				E			F
542		PAKISTAN	RASUL	/ /1929	1902	3M		R			F
543		AMERICA	READER GULCH	31/12/1901	1900	12M		E	987200		F
544		AMERICA	RED BLANKET CREEK	/ /1964	1932	9M		E			F
545		AMERICA	RED ROCK	15/07/1910	1910	32M	234M	E	16000000		F
546		CEYLON	RIDSEN		4944D			E			F
547		URUGUAY	RINCON DE BONETE	/ /1959	1946	37M	1170M	CG	6650000		F
548		URUGUAY	RIO NEGRO	/ /1959	1946	37M	1170M	CG	6650000		F
549		AMERICA	ROBERTS POND	/ /1922				E-C			F
550		GREAT BRITAIN	ROGERS POND	14/07/1875	1792	12M		E	80210		F
551		AMERICA	ROOSEVELT DIV.	28/01/1916	1906	4M	122M	R			F
552		SUDAN	ROSEIRES 1			COFFERDAM		R	NA		U/C-F
553		SUDAN	ROSEIRES 2			COFFERDAM		R	NA		U/C-F
554		GERMANY	ROT IN BADEN	/ /1969	1958	6M		E	500000		F
555		AMERICA	ROTHSCHILD	/10/1911				E	NA		U/C-F
556		GREAT BRITAIN	ROUND DAM	03/06/1908							F
557		AMERICA	ROXBOROUGH RES.	12/01/1895							F
558		AMERICA	ROXBURY	01/04/1903	1870	8M	14M	M+E			F
559		GREAT BRITAIN	RUNNEYMEDE	/ /1961	1887	12M	M	E			F
560		EGYPT	SADD-EL-KAFARA	2850 BC	2850BC	12M	106M	E-C	567640		F
561	10	ALGERIA	SAINT DENIS	/ /1865	1845	21M		M			F
562		AMERICA	SALT RIVER DIV.	28/01/1916	1906	4M	122M	R			F
563	4000	MEXICO	SAN ILDEFONSO	15/03/1626		8M	500M	E	431900		F
564	1	MEXICO	SANTA CATALINA	/10/1906		15M		M			F
565		AMERICA	SANTELLE	21/02/1914	1914	20M		E-C	155904		F
566		INDIA	SARDAR SAGAR	/ /	1961	16M		E	47500000		F
567		AMERICA	SANTELLE	21/02/1914	1914	20M		E-C	155904		F
568		AMERICA	SCHAEFFER	05/06/1921	1911	30M	335M	E+T	4753368		F
569		AMERICA	SCHENECTADY ,NY.	20/10/1916		10M	46M	E-C			F
570		AMERICA	SCHOFIELD	21/05/1923	1926	8M		E			F
571		AMERICA	SCHOFIELD.WIS.	24/07/1912		3M	107M	T			F
572		AMERICA	SCOFIELD.UTAH	/ /1925		21M		E-C			F
573		CANADA	SCOTT FALLS	/ /1923	1921	15M	610M	CG+E	30000000		F
574		GREAT BRITAIN	SCOTT FOUNDRY DAM	21/11/1835			83M	E			F
575		SYRIA	SEDJAR	/ /		6M		M			F?
576		AMERICA	SHEFIELD	/ /1925		40M		E			F
577	160	INDONESIA	SEMPUR	01/12/1967	1967	53M	228M	R	NA		U/C-F
578		AMERICA	SEPULTRIDA	21/02/1914	1914	20M		E-C	155904		F
579		AMERICA	SEPULVEDA CAL.	21/02/1914	1914	20M		E-C	155904		F
580		GREAT BRITAIN	SHAWS WATER CO.	21/11/1835	1831			E	545000		F
581		AMERICA	SHELTON LOWER	22/02/1903	1901	9M	18M	M			F
582		AMERICA	SHELTON MIDDLE	22/02/1903	1901		10M	M			F
583		GREAT BRITAIN	SHERBOURNE	/ /	1756	7M	150M	E	55000		I
584		AMERICA	SHERBURNE	03/09/1905	1892	10M	91M	E	90920		F
585		AMERICA	SHERBURNE LOWER	03/09/1905	1893	10M	91M	E-C	45460		F
586		AMERICA	SHORT CREEK.ARK.	16 05/1939	1939	17M	226M	E+R			F
587	10	ALGERIA	SIG	/ /1865	1845	21M		M			F-D
588		AMERICA	SIX MILE CREEK	21/06/1905	1903	5M		E	3825400		F
589	16	GREAT BRITAIN	SKELMORLIE	/04/1926	1878	5M		E	15911		F
590		GREAT BRITAIN	SKELPER UPPER DAM	28/01/1809							F
591		GREAT BRITAIN	SKILPER LOWER DAM	28/01/1809							F
592		GREAT BRITAIN	SLAUGHAM MILL POND			4M		E	81828		D
593		GREAT BRITAIN	SLIGNEAGH	/ /1957	C1925	4M	39M	E	220000		F
594		GREAT BRITAIN	SLIGNEAGH	/ /1961	C1925	4M	39M	E	220000		F
595		AMERICA	SOUTH FORK	/ /1880	1852	21M	275M	E	22730000		F
596	2280	AMERICA	SOUTH FORK	31/05/1839	1852	23M	283	E	22730000		F
597		AMERICA	SPARTANSBURG.PA.	04/06/1892		3M	55M	E/R			F
598	12	AMERICA	SPAULDING POND	06/03/1963	1853	6M		E+R	204570		F
599		GREAT BRITAIN	SPINDLE MILL	03/06/1908				F			F
600		GREAT BRITAIN	SPOTT LAKE	12/08/1943	1900	9M	18M	E			F

A/10

601		AMERICA	SOUYTEN DUYVIL 1	/ /1893		1M	107M	E		F
602		AMERICA	SOUYTEN DUYVIL 2	/ /1893		1M	107M	E		F
603		AMERICA	ST MAURICE RIVER	10/05/1939		COFFERDAM			NA	U/C-F
604		AMERICA	STANLEY	/ /1927	1840	14M	185M	E	612000	I
605	5	AMERICA	STAUNTON	30/09/1896		5M		E		F
606		GREAT BRITAIN	STAUNTON HARCLD	/ /	1781	7M	100M	E	6000	I
607		GREAT BRITAIN	STODSMIEL	12/08/1948	1911	10M	70M	CG	245000	I
608		GREAT BRITAIN	STOUR	/ /1917				E	141600	F?
609		ITALY	SUBIACO	1305		50M				F
610		AMERICA	SUTHERLAND	/ /1942	1940	18M		E		F
611		AMERICA	SWEETWATER	/ /1895				CG	22212000	I
612		AMERICA	SWEETWATER	17/01/1916	1898	34M	115M	CG	22212000	F
613		AMERICA	SWIFT.MONT.	18/06/1964	1914	58M	226M	R+E	37000000	F
614		GREAT BRITAIN	SWINESHAW	/02/1952	1865	11M		E-C	254000	I
615		ALGERIA	TABIA	/ /1890	1876	25M		E	NA	U/C-F-D
616		AMERICA	TALLASSEE LOWER	29/12/1901	1901	9M	396M	M		F
617		AMERICA	TALLASSEE UPPER	29/12/1901		19M	55M	R		F
618		AMERICA	TELLURIDE	/09/1909	1894	9M	183M	E+R		F
619		AMERICA	TELLURIDE CRIB	/09/1909	1894			T		F
620		FRANCE	TETL ROUSSE	/ /1892				L		F
621		GREAT BRITAIN	THORTERS	12/08/1948	1914	15M	20M	E	286000	I
622		AMERICA	THROTTLE	22/04/1942	1914	23M	320M	E	51000000	I
623		SPAIN	TIBI	13/11/1697	1594	41M	80M	M	4072200	F
624		AMERICA	TIFFIN 1,OHIO	/03/1913	1913	5M	122M	E		F
625		AMERICA	TIFFIN 2,OHIO	01/02/1915	1913	5M	122M	E		F
626		ISRAEL	TIMNA					E		F?
627		ALGERIA	TLELAT	/ /	1869	21M	99M	M	550000	F?
628		AMERICA	TOA VACA	09/10/1970	1971	65M		E-R	NA	U/C-F
629		GREAT BRITAIN	TODDBROOK	/ /1878	1838	21M	210M	E	1345616	I
630		CANADA	TORONTO	07/04/1912		11M	213M	E-C		I
631	16	SPAIN	TORRE LA VEGA	/ /1960				E		F
632		AMERICA	TOUCEY	29/07/1905		3M	30M	M		F
633		AMERICA	TROUT LAKE	/09/1909	1894	9M	183M	E+R		F
634		AMERICA	TROUT LAKE UPPER	/09/1909	1894			T		F
635		GREAT BRITAIN	TUMBLETON LAKE	22/07/1927	1879	10M	110M	E	50000	I
636		GREAT BRITAIN	TUMBLETON LAKE	13/08/1945	1879	10M	M	E	50000	I
637		GREAT BRITAIN	TUNNEL END	/ /1799	1798	9M	140M	E	13638	I
638		AMERICA	TURKEY CREEK	/ /1910	1910	18M	160M	E	NA	U/C-F
639		AMERICA	TWO MEDICINE	08/06/1964	1913	3M	262M	E	19744000	F
640		AUSTRALIA	UPBURUMBERKA	/ /1955	1915	41M	212M	A	13175000	I
641	1	CANADA	UNION BAY	10/02/1912	1907	6M	172M	E-C	7404000	F
642		GREAT BRITAIN	UPPER BLACK MOSS	08/08/1967	1894	11M		E-C	204570	F
643		AMERICA	UPPER CANADA POND	11/03/1901	1816	6M	183M	E		
644		GREAT BRITAIN	UPPER CREGGAN	04/08/1971	1851	8M		E	32000	F
645		GREAT BRITAIN	UPPER CREGGAN	08/09/1908	1851	10M		E		F
646		AMERICA	UPPER TALLASSEE	29/12/1901		19M	55M	R		F
647		CANADA	VAL MARIE	/ /1952	1939			E		I
648		AMERICA	VALENTINE	/ /1911	1911	9M		E		F
649	3030	ITALY	VAIONT	09/10/1963	1960	266M	191M	A		F
650		AMERICA	VEEDERS POND	20/10/1916		10M	46M	E-C		F
651		AMERICA	VERNON HEIGHTS	20/10/1896						F
652		AMERICA	VICTOR COLORADO.	19/05/1901		8M	131M	E	318220	F
653	96	BULGARIA	VRATSA	01/05/1966						F
654		CZECH.	VYSNE ZEBOJNA	/ /1960	1945	9M		E		F?
655		GREAT BRITAIN	WADHURST PARK LAKE	/ /1970			200M	E		
656	1	AMERICA	WAGNER CREEK	19/04/1938	1918	15M		E	740400	F
657		AMERICA	WAIIAKA	/01/1921	1906	42M	139M	R	10000000	I
658	150	AMERICA	WALNUT GROVE	22/02/1870	1888	34M	122M	R	25146000	F
659	2	AMERICA	WARD	09/07/1897				E		F
660		GREAT BRITAIN	WARMWITHENS	24/11/1970	1850?	10M		E	109104	F

661		GREAT BRITAIN	MASSOLL MILL	/ /1963		5M		E	>22700		I
662		AUSTRALIA	WAYATINAH ,S.			12M		R	NA		U/C-L
663		AMERICA	WEAR	/ /1938	1912	13M		E			F
664		GHANA	WEIJA	/ /	1959	9M		E			F
665		AMERICA	WEST RIVER 1	11/03/1901	1816	5M	183M	E			F
666		AMERICA	WEST RIVER 2	11/03/1901	1816	6M	183M	E			F
667		GREAT BRITAIN	WHARRY BURN	/ /1943	C1932	12M	M	E	113700		I
668		AMERICA	WHEATLAND,WYO	08/07/1969	1893	14M	2012M	E	11599600		F
669		GREAT BRITAIN	WHINHILL	15/03/1815	1796	13M	243M	E	262000		F
670	40	GREAT BRITAIN	WHINHILL 2	21/11/1835	1821	13M	243M	E	262000		F
671		GREAT BRITAIN	WHITE COPPICE RES	14/08/1871				E			I
672		GREAT BRITAIN	WHITEHEADS MIDDLE1	05/12/1852			27M	E-C			F
673		GREAT BRITAIN	WHITEHEADS MIDDLE2	05/12/1852			27M	E-C			F
674		AMERICA	WHITEWATER BROOK UP	4/05/1972	1943	19M	137M	E			I
675		AMERICA	WILBUR	/08/1940	1912	21M	111M	CG	882310		I
676		AMERICA	WINCONSIN DELLS	06/10/1911	1910	18M	87M	E-C	24660000		F
677		AMERICA	WINCONSIN RIVER	29/05/1928	1923	9M	244M	E			F
678		AMERICA	WINSTON NO2	15/03/1912	1902	8M	91M	E-C	795550		F
679		AMERICA	WISE RIVER,MONT.	14/06/1927	1903			E+R			F
680		GREAT BRITAIN	WOODHEAD	07/10/1849	1848	31M	174M	E	392000		I
681		AUSTRALIA	YALLOURN WEIR	/12/1934	1924	12M	59M	CG	490000		F
682		CHINA	YELLOW RIVER	29/09/1887		5M	1615M	E			F
683		AMERICA	YUBA RIVER	17/03/1907	1906	4M	373M	E			F
684		YUGOSLAVIA	ZAVOZ	/02/1963		NATURAL LANDSLIDE					F
685		ITALY	ZERBINO	13/08/1935	1925	15M	146M	CG	9850000		F
686		AMERICA	CENTRAL FLORIDA	/ /1960							F
687		AMERICA	UNKNOWN	03/04/1926				E			F
688		AMERICA	UNKNOWN	06/04/1912				E			F
689		GREAT BRITAIN	CONFIDENTIAL 1	17/07/1955				E	45000		F
690		GREAT BRITAIN	CONFIDENTIAL 2	/ /1960		5M	45M	E	100000		I
691		GREAT BRITAIN	CONFIDENTIAL 3	/ /1968		5M	45M	E	100000		I
692		GREAT BRITAIN	CONFIDENTIAL 5	/ /1968	1840	5M	358M	E	99000		F
693		GREAT BRITAIN	CONFIDENTIAL 6	24/02/1977	1807	5M	120M	E	25000		I
694		GREAT BRITAIN	CONFIDENTIAL 7	/ /1968	1850+	5M	76M	E	77250		I
695		GREAT BRITAIN	CONFIDENTIAL 8	/ /1968	1880?	3M	M	E			I
696		GREAT BRITAIN	CONFIDENTIAL 9	14/09/1968		M	M	E	27730		F
697		GREAT BRITAIN	CONFIDENTIAL10	14/09/1968		4M	8M	E	31339		F
698		GREAT BRITAIN	CONFIDENTIAL11	/ /1974		M	60M	E	77282		F
699											
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* A=ARCH
* C=CORED
* C+=CONCRETE AND
* CG=CONCRETE GRAVITY
* D=DISMANTLED;DESTROYED
* E=EARTHFILL
* F=FAILED
* G=GRAVITY
* H=HYDRAULIC
* I=INCIDENT - REPAIRED
* L=NATURAL LAKE
* M=MASONRY
* N=NATURAL DAM
* NA=NOT APPLICABLE
* R=ROCKFILL
* T=TIMBER
* U/C=UNDER CONSTRUCTION

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* /=+=-=AND                                                                    *
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*****************************************************************************
* COMMENT: THE INFORMATION REPORTED IN THIS LIST IS BELIEVED TO BE CORRECT,   *
* BUT AS IT HAS BEEN OBTAINED FROM A VARIETY OF SOURCES SOME SLIGHT           *
* INACCURACIES MAY BE INCLUDED.                                               *
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* ALL DIMENSIONS REPORTED TO THE NEAREST WHOLE METRE                          *
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*                                                                           *
*  COMMENT:  INCIDENTS LISTED HERE ARE NOT NECESSARILY 'PEDIGREE' CASES OF    *
*  OVERTOPPING. I.E. OVERTOPPING MAY HAVE RESULTED FROM MECHANICAL FAILURE OF *
*  CONTROL EQUIPMENT,SETTLEMENT,ETC. ;AND NOT ALL CASES LISTED ARE SUBJECT    *
*  TO ANY SAFETY LEGISLATION.                                                *
*                                                                           *
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* COMMENT: ALL HEIGHTS ARE THE MAXIMUM HEIGHT WHERE KNOWN.                    *
*                                                                           *
*****************************************************************************

CASES WHERE DAMS HAVE ALTERNATIVE NAMES.
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AGUACALIENTE     =    AGUACALIENTE
ALICANTE         =    TIBI
ALLESSANDRIA     =    ALLA SELLA ZERBINO
ALYCON           =    ALCYON
ANDERSON         =    PORTMAN SHOALS
ARMANDO          =    LIMOEIRO
ARI              =    CALLANDER
ASHLAND          =    READERS GULCH
AUSTIN,TEXAS     =    LAKE MCDONALD
BALLOCHDOWAN     =    KILANTRINGAN LOCH
BAKER CITY       =    GOODRICH CREEK
BARRENJACK       =    BURRENJUCK
BAYLISS          =    BAYLESS.
BEATH            =    BEITH
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781	BLAEN-Y-CYM	=	EBBW VALE IRONWORK	=	LLANHILLETH
782	BLUE WATER	=	BISHOP CREEK		
783	BRADFORD 2	=	GILBERT RUN 2		
784	BRAEBURN	=	DAM NO 4 PENN		
785	BREAKNECK	=	CONNELLSVILLE		
786	BUFFALO CREEK	=	MINE TAILINGS.		
787	CARTSBURN	=	WHINHILL	=	BEATH = BEITH
788	CHEUREAS	=	GRAN CHEUREAS		
789	CLINTON 1	=	KILLINGWORTH 1		
790	CYMCARN	=	ROGERS POND		
791	DALE DYKE	=	BRADFIELD		
792	DARWEN	=	OVER DARWEN	=	RADFIELD FOLD
793	DELLS	=	WINCONSIN DELLS		
794	DIXIE LAKE	=	MARTIN DAVEY		
795	EL MALI	=	ESMALI.		
796	ERINDALE	=	CREDIT RIVER		
797	FALMOUTH	=	COLLEGE 1		
798	FALMOUTH	=	COLLEGE 1		
799	FERGOUG	=	EL HABRA	=	PERRAUGAUX
800	FORTE MEADE	=	DEADMANS GULCH		
801	FISHKILL	=	MELZINGAH		
802	GLENOGIL	=	GLENOGLE		
803	HARLEM	=	SPUTYEN DUYVIL		
804	HAUPT ESTATES	=	FRACKVILLE		
805	HOLMFIRTH	=	BILBERRY		
806	HORTON	=	MISSION CREEK		
807	HYDRAULIC CO.	=	BRIDGEPORT		
808	IABIA	=	TABIA		
809	IRON RIVER	=	MUD LAKE		
810	IRUKA DAM	=	IRUKAIKA		
811	ITHACA	=	SIX MILE CREEK		
812	JEANETTE	=	OAKFORD PARK		
813	KABDAM	=	KADDAM.		
814	LAKE AVALON	=	AVALON		
815	LAKE GEORGE	=	LIDDERDALE		
816	LAKE HEMET	=	LAKE HELMET		
817	LOWERCROFT	=	WHITEHEADS MIDDLE LODGE		
818	MOLARE	=	ZERBINO		
819	NR.LOUGHBOROUGH	=	NANPANTAN		
820	OAKLAND	=	VERNON HEIGHTS.		
821	OCHLOCKONEE RV.	=	TALLASSEE LOWER.		
822	OIL CITY	=	SPARTANSBURG.		
823	OLDHAM	=	GREENFIELD		
824	PANGUITCH	=	CIRCLEVILLE		
825	PARDO	=	FRIAS DIQUE PARDO.		
826	PARIS	=	SHORT CREEK		
827	POINTA	=	POINT A		
828	POTOSI	=	SAN IDLEFONSO	=	KARIKARI DEL REY
829	POWER CANAL	=	SALT RIVER		
830	RIO NEGRO	=	RINCON DE BONETE		
831	SANTELLE	=	SEPULVIDA	=	SAWTELLE = SEPULVEDA = .
832	SCHNECTADY	=	VELDERS POND		
833	SHERBURNE	=	NORWICH,N.Y.		
834	SIG	=	SAINT DENIS		
835	SOUTH FORK	=	JOHNSTOWN		
836	SPAULDING POND	=	ZOHEGAN PARK	=	NORWICH
837	STAUNTON	=	LEWIS CREEK		
838	TIFFIN	=	MOHAWK		
839	TRENT LAKE	=	TELLURIDE		
840	WARD	=	JEFFERSON COUNTY		

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841     YALLOUP          =    YALLOURN
842     WEST RIVER       =    RANDALLS POND          =     UPPER & LOWER CANADA POND
843     YUBA             =    DEBRIS BARRIER NO. 1
844     ***************************************************************************
END OF FILE
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APPENDIX B

KNOWN INSTANCES OF DAMS WHICH HAVE BEEN OVERTOPPED

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BRITISH SUB-SAMPLE

KNOWN INSTANCES OF DAMS WHICH HAVE BEEN OVERTOPPED.

DEAD	COUNTRY	NAME	DATE	BUILT	HT	LT	TYPE	CAPACITY M3	INCIDENT TYPE	MR		
	GREAT BRITAIN	ALDENHAM	/ /1933	1795	8M	400M	E	78000	I	TQ	169	958
	GREAT BRITAIN	ARI	02/01/1933	1933	20M	189M	C	22730	U/C-I	NN	649	138
	GREAT BRITAIN	ASHBURNHAM 1	/ /	1762	5M		E		I	TQ	692	145
	GREAT BRITAIN	ASHBURNHAM 2	/ /	1762					I	TQ	692	145
	GREAT BRITAIN	ASHBURNHAM 3	/ /	1762	9M	260M	E		I	TQ	692	145
	GREAT BRITAIN	ASHFORD RES.	10/07/1968	1934	6M	220M	E	50006	I	ST	236	344
	GREAT BRITAIN	AUCHENDORES RES.	14/01/1969	1880	10M	348M	E	857000	I	NS	367	723
	GREAT BRITAIN	BALLOCHDOWAN	04/11/1843	1843		137M	E		F	NK	039	738
	GREAT BRITAIN	BALTHAYOCK FISHING	08/07/1916		8M		E	102285	F	NO	145	234
	GREAT BRITAIN	BARROWFORD	/ /1976		9M	1000M	E	459146	I	SD	870	400
	GREAT BRITAIN	BEATH	15/03/1815	1796	13M	243M	E	262000	F	NS	232	746
40	GREAT BRITAIN	BEATH	21/11/1835	1821	13M	243M	E	262000	F	NS	232	746
	GREAT BRITAIN	BEITH	15/03/1815	1796	13M	243M	E	262000	F	NS	232	746
40	GREAT BRITAIN	BEITH	21/11/1835	1821	13M	243M	E	262000	F	NS	232	746
	GREAT BRITAIN	BESOM HILL	/10/1856	1856		73M	E	45460	F	SD	933	065
81	GREAT BRITAIN	BILBERRY	05/02/1852	1838	20M	90M	E-C	390956	F	SE	103	071
	GREAT BRITAIN	BILBERRY	24/05/1944	1840	32M		E-C	390956	I	SE	103	071
	GREAT BRITAIN	BIRCHEN BRIDGE	14/09/1968		8M	60M	E	>22700	F	TQ	194	392
	GREAT BRITAIN	BIRCHEN BRIDGE	/ /1974		8M	60M	E	>22700	F	TQ	194	392
	GREAT BRITAIN	BIRSEMORE LOCH	23/04/1968	PRE1865	3M	425M	E	78000	F	NO	526	973
	GREAT BRITAIN	BLACK MOSS	/ /1810	1800	6M	320M	E	85000	I	SE	032	328
	GREAT BRITAIN	BLACKBROOK 1	20/02/1799	1796	23M	160M	E	2300276	F	SK	458	175
	GREAT BRITAIN	BLACKBROOK 2	/ /1804	1801	25M	160M	E	2300276	D	SK	458	178
	GREAT BRITAIN	BLACKSYKE	21/09/1921						UNKNOWN			
	GREAT BRITAIN	BLAEN-Y-CYM	16/01/1912						F	SO	175	130
	GREAT BRITAIN	BLACKMELINDWR				73M	E		I	SN	715	934
1	GREAT BRITAIN	BLAKENEY BROOK	15/07/1875		5M		M	296000	F	SD	644	143
	GREAT BRITAIN	BLEA BECK			7M	137M	E		F	SE	042	565
	GREAT BRITAIN	BLITHFIELD	/02/1962	1953	16M	856M	E	18123000	I	SK	072	228
	GREAT BRITAIN	BOTTOMS	/07/1872	1877	20M	250M	E	155000	U/C-I	SK	023	970
245	GREAT BRITAIN	BRADFIELD	11/03/1864	1859	29M	382M	E	3227660	F	SK	243	916
	GREAT BRITAIN	BRAITHWAITE	02/11/1898							NY	229	235
	GREAT BRITAIN	BROTON	/07/1917	1857				142312	F	ST		
	GREAT BRITAIN	BUCKIEBURN RES.	01/11/1970	1900	23M	259M	E	909000	I	NS	740	859
	GREAT BRITAIN	CALLANDER	02/01/1933	1923	20M	189M	C	22730	U/C-I	NN	649	139
	GREAT BRITAIN	CAMLOCK	/02/1859	1801	8M				F	NR	818	877
	GREAT BRITAIN	CANTSBURN 1	/ /1815	1796	13M	243M	E	262000	F	NS	232	746
	GREAT BRITAIN	CANTSBURN 2	/ /1835	1821	13M	243M	E	262000	F	NS	232	746
	GREAT BRITAIN	CASTLE HOWARD	/ /1967	1798	6M	520M	E	45000	I	SE	717	706
2	GREAT BRITAIN	CASTLE MALGWYN 1	10/08/1875							SN		
2	GREAT BRITAIN	CASTLE MALGWYN 2	10/08/1875							SN		
2	GREAT BRITAIN	CASTLE MALGWYN 3	10/08/1875							SN		
	GREAT BRITAIN	CHEW MAGNA	/03/1848	1849	9M	96M	E-C	113650	I	ST	5[illegible]	533
	GREAT BRITAIN	CHEW MAGNA	10/07/1968	1849	10M	108M	E-C	113650	F	ST	561	633
	GREAT BRITAIN	COCKERMOUTH	22/11/1899	TEMPORARY DAM			T			NY	115	327
	GREAT BRITAIN	COCKER CROSS RES.	24/11/1970					522800	I	SD	741	265
16	GREAT BRITAIN	COEDTY	02/11/1925	1924	11M	244M	E-C	318220	F	SH	755	657
	GREAT BRITAIN	COLLEGE 1	/ /	1850	6M	116M	E		I	SH	754	328
	GREAT BRITAIN	COMBS	/ /1878	1797	16M	304M	E	15450000	I	SK	035	795
	GREAT BRITAIN	COPE LODGE	20/09/1944	1860	12M	73M	E-C		I			

61		GREAT BRITAIN	CONLYD	21/12/1924	1921	14M	310M	E-C	10760000		I	SH	737	674
62	12	GREAT BRITAIN	CYNCARN	14/07/1875	1792	12M		E	80210		F	ST	222	935
63	245	GREAT BRITAIN	DALE DYKE	11/03/1864	1859	29M	382M	E	3227660		F	SK	243	916
64		GREAT BRITAIN	DANSON PARK	05/09/1958	1762	4M	162M	E	90000		I	TQ	475	747
65	13	GREAT BRITAIN	DARWEN	23/08/1848				E			F	UNKNOWN		
66		GREAT BRITAIN	DENTON	/ /1947	C1800	7M	1380M	E	278000		I	SK	870	[illegible]
67		GREAT BRITAIN	DEANHEAD	/ /1841	1840	19M	320M	E-C	318220		I	SE	039	154
68	..	GREAT BRITAIN	DEBDON LAKE	13/08/1946		M	M	E	39704		I	SK		
69		GREAT BRITAIN	DIGGLE	/ /1849	1830	12M	133M	E	55460		I	SE	021	081
70		GREAT BRITAIN	DOVESTONE CLOUGH	29/05/1881							I	SK	014	037
71		GREAT BRITAIN	DUFLEIGH	10/07/1968	1938	10M	433M	E	959206		I	ST	271	[illegible]
72		GREAT BRITAIN	EARLSWOOD	/ /1953	C1815	5M	400M	E	772820		I	SP	110	741
73		GREAT BRITAIN	EASTER DALTULLICH	15/08/1893				E			F			
74		GREAT BRITAIN	EBBW VALE IRONWORK	16/01/1912							F	SO		
75		GREAT BRITAIN	EMBSAY MOOR	03/06/1908	1908		430M	E	795550		U/C-F	SD	000	545
76		GREAT BRITAIN	FALMOUTH		1850	6M		E	331858		F	SW	764	328
77		GREAT BRITAIN	FIELDS DAM	/ /1926				M+E			F	SK	410	803
78		GREAT BRITAIN	FURNACE HOUSE POND	/ /1968				E			F	TQ	105	333
79		GREAT BRITAIN	FURNACE MILL COWDEN	05/09/1968		6M	120M	E			F	TQ	455	399
80	8	GREAT BRITAIN	GLANDERSTON	30/02/1942		M	M	E			F	NK		
81		GREAT BRITAIN	GLENHORDIAL	/05/1963	1820	SIDE BANK		E-C	63000		I			
82		GREAT BRITAIN	GLENOGIL	10/03/1975	1907			E+CG	273000		I	NO	449	643
83		GREAT BRITAIN	GLENOGLE	10/03/1975	1907			E+CG	273000		I	NO	449	643
84		GREAT BRITAIN	GLEN CLACHAIG	/01/1811	1801	10M					F	NR	813	873
85		GREAT BRITAIN	GLEN CLACHAIG	/02/1859	1801						F	NR	813	873
86		GREAT BRITAIN	GREENFIELD	29/05/1881				E			F	SD	955	094
87		GREAT BRITAIN	HANCH	/ /1960	1850	4M	259M	E	145472		I	SK	103	857
88		GREAT BRITAIN	HARRINGTON	30/11/1864	1864	6M	60M	E-C			U/C-F	SK	9	2
89		GREAT BRITAIN	HENBURY HALL 1	19/06/1972			72M	E			F	SJ	871	720
90		GREAT BRITAIN	HENBURY HALL 2	19/06/1972			48M	E			F	SJ	870	751
91		GREAT BRITAIN	HENBURY HALL 3	19/06/1972							F	SJ	865	736
92	81	GREAT BRITAIN	HOLMFIRTH	05/02/1852	1838	20M	90M	E-C	390956		F	SE	103	071
93		GREAT BRITAIN	HOLMFIRTH	/ /1944	1840	32M		E	390956		I	SE	103	071
94		GREAT BRITAIN	KENNEDY'S MILL DAM	21/11/1835				E			F	NS	235	730
95		GREAT BRITAIN	KERTYLE	/ /1944		BYWASH	WASHED OUT				F	NY	445	070
96		GREAT BRITAIN	KENWOOD HOUSE 1	/08/1970	PRE1777	6M		E			I	TQ	230	854
97		GREAT BRITAIN	KENWOOD HOUSE 2	/03/1973	PRE1777	5M		E			I	TQ	279	865
98		GREAT BRITAIN	KENWOOD HOUSE 3	/08/1975	PRE1777	1M		E			I	TQ	278	867
99		GREAT BRITAIN	KENWOOD HOUSE 4	/08/1975	PRE1777	6M		E			I	TQ	277	868
100		GREAT BRITAIN	KENWOOD HOUSE 5	/08/1975	PRE1777	4M	M	E			F	TQ	275	870
101		GREAT BRITAIN	APPLE COVE	29/10/1927		16M	M	E			F	NY	344	167
102		GREAT BRITAIN	KILANTRINGAN LOCH	04/11/1843	1843			E	102285		F	NX	039	781
103		GREAT BRITAIN	KILLAMARSH	/ /1968	C.1774	7M	240M	E	105000		I	SK	473	805
104		GREAT BRITAIN	KILLINGTON	30/07/1836	1820	16M	220M	E	3482000		F	SD	590	094
105		GREAT BRITAIN	L. NAN UREAC RUIDHE	/ /1965	1801			P			F	NH	813	84[illegible]
106		GREAT BRITAIN	LIGHTWOOD BYWASH	31/07/1881		BYWASH		E			F	SK	054	751
107		GREAT BRITAIN	LLANHILLETH	16/01/1912							F	SO	175	130
108		GREAT BRITAIN	LLYN BODGYNYDD BACH	/ /	1880	5M	M	E			F			
109		GREAT BRITAIN	LOCH AIGHIDH NA LIC	27/08/1933		4M		E			F	NG	405	39
110		GREAT BRITAIN	LOCH CLACHAIG	/02/1859							F	NR	813	87[illegible]
111		GREAT BRITAIN	LOCH DUNTELCHAIG	/ /1970	1930	1M	17M	C	6270000		I	NH	646	32[illegible]
112		GREAT BRITAIN	LONG DAM	03/06/1908				C			F	SD	991	5[illegible]
113		GREAT BRITAIN	LOWERCROFT 1	05/12/1852			27M	E-C	5228		F	SD	735	11[illegible]
114		GREAT BRITAIN	LOWERCROFT 2	05/12/1852			27M	E-C			F	SD	742	11[illegible]
115		GREAT BRITAIN	LOWER BLACK MOSS						295490		F	SD	027	01[illegible]
116		GREAT BRITAIN	LOWER NEUADD	/ /18	1884		200M	E	195500		I	SO	026	1[illegible]
117		GREAT BRITAIN	LUEGASHALL MILL	16/09/1968	PRE1650	5M	106M	E	92738		F	SU	940	239
118		GREAT BRITAIN	MCHOPE	/06/1936				E	59000		F	SO	060	972
119		GREAT BRITAIN	MARPANTAN	20/07/1875	1670	7M	4195M	E	131334		F	SK	309	17[illegible]
120		GREAT BRITAIN	NEVER FEAR DAM	/ /1926							F	SK	405	8[illegible]

121		GREAT BRITAIN	NORTHAMPTON CANAL	23/08/1865							F	NOT APPLIC	
122		GREAT BRITAIN	NR. BRISTOL	C.1940	1700	10M		M			F	ST	
123		GREAT BRITAIN	NR. BRISTOL.	C.1960	1700	10M		M			F	ST	
124		GREAT BRITAIN	NR. BRISTOL.	C.1968	1700	10M					I+D	ST	
125		GREAT BRITAIN	NR. PUMPSAINT	- LEAD MINING DAM - BREACHED - NOTICED ON T.V. DOCUMENTARY									
126		GREAT BRITAIN	NR.LOUGHBOROUGH	20/07/1875	1870			E	131834		I	SK 508 172	
127		GREAT BRITAIN	OLDHAM	29/05/1881				E			F?	SD 956 09?	
128		GREAT BRITAIN	OTTERHEAD LAKE	10/07/1968	1900	2M	76M	E-C	9092		F	ST 22 13	
129	13	GREAT BRITAIN	OVER DARWEN	23/08/1848				E	3000000		F	UNKNOWN	
130		GREAT BRITAIN	PEAK FOREST CANAL	/07/1973		60M OF CANAL		E			F	NOT APPLIC.	
131		GREAT BRITAIN	PICKUP BANK	/07/1974	1849			E	141000		I	SD 726 241	
132		GREAT BRITAIN	PIPINGFORD PARK	/ /1972	1696	6M	118M	E			F	TQ 450 31?	
133	13	GREAT BRITAIN	RADFIELD FOLD	23/08/1841				E			F	UNKNOWN	
134		GREAT BRITAIN	RAINBOW FORGE DAM	09/07/1926			70M	E			F	SK 419 940	
135	12	GREAT BRITAIN	ROGERS POND	14/07/1875	1792	12M		E	80210		F	ST 222 93?	
136		GREAT BRITAIN	ROUND DAM	03/06/1908				E			F	SD 991 526	
137		GREAT BRITAIN	RUNNEYMEDE	/ /1961	1887	12M	M	E			F	NX 372 53?	
138		GREAT BRITAIN	SCOTT FOUNDRY DAM	21/11/1935			83M	E			F	NS 285 75?	
139		GREAT BRITAIN	SHAAS WATER CO.	21/11/1835	1831			E	545000		F	NS 285 75?	
140		GREAT BRITAIN	SHERBOURNE	/ /	1736	7M	150M	E	55000		I	ST 647 16?	
141	16	GREAT BRITAIN	SKELMORLIE	/04/1825	1878	5M		E	15911		F	NS 202 975	
142		GREAT BRITAIN	SKELPER UPPER DAM	28/01/1909							F	SK 397 79?	
143		GREAT BRITAIN	SKELPER LOWER DAM	28/01/1909							F	SK 398 8??	
144		GREAT BRITAIN	SLAUGHAM MILL POND			4M		E	81828		D	TQ 253 277	
145		GREAT BRITAIN	SLIGNEAGH DAM	/ /1961	C1925	4M	39M	E	220000		F	NM 639 66?	
146		GREAT BRITAIN	SLIGNLAGH DAM	/ /1957	C1925	4M	39M	E	220000		F	NM 639 66?	
147		GREAT BRITAIN	SPINDLE DAM	03/06/1908				E			F	SE 006 535	
148		GREAT BRITAIN	SPOTT LAKE	12/08/1948	1900	9M	18M	E			F	NT 662 749	
149		GREAT BRITAIN	STANLEY	/ /1927	1840	14M	185M	E	612000		I	SJ 930 51?	
150		GREAT BRITAIN	STAUNTON HAROLD	/ /	1781	7M	100M	E	6000		I	SK 380 21?	
151		GREAT BRITAIN	STUBSHIEL	12/08/1948	1911	10M	70M	CG	245000		I	NT 501 62?	
152		GREAT BRITAIN	STOUR	/ /1917				E	141600		F?	ST 7 3.	
153		GREAT BRITAIN	SWINESHAW	/02/1962	1865	11M		E-C	254000		I	SK 043 957	
154		GREAT BRITAIN	TODBROOK	/ /1878	1838	21M	210M	E	1345616		I	SK 005 809	
155		GREAT BRITAIN	TUMPLETON LAKE	22/07/1927	1879	10M	110M	E	50000		I		
156		GREAT BRITAIN	TUMPLETON LAKE	13/08/1946	1879	10M	110M	E	50000		I		
157		GREAT BRITAIN	TUNNEL END	/ /1799	1798	9M	140M	E	13638		I	SE 040 120	
158		GREAT BRITAIN	THORTERS	12/08/1948	1914	15M	20M	E	236000		I	NT 605 696	
159		GREAT BRITAIN	UPPER CLACK MOSS	08/08/1967	1894	11M		E-C	204570		I	SD 824 411	
160		GREAT BRITAIN	UPPER CREGGAN	04/08/1971	1851	8M		E	32000		F		
161		GREAT BRITAIN	UPPER CREGGAN	08/09/1908	1851	10M		E			F		
162		GREAT BRITAIN	WARHURST PARK LAKE	/ /1970			200M	E				TQ 538 280	
163		GREAT BRITAIN	WALMSITHENS	24/11/1970	1850?	10M		E	109104		F	SD 739 247	
164		GREAT BRITAIN	WASSELL MILL	/ /1968		5M		E	>22700		I	SU 931 ?41	
165		GREAT BRITAIN	WHARNY BURN	/ /1943	C1932	12M	M	E	113700		I		
166		GREAT BRITAIN	WHINHILL	15/07/1815	1796	13M	243M	E	262000		F	NS 292 74?	
167	40	GREAT BRITAIN	WHINHILL 2	21/11/1935	1821	13M	243M	E	262000		F	NS 292 74?	
168		GREAT BRITAIN	WHITE COPPICE RES	14/06/1891				E			I	SD 617 192	
169		GREAT BRITAIN	WHITEHEADS MIDDLE1	05/12/1852			27M	E-C			F	SD 786 11?	
170		GREAT BRITAIN	WHITEHEADS MIDDLE2	05/12/1852			27M	E-C			F	SD 782 11?	
171		GREAT BRITAIN	WOODHEAD	07/10/1849	1848	31M	174M	E	392000		I	SK 082 994	
172		GREAT BRITAIN	CONFIDENTIAL 1	17/07/1955		M	M	E	45000		F		
173		GREAT BRITAIN	CONFIDENTIAL 2	/ /1960		5M	45M	E	100000		I		
174		GREAT BRITAIN	CONFIDENTIAL 3	/ /1968		5M	45M	E	100000		I		
175		GREAT BRITAIN	CONFIDENTIAL 5	/ /1968	1840	5M	358M	E	99000		F		
176		GREAT BRITAIN	CONFIDENTIAL 6	24/02/1977	1807	5M	120M	E	25000		I		
177		GREAT BRITAIN	CONFIDENTIAL 7	/ /1968	1850+	5M	76M	E	77250		I		
178		GREAT BRITAIN	CONFIDENTIAL 8	/ /1968	1880?	3M	M	E			I		
179		GREAT BRITAIN	CONFIDENTIAL 9	14/09/1968				E	27730		F		
180		GREAT BRITAIN	CONFIDENTIAL 10	14/09/1968		4M	M	E	31339		F		

```
GREAT BRITAIN   CONFIDENTIAL  11     /  /1974          8M    60M    E     77282     F

*****************************************************************************
END OF FILE
```

APPENDIX C

CALIBRATION CURVES

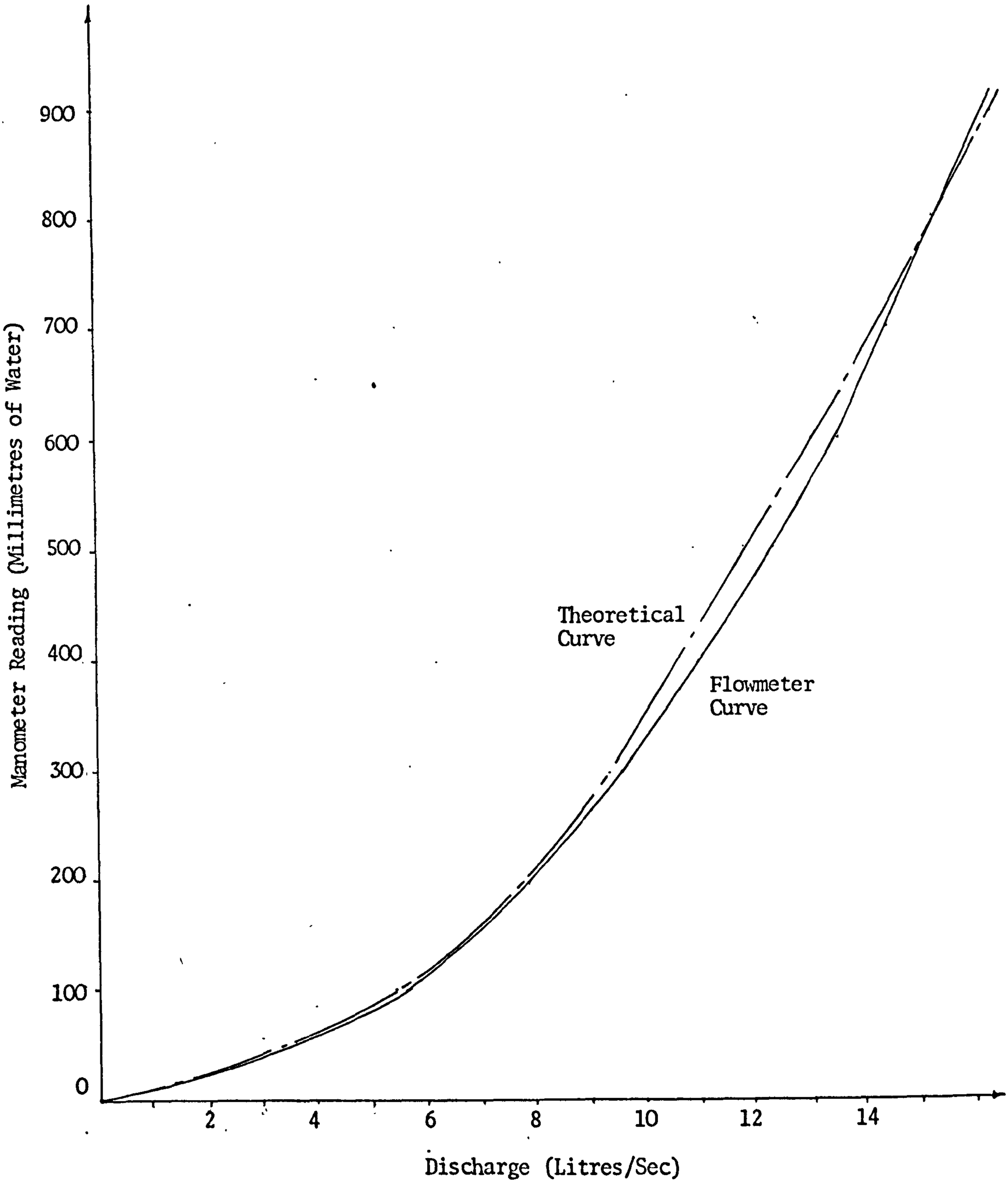


FIGURE C1: Orifice Plate/Turbine Flowmeter Calibration

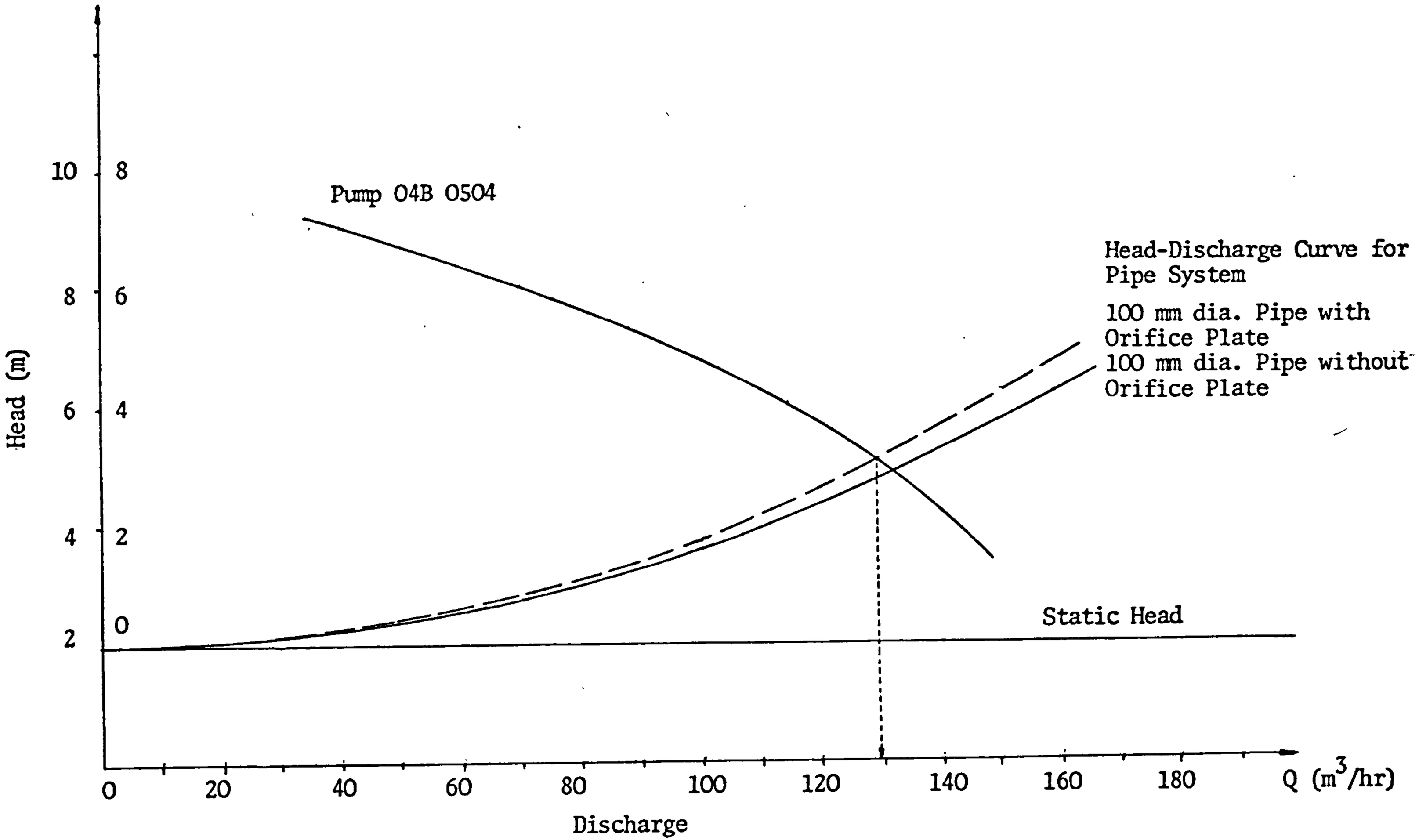
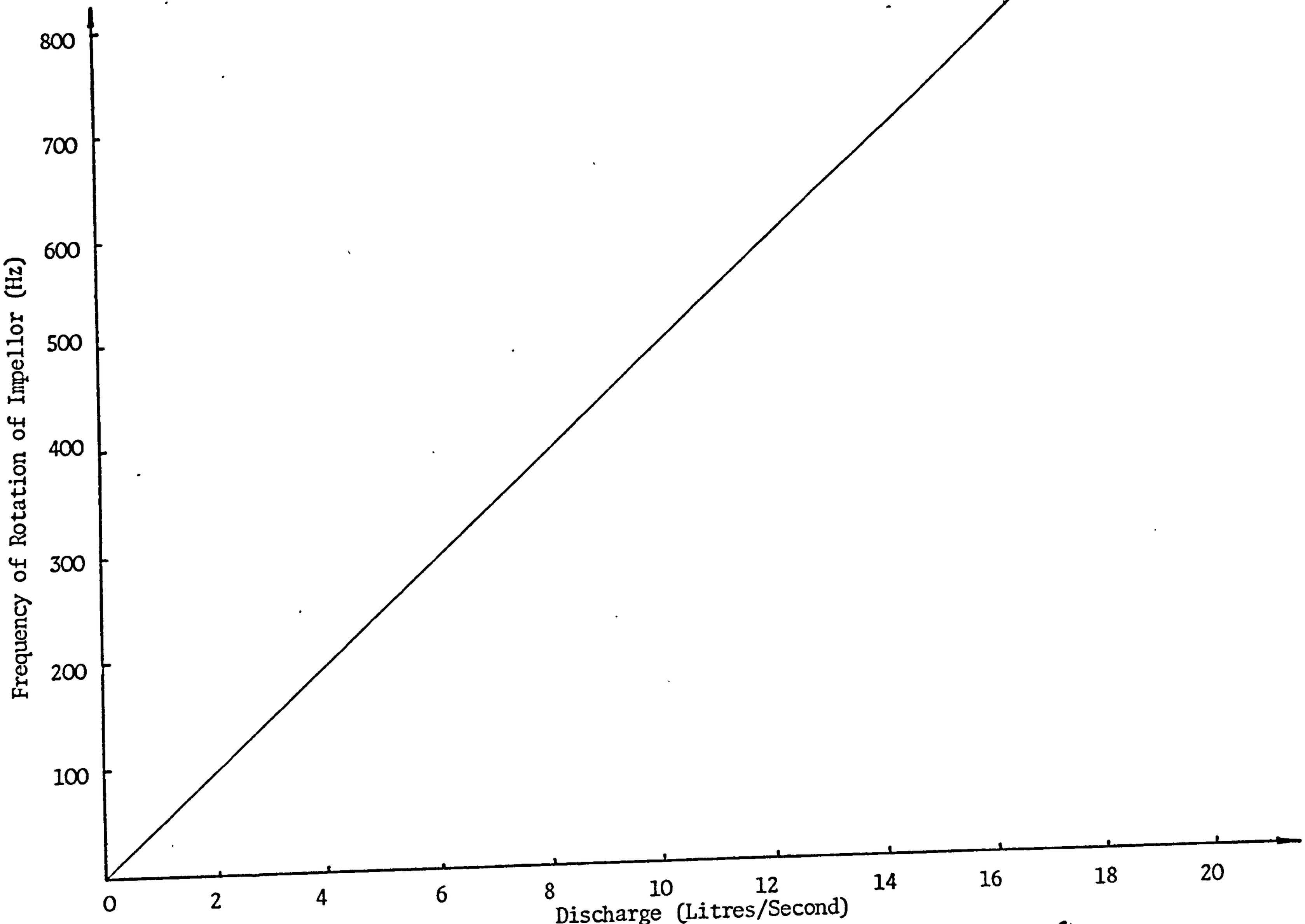
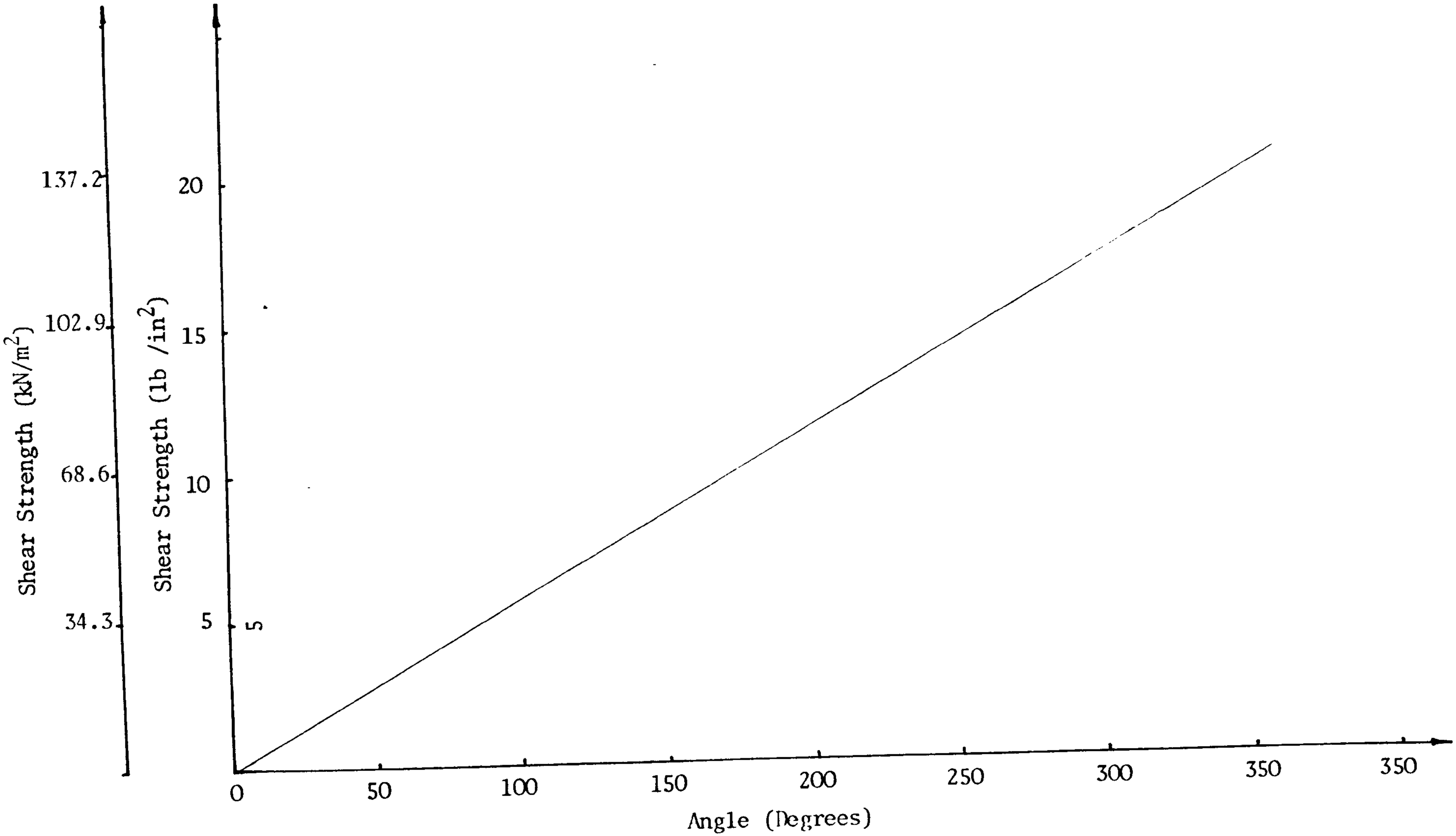


FIGURE C2: Pump Characteristics


Frequency of Rotation of Impellor (Hz)
800
700
600
500
400
300
200
100
0
2
4
6
8
10
12
14
16
18
20
Discharge (Litres/Second)


137.2
102.9
68.6
34.3
Shear Strength (kN/m²)
20
15
10
5
Shear Strength (lb /in²)
5
0
50
100
150
200
250
300
350
350
Angle (Degrees)

APPENDIX D

NOTATION AND ABBREVIATIONS

Appendix D

Notations/Abbreviations

The following symbols are used in this thesis that may not be defined elsewhere:-

b	width
c	cohesion
cc	cubic centimetres
^{o}C	degrees Celcius
d	depth
D	diameter
f	function
ft	feet
ft^2	square feet
ft^3	cubic feet
ft^3/sec	cubic feet per second (cusecs)
gm	grams
g	gravitational acceleration
h	height of vane
ha	hectare
hr	hours
Hz	hertz
kg	kilogramme
km	kilometre
kN	kilonewton
kph	kilometres per hour
L	unit of length
LL	Liquid Limit
lb	pound
m	metre
M	unit of Mass
m^2	square metres
m^3	cubic metres
mm	millimetres

m^3/sec	cubic metres per second (cumecs)
m/sec	metres per second
PL	Plastic Limit
PI	Plasticity Index
p.s.f.	pounds per square foot
s	shear strength
sec	seconds
t	tonne
t.cm	thousands of cubic metres
T	unit of Time
v	velocity
y	depth of flow
Å	Ångstrom
γ_B	bulk density
ρ	density of water
μ	microns

APPENDIX E

COMPUTER FILE DATA

Appendix E

Program to put on file data lines with more than

80 characters - use of *FILEPUNCH

TO PUT ON CARDS

$R *FILEPUNCH SPunch = *PUNCH* PAR = FILEPUNCH = file name.

If numbered lines required :-

PAR = FILE LINEPUNCH = filename

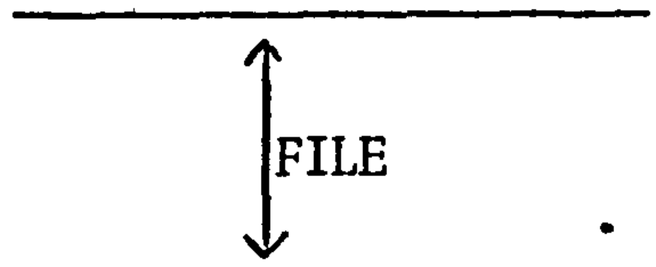
TO PUT BACK ON FILE

$ CREATE filename

$ GET filename

$R *FILEPUNCH

FILE = filename RESTORE



$ ENDFILE

```
$COMPILE
      REAL OAC,OADN,CP,CENS,CPM,TCP,P,C
      INTEGER C,M
      DIMENSION ISTRES(36,1)
      DIMENSION AREA(36,1)
      DIMENSION AREAM(36,1)
      READ(5,010)C
  010 FORMAT(I2)
      WRITE(6,011)C
  011 FORMAT( ' CELL PRESSURE                    ',I2, 'LBS:IN2')
      READ(5,012)M
  012 FORMAT(I2)
      WRITE(6,013)M
  013 FORMAT( ' APPROX MOISTURE CONTENT   ',I2, '%')
      READ(5,014)CENS
  014 FORMAT(F7.3)
      WRITE(6,015)CENS
  015 FORMAT( ' APPROX DENSITY              ',F10.3, 'KG:M3')
      WRITE(6,022)
  022 FORMAT(//)
      TCP=0.
      WRITE(6,009)
  009 FORMAT('    LOAD          LOAD        COMP STRESS     COMP STRESS'
     1/,'     LBS             N          LBS:IN2         N:MM2')
      DO 100 N=1,36
      READ(5,001)ISTRES(N,1)
  001 FORMAT(I3)
  100 CONTINUE
      DO 200 N=1,36
      READ(5,002)AREA(N,1)
  002 FORMAT(F5.3)
  200 CONTINUE
      DO 300 N=1,36
      READ(5,003)AREAM(N,1)
  003 FORMAT(F7.3)
  300 CONTINUE
      DO 005 I=1,36
      CAO=0.3334*ISTRES(I,1)
      CAON=1.482*ISTRES(I,1)
      CP=CAO/AREA(I,1)
      CPM=CAON/AREAM(I,1)
      WRITE(6,004) CAO,CAON,CP,CPM
  004 FORMAT(F8.3,4X,F8.3,5X,F7.3,5X,F10.6)
      IF (CPM.GT.TCP)TCP=CPM
  005 CONTINUE
      WRITE(6,023)
  023 FORMAT(/)
      WRITE(6,040) TCP
  040 FORMAT(' MAX VALUE OF COMPRESSIVE STRESS',F12.5,'N/MM2')
      WRITE(6,024)
  024 FORMAT(//)
      P=(TCP+(2*C*0.006895))/2
      C=TCP/2
      WRITE(6,016)  P
  016 FORMAT(' P VALUE IS  ',F10.6)
      WRITE(6,017) C
  017 FORMAT(' C VALUE IS  ',F10.6)
      STOP 100
      END
```

APPENDIX F

TABULATED RESULTS

Initiation of Erosion Test

Mean vel. m/sec	Depth of flow mm	Suspended Solid Conc. gm/litre x 10^{-3}
1.42	6.2	15
1.75	6.4	16
1.85	7.8	24
1.95	8.7	34
2.07	10.8	40
2.21	12.5	56
2.27	14.0	94
2.30	15.3	116
2.51	15.4	154

Density of sample : 1942 kg/m^3

m/c of sample : 19.7%

Clay only.

TRIAXIAL TEST

$DATA

CELL PRESSURE	20LBS:IN2	20
APPROX MOISTURE CONTENT	19%	19.35
APPROX DENSITY	1850.000KG:M3	1878.44

LOAD LBS	LOAD N	COMP STRESS LBS:IN2	COMP STRESS N:MM2
0.000	0.000	0.000	0.00000000
4.668	20.748	2.637	0.01816924
6.001	26.676	3.385	0.02332082
7.001	31.122	3.942	0.02716156
8.002	35.568	4.498	0.03098960
9.002	40.014	5.052	0.03480447
9.669	42.978	5.417	0.03732608
12.002	53.352	6.690	0.04609563
13.669	60.762	7.577	0.05220686
15.003	66.690	8.275	0.05701560
16.337	72.618	8.961	0.06174319
17.337	77.064	9.463	0.06520182
18.670	82.992	10.136	0.06983626
19.337	85.956	10.441	0.07193929
20.337	90.402	10.922	0.07525408
21.004	93.366	11.220	0.07730633
21.671	96.330	11.515	0.07933682
23.005	102.258	12.095	0.08333373
24.338	108.186	12.663	0.08724749
25.338	112.632	13.048	0.08989704
26.339	117.078	13.424	0.09249330
27.339	121.524	13.794	0.09503639
28.339	125.970	14.106	0.09718931
29.339	130.416	14.439	0.09948051
30.006	133.380	14.580	0.10045640
31.006	137.826	14.907	0.10270720
31.673	140.790	15.047	0.10366990
32.006	142.272	15.026	0.10353150
33.007	146.718	15.295	0.10538110
33.673	149.682	15.411	0.10618220
34.340	152.646	15.539	0.10705980
34.674	154.128	15.486	0.10669910
0.000	0.000	0.000	0.00000000
0.000	0.000	0.000	0.00000000
0.000	0.000	0.000	0.00000000

MAX VALUE OF COMPRESSIVE STRESS 0.10705980N/MM2

P VALUE IS 0.191430
Q VALUE IS 0.053530
STD 100

Erosion Rates

γ_B = 1840- 1850 kg/m^3	m/c (%)	Erosion rate gm/m^2/min	
		full sample	sub sample
v = 2.38 - 2.42 m/s	19.27	497	500
	19.71	471	478
	20.24	438	442
	20.74	418	414
	21.35	389	386
	21.80	384	371
	22.02	360	349
	22.69	382	372
	22.94	400	397
v = 2.51 - 2.74 m/s	18.74	503	515
	19.21	490	494
	19.54	464	469
	20.09	449	452
	20.79	424	406
	21.72	387	383
	22.40	393	380
	23.21	442	451
v = 3.0 - 3.15 m/s	19.07	630	616
	19.62	607	599
	20.26	577	557
	20.84	549	541
	21.18	520	515
	22.00	511	494
	22.60	498	488
	22.97	522	517

Erosion Rates (cont)

γ_B = 1840 - 1850 kg/m^3	m/c (%)	Erosion rate gm/m^2/min	
		full sample	sub sample
v = 2.4 m/sec	18.91	395	400
	19.37	366	370
	19.65	352	341
	20.39	288	283
	20.92	275	265
	21.44	254	226
	22.03	267	248
	22.76	312	308
v = 2.63 m/s	18.52	425	400
	19.03	402	376
	19.74	367	360
	20.52	325	316
	21.39	293	270
	22.00	302	297
	22.17	329	321
v = 3.04 m/s	19.01	498	490
	19.46	464	454
	20.18	440	426
	20.49	380	376
	21.00	367	364
	21.81	353	340
	22.64	423	396

γ (kg/m^3)	C(kN/m^3)	Ø (°)	m/c (%)
1850	3.4	0	21.50
	4.0	0	21.15
	5.2	0	20.40
	6.0	1	19.90
	7.4	0.5	19.30
	10.25	1	18.35
	13.20	0	17.75
	15.10	1	17.35
	16.75	0	17.00
1900	4.0	0	21.82
	5.3	0.5	21.10
	6.0	1.5	20.72
	7.7	1.0	19.69
	10.0	0	18.87
	10.4	0	18.72
	12.1	0	18.02
	14.0	1	17.80
	17.5	0	17.20
	18.7	1.5	16.95
1950	4.7	0	22.01
	6.35	0.5	21.11
	7.70	1.0	20.28
	11.92	1.0	18.82
	15.75	0.5	17.80
	19.50	0	17.10

TABLE F2 : Results of Triaxial Tests on 'Standard' Clay

m/c (%)	γ_D (kg/m^3)
10.99	1672.16
13.11	1748.52
14.07	1760.00
16.95	1778.97
17.01	1780.00
19.42	1704.04
21.05	1639.96

TABLE F3 : Compaction Test Results - Standard Clay.

(2.5 kg rammer)

	m/c (%)	γ_D (kg/m^3)
5% SAND	9.77	1699.43
	10.77	1725.21
	14.40	1780.66
	17.49	1763.58
	17.42	1749.29
	20.02	1684.00
10% SAND	8.74	1717.48
	10.38	1740.24
	14.04	1779.42
	16.99	1744.34
	19.50	1677.83
	21.46	1619.32
15% SAND	8.89	1735.17
	10.37	1751.42
	13.66	1781.22
	15.84	1761.48
	17.23	1739.17
	20.73	1620.00
20% SAND	16.56	1692.17
	9.06	1739.02
	11.49	1775.33
	16.87	1746.74
	19.05	1668.24
	20.35	1624.92

TABLE F4 : Compaction Test Results - 'Modified'Clay

(2.5 kg rammer)

	C(kN/m^3)	Ø (°)	m/c (%)
5% SAND	3.4	0	21.50
	5.4	1	19.82
	8.2	1.5	18.12
	9.9	1	17.17
	12.7	0	16.66
	14.95	1	16.45
10% SAND	3.5	0	21.68
	4.1	0	20.21
	5.2	0	18.90
	6.2	1.5	18.32
	9.8	1	17.02
	17.0	0	16.20
20% SAND	1.4	0	23.52
	2.2	1	21.15
	3.25	1.5	19.72
	3.75	0	19.22
	5.00	2	18.02
	5.50	1	17.55

TABLE F5 : Triaxial Test Results - 'Modified' Clay

γ_B = 1900 kg/m^3

% SAND	L.L.	P.L.	P.I.
0	39	18	21
5	39	17.80	21.2
10	38	17.50	20.5
15	37	16.82	20.18
20	35	16.20	18.80
25	33	15.56	17.44
30	31	15.00	16.00

TABLE F6 : Tabulated Results of L.L., P.L. and P.I. for 'Modified' Clay

	Area (m^2)	Volume (m^3)
'Large sample	1.285×10^{-1}	5.14×10^{-3}
'Sub-sample'	3.5×10^{-2}	1.4×10^{-3}
Ratio : $\frac{\text{Large sample}}{\text{Sub-sample}}$	3.67	3.67

TABLE F7 : Sample Data

m/c (%)	time (min)	swelling (in x 10^{-4})
17.56	5	25
	17	55
	20	64
	25	78
	45	137
	60	172
	85	238
	130	320
19.19	40	50
	57	100
	65	125
	80	159
	100	206
	120	231
	130	252
	135	260
	840	850
19.37	30	65
	40	105
	60	120
	75	145
	120	215
	180	282
	240	364

TABLE F8 : Swelling Test Results - 'Modified Clay'
(5% Sand)

m/c (%)	time (min)	swelling (in x 10^{-4})
22.22	7	10
	120	165
	125	170
	140	194
	155	210
	180	238
	210	264
	240	275
	275	320
	300	336
	390	375
26.02	35	14
	50	23
	70	32
	120	52
	280	96
	305	101
	325	123

TABLE F8 (cont'd) : Swelling Test Results - 'Modified Clay'
(5% Sand)

m/c (%)	Time (min)	Swelliing (in x 10^{-4})
18.45	5	100
	30	260
	60	414
	90	527
	120	592
	150	649
	165	663
	270	772
20.38	10	41
	20	70
	30	98
	70	208
	105	275
	155	342
	180	366
	330	466
	390	488
21.19	8	35
	25	72
	75	173
	100	226
	210	365
	300	437
	340	458
	360	462

TABLE F9 : Swelling Test Results - 'Modified Clay'
(15% Sand)

m/c (%)	time (min)	swelling (in x 10^{-4})
22.14	5	22
	50	112
	60	135
	80	166
	110	220
	150	262
	180	300
	285	371
	330	400
22.20	60	71
	90	100
	165	170
	210	300
	240	312
	300	345
	330	358
	360	370
	390	376

TABLE F9 (cont'd) : Swelling Test Results - 'Modified Clay'
(15 % Sand)

m/c (%)	time (min)	swelling (in x 10^{-4})
18.48	12	72
	20	110
	30	160
	55	260
	85	368
19.50	50	25
	90	130
	135	220
20.35	5	10
	10	16
	20	30
	70	110
	80	123
	90	138
	130	182
	155	215
	180	240
22.65	20	25
	55	50
	90	69
	150	107
	180	125
	240	150

TABLE F10 : Swelling Test Results - 'Modified Clay'
(20% Sand)

m/c (%)	time (min)	swelling (in x 10^{-4})
21.07	40	10
	80	32
	90	40
	100	51
	150	72
22.80	45	13
	90	26
	105	30
	120	36
	205	55
	320	80
25.00	60	8
	100	11
	148	11

TABLE F10 (cont'd) : Swelling Test Results - 'Modified Clay' (20% Sand)

APPENDIX G

DUNFORD BRIDGE

FULL SCALE TEST.

UNIVERSITY OF NEWCASTLE UPON TYNE
DEPARTMENT OF CIVIL ENGINEERING

NOTES ON POSSIBLE FIELD TRIALS, DUNFORD BRIDGE DAM

1. Introduction

The completion of Winscar Dam will lead to the abandonment and subsequent flooding of the old Dunford Bridge Dam. As a safeguard against possible instability of Dunford Bridge embankment in the event of a future draw-down of Winscar reservoir, the old embankment will be permanently breached. In the normal course of events, this operation would be carried out at a suitable stage of construction, having first decommissioned Dunford Bridge reservoir. It is suggested that the combination of circumstances is such as to provide a unique opportunity to conduct invaluable field trials.

2. Motivation

Dunford Bridge is typical of at least several hundred old embankment dams, mainly 19th century, which are still in service in the UK. In a large number of cases little or nothing is known of the details of design and construction, and the structural competence of many dams of this type and vintage is a matter of concern to the Panel Engineer charged with statutory inspection under the Reservoirs (Safety Provisions) Act.

Two questions of particular interest in the case of old clay embankments are relevant in the context of the Dunford Bridge/Winscar situation:

2.1 upstream slope stability in the event of rapid draw-down

2.2 resistance to overtopping consequent upon inadequate spillway capacity and/or local settlement

It is considered that the planned abandonment of Dunford Bridge affords an opportunity to carry out appropriate field trials under controlled conditions.

3. Possible Trials

Problem 2.1, stability

Useful data could be retrieved from a suitable piezometer array installed in Dunford Bridge Dam prior to draw-down for breaching.

Field data on the effects of rapid draw-down on pore pressures within clay embankments would usefully supplement existing knowledge. Considered in more detail, however, it is felt that a project to this end would be expensive to set up in relation to the value of the information gained. The circumstances surrounding Dunford Bridge are also such that experimental work restricted solely to measurement of pore pressures would, in our opinion, neglect a unique opportunity to conduct other trials.

Problem 2.2: overtopping

Some dispute exists as to the risk which overtopping represents to a clay embankment, and actual data is extremely limited in extent. The fact that Dunford Bridge will require to be breached in any event, combined with its location behind a much higher new dam, suggest the possibility of

conducting a controlled 'overtopping' experiment. It is felt that the return on an experiment of this nature would be such that it merits the more detailed consideration given in 4.

4. Overtopping Trial

4.1 A controlled overtopping trial on a section of the Dunford Bridge embankment would provide invaluable data on the following:

a) susceptibility of clay fill to erosion

b) influence of height and duration of overtopping on behaviour of clay slopes

c) the mechanics of vertical and lateral washout of clay fill

d) influence of a central puddle core on washout

A study of the influence of the very rapid draw-down associated with breaching on the stability of the upstream slope of the remainder of the dam could also be carried out in association with the breaching trial.

4.2 Considered in the particular context of Dunford Bridge/Winscar, the general procedure for conducting an 'overtopping' trial might be as set out in 4.3. Assumptions made at this stage are that the Dunford Bridge Dam is of homogeneous clay construction or contains a puddle-clay core, and that side slopes are 1 in 2. With regard to Winscar, it is appreciated that no risk of damage to the upstream membrane or to the draw-off arrangements is acceptable. To these ends the sudden increase of load against Winscar following breaching of the embankment must be minimised, and the washout flow directed away from the draw-off gallery.

4.3 The essential procedure for an overtopping trial would be:

4.3.1 Prepare a section of the crest of Dunford Bridge Dam for overtopping.

4.3.2 Having selected a suitable level for Winscar reservoir to act as a 'cushion' (see 5.), allow filling to that level while Dunford Bridge fills to spill level.

4.3.3 Initiate overtopping on Dunford Bridge by removal of sandbag barrier by mobile crane.

4.3.4 Failure pattern of Dunford Bridge to be followed by time-lapse and/or intermittent cine photography.

4.3.5 Examination of failure zone and of upstream slope of that portion of Dunford Bridge left standing.

5. Location of Overtopping and Breach

Two possible locations for the initiation of overtopping are indicated on the attached 1/2500 plan. Examination of provisional capacity curves indicates that appropriate starting and final water levels could be selected from the following alternatives:

Water levels at start

	Dunford Bridge	Winscar	Approx. final water level*	Outflow (m^3)	Rise on Winscar (m)
a)	327	315	322	325000	7
b)	327	318	323	265000	5
c)	327	320	324	230000	4
d)	327	322	325	190000	3

*assumes complete breaching to this level or below

Alternative d) provides, in our opinion, the most suitable combination of levels for the experiment.

With regard to duration of test, no meaningful estimate is really possible, but approximate initial discharge characteristics for overtopping through a 3 m wide breach are as follows:

Reservoir level	Breach level	H(m)	Q(m^3/sec)
327	326.8	0.2	0.7
327	326.6	0.4*	2.1
327	326.5	0.5	3.0
327	326.4	0.6	3.8

*min. figure likely to erode intact clay

6. Technical Problems

6.1 Haul Road

The presence of the haulage road running along the berm below the old dam causes us some concern in the event of its still being present upon breaching. We are of the opinion that the haulage road material will very readily wash down into the sump behind Winscar and, as the distance is only some 60 m, and would inevitably wash into the Winscar scour gallery entrance. A further possible hazard is the possibility of general instability of the haulage road, particularly at its N. end above the Dunford Bridge scour tunnel, when the water level behind Winscar rises.

6.2 Breach Material

It is a reasonable assumption that the clay fill washed out from a breach forming in Dunford Bridge Dam will, to a large extent, break down into fines and be left in suspension behind Winscar. Coarser material from this source, and from the haul road, will carry to the scour gallery as referred to in 1. Fine material, from both sources, presents another problem in that it is likely to remain in suspension for a considerable period and may thus prove an embarrassment as regards putting Winscar on public supply. An accurate estimate of settling time, or of concentration, is not possible at this stage, but the time required to completely settle out may possibly be of the order of 6 to 12 weeks.

6.3 Relative Water Levels

The question of optimum timing with regard to the relative water levels behind Dunford Bridge and Winscar dams at the commencement of breaching

is a complex one. Various factors are involved, but purely from the standpoint of a breaching experiment we consider that an initial differential head across the Dunford Bridge embankment should be not less than 5 m, and preferably of the order of 7 m or more. This is particularly relevant to the initial level behind Dunford Bridge, placing a lower limit of about 326 on the latter reservoir if complete breaching to, say, 322 is likely to be assured.

6.4 Timing

The timing of any work on Dunford Bridge is dependent upon the Water Authority's requirements as to maintaining continuity of supply from Dunford Bridge and, latterly, Winscar. Provisional 'target' date for a breaching operation has been determined as 29 January 1975. If the project is to go ahead, an early indication of the availability of support is essential in order to allow for adequate planning.

7. Responsibility

The Consultant and the Water Authority will, if approval is given, be prepared to accept all legal responsibility arising from the project provided that the work is carried out strictly in accordance with an agreed programme.

8. Estimated Cost

Support required divides into a) assistance on site and b) financial backing for the University contribution.

As regards a), it is appreciated that large-scale technical support will not be available, but a certain amount of assistance by Contractors staff on the following aspects of the projects would be necessary. The major item here would be excavation and preparation of the 'notch' used to initiate breaching, involving removal of up to 150 m^3 of fill if the 'notch' is carried down to level 325.

With reference to b), the University would require to make available all necessary assistance for setting out target markers, still and time-lapse photography, cine photography and control of operations. The logistic effort required would be considerable. A detailed estimate is not possible at this stage, but a provisional figure of some £1200-£1500 appears realistic. An absolute ceiling of, say, £2000 would provide a sufficient margin for any untoward circumstances arising.

A I B Moffat

November 1974

NEW ACCESS ROAD

PUMPING STATION

GALLERY HOUSE

GALLERY

DAM CREST

APPROXIM
OF MAIN QU

CLIFFORD BRIDGE DAM

HAUL ROAD USED
DURING CONSTRUCTION
OF WINSCAR DAM

POSSIBLE LOCATIONS
FOR INITIATION OF
BREACHING ACTION

CREST @ 329 mAod

AREA FOR STUDY OF
POSSIBLE INSTABILITY
FOLLOWING BREACHING
AND DRAWDOWN

N

1/2500

SPILLWAY

APPROXIM
FOR STOR

APPROXIMATE LI
SITE OFFICES SHA

APPENDIX H

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ADDENDUM

Additions to Appendix A

(1) GREAT BRITAIN HIGHER SWINESHAW / /1844 1800
E F

(2) 10000 INDIA MACHHU 1 10/08/1979
M M I

(3) 10000 INDIA MACHHU 11 11/08/1979
51M 3949 M E + C F

APPENDIX J

EXPERIMENTAL PROGRAMME -(Tabulated Results.)

SUMMARY OF EXPERIMENTS - SERIES I. - CLAY SAMPLES.

Test No.	Clay Type.	Bulk Density. (kg/m^3)	m/c. (%)	Velocity m/sec.	Duration of Test. (min)	Temp. (°C)	Material Eroded. (kg)	Observations.
1.	Standard.	1890	20.05	2.86	60	21.	Observation only.	
2.	Standard.	not recorded.		2.86	60.	21.	1.954	'clean water'
3.	Standard.	not recorded.		2.86	60.	21.	1.928	'dirty water'
4.	Standard.	1840-1850	19.27	2.38-2.42	60.	20.	3.831	pock marks, stones causing deeper hole.
5.	Standard.	"	19.71	"	"	21.	3.631.	ripples on surface some mass erosion.
6.	Standard.	"	20.24	"	"	21.	3.377	as above, material rolling, scour hole
7.	Standard.	"	20.74	"	"	21.	3.222	pebbles whirled round in holes.
8.	Standard.	"	21.35	"	"	20.	2.999	as above.
9.	Standard.	"	21.80	"	"	21.	2.960	same general erosion pattern.
10.	Standard.	"	22.02	"	"	21.	2.775	as above.
11.	Standard.	"	22.69	"	"	21.	2.945	as above, some deep holes eroded.
12.	Standard.	"	22.94	"	"	20.	3.084	as abpve.
13.	Standard.	1840-1850	18.74	2.51-2.74	"	21.	3.878	general pattern.
14.	Standard.	"	19.21.	"	"	21.	3.777	as above.

SUMMARY OF EXPERIMENTS - SERIES I. - CLAY SAMPLES.

Test No.	Clay Type.	Bulk Density. (kg/m3)	m/c. (%)	Velocity m/sec.	Duration of Test. (min)	Temp. ('C)	Material Eroded. (kg)	Observations.
15.	Standard.	1840-1850	19.54	2.51-2.74	60	21.	3.577	general erosion pattern.
16.	Standard.	"	20.09	"	"	21.	3.461	as above.
17.	Standard.	"	20.79	"	"	21.	3.269	as above.
18.	Standard.	"	21.72	"	"	21.	2.983	as above.
19.	Standard.	"	22.40	"	"	20.	3.030	rippled surface approx 2mm high
20.	Standard.	"	23.21	"	"	21.	3.407	general pattern of erosion.
21.	Standard.	1840-1850	19.07	3.00-3.15	"	22.	4.857	deep holes some mass erosion
22.	Standard.	"	19.62	"	"	21.	4.680	deep holes due to whirling stones.
23.	Standard.	"	20.26	"	"	21.	4.448	as above - high -turbulence.
24.	Standard.	"	20.84	"	"	21.	4.233	general pattern.
25.	Standard.	"	21.18	"	"	21.	4.009	as above
26.	Standard.	"	22.00	"	"	21.	3.939	as above.
27.	Standard.	"	22.60	"	"	21.	3.839	as above.
28.	Standard.	"	22.97	"	"	21.	4.024	as above.

SUMMARY OF EXPERIMENTS - SERIES I. - CLAY SAMPLES.

Test No.	Clay Type.	Bulk Density. (kg/m3)	m/c. (%)	Velocity m/sec.	Duration of Test. (min)	Temp. ('C)	Material Eroded. (kg)	Observations.
29.	Standard.	1920-1940	18.91	2.4	60	21.	3.045	general pattern.
30.	Standard.	"	19.37	"	"	21.	2.821.	as above.
31.	Standard.	"	19.65	"	"	21.	2.714.	as above.
32.	Standard.	"	20.39	"	"	22.	2.220.	as above.
33.	Standard.	"	20.92	"	"	21.	2.120	general pattern
34.	Standard.	"	21.44	"	"	21.	1.958	as above.
35	Standard.	"	22.03	"	"	21.	2.057	as above.
36	Standard.	"	22.76	"	"	21.	2.406	as above.
37	Standard.	1920-1950	18.52	2.63	"	22.	3.276	as above.
38	Standard.	"	19.03	"	"	21.	3.099	as above.
39	Standard.	"	19.74	"	"	20.	2.829	as above.
40	Standard.	"	20.52	"	"	21.	2.506	start of mass erosion at end
41.	Standard.	"	21.39	"	"	21.	2.259	general pattern
42.	Standard.	"	22.00	"	"	21.	2.328	as above.

SUMMARY OF EXPERIMENTS - SERIES I. - CLAY SAMPLES.

Test No.	Clay Type.	Bulk Density. (kg/m3)	m/c. (%)	Velocity m/sec	Duration of Test. (min)	Temp. ('C)	Material Eroded. (kg)	Observations.
43.	Standard.	1920-1950	22.17	2.63	60	21.	2.536	general pattern of erosion
44.	Standard.	1920-1940	19.01	3.04	"	21.	3.840	deep holes - mass erosion.
45.	Standard.	"	19.46	"	"	21.	3.577	as above to a lesser extent
46.	Standard.	"	20.18	"	"	21.	3.392	general erosion
47.	Standard.	"	20.49	"	"	20.	2.930	as above.
48.	Standard.	"	21.00	"	"	21.	2.829	as above.
49.	Standard.	"	21.81	"	"	21.	2.722	as above.
49a	Standard.	"	22.64	"	"	21.	3.261.	as above.

A
B
C
D
E
F
G
H

All tests in this group were carried out at the start of the experimental investigation and, due to a sample compaction technique which produced samples whose density was very variable, produced results which were not considered to be representative of the erosion process and which could not be reproduced.

SUMMARY OF EXPERIMENTS - SERIES II. - MODELS.

Test No.	Clay Type.	Bulk Density. (kg/m^3)	m/c. (%)	Downstream slope.	Material Eroded. (kg)	Duration of Test. (min)	Observations.
50	Standard.	1900	19.40	1:1	Observations only - rounding of crest, formation of step at regular intervals, erosion of toe.		
51	Standard.	1890	20.47	1:2	As above - eroded material seen to roll, bounce, etc..		
52	Standard.	1940	19.74	1:2.5	As above - rounding of crest, erosion at toe , step formation on regular spacing.		
53	Standard.	1985	19.75	1:2.5	Model formed in sloping channel, head over crest 65mm, discharge 11 l/sec., duration of test 9 hours, comparitively little erosion apart from rounding of crest,steps at regular intervals down the face, and erosion at the toe.		
54	Standard.	1935	19.00	1:1.5	1.804	50.	As above, vel (average) 2.56m/sec.
55	Standard.	1935	19.07	1:2	1.784	50.	As above, density in range 1920-1950.
56	Standard.	1935	19.04	1:2.5	1.720	50.	As above - in all tests layers horz.
57	Standard.	1935	19.14	1:3	1.719	50.	As above.
58	Standard.	1915	20.84	1:2.5	Model formed by inclined layers of clay - mass erosion took place where the 'feather - edges' of the layers occurred on the face - inadequate bonding of layers.		
59	Standard.	1890	18.94	1:1	Again mass erosion at 'feather - edge' and at toe- test stopped early.		